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Comparison between Eurocodes and UK standards (BDs) for structural assessment

The Case Study of Ashworth Viaduct

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Abstract

Within the scope of safety and preservation of historical memory of existing bridges, maintenance of bridges has become one of the most important issues to the economy, society and public interest. To achieve this objective, an efficient management of bridges is needed, being implied in the process, periodic inspections and structural assessments, followed by repair, rehabilitation or strengthening, if necessary. However, drastic actions, such as demolition and/or substitution have to be considered when the structure does not accomplish the minimum requirements of the new standards or it is not economically viable for any other intervention.

This work concentrates on an essential aspect within the process of maintenance, which is the structural assessment of existing bridges. Therefore, an approach regarding this subject is carried out using two different methods of assessment, one based in the Eurocodes and the other one guided by the UK standards for structural assessments, having as the main aim the comparison of these two standards.

To better understand the evolution of bridges in the United Kingdom, a historical approach is developed as first contact, highlighting the beginning of great constructions in history, using as material construction, steel, concrete and steel-concrete composite.

It is known that bridges have been through great changes in traffic load over time affecting thus, their maintenance and the need for structural assessment. On one side' this came to encourage the UK in creating unique standards made specifically for structural assessment (BDs) of bridges. On the other side, the Eurocodes, despite having been created specifically for design of new bridges, are adapted to the assessment of existing bridges, being used in many other countries around Europe.

Throughout this work, the main differences between these two standards, mentioned above, are described taking into account the actions applied on bridges, the used partial safety factors, the properties and capacity of construction materials and the effects of the actions observed in the structural members. To support this comparison, it is also introduced a composite viaduct existing in the UK, where it is possible to take the distinct results from the different models of forces acting on the structure.

In conclusion, an analysis of the two results is carried out in order to register the difference between both codes and discuss main features for future structural assessment works. BDs has been registered to be more conservative than Eurocodes although the results between them do not show a high discrepancy.

Keywords: structural assessment, composite bridges, structural strengthening, Eurocodes, BDs, Comparison between Eurocodes and BDs

Resumo

No âmbito da segurança e preservação da memória histórica de pontes existentes, a manutenção de pontes tem-se tornado numa das questões mais importantes para a economia, sociedade e interesse público. De forma a cumprir esse objetivo, é então necessário um sistema eficaz de gestão de pontes, estando implícito nesse processo uma série de inspeções periódicas e avaliações estruturais seguidas de reparação, reabilitação ou reforço estrutural, se a ponte assim o exigir. No entanto, será necessário tomar medidas drásticas, como por exemplo a demolição e/ou substituição de pontes, sempre que a estrutura não cumpra os requisitos mínimos estabelecidos pelas novas normas ou não seja economicamente viável para qualquer outro tipo de intervenção.

Este trabalho concentra-se num aspeto indispensável no processo de manutenção, sendo ele a avaliação estrutural de pontes existentes. Sendo assim, é feita uma abordagem acerca deste tema, recorrendo a dois métodos diferentes de avaliação, um baseado nas normas estabelecidas pelos Eurocódigos e o outro guiado pelas normas específicas de avaliação estrutural existentes no Reino Unido, tendo como objetivo principal a comparação destas duas normas.

De forma a perceber melhor a evolução das pontes no Reino Unido, uma contextualização histórica é desenvolvida como primeira abordagem, realçando o começo de grandes construções na história, tendo como material de construção, o aço, o betão e o composto dos dois.

Com o passar do tempo, as pontes têm sido sujeitas a grandes mudanças na sobrecarga de tráfego, afetando dessa forma, a respetiva manutenção da ponte e constante necessidade de avaliações estruturais. Isso veio encorajar o Reino Unido na criação de normas próprias para a avaliação estrutural de pontes (BDs). Relativamente aos Eurocódigos, apesar destes serem feitos especificamente para o dimensionamento de pontes novas, são utilizados por muitos países na Europa adaptando-se a avaliação estrutural de pontes.

Ao longo deste trabalho, as principais diferenças entre as duas normas mencionadas são descritas, tendo em consideração as ações aplicadas, os fatores parciais de segurança, as propriedades e capacidade estrutural dos materiais e os esforços provocados pelas ações, observados nos elementos estruturais. Para uma melhor comparação, recorreu-se à análise estrutural de dois modelos distintos de um viaduto misto existente no Reino Unido.

Em conclusão, é feito um estudo para registar a diferença entre ambas as normas e para discutir aspetos relevantes para futuros trabalhos de avaliação estrutural.

Palavras-Chave: avaliação estrutural, pontes mistas, reforço estrutural, Eurocódigos, BDs, Comparação entre Eurocódigos e BDs

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Abbreviations and Acronyms

AF	Adjustment Factors
AIP	Approval In Principle
AW	Authorized Weight
BAs	Advice Notes
BDs	Bridge Departmental
BS	British Standards
BSI	British Standard Institution
CEN	European Committee for Standardisation
DMRB	Design Manual for Roads and Bridges
DOT	Department of Transport
DSIR	Department of Scientific and Industrial Research
EC	Eurocodes
EN	European Standards
ENVs	Provisional European Standards
KEL	Knife Edge Load
LCC	London Country Council
MOT	Ministry of Transport
NA	National Annex
NAD	National Application Document
SLS	Serviceability Limit State
SO	Special Order vehicles
STGO	Special Types General Order vehicles
UDL	Uniformly Distributed Load
ULS	Ultimate Limit State

Symbols

Eurocodes

A_d	Design value of an Accidental Action
A_s	Area of tension reinforcement
A_v	shear area
B, b	Width of the section
D	Depth of cross-section
d	Effective depth to tension reinforcement
E_d	Design value of effects of Actions
E_c	Modulus of Elasticity (concrete)
E_s	Modulus of Elasticity (steel)
F_u	Tensile strength of the stud
f_{ck}	Characteristic cylinder strength
f_{cd}	Design value of concrete compressive strength
f_{cu}	Characteristic cube strength
f_y	Yield Strength
f_u	Ultimate Strength
$G_{k,j}$	Characteristic value of of permanent action j
I_{xx}	Second moment of area x-x
I_{yy}	Second moment of area y-y
J, I_t	Torsional constant
M_{cr}	Elastic cristical moment for lateral-torsional buckling
M_{Ed}	Design Bending Moment
$M_{c,Rd}$	Design Resistance for Bending
$M_{b,Rd}$	Design buckling Resistance Moment
N_{Ed}	Design normal force
$N_{c,Rd}$	Design resistance to normal forces
$N_{b,Rd}$	Design buckling resistance of a compression member
n_l	Number of notional lanes

P Representative value of prestressing action

P_f Probability of failure

Q_{fwk} Characteristic value of Concentrated Load

Q_{ik} Magnitude of characteristic axle loads on notional lane number i

$Q_{k,i}$ Characteristic value of the accompanying action on notional lane number i

$Q_{k,l}$ Characteristic value of the leading variable l

Q_{lk} Magnitude of the characteristic longitudinal forces

q_{ik} Magnitude of the characteristic vertical distributed load on notional lane number i

L Loaded length

R_d Design value of the Resistance

T_{Ed} Design value of total torsional moments

T_{Rd} Design resistance to torsional moments

t_w Thickness of the web

t_{weld} Thickness of the weld

t_f Thickness of the flange

V_{Ed} Design vertical load

$V_{pl,TRd}$ Reduced design plastic shear esistance

V_{Rd} Design shear resistance

$V_{L,Ed}$ Longitudinal Force

W_y Section modulus

W_{el} Elastic section modulus

W_{pl} Plastic section modulus

Y_p Plastic neutral axis

Y_e Distance to combine centroide

w_l Width of lane

z lever arm

α_{Qi}, α_{qi} Adjustment factors of some load models on lanes i

γ_{M0} Partial factor Resistance of cross-section whatever the class is

γ_{M1} Partial factor for Resistance of memebers to instability assessed by member checks

γ_m Partial safety factor for a material property

γ_G Partial safety factor for Permanent Actions

γ_{res} Partial safety factor for Resistance

η, ξ Factors defined in text

$\overline{\lambda_{LT}}$ Non-Dimensional slenderness for lateral-torsional buckling

ρ Reduction factor

σ Standard Deviation

τ_{Ed} Design value of the load shear stress

$\tau_{t,Ed}$ Design shear stresses due to St. Venant torsion

χ_{LT} reduction factor for lateral-torsional buckling

β Reliability Index

ψ_0 Factor for combination value of a variable action

ψ_1 Factor for a frequent value of a variable action

ψ_2 Factor for a quasi-permanent value of a variable action

UK standards for structural Assessment (BDs)

A_e Effective Area

A_{fe} Area of the effective flange section

d_f Distance between the centroids of the two flanges

d_w Overall depth of a rolled section

F_f Limiting force in the flange

h_h Height of the largest hole or cut-out

Q_A^* Assessment Loads

Q_k^* Nominal Loads

r_y Radius of gyration around y axis

r_z Radius of gyration around z axis

L Loaded length

l_e Effective length

M_D Bending Resistance

M_r Limiting moment of resistance

m	Ratio
P_{max}	Maximum axial load
r_{xx}	Radius of gyration x-x
r_{yy}	Radius of gyration y-y
S_A^*	Load effects
z_{el}	Elastic section modulus
Z_{xc}	Elastic section modulus x-x compression
Z_{xt}	Elastic section modulus x-x tension
z_{pe}	Plastic section modulus
γ_{fl}	Partial factor for actions
γ_{f3}	Partial factor for the effects of actions
σ_f	Nominal yield stress of the falnge
σ_{yw}	Nominal yield stress of the web

Chapter 1

1. Introduction

1.1 Preliminary Remarks

Since ancient times, there has always been a need of communication between nations, whether it was for commercial, military and spiritual reasons or for simply spreading the evolution of technology and knowledge, between cities, ports or any kind of separate land.

By levelling up uneven terrain and span mountains, rivers, lakes or even oceans, Roman civilization began to revolutionize the meaning of bridges with the introduction of arch bridges in the world, being United Kingdom, no exception. The concept of bridge was quite unknown until the first types of bridges started to show up in an arch form either from wood, stone or brickwork [1].

Despite of their medieval existence, the majority of arch bridges in UK were constructed from the 18th and 19th century, as first the canal, then the railway and suddenly the road networks were subject to rapid expansion. The Roman Empire was the undoubtedly pioneer in this type of bridges by showing the great impact that this period had on history [2].

Nowadays, stone bridges are no longer being built and they are only a national heritage in various countries of the world noticing, though, that many of them are still standing and most are in service. In the early medieval England, known by Anglo-Saxon times, there was an increase of concern on these bridges preservation and maintenance, not only because of the noticeable increasing of traffic load overtime, as also the fact that they are vulnerable to natural hazards, human actions and aggressive environmental conditions. Therefore, methods of analysis have been made just to repair, restore or rebuild these types of bridges in order to avoid disturbance of the network [3].

Maintenance, repair and rehabilitation of bridges are a very important aspect to be taken into consideration as the design of new ones. The old bridges have therefore a more important role regarding to this aspect since they did not use to consider fundamental actions or even appropriate surcharge in their first analysis as also they withstood the passage of time and their climate changes. Knowing that it can affect even the structural part of the bridge there are some strategies concerning this issue to guarantee the safety of their use.

Therefore, maintenance, repair and rehabilitation have been looking for an equilibrium between the first conception of the bridges and their new requirements and trying to adapt them to the new regulations in force, always respecting the preservation and integrity of the structure. However, as it is expected, ancient bridges in general, will never have more durability than the new ones [4].

It is also important to make a clear distinction between the meaning of roadway and railway whereby the first one consists on a pavement with a specified width usually with shoulders on each side whilst regarding the railways, the routes consist in a pair of steel rails which are laid parallel to each other. The traffic destined for roadways are all type of cars, truck, buses, cycles, pedestrians etc. whilst the railways are only made for the movement of trains.

However, a few bridges of this type were constructed after the first world war not only because most part of them had been destroyed and needed repair but also because like any other kind of construction or technology, always go through evolution and development. Thus, construction of bridges, their materials and performance rapidly became better understood as time went by. Therefore, it is possible to divide the history of bridges in many different parts, more or less independent from each other, being the increase in demand for concrete, iron and steel as construction materials the most important part of that.

As many other characteristics, large spans are a fundamental part in the history of bridges because they are the most representative element of progress and development. It was first started with a simple trunk of wood, going through a junction of several trunks laid side by side forming thus a simply supported beam, going bigger as time went by and being replaced by portal frames and arches truss beams [3].

1.2 Scope

It is a reality that road traffic has been increasing over time, not only in volume but also in gross weight, which has brought, as expected, a constant need of concern of assessing the condition and strength capacity of bridges as time goes by. It can be justified by the fact that these new loads may not be compatible with the design of the existing bridges under consideration, which could have been constructed in a totally different period when the requirements for load traffic and materials strength were completely different.

Therefore, the main aim of this work is focused on the main differences between the Eurocodes and UK standards (BDs) in the assessment of existing bridges, having as a case study, the composite bridge of Ashworth Road Viaduct, in West Yorkshire, United Kingdom.

This comparison will be based in the main requirements of both standards when regards to actions applied on general road bridges and also in the resistances of steel, concrete and steel-concrete composite structures. It is intended to analyse the results obtained in the structural members of both models, to understand their differences and similarities and take valid conclusions that can possibly help in a positive upgrade of both standards regarding the assessment of existing bridges.

1.3 Organization of the Dissertation

This dissertation is divided in 7 chapters described below:

Chapter 1 – *Introduction*: This Chapter presents an introduction of the underlying subject, highlighting the importance of the assessment and maintenance of bridges nowadays, which leads to the main scope of this work;

Chapter 2 – *Evolution of steel, concrete and composite bridges in the UK*: it Is made a historical context regarding the evolution of steel, concrete and composite bridges in the UK, connecting the materials used according to the period under consideration. Furthermore, it is made an approach of the composition and evolution of those materials;

Chapter 3 – *Introduction of the Eurocodes and UK standards (BDs)*: In this Chapter, an introduction is made regarding the standards that are going to be compared throughout this work. A brief historical contextualization is carried out both in Eurocodes and BDs standards, followed by its respective organization, aims and contents. When comes to BDs standards, it is made an approach regarding the different kind of inspections considered in the UK for bridges maintenance;

Chapter 4 – *Main Differences between the Eurocodes and UK standards (BDs)*: This chapter is carried out towards a detailed comparison between the two codes enumerating the different requirements regarding materials properties, actions, combinations of actions and materials resistance related to structural forces;

Chapter 5 – *Case Study of Structural Assessment- Ashworth Road Viaduct*: This Chapter is entirely dedicated to the case study of Aswhorth Road Viaduct, where is made a brief introduction of the case such as its location, historical information and description. Afterwards, the model of the case is introduced having as basis the principles mentioned in chapter 4 when regards to the actions applied in both standards as well as the results of the structural members capacity;

Chapter 6 – *Discussion of Results*: This Chapter has been created to allow a better understanding of the differences and similarities between the two codes regarding the main efforts acting on the bridge such as bending moment, shear forces, axial forces and the combined effects. This comparison is made using graphical illustrations to simplify it;

Chapter 7 - *Summary, Conclusions and Future Works*: At last, Chapter 7 is reserved for the conclusion of this work, where is commented the existing differences in both standards, mentioned in Chapter 4, and the respective results obtained in Chapter 6. Giving, in this way, some advices for future works associated with the upgrade of standards in the assessment of existing bridges.

Chapter 2

2. Evolution of Steel, Concrete and Composite Bridges in the UK

2.1 Structural Materials

2.1.1 Concrete

The use of concrete as a construction material, in a gross way of its meaning, has started long time ago in the Middle East when, unintentionally, the builders mixed pounded-clay and thin damp limestone, which, reacting with gases from the air formed a strong protective surface that is nowadays called cement. Overtime, the need of building strong structures and demanding a greater durability, forced the Romans to improve this type of cement, discovering then the pozzuolana[1]. They used this material in marine structures and in those that were exposed to water such as docks, aqueducts and bridges. However, the material that exists today was still unknown until 1824, when Joseph Aspdin, an English entrepreneur and manufacturer in the United Kingdom, created equivalent to Portland cement. The name Portland arose after the high-quality building stones discovered in Portland, England. Although it was not that common for aesthetic reasons, in the early 19th century, concrete started to be used in industrial buildings and private houses, generally as a non-reinforcement mass concrete. [5]

Concrete is a material formed by the admixture of a hydraulic binder (cement), large and fine aggregates and water. It can also contain additions and adjuvants to improve its characteristics. The kind of cement has an important role in the reactivity and the water amount being used in the process, affecting also the concrete strength. The aggregate can be chosen specifically to produce concrete in order to develop certain properties, for instance using materials like clay and expanded shale for light concrete. This constituent is a large portion of the concrete material, varying from 70 to 80% of the total volume of concrete [5].

Regarding the process that concrete goes through, it is possible to distinguish two different phases such as the fresh concrete state and the hardened concrete state. In the first one, the

[1] a mixing of siliceous and aluminous materials in a finely divided form which in the presence of water, reacts in way that produce cementitious properties

material is still malleable, and it can be compacted while it does not obtain its resistance. Then it starts to harden, gaining thus some resistance and stiffness reaching its high potential at more or less 28 days. Therefore, concrete can be classified into three types according to its density. They are known by normal concrete with a density varying between 2000kg/m^3 and 2600kg/m^3, heavy concrete when its density is above 2600kg/m^3 and light weight concrete which has a density not greater than 2000kg/m^3 [6].

After concrete has reached its full capacity, there is a significant subject that must be taken into account, it is the characterization of the fracture surface. It is known that concrete has little resistance under tensile stresses, so although reinforced concrete subjected to flexure has been designed to crack, it is controlled by the distribution of steel reinforcement. Thus, the cracking of concrete, mainly based on concrete composition, plays an essential part in the evaluation of fractures mechanics properties of cementitious materials. This characterization also depends on the load applied during the test, being afterwards an important process for the mechanical behaviour of this material. As far as it is known, water binder ratio, maximum aggregate size and aggregate type can have a great influence on the fractal dimension verified in the concrete when subjected to a certain load. Some tests have been carried out to see how much the fractal dimension varies with the amount of these constituents and the conclusions are clear that the cracks increase almost linearly with the water binder ratio, so as the size of the aggregate.

In the end, the combination of the three constituents of concrete in addition to these characteristics mentioned above have to satisfy the minimum performance specifications such as shrinkage, creep, modulus of elasticity, compressive strength, durability and workability, required for structural concrete [5].

2.1.2 Steel

The origin of the production of steel is still a bit unknown. However, it is thought to have started with some investigations using bloomery hearth furnaces and taking into account some aspects such as the quality and preparation of the ore being processed, the material and the shape of furnace walls. With these experiences, it was defined different designations for iron and steel depending on the way they were produced and processed, being thus steel characterized by having a higher tensile strength and greater hardness due to the higher quantity of carbon. [7]

Cast iron was known by its brittleness and low shock resistance whereby plenty of variations were made to improve the qualities of this material, despite not having succeeded. However, it was noticed that this material showed a compressive strength 100 times greater than the one observed in stone, it was not long before the building industry became interested in the new material and so they started their first tentative in industrial buildings and bridges [7].

Wrought iron can be defined as a resilient malleable iron appropriate for making shapes, rather than smelting, which cannot be hardened.

Finally, steel is the last material showing up, being its properties and performance consequence of the combination of its chemical composition and process of production, including its heat treatment [8].

The use of these materials can be divided into three periods in history, cast iron (1780-1850), wrought iron (1850-1900) that replaced cast iron and at least steel, used since 1880 until the present which enabled the construction of bridges to become larger and lighter. The material is a fundamental part regarding bridge design, not only because resistant properties of the bridge heavily depend on the dimensions of the material, but also because of their technological properties such as manufacturing capacity, joints or shapes of basic elements. Metal has always been ahead of concrete when it comes to dimensions of bridges, which means it has a greater capacity to span big bridges due to its higher specific strength. There are three different types of metal, showing different characteristics among them, that have been through evolution, not only in the composition but also in the way of manufacturing them, which enables to distinguish them as wrought iron, cast iron and steel [7].

2.1.2.1 Cast Iron

Cast iron is an iron-carbon metal, known by containing more than 2% of carbon which is considerated a high level of carbon. Cast iron can be divided in three most common types: white, gray and nodular cast iron. With the gray one being the most used due to its lower costs, although it has a good behaviour when subjected to compression, the opposite occurs when it is subjected to tension. Thus, this material is often used in parts of the bridge that are subjected to compression, such as arches, columns etc. Overall, cast iron combines many features such as vibration damping and long lifecycle as a result of various conditions and graphite formations. However, it has some negative aspects which are: being a brittle material and having a poor resistance to impact and shock. Hence, cast iron is not suitable for welding since it can lead to brittle cracks in and around welded joints [9].

2.1.2.2 Wrought iron

Wrought iron was the first metal classed as structural steel. However, its characteristics are noun-homogeneous and especially bad in the thickness direction due to the manufacturing process. Contrary to the cast iron, it has a low content of carbon, an average of 0.08% but a high quantity of phosphor and nitrogen which reduces the ductility capacity and accelerates the process of ageing. The metal was used in structures until it was replaced by steel at the end of 19th century [7].

2.1.2.3 Steel

The percentage verified on a steel with a medium content of carbon is between 0,3 and 0,6%, being this material considered an upgrade of cast iron and wrought iron, improving its mechanical properties such as strength durability and ductility. This last property is essential to structural safety, since it gives a previous warning when there is a structural failure, with a reaction produced when the plastic state of the material is achieved. Although the fact that steel is a good material in multiple aspects, there are some factors that must be considered about bridge design and maintenance, such as fatigue, corrosion and connections between structural parts, known to weaken and influence the time of steel [7].

2.2 The Evolution of Steel Bridges in the UK

The use of iron as a construction material on bridges was recorded around 1780, being substituted by steel in 1880. This one became a permanent position in modern construction due to its special properties.

It is thought that the main history of metal bridges in the UK began in 1779 with the greatest construction of a cast iron arch bridge in the world, called Iron Bridge with a 30m span built by Abraham Darby III. This bridge was considered the dawn of the Industrial Revolution. The bridge is located in Coalbrookdale, England, crossing the River Severn, exemplified in Figure 2.1. Cast iron began to be used in the construction of bridges and it was an authentic revolution in bridges [10].

Figure 2.1 - The Iron Bridge [54]

Twenty years later, it was finally constructed the bridge that is still considered the largest cast iron arch bridge with a span of 73m, the Southwark bridge crossing river Thames, also situated in UK and illustrated in Figure 2.2 [10].

Figure 2.2 - Southwark bridge [11]

Many pointed arch bridges were built during the Middle Age, however it did not make a difference in its structural type, only in the shape of the arch that was considered of being less suited than the semi-circular arches for load actions. In the mid 19th century, engineers first used this material as a reinforcement because of its advantage to accommodate tensile forces.

In the early nineteenth century, suspension bridges, a type of cable-supported bridges, came also to replace the arch bridges since they could span larger decks, being ideal for covering big waterways. The cables for this kind of bridges are suspended vertically from the main cable, anchored at both ends of the bridge and running between the towers. The first to appear and rapidly spread all over the world were catenary bridges, the classical catenary bridges where the cables are separated from the deck and every part of it are in perfect equilibrium [3].

The first catenary cables being built were made of natural fibre strings, later substituted for iron chains. These ones have also suffered an evolution through the years and late in that century, it was already used high tensile cables. Today, suspension cables are made of thousands of individual steel strings bound tightly together, which gives an easier access for the maintenance of this type of bridges. This material is perfect for this kind of bridges since it can support huge tensile strengths. The first suspension bridge being constructed was made of wrought iron chains and had a span of 21.5m, being followed by a bigger one in 1823 that had parallel cables and spanned a length of 40m. Later on, these constructions permitted the building of the famous road bridge Clifton Suspension Bridge which was constructed in 1864 and has 214 m spanning Avon Gorge and the river Avon Clifton Bridge in Bristol, UK, as it is represented in Figure 2.3 [12].

Figure 2.3 - Clifton Suspension Bridge [55]

This kind of bridges are then used not just because they can span larger decks but also due to their lightweight, aesthetic appearance, high strength and capacity and finally ease of construction. However, they have a high sensitivity to dynamic loads, since the long decks and the cables create a flexibility that allow the bridge to be more sensitive to that kind of effects [12].

The first bridges constructed with wrought iron were two large box girder railway bridges named Canway and Britannia bridges over the Menai Strait, built by Robert Stephenson around 1850. These two constructions were quite popular back then, not only because they were the first made of wrought iron but they were also the first box girder being constructed. Later, in the end of 19th century, there was a spectacular development of this material, beginning the era of steel known by the “metal bridge boom” depending largely in the need of making a great number of bridges and viaducts for railway lines purpose, some of them requiring a considerable size [3].

By the 19th century, not much time after suspension bridges appeared, trussed girder bridges started to be developed, with a combination between cast iron for compression and tension members respectively. This type of bridges was especially used in the railway construction due to the need of carrying heavy loads and maintenance facilities. Shortly, bridges’ construction evolution saw a huge development related to trussed girders. The first iron truss railroad bridge, with a peculiar construction of lattice trusses, was constructed by George Stephenson in 1824 and it was called Gaunless Viaduct. This bridge used to be located in West Auckland crossing the river Gaunless, ending demolished a little over one hundred years after its opening. It can be seen an example of the ancient bridge in Figure 2.4 [13].

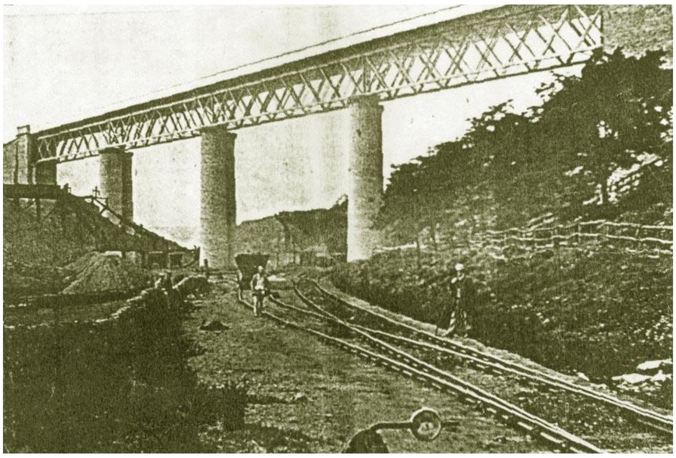


Figure 2.4 - Gaunless Viaduct [56]

Cable-stayed bridges, another type of cable-supported bridges, were recorded to be the last ones to appear in the mid twentieth century and they had a really fast development in history, both in steel and concrete. They have basically the same concept as the suspension bridges such as the presence of cables, towers and girders. However, in this case, the cables are directly attached to the deck, supporting this one and normally forming a series of parallel lines. Similar to suspension bridges, cable-stayed bridges have also a high sensitivity to dynamic behaviours and almost an equal capacity for spanning large decks, reaching lengths' up to 800m. Due to the first parameter and in addition to some probably lack of technical and analytical understanding, cable-supported bridges led to a few historic bridges' collapse. There are a lot of causes that can lead to a bridge collapse such as natural causes, accidental impacts, structural and design mistakes, construction deficiencies or even the lack of maintenance and inspection of bridges which represents only 5% of the collapses. A great example of bridge disasters happened in the UK due to miscalculation of dynamic forces is the Tay bridge disaster, designed by the railway engineer Thomas Bouch, occurred in 1879, Scotland (Figure 2.5) [3].

Tay rail Bridge was the first railway bridge being constructed in Scotland and it was classified as the longer bridge in the world, back in 1877. The bridge consisted of steel lattice girders, combining cast and wrought iron, high pillars of cast iron tube as supports and a total deck length of 3264m. In the year of 1879, a disaster derived of a storm happened while a train was passing by, ending up destroying the whole central part of the bridge along with the train. Due to the load combination of the storm and the train, a pillar ended up buckling and took the continuous bridge girder with it. It is thought that the causes behind this epic disaster are derived from the insufficient assumption of wind forces, poor workmanship and low-quality material, becoming this bridge an issue of investigation, even nowadays [3].

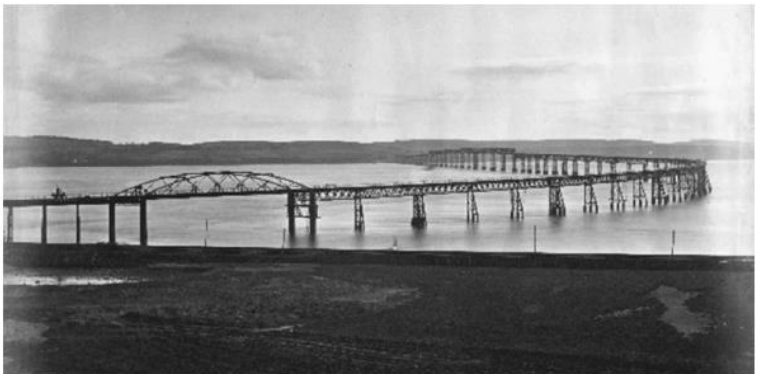


Figure 2.5 - Tay Rail Bridge [3]

Today there is another bridge replacing the old Tay bridge (Figure 2.6). It was constructed between 1882 and 1887 and spans a length of 2250 m in total. [3] This bridge was the first greater box girder bridge in Britain. The superstructure was made of a series of simply supported spans of 55m each, with two decks made of a single closed box section. For the areas of sagging moment only, it was considered composite sections whilst in the hogging moment regions have as a resistant section, the steel box section only [14].

Figure 2.6 - Tay Road Bridge [57]

2.3 The Evolution of Concrete Bridges in the UK

Early concrete bridges started to appear in an arch form, like masonry bridges, using just the compressive strength of concrete since this material has low resistance in tension. The history of concrete in the United Kingdom started in Inverness and Edinburgh, Scotland with the beginning of construction of concrete roads in 1865 when it was built the first plain concrete bridge designed by Thomas Marr Johnson, tooking place in West London. Overtime, plain concrete started to be used more and more for the construction of bridges by British engineers, having as an example of this, a three-span concrete arch, including a 15.2 m middle span, built by Philip Brannan in Devon, in 1877. Railway engineers were remarkable as well in single plain concrete in examples such as the Dochart Viaduct (Figure 2.7) built in 1886, Killin, Scotland [15] and the Cannington Viaduct in 1903. This last one shows a 186 m length with ten elliptical arches 15 m long and 28 m high, connecting the seaside town of Lyme Regis with the main line railway network at Axminster, southwest England, as it can be seen in Figure 2.8 [16].

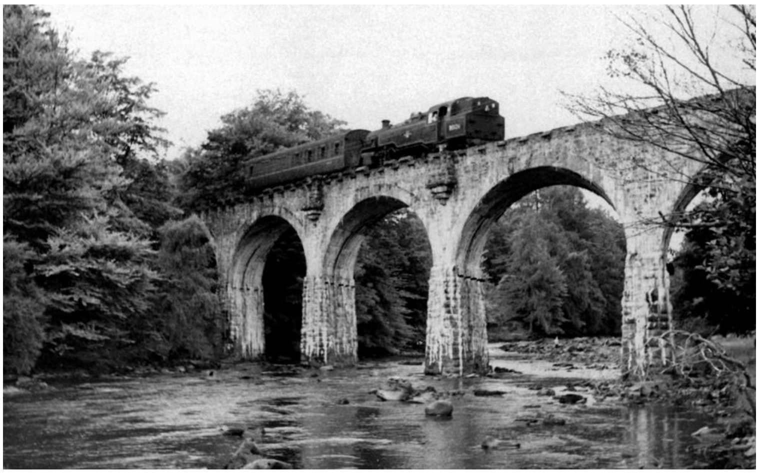


Figure 2.7 - Dochart Viaduct [50]

Figure 2.8 - Cannington viaduct [60]

In Great Britain, an example of an early mass concrete bridge is the spectacular Glenfinnan Railway viaduct in Scotland, constructed in 1897 by Robert McAlpine and opened in 1901. This viaduct has a length of 380m, 21 arches and shows a 15 m length for the biggest span. Crossing the river Finnan, this viaduct connects Glasgow with Maillag through the West Highland line which is situated 30 m above the ground and shows a curve geometry as it can be seen in Figure 2.9 [16].


Figure 2.9 - Glefinnan Viaduct [51]

In 1867, without any knowledge or any method for design calculations, Joseph Monier invented reinforced concrete, based on the idea of putting steel and concrete together and bringing the best quality of each material into performance. The first bridge constructed using reinforced concrete was probably Homersfield Bridge over the river Waveney on the Norfolk/Suffolk border in 1870 (Figure 2.10). It has 15 metres span and it is made of wrought iron embedded in concrete. About the first reinforced concrete railway bridge, Mouchel (Bristol, 1907) and Coignet (Bargoed, Wales) worked together contributing with the design of an 8.5 m span structure built in Dundee, Scotland, in 1903 [15].


Figure 2.10 - Homersfield Bridge [52]

This technique of using reinforced concrete was perfected by Francoise Hennibique, and its application to bridges was carried out by Robert Maillart during the first part of the 20th century [16]. Hence, by the 1930s, there was a significant increase in the use of reinforced concrete and so it began to have a very important role in society due to its strength, design flexibility and natural durability, becoming thus, the major construction material [17]. Around this time, there were about 2000 reinforced concrete bridges in the UK and it was still being used in the 1950s for larger bridges, especially arches. However, by the end of the 1960s it had been mostly superseded by prestressed concrete [18].

After reinforced concrete had been developed, it was noticed that the performance of the bars was improved when they were placed in tension, leaving the concrete in compression. During some experiences, although the trend to crack in tension was notoriously reduced, the cracks started to reappear after some months, it was then realised that creep occurred. So, it began the era of prestressed concrete invented by Eugene Freyssinet around 1929 [16]. In the UK, the use of prestressed concrete in bridges started in London around 1947 when it was used precast prestressed in the girders for the reconstruction of two bridges carrying roads over railways. Thus, it led to a major construction such as Nunn's bridge (Figure 2.11) in Boston, 1948, which is thought to be the first in-situ prestressed post-tensioned concrete bridge, with a span of 22,5m, result of a partnership between prestressed Concrete Company and Mouchel Consulting [19].

Figure 2.11 - Nunn's Bridge [19]

Later on, prestressed concrete rapidly superseded reinforced concrete, gaining a more important role in the late 1960s with the arising of huge constructions such as Hammersmith Flyover (Figure 2.12), the first modern elevated urban motorway constructed in 1961 that became a landmark structure in the UK. The increasing need of construction of motorways due to the vast growth of load traffic led to a large number of concrete bridges, not specifically because of the style, but due to economy and durability issues that became more important overtime [18].

Figure 2.12 - Hammersmith Flyover [53]

As well as the innovation of prestressed concrete, precast concrete showed up just in the right time also showing a constant growth in its techniques such as the number of prefabricated bridges, its size and weight, helping thus the economy section by being a faster solution. [19]

2.4 The evolution of steel-concrete composite bridges in the UK

The beginning of construction of composite bridges came with the development of concrete structures in the 1800s when it was experimented the embedment of iron into the concrete, producing the first iron–concrete composite structures. An example of this type of structure is Brunel's bridge near Paddington [20].

United Kingdom has been recognized by their tradition in the construction of steel bridges, being one of the first countries introducing the construction of composite bridges. During the 1950s and 1960s a big evolution of the railways and motorways has been registered, helping in the growth of this type of bridges. Therefore, Pelham bridge (Figure 2.13) constructed in 1958 and located in Lincoln, UK, was one of the earliest bridges introduced in this type of construction. It is constituted by four plate girders site welded with steel-rubber laminated bearings. The design for shear connectors has been based in tests carried out by the University of Illinois, USA, since at the time it did not exist standards for composite bridges [21].

Figure 2.13 - Pelham bridge [21]

In the 1960s, the industry of steel in the UK was known by their high yield steel quality and their fast improvement in welding techniques, which led to an increase of competitiveness regarding steel and composite constructions. It has also been noticed that composite constructions started with medium spans being rapidly extended for large spans which was only verified in prestressed concrete box girders until then.

The first publication related to composite bridges showed up in 1967 with the publication of *CP117 – Composite Construction in Structural Steel and Concrete, Part 2: Beams for Bridges.* In 1969 was constructed one of the first modern composite bridges that presented a complete continuousness of the girders over the columns, called M74 Raith bridge (Figure 2.14) spanning Clyde river. Raith bridge is composed by a pair of trapezoidal open top box sections with transverse steel cross girders supporting a reinforced concrete slab [14].

Figure 2.14 – Raith bridge [14]

In 1973, a new approach for composite bridges has been brought by G. Maunsell and Partners in South Wales. The bridge was named Saltings viaduct and consisted in a continuous structure with spans of 31 m and a plan radius of 1000 m, having as structural elements four steel box girders with a reinforced concrete deck slab, as can be seen in Figure 2.15 [14].

Figure 2.15 – Saltings Viaduct [14]

After the new regulations established in the UK with the publication of *BS 5400: steel*, *concrete and composite bridges*, in 1979, Friarton Bridge (Figure 2.16) arose as a typical bridge designed under those standards. The bridge is also a steel box girder bridge, displaying a tapered section, with a concrete slab, across the River Tay in the outskirts of Scotland, Perth [14].

Figure 2.16 – Friarton bridge [59]

It is recognised nowadays, that composite bridges represent 8% of the total existing bridges in the UK, which is already a significant number, knowing that this type of construction is the most recent one [14]. This fast growth of the composite bridge industry can be justified by the several advantages existent in this type of construction such as the high tensile and shear strength united with the low cost of the compressive strength of the concrete, the rapidly construction and having a less critical demolition than prestressed concrete box girders (significant for medium span bridges). However, composite bridges are more suitable for simply supported beams since, when considering continuous beams, although they can take full advantage of the sections resisting to sagging moments, the same does not apply for the negative bending regions. It can be justified by the fact that concrete is exposed to tension and steel is exposed to compression [22].

Within composite bridges construction, there are several types of bridges that can be adopted by the British design, being the more common described below:

- Multi girder systems
- Twin I-girders
- Multiple box girders
- Open top trapezoidal box sections

The first composite bridges to appear was the multi-girder bridges (Figure 2.17) being distinguished by having multiple I-girders or trusses supporting the concrete slab. The main idea was to secure a substitute load path in the case of a girder fail. Besides that, the spacing of the multiple I-girders is usually small which allows the use of thinner (200-250 mm) precast deck panels [14].

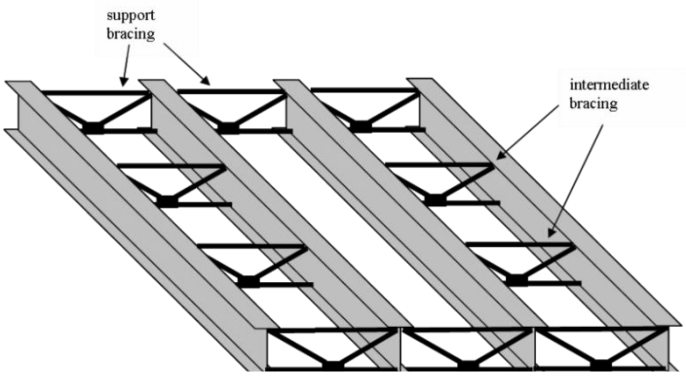


Figure 2.17 – Multy-girder system [8]

Overtime, this kind of construction has been abandoned and replaced by other types of composite bridges, already mentioned above. Twin I-girders (Figure 2.18) started to develop with the advances in welding techniques of thick plates. These bridges can span from 50 m to the much longer span of cable-stayed bridges (approximately 800 m) and besides the fact of needing less material than the multi girders system, they also need less web stiffeners, cross-frames and lateral bracing [22].

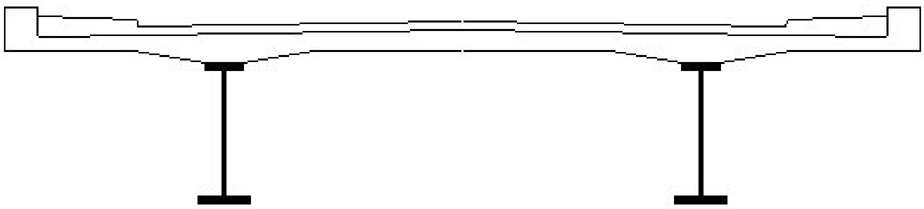


Figure 2.18 -Twin I-girders without cross-girders [49]

However, if it is intended a better stability of the system, it can be obtained by the invert U-frame action using cross-girders that provide restraint to the lateral torsional buckling of the main girders.

The multiple box sections (Figure 2.19) are usually appropriate for medium spans with curved alignments, due to the higher torsional effects introduced by it, on the girders. Despite of the initial high costs, it can bring some advantages, such as the box stability, span long sections can be erected without bracings and the span lengths are more economic than for concrete constructions. This type of composite bridges is very common in the UK [23].

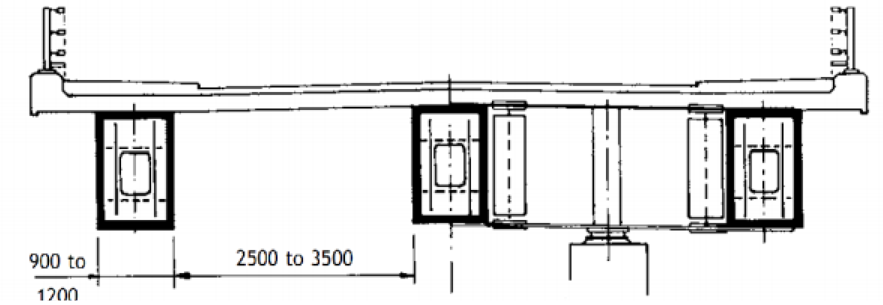


Figure 2.19 - Multiple box girders [14]

At last, open top trapezoidal box sections with inclined webs (Figure 2.20) are also frequently used in the UK since the 1990s. This solution is even more indicated for significant curvature plans requiring a high torsional stiffness. Despite of the similar characteristics with box girders, this type of construction shows other advantages such as the inclined webs design since it allows the reduction of the width of the bottom flange, the thickness of the concrete slab and finally the need for less number of longitudinal girders [23].

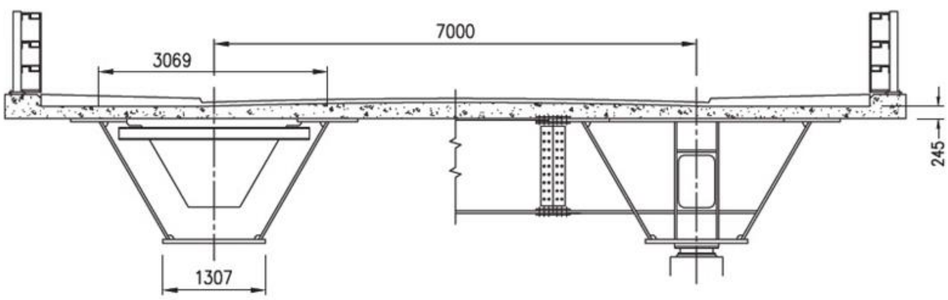


Figure 2.20 - Open top trapezoidal box sections [8]

Chapter 3

3. Introduction of the Eurocodes and UK standards (BDs)

3.1 Eurocodes Standards

3.1.1 Brief History of the Eurocodes

The need for the use of standard rules in construction's field was first discussed in 21 July 1908 by the Concrete Institute, in the Ritz Hotel, London. Their work was recognized in the London County Council (LCC) Regulations of 1916 which kept in collaboration with the Department of Scientific and Industrial Research (DSIR). In 1922, the Concrete Institute have grown into the Institution of Structural Engineers. Later in 2008 the Institution, that first started with just 100 people, was already counting with a body of 22 000 members, in more than 100 countries.

Although reinforced concrete had already been used some years before, the main aims were based on the design rules for the new material. Thus, the institution followed the history from a concrete design point of view, allowing other engineers to apply the new technology. Therefore, several codes and guidelines had been developed since then and have been extended beyond many National Requirements, taking the UK a full part in this process.

The Eurocodes' history started around 1971-1976, when a steering committee was organized by the Public Procurement Directive in order to purpose the development of a common European code that could cover the design of a great diversity of construction works. Their goals were specially to harmonize technical specifications, eliminating all the technical obstacles. Hereupon, the first set of technical documents were drafted between 1976 and 1990. Their conversion of the first Eurocodes into provisional European standards (ENVs) was made by CEN (European Committee for Standardisation) technical Committee around 1990. At last, the conversion of the provisional European standards into European Standards (EN) was made between 1998 and 2006 after a test period, going only through maintenance and evolution from there on. Nowadays, it is conclusive that their format and content are due to an association between professional institutions, the national legal authorities, academic study and practising engineers. This has resulted in a cultural, legal and technical development not only for the authors of the documents but also for society who rely on them, despite of their unfamiliarity with its content.

Public safety, durability and serviceability are really important issues for society, becoming significant aspects for regulation of construction. It brought then, new rules into the Eurocodes regarding existing structures which accurately are object of concern since they have undergone a long history of evolution. However, this concept does not mean that structures became suddenly unsafe, this only draws attention to the need of understanding, for instance, the changes on the load and consequent behaviour in the structures [24].

3.1.2 Organization of the Eurocodes

Nowadays, the Eurocodes can be found in 10 significant categories, discriminated below (Table 3.1):

Table 3.1 – The 10 categories within the Eurocodes standards

EN 1990	Eurocode 0	Basis of Design
EN 1991	Eurocode 1	Actions on Structures
EN 1992	Eurocode 2	Design of concrete structures
EN 1993	Eurocode 3	Design of steel structures
EN 1994	Eurocode 4	Design of composite steel and concrete structures
EN 1995	Eurocode 5	Design of timber structures
EN 1996	Eurocode 6	Design of masonry structures
EN 1997	Eurocode 7	Geotechnical design
EN 1998	Eurocode 8	Design of structures for earthquake resistance
EN 1999	Eurocode 9	Design of aluminium structures

These codes have been approved by CEN and they could co-exist with the appropriate National code under the rules of the same authority.

3.1.3 Aims of the Eurocodes

With the introduction of the Eurocodes, many aspects regarding structural design became easier to manage. Their purpose was to serve as reference documents for structural design, trying to include the project and verification of any type of construction. To accomplish that, they had to follow some basic common rules such as:

- The requirement for public safety and serviceability of structures, based on the principle of risk, particularly emphasizing the safety in case of fire, the mechanical resistance and stability;
- Health and safety and environment protection;

- The independence of advice and updated information;
- References of material properties, taking an appropriate theory approach for structural use;
- The durability of the structure under normal maintenance conditions, regarding economic concerns;
- To make harmonized technical specifications for construction products and engineering services, being a support for contract and specification and at last;
- Improving the competitive market between European construction Industry and the industry outside the European Union [25].

In summary, these codes are mainly to simplify the process of design, providing common structural design rules based on calculation methods, current tests, non-linear and finite element analysis, used all over the European Union [24].

3.1.4 Eurocodes Format

The common standards around European Union have a common format, being distinguished from the other codes by the clauses. They are only designated either as Principles or Rules of Application. Principles are known as the fundamental bases of structural performance that must be attained. They are identified by the letter “P”, following the paragraph number.

Rules of application are appropriate methods that must be used in order to comply those principles and satisfy their requirements. If there are alternative methods replacing the rules of Application, those ones have to be according the principles and provide the same level of reliability. Regarding rules of application, there are many parameters where only indicative values are given, being thus necessary for each country to specify their own equivalent values. These values are indicated by being enclosed by a box (|____|) and are established in the National Application Document (NAD). In this document, it is also possible to find amendments to the Eurocodes if the rules either do not apply or are incomplete.

Furthermore, The Eurocodes also have an open part that is intended for National choice, known as Nationally Determined Parameters and it is disposed in the National Annexes. The National annex may only contain information on those parameters already left open for that purpose, such as country specific data. In the other hand, everything that is considered normative, with no choice, it is transformed into National standard, comprising the full text of the Eurocode. This part may be preceded by a National title page and a national foreword.

Finally, the codes always present a set of terms and definitions to clarify the meaning of some technical expressions and symbols used further ahead [25].

3.1.5 Materials

Since it is being made an approach regarding steel, concrete and composite bridges, the references further ahead will be only around these materials. The codes associated with the mentioned materials are EN 1992, EN1993 and EN 1994, respectively. The part 2 (EN 1992-2, EN1993-2 and EN 1994-2) of each Eurocode is particularly apply on bridges.

These materials can appear in many forms, categories and conditions, depending on the process of fabrication or even on the need for the structure's design. Hence, they will have a wide variety of resistance that can be checked for limit states, through specific tests according to the rules.

3.1.6 Actions

The actions applied will focus on the existent actions on bridges. These include all the general actions indicated by the Eurocode 1, part 1 (EN 1991-1) and the specific actions of traffic load described in the part 2 of the Eurocode 1 (EN 1991-2).

The Eurocodes are a practical way of assisting with design and structural verification (here after designated assessment), through an organized group of codes and standards. Hence, they have methods of procedure according with the following criteria:

- ❖ Verification of break or excessive deformation of the structure (STR)
- ❖ Verification of loss of the static equilibrium of the structure or their structural elements (EQU)
- ❖ Verification of break or excessive deformation of the soil that support the structure (GEO)

They include calculations procedures complying limit states approach that must be checked to verify safety requirements of structures. A limit state is a threshold that defines the moment when a structure no longer complies with a specific requirement, being then limited in some functionalities. They are:

- ❖ Ultimate Limit State: A state associated with structural failure and may compromise the safety of the structure and people.
- ❖ Serviceability Limit State: A state associated with the serviceability of the structure, not compromising people safety, but only the functioning of the structure, its appearance or the comfort of people [26].

3.2 UK Standards (BDs)

3.2.1 Brief History of BD Standards

In the late 1950s was witnessed a rapid development of the UK road and motorway regarding structural design. Ministry of Transport (MoT) was the entity who introduced a new guidance document regarding this subject. This document consisted in guidelines for standards, methods of construction and loadings.

The document relied on the existing British Standards and codes of practise for the essential requirements of steel and concrete. Afterwards, a Specification for road and Bridgeworks was also published in 1951 by the Department of Transport (DoT) as a first edition which has passed through several reviews and updates. With this, supervision of design development became the main concern for bridge engineering. Consequently, there was a huge development in the knowledge and techniques of design, which forced the need to review codes of practise every four years as well as the need for the creation of new codes.

Only by mid 1960s, due to the impossibility of a general code that could embrace all of the bridges' design, the British Standards Institution (BSI) set up a committee to create a standard code specifically for steel, concrete and composite bridges. Later, in 1978, as expected, they have been part of the national code within BS5400, the British Standard associated with steel design. This code was the first limit state code of practise for bridge design and construction in the world.

Similar to the making process of the Eurocodes, the Bridge Engineering Division also had an important role with regard to ensuring that the new information and development was translated into departmental and national standards, design guidance and specification. By 1965, 1000 designs of Bridges had been already approved by that division. Nevertheless, and apart from the constant appearance of new works, a growth concern of the maintenance of existing bridges started to take place due to the increase of traffic volume and weight.

With the Bridge Engineering Division also responsible for the issue mentioned previously, the "Operation Bridgeguard" emerged in 1967 with the purpose of assessing the load-carrying capacity and the condition of the nation's bridges, constructed before 1922. However, this programme could not be put in practise straight away, due to the lack of technical and financial resources, being completed only some years later. This introduction transformed the assessment of the Nation's bridges into a permanent task undertaken and managed by the DOT, local highway authorities and other bridge owners.

Thus, bridges considered as weak, started to have weight limits, as well as, in some cases, width restrictions, while they could not be strengthened. Although in other cases, some of them were even demolished and rebuilt afterwards.

By the 1970s, a notorious evolution of standards and research had already undergone a great evolution. Understandably, Bridge failures played a really important role in this subject, bringing radical changes to procedures [27]. In 1973 the maximum weight allowed on bridges was 32 tonne gross vehicle and a 10-tonne axle. With the constant increase of vehicles, in 1983 the European Commission extended it to a minimum of 40 tonnes [28]. However, if the structure cannot carry that capacity, it should be assessed for 26 tonnes, 18 tonnes or 7,5 tonnes Assessment live loading [29]. Following a number of bridges failures, DoT's system of certification of bridges was then extended. The changes made to the technical approval procedures were the following:

- The responsibility to evaluate the design criteria and methods was kept entirely left for DoT.
- DoT required for every engineer to provide an independent check of the design and calculations.
- An Approval in Principle' stage was required for all structures, with exception for the minor ones [27].

Approval in Principle (AIP) for a bridge assessment is a document describing the bridge type, the agreed basis, standards and methods of analysis for the detailed assessment of the bridge. Afterwards, this document is delivered to Technical Approval Authority (TAA) for approval [30].

These changes to the Certification procedures were mainly to reduce the risk of structural failure but also for effective management of existing bridges. Therefore, besides the prevention regarding loading and material risks, certification also tried to reduce the human risk by requiring second opinion of an independent checker, at three levels of rigour, as can be seen in Figure 3.1 [27]. These assessment checks are carried out according to *BD 2/12,* based on the structure category which is defined by its complexity, size, value and economic importance [28].

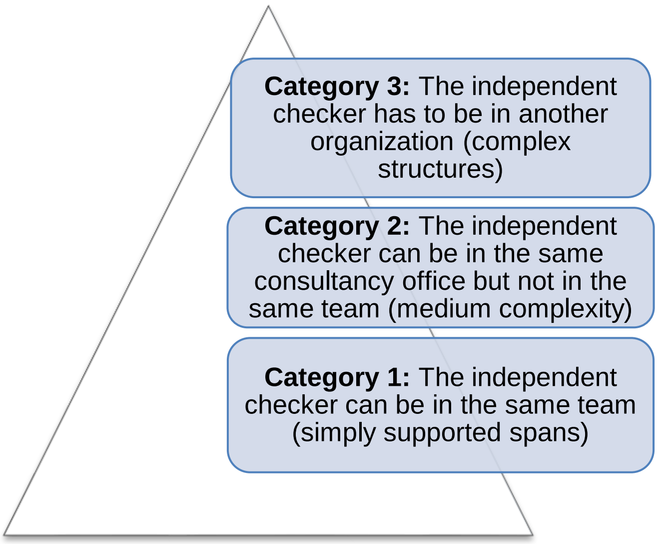


Figure 3.1 – Assessment check categories

DOT, the Technical Approval Authority assigns the category 3, the highest one, which is associated with the bridges of bigger complexity and economic importance. For this category, there is another independent checking team that has to be in agreement with the design team, otherwise, DOT will be the entity who makes the final choice. Category 2 is a lower category where it is only required an agreement with an independent checking team within the same consultancy office. Finally, for category 1, the simplest type, the designer and the checker can be in the same team.

It is important to highlight the fact that the assessing engineer must agree with the Technical Approval Authority for the category of the structure, being that agreement registered in the AIP [28]. The basis for these decisions are recorded in the standards previously mentioned, with *BD 21/84 The Assessment of Highway Bridges and Structures* being the first introduced and its additional Advice Note BA 16/84 which were mainly concerned with the simple methods of load distribution. Many other standards BDs and BAs (standing for Bridge Departmental standards and Bridge Advice Notes, respectively) were then developed overtime changing significantly the Bridge Engineering Department in the UK [31]. In conclusion, the assessment and maintenance of bridges, can be as important as design itself, bringing sometimes a higher challenge for bridge engineering [27].

3.2.2 Organization of BD standards

Codes and standards for the Assessment of bridges have a different form of organization from the Eurocodes. These codes, included in DMRB (Design Manual for Roads and Bridges) standards are an ample group of documents that can be consulted or not, depending on each type of bridge, or even the type of assessment. They can be found in a collection of volumes that goes from 1 to 15 and they relate to Standards for Highway works. Each volume is divided by sections which are organized in parts that can be defined as BDs. In other words, BDs and BAs are codes divided in categories needed or not, for the assessment of a specific bridge type. The examples below (Table 3.2) are particularly used for the case studies discussed in the next chapter:

Table 3.2 – Some codes of practise included in DMRB standards

BD 63/07	Inspection of Highway Structures
BD 21/01	The Assessment of Highway Bridges and Structures
BD 37/01	Loads for Highway Bridges
BD 44/15	Assessment of Concrete Highway Bridges of Structures
BD 86/11	The Assessment of Highway Bridges and Structures for the effects of Special Types General Order (STGO) and Special Order (SO) vehicles
BD 56/10	The Assessment of Steel Highway Bridges and Structures
BD 61/10	The Assessment of Composite Highway Bridges and Structures

It is important to emphasize that each code applied to the motorway road bridge stock in the UK is defined by an application standard in DMRB. For example, the current British Standard for concrete bridges *BS5400-3*, is implemented by *BD56-10*. Therefore, the BDs and BS are used together in parallel. [31]

3.2.3 Aims of BD standards

As discussed previously, the UK standards (BDs) for structural assessments are codes whose main objective is to set the minimum requirements to guarantee certain levels of performance. These are mainly to improve the economy, safety, integration and accessibility as follows:

- Reducing the life cost of a structure by developing management systems that help bring good conditions for inspection, maintenance and repair works;
- Trying to make bridges safe during its serviceability period, with the minimum effort but the maximum accuracy;
- Making periodic inspections that can provide sufficient information regarding the type of construction and the conditions established, helping management and maintenance of the structures [27].

3.2.4 Inspections

Inspections are a major subject when it comes to structural assessments, besides being important regarding safety of bridges during its serviceable state. To reach the phase of assessment requirement, it means that the process has already went through a long path or a special proposal was simply required by a superior entity. These inspections are carried out in order to analyse the construction of the bridge, its dimensions and condition. In this process, they attempt to manage and plan the maintenance of bridges, acting consequently in order to preserve them, through procedures such as repair, rehabilitation or strengthening [32].

Inspections can be divided in five different types. They are:

- General Inspection
- Principal Inspection
- Safety Inspection
- Special Inspection
- Inspection for Assessment

3.2.4.1 General Inspections

General inspection is a type of inspection carried out within a visual distance. Usually, it is repeated in 2 years' time and it does not require any type of equipment or traffic restriction. The main intent is to acquire every possible information and the restrictions found around the structure.

Therefore, it is necessary all the previous data of the bridge, such as historical information, previous maintenance works, features of the bridge and previous defects or inspection reports [28].

3.2.4.2 Principal Inspection

Principal inspection goes into more detail than the one mentioned above and is based in a close distance check (within an arm distance), also known as “close visual inspection” (CVI). For this inspection, all the parts of the bridge have to be inspected and it is carried out every 6 years, although it can also be done in shorter or longer intervals, if a risk assessment thus justifies it [28].

3.2.4.3 Safety Inspection

This inspection is not that common and is only required at special times. In other words, this is for bridges that are under imminent risk or present obvious deterioration or defects that can cause serious damage to the public or the structure [28].

3.2.4.4 Special Inspection

Special Inspection is similar to safety inspection, although it is focus in just an element that can be causing some concern to the assessment team of engineers. For example, the check for any defects caused by a vehicle impact. This inspection is carried out at any time when needed. This inspection can be only visual or include testing and/or monitory. It depends on the condition of the bridge and urgency to act upon [28].

3.2.4.5 Inspection for Assessment

Last but not least, there is the inspection for assessment which is also specially required for structural assessment of bridges. For this one, it is important the possession of every information possible, that is, all the reports from previous inspections, historical information, assessment drawings and elements data. This inspection is normally done together with the principal or special inspection [28].

3.2.5 Materials

Similarly to Eurocodes, standards for structural assessments in the UK, introduced national standards and guidance codes that allow all bridge engineers to follow the same process. Although in this case, BDs are specifically used for existent bridges, being thus consulted in parallel with the British Standards.

As already mentioned, there are some specific standards (BDs and BSs) relative to concrete, steel and composite bridges, specifically:

- BD 44/15 – Assessment of Concrete Highway Bridges of Structures
- BD 56/10 – The Assessment of Steel Highway Bridges and Structures BD 61/10
- BD 61/10 – The Assessment of Composite Highway Bridges and Structures
- BS 5400 (Part 3 and 6)– Steel, Concrete and Composite Bridges

3.2.6 Actions

The relevant actions and respectively combinations for the assessment of short/medium span bridges are discriminated in the following codes:

- BD 21/01 – The Assessment of Highway Bridges and Structures
- BD 37/01 – Loads for Highway Bridges
- BD 86/11 – The Assessment of Highway Bridges and Structures for the Effects of Special Types General Order (STGO) and Special Order (SO) Vehicles.

These codes contain all the information regarding magnitudes, derivation and application of these loads on most type of bridges.

3.2.7 Comparison between design and assessment of bridges

There are many differences regarding design and assessment, starting with the fact that structural assessment is made for existing bridges. Therefore, the first stage of structural assessment is focused on checking the load carrying capacity based on existing properties and constraints. In the beginning of the process they are provided of existing information such as as-built drawings, inspection reports, surveys, maintenance history, previous reports and similar documents. With this information, the current condition of the bridge is determined in order to carry an inspection for assessment according with BD 21/01 and confirm if the last inspection is still accurate, regarding geometrical information, material properties and so on.

An AIP is then prepared to send to TAA for approval and receive feedback in the chosen procedure. Afterwards, a visit to the site and following assessment, if needed, is carried out for posterior results and conclusions. If the bridge passes the assessment, the results are documented for managing and maintenance. If the bridge fails in the assessment, it has to be properly managed by the Overseeing Organisation.

In the design procedure, in the first stage, a range of viable solutions is given in order to compare them and choose one that suites the client's requirements and site restrictions. Then, a detailed design of the elements and materials is put into practise in order to achieve sufficient structural capacities [28].

Chapter 4

4. Main Differences between the Eurocodes and UK standards (BDs)

4.1 Classification of Actions

In a development of a structural design, it is necessary to quantify loads acting on the structure, as well as various combinations of different actions of loads. For assessments, the same principle is applied since the main objective is to check the load carriageway capacity of the structure in view of its properties and the most up-to-date actions set up by standards or codes. Depending on the time variable, they can be inserted in three different categories:

- Permanent Action: Action that, in most of cases, stays constant over time and can be defined as the self-weight of the structure, as well as the non-structural elements that are part of the structure, such as surfacing, filling, kerbs and parapets. However, it can also represent indirect actions caused by prestress, shrinkage or creep, which despite tending to stabilize, change overtime.
- Variable Action: Action that varies over time such as surcharge, wind, snow, temperature or even moving loads of vehicles;
- Accidental Action: Actions that have a lower probability of happening but can cause serious damages, such as explosions or impact from vehicles [14].

4.2 Material Properties

Both Eurocodes (ECs) and UK standards (BDs) state that properties of materials should be represented by a characteristic value. A characteristic value is associated with a probability of non-exceedance different from 50%. A probability of 50% usually corresponds to the mean value (assuming a normal density function/distribution). As materials properties are favourable, they contribute to structures' capacity, the characteristic values are usually defined by standards with a probability of non-exceedance of 5%. For actions, unfavourable to structures' performance, their characteristic values are usually set up by standards/codes with a probability of non-exceedance of 95% (which evidently corresponds to a probability of exceedance of only 5%), as shown in Figure 4.1. In the case of EC0, a Gumbel distribution is considered, and the required characteristic values are quantified for a probability of non-exceedance of 98% (2% of exceedance).

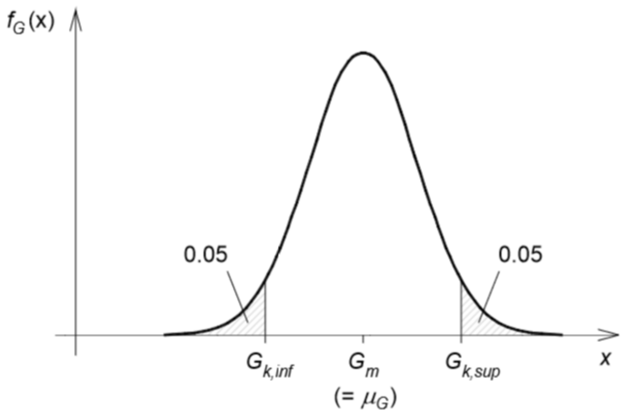


Figure 4.1 – Gaussian distribution of the characteristic values

As regards properties of materials, these values, obtained from a limited number of standardised tests carried out under certain conditions, are tabled in the appropriate places within each code.

4.2.1 Unit weight of materials

The table with the Unit Weight of materials can be found in *Table A.6* of *Annex A*, *EN 1991-1-1* [26] and *Table 4.1* of *BD 21/01* [29].

4.2.2 Modulus of Elasticity (Young Modulus)

The Young Modulus values used for both steel and concrete materials in BDs are characteristic values [29]. In the case of the Eurocodes, the values used for concrete are mean values whilst the values adopted for steel can range from 195 and 210 GPa, depending on the manufacturing process [33].

4.2.3 Yield Stress

1) <u>Steel</u>

EC - Nominal values of yield strength (f_y) and ultimate strength (f_u) for structural steel are directly taken from the product or using the *Table 3.2*, *EN 1993-1-1* [33].

BD - When the information about the material properties allow to identify the grade and specification of steel, it is recommended to use *Annex H, BD 56/10*. Table 4.1 provides consistent values of the ratio of the product of yield stress and thickness to that of nominal thickness and specified minimum yield stress [34].

Table 4.1 – Yield stress to be considered in an assessment [34]

Standard	Grade	Quality	Minimum yield stress (MPa) for thickness t in mm (lower of values for flat and long products)					Tensile strength range (MPa)	Notch impact test temp (ºC)
		D							-20
	A330-1 or A330-2	B	195	185	175	°	°	330-410	-
	AE255 or A410	B	235	225	215	°	°	360-440	20
		C							0
		D or DD							-20
	AE355 or A510	B	355	345	335	°	°	510-610	20
		C							0
		D							-20
		DD						490-610	-20
	AE410	DD	410	400	390	°	°	530-670	-20
BS 4360-1972 (Britain) (earlier norms BS 15, BS548, BS968, BS 2762 and BS 3706)	°		t≤16	16<t≤25	25<t≤40	40<t≤63	63<t≤100		
	40	A	-	-	-	-	-	400-480	-
		B	230	225	225	220	210		20
		C	230	225	225	220	210		0
		D	240	230	225	220	210		-20
		E	255	245	240	230	225		-50
	43	A	245	240	240	230	220	430-510	-
		B	245	240	240	230	220		20
		C	245	240	240	230	225		0
		D	255	245	240	230	225		-20
		E	270	260	255	245	225		-50
	50	B	355	345	345	340	325	480-620	-15
		C	355	345	345	340	325	480-620	-30
		D	355	345	345	340	°	490-620	-50
	55	C	450	430	415	°	°	550-700	0
		E	450	430	415	400	°		-50

2) Concrete

EC – Strength classes in this code are represented by the characteristic cylinder strength f_{ck}, obtained at 28 days. The Eurocodes also present a correlation between cylinders and cubes for each class [35].

BD - The concept of *worst credible strength* is introduced in BD standards. This strength can be defined as the worst realistic strength that, based on test experiences and knowledge of the material, could be obtained in the element under consideration. This value may be greater or less than the actual value defined at the design stage. This concept should only be used in the following conditions [36]:

- ❖ If the assessment to the structural element has shown that, using its characteristic value, the element is incapable of carrying the full assessment loading;
- ❖ If the structure has suffered deterioration in such a way that the actual strengths are known of being less than the characteristic values;
- ❖ Where the information about the characteristic value used in design, does not exist and it is not appropriate to adopt a value from BD 21/01.

Otherwise, strength classes in this code should be represented by the characteristic cube strength of concrete, f_{cu}. Table 4.2 summarizes where some properties of materials can be found in both standards.

Table 4.2 - Specified documents to find some properties of materials

	Eurocodes	***BDs***
Unit weight of materials	*EN 1991-1, Annex A.6*	*BD21/0, Table 4.1,*
Modulus of Elasticity	*EN 1992 to EN 1994*	*BD21/01, Table 4.2,*
Linear thermal Expansion	*EN 1992 to EN 1994*	*BD21/01, Table 4.3*
Yield Stress (Steel)	*EN 1993-1, Table 3.2*,	*BD 56/10, Annex H*
Yield Stress (Concrete)	*EN 1992-2, Clause 3.1.6*	*BD 44/15, Clause 2.1.3.1*

4.2.4 Partial Safety Factors

Several methods have been proposed to try to guarantee certain levels of safety of structures. These are basic sciences, such as mathematics, computer science, statics and probabilistic; structural mechanics and legal frame works, such as norms and codes.

The partial factor method is a semi-probabilistic approach which is a common system used in all standards related to design and assessment. It is used to increase the safety of structures by reducing the probability of any limit state being exceeded, as shown in the equation below (4.1):

$$S_d(\gamma_p \times p_k) \leq \frac{R_k}{\gamma_{res}} \tag{4.1}$$

Where S_d, represents the computational model, p_k the characteristic value, γ_p and γ_{res} are the partial factors related to the action and the resistance of the material, respectively.

Based on Structural Reliability approaches, safety factors are associated with the uncertainty of occurrence of actions or deviations from the real capacity of the material. They are established according to a fixed level of reliability associated with a probability of failure, considered as being acceptable, e.g. 10^{-5}.

Reliability is defined from a method that implies limit state probabilities of a structural system under adverse loads. Therefore, considering a function, $Z = R - S$ as the limit state function which represents the margin-to-failure, the probability of failure (P_f) corresponds to the probability of this function being less than zero [$P_f = p(Z < 0)$], it means that the resistance will be less than the action [37].

Knowing that the mean value of the function can be described as the following:

$$\mu_Z = \mu_R - \mu_S \tag{4.2}$$

And the standard deviation according to:

$$\sigma_Z = \sqrt{\sigma_R^2 + \sigma_S^2} \tag{4.3}$$

Then, if Z is Normally distributed, the probability of failure will be:

$$P_f = \Phi\left(-\frac{\mu_Z}{\sigma_Z}\right) = \Phi(-\beta) \tag{4.4}$$

Which finally leads to the value of Reliability Index:

$$\beta = \frac{\mu_z}{\sigma_z} \tag{4.5}$$

Therefore, as graphically represented in Figure 4.2, the reliability index β increases as the probability of failure decreases.

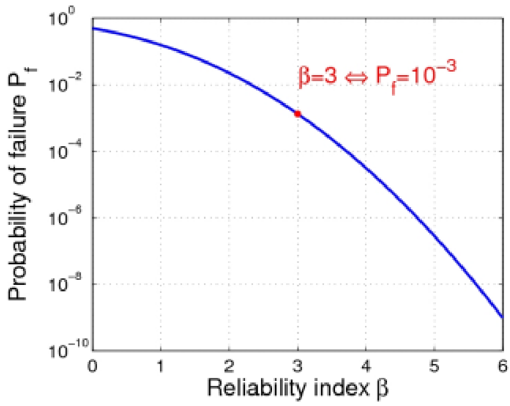


Figure 4.2 – Reliability Index depending on the probability of failure [58]

Reliability associated with partial safety factors is then focused on the accuracy of the latter, when related to an action or a resistance. They are not just based on the inherent variability of the properties, but also take into account the model errors and approximations, such as the lack of knowledge or even numerical algorithm errors [37].

Table 4.3 shows the different partial factors in both standards. It is also possible to consult in Table 4.4 an example of the partial factors used in the standards related to actions of different combinations. In this case and associated with the case study presented further ahead, these partial factors used in the combination of actions are specifically used in fundamental combination described in (4.7) and combination 1 described in section 4.3.2.

Table 4.3 - Description, symbol and localization of the partial factors in both standards

	DESCRIPTION	EUROCODES	BDs
ACTIONS	Takes account of the uncertainty in the effects of actions	γ_F *EN 1990-1, Annex A2,*	γ_{fl} *BD37/0, Table 1*
RESISTANCES/ MATERIALS	Takes account of the uncertainty of the resistance model and geometric deviations	γ_m *EN 1990-1, cl. 6.3.5,*	γ_m *BD21/01, Table 3.2*
MATERIALS	Takes account of the possibility of an unfavourable deviation of a material	γ_m *EN 1990-1, cl. 6.3.5*	γ_m *BD21/01, Table 3.2*
ASSESSMENT	Takes account of inaccurate assessment	_____	γ_{f3} *BD21/01, cl. 3.10,*

Table 4.4 - Partial Factors used in the case study for fundamental combination and combination 1, respectively

Eurocodes

Actions (fundamental comb)	
γ_G - Permanent Actions	1,35
γ_Q - Variable Actions	1,50

	Resistance/Materials	
Persistent, transient Actions	γ_c - Concrete	1,50
	γ_s - Steel	1,15
Accidental Actions	γ_c - Concrete	1,20
	γ_s - Steel	1,0

BDs

Actions (Combination 1)	
γ_{fl} - Concrete	1,15
γ_{fl} - Steel	1,05
γ_{fl} - Surfacing	1,75
γ_{fl} - Other Loads	1,20
γ_{fl} - Live Load	1,50

Resistance/Materials	
γ_m - Concrete	1,50
γ_m - Steel	1,05
γ_m - R. Steel	1,15

Assessment	
γ_{f3}	1,10

4.3 Combination of Actions

4.3.1 Combination of Actions according to the Eurocodes

In the present work, the structural assessment was carried out only for the Ultimate Limit State, as the bridge has not shown evident signs of underperforming for deflection or vibration. Furthermore, it will only be made reference to static analysis whereby it has not been considered dynamic actions such as earthquakes, wind and snow. An initial assessment estimate has shown collapse (ULS) as governing. For a structure to be compliant regarding safety, the following equation shall be verified:

$$E_d < R_d \tag{4.6}$$

Where E_d corresponds to the value of the effect of actions, such as an internal force, and R_d is the equivalent resistance of the structural element. Therefore, actions that occur simultaneously, should be combined among each other in order to obtain the critical case of the effects of actions. Each combination should include a leading variable action or an accidental action, being two main equations defined by *EN 1990* [38]:

Combinations of actions for persistent or transient situations (Fundamental Equation):

$$\sum_{j\geq 1} \gamma_{G,j} G_{k,j} + \gamma_p P + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i}\, \psi_{0,1} Q_{k,i} \tag{4.7}$$

Combinations of actions for accidental situations:

$$\sum_{j\geq 1} G_{k,j} + A_d + (\psi_{1,1}\ or\ \psi_{2,1}) Q_{k,1} + \sum_{i>1} \psi_{2,1} Q_{k,i} \tag{4.8}$$

It is important to mention that when an accidental action is taken into account, no other accidental action needs to be considered at the same time. The recommended values of ψ factors for road bridges can be found in *Table A2.1, EN 1990* [38].

4.3.2 Combination of Actions according to BDs

There are five different combinations of actions that can be applied on road bridges specified in *BD 37/01*. Although in chapter 5 the only used combinations have been combinations 1 and 3, the first three combinations are the main ones and the last two, not so important. The actions within the combinations should be chosen and applied in order to produce the most critical effect in the element or structure under consideration. The combinations are described in the list below [39]:

- Combination 1: For highway and foot/cycle track bridges, the permanent loads shall be combined with the primary live loads;
- Combination 2: For all bridges, the loads considered in combination 1, shall be combined with the action of wind and erection load;
- Combination 3: For all bridges, the loads in combination 1 shall be combined with the action of temperature, erection, or other restrain actions;
- Combination 4: For highways bridges, the loads in combination 1 shall be combined with the secondary live loads and primary live loads associated with them;
- Combination 5: For all the bridges, the permanent loads shall be combined with due to friction at bearings[2].

The consultation of *Table 1* from *BD 37/01* is suggested for more specific details. The assessment Load effects are then obtained from the following equation defined in *BD 21/01*:

$$S_A^* = \gamma_{f3}(effects\ of\ Q_A^*) \tag{4.9}$$

Where,

$$Q_A^* = \gamma_{fl} Q_k \tag{4.10}$$

4.4 Road Bridge Assessment Loads

4.4.1 Dead Load and Live Load

The assessment loads considered in both standards cover the three types of actions previously discussed in section 4.1. The permanent actions can be divided into dead loads and superimposed dead loads whilst the variable actions are considered as the live loads, as explained below:

- Dead Load: defined as the self-weight of the structural elements such as the beams, trestles, slab and so on;
- Superimposed Dead Load: defined as the weight of the non-structural elements present in the structure, such as the filling, the surface and so on.
- Live Load: defined as traffic load and the pedestrian load, when regard to bridges.

4.4.2 Notional Lanes

To describe the load models, it is first necessary to define the concept of notional lane. Notional lane is a rational way of dividing the carriageway width in equal parts, measured between the

[2] When the member is required to resist the frictional restraint caused by a temperature-induced movement.

kerbs. Thus, the calculation for the number, width and remaining area of the notional lane, according to the Eurocodes, can be found in the table below (Table 4.5):

Table 4.5 – Number and width of Notional lanes, according to the Eurocodes [26]

Carriageway width w	Number of notional lanes	Width of a notional lane w_l	Width of the remaining area
$w < 5{,}4$ m	$n_l = 1$	3 m	w - 3 m
5,4 m $\leq w < 6$ m	$n_l = 2$	$\frac{w}{2}$	0
6 m $\leq w$	$n_l = Int\left(\frac{w}{3}\right)$	3 m	$w - 3 \times n_l$
NOTE For example, for a carriageway width equal to 11m, $n_l = Int\left(\frac{w}{3}\right) = 3$, and the width of the remaining area is 11 - 3×3 = 2m.			

Table 4.6 - Number of Notional Lanes, according to BDs [29]

Carriageway Width (m)	Number of Notional Lanes
below 5.0	1
from 5.0 up to and including 7.5	2
above 7.5 up to and including 10.95	3
above 10.95 up to and including 14.6	4
above 14.6 up to and including 18.25	5
above 18.25 up to and including 21.9	6

Similarly to the Eurocodes, BDs also introduce their method of determining the characteristics of notional lane. According to the UK Standards for Structural Assessments, each lane of the carriageway must be neither less than 2.5m nor greater than 3.65m. In both standards, if there are already actual lane markings, the notional lanes should be based in that physical separation. However, in the BDs, if that notional lane exceeds 3.65m of width, its determination should be done through Table 4.6 [29].

The concept of remaining areas is not defined in BD 21, yet the idea of hard shoulder is introduced, representing practically the same thing. It means that, according to BD standards, the carriageway is divided in the intended number of notional lanes, where the hard shoulder is included in each notional lane.

For future assessments, loads must be applied throughout the length of each notional lane, in place where it produces the most adverse effect. Regarding the Eurocodes, the same applies to the remaining areas. When relevant, traffic and pedestrian loads must be combined.

4.4.3 Traffic Load Models according to the Eurocodes

Traffic loads on bridges can be classified as vertical, horizontal, static and dynamic. In this content, loads will be considered as vertical and horizontal loads.

Traffic load models on road bridges are an approach based on the various categories of vehicles. It means that the traffic loads can vary between bridges, depending, for instance, on the percentage of lorries, in the average number of vehicles per year, traffic jam frequency or even in the higher possible weight of vehicles.

Therefore, adjustment factor α, which is related with the previous concern, will be used for the accounting of the resulting traffic load models. These adjustment factors can be found in the National Annex associated with the location of the bridge. For the specific case study introduced further ahead, they can be found in Table 5.6.

Vertical loads according to the Eurocodes can be divided in four different groups:

- Load Model 1(LM1): Double axle Concentrated Load (Figure 4.3), a Tandem system representing the vehicle type, and uniformly distributed loads (UDL system);
- Load Model 2 (LM2): Single axle load applied on specific tyre contact areas (covers the dynamic effects);
- Load Model 3 (LM3): Assemblies of axle loads representing Special Vehicles;
- Load Model 4 (LM4): Crowd loading.

However, the only loads of traffic effects relevant for future developments are Load Model 1 and Load Model 3. Values of loads Q_{ik} and q_{ik} can be determined according to Table 4.7 [40].

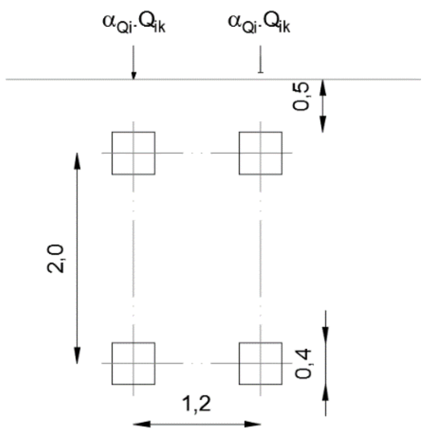


Figure 4.3 -Schematic representation of Double axle concentrated Load from Load model 1 [40]

Regarding Load model 1, it is known that:

- *no more than one tandem system should be taken into account per notional lane*;
- *For the assessment of general effects, each tandem system should be assumed to travel centrally along the axis of notional lanes;*
- *Each axle of the tandem system should be taken into account with two identical wheels, the load per wheel being therefore equal to $0{,}5\alpha_Q Q_k$.*
- *The uniformly distributed loads should be applied only in the unfavourable parts of the influence surface, longitudinally and transversally. Where two tandem systems on adjacent notional lanes are taken into account, they may be brought closer, with a distance between wheel axles not below 0,50 m* [40].

Table 4.7 – Values of Loads used in Load Model 1 [40]

Location	**Tandem system *TS***	***UDL* system**
	Axle loads Q_{ik} (kN)	AC1 q_{ik} **(or** q_{rk}) (kN/m^2) AC1
Lane Number 1	300	9
Lane Number 2	200	2,5
Lane Number 3	100	2,5
Other lanes	0	2,5
Remaining area (q_{rk})	0	2,5

Load Model 3 represents a most complex model since it portrays a few models of special vehicles with a convoy of axles that can be found in the respective National Annex of each country. Those models will be introduced further ahead.

4.4.4 Traffic Load Models according to BDs

The assessment loading defined in the UK standards can also be divided into main categories of vehicles. The first one is characterized as normal vehicles and the other one as abnormal vehicles:

Normal Vehicles: This type of vehicles is under the Road Vehicles Authorized Weight (AW) Regulations 1998 which cover regular vehicles up to 40/44 tonnes gross weight.

Abnormal vehicles: This type of vehicles is not covered by the AW Regulations 1998 since the weight of these vehicles exceed the limit established by those regulations.

Representing the first category, normal vehicles or normal traffic is introduced. According to *BD 21/01,* HA live load type consists in the combination of a uniformly distributed load (UDL) and a knife edge load (KEL). Those loads must be applied to each notional lane in the appropriate influence line for the structural element under consideration. The KEL should be applied at one point only in the loaded length of each notional lane. Loaded length varies according to the length of the span, it means that if it is a simply supported beam, the load must be applied over the full length. Otherwise, if the bridge has more than one span, the load should be applied over the places that will cause the worst effect possible [29].

Influence lines are diagrams that express the value that a certain internal force holds when a certain load is applied to the structure, as a function of the magnitude and the location of this load. Considering that the internal forces (moments, support reactions, shear force, stresses etc.) are a linear function of the load, the ordinate of the influence line of a given point is the value of the internal force when a unit load is placed at this point.

4.4.4.1 Normal vehicles

1) **Type HA UDL and KEL**

This kind of load is determined based on the following equation:

$$W = 336(1/L)^{0,67} \tag{4.11}$$

Where W represents the UDL in kN per metre of lane and L the respective loaded length in metres. KEL is always considered as a knife edge of 120 kN. The respective graphic of W (UDL) can be found in *BD 21/01, Figure 5.1* [29].

2) **Reduction factors for HA UDL and KEL**

There are three reduction factors required for the HA loading. K is defined as the ratio between the Assessment live loading and the HA loading, AF (adjustment factor) is a factor that intends to eliminate the lateral bunching factor. The Lane factor is used to reduce the probability of the HA loading occurring at the same time in adjacent lanes.

The HA loading was obtained using a deterministic approach that led to six categories of bridge, related to road surface characteristics and daily traffic flow (both directions). Therefore, K is a value used in order to consider those categories in the determination of the HA loading.

Traffic flow can be separated in different groups as well as the road surface. The traffic flow depends on the Annual Average Hourly HGV flow and it can be defined as:

- High (H): 70 < AAHHGVF
- Medium (M): 7 < AAHHGVF <70
- Low (L): AAHHGVF < 7

For the road surface, two groups can be distinguished:

- “Good”: Roads that are in a good state, showing no visible deterioration;
- “Poor”: Roads that shows visible deterioration.

Combined together, the six categories can be cited as Hg, Mg, Hp, Lg, Mp and Lp. Hence, it is possible to determine the factor K based on the loaded length, the required weight for the structure to carry and those categories determined above, through existent charts, available in *BD 21/01, Figures 5.2 to 5.7*. The one used for the specific case study is shown in Figure 5.12 [29].

The HA loading has been determined using a lateral bunching factor that, *in slow moving situations, more lanes of traffic than the marked or notional lanes could use the bridge.* (BD 21/01, 2001, p.5). However, some researches have concluded that the most onerous effect occurs for the high-speed impact but with no lateral bunching. So, AF was created in order to reduce those effects, and it can be translated as [29]:

$$AF = a_L/2{,}5 \tag{4.12}$$

For spans between 0 and 20 m, $a_L = 3{,}65$

The Lane factors shall be as follows [29]:

Lane 1	1,0
Lane 2	1,0
Lane 3	0,5
Lane 4 and subsequents	0,4

4.4.4.2 Abnormal vehicles

There are two types of abnormal vehicles covered by the UK Standards for Structural Assessment of highway bridges and structures. They are the Special Types General Order (STGO) vehicles and Special Order (SO) vehicles, introduced in BD 86/11 where requirements and guidance are given for the load carrying capacity of structures to support the effects of those type of vehicles

However, STGO vehicles will only have a brief mention as these are included in BDs that do not follow the AW Regulations for reasons of gross weight, height, length, axle weight and space configurations. Standard BD 86/11 provides an analysis of the effects of this type of vehicles using SV load models, allowing more accurate results than the ones obtained from the HB type introduced in BD 37/01. According to BD 37/01 of the Design Manual for Roads and Bridges (DMRB), HB loading is a type of load that derives from the nature of exceptional industrial loads such as electrical transformers, generators pressure vessels and others [39].

1) <u>HB loading</u>

HB loading represents a type of vehicle constituted by four axles, each one with four wheels that have the same space between them. The load on each axle is defined by the number of units which will depend on the class of the road that controls the allowed weight. For instance, motorways and trunk roads require for up to 45 units of carrying capacity, principal roads until 37,5 units and public roads a minimum of 30 units. Where one unit of HB corresponds to 10 kN per axle. This type of loading has recently been superseded for new bridges and existing bridges that have not yet been assessed. In the end, there are five types of HB loading to check, with an exception for the bridges that have been inspected, assessed for this type of load and compliant with it. An example of an HB type is shown in Figure 4.4 [39].

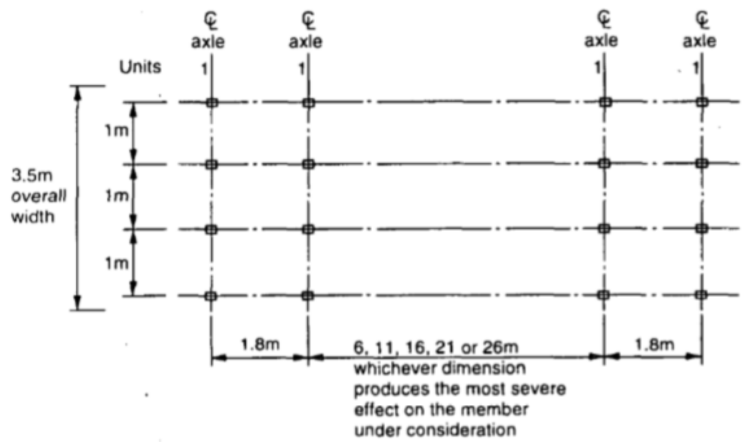


Figure 4.4 – Schematic representation of HB vehicle loading [39]

2) <u>SGTO vehicles</u>

As implied previously, STGO vehicles, represented by an SV model, came to improve HB loading qualities such as the increase of the capacity of lower classified bridges, especially the ones with a loaded length less than 10 metres. As well as the improvement of safety for highway bridges and structures of different spans.

Thus, it is intended that the assessment for HB loading stops being carried out and give place to SV load models. For some continued operation of load management systems its use is still permitted as an addition to SV load models, with the agreement of the Overseeing Organization. It is expected that the use of HB classification falls in disuse.

Therefore, BD 86/11 presents five load models that simulate the most onerous effects that can be obtained from STGO vehicles, although they do not represent real vehicles. The basic axles from these models do not exceed 16,5 tonnes and the axles from military tank transporter vehicles do not exceed 25 tonnes. The axle weight and spacing from these models are then an approximation to the allowable limits stablished by the STGO Regulations, but not equal to it.

An example of an SV model is the SV 80 represented in the Figure 4.5, which is intended to model the effects of STGO vehicles from category 2, with a maximum gross vehicle weight of 80 tonnes and a maximum axle load of 12,5 tonnes [41].

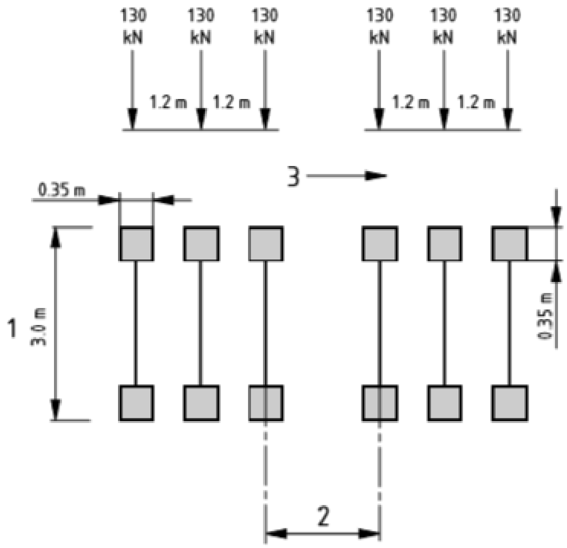


Figure 4.5 - SV 80 load model [41]

Based on the same method, STGO Regulations also define models for[3]:

- SV 100: STGO category 3 vehicles with a maximum weight of 100 tonnes and a maximum basic axle load of 16,5 tonnes;
- SV 150: STGO category 3 vehicles with a maximum weight of 150 tonnes and a maximum basic axle load of 16,5 tonnes;
- SV-train: Single locomotive pulling a category 3 trailer;
- SV-TT: Military tank transporter vehicles with a maximum basic axle load of 25 tonnes.

In section 4.4.3, load model 3 was mentioned regarding traffic loads from the Eurocodes. This type of loads is related to special vehicles, which are mainly used as industrial transports and are defined as a set of assemblies of axle loads. Thus, similarly to UK Standards for Structural Assessments, they are also treated as abnormal vehicles and can be consulted with the help of the National Annex of each country [41].

In the UK, both standards show the same load models for those special vehicles referred in the Eurocodes and BDs, except for the model SV 150, SV-train and SV-TT, which only exist in BDs and the model SV 196 that only exists in the Eurocodes.

Despite of the differences regarding load distribution, load model SV 196 (Figure 4.6), introduced in the National Annex of *EN 1991-2*, has a load value similar to HB 45 which can still be used for structural assessment, according to *BD 37/01* [42]. Therefore, these two models will be compared further ahead regarding load capacity and effects that produce in the structural members.

[3] These load models can be found in Annex A.

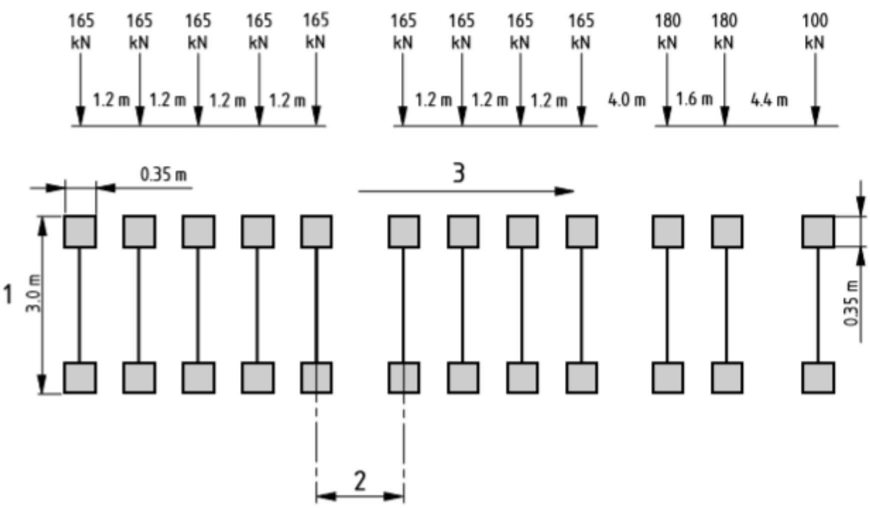


Figure 4.6 - SV 196 load model [41]

4.4.5 Braking Loads

According to both standards, a braking load is a longitudinal force resulted from a traction, acting at the surfacing level of the carriageway. It should be applied only in one of the notional lanes[4] and parallel to it. The value of this load (Q_k) is limited to 900 kN according to the Eurocodes, and restricted to 750 kN according to BDs.

Eurocodes define that this load should be considered as a 60% of the total vertical loads from the vehicle-type and 10% from the uniformly distributed loads. Both loads from load model 1 are likely to be applied on lane number 1, translated as follows [40]:

$$0{,}6\alpha_{Q1}(Q_k) + 0{,}1\alpha_{q1}q_{1k}w_l L \tag{4.13}$$

Whereas in BDs it should be the nominal load for type HA explained in the following way [39]:

$$8 \times L + 250 \tag{4.14}$$

If it is intended to obtain the braking loads caused by accidental loads, they usually correspond to 60% of the vertical loads, according to BD standards.

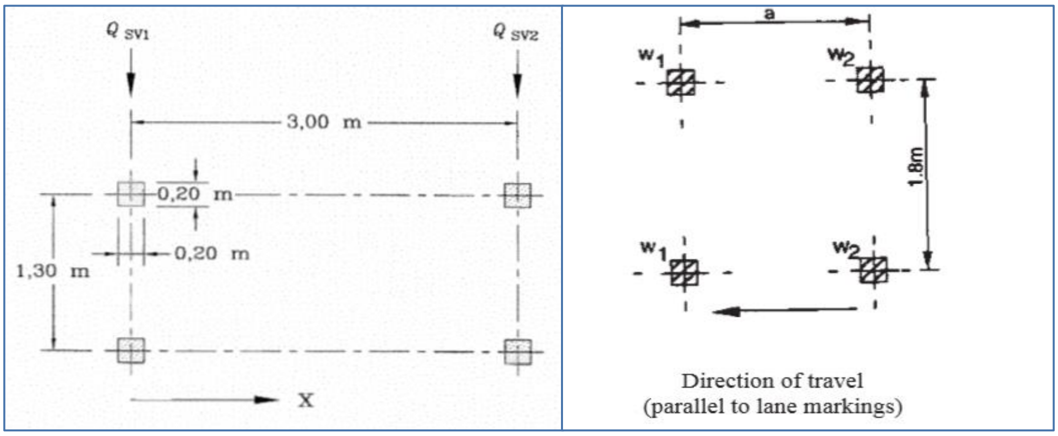
4.4.6 Accidental Loads

An accidental load can be defined as a vehicle collision against the columns of the bridge, the presence of wheels or vehicle on footways when they are not protected by any type of restraint, or even a vehicle collision against the parapets or safety barriers. Throughout this work a

[4] However, in the Eurocodes the lanes have a proper notation, which is not the case in BDs.

description of the accidental loads due to the incident of vehicle wheels on the footways will be carried out when there is no permanent obstacle preventing it to happen [29].

Therefore, the two models that should be used in the Eurocodes and in the UK standards for structural assessment are shown in the figure below (Figure 4.7). Regarding Figure 4.7 (a), the values Q_{SV1} and Q_{SV2} are considered as being axles of 80kN and 40 kN, respectively, separated by a wheel base of 3m with a track (wheel-centre to wheel-centre) of 1,3m and square contact areas of 0,2m sided [40]. In terms of w_1 and w_2, they depend on the assessment live loading, as well as the parameter "a", so these values are tabled according to it, possible to determine through Table 4.8. The contact areas shall be uniformly distributed in a circular or square geometry and assuming an actual pressure of 1,1 N/mm^2 [29].



a) b)

Figure 4.7 - Representation of accidental load models according to the Eurocodes (a) [26] and BDs (b) [29]

Table 4.8 – Assessment Live loading for accidental actions, according to BDs [29]

Assessment Live Loading	W_1 (kN)	W_2 (kN)	a (m)
40 tonnes	100	60	1.5
26 tonnes	100	40	1.5
18 tonnes	100	10	1.5
7.5 tonnes	50	10	1.5
3 tonnes	25	-	-
FE Group One	60	10	1.5
FE Group Two	30	20	1.5

4.4.7 Footway Loads

For road bridges supporting footways, a uniformly distributed live load, representing the pedestrian load, should be applied in the unfavourable parts of the influence surface, assuming in both standards, the recommended value of 5 kN/m^2.

In the Eurocodes, beyond the pedestrian load q_{fk}, it is also required the application of two more loads, being them a concentrated load Q_{fwk} and loads representing service vehicles Q_{serv}. The concentrated load Q_{fwk} should be taken into account for local effects and equal to 10 kN with an application surface of a square with 0,10 m side. Regarding the load for service vehicles Q_{fwk}, they are already included in the assessment for accidental loads on footways. However, if permanent provisions have been made to prevent the access from this type of vehicles, it is not necessary to consider them for the structural assessment [40].

4.4.8 Summary

For a better comparison, Table 4.9 summarize the actions mentioned above and shows the clauses where they can be consulted in both standards.

Table 4.9 - Summary of actions and respective clauses in both standards

	Eurocodes	**BDs**
LM1	*EN 1991-2, clause 4.3.2*	______
HA UDL and KEL	______	*BD 21/01, Clause 5.8*
LM 3/SV vehicles	*EN 1991-2, clause 4.3.4*	*BD 86/11, clause 3.10*
Braking Load	*EN 1991-2, clause 4.4.1*	*BD 37/01, clause 6.10*
Accidental Loads	*EN 1991-2, clause 5.6.3*	*BD 21/01, Clause 5.34*
Footway Loads	*EN 1991-2, clause 5.3.1 (2)*	*BD 21/01, Clause 5.35*

4.5 Resistance of Elements of a Composite Bridge

4.5.1 Resistance of box girders

4.5.1.1 Classification of Composite Box Girders

In order to meet the requirements stipulated, both in the *EN 1990-2* and *BD 21/01*, regarding structural resistance, serviceability and durability, the structural elements should satisfy some principles related to internal failure or excessive deformation.

A composite beam, when loaded, is mainly subjected to bending, being its structural analysis done mostly in this direction.

The first step to carry out in a structural assessment of a composite box girder is to identify the class of the cross-section in order to understand the likelihood of any element within the cross-section to reach its local buckling resistance before its yield stress. Limiting thus the axial load capacity and bending resistance of the section.

According to EN 1993-1, a steel cross-section can be separated in four classes, depending on its capacity of resisting a compression stress. Classes are then defined in terms of the requirements for bending resistance represented in Figure 4.8 and explained as follows:

- Class 1: These cross-sections have the capacity to develop the plastic moment capacity, with the respective rotation capacity required to achieve a plastic hinge;
- Class 2: Cross-sections that can also reach the plastic moment resistance in all over the section but have limited rotation capacity due to local buckling;
- Class 3: Sections with the capacity of achieving the yield stress resistance in the extreme compression fibre, forming an elastic distribution of stresses;
- Class 4: These cross-sections cannot even achieve the yield stress due to the occurrence of local buckling. These sections shall be reduced to an effective area and be then considered as class 3.

Hence, the classes of a steel cross-section depend on its width-to-thickness ratio and can be determinate using the *table 5.2* from *EN 1993-1*. *EN 1993-1* says that it is necessary to determine the class of all the members of a cross-section and classify it with the least favourable class of all the members in compression.

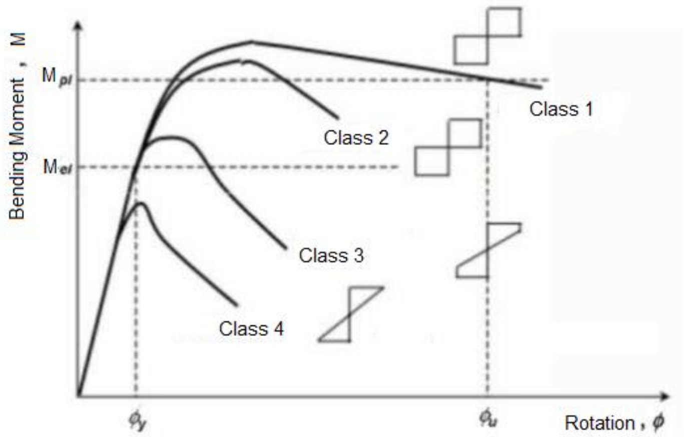


Figure 4.8 – Diagram of classes of steel sections

Nevertheless, if a cross-section has a class 3 web and class 1 or 2 flanges, the cross-section in general may be considered as class 2, if the effective web is in accordance with *6.2.2.4, EN 1993-1*. This section says that the web in compression should be substituted by a part of $20\varepsilon t_w$ next to to the compression flange and a part adjacent to the neutral axis of the section [33].

It is known that the girders under consideration are steel-concrete composite, whereby *EN 1994-2* must also be taken into consideration. This aligns with the classification system defined in the Eurocode 3. Although it is known that a steel compression element restrained by a reinforced concrete element should be considered in a more favourable class, if an improvement in the section has been made [43].

In BD standards, there is also a distinction made with regard the capacity of the cross-section to resist the plastic moment. Although in this case, the classification is only divided in two: a compact section or a non-compact section [44].

- Compact Section: The plastic moment can be developed and maintained beyond yielding.
- Non-compact Section: The cross-section cannot achieve the plastic moment resistance, being subject to buckling beforehand.

Thereafter, a similar process, is made to determine if the cross-section is compact, as represented below.

The depth of the web shall not be greater than:

$$d_w < \frac{34t_w}{m}\sqrt{\frac{355}{\sigma_{yw}}}, when\ m\ does\ not\ exceed\ 0{,}5 \quad (4.15)$$

$$d_w < \frac{374t_w}{(13m-1)}\sqrt{\frac{355}{\sigma_{yw}}}, when\ m\ exceeds\ 0{,}5 \quad (4.16)$$

The clear width of the compression flange shall not be greater than:

$$w_f < 24t_f\sqrt{\frac{355}{\sigma_{yw}}} \quad (4.17)$$

In conclusion, it is possible to make a comparison between both standards. Classes 1 and 2 define sections that are able to mobilise plastic moment, representing compact sections in BD standards, while class 3 and 4, cannot mobilise plastic moment and elastic moment, respectively, being comparable to non-compact sections in BDs, as exposed in the Table 4.10 [44].

Table 4.10 – Classification of steel cross-sections

EC	BDs
Class 1	Compact
Class 2	
Class 3	Non-Compact
Class 4	

4.5.1.2 Shear Connectors

Shear connectors are typically used in composite sections to connect the concrete to steel members in order to prevent the occurrence of slip between them and accomplish a much stiffener and stronger beam. In the case introduced further ahead, the connection of the concrete slab to the supporting steel girders is made by transferring longitudinal shear force between the structural members using stud shear connectors, one of the most popular types of connecting device, used in composite construction. The strength of the composite girders is significantly influenced by the strength and service capacity of the shear connectors which is defined after the study of their behaviour through push-out tests. They are carried out specifically to know their strength, the ability to resist longitudinal forces, the natural consequence that occurs for a composite action, and the degree of slip that arises at the interface of the steel and concrete.

The connector is considered to be sufficiently strong and stiff, if the separation measured in those push-out tests does not exceed half of the longitudinal slip at the corresponding load level. In the case of British Standards, it is only considered load cases up to 80% of the nominal static strength of the connector. The study of the behaviour of the embedded stud shear connector is extremely important due to the inelastic deformations under the combined effects of shear, bending and torsion, when the headed stud approaches failure. It will also affect the concrete surrounding the headed stud shear connector which is subjected to cracks due to high splitting forces caused by that. It will probably lead to non-ductile failure [45].

Various studies suggest an upper limit equation on headed stud shear connector strength of $0{,}8A_sF_u$, where F_u is the tensile strength of the stud. According to *EN 1994-2*, to prevent the separation of the slab, the shear connectors are required to resist in tension at least 10% of shear resistance [43]. The most types of shear connection used in composite sections are the 19 mm headed stud shear connectors. However, in practice, the scale of shear flows varies along the beam which requests different resistance from the shear connector, despite of being much more easy and economical to keep a uniform resistance over the whole girder [45]. The shear connection is usually provided in rows with a spacing that varies consonant the needs required. For instance, they are normally less spaced near the supports since it is where shear is highest. They should be provided in an inelastic length L_{AB} to resist the longitudinal force V_{LEd} [43]. The rules regarding longitudinal and transverse spacing, associated with shear connectors, are stablished in the Eurocodes and BDs standards as discriminated in Table 4.11:

Table 4.11 - Allowable longitudinal and transverse space of shear connectors

	EC	BDs
Max Longitudinal Spacing	$22t_f\sqrt{235/f_y}$	$22t_f\sqrt{355/f_y}$
Max Transverse Spacing	$min(4 \times t_{slab}; 800)$	$30t_f\sqrt{355/f_y}$
Clauses	*EN 1994-2, cl. 6.6.5.5*	*BS 5400-3, cl. 9.3.7.3*

4.5.1.3 Bending Resistance and Lateral Torsional Buckling

When talking about composite bridges, it is assumed that there is a perfect bond between materials, creating a full interaction between the structural steel, reinforcement and concrete. Thus, for verification of bending resistance of this type of sections, it is important to highlight the fact that both structural steel and reinforcement, are considered in tension resistance while concrete is considered only for compression.

Within composite sections it is then needed to transform the concrete material in steel material through the modular ratio E_s/E_c in order to compute the transformed plastic neutral axis for the sections resisting sagging moments. Regarding the sections resisting hogging moments, neutral axis has been determined based on the combined section of concrete slab and steel girder. Afterwards, it is considered the effective area of the concrete in compression resisting to a stress of $0{,}85f_{cd}$, as estipulated in *EN 1994-2*.

Thus, the following equation 4.18, according to *EN 1993-1* and *BS 5400-3*, must be checked:

$$\frac{M_{Ed}}{M_{c,Rd}\,or\;M_D} \leq 1{,}0 \tag{4.18}$$

The Moments of Resistance are defined based on the classification of the composite cross-sections and according to each standard, introduced on the Table 4.12:

Table 4.12 - Moment of Resistance defined by each standard

EN 1993-1, cl. 6.2.5		BS 5400-3, cl. 9.9.1
Class 1 and 2	Class 3	Compact and Non-compact sections
$M_{c,Rd} = \frac{W_{Pl}f_y}{\gamma_{M0}}$	$M_{c,Rd} = \frac{W_{el,min}f_y}{\gamma_{M0}}$	$M_D = \frac{M_R}{\gamma_m\gamma_{f3}}$

M_R is defined as the limiting moment of resistance which depends on the resistance of the beam to prevent lateral-torsional buckling, and can be obtained from Figure 4.9.

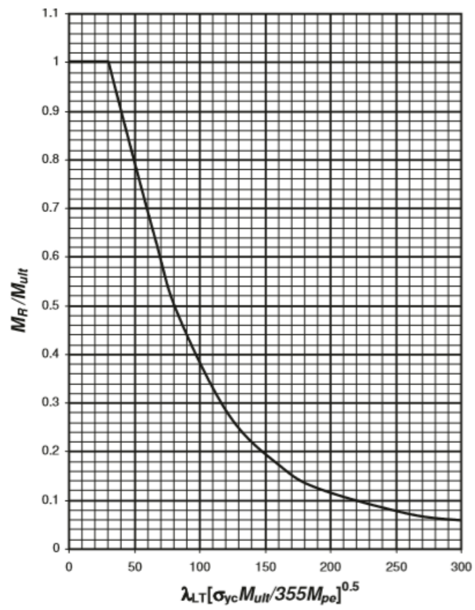


Figure 4.9 – Graphical Illustration of the limiting moment of resistance [34]

If there is no possibility of lateral torsional buckling, then the reduction factor will be 1,0 and so $M_r = M_{ult}$. This means that the beam will not fail due to lateral buckling but due to the fact of the cross section reach the bending resistance in the case of compact sections, or elastic resistance for non-compact sections [34].

In the case of Eurocodes, when a laterally unrestrained member is under consideration, the resistance of the beams to lateral torsional buckling should be also verified apart from the Moment of Resistance. So, equation 4.19 shall be satisfied [33]:

$$\frac{M_{Ed}}{M_{b,Rd}} \leq 1{,}0 \tag{4.19}$$

Where,

$$M_{b,Rd} = \chi_{LT} W_y \frac{f_y}{\gamma_{M1}} \tag{4.20}$$

4.5.1.4 Resistance to Pure Shear

Equivalent to Bending Resistance, shear force must also verify the following equation:

$$\frac{V_{Ed}}{V_{c,Rd}\ or\ V_D} \leq 1{,}0 \tag{4.21}$$

According to *EN 1993-1* and *BS 5400-3*, respectively, the equations in

Table 4.13 should be applied in order to verify the resistance of the box girder under pure shear, either through elastic analysis or plastic analysis:

Table 4.13 - Equations used to verify the capacity of a girder under pure shear

EN 1993-1, cl. 6.2.6		BS 5400-3, cl. 9.9.2.2
Class 1 and 2	Class 3	Compact and Non-compact sections
$\boldsymbol{V_{pl,Rd} = \frac{A_v(f_y/\sqrt{3})}{\gamma_M}}$	$\frac{\tau_{Ed}}{f_y/(\sqrt{3\gamma_{M0}})} \leq 1{,}0$	$V_D = \left[\frac{t_w(d_w - h_h)}{\gamma_m \gamma_{f3}}\right]\tau_l$

When considering composite cross-sections, it is known that the resistance to pure shear should be taken as the resistance of the structural steel section, unless the reinforcement of the concrete is taken into consideration. However, for a more conservative process that part is usually negligible. With regards to pure shear resistance according to *BS 5400-3* (Table 4.13), the equation (V_D) already takes account of the possibility of the failing of the structural member due to shear buckling of the webs. Since in the Eurocodes the same is not applied, the following equation must be checked in order to dismiss that assumption [33].

$$\frac{h}{t_w} \leq \frac{72}{\eta}\varepsilon \tag{4.22}$$

For unstiffened webs, and

$$\frac{h}{t_w} \leq \frac{31}{\eta}\varepsilon\sqrt{k_\tau} \tag{4.23}$$

For stiffened webs.

4.5.1.5 Combined Bending and Shear

When shear is present, its interaction with bending moment should be analysed. However, this combined effect may be neglected if the shear force is less than half of the plastic shear resistance according to *EN 1993-1*.

Regarding *BS 5400-3*, verification to shear effect on moment resistance for sections with intermediate stiffeners should only be done if $M > M_f$ where,

$$M_f = \frac{F_f d_f}{\gamma_m \gamma_{f3}} \tag{4.24}$$

$F_f = \sigma_f A_{fe}$ and d_f is the distance between the centroids of the two flanges. M_f should not be greater than M_D.

In addition, $V > V_R$ must also be confirmed, where $V_R = V_D$, if $m_{fw} = 0$. This last equation means that there is no contribution to tension field action from the flanges.

Otherwise, if the combined effect of shear force and bending moment must be definitely taken into account, the following equations should be verified according to *BS 5400-3, clause 9.9.3.1* [44]*:*

$$If\ M > M_f, then\ \frac{M}{M_D} + \left(1 - \frac{M_f}{M_D}\right)\left(\frac{2V}{V_R} - 1\right) \leq 1{,}0 \quad (4.25)$$

$$If\ V > V_R, then\ \frac{V}{V_D} + \left(1 - \frac{V_R}{V_D}\right)\left(\frac{2M}{M_f} - 1\right) \leq 1{,}0 \quad (4.26)$$

From *EN 1993-1, clause 6.2.8*, it is obtained the reduced yield strength $(1-\rho)f_y$ which should be multiplied by the moment resistance, where [33]:

$$\rho = \left(\frac{2V_{Ed}}{V_{pl,,Rd}} - 1\right)^2 \quad (4.27)$$

4.5.1.6 Torsion

Although it can be verified a much better torsional resistance in the box girders than it is observed in an open section, for example an I beam, a member subjected to torsion should verify the following equation:

$$\frac{T_{Ed}}{T_{Rd}} \leq 1{,}0 \quad (4.28)$$

Torsion is normally represented as a pair of forces, being resisted in a box section by a shear flow around the whole perimeter. Then, it can be divided in two parts, the internal St. Venant torsion moment and the internal warping torsional moment. Knowing that for hollow sections the effects of torsional wrapping can be neglected, it should only be taken into account the shear stress related to St. Venant torsion [33].

As it is shown on Figure 4.10, each force can finally be obtained as $q = T/x$, where

$$x = B - 2t_w \quad (4.29)$$

In the end of the process of checking box girder against combined shear force and torsional moment effects, the following equations must be met, according to *EN 1993-1, clause 6.2.7*:

$$\frac{V_{Ed}}{V_{pl,T,Rd}} \leq 1{,}0 \quad (4.30)$$

Where,

$$V_{pl,T,Rd} = \left[1 - \frac{\tau_{t,Ed}}{\left(\frac{f_y}{\sqrt{3}}\right)/\gamma_{M0}}\right] V_{pl,Rd} \qquad (4.31)$$

According to *BS 5400-3* and considering Figure 4.10, the following must be checked:

$$\frac{T}{x} + \frac{V}{2} \leq \frac{V_D}{2} \qquad (4.32)$$

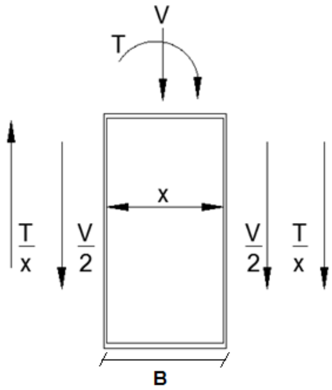


Figure 4.10 - Schematic representation of torsion moment applied on a box girder

4.5.2 Resistance of steel columns

4.5.2.1 Resistance to axial compression and Buckling

A steel column is mainly subjected to axial force which can lead to buckling, being them the main subject of concern, not forgetting although, the interaction between bending and compression. An element under axial compression should be such that its axial load is less than the following equations within Table 4.14.

Table 4.14 – Equations related to Compression Resistance

EN 1993-1, cl. 6.2.4	BS 5400-3, cl. 10.6.1
Class 1, 2 and 3	Compact and Non-compact sections
$N_{c,Rd} = \frac{Af_y}{\gamma_{M0}}$	$P_D = \frac{A_e \sigma_c}{\gamma_m \gamma_{f3}}$

σ_c is the less compressive stress value for buckling about any axis, to be obtained from Figure 4.11 [44].

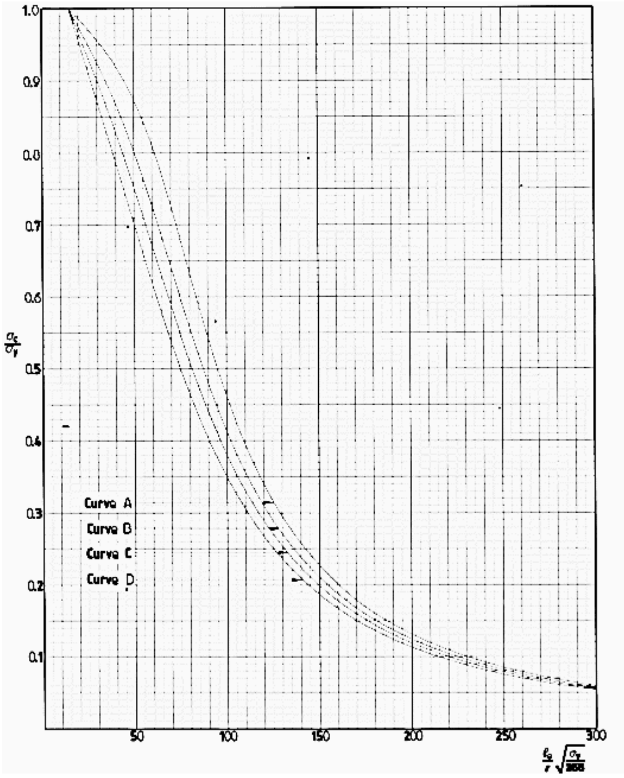


Figure 4.11 - Ultimate compressive stress σ_c [44]

Regarding EN 1993-1, the verification against flexural buckling resistance is calculated in order to check the following equation:

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1{,}0 \tag{4.33}$$

Where,

$$N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}}, for\ classes\ 1, 2\ and\ 3 \tag{4.34}$$

The slenderness of the column is obtained through equations 4.35 and 4.36 according to Eurocodes and BDs, respectively. When considering a column hold in position at both ends and restrained in direction at one end, Eurocodes consider an effective length of $0{,}7l_e$ whilst BDs consider an effective length of $0{,}85l_e$.

$$\lambda = \frac{l_e}{i} \tag{4.35}$$

$$\lambda = \frac{l_e}{r}\sqrt{\frac{\sigma_y}{355}} \tag{4.36}$$

4.5.2.2 Combined Compression and Bending

When subjected to coexistent compression and bending, a member should satisfy the subsequent procedures. According to EN 1993-1, class 1 and 2 cross-sections, shall satisfy the following equation:

$$M_{Ed} \leq M_{N,Rd} \tag{4.37}$$

Where,

$$M_{N,Rd} = M_{pl,Rd}(1-n)/(1-0{,}5a_w) \tag{4.38}$$

For the minor axis, and,

$$M_{N,Rd} = M_{pl,Rd}\left(1-\left(\frac{n-a}{1-a}\right)^2\right) \tag{4.39}$$

For the major axis.

Where $n = N_{Ed}/N_{pl,Rd}$ and $a = (A - 2bt_f)/A \leq 0{,}5$. From these equations is then possible to take the capacity of the cross-sections against the interaction between axial force and bending moment (N-M) [33].

When the resistance of a cross-section to bending moment must increase due to a presence of an axial force, the following equation, more conservative, shall be satisfied, according to *BS 5400-3*, *clause 10.6.2.1*:

$$\frac{P_{max}}{P_D} + \frac{M_{x,max}}{M_{Dxc}} + \frac{M_{y,max}}{M_{Dyc}} \leq 1{,}0 \tag{4.40}$$

4.5.3 Resistance of slab concrete

4.5.3.1 Bending Resistance

A reinforced concrete slab supported on steel girders in a composite bridge can be analysed as a line beam in the transverse direction where, according to *BD 44/15*, the ultimate moment resistance in sections without compression reinforcement can be obtained from the lesser of the following equations:

$$M_{u,steel} = (f_y/\gamma_{ms})A_s z \tag{4.41}$$

$$M_{u,con} = (0{,}225 f_{cu}/\gamma_{mc})bd^2 \tag{4.42}$$

The ultimate resistant moment according to *EN 1992-2*, can be obtained through the equation below:

$$M_{Rd} = \mu b d^2 f_{cd} \tag{4.43}$$

Considering *BD 44/15*, z is assumed to be the lever arm, never greater than $0{,}95d$, determined through the following equation:

$$z = \left[1 - \frac{0{,}84(f_y/\gamma_{ms})A_s}{(f_{cu}/\gamma_{mc})bd}\right]d \tag{4.44}$$

In *EN 1992-2*, μ is the reduced moment, taken as:

$$\mu = \omega(1 - 0{,}514\omega) \tag{4.45}$$

Where,

$$\omega = A_s f_{yd}/bdf_{cd} \tag{4.46}$$

4.5.3.2 Shear Resistance

The process of verifying a slab concrete without shear reinforcement differ in a few things, in the standards under consideration. The equations associated with it according to *EN 1992-2* are the following:

$$V_{Rd,c} = \left[C_{Rd,c}k(100\rho_l f_{ck})^{1/3} + k_1\sigma_{cp}\right]b_w d \tag{4.47}$$

Where,

$$C_{Rd,c} = 0{,}18/\gamma_c \tag{4.48}$$

$$k = 1 + \sqrt{\frac{200}{d}} \leq 2{,}0 \tag{4.49}$$

$$\rho_l = \frac{A_{sl}}{b_w} \tag{4.50}$$

$$k_1 = 0{,}15 \tag{4.51}$$

$$\sigma_{cp} = \frac{N_{Ed}}{A_c} < 0{,}2f_{cd} \tag{4.52}$$

The way to obtain the ultimate shear resistance in *BD 44/15* is through the equations below:

$$V_u = \xi_s v_c b_w d \tag{4.53}$$

Where,

$$\xi_s = \left(\frac{500}{d}\right)^{0,25} \geq 0,7 \qquad (4.54)$$

$$\nu_c = \frac{0,24}{\gamma_{mv}}\left(\frac{100A_s}{b_w d}\right)^{1/3} (f_{cu})^{1/3} \qquad (4.55)$$

Chapter 5

5. Case study of structural assessment – Ashworth Road Viaduct

Within the scope of the ongoing Kirklees Council Bridge Assessment programme 2016/2017, it was proposed to Mouchel Consulting to determine the load carrying capacity of Aswhorth Viaduct, in accordance with the Addendum nº 1 to the AIP, dated April 2017 (Figure 5.1).

Figure 5.1 – West Elevation of Ashworth Road Viaduct

5.1 Location of Ashworth Road Viaduct

The Ashworth Road Viaduct is a composite curved in plan bridge, carrying the unclassified Ashworth Road over the A638 Dewsbury Ring Road. It is located in the outskirts of Dewsbury centre Town, West Yorkshire, UK, as indicated in Figure 5.2 with a red circle. The UK National Grid Reference is 424266 East, 421652 North.

Figure 5.2 – Location of Ashworth Road Viaduct

5.2 Historical Information

The construction of this Viaduct was concluded in the early 1970s and it was designed for a maximum speed of 50 km/h. This bridge is owned and maintained by Kirklees Council Highways Service, having records that say it has passed through a few works and inspections, such as:

- Concrete test in 2000;
- A Principal Inspection in August 2001, inspected by Roger Lant and prepared by Carl Bro, Intelligent Solutions Company;
- A Feasibility Report, in July 2006;
- A Report of bat survey in August 2006
- A full refurbishment scheme in September 2007 recommended by the previous inspection in order to repair some of the defects identified, which included some improvements in the strengthening of the bridge such as an upgrade to the existing steel vehicle restraint parapet. The bridge was then assessed to have capacity of undertake 40T and 18 units of HB Loading.

5.3 Description of Ashworth Road Viaduct

The viaduct is a single carriageway that consists of five spans comprising five curved steel box girders composite with an insitu reinforced deck slab, as it can be seen in Figure 5.3. The superstructure is continuous over four intermediate steel hollow section columns and a transverse hollow section girder, having both end spans supported in two masonry mass concrete abutments. The steel box girders are supported on pinned mechanical bearings at the columns (Figure 5.4) and east abutment, with roller bearings at north abutment.

Figure 5.3 - Steel Box Girders and Reinforced Concrete Slab

Figure 5.4 - Pinned Mechanical Bearing at Trestle

The abutments, despite of being constructed of a mass concrete with integral reinforced concrete ballast walls, are faced in coarse masonry. The wingwalls to the east abutment are in line mass concrete walls with a stone facing, opposing the north abutment that has no wingwalls.

The trestles are portal frames comprising two square sections uprights and a rectangular section crosshead. The girders are also laterally restricted by a beam (trimmer) at the trestles zone, as it can be seen in Figure 5.5 a). Lastly, the base of each column incorporates a pin joint with a rubber bearing and central dowel (Figure 5.5 b) and it is simply supported on the reinforced concrete foundation.

Figure 5.5 – Example of one of the trestles (a) and support of the column (b)

5.4 Introduction to the Structural Assessment

As previously mentioned, based on the condition of the bridge recorded in the previous principal inspections of 2008 and 2016, Mouchel was commissioned to assess Ashowrth Viaduct in order to make future improvements in its strength if necessary. Therefore, the whole study of its structural elements is needed for eventual upcoming changes regarding higher loads to be carried.

In the scope of the subject of this dissertation, the assessment of the viaduct will be calculated following the Eurocodes and the BDs, already debated in the previous chapter and discussed afterwards.

This viaduct has four curved spans in plan, being the fifth and last one, straight, as it can be consulted on drawing BB.1, Annex B. Each span has an approximate length of 16m in plan which makes the viaduct length approximately 80 m long. From the transverse point a view, the viaduct presents 11 m width, including the verges, footways and parapets (drawing BB.4, Annex B).

Regarding the structural part, this bridge can be divided in three main structural elements, the steel box girders, the trestles and the slab. Not forgetting the others such as the trimmer, the parapets and the bearings.

a) Girders

The Girders are made of steel box sections and since it is a curved bridge, the length of each girder increases with the distance to the inner side of the bridge. As the length varies, so does the cross-section of each one, not only in height but also in the thickness of the flanges and webs.

In addition, each girder has also different sections, depending on where they are. For instance, if the girder is in an inner span, the section is different from other girder situated in an outer span, or even if it is in the centre of the span or right above the columns. For a better understanding, there is a drawing below representing, by colours, the different existent cross-sections in Girder 1, together with the respective notation of the Spans, Columns and Girders. Altogether, it can be counted, 20 different cross-sections in all the girders of the bridge (Figure 5.6). The girders are enumerated as G1 to G5, standing for girder 1 to girder 5. There are also tables B1 to B4, with the respective dimensions of each cross-section that can be found in Appendix B.

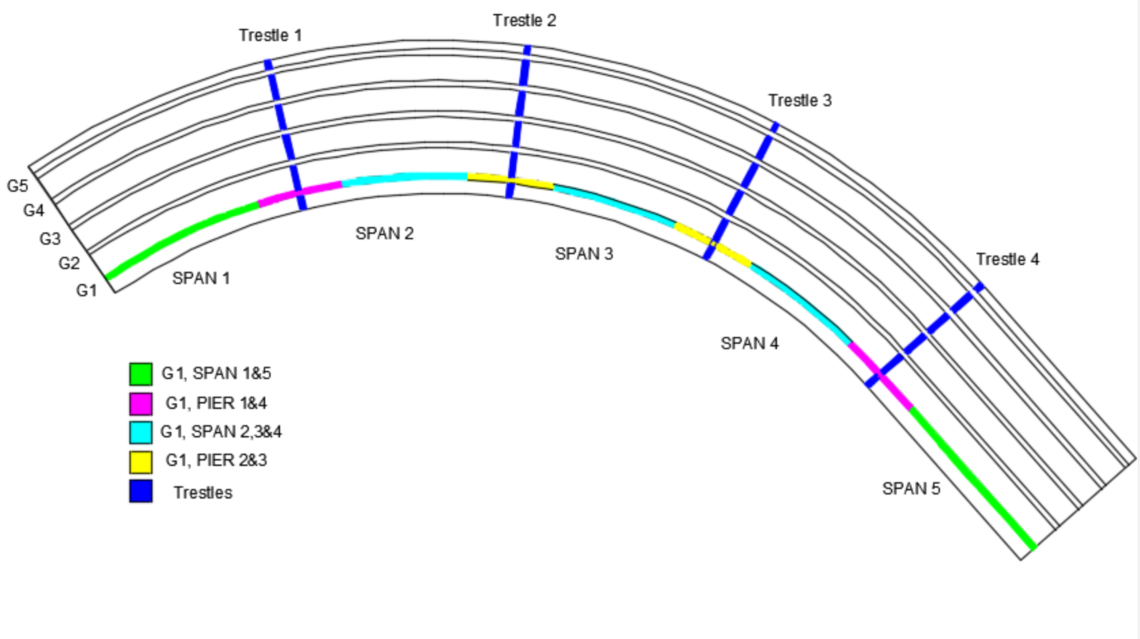


Figure 5.6 - Illustration of the different sections of the Girders (Top Plan of Ashworth Viaduct)

The cross-section shape is the same for all the girders, a rectangular hollow section made of steel, and connected to the reinforced concrete slab, forming a composite section. As mentioned, they are restricted to lateral movements at the trestles zone. All the girders have transverse stiffeners with a maximum longitudinal spacing of 690 mm and, for the sections positioned above the columns, there are also longitudinal stiffeners. An example of G1, spans 1&5 regarding its steel cross-section and the composite cross-section used in terms of calculations is given in Figure 5.7 (a) and (b), respectively. Details about the girders can be consulted in drawing BB.2 from Annex B.

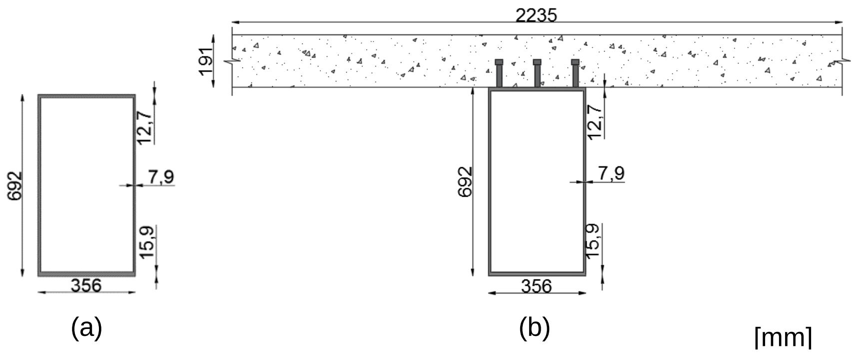


Figure 5.7 – Cross-section G1, span 1&5, steel only (a) and composite (b) (dimensions in mm

b) Trestles

The girders are then supported on the columns, whose name is also known by portal frame or trestles, through mechanical bearings (pinned). There are four trestles along the bridge that vary in height together with the slope shown by the slab. The designation of the trestles is made from the highest to the smallest, as it is represented in Figure 5.8.

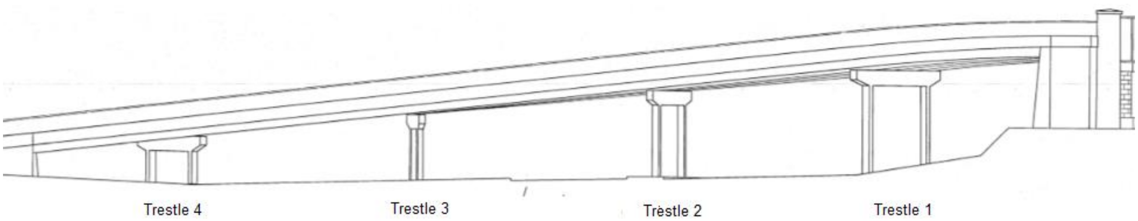


Figure 5.8 – Schematic representation of the Trestles in perspective

All the trestles show the same model, two columns and a crossbeam made of steel. Both columns have, each one, two square cross-sections, depending if the cross-section is in the bottom or in the top (Figure 5.9). The bottom one is referred as cross-section BB occupying approximately ¾ of the column and the top one as cross-section DD, filling approximately just ¼ of the column.

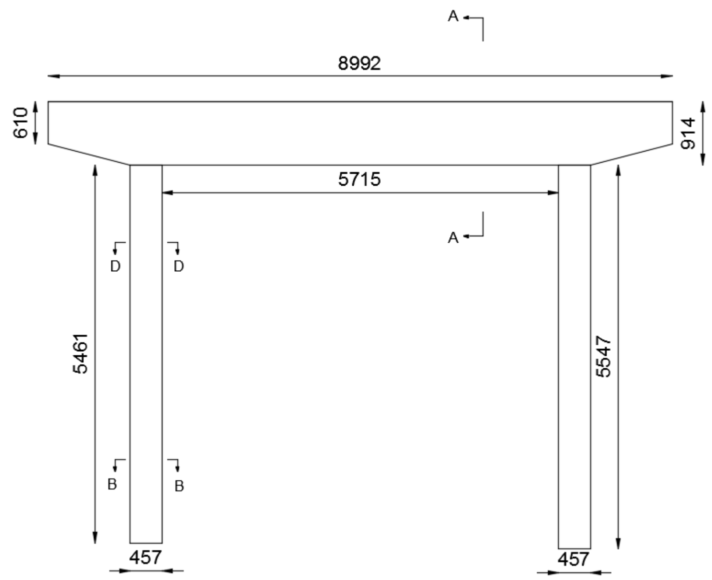
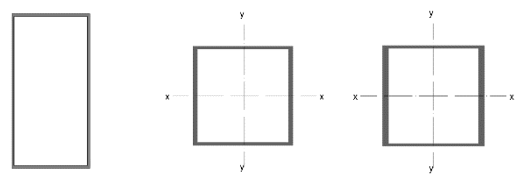


Figure 5.9 - Schematic Representation of Trestle 1 (dimensions in mm)

The Cross Beam is a rectangular hollow section that support the mechanical bearings, fixed into the columns. This beam also has transverse stiffeners spaced of 1067 mm and its cross-section is referred as cross-section AA. The three cross-sections are represented in the table below (Table 5.1), as well as their related dimensions.

Table 5.1 – Dimensions of the cross-sections of Trestles



Cross-Section	**AA**	**BB**	**DD**
Height (mm)	914,0	457,0	457,0
Width (mm)	457,2	457,0	457,0
Web thickness (mm)	13,0	13,0	13,0
Flanges thickness (mm)	15,9	19,1	25,4
Weld thickness (mm)	6,4	6,4	6,4
Area (mm^2)	36941	28095	33533

The dimensions of the four trestles can be found in drawing BB.3 (Annex B) and in table B.9 from Appendix B.

c) Deck slab

At last, there is the deck slab which is made of reinforced concrete and it is connected to the main girders by shear connectors, forming as it is known, a composite bridge. The deck slab also shows a transverse slope due to the variation height of the girders, and it can be seen in Figure 5.10.

The information regarding dimensions of the reinforcement and the slab are shown in Table B.6 in Appendix B.

Another important structural element is the parapet string course, an important barrier for accidents that is represented in Figure 5.10 as parapet A (left) and B (right).

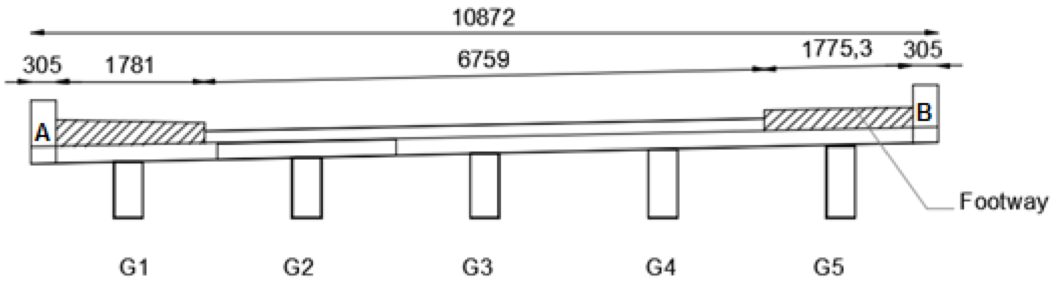


Figure 5.10 – Schematic representation of the deck slab supported in the girders (dimensions in mm)

The non-structural elements such as the carriageway, the footways, verges and so on are represented as well, and can be found in drawing BB.4, Annex B. They were also used, only as permanent loads, for calculations of the Structural Assessment.

5.5 Modelling the viaduct on MIDAS

For the Structural Assessment of Ashworth Viaduct through both Eurocodes and BDs, a finite element programme, called MIDAS, was used as to model and analyse the structure. Therefore, a model of the distribution of global loads to the box girders was created using a computer based grillage method (Figure 5.11). The properties of all sections were based on the as-built construction drawings, since the structure has no major defects. The properties of the deck slab grillage members were based on the as built deck thickness.

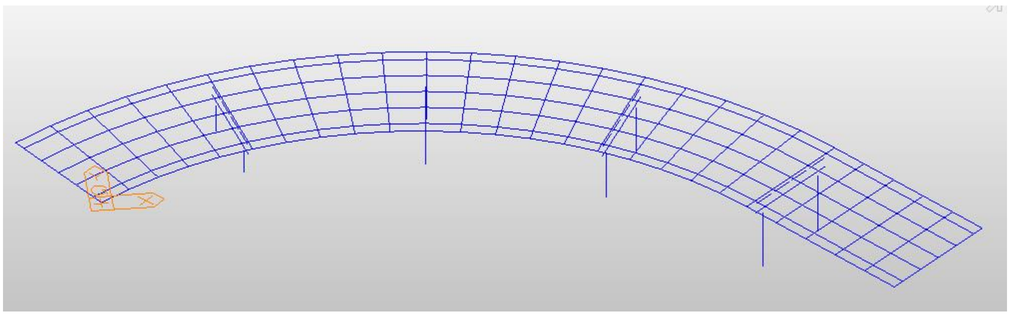


Figure 5.11 - Grillage model used for the Structural Assessment of Ashworth Viaduct

The columns were considered pinned at the supports as well as span 1 at the abutments. Span 5 has been considered as having roller bearings at north abutment. Between the girders and the slab, a link that restricts all the movements has been considered using the same for the connection between the cross-beam and the columns of the trestles. To represent the bearings above the trestles, a connection restricting all the lateral and vertical movements, although allowing the rotations, has been used.

A trimmer between the girders has also been modelled above the trestles. For the parapets, a simple solid rectangular beam has been used with the related dimensions of both parapet A and B.

5.5.1 Material Properties

The parameters used for the properties of materials related to steel and concrete were based on the information taken from the drawings BB.1 and BB.4, Annex B. The steel used in the girders is referred as being grade 50B which can be classified as having a strength of 355 MPa if the thickness is up to 16 mm and 345 MPa if the thickness is above 16 mm, according with Table 4.1. When considering concrete, the material is mentioned as having a 4500 psi compressive strength, which is equivalent to 31 MPa. The reinforcement used in the concrete slab is according with BS 785 where it is considered to have a yield strength of 250 MPa.

Modulus of elasticity has been chosen according with *BD 44/15* and *EN 1992-1-1* with regard concrete and *BD 21/01* and *EN 1993-1* regarding steel. With regard concrete, *BD 44/15* states that short term elastic modulus is equal to $(20 + 0.27 f_{cu})$ GPa with f_{cu} in MPa. However, when considering the effect of creep under a long-term loading, it is usually used half of the short-term elastic modulus for the modulus of elasticity. Therefore, the properties of the materials can be consulted in Tables B.10 to B.12, Appendix B.

Finally, despite the differences shown by the standards, the values used for the properties of materials were the same in order to ease the process of calculation and due to the fact of not influence future results.

5.5.2 Applied Loads

Before the application of loads, the notional lanes have been defined according to both standards. So, guided by the rules in section 4.4.2 regarding notional lanes, it was obtained the following Table 5.2.

Table 5.2 – Division of lanes, according to Eurocodes and BDs

	Eurocodes			**BDs**	
width of the carriageway	nº lanes	width	width of remaining area	nº lanes	width
6,7 m	2	3 m	0,7 m	2	3,35 m

The loads applied in the model under consideration according to the requirements referred throughout the different standards and consequently mentioned in the last chapter, are enumerated as follows:

- Dead Load and Superimposed Dead Load:

$$\rho_{material} \times Area \tag{5.1}$$

- Braking load due to traction of vehicles from load model 1 (Eurocodes) and HA loading (BDs) according to section 4.4.5, applied in the most onerous span (Table 5.3);

Table 5.3 – values of Braking Load for both Standards

Eurocodes	**BDs**
386,4 kN	378,0 kN

- Accidental load applied in the most unfavourable span, in terms of shear and bending moment effects at girders, according to section 4.4.6;

From Table 4.8, it is possible to obtain w_1, w_2 and "a" parameters, according to BD standards. Knowing that this road bridge is being assessed for 40 tonnes capacity:

- w_1 = 100 kN;
- w_2 = 60 kN;
- a = 1,5 m

- Pedestrian Load according to section 4.4.7

- Moving load cases

Table 5.4 – Moving loads applied in the models related to each standard

<u>Eurocodes</u>

- Load Model 1:
  - Tandem System (TS)
  - UDL
- Load Model 3:
  - SV 80
  - SV 100
  - SV 196

<u>BDs</u>

- HA UDL + KEL
- SV 80
- SV 100
- HB 45

To get the value of HA UDL load, the following process has been carried out:

Assuming L as the length of the span and equals to approximate 16 m, then, with the help of equation 4.11, it was obtained:

$$W_{UDL} = 336\ \times (1/16)^{0,67} = 52,4\ kN/m\ per\ national\ lane \tag{5.2}$$

Considering $a_L = 3{,}65$ and $AF = a_L/2{,}5$ for spans between 0 and 20 m, AF, as in adjustment factor, is equal to 1,46.

The factor K is then take it as 0,81 from Figure 5.12. Therefore:

$$HA\ UDL = 0{,}81 \times \frac{52{,}4}{1{,}46} = 28{,}8\ kN/m \tag{5.3}$$

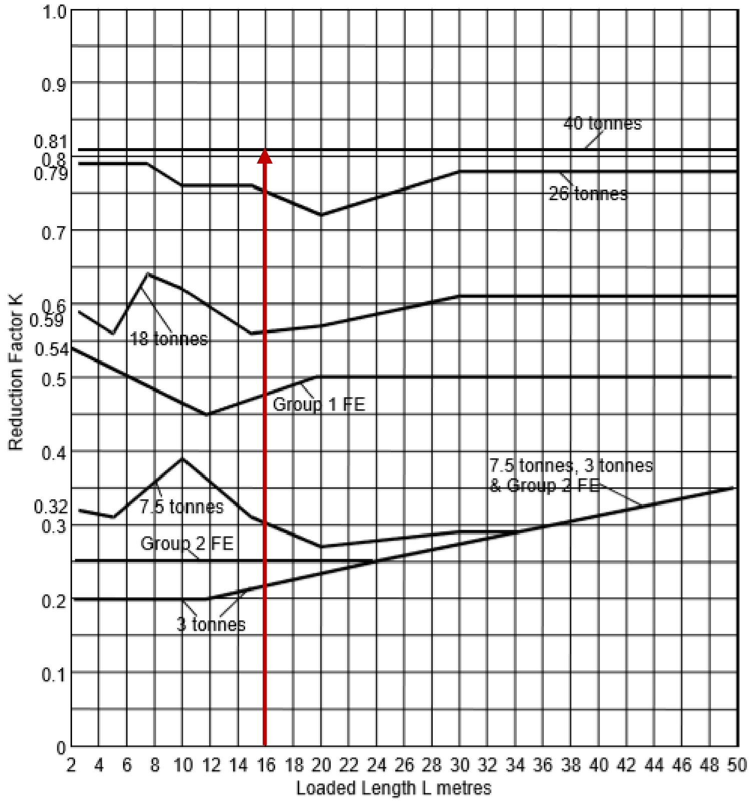


Figure 5.12 – Graphical illustration of K factors for Heavy Traffic Good Surface (Hg) [29]

According to *EN 1991-2,* an adjustment factor α is used in load model 1 as described in Table 5.5

Table 5.5 – Adjustment factors for Load model 1

	α_Q (Tandem System)	α_q (UDL loading)
Lane 1	$\alpha_{Q1} = 1{,}0$	$\alpha_q = 0{,}61$
Lane 2	$\alpha_{Q2} = 1{,}0$	$\alpha_q = 2{,}2$

The combinations of actions applied to the structure under consideration were based in equations 4.7 and 4.8 when regards to Eurocodes. For BD standards, it was only considered combination 1 with the application of equations 4.9 and 4.10. The partial safety factors used in both models are shown in Table 4.4.

The values of ψ factors used in this case, are described below (Table 5.6):

Table 5.6 – Recommended values of ψ factors for road bridges

		ψ_0	ψ_1	ψ_2
LM 1 + pedestrian loads	TS	0,75	0,75	0
	UDL	0,4	0,4	0
	pedestrian	0,4	0,4	0

The actions applied according to LM1 are then represented in the Table 5.7:

Table 5.7 – Actions applied according to LM1

	TS (Tandem System)	UDL loading
Lane 1	$1{,}0 \times 600 = 600$ kN	$0{,}61 \times 9 = 5{,}5$ kN/m²
Lane 2	$1{,}0 \times 400 = 400$ kN	$2{,}2 \times 2{,}5 = 5{,}5$ kN/m²

The different combinations used for structural assessment are described in Appendix A.

5.6 Resistance of Sections

The calculations associated with the resistance of the structural elements of Ashworth Road Viaduct have been done in accordance with *EN 1992-2, 1993-2; BS 5400-3, BD 56/10 and BD 44/15*. Hence, the principles and equations applied for the structural assessment have been already mentioned in chapter 4, regarding the main differences between Eurocodes and BDs.

Throughout the current chapter, it is going to be considered just one example as a model of analysis for each structural member regarding the composite girders and the trestles. However, the data and results of all the structural elements can be found in Appendixes B and D, respectively.

5.6.1 Structural Capacity of Girder 1

All the information related to the geometry of Girder 1 can be consulted in Appendix B. The intermediate calculations that served as a mean to the final results, such as the inertia along the main and the secondary axis, the radius of gyration, the neutral axis and the section modulus, can also be found in Appendix C.

In order to simplify the process, the concrete slab was firstly considered as a steel material, so the long-term elastic modular ratio (m_{lt}) or just modular ratio (m) is obtained as follows:

$$m_{lt} \text{ or } m = E_s/E_c \quad (5.4)$$

Hence, b_{eff} according to the Eurocodes and according to BDs is (Table 5.8):

Table 5.8 – Modular ratio, width and effective width of the composite flange (girder 1)

	Eurocodes	**BDs**
m_{lt} or m	6,8	14,4
b (mm)	2235	2235
b_{eff} (mm)	329	310

i. Classification of girder G1, span 1&5

The classification of the box girders was carried out based on the rules introduced in section 4.5.1.1 and respecting the longitudinal spacing between shear connectors referred in section 4.5.1.2. The section situated in span 1&5 from Girder 1 (Figure 5.7) can be then defined as being:

Table 5.9 - Classification of Girder 1 on spans 1&5

Class	**Eurocodes**	**BDs**
Compression flange	Class 1	Non-Compact[5]
webs	Class 3	Not-compact
Overall section	Class 3	Non-compact

ii. Assessment Resistance of G1, span 1&5

From section 4.5.1 and knowing that the girder is already composite in the last stage, that is, the compression flange is fully restrained by the concrete slab, it is possible to conclude that the effective length for lateral torsional buckling (l_e) is zero. Therefore, the effects of lateral torsional buckling can be automatically excluded from the process and thus, the element is considered to fail due to its resistance against bending only.

Thus, considering the class of the section and the principles from both standards, shown in section 4.5.1.3, it is possible to obtain the equations and the results in Table 5.10 shown below. The equations are based on the data existing in Appendices B and C, according to EN 1993-1 and BS 5400-3, respectively.

$$M_{c,Rd} = \frac{W_{el,min} f_y}{\gamma_{M0}} = \frac{7{,}4E6 \times 355}{1{,}0} \quad (5.5)$$

[5] The compression flange was identified as non-compact, not only because it did not comply with the rules established by the classification of box girders, but also because it did not meet the longitudinal shear rules.

$$M_D = \frac{Z_{xt} \times f_{yc}}{\gamma_{f3}\gamma_{m,steel}} = \frac{7{,}4E6 \times 345^6}{1{,}1 \times 1{,}05} \quad 5.6$$

Table 5.10 – Bending Resistance of G1, spans 1&5, obtained in both Standards

	Eurocodes	**BDS**
$M_{c,Rd}$ or M_D (kNm)	2550	2429

Using the characteristics of structural steel cross-section from G1, span 1&5, the checking process of shear resistance under pure shear was carried out taking into account the existent transverse stiffeners (Figure D.1, Appendix D).

According to *EN 1993-1*, the cross-section under consideration has satisfied the equation from Table 4.14 regarding class 3 cross-sections and the equation 4.23 (for stiffened webs), which mean it did not have to be checked against shear buckling. According to Table 4.13 related to *BS 5400-3*, the following result was obtained:

$$V_D = \left[\frac{7.9 \times 651}{1{,}05 \times 1{,}1}\right] \times 205 = 1860\ kN \quad 5.7$$

The spacing between transverse stiffeners is given through the following equation:

$$a = \frac{L}{N_{panel}} = 693\ mm, \quad 5.8$$

Where L is the length of the span and N_{panel}, corresponds to the number of the existent transverse stiffeners in each span. The parameter τ_l is the limiting shear strength of the web panel and can be determined according to Figure 5.13.

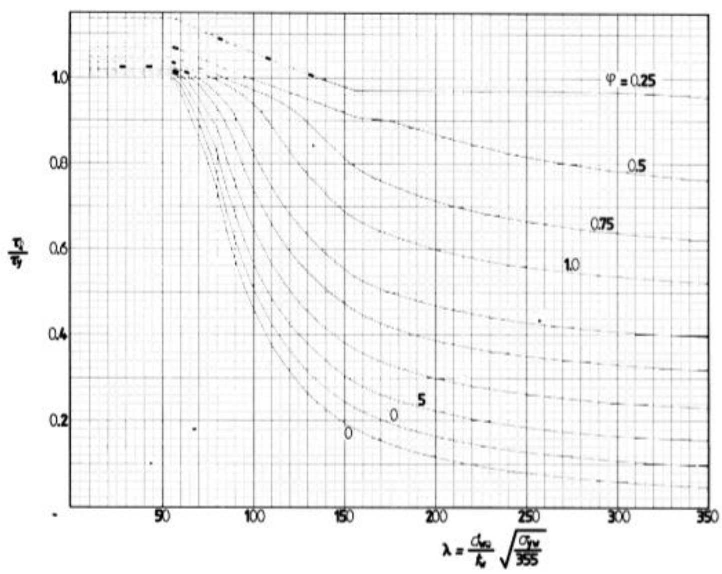


Figure 5.13 – Graphical representation of limiting shear strength τ_l for $m_{fw} = 0{,}005$ [44]

[6] See note from Appendix B

The same process has been repeated for the cross-sections in spans 2,3 and 4 from Girder 1. All the assessment calculation and results related to girder 1 can be found in Appendix D.

iii. Classification of girder G1, span 1&4

The same process of section classification has been used in this section, being the results exposed in the following Table 5.11:

Table 5.11 - Classification of Girder 1 on trestles 1&4

Class	Eurocodes	BDs
Compression flange	Class 1	Compact[7]
webs	Class 3	Non-compact
Overall section	Class 2	Non-compact

In accordance with 4.5.1.1, the overall cross-section can be considered as being class 2 because it met the requirements established by the Eurocodes. So, in this case, the section can mobilise plastic moment according to the Eurocodes, but not according to BD standards.

These cross-sections from girder 1 are resisting a negative moment (hogging), which means that the part of the section in compression is not laterally restrained by the concrete slab as it happens in the sections at mid-span. Although it would make sense to check the capacity of these elements against lateral torsional buckling, box girders have a high resistance against this, hence a low probability of failing due to lateral buckling. For this reason and the fact these cross-sections have a longitudinal stiffener[8] in the compression part, they have not been checked against lateral torsional buckling.

The effects of Bending Resistance for both cases, can be seen in Table 5.12.

Table 5.12 – Bending Resistance of G1, Trestles 1&4

	Eurocodes	BDs
$M_{C,Rd}$ or M_D (kNm)	3308	2465

The cross-section under consideration is subjected to hogging moments and therefore, a plastic analysis has been considered according to the Eurocodes. The results, based on Table 4.13 are shown in Table 5.13.

[7] The compression flange was identified as non-compact, not only because it did not comply with the rules established by the classification of box girders, but also because it did not meet the longitudinal shear rules.

[8] See drawing BB.2 from Annex B

Table 5.13 – Shear Resistance of G1, Trestles 1&4

	Eurocodes	**BDs**
$V_{C,Rd}$ or V_D (kN)	2325	2056

Afterwards, it was necessary to verify the combined bending and shear due to the fact that they can both be present in the places under consideration. Since shear force is less than half of the plastic shear resistance, this verification may be neglected, according to clause *6.2.8 (2), EN 1993-1*.

The principles stated in section 4.5.1.5, according to BS 5400-3, were applied for all combinations and, hence, satisfied for the combination that produces the highest acting bending moment. Thus, all of them satisfied the following equations (5.9 and 5.11):

$$M < M_f \tag{5.9}$$

Where,

$$M_f = M_D \tag{5.10}$$

And,

$$V < V_R \tag{5.11}$$

And, as it was already mentioned in section 4.5.1.5, $V_f = V_D$.

When putting the effects of torsion under verification it is necessary to take account of the effects of St. Venant. The warping effects can be ignored, when considering box girders, as said before.

The results obtained for the reduced resistance of shear due to torsion effects were the following (Table 5.14):

Table 5.14 – Resistance against Torsion

	Eurocodes	**BDs**
$V_{pl,T,Rd}$ (kN)	1240	______
$V_D/2$ (kN)	______	1028

The same calculations have been repeated for the section above trestles 2 and 3 from Girder 1. Its results can be found in Appendix D.

5.6.2 Structural Capacity of Trestle 1

The trestle is divided into three different types of cross-sections which are subjected to different kinds of actions, specially between the cross beam and the columns. Therefore, and similar to the verification process of the girders, the resistance of the trestle has also been determined according to the principles already introduced in sections 4.5.1 and 4.5.2.

i. Classification of sections

For the classification of the cross-sections of the columns of trestle 1 (Table 5.1), it has been considered the cross-sections completely under compression in order to obtain the most critical case and therefore the lowest possible class (Table 5.15).

Table 5.15 – Classification of the sections of the trestle

		Eurocodes	**BDs**
Section AA	webs	Class 3	Non-Compact
	flanges	Class 1	Compact
	Cross-section	Class 3	Non-Compact
Section BB	webs	Class 2	Compact
	flanges	Class 1	Compact
	Cross-section	Class 2	Compact
Section DD	webs	Class 2	Compact
	flanges	Class 1	Compact
	Cross-section	Class 2	Compact

ii. Assessment Resistance of Trestle 1

The results of moment resistance for all cross-sections are registered from Tables 5.16 to 5.18:

Table 5.16 -Bending Resistance of section AA

	Eurocodes	**BDs**
M_D (kNm)	4071	3152

Table 5.17 - Bending Resistance of section BB

	Eurocodes	**BDs**
M_{D_xx} (kNm)	1547	1432
M_{D_yy} (kNm)	1756	1622

Table 5.18 - Bending Resistance of section DD

	Eurocodes	**BDs**
M_{D_xx} (kNm)	1755	1625
M_{D_yy} (kNm)	2151	1990

The results of shear resistance for all cross-sections are shown in Table 5.19:

Table 5.19 - Shear Resistance of Trestle 1

V_D (KN)	Eurocodes	BDs
Cross Section AA	4758	4376
Cross Section BB	2380	2020
Cross Section DD	2380	1958

When regards to axial resistance, the results of the columns are shown in Table 5.20

Table 5.20 – Axial Compression Resistance of the columns of trestle 1

$N_{C,Rd}$ or P_D (kN)	Eurocodes	BDs
Cross Section BB	9974	8954
Cross Section DD	11904	10687

According to section 4.5.2.1 regarding axial compression, the checking of the columns against buckling has been carried out. Although it is not conditioning for the structure capacity, the slenderness of the column determined from the different methods, where $L = 5547$ m, is shown in Table 5.21.

Table 5.21 – Slenderness of the columns of trestle 1

λ (m)	Eurocodes	BDs
Cross Section BB	20,7	25,0
Cross Section DD	20,3	24,7

5.6.3 Structural Capacity of the Concrete Slab

As mentioned previously in Chapter 4, the concrete slab has been analysed considering a simply supported beam. The capacity of the reinforced concrete slab is considered only 1.8m of the slab section as minimum resistance for conservative assessment results. According to Eurocodes and BDs standards, it was possible to obtain the results of bending and shear resistance of the concrete slab, shown in Tables 5.22 and 5.23.

Table 5.22 – Bending capacity of the concrete slab

	Eurocodes	BDs
M_{Rd} or M_D (kNm)	171	104

Table 5.23 -Shear resistance of the concrete slab

	Eurocodes	BDs
V_{Rd} or V_D (kN)	395	277

The data regarding the structural capacity of the concrete slab can be found in Table B.6, Appendix B.

Chapter 6

6. Discussion of results

6.1 Output results from Girders

Throughout this work, the differences between the Eurocodes and BDs Standards have been carried out in order to understand in which way they differ and how can they be comparable. The main differences focus on the actions applied on bridges, the procedure of obtaining the capacity of the structural elements and their consequent results. For a deeper analysis between the two standards, the effects of those actions have been taken into account as well as the capacity of the structural elements, plotting the relative amount between them into graphical illustrations.

Girder 1 and Girder 3, represented in Figure 6.1, have been assessed for all the combinations[9], as an example of comparison.

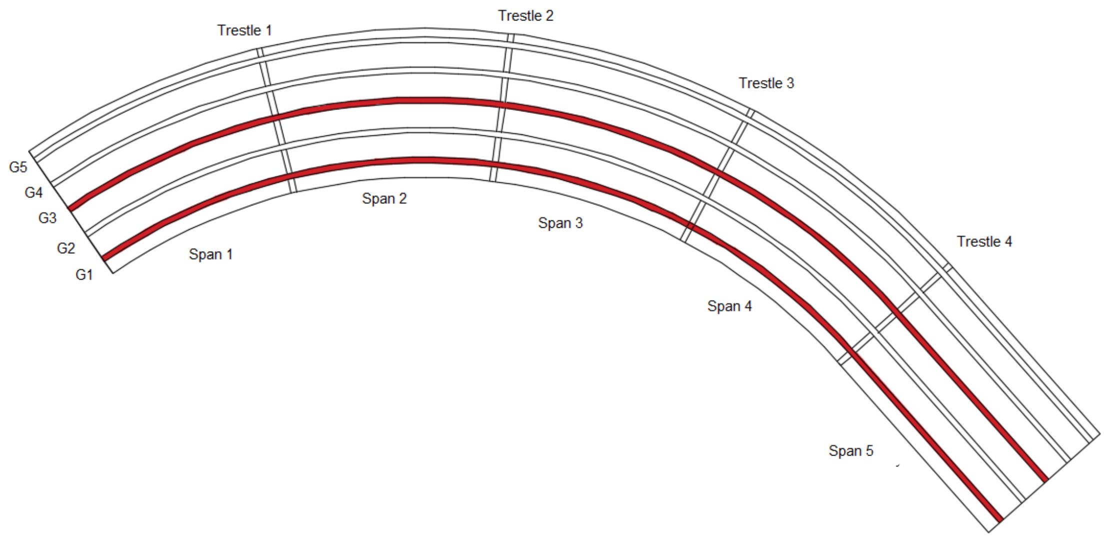


Figure 6.1 – Schematic representation of the girders under consideration (G1and G3)

[9] See Appendix A.

According to equations (4.18) and (4.21) and considering the structural capacity of Girder 1 and Girder 3 regarding bending resistance and shear capacity, it was possible to obtain the following graphical illustrations. It is also important to highlight the fact that all the partial safety factors mentioned in section 4.2.4 have been already considered both in actions and resistances. All de data related to the structural members and the intermediate calculations for the results of structural capacity can be found in Appendices B, C and D.

6.1.1 Results from girder 1

The graphical results regarding $M_{Ed}/|R|$ and $V_{Ed}/|R|$ displayed by Girder 1, in both standards, due to self-weight, are shown in Figure 6.2 and 6.3:

Figure 6.2 – Graphical illustration of $M_{Ed}/|R|$ results due to Self-weight along Girder 1

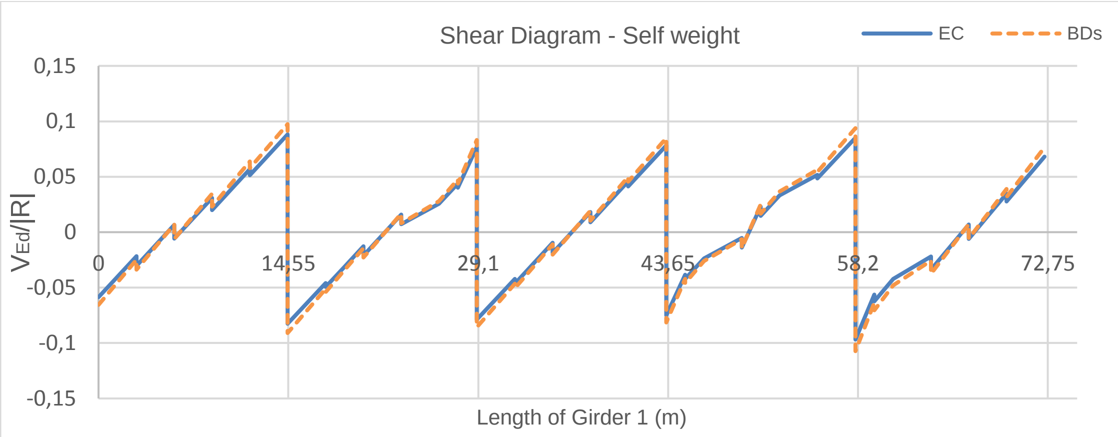


Figure 6.3 - Graphical illustration of $V_{Ed}/|R|$ results due to Self-weight along Girder 1

The scale of the abscissa is divided according with the position of the supports along the girder which is 14,55 m according to the length of girder 1. For the input of the self-weight of the structure, the only parameter changing in the standards is the partial factor used to increase the action, introduced in Table 4.4. Since the load applied and the output obtained (bending moment and shear force) are directly proportional, the results are expected to be similar in both standards due to the fact of the values of the partial factors do not differ that much between codes. The effects of actions due to self-weight are shown in Figures 6.4 and 6.5.

Figure 6.4 - Graphical illustration of Bending Moment results due to Self-weight along Girder 1

Figure 6.5 - Graphical illustration of Shear Force results due to Self-weight along Girder 1

As can be confirmed in the graphics above, the effects of actions shown by girder 1 under both standards differ in only 3%. However, when considering the resistances calculated in chapter 5, the differences are notorious, which means that they are higher than the ones verified for the effects of actions.

When comparing the results between the two standards from tables 5.10 and 5.12 regarding bending moment capacity of girder 1, higher values are registered for the Eurocodes. However, the percentage of difference varies with the section of the girder, being an average of 5% when regards to sections resisting sagging moments and 30% when regards to sections resisting hogging moments.

Since it was considered that the sections resisting hogging moments could mobilise the plastic moment according to the Eurocodes, the difference is expected to be higher.

Regarding shear capacity of girder 1, it was notice an average percentage of 15% difference, showing also higher results for the Eurocode.

6.1.1.1 Results related to normal traffic

Despite of the models related to regular traffic, mentioned in chapter 4, being dissimilar, they cover the same concept. Thus, a comparison between them, associated with the maximum effects of actions produced, have been also carried out

Therefore, the graphical results regarding $M_{Ed}/|R|$ and $V_{Ed}/|R|$ displayed by Girder 1 due to combination 1a and combination 2a (see Appendix A) are shown in Figure 6.6 and 6.7:

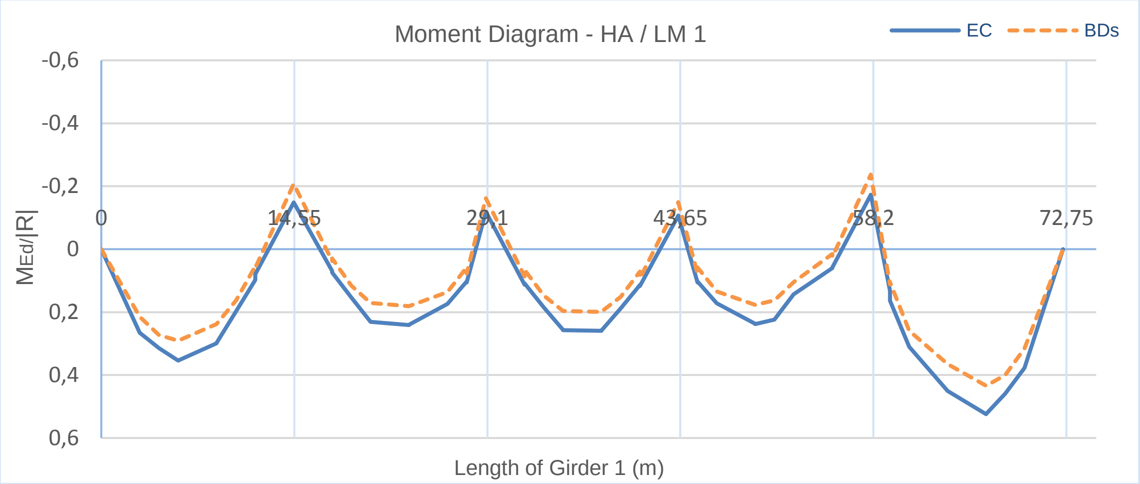


Figure 6.6 - Graphical illustration of $M_{Ed}/|R|$ results due to combinations 1a and 2a along Girder 1

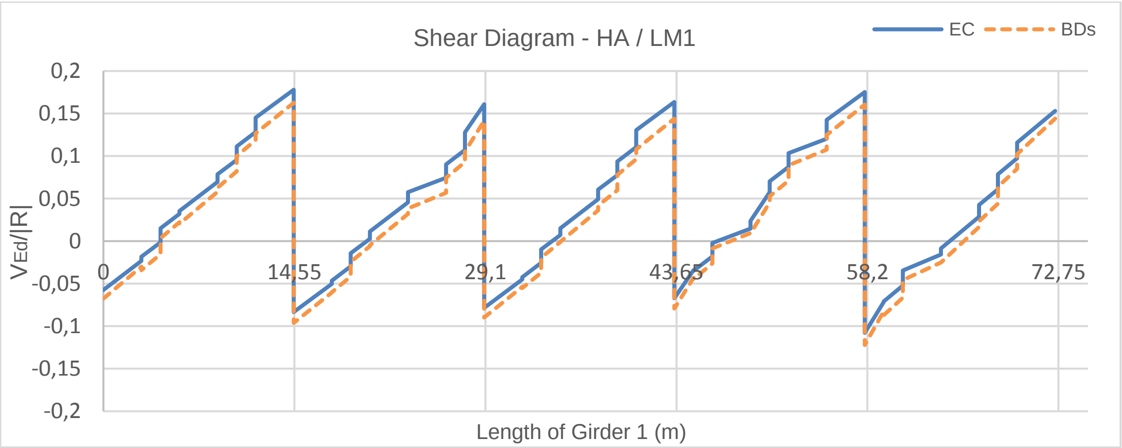


Figure 6.7 - Graphical illustration o$V_{Ed}/|R|$ f results due to combinations 1a and 2a along Girder 1

As mentioned previously, the Eurocodes consider as Load Model 1 a vehicle type and a uniform distributed Load, whilst BD standards consider a uniform distributed load and a knife edge. Based on the loads associated with each action, described in sections 4.4.3 and 4.4.4, the effects of actions are expected to be higher in the Eurocodes as shown by Table 6.1.

Table 6.1 – Comparison between LM1 and HA

	BDs	Eurocodes
	HA UDL + KEL	LM 1
UDL	28,8 kN/m per Notional Lane	Lane 1: $5{,}5 \times 3{,}65 = 20$ kN/m Lane 2: $5{,}5 \times 3{,}65 = 20$ kN/m
Concentrated Load	120 kN per National Lane	Lane 1: 600 kN per NL Lane 2: 400 kN per NL

Furthermore, it is also known that the capacity results are higher in the Eurocodes with an overall average of 10%, considering sagging and hogging sections together. This fact approximates the results of $M_{Ed}/|R|$ from both standards due to the higher reduction of the effects of actions induced in the Eurocodes.

In conclusion, a difference of 40% is registered in the effects of actions of the two combinations (Figure 6.8) that leads to a difference of 30% when regards to the relative results of $M_{Ed}/|R|$.

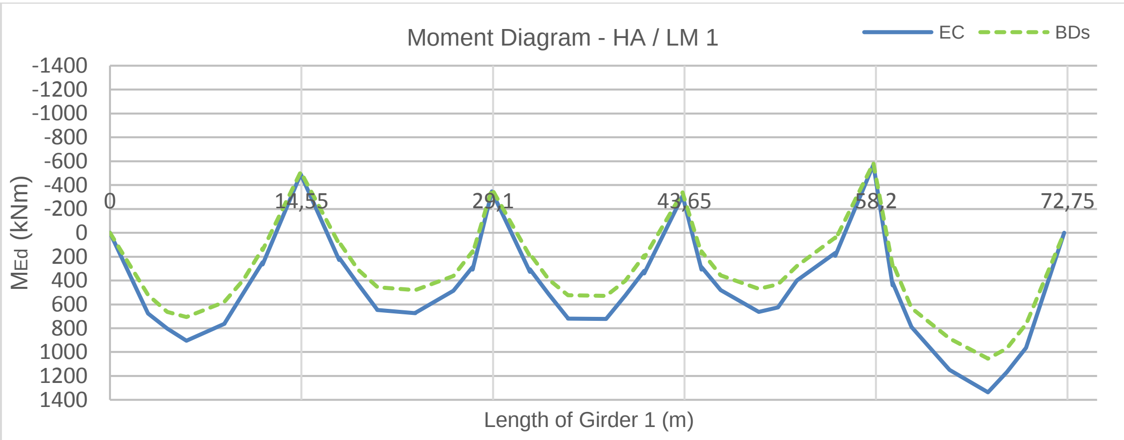


Figure 6.8 - Graphical illustration of bending moment results due to Combinations 1a and 2a along Girder 1

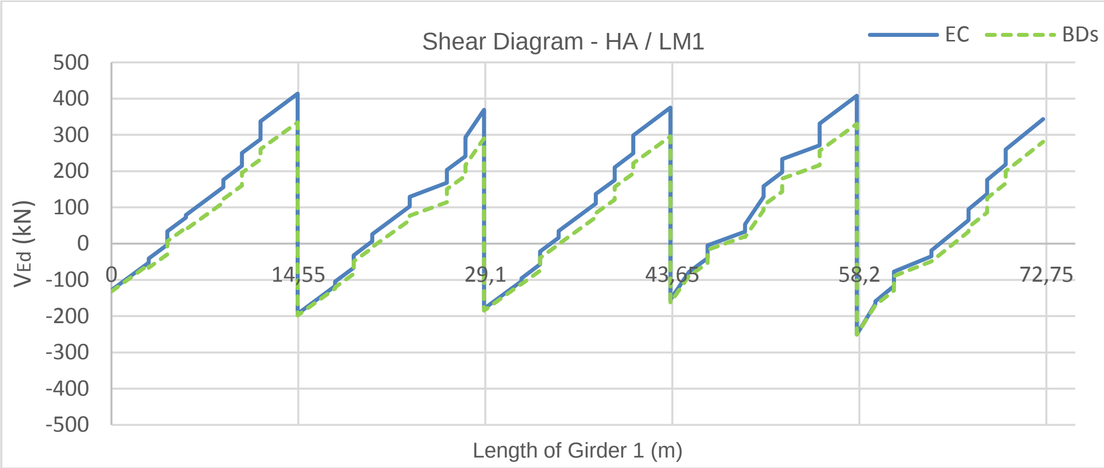


Figure 6.9 - Graphical illustration of shear force results due to Combinations 1a and 2a along Girder 1

With regards to shear diagram, the effects of actions produce 20% higher results in the Eurocodes. Therefore, knowing that the capacity results have shown an average of 15% difference, the relative results of $V_{Ed}/|R|$ are approximately 10% different.

6.1.1.2 Results related to SV 80

Within the scope of special vehicles, it is possible to conclude that the models introduced in sections 4.4.3 and 4.4.4 are pretty much the same when related to SV 80 and SV100. The graphical illustrations for the relative results of $M_{Ed}/|R|$ and $V_{Ed}/|R|$ between the standards for combination 1b and 2b are shown in Figures 6.10 and 6.11, respectively.

The results have also been taken for the maximum effects produced by this type of vehicles. The higher results are shown by span 5 and Figure 6.10 helps to understand the position of vehicles in the structure when the maximum bending effects are verified.

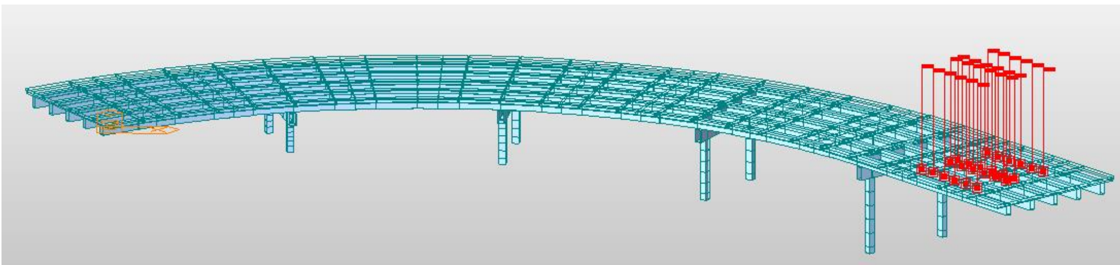


Figure 6.10 – Position of SV 80 producing the maximum bending effects

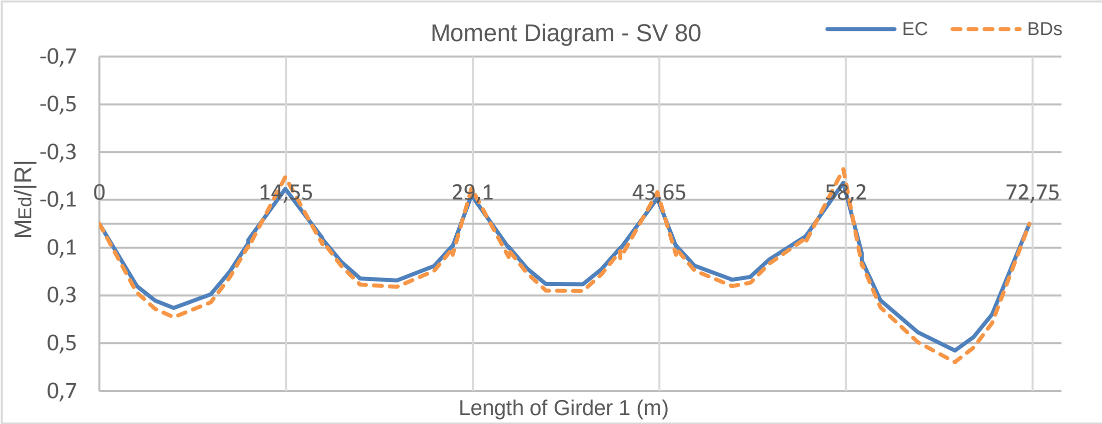


Figure 6.11 - Graphical illustration of $M_{Ed}/|R|$ of results due to combination 1b and 2b along Girder 1

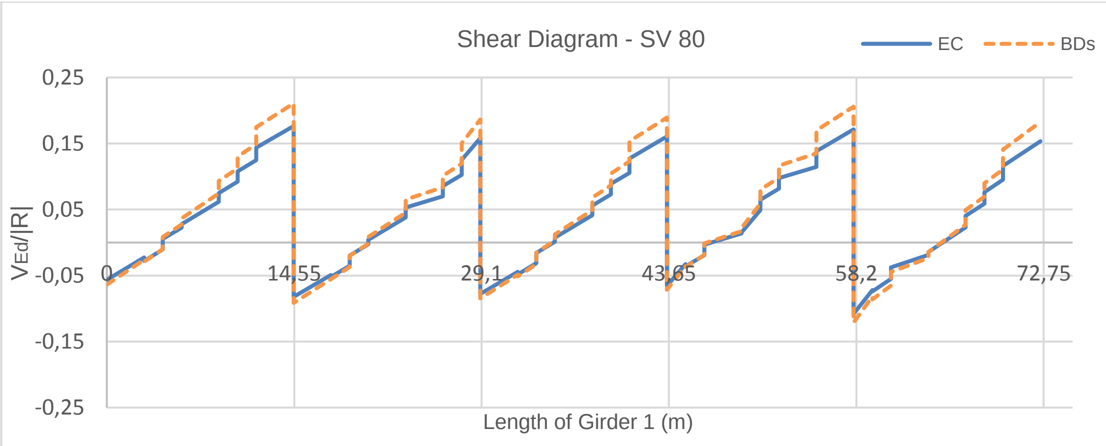


Figure 6.12 - Graphical illustration of $V_{Ed}/|R|$ f results due to combination 1b and 2b along Girder 1

The results obtain directly from the graphics are really similar between both standards, showing a difference of 15% higher results in BDs. To help this interpretation, Figures 6.13 and 6.14 are introduced to represent only the effects of actions produced by the combinations mentioned above.

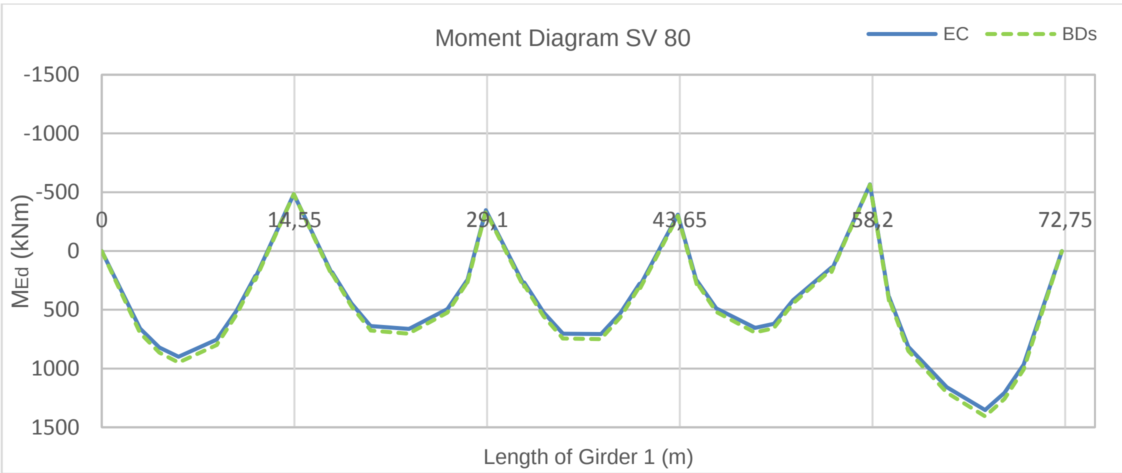


Figure 6.13 - Graphical illustration of bending moment results due to Combinations 1b and 2b along Girder 1

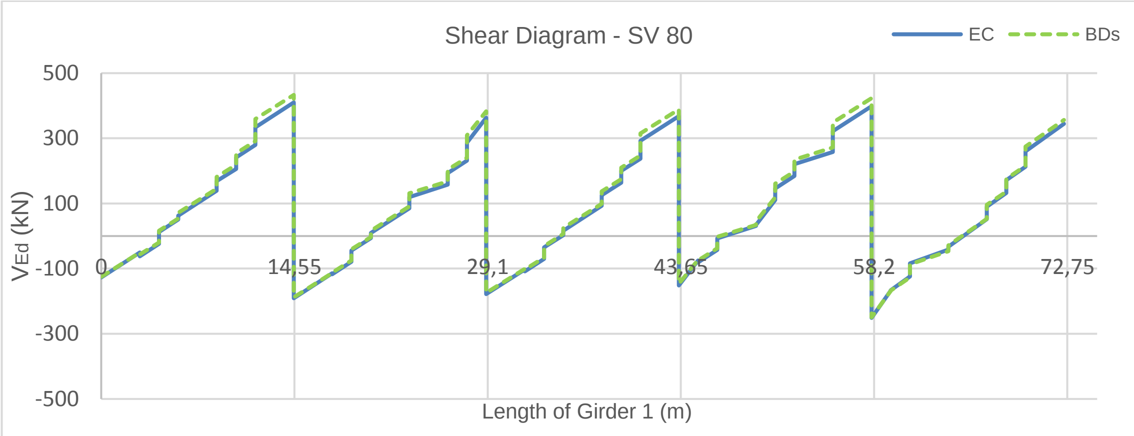


Figure 6.14 - Graphical illustration of shear force results due to Combinations 1b and 2b along Girder 1

Despite the higher values registered in BDs standards in Figures 6.11 and 6.12, the difference of results from the effects of actions are negligible. Furthermore, the fact that the member under analysis is an outer girder must be highlighted since the effects of traffic loads are not that significant when compared to an inner girder.

6.1.1.3 Results related to SV 100

The same process has been made for combinations 1c and 2c and the graphical illustrations for the relative results of $M_{Ed}/|R|$ and $V_{Ed}/|R|$ related to it are shown in Figures 6.15 and 6.16, respectively.

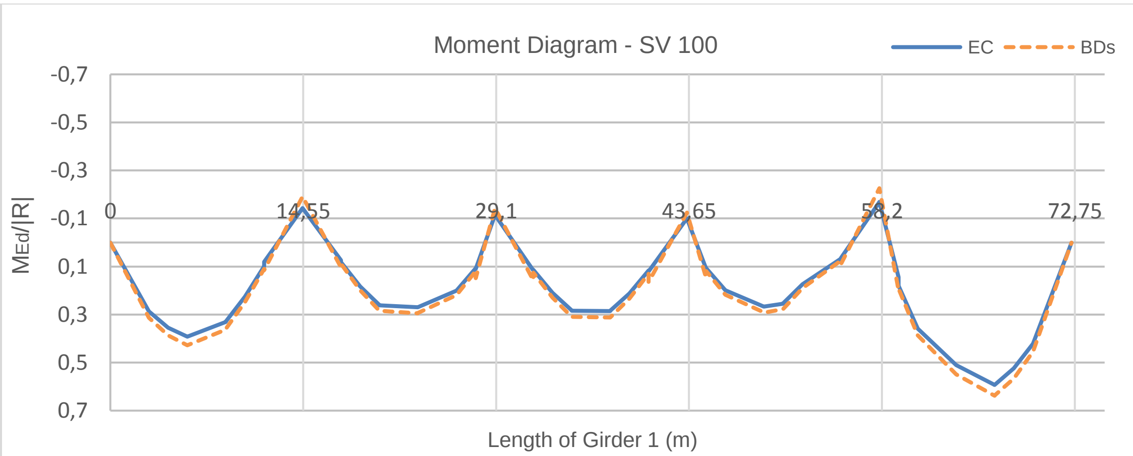


Figure 6.15 - Graphical illustration of $V_{Ed}/|R|$ of results due to combination 1c and 2c along Girder 1

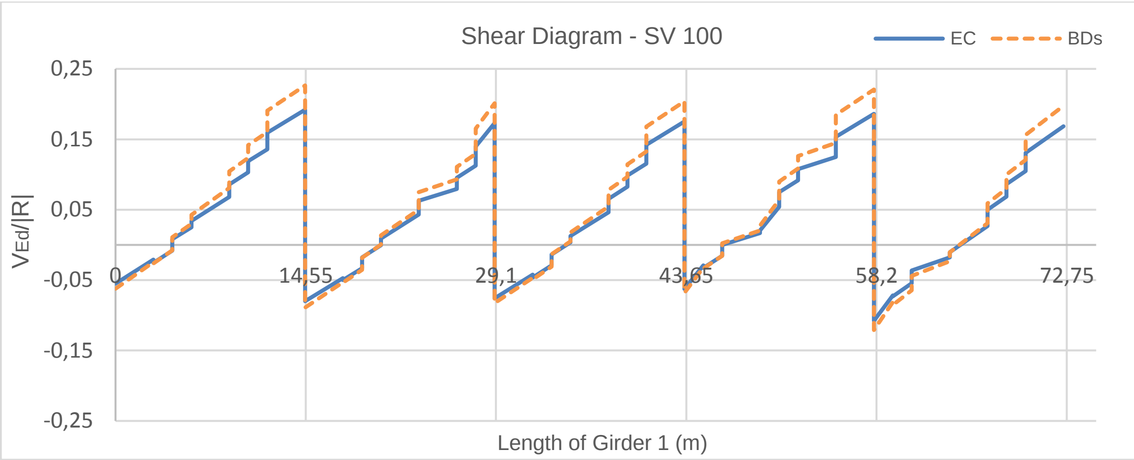


Figure 6.16 - Graphical illustration of $M_{Ed}/|R|$ of results due to combinations 1c and 2c along Girder 1

The results in this combination are similar to the one related to SV 80, showing also an average of 15% difference for the relative values of $M_{Ed}/|R|$ and $V_{Ed}/|R|$, with higher results in BDs.

6.1.1.4 Results related to pedestrian action

Another important aspect is the pedestrian load, since this one is directly applied above girder 1. For that reason, an analysis regarding just pedestrian load has been carried out for both standards. The percentage difference of resistances is already known, so this analysis is only focused in the actions as it can be seen in Figure 6.17.

The 20% difference is mostly due to the fact that the programme has considered the pedestrian load in different places for the same maximum moment when regards to the models produced by the Eurocodes and BDs. When considering the loads showed in section 4.4.7, they are both similar.

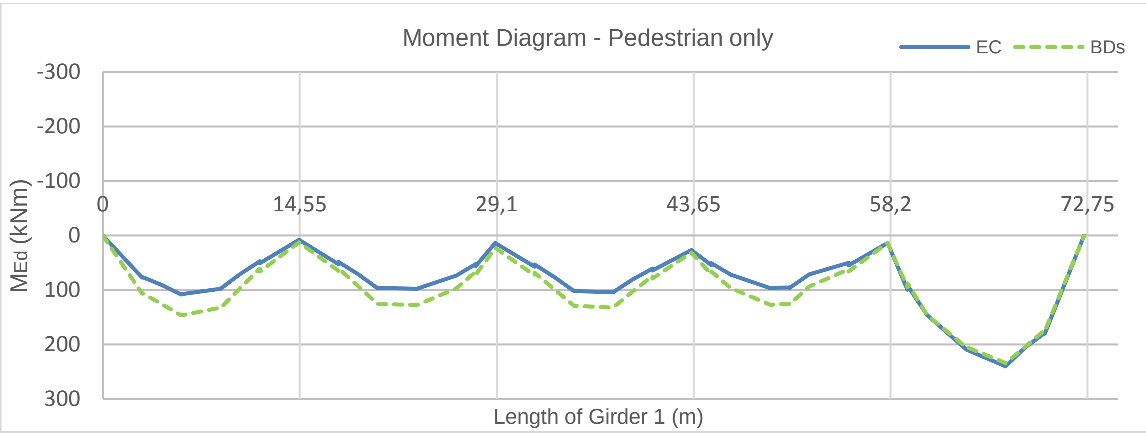


Figure 6.17 - Graphical illustration of bending moment results due to pedestrian load along Girder 1

6.1.1.5 *Results related to Accidental combination*

To conclude the analysis of Girder 1, a graphical analysis has been carried out for the most onerous span (span 5), when subjected to the accidental action mentioned in section 4.4.6 (Figures 6.18 and 6.19).

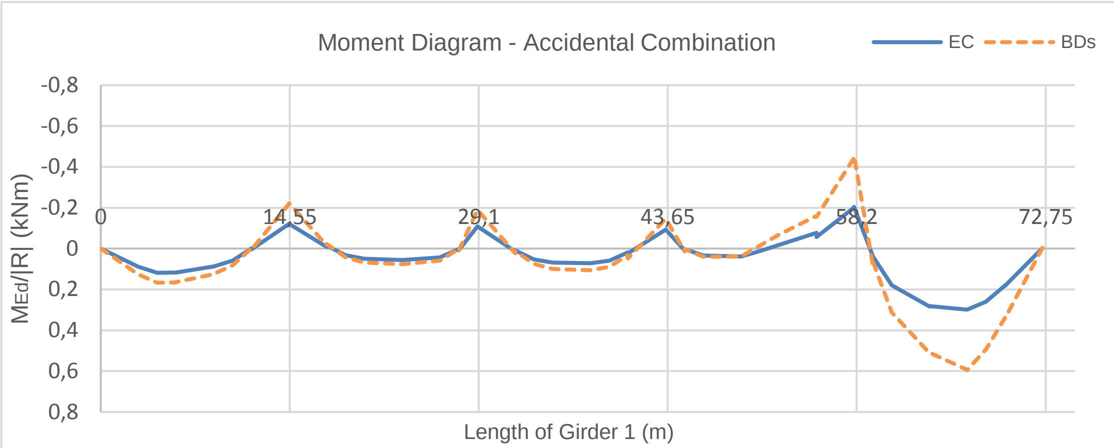


Figure 6.18 - Graphical illustration of $M_{Ed}/|R|$ of results due to combination 1d and 2d along Girder 1

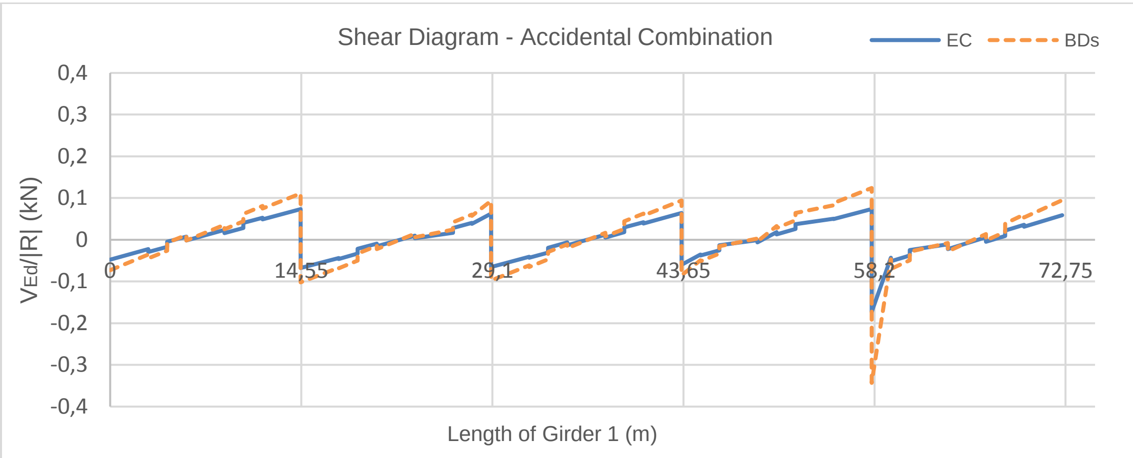


Figure 6.19 - Graphical illustration of $V_{Ed}/|R|$ of results due to combinations 1e and 2e along Girder 1

Based on the graphics above, it was possible to obtain 40% and 30% of difference for the relative difference of $M_{Ed}/|R|$ and $V_{Ed}/|R|$, respectively. Considering the 10% and 15% difference verified in the resistance of the elements against bending moment and shear force, respectively, an average of 30% and 15% is shown for bending and shear effects produced by accidental actions.

6.1.2 Results from girder 3

For a deeper analysis of the main differences between the two standards, girder 3 has also been an object of study, as shown in Figure 6.1. The loaded length of girder 3 is 16 m and so it is the scale defined for the abscissa of the graphical illustrations shown in Figures 6.20 and 6.21 related to the self-weight of girder 3.

Figure 6.20 - Graphical illustration of $M_{Ed}/|R|$ of results due to self-weight along Girder 3

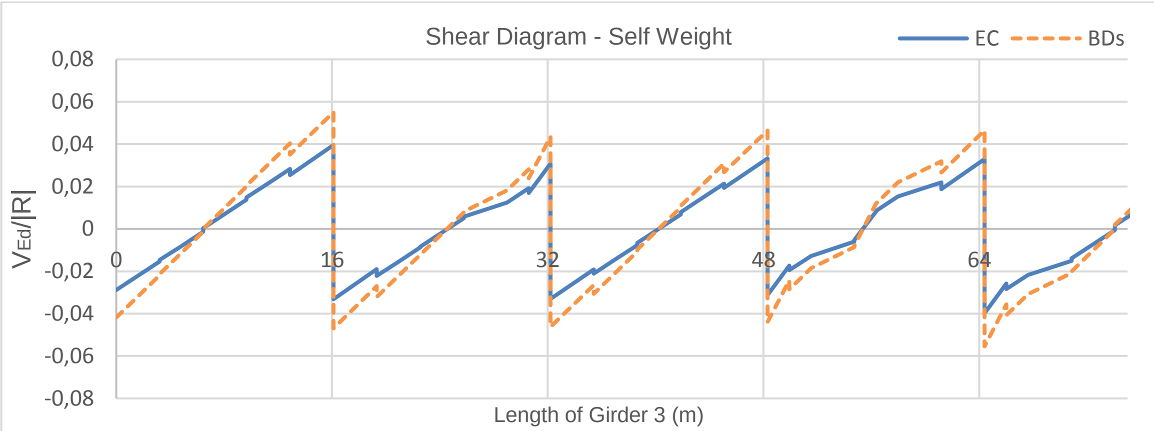


Figure 6.21 - Graphical illustration of $V_{Ed}/|R|$ of results due to self-weight along Girder 3

Considering the capacity of girder 3, that also shows a 10% difference regarding bending moment and 14% variance regarding shear force, the relative difference of $M_{Ed}/|R|$ and $V_{Ed}/|R|$ are respectively 15% and 20 %.

However, in this case, the differences shown by the effects of actions are higher than in girder 1, with 20% higher results in BDs for both bending moment and shear force (Appendix E). This variance must be related to the dead load and superimposed dead load that is directly applied in the girder under consideration, according to Figure 5.10.

6.1.2.1 Results related to SV 80

The results of the effects of actions produced by combinations 1b and 2b associated with SV 80, in girder 3 are possible to see in Figures 6.22. and 6.23.

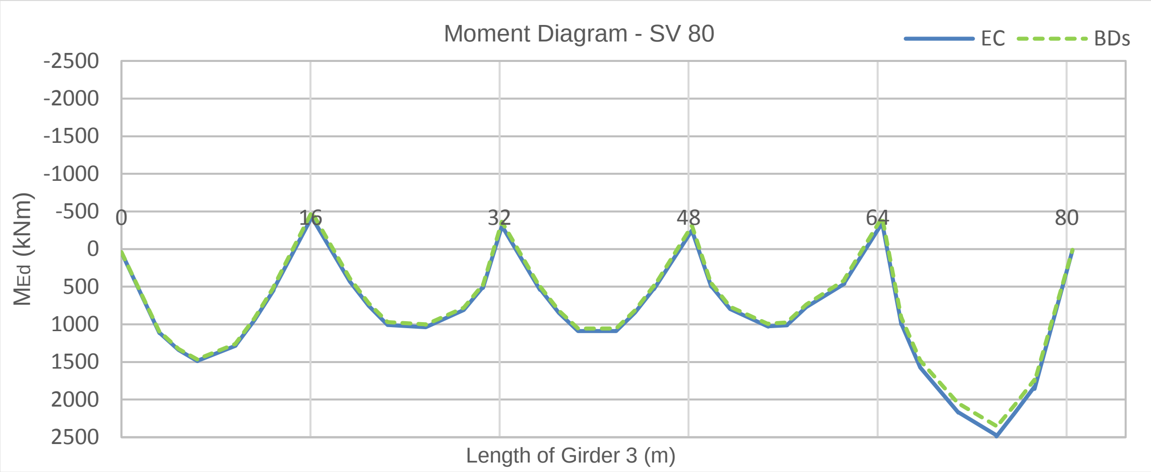


Figure 6.22 - Graphical illustration of bending moment results due to combinations 1b and 2b along Girder 3

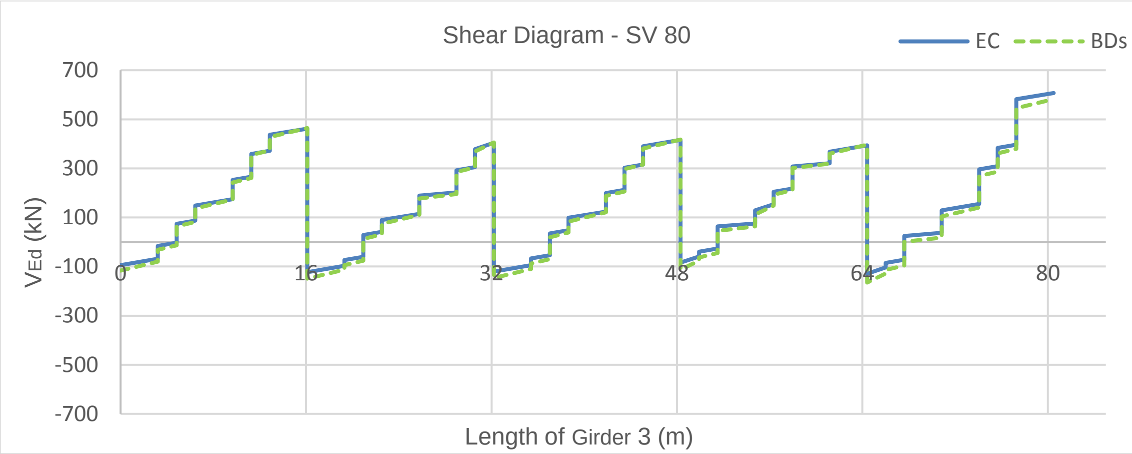


Figure 6.23 - Graphical illustration of shear force results due to combinations 1b and 2b along Girder 3

When looking at the results above, it is possible to observe that Eurocodes show higher values (2%) than BDs which did not happen in girder 1. In fact, the values are really close and the capacities of the elements calculated in chapter 4 usually make the difference.

The relative results of $M_{Ed}/|R|$ and $V_{Ed}/|R|$ can be seen in Figures 6.24 and 6.25.

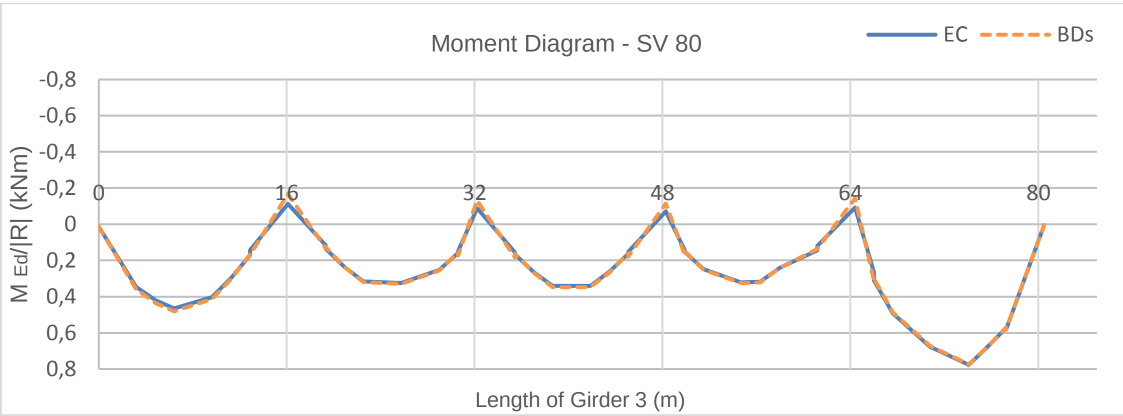


Figure 6.24 - Graphical illustration of $M_{Ed}/|R|$ of results due to combinations 1b and 2b along Girder 3

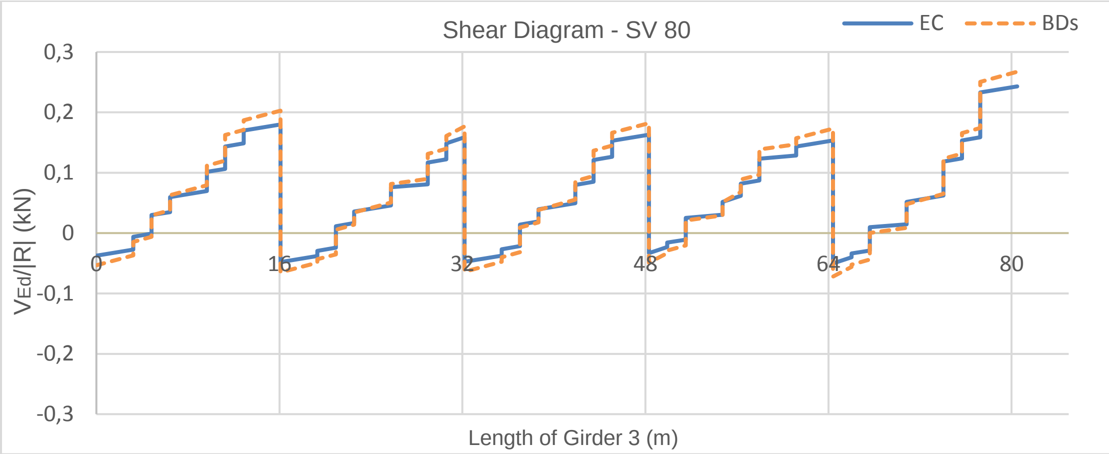


Figure 6.25 - Graphical illustration of $V_{Ed}/|R|$ of results due to combinations 1b and 2b along Girder 3

The last graphic related to $M_{Ed}/|R|$ (Figure 6.26) allow to understand that the results of the standards got closer and the results from BDs got 10% higher than the ones showed by the Eurocodes.

6.1.2.2 Results related to SV 196 and HB 45

At last, the comparison between the combinations 1f and 2f, having the special vehicle SV196 and HB 45 as the main variable, respectively, have also been carried out. The results are shown in Figures 6.26 and 6.27.

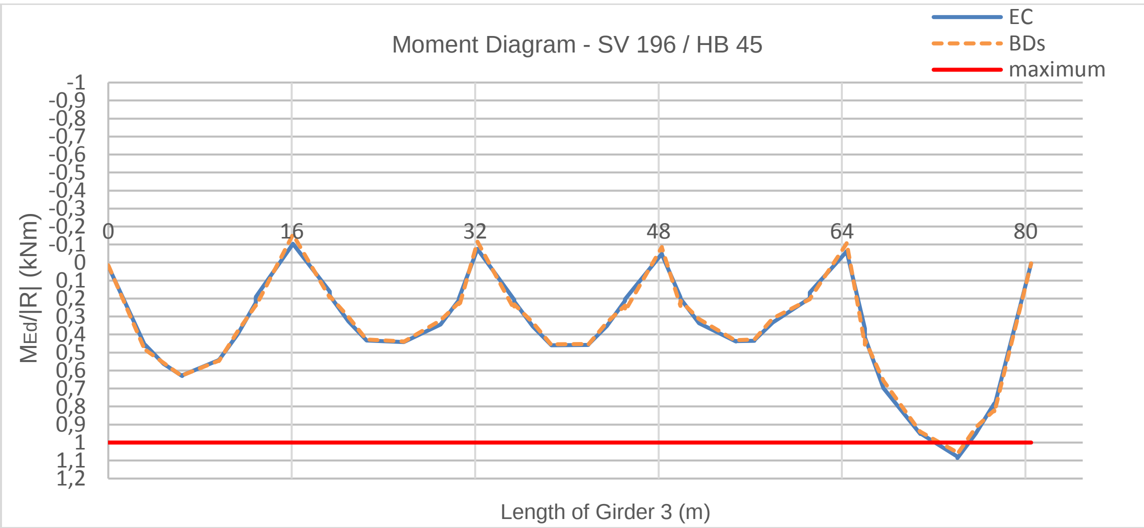


Figure 6.26 - Graphical illustration of $M_{Ed}/|R|$ of results due to combinations 1f and 2f along Girder 3

As it can be seen in Figure 6.26, the results of the effects of actions produced by combinations 1f and 2f are similar in both standards. However, the results shown in $M_{Ed}/|R|$ present higher values for the Eurocodes whilst the opposite happens for the results obtained in $V_{Ed}/|R|$ diagram.

Through this last graphical illustration, it is also possible to conclude that both combinations failed for structural assessment, which means that the effects produced by the actions exceeded the limit capacity of girder 3 against bending moment, by approximately 10%.

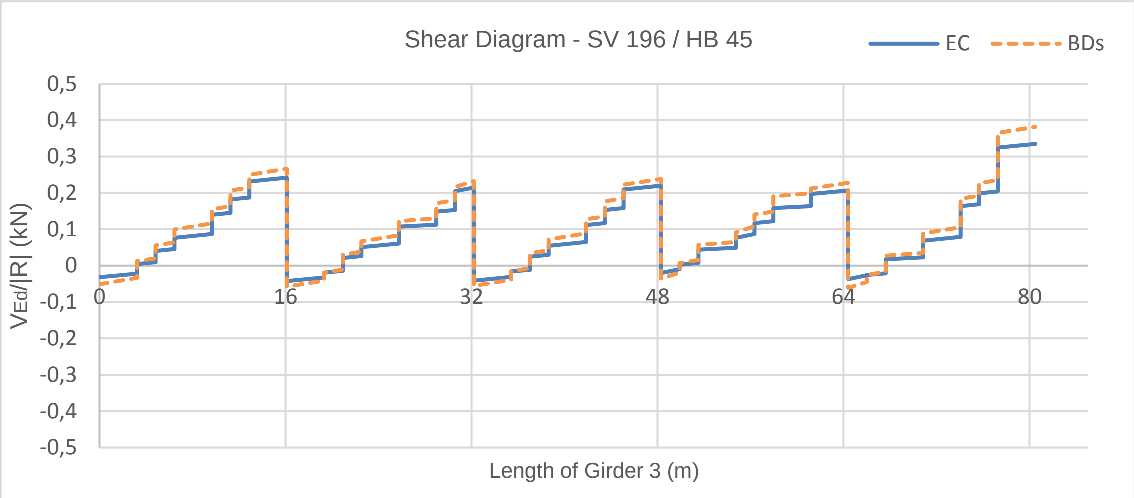


Figure 6.27 - Graphical illustration of $V_{Ed}/|R|$ of results due to combinations 1f and 2f along Girder 3

For a better analysis, Figures 6.28 related to the effects of bending moment produced by combinations 1f and 2f is shown below:

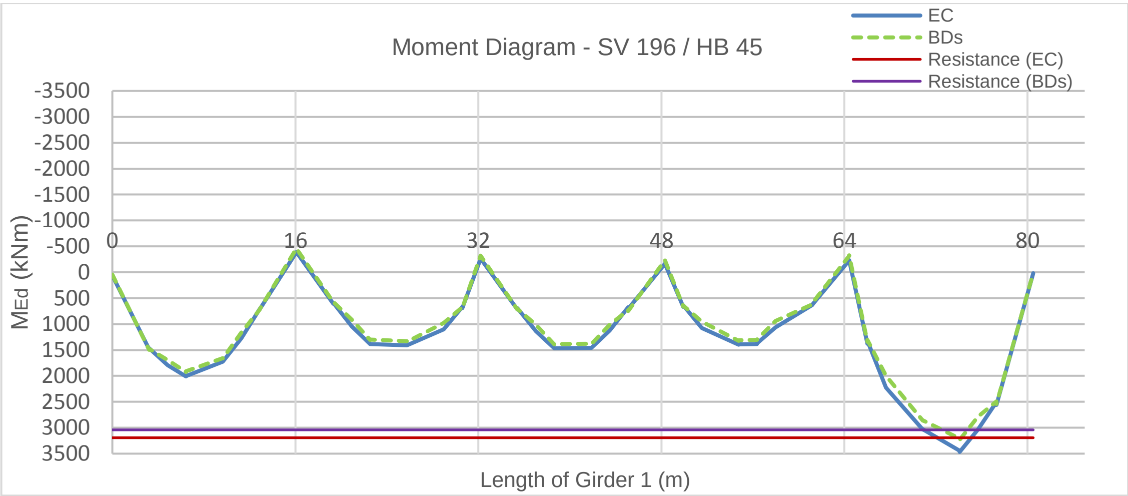


Figure 6.28 - Graphical illustration of bending moment results due to combinations 1f and 2f along Girder 3

From this comparison is possible to get the maximum capacity of girder 3 in span 5, against bending moment, obtained both through Eurocodes and BDs. It is also possible to extract the maximum effects verified in span 5 due to combinations 1f and 2f.

Therefore, Table 6.2 is based on those results mentioned above in order to understand the relative difference between the two standards in this extreme case.

Table 6.2 – Comparison of the relative results of $M_{Ed}/|R|$ between the standards

	Eurocodes	**BDs**
Maximum M (kNm)	3468	3234
M_{rd} or M_D (kNm)	3193	3041
$M_{Ed}/\lvert R\rvert$	1,09	1,06

Table 6.2 shows that results obtained from the Eurocodes present a higher exceedance of the capacity of girder 3 against bending moment than the results verified in BDs. The results obtained in girder 1 regarding the minimum effects verified in the structure can be consulted in Appendix E.

In order to work out this concern regarding the sagging capacity of girder 3 against bending, some valid solutions can be proposed. The first may be to restrict this road bridge to the transit of abnormal vehicles HB 45 and/or SV 196 since they represent exceptional weight that is not cover by the AW Regulations. Another option would be the strengthening of girder 3 with an additional steel plate attached to the bottom flange at mid-span.

Since structural safety is exceeded by only 10% approximately, in both standards, other options are also valid. They can be allowing the passage of just one vehicle at a time or even restrict the passage of pedestrians at the same time of the vehicle, since the results from Figure 6.28 are regarding the combination of the two (see Appendix A).

6.2 Output results from the columns

The analysis of the trestles has been carried out based on the capacity of the sections of the columns against compression and bending. Trestle 1, as indicated in Figure 5.8 has been chosen for the comparison of the effects of actions and resistances between standards, since it is the most onerous one.

According to section 4.5.2 and considering the capacity of sections BB and DD introduced in Table 5.1, the following graphical illustration (Figure 6.29) regarding cross-section BB was obtained for the comparison between the effects of actions and respective resistance against composite bending.

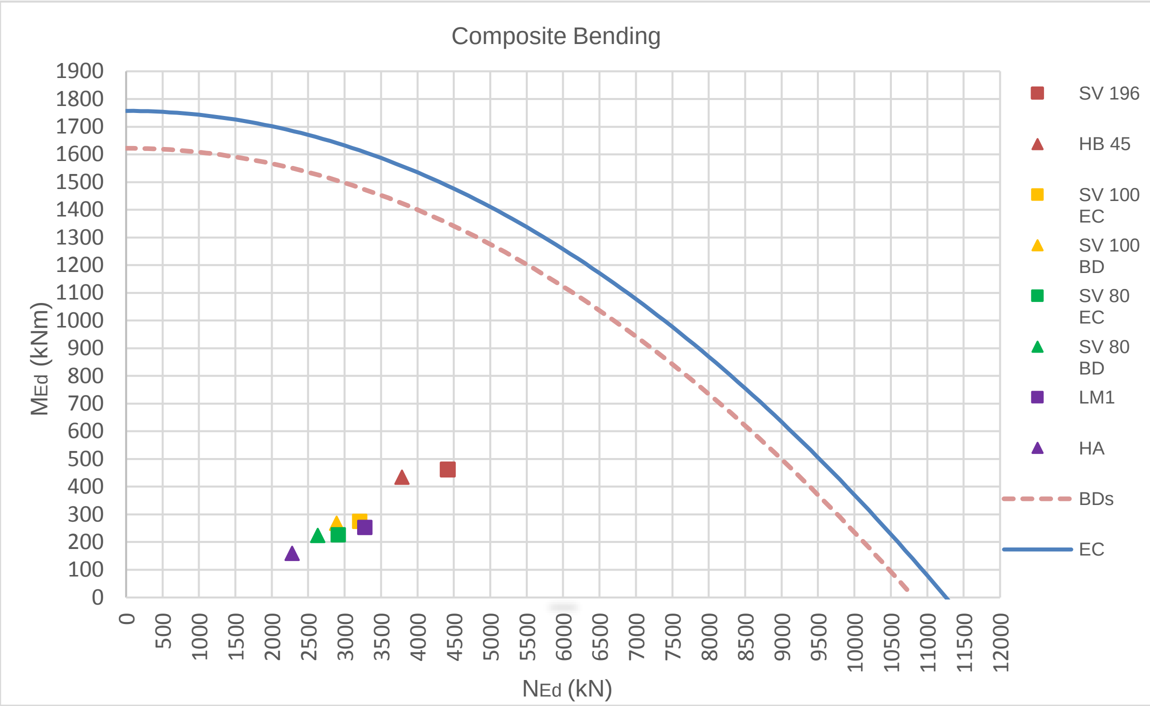


Figure 6.29 – Comparison between the maximum results obtained by the standards and its respective composite bending resistance in section BB from trestle 1

Considering the graphical illustration related to cross-section BB from trestle 1, it is possible to conclude that in general, the effects of actions from the Eurocodes are usually higher than the ones verified in BD standards, as well as their calculated resistances. As expected, the results shown by the combinations 1b, 2b, 1c and 2c, which have SV 80 and SV 100 as the main variable, are more similar than the results from the rest of the combinations.

Anyway, the effects of actions verified both in Eurocodes and BDs have shown relatively low results when compared to the capacity of cross-sections BB and hence, cross-section DD of the columns. Thus, the safety of the columns against composite bending has been verified for the combinations applied.

Chapter 7

7. Summary, Conclusions and Future Works

7.1 Summary and Conclusions

The motivation for this work rely mainly on understanding the existent differences between Eurocodes and BD standards in the application, requirements and results obtained with regards to structural assessment. It is intended to recognise the influence that the assessment methodology of an existing structure have in the maintenance and economy of bridges, using one of the standards mentioned. It is important to highlight the huge importance that this subject has nowadays due to the constant increase of traffic load on bridges and its corresponding response.

There are several differences in the evaluation of an existing bridge and the evaluation of a newly designed bridge such as the information of the bridge condition, which can give the actual certificates of material properties; real geometry; collection of load data; results of proof load testing; and the results of periodic inspections

The evaluation of existing bridges became one of the most important subjects for Bridge Management System (BMS) not only due to the maintenance of bridges as a response to the traffic loads growth but also to help in the management process and rehabilitation strategy of the existing bridges. Beyond that, it is also important to record any damage or deviation from the expected actual condition of the bridge under consideration in order to calculate the reliability of the bridge for remaining bridge lifetime [46].

Within the scope of this work, comparisons have been made regarding the existent actions models for each standard, as well as the procedures needed to calculate the resistance of structural materials. From this comparison, it is possible to notice the difference in the composition and application of traffic loads, such as the variety, number, intensity of load and axles spacing as it can be observed in chapter 4. When it comes to special vehicles the same principles are applied in both standards, except the fact that in BD standards, HB load can still be considered in structural assessment. Beyond that, the SV-TT and SV-train are also defined in the assessment standards since this type of vehicles had been used associated with British Standards, that is, before the implementation of the Eurocodes.

The assessment of Ashworth Road Viaduct has been carried out according to both standards, in order to comprehend the impact that the effects of different actions and its possible combinations have in the structure in general and in the structural elements in particular. It has been given special attention to the requirements of each standard related to notional lanes, distribution of load models and its partial safety factors when regards to their combinations.

In the calculation method for resistance of materials, lower bending resistance results in BDs have been registered, mainly due to partial safety factors regarding sections subjected to sagging moment. Considering sections under hogging effect, the process to determine the bending resistance of the cross-sections was not the same as it has been considered that the cross-sections, according to the Eurocodes, were able to mobilise the plastic moment. The same could not be verified in BD standards, since it has been considered class 2 in the Eurocodes and non-compact in the case of BDs.

When looking at the results obtained from girders 1 and 3 in chapter 6, it is possible to conclude that the values from both models do not have a high discrepancy with regard to bending moment and shear force. However, it has been verified in general, more conservative values for BDs due to the fact that these standards show lower values (10%) for the capacity of the structural members, being this a determining factor. Furthermore, it is also possible to notice that the effects of actions based on the Eurocodes show always higher values than the ones from BDs. A difference of 40% and 20% in bending and shear effects, respectively, has been specially noticed for regular vehicles.

In the analysis of trestle 1, the same can be concluded that Eurocodes show higher results for the effects of actions as well for the capacity of the elements. However, the results from the effects of actions are not great enough to cover the high values of resistance, turning BDs into more conservative standards.

Eurocodes suggest a greater resistance from the materials than the corresponding assessment standards which, in some way, can be useful in rejecting the need for unnecessary strengthening of the structure, trying however, not to compromise the safety of the structure.

The Eurocodes are based on more ample calculations methods such as non-linear analysis and recent tests which turns it into a more efficient standard regarding design of bridges. However, Eurocodes standards try to adapt, when considering assessment of existing bridges, being their rules outside of this subject and mainly focused in first principles. Therefore, this standard is focused on determining the true ultimate strength of a structure instead of giving a more empirical perspective. For instance, Eurocodes have not been designed to directly deal with a reduction in strength of a material due to deterioration of an unforeseen action [47].

7.2 Future Works

Despite the increasing concern regarding assessment of existing bridges, there is still a lack of dedication when it comes to standards applied in this specifically issue. Structural codes created purposely for design attempt to adapt for assessment in most countries.

Therefore, a case study has been carried out towards the major differences between the Eurocodes and the specific standards for structural assessment in the UK (BDs), in order to understand the effects and benefits each one can have directly in the maintenance and economy of structures or in their use by public in general.

As the presented analysis was focused on a composite bridge, a deeper study is suggested regarding the underlying issue herein developed, considering existing concrete and steel bridges in the UK. This will bring a wide range of results and hence, a more solid and credible conclusion with regards to the use of the standards under consideration in the assessment of existing bridges.

As a sequence to these analyses, it is also proposed to be explored the impact of the partial safety factors on the results towards further details regarding reliability index values adopted by each code.

This work can also serve as reference for the comparison between other standards made specifically for design of new bridges and the UK Standards for Structural Assessments. This can be used in order to understand the greater impact that they can have in the maintenance of existing bridges or even in its medium to long term failure.

A new approach could be put in place regarding the topics that could possibly be changed in the Eurocodes in order to provide a better application of the already existing principles to the assessment of old bridges. After comprehensive development, these could be integrated into the Eurocodes through National Annexes created specifically for the structural assessment of existing bridges.

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Appendix A

A. Combinations of Actions

Eurocodes:

Combination 1a: 1,35 x self-weight + 1,50 x (LM1 + 0,40 x pedestrian)

Combination 1b: 1,35 x self-weight + 1,5 x (SV 80 + 0,40 x pedestrian)

Combination 1c: 1,35 x self-weight + 1,5 x (SV 100 + 0,40 x pedestrian)

Combination 1d: 1,0 x self-weight + 1,0 x (Accidental-max bending)

Combination 1e: 1,0 x self-weight + 1,0 x (Accidental-max shear)

Combination 1f: 1,35 x self-weight + 1,5 x (SV 196 + 0,40 x pedestrian)

BDs:

Combination 2a: 1,1 x γ_{fl} x self-weight + 1,50 x 1,1 x (HA + pedestrian)

Combination 2b: 1,1 x γ_{fl} x self-weight + 1,21 x SV 80 + 1,65 x pedestrian

Combination 2c: 1,1 x γ_{fl} x self-weight + 1,21 x SV 100 + 1,65 x pedestrian

Combination 2d: 1,1 x γ_{fl} x self-weight + 1,50 x 1,1 x (Accidental-max bending)

Combination 2c: 1,1 x γ_{fl} x self-weight + 1,50 x 1,1 x (Accidental-max shear)

Combination 2f: 1,1 x γ_{fl} x self-weight + 1,50 x 1,1 x (HB 45 + pedestrian)

Appendix B

B. Dimensions of structural elements and material properties

Table B.1 - Section Characteristics of girders from span 1&5 (midsection)

Girders	D (mm)	w (mm)	t_w (mm)	t_{tf} (mm)	t_{bf} (mm)	t_{weld} (mm)	A (mm^2)
Girder 1	692,0	356,0	7,9	12,7	15,9	6,4	20663,3
Girder 2	740,0	356,0	7,9	12,7	15,9	6,4	21421,7
Girder 3	771,5	356,0	7,9	12,7	19,1	6,4	23016,0
Girder 4	838,2	356,0	7,9	12,7	19,1	6,4	23553,2
Girder 5	889,0	356,0	7,9	12,7	22,2	6,4	24864,6

Table.B.2 - Section Characteristics of girders from span 2,3&4 (midsection)

Girders	D (mm)	w (mm)	t_w (mm)	t_{tf} (mm)	t_{bf} (mm)	t_{weld} (mm)	A (mm^2)
Girder 1	695,3	356,0	7,9	12,7	19,1	6,4	21752,0
Girder 2	743,0	356,0	7,9	12,7	19,1	6,4	22510,4
Girder 3	771,0	356,0	7,9	12,7	19,1	6,4	23016,0
Girder 4	838,2	356,0	7,9	12,7	19,1	6,4	23553,2
Girder 5	885,8	356,0	7,9	12,7	19,1	6,4	24864,6

Table B.3 - Section Characteristics of girders from trestles 1&4

Girders	D (mm)	w (mm)	t_w (mm)	t_{tf} (mm)	t_{bf} (mm)	t_{weld} (mm)	A (mm^2)
Girder 1	718,0	356,0	7,9	25,4	28,6	6,4	29304,4
Girder 2	765,2	356,0	7,9	25,4	28,6	6,4	30062,8
Girder 3	793,8	356,0	7,9	25,4	28,6	6,4	30568,4
Girder 4	860,0	356,0	7,9	25,4	28,6	6,4	31105,6
Girder 5	920,8	356,0	7,9	31,8	34,9	6,4	36737,5

Table B.4 - Section Characteristics of girders from trestles 2&3

Girders	D (mm)	w (mm)	t_w (mm)	t_{tf} (mm)	t_{bf} (mm)	t_{weld} (mm)	A (mm^2)
Girder 1	708,0	356,0	7,9	22,2	22,2	6,4	26038,5
Girder 2	755,7	356,0	7,9	22,2	22,2	6,4	26796,9
Girder 3	787,4	356,0	7,9	22,2	25,4	6,4	28391,1
Girder 4	850,9	356,0	7,9	22,2	22,2	6,4	27839,7
Girder 5	905,0	356,0	7,9	22,2	28,6	6,4	31328,4

Table B.5 – General dimensions of the Trimmer

Cross beam section size 18''x 6'' 50lbs		
Depth of the beam	461,3	mm
Width of the beam	152,7	mm
Thickness of the flange	17,0	mm
Thickness of the web	9,9	mm
Approximate length of cross beam	1778,0	mm
Cross section area	9490,0	mm

Table B.6 – Dimensions and details of the concrete slab

Thickness of the slab (t_{slab})	191 mm
Diameter of primary bar (ϕ_p)	19 mm
Diameter of secondary bar (ϕ_s)	16 mm
Average primary spacing	178 mm
Average secondary spacing	216 mm
Average concrete cover	19 mm

Table B.7 - Lengths of the spans

Bridge spans		L (mm)
Straight span distance	$S_{\_G1\text{-}G5}$	16,0
Curvature span distance	$S_{\_G1}$	14,6
Curvature span distance	$S_{\_G2}$	15,3
Curvature span distance	$S_{\_G3}$	16,1
Curvature span distance	$S_{\_G4}$	16,9
Curvature span distance	$S_{\_G5}$	17,6

Table B.8 – Carriageway Details

Width of carriageway	6706	mm
Width of Footways	1778	mm
Average thickness of footway	281	mm
Thickness of parapet stand A,B	305	mm
Depth of parapet beam A	686	mm
Depth of parapet beam B	762	mm
Cross-sectional area of the footway	499618	mm
Cross fall of the deck	152	mm
Height of the kerb	102	mm

Table B.9 - Dimensions of the Trestles in mm

Trestle 1 (T1)	
Approximate left length to bottom of cross beam trestle	5461
Approximate right length to bottom of cross beam trestle	5547

Trestle 2 (T2)	
Approximate left length to bottom of cross beam trestle	4372
Approximate right length to bottom of cross beam trestle	4632

Trestle 3 (T3)	
Approximate left length to bottom of cross beam trestle	3175
Approximate right length to bottom of cross beam trestle	3620

Trestle 4 (T4)	
Approximate left length to bottom of cross beam trestle	2092
Approximate right length to bottom of cross beam trestle	2651

Table B.10 – Unit Weight of the materials (kg/m^3)

concrete (wet)	25,0
concrete (dry)	24,0
surfacing	23,0
gravel (wet)	21,0
structural steel	78,5

Table B.11 - Steel properties according to BDs (left) and the Eurocodes (right)

Elastic modulus (E)	205 GPa
Shear Modulus (G)	81 GPa
Yield strength (t<16)	355 MPa
Yield strength (16<t<25)	345 MPa
Yield strength 25<t<40	345 MPa

Elastic modulus (E)	210 GPa
Shear Modulus (G)	81 GPa
Characteristic Yield strength	355 MPa

Note: From the drawing b2 in Annex B, it is possible to obtain the grade of the girders, 50 B. Thus, and with the help of the table 3.2 from section 3.3.2.3, the given yield stress is shown in table B11.

Table B.12 - Concrete Properties according to BDs (left) and the Eurocodes (right)

Compressive strength (f_{cu})	31 MPa
Short term Elastic Modulus (E_{lt})	28,4 GPa
Long term Elastic Modulus (E_{lt})	14,2 GPa
Poisson coefficient (v)	0,2

Compressive strength (f_{cu})	31 MPa
Elastic Modulus (E_{lt})	33 GPa
Poisson coefficient (v)	0,2

Appendix C

C. Intermediate output of structural elements’ properties

i. Cross-section of girders (steel only)

Table C.1 – Section properties of girders from spans 1&5 (midsection)

Girders	I_{xx} (mm^4)	I_{yy} (mm^4)	r_{xx} (mm)	r_{yy} (mm)	Z_{pe} (mm^3)	Z_{xc} (mm^3)	Z_{xt} (mm^3)	Y_p (mm)	Y_e (mm)
Girder 1	1,5E+09	4,3E+08	274	143	5,2E+06	4,3E+06	4,7E+06	312	329
Girder 2	1,8E+09	4,5E+08	290	145	5,7E+06	4,7E+06	5,1E+06	336	352
Girder 3	2,1E+09	4,7E+08	303	144	6,4E+06	5,0E+06	6,0E+06	317	351
Girder 4	2,3E+09	4,9E+08	327	145	6,8E+06	5,3E+06	6,4E+06	334	367
Girder 5	2,9E+09	5,3E+08	344	146	7,8E+06	6,1E+06	7,2E+06	376	407

Table C.2 - Section properties of girders from spans 2,3&4 (midsection)

Girders	I_{xx} (mm^4)	I_{yy} (mm^4)	r_{xx} (mm)	r_{yy} (mm)	Z_{pe} (mm^3)	Z_{xc} (mm^3)	Z_{xt} (mm^3)	Y_p (mm)	Y_e (mm)
Girder 1	1,6E+09	4,4E+08	275	142	5,5E+06	4,3E+06	5,3E+06	277,1	313,0
Girder 2	1,9E+09	4,6E+08	292	143	6,0E+06	4,8E+06	5,7E+06	301,1	335,8
Girder 3	2,1E+09	4,7E+08	304	143	6,4E+06	5,0E+06	6,0E+06	317,1	351,0
Girder 4	2,3E+09	4,9E+08	316	144	6,8E+06	5,3E+06	6,4E+06	334,1	367,2
Girder 5	2,9E+09	5,3E+08	344	146	7,8E+06	6,1E+06	7,2E+06	375,6	407,0

Table C.3 - Section properties of girders from trestles 1&4

Girders	I_{xx} (mm^4)	I_{yy} (mm^4)	r_{xx} (mm)	r_{yy} (mm)	Z_{pe} (mm^3)	Z_{xc} (mm^3)	Z_{xt} (mm^3)	Y_p (mm)	Y_e (mm)
Girder 1	2,5E+09	5,1E+08	290	132	8,0E+06	6,9E+06	7,4E+06	312	334
Girder 2	2,9E+09	5,3E+08	309	133	8,7E+06	7,5E+06	8,0E+06	336	358
Girder 3	3,2E+09	5,5E+08	321	134	9,2E+06	7,9E+06	8,4E+06	352	373
Girder 4	3,5E+09	5,6E+08	334	135	9,7E+06	8,3E+06	8,9E+06	369	390
Girder 5	5,1E+09	6,4E+08	372	132	1,3E+07	1,1E+07	1,2E+07	411	433

Table C.4 - Section properties of girders from trestles 2&3

Girders	I_{xx} (mm^4)	I_{yy} (mm^4)	r_{xx} (mm)	r_{yy} (mm)	Z_{pe} (mm^3)	Z_{xc} (mm^3)	Z_{xt} (mm^3)	Y_p (mm)	Y_e (mm)
Girder 1	2,1E+09	4,8E+08	286	135	7,0E+06	6,2E+06	6,2E+06	346	346
Girder 2	2,5E+09	5,0E+08	304	137	7,6E+06	6,7E+06	6,7E+06	370	370
Girder 3	2,9E+09	5,3E+08	318	136	8,4E+06	7,2E+06	7,7E+06	352	372
Girder 4	3,0E+09	5,3E+08	329	138	8,5E+06	7,5E+06	7,5E+06	403	403
Girder 5	4,1E+09	5,9E+08	363	137	1,1E+07	8,7E+06	9,9E+06	376	415

ii. Cross-section of composite girders

Table C.5 – Transformed section properties of girders from spans 1&5 (midsection)

Girders	I_{xx} (mm4)	I_{yy} (mm^4)	r_{xx} (mm)	r_{yy} (mm)	Z_{xc} (mm^3)	Z_{xt} (mm^3)	Y_p (mm)
Girder 1	4,8E+09	1,6E+10	246	453	2,3E+07	7,4E+06	668
Girder 2	5,6E+09	2,2E+10	269	529	2,4E+07	8,0E+06	701
Girder 3	6,6E+09	2,2E+10	290	524	2,7E+07	9,3E+06	717
Girder 4	7,2E+09	2,2E+10	302	523	2,9E+07	9,7E+06	743
Girder 5	8,7E+09	1,6E+10	338	466	3,0E+07	1,1E+07	794

Table C.6 – Transformed section properties of girders from spans 2,3&4 (midsection)

Girders	I_{xx} (mm^4)	I_{yy} (mm^4)	r_{xx} (mm)	r_{yy} (mm)	Z_{xc} (mm^3)	Z_{xt} (mm^3)	Y_p (mm)
Girder 1	5,22E+09	1,63E+10	268,4	474,7	2,20E+07	8,10E+06	645
Girder 2	6,09E+09	2,17E+10	278,6	525,9	2,56E+07	8,81E+06	692
Girder 3	6,63E+09	2,17E+10	289,7	524,4	2,71E+07	9,26E+06	717
Girder 4	7,24E+09	2,18E+10	301,6	522,8	2,86E+07	9,74E+06	743
Girder 5	8,66E+09	1,64E+10	338,5	466,4	3,04E+07	1,09E+07	794

Table C.7 - Combined section properties of girders from trestles 1&4

Girders	I_{xx} (mm^4)	I_{yy} (mm^4)	r_{xx} (mm)	r_{yy} (mm)	Z_{xt} (mm^3)	Z_{xc} (mm^3)	Y_e (mm)	Z_{pe} (mm^3)	Y_p (mm)
Girder 1	3,4E+09	2,3E+09	315	259	8,5E+06	7,5E+06	403	9,5E+06	424
Girder 2	3,9E+09	2,3E+09	334	257	9,1E+06	8,1E+06	428	1,0E+07	448
Girder 3	4,2E+09	2,3E+09	343	256	9,5E+06	9,0E+06	442	1,1E+07	464
Girder 4	4,5E+09	2,4E+09	355	255	9,9E+06	9,6E+06	459	1,1E+07	481
Girder 5	6,4E+09	2,4E+09	392	241	1,3E+07	1,2E+07	499	1,5E+07	523

Table C.8 – Combined section properties of girders from trestles 2&3

Girders	I_{xx} (mm^4)	I_{yy} (mm^4)	r_{xx} (mm)	r_{yy} (mm)	Z_{xt} (mm^3)	Z_{xc} (mm^3)	Y_e (mm)	Z_{pe} (mm^3)	Y_p (mm)
Girder 1	3,0E+09	2,3E+09	312	270	7,2E+06	6,9E+06	420	8,4E+06	458
Girder 2	3,5E+09	2,3E+09	330	268	7,8E+06	7,5E+06	446	9,1E+06	482
Girder 3	3,9E+09	2,3E+09	341	263	8,7E+06	8,4E+06	446	1,0E+07	464
Girder 4	4,0E+09	2,3E+09	349	266	8,4E+06	8,8E+06	477	1,0E+07	515
Girder 5	5,5E+09	2,4E+09	389	256	1,1E+07	1,0E+07	494	1,2E+07	488

iii. Cross-sections of trestles

Table C.9 –Properties of cross-section AA

	I_{xx} (mm^4)	I_{yy} (mm^4)	r_{xx} (mm	r_{yy} (mm)	Z_{pe} (mm^3)	Z_{xc} (mm^3)	Z_{xt} (mm^3)	Y_p (mm)	Y_e (mm)
AA	4,4E+09	1,4E+09	344	192	1,1E+07	9,6E+06	9,6E+06	457	457

Table C.10 - Properties of cross-sections BB and DD

	I_{xx} (mm^4)	I_{yy}(mm^4)	r_{xx} (mm)	r_{yy} (mm)	$Z_{pe,xx}$ (mm^3)	$Z_{pe,yy}$ (mm^3)	Z_{xc} (mm^3)	Z_{xt} (mm^3)	Y_p (mm)	Y_e (mm)
DD	9,1E+08	1,2E+09	165	191	4,9E+06	6,1E+06	5,4E+06	5,4E+06	229	229
BB	8,3E+08	9,9E+08	172	188	4,4E+06	4,9E+06	4,3E+06	4,3E+06	229	229

Appendix D

D. Intermediate calculations and results for the capacity output of structural members

i. Calculations according to the Eurocodes - Example of Girder 1

According to Figure AA.5 from Annex A

	web			Compression flange			class
Girder 1	c	c/t	class	c	c/t	class	
Spans 1&5	650,6	82,4	class 3	340,2	26,8	class 1	class 3
Spans 2,3&4	650,7	82,4	class 3	340,2	26,8	class 1	class 3
Trestles 1&4	651,2	82,4	class 3	340,2	11,9	class 1	class 3
Trestles 2&3	650,8	82,4	class 3	340,2	15,3	class 1	class 3

The area of hogging sections has been reduced, whereby:

Girder 1	A (mm^2)	$20\varepsilon t_w$	A_{negl} (mm^2)	A_{eff} (mm^2)	$Z_{pe,\ eff}$
Trestle 1&4	34330,9	128,0	2648,2	31682,7	9,3E+06
Trestle 2&3	31823,4	128,0	3280,2	28543,2	1,1E+07

<u>Allowable space of connectors (Classification of composite compression flanges)</u>

$$22 \times t_f \sqrt{235/f_y} = 227$$

Maximum longitudinal spacing: 381 mm, it fails, however the flange is already class 1.

a) <u>Shear and Torsion Resistance of box girder 1</u>

Spans 1,2,3,4&5 (Mid-section)

Girder 1	Combination	Bending M (max)	Shear V (max)
Spans 1&5	C1f: SW + SV196 +ped	1417,4	345,7
Spans 2,3&4	C1f: SW + SV196 +ped	668,5	239,8

h_{con}	=	191	mm	b_{eff}	310	mm
L (G1 &G5)	=	2235	mm			
L (in Girders)	=	2134	mm			

Girder 1	V_{Ed} (kN)	I_{xx} (mm^4)	t (mm)	$\bar{y}$	D-$\bar{y}$	S	τ (Mpa)	Checking
Spans 1&5	345,7	4,8E+09	15,8	667,6	24,4	7,2E+06	32,8	check
Spans 2,3&4	239,8	5,2E+09	15,8	644,6	50,7	8,9E+06	25,8	check

$S = A \times z'$

Rolled rectangular hollow sections

$A_v = \eta \sum h_w t_w$

Girder 1	A (mm²)	h (m)	b (m)	A_v (mm²)	$V_{pl,rd}$ (kN)	γ_{M0}	1,0
						η	1,0
Trestles 1&4	29304,4	718,0	356,0	11344,4	2325,1	tw	7,9
Trestles 2&3	26038,5	708,0	356,0	11186,4	2292,8	f_y	355

Transverse stiffeners

η	1,2
ε	0,81

$I_{yy} = I_{sl}$	2,20E+07
f_{yw}	355

tstiff	25,4	mm
bstiff	340,2	mm
Npanel	21	

a = L/Npanel

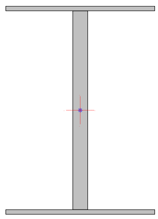


Figure D.1 – Intermediate transverse stiffener

The checking is done according with equation (4.23)

Girder 1	h_w	L	a	a/h_w	$\kappa\tau_{sl}$	$\kappa\tau$	h/t_w		Assessment
Midsection	663,4	14557	693,2	1,01	14,0	23,0	44	100,3	check

Girder 1	h_w	L	α	a	$\kappa\tau$	h/t_w		Assessment
Trestles	664,0	14557	1,01	693,2	13,3	45	76,2	check

Girder 1	Combination	Torsion (max)
Trestles	CBf1 : SW + SV196	233,5

$$I_t = \frac{4A^2}{\int_s ds/t}; \ \tau_{,Ed} = \frac{T \times t}{I_t}$$

Girder 1	T_{ed} (kNm)	I_t	t (mm)	$\tau_{t,Ed}$ (Mpa)	$V_{pl,T,Rd}$ (kN)
Trestles	233,5	1,9E+07	7,9	95,6	1240,1

ii. Caculations according to the Eurocodes – Example of Trestle 1

	web			Compression flange			class
Cross-section	c	c/t	class	c	c/t	class	
AA	869,4	68,5	class 3	431,6	27,1	class 1	class 3

		flanges			web			class
Sections	α	c	c/t	class	c	c/t	class	
Section D-D	1,0	419,2	16,5	class 1	394,4	30,3	class 2	class 2
Section B-B	1,0	419,2	22,0	class 1	407,1	31,3	class 2	class 2

a. Axial Capacity

Sections	A (mm^2)	A_v (mm^2)	$N_{c,Rd}$ (kN)	$M_{pl,Rd\text{-}xx}$ (kNm)	$M_{pl,Rd\text{-}yy}$ (kNm)	$V_{pl,Rd}$ (kN)	0,5 x $V_{pl,Rd}$ (kN)
DD	33533	11887	11904	1755	2150	2436	1218
BB	28095	11887	9974	1547	1752	2436	1218

b. Bending and Axial Force

Sections	N_{ed}	$N_{pl,Rd}$	1st condition	2nd condition	Test
Section D-D	2246,6	11904,2	2976,1	1055,0	fail
Section B-B	2246,6	9973,7	2493,4	1055,0	fail

$n = N_{Ed} / N_{pl.Rd}$ $\qquad$ $a = (A\text{-}2bt_f)/A$ but $a \leq 0{,}5$

Sections	a	n	$M_{pl,Rd\text{-}yy}$ (kNm)	$M_{N,y,Rd}$ (kNm)
Section D-D	0,31	0,2	2150,5	2106,8
Section B-B	0,38	0,2	1752,3	1684,6

c. Buckling Resistance

$$N_{cr} = \frac{\pi^2 EI}{L^2}$$

E	205	Gpa
Lcr D	5546,7	m
Lcr B	5546,7	m
Ncr	80511	kN
Iyy	1,2E+09	mm^4

Sections	A (mm^2)	Lcr (m)	I (m)	$\bar{\lambda}$	Φ	χ	$N_{b,rd}$ (kN)
Section D-D	33533,1	5546,7	190	0,38	0,59	0,96	11398,9
Section B-B	28094,9	5546,7	190	0,39	0,59	0,96	9534,2

iii. Calculations according to BDs- Example of Girder 1

Girder 1 at SPANS 1&5

BS 5400-3

Shape Limitations - classification of box girder

Cl 9.3.7.2

- *Classification of Web*

The depth of the web should not exceed:

Equation 1 : $\frac{34t_w}{m}\sqrt{\frac{355}{\sigma_{yw}}}$ when m does not exceed 0.5; and

Equation 2 : $\frac{374t_w}{(13m-1)}\sqrt{\frac{355}{\sigma_{yw}}}$ when m exceeds 0.5;

where

m is the ratio of the depth of the web plate which is on the compressive side of the plastic neutral axis of the beam to the depth of the web plate;

t_w is the thickness of the web plate;

σ_{yw} is the nominal yield stress of the web material as defined in **6.2**.

D_{wp} is depth of the web plate on the compressive side of the plastic neutral axis of the beam

D_w is depth of the web plate (clear of welds)

$D_w = d_{G1} - 2w_t = 692 - 2 \times 6{,}4 = 679$

$D_{wp} = d_{G1} - Y_p = 692 - 312 = 381$

$m = D_{wp}/D_w = 381 / 679 = 0{,}56$

Note : m exceeds 0.5, thus use equation 2 above

Equation 2 : $\frac{374t_w}{(13m-1)}\sqrt{\frac{355}{\sigma_{yw}}} = \frac{374 \times 7{,}9}{((13 \times 0{,}56)-1)}$ x (355 /355)^1/ = 472,6

Note : Depth of the existing web does not exceeds the obtained classification of the Web. Thus, Web is compact

BS 5400-3
Cl 9.3.7.3.2

- *Classification of flanges*

The clear width of the compression flange should not exceed:

tf,top = 12,7 mm

Qyf,50 = *355* N/mm2

Dwg 2110/48/A

bfc = *B - (2 x tw)* = 356 - (2 x 7,9) = 339,725 mm

$24t_f\sqrt{\frac{355}{\sigma_{yf}}}$ = 12,7 x (355 /355)^1/2 = 304,8 mm

Note : The clear width of the compression flange exceeds the obtained classification of the flange width. Thus, Compression flange is not compact

BS 5400-3
Cl 9.3.7.3.3

- *Classification of composite compression flanges*

In composite compression flanges, where the flange plate is connected to the concrete by shear connectors in accordance with BS 5400-5, the limits of 9.3.7.3.1 or 9.3.7.3.2 may be exceeded, provided that the spacing of the shear connectors perpendicular to the direction of compression does not exceed:

Max longitudinal spacing of shear connector S_{sc_Long} 381 mm

Transverse spacing of shear connector S_{sc_trans} 140 mm

Allowable transverse shear connector spacing ($S_{sc\_trans\_all}$)

$S_{SC\_trans\_all}$ = $30t_f\sqrt{\frac{355}{\sigma_{yf}}}$ = 30 x 12,7 x (355 /355)^1/2

$S_{SC\_trans\_all}$ = 381 mm

Allowable longitudinal shear connector spacing ($S_{SC\_long\_all}$)

$S_{SC\_long\_all}$ = $15t_f\sqrt{\frac{355}{\sigma_{yf}}}$ = 15 x 12,7 x (355 /355)^1/2

$S_{SC\_long\_all}$ = 191 mm

Or

$S_{SC\_long\_all}$ = $22t_f\sqrt{\frac{355}{\sigma_{yf}}}$ = 22 x 12,7 x (355 /355)^1/2

$S_{SC\_tras\_all}$ = 254 mm

Thus, Max longitudinal spacing of shear connector exceeds Allowable transverse shear connector spacing . Thus, Compression flange is not compact

BS 5400-3 Cl 9.9.1 — ***Composite bending resistance of the box girder***

Cl 9.9.1.2 — The bending resistance M_D of a beam **(non-compact section)** will be determined as following :

Cl 9.8 — - *Limiting moment of resistance*

The limiting moment of resistance, M_R, should be obtained from Figure 11a) for beams fabricated by welding (excluding local welding of stiffeners) or Figure 11b) for all other sections (including stress relieved weldedsections) according to the value of:

$$\lambda_{LT}\sqrt{\left(\frac{\sigma_{yc}}{355}\right)\left(\frac{M_{ult}}{M_{pe}}\right)}$$

BS 5400-3 Cl 9.7.3 Cl 9.7.3.1 — - *Slenderness (λ_{LT})* for other uniform sections : Uniform rectangular or trapezoidal box sections

Note: At this stage, the compression flange is fully restrained by the deck slab and the effective length *(l_e)* for lateral torsional buckling

l_e = 0 mm

λ_{LT} = $2.25\eta\xi\sqrt{\frac{Z_{pe}\ell_e}{r_y\sqrt{AJ}}}$ = 0

BS 5400-3 Cl 9.7.1

Characteristic yield strength (compression)	δ_{y_c}	355	N/mm^2
Characteristic yield strength (tension)	δ_{y_t}	345	N/mm^2
Transformed section modulus X-X top	Z_{xc}	23021300	mm^3
Transformed section modulus X-X Bottom	Z_{xt}	7393000	mm^3
Plastic section modulus X-X Bottom	Z_{pe}	9160872	mm^3

Elastic Bending Resistance (M_{ult})

$M_{ult,com}$ = Z_{xc} x δ_{yc} =

$M_{ult,com}$ = 8172,6 kNm

$M_{ult,ten}$ = Z_{xt} x δ_{yt} =

$M_{ult,ten}$ = 2550,6 kNm

Therefore, M_{ult} = Minimum of $M_{ult,com}$ & $M_{ult,ten}$

M_{ult} = 2550,6 kNm

Calculations

Project				Part of structure/scheme and status		
Kirklees Assessment 2015-16				Ashworth Road Viaduct Assessment		
Office of Issue	Telephone No		Division	Calculations by		Date
Manchester	0616 8838 6069		100000	MM		
Code Ref	Calculations	Intermediate calculations according to BDs (Girder 1)				

Plastic Bending Resistance (M_{pe})

M_{pe} = $Z_{pe} \times \delta_{yc}$ =

Therefore, M_{pe} = 3160,5 kNm

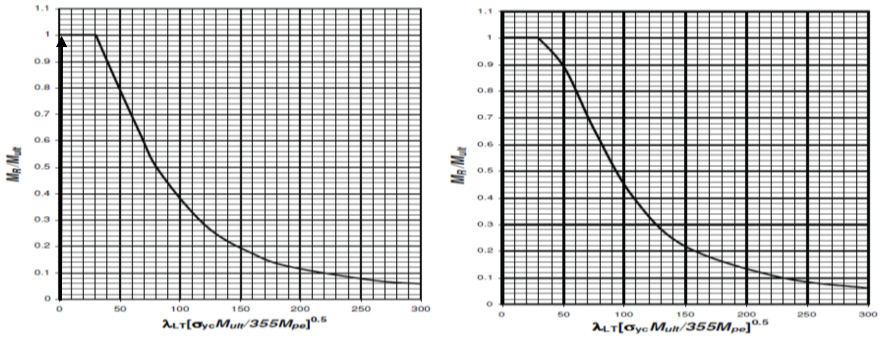
Cl 9.8

Thus; *Limiting moment of resistance* = 0 x [(355 / 355) x (2550,6 / 3160,5)] ^ 0.5

$$\lambda_{LT}\sqrt{\left(\frac{\sigma_{yc}}{355}\right)\left(\frac{M_{ult}}{M_{pe}}\right)} = 0{,}0$$

Delete the definitions of ℓ_w, ℓ_e and L.

Replace Figures 11a) and 11b) by those given below (see expressions for curves in **Annex G.8**)



a) Beams fabricated by welding b) All other sections

Figure 11: Limiting moment of resistance M_R

M_R/M_{ult} = 1,0

M_R = M_{ult} x 1.0 = 2550,6 x 1

M_R = 2550,6 kNm

Cl 9.9.1

Therefore ; *Bending resistance (M_D)*

$$M_D = \frac{M_R}{\gamma_{f3}\gamma_{m,steel}} \qquad \frac{2550{,}585}{1 \times 1{,}05}$$

M_D = 2429,1 kNm

BS 5400-3
cl 9.9.2
cl 9.9.2.2

Shear resistance (V_D)

Yield stress of the web	σ_{yw}	=	355
Yield stress of the flange	σ_{yf}	=	355
Total web thickness	t_w	=	7,9
Approximate depth of the box girder	d_{G1}	=	692
Width of the box girder	b	=	356
Thickness of the top flange	$t_{f\ top}$	=	12,7
Thickness of the bottom flange	$t_{f\ bot}$	=	15,9
Depth of web for shear	$d_{we} = d_G - t_{f,bottom} - t_{f,top}$	=	663,6
Slenderness ratio	$\lambda = (d_{we} / t_w) \times \sqrt{(\sigma_{yw} / 355)}$	=	84
	$\tau_y = \sigma_{yw} / \sqrt{(3)}$	=	205,0
Smaller thickness of the flange plate	t_f	=	12,7
Flange plate width	PT_f	=	
Yield stress of the flange	$\sigma_{yt} = \sigma_{yw}$	=	355
There are no holes or cut outs in the web section h_h		=	0
Clear length of panel between stiffeners	$a = L / N0_{\_trastiff}$	=	693
Aspect ratio of the web panel	$\varphi = a / d_{we}$	=	1,0

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Code Ref	Calculations	Intermediate calculations according to BDs (Girder 1)

b_{fe}

a) $10t_f \sqrt{355/\sigma_{yt}}$ = 127

b) The distance from the mid-plane of the web to the nearer edge of the flange = 174

c) half the clear distance between webs = 174

Thus, b_{fe} = 127

$m_{fw} = \dfrac{\sigma_{yf} b_{fe} t_f^2}{2\sigma_{yw} d_{we}{}^2 t_w}$ = 0,003

Therefore, from figure 5.13 determine Limiting shear strenght using the obtained m_{fw}

τ_l / τ_y = 0,95

$\tau_l = \tau_y \times 0.95$

$\tau_l = 205 \times 0,95$

$\tau_l = 194,7$

$V_D = \left[\dfrac{t_w(d_w - h_h)}{\gamma_m \gamma_{f3}}\right]\tau_\ell$

V_D = [7,9 x (663,6 - 0) / (1 x 1,05)]194,7

V_D = 976,7 kN

Girder 1 at TRESTLES 1&4

BS 5400-3

10.11.1 Shape Limitations - classification of box girder

Cl 9.3.7.2

- *Classification of Web*

The depth of the web should not exceed:

Equation 1 : $\dfrac{34t_w}{m}\sqrt{\dfrac{355}{\sigma_{yw}}}$ when m does not exceed 0.5; and

Equation 2 : $\dfrac{374t_w}{(13m-1)}\sqrt{\dfrac{355}{\sigma_{yw}}}$ when m exceeds 0.5;

where

m is the ratio of the depth of the web plate which is on the compressive side of the plastic neutral axis of the beam to the depth of the web plate;

t_w is the thickness of the web plate;

σ_{yw} is the nominal yield stress of the web material as defined in **6.2**.

$D_w = d_{G1} - 2w_t$ = 718 - 2 x 6 = 704,85 mm

$D_{wp} = d_{G1} - Y_p$ = 718 - 312 = 406,001 mm

$m = D_{wp}/D_w$ = 406 / 705 = 0,57601

Note : m exceeds 0.5, thus use equation 2 above

Equation 2 : $\dfrac{374t_w}{(13m-1)}\sqrt{\dfrac{355}{\sigma_{yw}}}$ = 374 x 7,9 / ((13 x 0,58) -1) x (355 /355)^1/2 = 457,5 mm

Note : Depth of the existing web exceeds the obtained classification of the Web. Thus, Web is not compact

BS 5400-3
Cl 9.3.7.3.2

- *Classification of flanges*

The clear width of the compression flange should not exceed:

$t_{f,top}$ = 25,4 mm

dwg 2110/48/A

$Q_{yf,50}$ = 345 N/mm2

b_{fc} = B - (2 x tw) 356 - (2 x 7,9) = 340 mm

$24t_f\sqrt{\dfrac{355}{\sigma_{yf}}}$ = 24 x 25,4 x (355 /345)^1/2 = 618 mm

Note : The clear width of the compression flange exceeds the obtained classification of the flange width. Thus, Compression flange is not compact

Calculations

mouchel

Project		Part of structure/scheme and status	
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Office of Issue	Telephone No	Division	Calculations by	Date
Manchester	0616 8838 6069	100000	MM	
Code Ref	Calculations	Intermediate calculations according to BDs (Girder 1)		

BS 5400-3
Cl 9.3.7.3.3

- *Classification of composite compression flanges*

Max longitudinal spacing of shear connector S_{sc_Long} = 229 mm
Transverse spacing of shear connector S_{sc_trans} = 140 mm

Allowable transverse shear connector spacing ($S_{SC\_trans\_all}$)

$S_{SC\_trans\_all}$ = $30t_f\sqrt{\frac{355}{\sigma_{yf}}}$ = 30 x 25,4 x (355 /345)^1/2

$S_{SC\_trans\_all}$ = 773,0 mm

Thus, Max Transverse spacing of shear connector does not exceeds Allowable transverse shear connector spacing. Thus, Compression flange is compact

Allowable longitudinal shear connector spacing ($S_{SC\_long\_all}$)

$S_{SC\_long\_all}$ = $15t_f\sqrt{\frac{355}{\sigma_{yf}}}$ = 15 x 25,4 x (355 /345)^1/2

$S_{SC\_long\_all}$ = 386 mm

Or

$S_{SC\_long\_all}$ = $22t_f\sqrt{\frac{355}{\sigma_{yf}}}$ = 22 x 25,4 x (355 /345)^1/2

$S_{SC\_tras\_all}$ = 515,3 mm

Thus, Max longitudinal spacing of shear connector does not exceeds Allowable transverse shear connector spacing. Thus, Compression flange is compact

BS 5400-3
Cl 9.9.1
Cl 9.9.1.2

10.11.2 Composite bending resistance of the box girder

The bending resistance M_D of a beam **(non-compact section)** will be determined as following :

δ_{y_c} = 345 N/mm^2
δ_{y_t} = 345 N/mm^2
Z_{xc} = 8482637,8 mm^3
Z_{xt} = 7500900 mm^3
Z_{pe} = 9518024,1 mm^3

BS 5400-3
Cl 9.7.1

Elastic Bending Resistance (M_{ult})

$M_{ult,com}$ = $Z_{xc} \times \delta_{yc}$ = 8482637,8 x 345
$M_{ult,com}$ = 2926,5 kNm
$M_{ult,ten}$ = $Z_{xt} \times \delta_{yt}$ = 7500900 x 345
$M_{ult,ten}$ = 2587,8 kNm
Therefore, M_{ult} = Minimum of $M_{ult,com}$ & $M_{ult,ten}$
M_{ult} = 2587,8 kNm

Plastic Bending Resistance (M_{pe})

M_{pe} = $Z_{pe} \times \delta_{yc}$ = 9518024,1 x 345
Therefore, M_{pe} = 3283,7 kNm

Cl 9.9.1

<u>Therefore ;</u> *Bending resistance (M_D)*

$$M_D = \frac{MR}{\gamma_{f3}\gamma_{m,steel}} = \frac{2587{,}811}{1 \times 1{,}05}$$

M_D = 2464,6 kNm

Cl 9.9.3

<u>10.11.3 Combined bending and shear checked</u>

Note : d_f for a composite flange the distance should be measured from the centroid of the transformed flange section and F_t is the limiting force in the flange and the A_{fe} of the bottom flange will be taken as the lower value for the two flanges

df	=	dG + (tslab /2) - (tbot /2)	=	718 + (191/2) - (28,6/2)			
df	=	798,5125	mm				
Afe	=	B x tbot	=	356 x 28,6	=	10161,3	mm2
δy_t	=	345	N/mm2				
Ff	=	δy_t x Afe	=	10161 x 345	=	3505,64	kN
Mf	=	Ff x df / γf3γm,steel	=	3505,6 x 798,5 / 1 x 1,05	=	2665,996	kNm

Note : M_f not greater than M_D

<u>Load effects obtined from the structural analysis</u>

Load case	CB1c : DL + SDL + 45 units of HB		
Maximum bending effect (hogging)	M	=	1406,0
Maximum shear effect (at support)	V	=	616,0
Bending capacity	M_D	=	2464,6
Shear capacity	$V_R = V_D$	=	2056,3

a) $V \leq V_D$ Satisfy

b) $M \leq M_D$ Satisfy

c)

$$\text{if } M > M_f, \text{ then } \frac{M}{M_D} + \left(1 - \frac{M_f}{M_D}\right)\left(\frac{2V}{V_R} - 1\right) \leq 1;$$

$M < M_f$

Equation c) Not Apply

d)

$$\text{if } V > V_R, \text{ then } \frac{V}{V_D} + \left(1 - \frac{V_R}{V_D}\right)\left(\frac{2M}{M_f} - 1\right) \leq 1;$$

$V < V_R$

Equation c) Not Apply

<u>Torsion</u>

<u>Load effects obtined from the structural analysis</u>

Load case	CB1c : DL + SDL + 45 units of HB		
Maximum torsion effect	T	=	229
Maximum shear effects (at support)	V	=	595
Shear capacity	V_D	=	2056,3

Where $x = B-2t_w$ = 355,6 - (2 x 7,9)

x = 340 mm

Note : The torsion plus shear effects need to be less than the shear resistant of the box girder

Calculations

mouchel

Project	Part of structure/scheme and status
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Code Ref	Calculations	Intermediate calculations according to BDs (Girder 1)

$$\frac{V_D}{2} \quad \frac{T}{x} + \frac{V}{2}$$

$$\frac{2056}{2} \quad \frac{229}{0{,}33973} + \frac{595}{2}$$

1028,14 > 971,5324

PASS

TRESTLE 1 - cross-section DD

BS5400 cl10.6.1.1

Axial compression resistance

From the table 10 BS5400-3 take that

L	=	5546,7	mm
l_e	=	0.7L	
l_e	=	3882,7	mm
r	=	191	mm
y	=	228,5	mm
r/y	=	0,8	
σ_y	=	345	N/mm2

$$\frac{\ell_e}{r}\sqrt{\frac{\sigma_y}{355}} = 20{,}3$$

From figure 4.14 $\sigma_c = \sigma_y \times 0{,}97$

$\sigma_c = 334{,}65$ N/mm^2

The effective area of a section $A_e = A_{c,net}$

Therefore, $P_D = \dfrac{A_e\sigma_c}{\gamma_M\gamma_{f3}}$

$P_D =$ **10687**

Combined compression and bending check

Worst load case considered for the checking Trestle 1 DL+SDL+HB 45

Maximum axial force in trestle (P)	=	3786,0	kN
Maximum bending moment in trestle (M	=	434,0	kNm
Maximum bending moment in trestle (M	=	0,04	kNm
The effective area of a secti A_e	=	33533,1	
Characteristic yield strength δy	=	345,0	kN/m2

$$\frac{P_{max}}{P_D} + \frac{M_{x,max}}{M_{Dx}} + \frac{M_{y,\,max}}{M_{Dy}} \le 1{,}0$$

$$0{,}354 + 0{,}041 + 3{,}7E\text{-}06 \le 1$$

≤ 1

PASS

iv. Results of the capacity of composite box girders

Table D.1 – Results for the capacity of girders against bending and shear (Spans 1&5)

	EC		BD	
Girders	$M_{el,rd}$ (kNm)	$V_{el,rd}$ (kNm)	$M_{el,rd}$ (kNm)	$V_{el,rd}$ (kNm)
Girder 1	2550,1	2241	2429,1	1953,1
Girder 2	2750,6	2396	2619,6	2093,7
Girder 3	3193,2	2498	3041,1	2177,7
Girder 4	3361,0	2714	3200,9	2324,1
Girder 5	3760,5	2879	3581,4	2381,9

Table D.2 - Results for the capacity of girders against bending and shear (Spans 2,3&4)

	EC		BD	
Girders	$M_{el,rd}$ (kNm)	$V_{el,rd}$ (kNm)	$M_{el,rd}$ (kNm)	$V_{el,rd}$ (kNm)
Girder 1	2793,3	2252	2660,3	2015,2
Girder 2	3038,7	2406	2894,0	2203,9
Girder 3	3193,2	2497	3041,1	2177,8
Girder 4	3361,1	2714	3200,9	2299,1
Girder 5	3760,5	2869	3581,4	2381,9

Table D.3 - Results for the capacity of girders against bending and shear (at Trestles 1&4)

	EC		BD	
Girders	$M_{pl,rd}$ (kNm)	$V_{pl,rd}$ (kNm)	$M_{el,r}$ (kNm)	$V_{el,r}$ (kNm)
Girder 1	3307,9	2325,1	2464,6	2056,3
Girder 2	3588,4	2478,0	2677,3	2159,8
Girder 3	3756,9	2570,6	2951,1	2292,4
Girder 4	3953,0	2785,0	3148,9	2400
Girder 5	5122,2	2981,9	3907	2514,3

Table D.4 - Results for the capacity of girders against bending and shear (at Trestles 2&3)

	EC		BD	
Girders	$M_{pl,rd}$ (kNm)	$V_{pl,rd}$ (kNm)	$M_{el,r}$(kNm)	$V_{el,r}$(kNm)
Girder 1	2898,9	2292,8	2264,2	2056,3
Girder 2	3151,7	2447,2	2457,7	2203,9
Girder 3	3478,2	2549,9	2767,7	2292,4
Girder 4	3478,0	2755,5	2763,2	2449
Girder 5	4348,4	2930,7	3327,4	2514,3

Appendix E

E. Graphical Ilustrations of the effects of actions on the girders

i. Minimum effects of actions on Girder 1 due to combinations shown in Appendix A

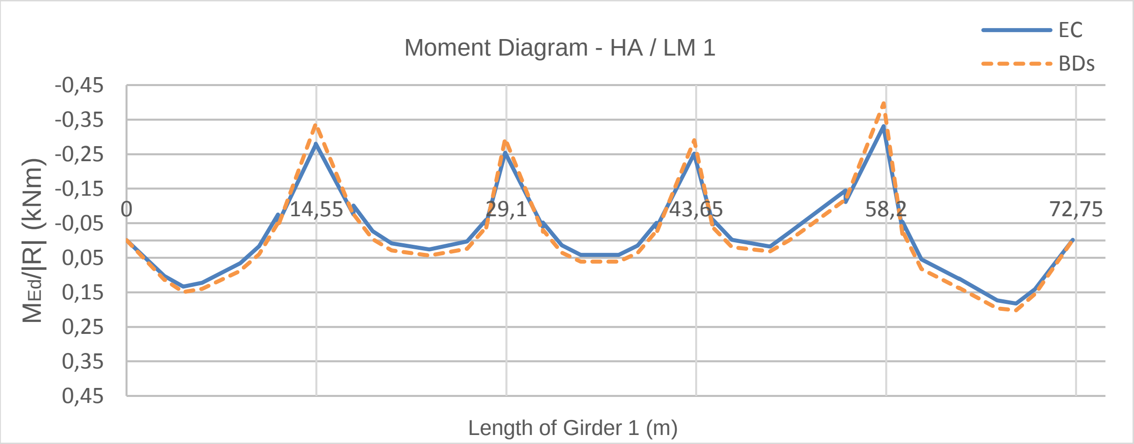


Figure E.1 - Graphical illustration of $M_{Ed}/|R|$ results due to combinations 1a and 2a along Girder 1

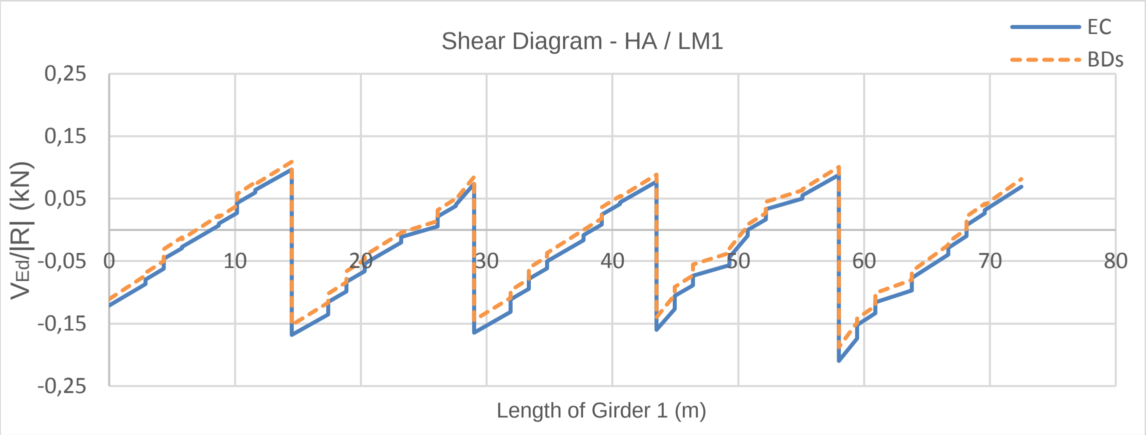


Figure E.2 - Graphical illustration of $V_{Ed}/|R|$ results due to combinations 1a and 2a along Girder 1

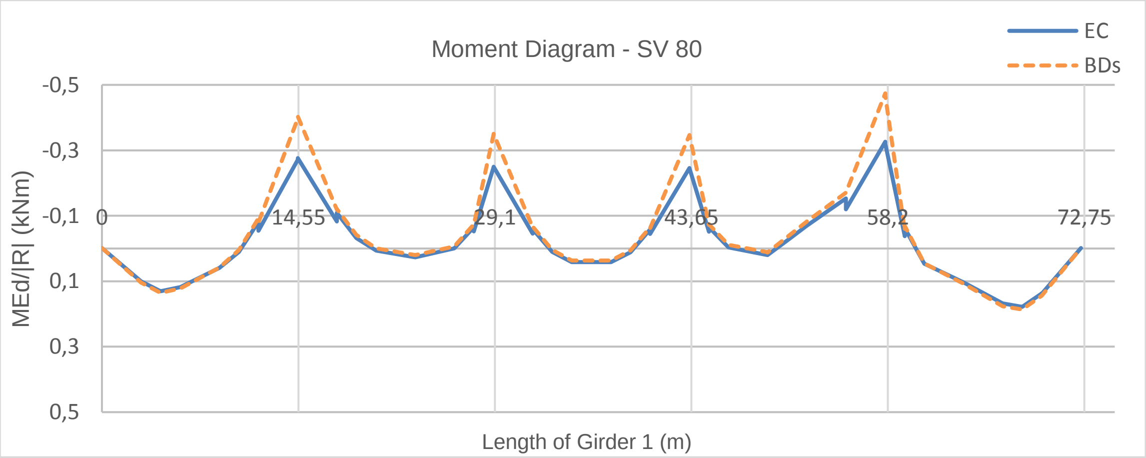


Figure E.3 - Graphical illustration of $M_{Ed}/|R|$ results due to combinations 1d and 2d along Girder 1

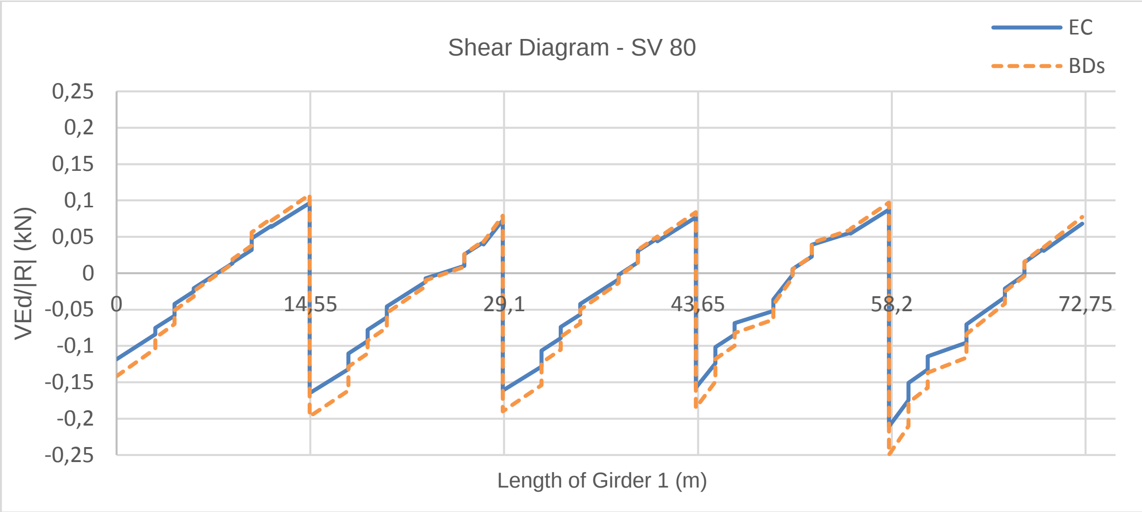


Figure E.4 - Graphical illustration of $V_{Ed}/|R|$ results due to combinations 1d and 2d along Girder 1

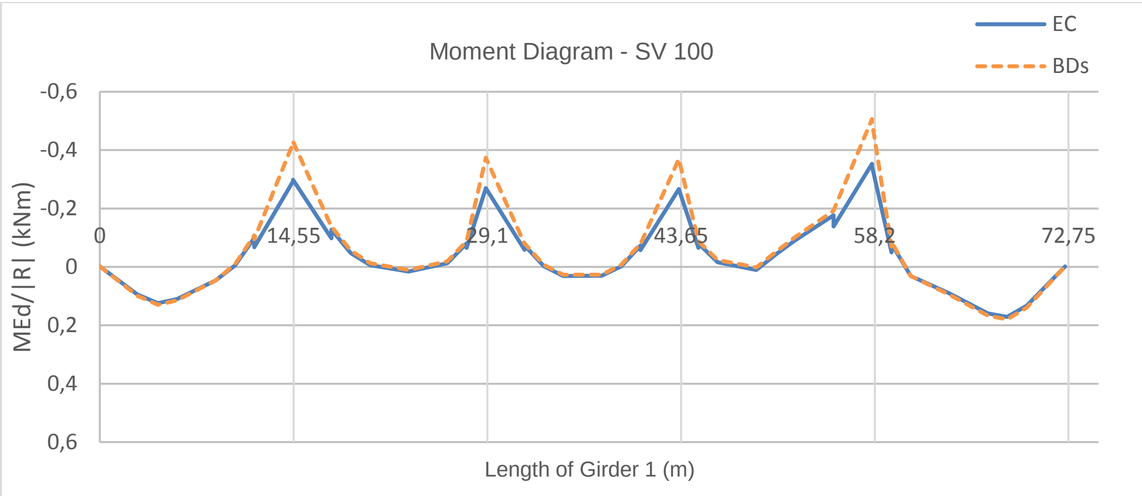


Figure E.5 - Graphical illustration of $M_{Ed}/|R|$ results due to combinations 1e and 2e along Girder 1

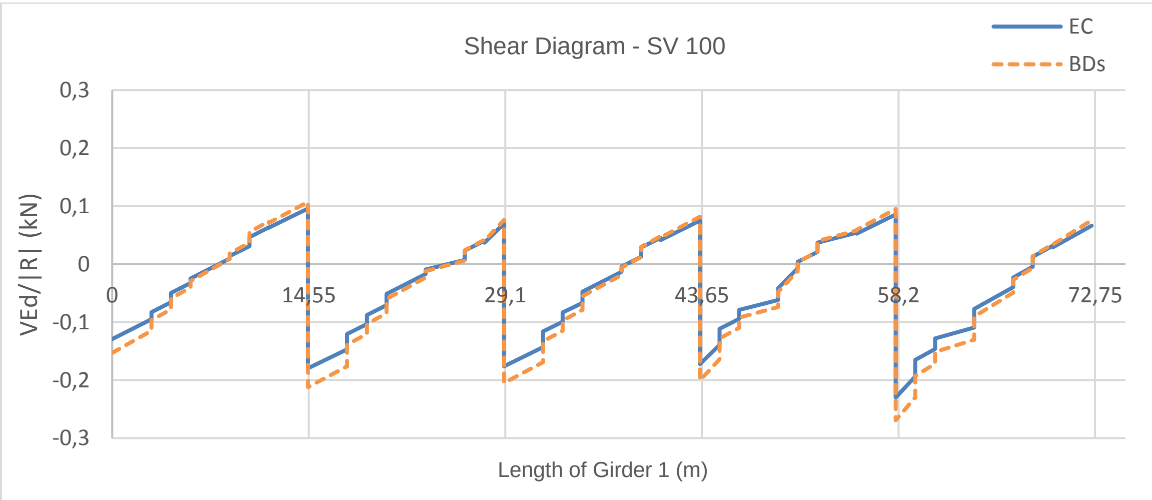


Figure E.6 - Graphical illustration of $V_{Ed}/|R|$ results due to combinations 1e and 2e along Girder 1

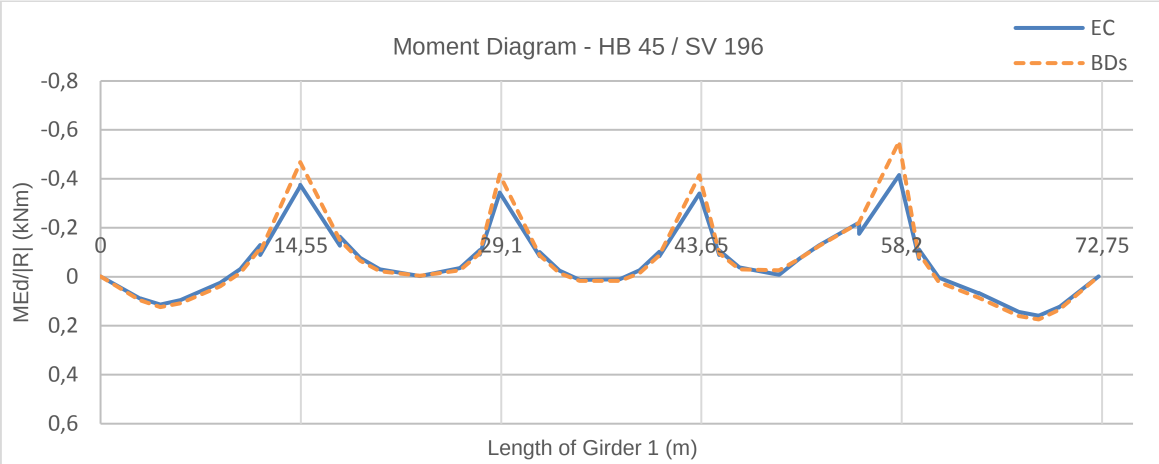


Figure E.7 - Graphical illustration of $M_{Ed}/|R|$ results due to combinations 1f and 2f along Girder 1

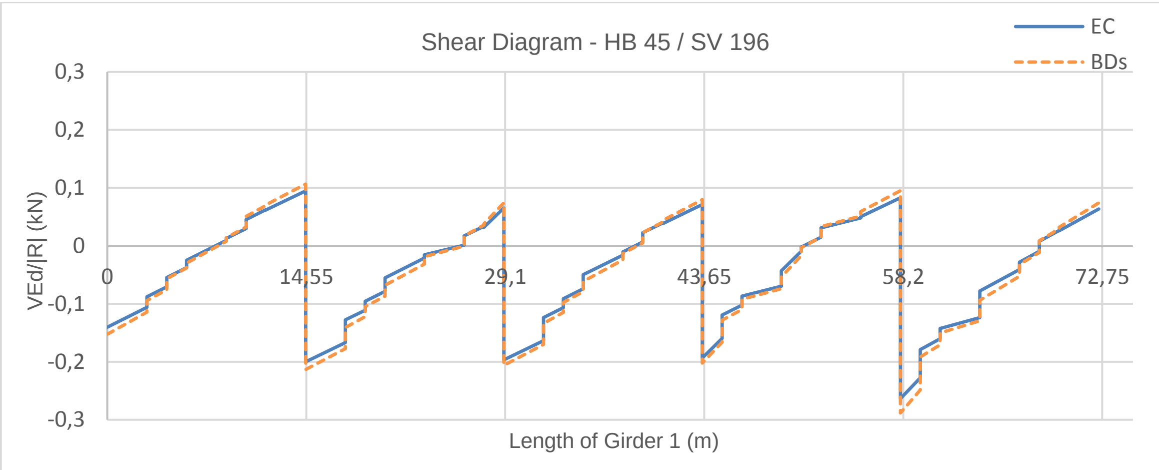


Figure E.8 - Graphical illustration of $V_{Ed}/|R|$ results due to combinations 1f and 2f along Girder 1

ii. Maximum effects of actions on Girder 3 due to combinations shown in Appendix A

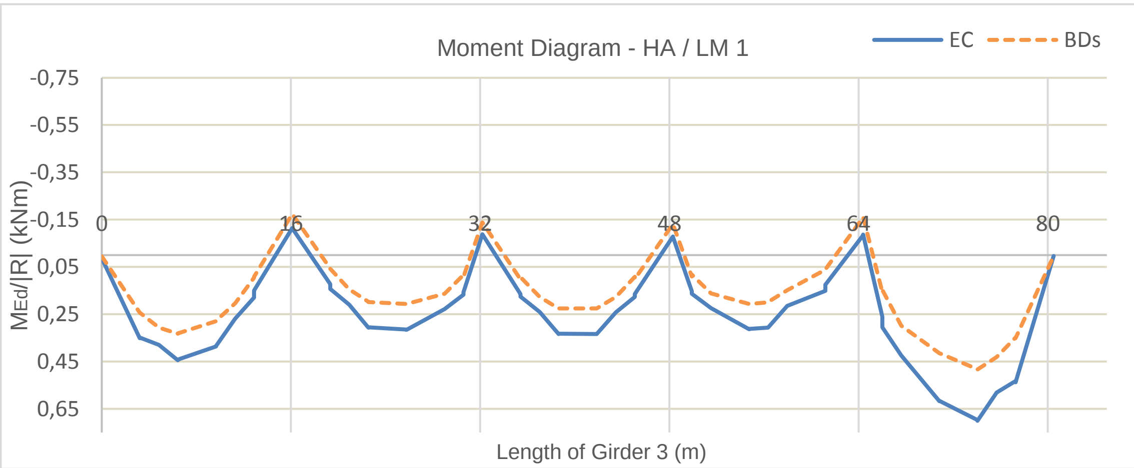


Figure E.9 - Graphical illustration of $M_{Ed}/|R|$ results due to combinations 1a and 2a along Girder 1

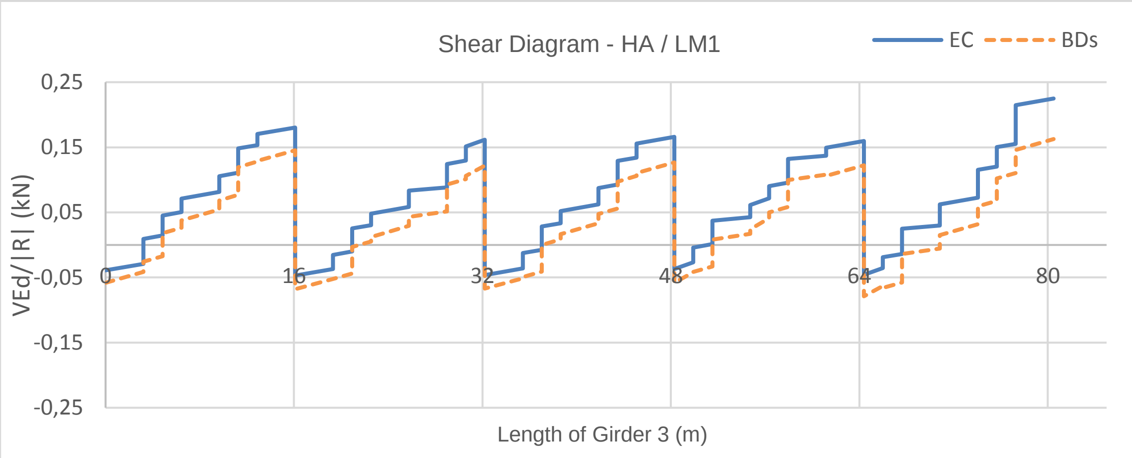


Figure E.10 - Graphical illustration of $V_{Ed}/|R|$ results due to combinations 1a and 2a along Girder 1

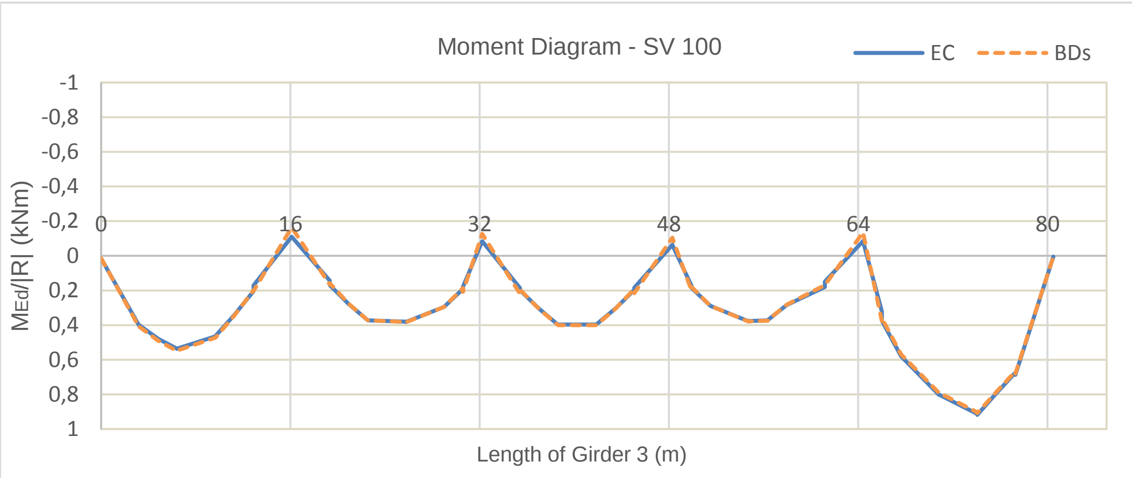


Figure E.11 - Graphical illustration of $M_{Ed}/|R|$ results due to combinations 1e and 2e along Girder 1

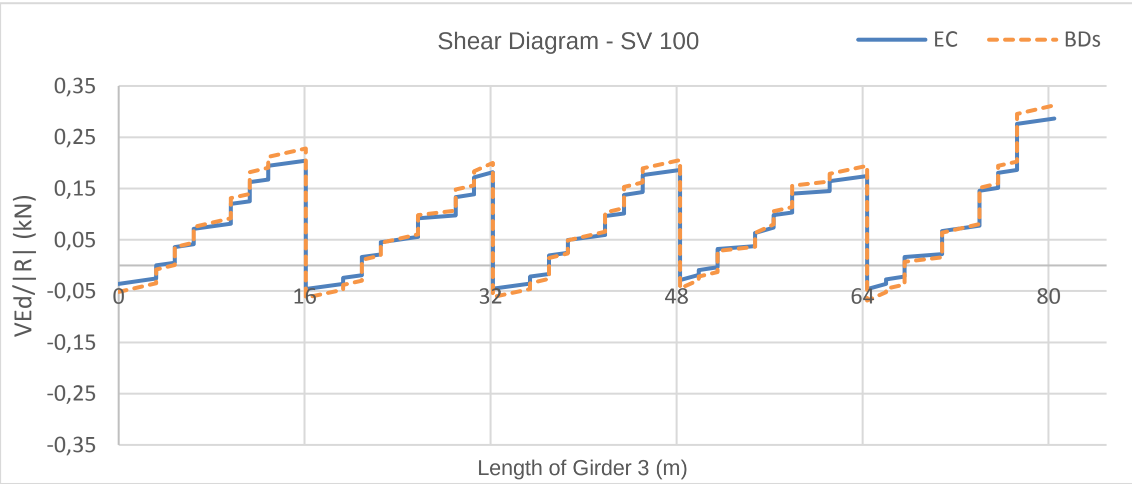


Figure E.12 - Graphical illustration of $V_{Ed}/|R|$ results due to combinations 1e and 2e along Girder 1

Annex A

a. Load Models of Special Vehicles

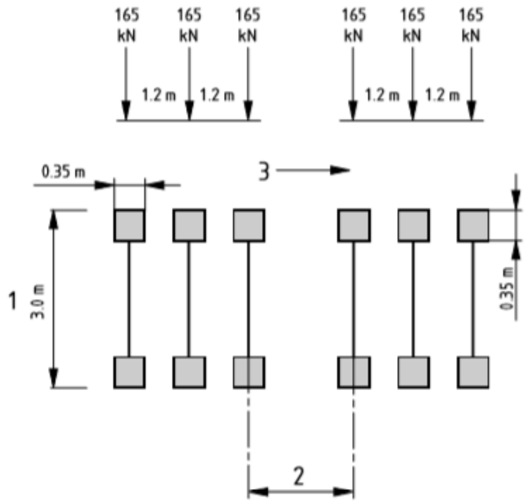


Figure AA.1 – SV 100 load model [42]

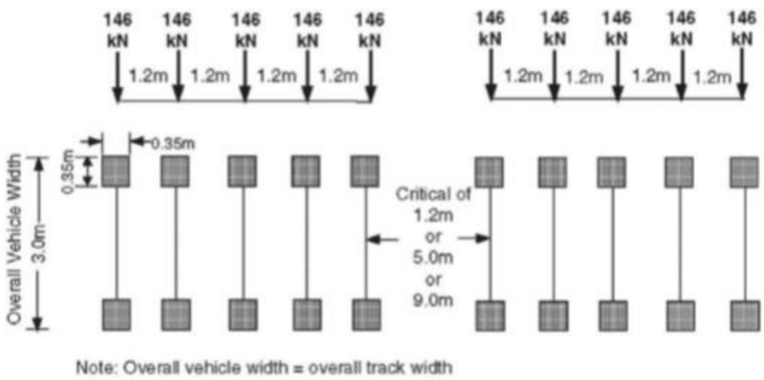


Figure AA.2 – SV 150 load model [41]

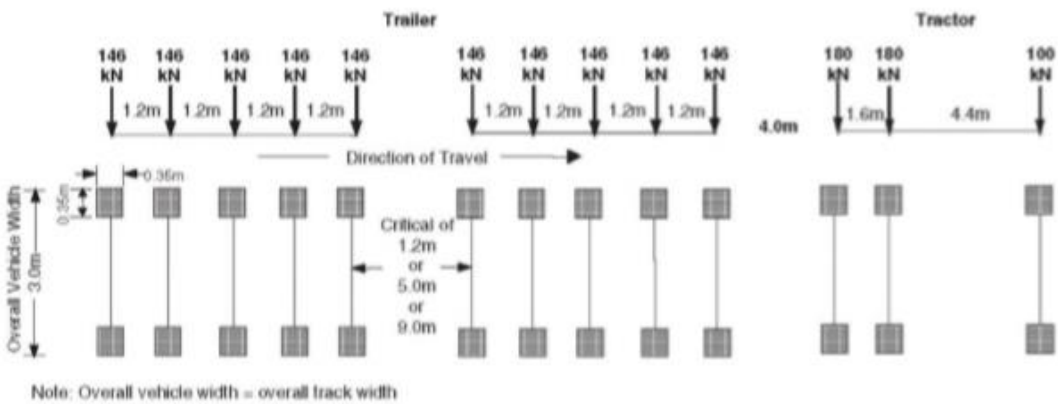


Figure AA.3 – SV-Train Load Model [41]

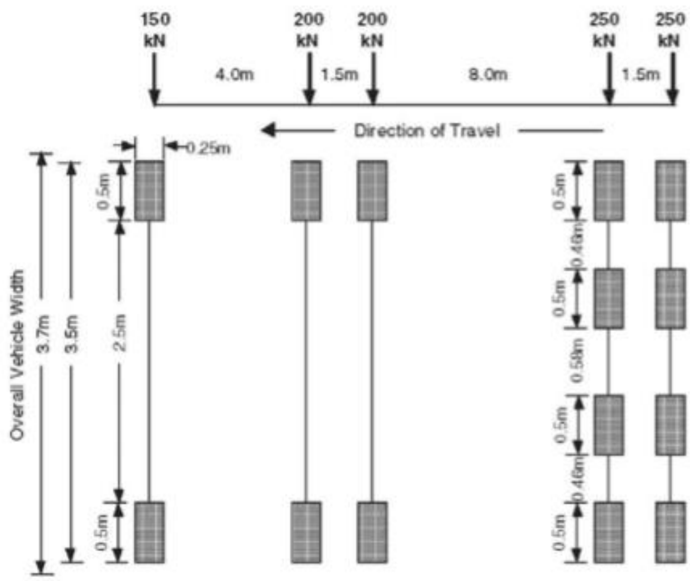
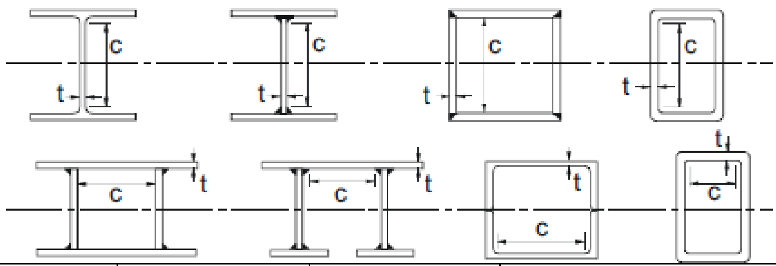
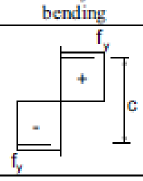
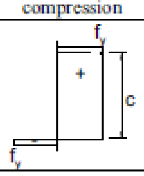
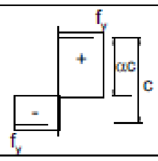
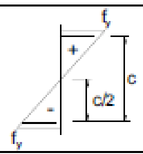
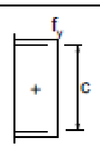
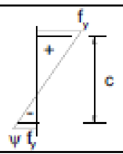


Figure AA.4 – SV TT Load Model [41]

Internal compression parts



Class	Part subject to bending	Part subject to compression	Part subject to bending and compression
Stress distribution in parts (compression positive)			
1	$c/t \le 72\varepsilon$	$c/t \le 33\varepsilon$	when $\alpha > 0{,}5$: $c/t \le \dfrac{396\varepsilon}{13\alpha - 1}$ when $\alpha \le 0{,}5$: $c/t \le \dfrac{36\varepsilon}{\alpha}$
2	$c/t \le 83\varepsilon$	$c/t \le 38\varepsilon$	when $\alpha > 0{,}5$: $c/t \le \dfrac{456\varepsilon}{13\alpha - 1}$ when $\alpha \le 0{,}5$: $c/t \le \dfrac{41{,}5\varepsilon}{\alpha}$
Stress distribution in parts (compression positive)			
3	$c/t \le 124\varepsilon$	$c/t \le 42\varepsilon$	when $\psi > -1$: $c/t \le \dfrac{42\varepsilon}{0{,}67 + 0{,}33\psi}$ when $\psi \le -1$ *): $c/t \le 62\varepsilon(1-\psi)\sqrt{(-\psi)}$

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

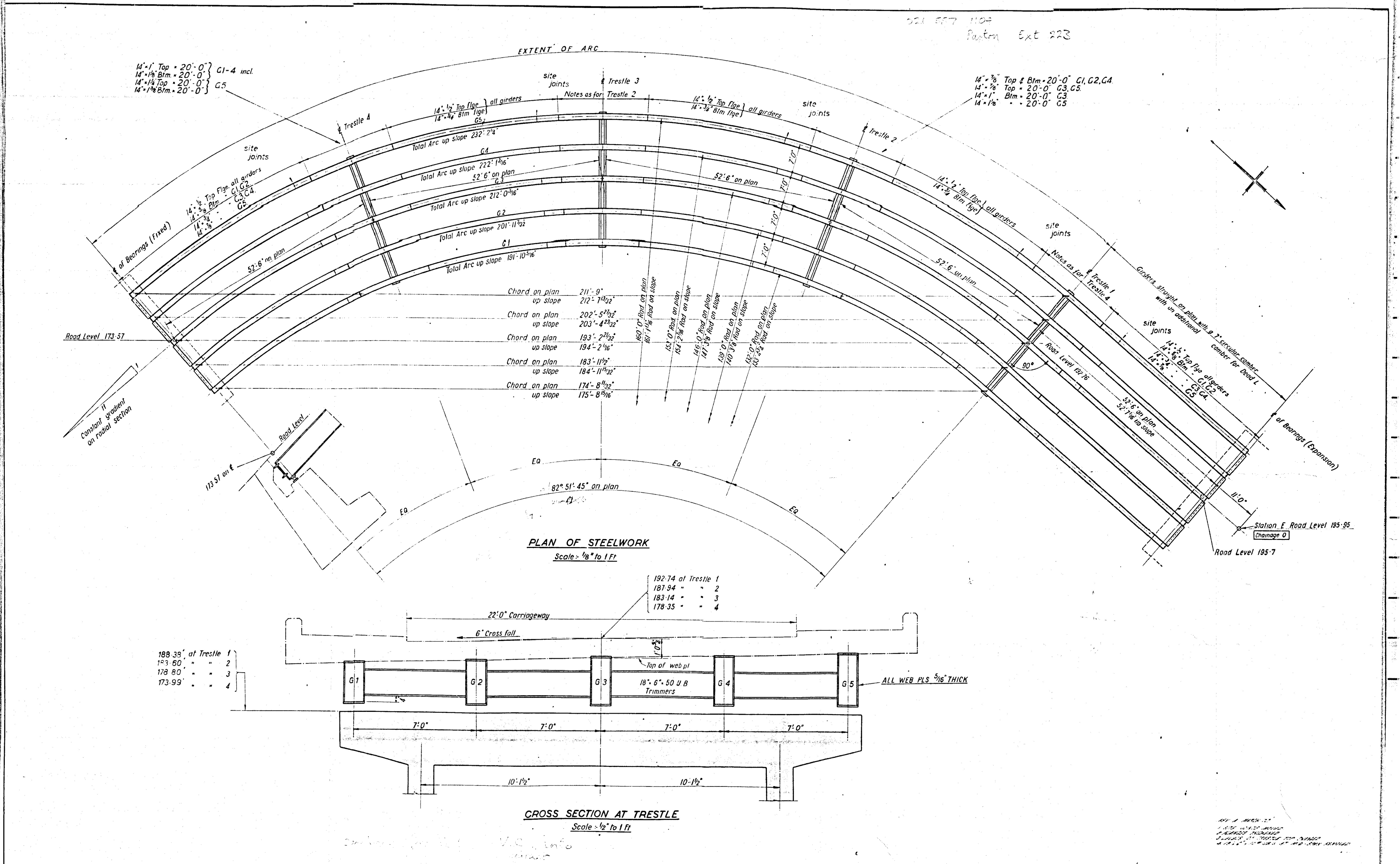
Figure AA.5 – Classification of steel cross-sections

Annex B

b. Drawings for Structural Assessment

Drawing BB.1

R019872



COUNTY BOROUGH OF DEWSBURY

A 652 WELLINGTON ROAD IMPROVEMENT STAGE I. BRIDGEWORKS

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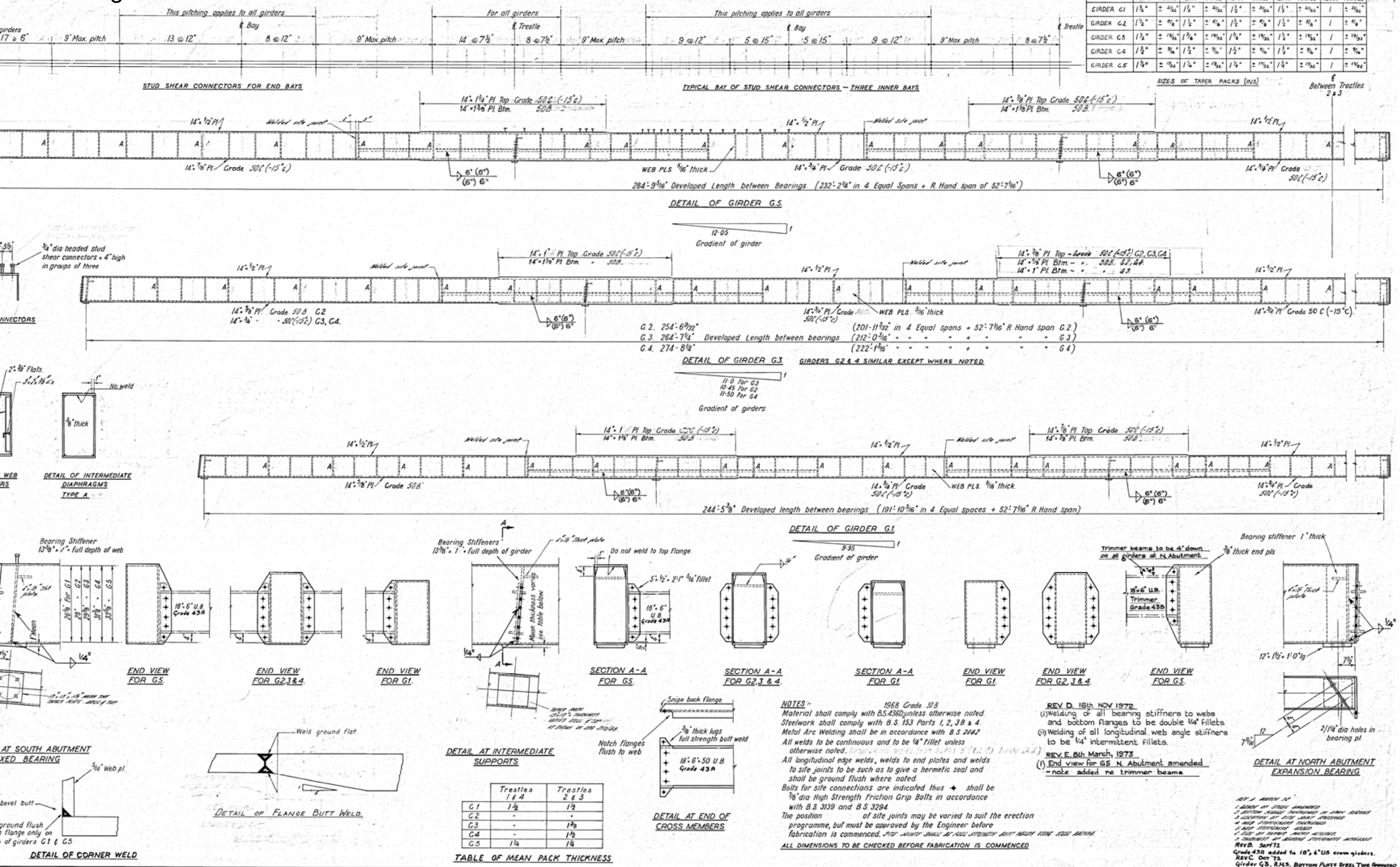
BOROUGH ENGINEER & SURVEYOR.
DEWSBURY MARCH 1972
DRAWING NO. 2110 / 49 / 4.

Drawing BB.2



LOCATION	TRESTLE 1. MEAN	TRESTLE 1. RANGE	TRESTLE 2. MEAN	TRESTLE 2. RANGE	TRESTLE 3. MEAN	TRESTLE 3. RANGE	TRESTLE 4. MEAN	TRESTLE 4. RANGE	SOUTH ABUTMENT MEAN	SOUTH ABUTMENT RANGE
GIRDER G1	1 1/2"	± 21/32"	1 1/2"	± 21/32"	1 1/2"	± 21/32"	1 1/2"	± 21/32"	1	± 21/32"
GIRDER G2	1 1/2"	± 5/8"	1 1/2"	± 5/8"	1 1/2"	± 5/8"	1 1/2"	± 5/8"	1	± 5/8"
GIRDER G3	1 1/2"	± 19/32"	1 3/8"	± 19/32"	1 3/8"	± 19/32"	1 1/2"	± 19/32"	1	± 19/32"
GIRDER G4	1 1/2"	± 9/16"	1 1/2"	± 9/16"	1 1/2"	± 9/16"	1 1/2"	± 9/16"	1	± 9/16"
GIRDER G5	1 1/4"	± 17/32"	1 1/4"	± 17/32"	1 1/4"	± 17/32"	1 1/4"	± 17/32"	1	± 17/32"

SIZES OF TAPER PACKS (INS)

	Trestles 1 & 4	Trestles 2 & 3
G1	1 1/2	1 1/2
G2	"	"
G3	"	1 3/8
G4	"	1 1/2
G5	1 1/4	1 1/4

TABLE OF MEAN PACK THICKNESS

NOTES:-

Material shall comply with B.S.4360 1968 Grade 50B unless otherwise noted.
Steelwork shall comply with B.S. 153 Parts 1, 2, 3B & 4.
Metal Arc Welding shall be in accordance with B.S. 2642
All welds to be continuous and to be 1/4" fillet unless otherwise noted.
All longitudinal edge welds, welds to end plates and welds to site joints to be such as to give a hermetic seal and shall be ground flush where noted.
Bolts for site connections are indicated thus ✦ shall be 7/8" dia High Strength Friction Grip Bolts in accordance with B.S. 3139 and B.S. 3294
The position of site joints may be varied to suit the erection programme, but must be approved by the Engineer before fabrication is commenced.

ALL DIMENSIONS TO BE CHECKED BEFORE FABRICATION IS COMMENCED

REV D. 16th NOV 1972.
(i) Welding of all bearing stiffners to webs and bottom flanges to be double 1/4" Fillets.
(ii) Welding of all longitudinal web angle stiffners to be 1/4" intermittent Fillets.

REV. E. 8th March, 1973
(I) End view for G5 N. Abutment amended - note added re trimmer beams.

REV B. SEPT 72
Grade 43A added to 18" x 6" UB cross girders.
REV C OCT '72
Girder G3. R.H.S. Bottom Plate Steel Type Amended

COUNTY BOROUGH OF DEWSBURY

A 652 WELLINGTON ROAD IMPROVEMENT STAGE I. BRIDGEWORKS

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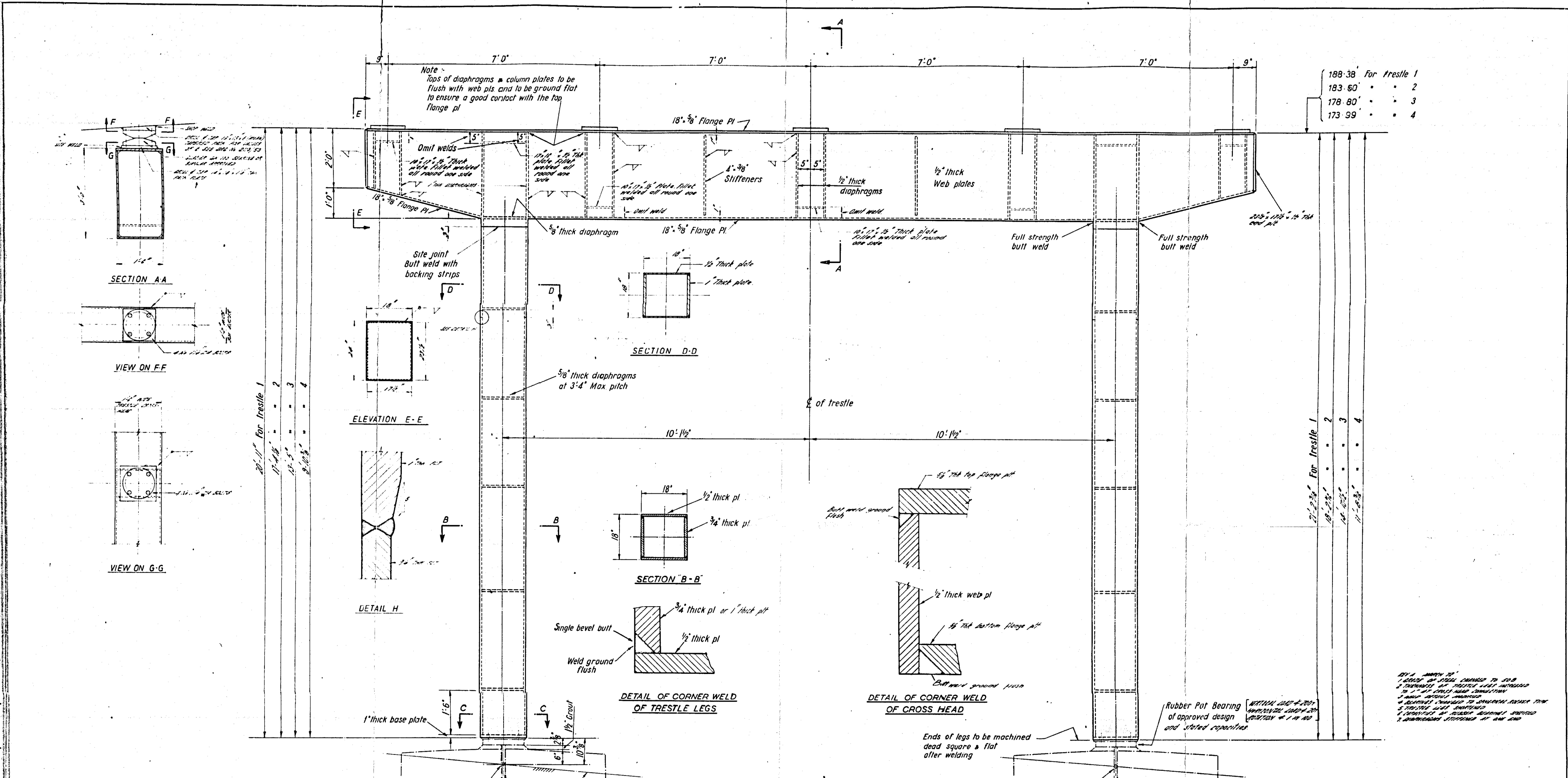
DEWSBURY MARCH 1972
DRAWING NO. 2110 / 50 / E

NG00500 K198

K0198/1



NOTES:-
Material shall comply with B.S. 4360:1968 Grade 50B unless otherwise noted
Steelwork shall comply with B.S. 153 Parts 1, 2, 3B & 4
Metal Arc Welding shall be in accordance with B.S. 2642 for Grade 50 steels

All welds to be continuous and to be ¼" fillet unless otherwise noted.
All edge welds and butt welds to site joints to be such as to give a hermetic seal and shall be ground flush.
The position and detail of site joints may be varied to suit the erection programme but must be approved by the Engineer before fabrication is commenced

ALL DIMENSIONS TO BE CHECKED BEFORE FABRICATION IS COMMENCED.

COUNTY BOROUGH OF DEWSBURY

A 652 WELLINGTON ROAD IMPROVEMENT STAGE I. BRIDGEWORKS

G.J. DRUMMOND. F.I.C.E. F.I.MUN.E.
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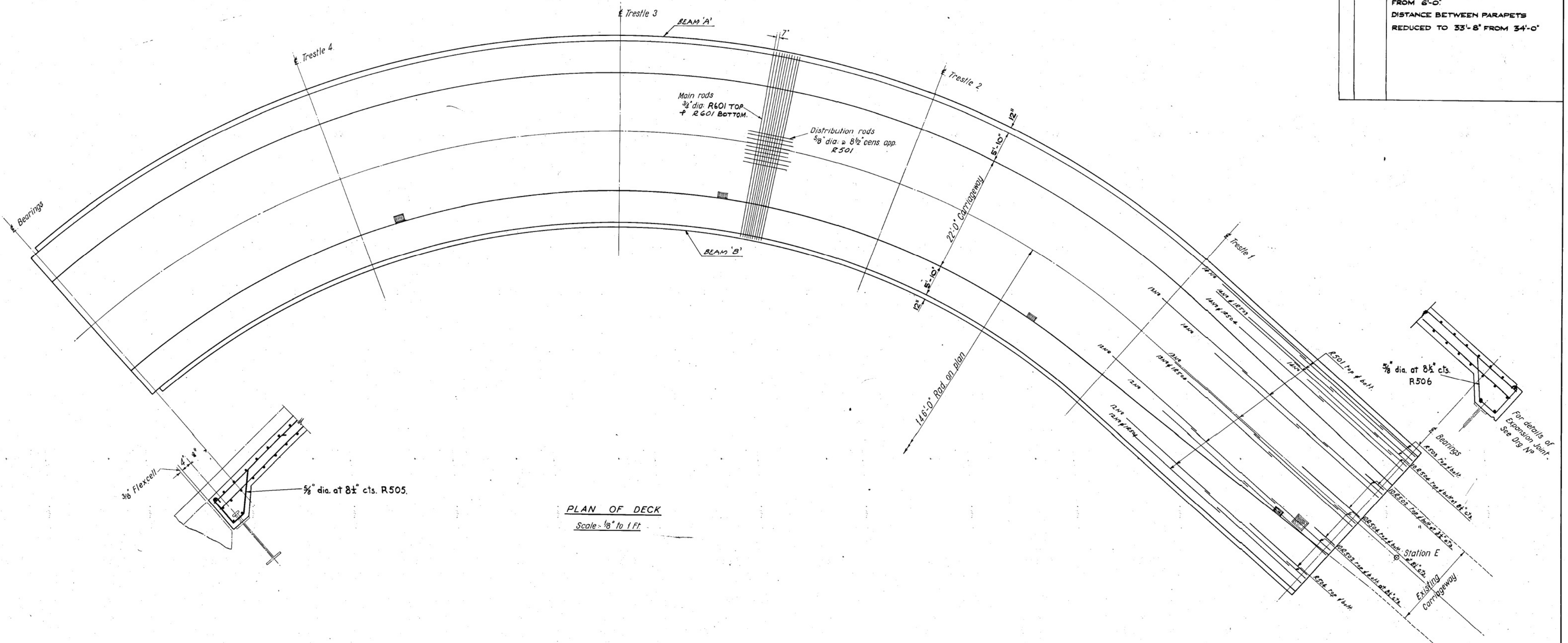
DEWSBURY. MARCH 1972
DRAWING NO. 2110 / 48 / A

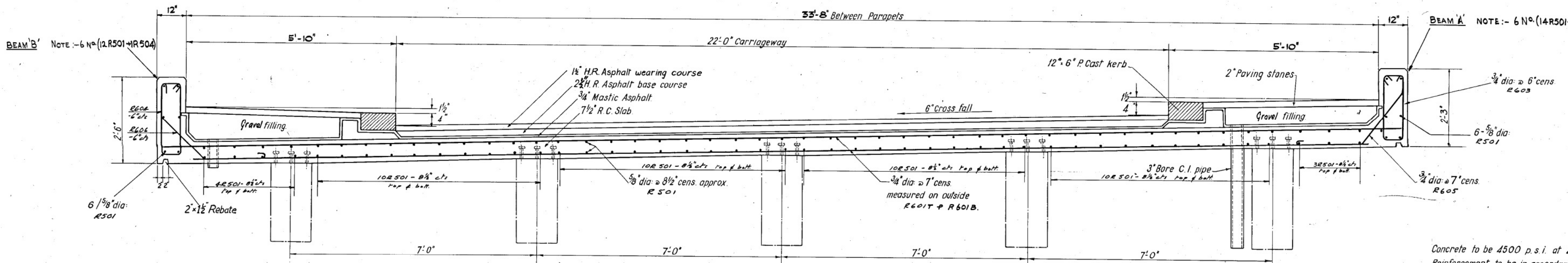
Drawing BB.4

		AMENDMENTS.
No.	DATE	DESCRIPTION
A.	JUNE '73	COPING WIDTH INCREASED TO 12" FROM 10". WIDTH OF FOOTWAY REDUCED TO 5'-10" FROM 6'-0". DISTANCE BETWEEN PARAPETS REDUCED TO 33'-8" FROM 34'-0"



PLAN OF DECK

Scale - 1/8" to 1 Ft



TYPICAL CROSS SECTION FROM THRESTLE ONE TO CONTRACTION JOINT.

Scale - 3/4" to 1 ft

Concrete to be 4500 p.s.i. at 28 days
Reinforcement to be in accordance with B.S. 785
Laps in longitudinal bars to be staggered. Lap 12" min.
Each span of the bridge must be supported in at least two intermediate positions until the deck concrete has reached the appropriate strength.

Reinforcement listed on Sheet No 2110/45 – Sheet 1.
Minimum cover to main steel, in beams 1½", in slab 3/4".

COUNTY BOROUGH OF DEWSBURY

A 652 WELLINGTON ROAD IMPROVEMENT STAGE I. BRIDGEWORKS

DETAIL OF BRIDGE DECK SHOWING REINFORCEMENT

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BOROUGH ENGINEER & SURVEYOR.
DEWSBURY. MARCH 1972
DRAWING NO. 2110/ 45 /A

K19811

NG00491