

ISTANBUL TECHNICAL UNIVERSITY ★ GRADUATE SCHOOL OF SCIENCE
ENGINEERING AND TECHNOLOGY

**DESIGN OF A STAINLESS STEEL COMPOSITE BRIDGE ACCORDING TO
THE EUROCODES AND COST ANALYSIS**

M.Sc. THESIS

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Department of Civil Engineering

Earthquake Engineering Programme

JUNE 2014

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İSTANBUL TEKNİK ÜNİVERSİTESİ ★ FEN BİLİMLERİ ENSTİTÜSÜ

**PASLANMAZ ÇELİK İLE KOMPOZİT BİR KÖPRÜNÜN EUROCODE'A
GÖRE TASARIMI VE MALİYET ANALİZİ**

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To my dear mother, Reyhane and my precious family,

FOREWORD

The study presented here is part of the requirements to achieve the Master of Science in Engineering (M.Sc.Eng.) at the Department of Civil Engineering at the Istanbul Technical University.

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DESIGN OF A STAINLESS STEEL COMPOSITE BRIDGE ACCORDING TO THE EUROCODES and COST ANALYSIS

SUMMARY

A constant research of more ecological and efficient structures has enabled bridges to be more innovative through the years. Nowadays, as the need is greater than ever, a new kind of bridge is expanding in the entire world: the stainless steel bridges. On 17 October 1912, Krupp Iron Works in Germany, Benno Strauss and Eduard Maurer patented austenitic stainless steel as Thyssen-Krupp Nirosta and it has been improved since then. Recently, there are different types of stainless steels: when nickel is added, for instance, the austenite structure of iron is stabilized. This crystal structure makes such steels virtually non-magnetic and less brittle at low temperatures. For greater hardness and strength, more carbon is added. with proper heat treatment, these steels are used for such products as razor blades, cutlery, and tools. In this study of thesis, duplex stainless steel which has a mixed microstructure of austenite and ferrite, the aim usually being to produce a 50/50 mix, although in commercial alloys the ratio may be 40/60. Duplex stainless steels have roughly twice the strength compared to austenitic stainless steels and also improved resistance to localized corrosion, particularly pitting, crevice corrosion and stress corrosion cracking. They are characterized by high chromium (19–32%) and molybdenum (up to 5%) and lower nickel contents than austenitic stainless steels is preferred for the bridge design.

The aim of the thesis is to investigate the structural behavior of these a stainless steel composite bridge in accordance with Eurocode norms and a comparison of the composite bridge versus a traditional post-tensioned bridge in Denmark. During this cost estimation, construction unit prices according to Turkey Road Directorate was used for Turkish construction unit prices , on the other hand construction unit prices according to Denmark Road Directorate was used for construction unit prices in Denmark. To analysis structure, a 2D model has been designed to be optimal and tested under national annex for the loads defined in the Eurocodes. Guidelines from the literature has been used to determine the optimal geometry of the bridge.

PASLANMAZ ÇELİK İLE KOMPOZİT BİR KÖPRÜNÜN EUROCODE'A GÖRE TASARIMI VE MALİYET ANALİZİ

ÖZET

On dokuzuncu yüzyılda demir ve çelik köprülerde önemli ilerlemeler kaydedilmiştir. Önceleri taş ve ahşap köprü yapım tekniğine uygun demir ve çelik köprüler yapılmışsa da, daha sonra bu tür malzemenin kendine has imkânlarının olduğu anlaşılmıştır. Bu köprülerin bir türü olan asma köprülerde, köprü tabliyesi, iki ayağa bağlı çelik halatlara asılı olarak taşınır. Diğer bir tür de kafes taşıyıcı sistemi olan çelik köprülerdir. Bunların özelliği yüksek titreşimlere mukavim olmalarıdır. Yapı malzemesi olarak karbon çelik, demir elementi ile genellikle %0,2 ila %2,1 oranlarında değişen karbon miktarının bileşiminden meydana gelen bir alaşımdır. Çelik alaşımındaki karbon miktarları çeliğin sınıflandırılmasında etkin rol oynamaktadır. Karbon genel olarak Demir'in alaşımlayıcı maddesi olsa da demir elementini alaşımlamada Magnezyum, Krom, Vanadyum ve Volfram gibi farklı elementler de kullanılabilir. Karbon ve diğer elementler demir atomundaki kristal kafeslerin kayarak birbirini geçmesini engelleyerek sertleşme aracı rolü üstlenirler. Alaşımlayıcı elementlerin, çelik içerisindeki, değişen miktarları ve mevcut bulundukları formlar (çözünen elementler, çökelti evresi) oluşan çelikte sertlik, süneklilik ve gerilme noktası gibi özellikleri kontrol eder. Karbon miktarı yüksek olan çelikler demirden daha sert ve güçlü olmasına rağmen daha az sünektirler. Yüksek Karbon içeren alaşımlar, düşük erime noktaları ve dökme kabiliyetleri nedeniyle dökme demir olarak bilinirler. Çelik ayrıca az miktarda karbon içeren fakat demir cürufarını da kapsayan dövme demir olarak da ayırt edilir. İki ayırt edici faktör de çeliklerin pas önleyiciliklerini artırır ve daha iyi kaynaklanabilirlik sağlar. Her ne kadar Rönesans'tan uzun süre önceleri çelik çeşitli etkisiz metotlarla üretilmişse de 17. yüzyılda icat edilen daha etkili üretimlerden sonra kullanımı yaygın bir hâl almıştır. 19. yüzyılın ortalarında Bessemer Değiştirgeci'nin icadıyla çelik pahalı olmayan seri üretim materyali olmaya başladı. İlerleme sürecinde ilave edilen temel oksijen ile çelik yapımı gibi mükemmelleştirmeler üretimin maliyetini düşürürken metalin kalitesini arttırmıştır. . Çeliğin ucuzlaması bunun daha yaygın kullanılmasını sağlamıştır. Ayrıca kaynak tekniğinin ilerlemesi daha sağlam birleşme yerlerinin yapımını sağlamıştır. Çeliğin mukavemetinin yüksekliğinden dolayı, taşıdığı yükün ağırlığına oranı yüksektir. Değişik çelik alaşımlarının kullanılması çelik köprülerin mukavemetinin daha yüksek ve dış tesirlere daha dayanıklı olmasını sağlamıştır. Her ne kadar alüminyum kullanılmasıyla, daha hafif köprüler elde edilebilirse de pahalı olması bunların yaygınlaşmasını önlemiştir. Ancak daha sonra betonarme yapı türünün gelişmesi ve 1950'lerden sonra önerilmeli betonun uygulanmağa başlanması, köprü inşaatında önemli adımların atılmasına yol açmıştır. Köprü inşaatında önemli gelişmeleri ayrıca matematiksel ve deneysel araştırmalar, bilgisayar kullanımı hızlandırmıştır. Bu tür metodları kullanarak, mühendisler statik ve dinamik yükler altında köprüde ortaya çıkacak gerilmeleri daha kesin elde edebilecek duruma gelmişlerdir. Bu suretle daha büyük açıklıkların daha narin köprülerle geçilmesi mümkün olmuştur. Çeliğin gelişimi ise daha eskilere

dayanmaktadır. Günümüzde, her yıl 1300 milyon ton üretimi ile, çelik dünyada en çok kullanılan ortak malzemelerden birisidir. Binalarda, altyapı üretiminde, aletlerde, gemilerde, otomobillerde, makinelerde, aksesuarlarda ve silahlarda ana malzemedir. Modern çelik çeşitli standartlar kuruluşları tarafından çeşitli özelliklerine göre sınıflandırılır. Demir, birçok metal gibi, yeryüzü kabuğunda oksijen ve ya sülfür gibi diğer elementlerle kombine olmuş halde, sadece cevher şeklinde bulunur. Standart demirin içerdiği mineraller arasında demir oksit (esmer renkte olan doğal demir oksidinden ibaret bir maden filizi) ve pirit (budala altını) vardır. Demir, oksijenin uzaklaştırılması ve cevherin kimyasal açıdan tercih edilen eşi karbon ile birleştirilmesi ile cevherden çekilir. Bu süreç, ilk olarak kalay (yaklaşık olarak erime noktası 250 °C (482 °F)) ve bakır (yaklaşık olarak erime noktası 1.000 °C (1,830 °F)) gibi erime noktası düşük metallerde tatbiki yapılmış ve madeni tasviye etme işlemi olarak bilinmektedir. Karşılaştırma yapılırsa dökme demirin yaklaşık olarak 1.370 °C (2,500 °F) civarında eridiği görülür. Bütün bu sıcaklıklara Bronz Çağı'ndan buyana uygulanan eski metodlarla ulaşmak mümkündür. Oksijen oranının kendi kendini hızlıca 800 °C nin civarına yükseltmesinden beri, madenin tasviyesi işleminin düşük oksijen ortamında yer alması önem taşımaktadır. Bakır ve kalaya benzemeyen sıvı demir karbonu kolayca çözer. Maden tasviye işlemi çelik adı verilen yüksek karbon içeren alaşım pik demir olarak sonuçlanır. Fazla gelen karbon ve diğer katkı maddeleri bir sonraki basamakta uzaklaştırılır.

Paslanmaz çelikler ise, günümüz çelik sektörünün üzerine yoğunlaştığı ve en çok kullanılan çelik türüdür ayrıca neredeyse tüm sanayi kollarında kullanılmaktadır. Yıllardır süregelen etkin ve sürekli araştırmalar ile daha ekolojik ve daha yenilikçi nitelikler köprü gibi yapıların tasarımında olumlu gelişmeler sağlamıştır. Paslanmaz çelik köprüler tüm dünyada kullanılmaya başlanmış olup bu yeni malzeme türü ile daha ekolojik ve yenilikçi tasarımlar mümkün hale gelmiştir: Antik çağlarda pasa ve korozyona dirençli demir yapı elemanları için çalışmalar yapılmaya başlanmış olup, Hindistan'ın Delhi şehrinde bulunan 7 metre yüksekliğindeki "Delhi Demir Ayağı" Gupta Kralı I. Kumaragupta tarafından yaptırılmıştır. Pasa dirençli bu demir direkt krom yerine fosfor kullanılarak direnç sağlanmıştır. 1821 yılında demir-krom alaşımları kullanarak korozyona karşı direnç sağlamaya çalışan ilk kişi Fransız metalurjist Pierre Berthier olmuştur. 19. yüzyıl metalurjistlerinin düşük karbon ve yüksek krom alaşımları kullanarak paslanmaz çelik elde etmelerine karşın bu alaşımların kırılganlığı yüksek olmuştur. Daha sonrasındaki gelişim sürecinde ise 17 Ekim 1912 tarihinde ise, Krupp mühendisleri Benno Strauss ve Eduard Maurer Thyssen-Krupp Nirosta olarak ostenitik paslanmaz çelik patentini alması ile başlayıp günümüze kadar gelişim göstermiştir.

Ekolojik ve yenilikçi bir yapı malzemesi olarak köprülerde kullanılmaya başlanan paslanmaz çelik, metalurjide çeliğin, minimum % 10,5 il % 11 arasında krom eklenerek elde edilen alaşım olarak tanımlanır. Eklenen krom, çeliğin yapısında çeliğin yüzeyine çıkarak kromoksit tabakası oluşturur ve demirin oksitlenmesini engeller. Paslanmaz çelik, beş ana gruba ayrılır; ostenitik, ferritik, martenzitik, duplex ve de çökeltme sertleşmesi uygulanabilen alaşımlar olup bu malzemenin en belirgin özelliklerinin başında korozlanma veya paslanma yapmaması ve sıradan çelikler gibi üzerinde su lekeleri bırakmaması gelmektedir. Ancak tamamen leke geçirmez olduğu söylenemez. Düşük oksijenli ve zayıf sirkülasyonlu ortamlar ile yüksek tuzluluk paslanmaz çeliği etkiler. Krom oranı ve pasivasyon oranı çeliğin korozyona ve pasa direncini belirlemektedir. Alışlagelmiş yapılarda kullanılan karbon çeliği ise demir elementi ile genellikle % 0,2 ila %2,1 oranlarında değişen karbon miktarının bileşiminden meydana gelen bir alaşımdır. Alaşıyımlayıcı elementlerin, çelik

içerisindeki, değişen miktarları ve mevcut bulundukları formlar (çözünen elementler, çökelti evresi) oluşan çelikte sertlik, süneklilik ve gerilme noktası gibi özellikleri kontrol eder. Karbon miktarı yüksek olan çelikler demirden daha sert ve güçlü olmasına rağmen daha az sünek olduğu söylenebilir. Bu iki yapı malzemesi kıyaslandığında ise ana farklar arasında karbon çeliğin içerdiği daha yüksek karbon içeriği ile daha düşük bir erime noktasına sahip olup daha fazla yumuşaklık ve dayanıklılığa sahip olup iyi ısı dağılımı göstermektedir. Paslanmaz çelik ise, içeriğindeki yüksek krom oranı ile korozyon ve boyama önlemek için çelik üzerinde görünmez bir tabaka oluşturması ve bu sayede karbon çeliğinden daha az bakım gerektirmesi, fiyat optimizasyonu sağlamasıdır.

Bu tez çalışmasında paslanmaz çelikten kompozit bir köprünün Eurocode standartlarına göre dizaynına, maliyet analizine ve Danimarka'da yer alan geleneksel ardgermeli betonarme bir köprünün maliyet kıyasına yer verilmiştir. Fiyat analizinde Bayındırlık bakanlığının birim fiyat verileri ile Danimarka Karayolları genel müdürlüğünün birim fiyat verileri temel esas alınarak kıyas yapılmıştır.

Kompozit köprü tasarımı ile paslanmaz çelik yapı malzemesi kullanılarak çelik profiller üzerlerindeki betonarme döşemeden gelen yükleri kenar ve orta ayaklara aktararak, çelik köprülerden farklı olarak çelik profillerle üzerlerindeki döşemenin uygun bir detayla birleştirilerek birlikte çalışmaları sağlanmıştır. Bu bağlamda, yurdumuzda sıkça kullanılan, elastomer mesnetlere serbestçe oturan paslanmaz çelik kirişler tasarımı ele alınmıştır. Paslanmaz çelik ile kompozit köprü tasarımı konusunda ülkemizde köprü tasarım yönetmeliği olmaması, köprü projelendiren mühendisler için belirsizlikler oluşturmaktadır. Bu çalışmada ayrıca Eurocode koşullarının uygulanmasında karşılaşılan sorunlara çözümler geliştirilmiştir. Depremsellik açısından yüksek ivmeye hakim olmayan Danimarka'da, tasarımı belirleyen etkinin depremsellik olmadığı görülmüştür. Tasarım parametreleri ağırlıklı olarak Eurocode (EN) 1993-2 Çelik yapılar, Eurocode (EN) 1994-2 Kompozit köprüler, Eurocode (EN) 1992-2 Betonarme köprüler, Eurocode (EN) 1990 tasarım kombinasyonları, Eurocode (EN) 1991 yüklemeler ve Danimarka Karayolları Müdürlüğü'nün Eurocode'a ilave olarak belirttiği ekler kullanılmıştır. Ayrıca paslanmaz çelik ile kompozit köprü tasarımı hakkında Avrupa ve Amerika başta olmak üzere dünya çapında yapılmış olan birçok araştırma dikkatle incelenerek tasarım kriterleri belirlenmiştir ve fiyat analizi için detaylı çalışmalar yapılmıştır. Doğru uygulanan paslanmaz çeliklerin korozyona karşı mükemmel direnç göstermeleri bakım maliyetlerini düşürmektedir. Bunun bir sonucu olarak, korozyon payı ya da herhangi bir koruyucu kaplamaya ihtiyaç duyulmamaktadır. Bu geleneksel malzeme eşdeğer bileşenleri dışında imal etmek için, daha hafif ve daha küçük kesitler ile paslanmaz çelik bileşenlerin kullanılmasına izin vermektedir. Yapı tasarımında karbon veya düktil çelikten paslanmaz çeliğe gidildiğinde ağırlık %25 oranında azalabilirken köprü maliyeti %30 ile %60'a kadar optimizasyon sağlayabilmektedir. Sonuç olarak yapı malzemesi olarak paslanmaz çelik seçimi yaşam döngüsü maliyetlerinin (LCC) 'yükleme, işletim ve tam ömrü boyunca bakımı ile ilgili toplam maliyetlerin analizi oldukça optime sonuçlar çıkardığı görülmüştür.

1. INTRODUCTION

1.1 General

1.1.1 What is stainless steel?

Stainless steel [8] is an alloy of steel, chromium and eventually other elements such as nickel or molybdenum. It is the chromium that renders the steel stainless. In fact, by virtue of the chromium's reaction with oxygen, the surface of stainless steel consists of a self-protecting passive layer that automatically regenerates itself if damaged. The other alloying elements (molybdenum in particular) further enhance this corrosion resistance.

According to its constituent elements and their relative percentages, stainless steel breaks down into more than a hundred grades grouped into several major categories that are found in European standard EN 10088. Thus, stainless steel may be ferritic, austenitic, duplex or martensitic. Each category has specific mechanical properties – such as hardness, yield strength, tensile strength, elongation, etc. These properties are decisive in the choice of a grade for one application or another.

Austenitic stainless steels are the most widely used category and today still account for almost 70% of stainless production. However, with the fluctuating cost of nickel in particular, but also of molybdenum, use of the austeno-ferritic duplex category, with a lower content of alloying elements, is increasing. Today, it offers equivalent or even superior quality in terms of corrosion resistance and its superiority in terms of mechanical performance makes it a particularly competitive material in the bridges and footbridges market.

1.1.2 History of stainless steel

A few corrosion-resistant iron artifacts survive from antiquity.

- The corrosion resistance of iron-chromium alloys was first recognized in 1821 by French metallurgist Pierre Berthier

- In the late 1890s, Hans Goldschmidt of Germany developed an aluminothermic (thermite) process for producing carbon-free chromium.
- Between 1904 and 1911, several researchers, particularly Leon Guillet of France, prepared alloys that would today be considered stainless steel.
- In 1912, Elwood Haynes applied for a US patent on a martensitic stainless steel alloy, which was not granted until 1919.

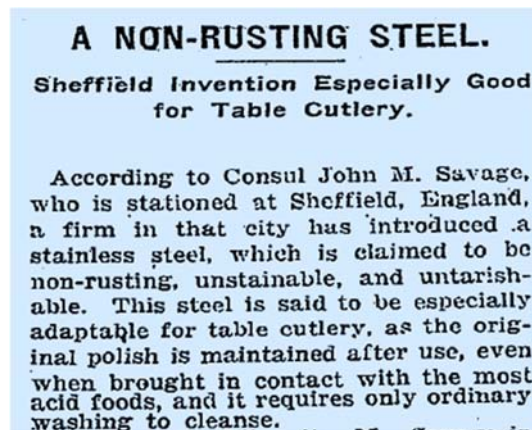


Figure 1.1 : An announcement, in the 1915 New York Times

1.1.3 Bridge projects with stainless steel

First footbridge in duplex stainless steel Likholefossen in Norway– 2004

Intended material was aluminum and replaced by Stainless LDX 2101 which needs no maintenance and gives cost efficiency, Aesthetics, stiffness, strength, low weight, easy erection (by helicopter).



Figure 1.2 : The world's first foot bridge Likholefossen

The Apate Bridge, Stockholm, Sweden

It is pedestrian bridge, built in 2004 and Outokumpu duplex grade 2205. It is supplied prefabricated segments at degerfors mill, tailor-made to customer requirements.

(Photo: SBI) The right hand photo shows the inner structure of the bridge body.



Figure 1.3 : The Apate Bridge,

The world's first stainless traffic road bridge Island of Menorca, Spain, 2005

It is replaced an existed concrete structure and built in the late sixties. In less than forty years have to replace due to corrosion. Its Steel grade is 2205. Figure 1.4 is showing both the finished bridge and the assembly of the bridge from prefabricated section (Photo: Pedelta),



Figure 1.4 : The Cala Galdana Bridge

The Nynäshamn composite bridge Sweden – 2011

Innovative design enabled use of sustainable stainless steel without cost disadvantage.

(Photos courtesy of NCC Construction)



Figure 1.5 : One of stainless beams of the load bearing structure.

First stainless railway bridge in the world Añorga Bridge – Spain, 2011

Old bridge was in carbon steel and replaced due to heavy corrosion damage. Hence, heavy maintenance demand. New bridge is in Duplex LDX 2101. The owner required 120 year design life and minimized maintenance is observed. LDX 2101 has light weight (high strength) structure due to access during construction. Thickness of members are 12-23 mm, and weight 130 tons.



Figure 1.6 : The entire truss structure

Across the Ljunga bay in Sölvesborg, Sweden

The bridge has 756 m total length, 3,5 m wide; 3 arches, 60 m span, carrying the higher deck section. Arches are made from duplex stainless steel plate S32101. Railings and deck support structure in lean duplex stainless steel S32101.

Ronny Södergren, MD, Stål- och Rörmontage AB said “Since we are building the bridge in stainless steel it will have a life time of several hundred years. The environmental impact will be zero compared to if you paint with anti-rust paint that peel off. Therefore Sölvesborgs town council selected durable high strength steel S32101”.



Figure 1.7 : The bridge ‘Stål- och Rörmontage AB, Sölvesborg’

1.2 Aim and Scope of the Study

The aim of the thesis is to investigate the structural behavior of these a stainless steel composite bridge and a comparison of the composite bridge versus a traditional post-tensioned bridge in Denmark. A 2D model has been designed to be optimal and tested under national annex for the loads defined in the Eurocodes. Guidelines from the literature has been used to determine the optimal geometry of the bridge.

Scope of Study is

- High level assessment
- Typical steel composite highway bridge
- Eurocodes
- Optimize for stainless steel and construction cost estimates.

2. DESIGN OF A STAINLESS STEEL COMPOSITE BRIDGE

Eurocodes which is used are a set of harmonized technical rules developed by the European Committee for Standardisation for the structural design of construction works in the European Union.

By March 2010, the Eurocodes are mandatory for the specification of European public works and are intended to become the de facto standard for the private sector. The Eurocodes therefore replace the existing national building codes published by national standard bodies, although many countries had a period of co-existence. Additionally, each country is expected to issue a National Annex to the Eurocodes, which will need referencing for a particular country. At present take up of Eurocodes is slow on private sector projects and existing national codes are still widely used by engineers.

European standards for structural design (Eurocodes) includes ten main subjects, covering Basis of Structural Design, Actions (Loading), Geotechnics, Earthquake Resistance, and each of the main structural materials, as follows:

EN 1990 Basis of structural design

EN 1991 Actions on structures

EN 1992 Design of concrete structures

EN 1993 Design of steel structures

EN 1994 Design of composite steel and concrete structures

EN 1995 Design of timber structures

EN 1996 Design of masonry structures

EN 1997 Geotechnical design

EN 1998 Design of structures for earthquake resistance

EN 1999 Design of aluminium structures

In this study, mainly used codes and guidelines for loads and design parameters:

Part of Eurocode 1 : Actions on structures	Title (Subject)	Issued
EN 1991-1-1	General actions – Densities, selfweight, imposed loads for buildings	April 2002
EN 1991-1-2	General actions – Actions on structures exposed to fire	November 2002
EN 1991-1-3	General actions – Snow loads	July 2003
EN 1991-1-4	General actions – Wind actions	April 2005
EN 1991-1-5	General actions – Thermal actions	November 2003
EN 1991-1-6	General actions – Actions during execution	June 2005
EN 1991-1-7	General actions – Accidental actions	July 2006
EN 1991-2	Traffic loads on bridges	September 2003
EN 1991-3	Actions induced by cranes and machinery	July 2006
EN 1991-4	Silos and tanks	May 2006

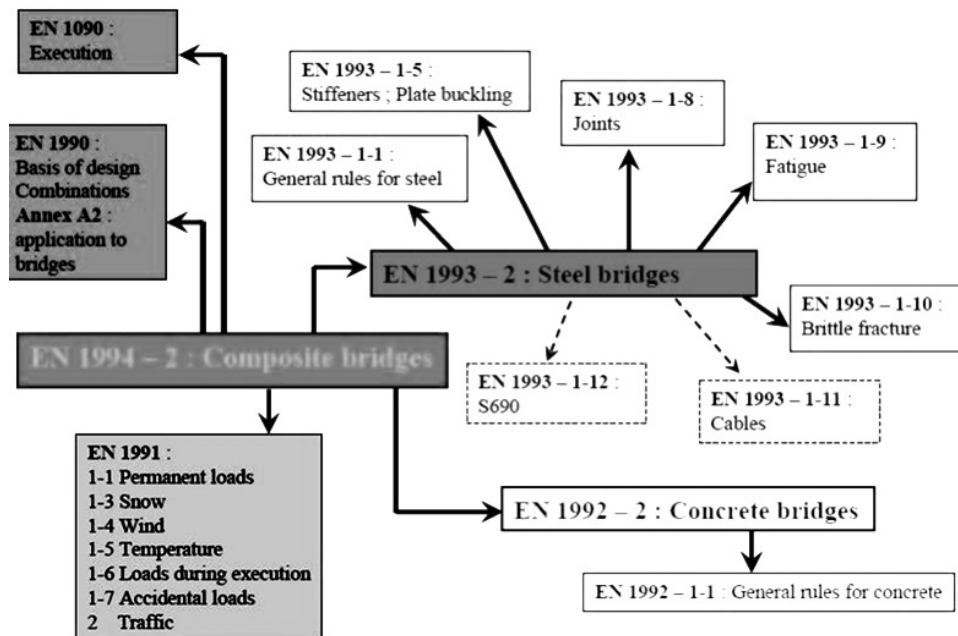


Figure 2.1 : Reference list of EN Norms

2.1 Design of Composite Steel and Concrete Structures-EN 1994

EN 1994-1-1 : general rules and rules for buildings

EN 1994-1-2: structural fire design

EN 1994-2: general rules and rules for bridges

EN 1994-1-1	
Forward	
Section 1	General
Section 2	Basis of design
Section 3	Material
Section 4	Durability
Section 5	Structural analysis
Section 6	ULS
Section 7	SLS
Section 8	Composite joints in frames for buildings
Section 9	Composite slabs for buildings
Annex A	(informative) Stiffness of joint in buildings
Annex B	(informative) Standard tests
Annex C	(informative) Shrinkage of concrete for buildings

5.1 Structural modelling for analysis
5.2 Structural stability
5.3 Imperfections
5.4 Calculation of action effects
5.5 Classification of cross-section

6.1 Beams
6.2 Resistance of cross-sections of beams
6.3 Filler beam decks
6.4 Lateral torsional buckling
6.5 Transverse forces on webs
6.6 Shear connection
6.7 Composite columns and composite compression members
6.8 Fatigue
6.9 Tension members in composite bridges

7.1 General
7.2 Stresses
7.3 Deformation of bridges
7.4 Cracking of concrete
7.5 Filler beam decks

Scope of EN 1994-1-1

Composite members

Composite beams

Composite columns

Composite slabs

Composite joints

EN 1994-2	
Forward	
Section 1	General
Section 2	Basis of design
Section 3	Material
Section 4	Durability
Section 5	Structural analysis
Section 6	ULS
Section 7	SLS
Section 8	Precast concrete slabs in bridges
Section 9	Composite plates in bridges
Annex C	Headed studs that cause splitting in the slab thickness

Figure 2.2 : Reference list of EN 1994

2.2 Stainless Steel as a Structural Material in Bridges

2.2.1 Materials: properties, selection and durability

2.2.1.1 Material grades

There are many different types of stainless steel. Not all of these are suitable for structural applications, particularly where welding is contemplated. There are five basic groups of stainless steel, classified according to their metallurgical structure: these are the austenitic, ferritic, martensitic, duplex and precipitation-hardening groups. The austenitic [1] stainless steels and the duplex stainless steels are generally the more useful groups for structural applications.

Austenitic stainless steels provide a good combination of corrosion resistance, forming and fabrication properties. Duplex stainless steels have high strength and wear resistance with very good resistance to stress corrosion cracking.

The most commonly used grades, typically referred to as the standard austenitic grades, are 1.4301 (widely known as 304) and 1.4401 (widely known as 316). They contain about 17-18% chromium and 8-11% nickel. Grade 1.4301 is suitable for rural, urban and light industrial sites whilst grade 1.4401 is a more highly alloyed grade and will perform well in marine and industrial sites.

2.2.2 Relevant standards

The relevant standard is EN 10088, stainless steels. It comprises three parts:

- Part 1, Lists of stainless steels, gives the chemical compositions and reference data on some physical properties such as modulus of elasticity, E.
- Part 2, Technical delivery conditions for sheet, plate and strip of corrosion resisting steels for general purposes, gives the technical properties and chemical compositions for the materials used in forming structural sections.
- Part 3, Technical delivery conditions for semi-finished products, bars, rods, wire, sections and bright products of corrosion resisting steels for general purposes, gives the technical properties and chemical compositions for the materials used in long products.
- Parts 4 (flat products) and 5 (long products) of EN 10088 are now in preparation to cover material for construction purposes.

The designation systems adopted in EN 10088 are the European steel number and a steel name.

For example, grade 304L has a steel number 1.4307, where:

1.	43	07
Denotes steel	Denotes one group of stainless steels	Individual grade identification

The steel name system provides some understanding of the steel composition.

The name of the steel number 1.4307 is X2CrNi18-9, where:

X	2	CrNi	18-9
Denotes high alloy steel	100 x % of carbon	Chemical symbols of main alloying elements	%of main alloying elements

Both austenitic and duplex stainless steels can be assumed to be adequately tough and not susceptible to brittle fracture for service temperatures down to – 40°C.

Table 2.1 : Specified mechanical properties of common stainless steels

	Grade	Product form ⁽¹⁾	Max thickness (mm)	Minimum 0.2% proof strength ⁽²⁾ (N/mm ²)	Ultimate tensile strength (N/mm ²)	Elongation after fracture (%)
Basic chromium-nickel austenitic steels	1.4301	C	8	230	540 – 750	45 ⁽³⁾
		H	13,5	210	520 – 720	45 ⁽³⁾
		P	75	210	520 – 720	45
	1.4307	C	8	220	520 – 700	45
		H	13,5	200	520 – 700	45
		P	75	200	500 – 700	45
Molybdenum-chromium-nickel austenitic steels	1.4401	C	8	240	530 – 680	40
		H	13,5	220	530 – 680	40
		P	75	220	520 – 670	45
	1.4404	C	8	240	530 – 680	40
		H	13,5	220	530 – 680	40
		P	75	220	520 – 670	45

Stabilised austenitic steels	1.4541	C	8	220	520 – 720	40
		H	13,5	200	520 – 720	40
		P	75	200	500 – 700	40
	1.4571	C	8	240	540 – 690	40
		H	13,5	220	540 – 690	40
		P	75	220	520 – 670	40
Low carbon, high nitrogen austenitic steel	1.4318	C	8	350	650 – 850	35
		H	13,5	330	650 – 850	35
		P	75	330	630 – 830	45
Duplex steels	1.4362	C	8	450	650 – 850	20
		H	13,5	400	650 – 850	20
		P	75	400	630 – 800	25
	1.4462	C	8	500	700 – 950	20
		H	13,5	460	700 – 950	25
		P	75	460	640 – 840	25
Notes:						
(1) C = cold rolled strip, H = hot rolled strip, P = hot rolled plate						
(2) Transverse properties						
(3) For stretcher levelled material, the minimum value is 5% lower						

Table 2.2 : Chemical composition to EN 10088-2 [1]

Grade	Content of alloying element (maximum or range permitted) weight %				
	C	Cr	Ni	Mo	Others
Austenitic steels	1.4301	0,07	17,5 – 19,5	8,0 – 10,5	
	1.4307	0,03	17,5 – 19,5	8,0 – 10,5	
	1.4401	0,07	16,5 – 18,5	10,0 – 13,0	2,0 – 2,5
	1.4404	0,03	16,5 – 18,5	10,0 – 13,0	2,0 – 2,5
	1.4541	0,08	17,0 – 19,0	9,0 – 12,0	Ti: 5xC – 0,7 ⁽¹⁾
	1.4571	0,08	16,5 – 18,5	10,5 – 13,5	2,0 – 2,5 Ti: 5xC – 0,7 ⁽¹⁾
	1.4318	0,03	16,5 – 18,5	6,0 - 8,0	N: 0,1 – 0,2
Duplex steels	1.4362	0,03	22,0 – 24,0	3,5 – 5,5	0,1 – 0,6 N: 0,05 – 0,2
	1.4462	0,03	21,0 – 23,0	4,5 – 6,5	2,5 – 3,5 N: 0,1 – 0,22
Note:					
(1) Titanium is added to stabilise carbon and improve corrosion performance in the heat affected zones of welds. However, except for very heavy section construction, the use of titanium stabilised austenitic steels has been superseded largely by the ready availability of the low carbon grades, 1.4307 and 1.4404.					

2.2.3 Mechanical behaviour and design values of properties

The stress-strain behaviour of stainless steels differs from that of carbon steels in a number of respects. The most important difference is in the shape of the stress-strain curve. Whereas carbon steel typically exhibits linear elastic behaviour up to the yield stress and a plateau before strain hardening is encountered, stainless steel has a more rounded response with no well-defined yield stress (see Figure 2.1). Therefore, stainless steel “yield” strengths are generally quoted in terms of a proof strength defined for a particular offset permanent strain (conventionally the 0,2% strain), as indicated in the figure 2.1.

Note that figure 2.1 shows typical experimental stress-strain curves. The curves shown are representative of the range of material likely to be supplied and should not be used in design.

Stainless steels can absorb considerable impact without fracturing due to their excellent ductility (especially the austenitic grades) and their strain hardening characteristics.

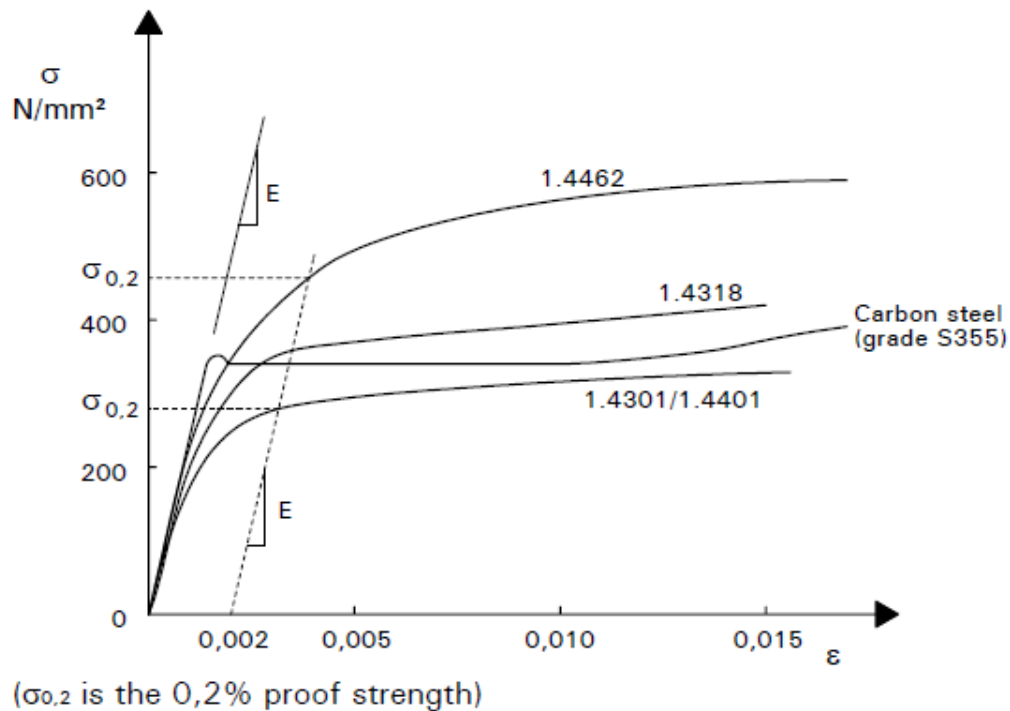


Figure 2.1 : Typical stress-strain curves for stainless steel and carbon steel in the annealed condition (for longitudinal tension) [1]

2.2.4 Factors affecting stress-strain behaviour

There are factors that can change the form of the basic stress-strain curve for any given grade of stainless steel. These factors are to some extent interdependent and include:

2.2.5 Cold working

Strength levels of austenitic and duplex grades are enhanced by cold working (such as imparted during cold forming operations including roller levelling/flattening and also during fabrication). Associated with this enhancement is a reduction in ductility but this normally is of slight consequence due to the initial high values of ductility, especially for the austenitic stainless steels.

Table 2.3 gives the cold worked levels specified in EN 1993-1-4 which are taken from the European material standard for stainless steel, EN 10088. Cold worked steels may be specified either in terms of minimum 0,2% proof strength or ultimate tensile strength or hardness, but only one parameter can be specified.

As stainless steel is cold worked, it tends to exhibit non-symmetry of tensile and compressive behaviour and anisotropy (different stress-strain characteristics parallel and transverse to the rolling directions). The degree of asymmetry and anisotropy depends on the grade, level of cold working and manufacturing route. Figure 2.2 shows stress-strain curves for grade 1.4318 cold worked to C850; the compression strength in the longitudinal direction lies below the tensile strength in both the transverse and longitudinal direction (the values traditionally given in material standards such as EN 10088 and reported accordingly by suppliers). Care is therefore needed in the choice of design strength for cold worked material. Additional information about values related to other types or directions of loading should be sought from the supplier.

The price of cold worked stainless steel is slightly higher than the equivalent annealed material, depending on the grade, product form and level of cold working.

During the fabrication of a section, a 0, 2% proof strength enhancement by a factor of about 50% is typical in cold formed corners of cross sections. However, the effect is localised and the increase in member resistance is dependent on the location of the corners within the section; e.g. in a beam, little benefit would be obtained for corners close to the neutral axis. The strength enhancement easily compensates for any effect

Table 2.3 : Cold worked strength levels in EN 10088-2

Cold worked condition	Minimum 0,2% proof strength ^(1) 2)	Minimum tensile strength ^(1) 2)	Available steel grades in the cold worked condition
CP350	350	700 ³⁾	1.4301, 1.4541, 1.4401, 1.4571
CP500	500	850 ³⁾	1.4301, 1.4541, 1.4401, 1.4571, 1.4318
CP700	700	1000 ³⁾	1.4318, 1.4301
C700	350 ³⁾	700	1.4301, 1.4541, 1.4401, 1.4571
C850	500 ³⁾	850	1.4301, 1.4541, 1.4401, 1.4571, 1.4318
C1000	700 ³⁾	1000	1.4318, 1.4301

Note:

(1) Intermediate 0,2% proof strength or tensile strength values may be agreed.

(2) Maximum product thickness for each strength level decreases with strength. Maximum product thickness and remaining elongation are also dependent on the work hardening behaviour of the steel and the cold working conditions - more exact information may be requested from the manufacturer.

(3) Not specified, indicative minimum values.

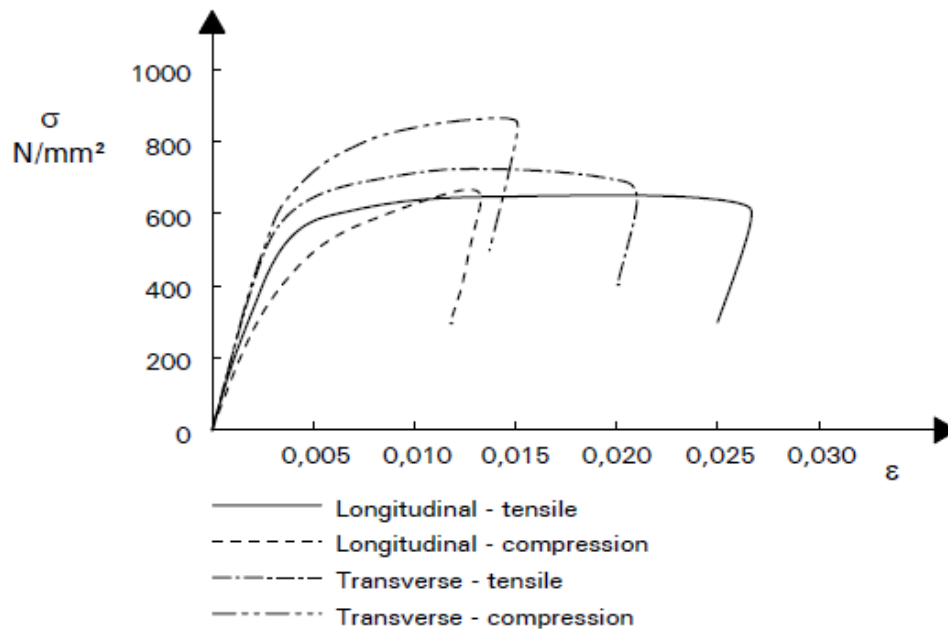


Figure 2.2 : Typical stress-strain curves for grade 1.4318 cold worked to strength level C850 [1]

due to thinning of the material at cold worked corners. If advantage is to be made of localised increased strength arising from fabrication, this should be proved by testing.

2.2.6 Strain-rate sensitivity

Strain-rate sensitivity is more pronounced in stainless steels than in carbon steels. That is, a proportionally greater strength can be realised at fast strain rates for stainless steel than for carbon steel.

2.2.7 Heat treatment

Annealing, or softening, reduces the strength enhancement and the anisotropy.

2.2.8 Typical values of properties

It should be apparent that more factors are involved when considering the mechanical properties of stainless steels than for carbon steels. Their metallurgy is more complex and the manufacturing process has a higher impact on their final properties. For any given grade, it is to be expected that there will be differences in properties for materials made by different manufacturers. However, the mechanical properties, being dependent on chemical composition and thermo-mechanical treatment, are therefore largely under the control of the manufacturers and it is possible to negotiate desired properties with individual manufacturers.

From a structural point of view, the margin by which the actual 0,2% proof stress exceeds the minimum specified value is significant. Typical mean proof stresses lie between 20 and 35% above the specified minima. The proportions of enhancement observed for proof stresses are not shared by ultimate tensile strength values, which typically are only about 10% above specified minima.

2.2.9 Design values of properties

2.2.9.1 Flat products

Three options may be considered: minimum specified values, verified material test data or mill certificate data.

(i) Design using minimum specified values

Annealed material

Take the characteristic yield strength, f_y , and the characteristic ultimate tensile strength, f_u as the minimum values specified in EN 10088-2 (given in Table 2.1).

Cold worked material

Increased nominal values of f_y and f_u may be adopted for material delivered in the cold worked conditions specified in EN 10088.

For material delivered to a specified 0,2% proof strength (e.g. CP350), the minimum 0,2% proof strength in Table 2.3 may be taken as the characteristic strength. To take into account asymmetry of the cold worked material in those cases where compression in the longitudinal direction is a relevant stress condition (i.e. column behaviour or bending), the characteristic value should be taken as $0.8 \times 0.2\%$ proof strength in Table 2.3. A higher value may be used if supported by appropriate experimental data.

For material delivered to a specified tensile strength (e.g. C700), the minimum tensile strength in Table 2.5 may be taken as the characteristic strength; the minimum 0,2% proof strength should be obtained from the supplier.

This should only be considered as an option where tensile testing has been carried out on coupons cut from the plate or sheet from which the members are to be formed or fabricated. The designer should also be satisfied that the tests have been carried out to a recognised standard, e.g. EN 10002-1, and that the procedures adopted by the fabricator are such that the member will be actually made from the tested material and positioned correctly within the structure.

It is recommended that the characteristic ultimate tensile strength, f_u , should still be based on the specified minimum value given in EN 10088-2.

2.2.9.2 Physical properties

Table 2.4 gives the room temperature physical properties in the annealed condition of the range of EN 10088. Physical properties may vary slightly with product form and size but such variations are usually not of critical importance to the application.

From a structural point of view, the most important physical property is the coefficient of linear expansion which, for the austenitic grades, differs considerably from that for carbon steel ($12 \times 10^{-6}/^{\circ}\text{C}$). Where carbon and stainless steel are used together, the effects of differential thermal expansion should be considered in design.

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Table 2.4 : Room temperature physical properties, annealed condition [1]

Grade	Density (kg/m ³)	Thermal expansion 20 – 100°C (10 ⁻⁶ /°C)	Thermal conductivity (W/m°C)	Heat capacity (J/kg°C)
1.4301	7900	16	15	500
1.4307	7900	16	15	500
1.4401	8000	16	15	500
1.4404	8000	16	15	500
1.4541	7900	16	15	500
1.4571	8000	16,5	15	500
1.4318	7900	16	15	500
1.4362	7800	13	15	500
1.4462	7800	13	15	500

Where carbon and stainless steel are used together, the effects of differential thermal expansion should be considered in design.

Duplex and ferritic grades are magnetic. Where the non-magnetic properties of the austenitic grades are important to the application, care must be exercised in selecting appropriate welding consumables to minimise the ferrite content in the weldment. Heavy cold working, particularly of the lean alloyed austenitic steel, can also increase magnetic permeability; subsequent annealing would restore the non-magnetic properties. For non-magnetic applications, it is recommended that further advice be obtained from a steel producer.

2.2.10 Effects of temperature

Austenitic grades are used for cryogenic applications. At the other end of the temperature scale, austenitic and duplex grades retain a higher proportion of their strength above about 550°C than carbon steel.

Duplex steels should not be used for long periods at temperatures above about 300°C, due to the possibility of embrittlement.

2.2.11 Life cycle costing

There is increasing awareness that life cycle (or whole life) costs, not just initial costs, should be considered when selecting materials. Life cycle costs take account of:

- Initial costs,

- Operating costs,
- Residual value.

Stainless steel is sometimes considered to be an expensive material. However, experience has shown that using a corrosion resistant material in order to avoid future maintenance, downtime and replacement costs can save costs which far outweigh higher initial material costs.

The initial cost of a structural stainless steel product is considerably higher than that of an equivalent carbon steel product, depending on the grade of stainless steel. However, savings will arise from the omission of surface coatings at regular (repeated) intervals in time.

The excellent corrosion resistance of stainless steel can offer many benefits including:

- Reduced inspection frequency and costs,
- Reduced maintenance costs,
- Long service life.

Stainless steel has a high residual value (i.e. value at the end of a structure's life), though this is rarely a deciding factor for a structure with a long projected life (for instance over 50 years).

Life cycle costing uses the standard accountancy principle of discounted cash flow to reduce all those costs to present day values. The discount rate encompasses inflation, bank interest rates, taxes and, possibly, a risk factor. This allows a realistic comparison to be made of the options available and the potential long-term benefits of using stainless steel to be assessed against other material selections.

2.2.12 Selection of materials

In the great majority of structural applications utilising stainless steel, it is the metal's corrosion resistance, which is being exploited, whether this be for reasons of aesthetics, minimal maintenance or long-term durability. Corrosion resistance must therefore be the primary factor in choosing a suitable grade. Stainless steels derive their corrosion resistance from the presence of a passive surface film which, given adequate access to oxygen or suitable oxidising agents, tends to be self-healing when damaged. This oxide film is primarily a consequence of the chromium content of the steel, though the addition of nickel and other alloying elements can substantially

enhance the protection offered by the film. In particular, a small percentage of molybdenum is used to improve the pitting resistance of the steel.

It is when the surface oxide film is damaged, possibly by electro-chemical attack or by mechanical damage, that corrosion might initiate.

Careful design should ensure trouble-free performance, but designers should be aware that even stainless steels may be subject to various forms of corrosion under certain circumstances. Notwithstanding the existence of these degradation effects, it is perfectly possible to employ stainless steels extremely effectively, provided that a few elementary principles are kept in mind. It is only when these materials are used without consideration for the principles behind their corrosion properties that problems might be encountered.

The selection of the correct grade of stainless steel must take into account the environment of the application, the fabrication route, surface finish and the maintenance of the structure. It might be noted that the maintenance requirement is minimal: merely washing down the stainless steel, even naturally by rain, will markedly assist in extending the service life.

The first step is to characterise the service environment, including reasonably anticipated deviations from the design conditions. In categorising atmospheric environments, special attention should be given to highly localised conditions such as proximity to chimneys venting corrosive fumes. Possible future developments or change of use should also be considered. The surface condition and the temperature of the steel, and the anticipated stress, could also be important parameters. Candidate grades can then be chosen to give overall satisfactory corrosion resistance in the environment. The selection of a candidate steel should consider which possible forms of corrosion might be significant in the operating environment. To do this requires some appreciation of the nature of corrosion found in stainless steels. It is intended to illustrate general points of good practice, as well as the circumstances where stainless steels might have to be used with caution. In these latter conditions specialist advice should be sought, for in many cases the steels can still be successfully used.

Assessing the suitability of grades is best approached by referring to experience of stainless steels in similar applications and environments. Table 2.5 gives guidance for selecting suitable grades for atmospheric environments. National regulations should

also be checked, since in some cases they may be more onerous. In the case of immersed stainless steel. When stainless steel comes into contact with chemicals, expert advice should always be sought.

Caution should be exercised when considering the use of free-machining stainless steels for fasteners. The addition of sulfur in the composition of these steels in the austenitic class renders them more liable to corrosion, especially in industrial and marine environments. In particular, this applies to fasteners in EN ISO 3506 grade A1 materials.

Table 2.5 : Suggested grades for atmospheric applications [1]

Steel grade	Location											
	Rural			Urban			Industrial			Marine		
	L	M	H	L	M	H	L	M	H	L	M	H
Basic chromium-nickel austenitic steels (e.g. 1.4301, 1.4307, 1.4541, 1.4318)	✓	✓	✓	✓	✓	(✓)	(✓)	(✓)	X	✓	(✓)	X
Molybdenum-chromium-nickel austenitic steels (e.g. 1.4401, 1.4404, 1.4571) and duplex 1.4362	0	0	0	0	✓	✓	✓	✓	(✓)	✓	✓	(✓)
Duplex steel grade 1.4462	0	0	0	0	0	0	0	0	✓	0	0	✓
L Least corrosive conditions within that category, e.g. tempered by low humidity, low temperatures. M Fairly typical of that category. H Corrosion likely to be higher than typical for that category, e.g. increased by persistent high humidity, high ambient temperatures, and particularly aggressive air pollutants. 0 Potentially over-specified from a corrosion point of view. ✓ Probably the best choice for corrosion resistance and cost. X Likely to suffer excessive corrosion. (✓) Worthy of consideration if precautions are taken (i.e. specifying a relatively smooth surface and if regular washing is carried out). NOTE: National regulations may contain more onerous requirements.												

2.2.13 Availability of product forms

Sheet, plate and bar products are all widely available in the grades of stainless steel considered in this Design Manual. Tubular products are available in austenitic grades and also the duplex grade 1.4462 (2205). Tubular products in the duplex grade 1.4362 (2304) are not widely available as this is a relatively new grade to the construction industry, although it has been used for some years for offshore blast walls.

There is a range of rolled sections (angles, channels, tees, rectangular hollow sections and I-sections) in standard austenitic grades such as 1.4301 and 1.4401 but none for duplex grades. Generally, sections may be produced by cold forming (rolling or bending), or fabricated by welding.

Material in the cold worked condition is available in various product forms including plate, sheet, coil, strip, bars and hollow sections:

- Plate, sheet, coil, strip (in thicknesses typically ≤ 6.0 mm)
- Round bar (diameters from 5 mm to 60 mm)
- Square and rectangular hollow sections (cross-section dimensions up to 40 mm, thicknesses from 1.2 to 6 mm).

The grades of stainless steel which are commercially available in the cold worked condition are given in Table 2.3.

2.2.14 Cold forming

It is important that early discussion with potential fabricators takes place to ascertain cold forming limits as stainless steels require higher forming loads than carbon steels. The length of brake-pressed cold formed sections is necessarily limited by the size of machine or by power capability in the case of thicker or stronger materials. Duplex grades require approximately twice the forming loads used for the austenitic materials and consequently the possible range of duplex sections is more limited. Furthermore, because of the lower ductility in the duplex material, more generous bending radii should be used.

2.2.15 Surface finish

In certain applications, surface finish and appearance are important. Manufacturers offer a range of standard finishes, from mill finish through dull finishes to bright polish. They may also offer proprietary textured finishes. It should be noted that although the various finishes are standardised, variability in processing introduces differences in appearance between manufacturers and even from a single producer. Bright finishes are frequently used in architectural applications and it should be noted that bright finishes will exaggerate any out-of-flatness of the material, particularly on panel surfaces. Rigidised, embossed, textured, patterned or profiled sheets with a rigid supporting frame will alleviate this tendency.

2.2.16 Durability

Stainless steels are generally very corrosion resistant and perform satisfactorily in most environments. The limit of corrosion resistance of a given stainless steel depends on its constituent elements, which means that each grade has a slightly different response when exposed to a corrosive environment.

Care is therefore needed to select the most appropriate grade of stainless steel for a given application. Generally, the higher the level of corrosion resistance required, the greater the cost of the material. For example, grade 1.4401 steel costs more than grade 1.4301 because of the addition of molybdenum.

Material in the cold worked condition has a similar corrosion resistance to that in the annealed condition.

The most common reasons for a metal to fail to live up to expectations regarding corrosion resistance are:

- (a) Incorrect assessment of the environment or exposure to unexpected conditions, e.g. unsuspected contamination by chloride ions
- (b) The way in which the stainless steel has been worked or treated may introduce a state not envisaged in the initial assessment.

Although stainless steel may be subject to discolouration and staining (often due to carbon steel contamination), it is extremely durable in buildings. In aggressive industrial and marine environments, tests have shown no indication of reduction in component capacity even where a small amount of weight loss occurred. However, the user may still regard unsightly rust staining on external surfaces as a failure. As well as careful material grade selection, good detailing and workmanship can significantly reduce the likelihood of staining and corrosion.

In certain aggressive environments, some grades of stainless steel will be susceptible to localised attack. Six mechanisms are described below although the last three are very rarely encountered in buildings onshore.

2.2.17 Types of corrosion and performance of steel grades

As the name implies, pitting takes the form of localised pits. It occurs as a result of local breakdown of the passive layer, normally by chloride ions although the other halides and other anions can have a similar effect. In a developing pit, corrosion

products may create a very corrosive solution, often leading to high corrosion rates. In most structural applications, the extent of pitting is likely to be superficial and the reduction in section of a component is negligible. However, corrosion products can stain architectural features. A less tolerant view of pitting should be adopted for services such as ducts, piping and containment structures.

Since the chloride ion is by far the most common cause of pitting, coastal and marine environments are rather aggressive. The probability of a certain medium causing pitting depends on, besides the chloride content, factors such as the temperature, acidity or alkalinity and the content of oxidising agents. The pitting resistance of a stainless steel is dependent on its chemical composition. Chromium, molybdenum and nitrogen all enhance the resistance to pitting.

An approximate measure of pitting resistance is given by the Pitting Index or Pitting Resistance Equivalent (PRE) defined as:

$PRE = \% \text{ wt Cr} + 3,3(\% \text{ wt Mo}) + 30(\% \text{ wt N})$ for austenitic grades

$PRE = \% \text{ wt Cr} + 3,3(\% \text{ wt Mo}) + 16(\% \text{ wt N})$ for duplex grades

2.2.18 Crevice corrosion

Crevice corrosion occurs in the same environments as pitting corrosion. Corrosion initiates more easily in a crevice than on a free surface because the diffusion of oxidants necessary for maintaining the passive film is restricted. The severity of a crevice is very dependent on its geometry: the narrower and deeper the crevice, the more severe the corrosion conditions. It is only likely to be a problem in stagnant solutions where a build up of chlorides can occur.

Crevices may result from a metal-to-metal joint, a gasket, biofouling, deposits and surface damage such as deep scratches. Every effort should be made to eliminate crevices, but it is often not possible to eliminate them entirely.

2.2.19 Bimetallic (galvanic) corrosion

When two dissimilar metals are in electrical contact and are also bridged by an electrolyte (i.e. an electrically conducting liquid such as seawater or impure fresh water), a current flows from the anodic metal to the cathodic or nobler metal through the electrolyte. As a result, the less noble metal corrodes.

This form of corrosion is particularly relevant when considering joining stainless steel and carbon or low alloy steels. It is important to select welding consumables that are at least as noble as the parent material. In corrosive environments where water may be present such as heavy industrial environments, marine atmospheres, and where immersion in brackish or seawater may occur, martensitic and ferritic bolts should be avoided for joining austenitic stainless steels.

Bimetallic corrosion need not be a problem with stainless steels, though sometimes its prevention can require precautions that at first sight might seem surprising. The prevention of bimetallic corrosion, in principle, is to prevent current flow by:

- insulating dissimilar metals, i.e. breaking the metallic path.
- preventing electrolyte bridging, i.e. breaking the electrolytic path by paint or other coating. Where protection is sought by this means and it is impracticable to coat both metals, then it is preferable to coat the more noble one (i.e. stainless steel in the case of a stainless/carbon steel connection).

The risk of a deep corrosion attack is greatest if the area of the more noble metal (i.e. stainless steel) is large compared with the area of the less noble metal (i.e. carbon steel). Special attention should be paid to the use of paints or other coatings on the carbon steel. If there are any small pores or pinholes in the coating, the small area of bare carbon steel will provide a very large cathode/anode area ratio, and severe pitting of the carbon steel may occur. This is, of course, likely to be most severe under immersed conditions. For this reason it is preferable to paint the stainless steel; any pores will lead to small area ratios.

Adverse area ratios are likely to occur with fasteners and at joints. Carbon steel bolts in stainless steel members should be avoided because the ratio of the area of the stainless steel to the carbon steel is large and the bolts will be subject to aggressive attack. Conversely, the rate of attack of a carbon steel member by a stainless steel bolt is much slower. It is usually helpful to draw on previous experience in similar sites because dissimilar metals can often be safely coupled under conditions of occasional condensation or dampness with no adverse effects, especially when the conductivity of the electrolyte is low.

The prediction of these effects is difficult because the corrosion rate is determined by a number of complex issues. The use of electrical potential tables ignores the presence

of surface oxide films and the effects of area ratios and different solution (electrolyte) chemistry. Therefore, uninformed use of these tables may produce erroneous results. They should be used with care and only for initial assessment. Stainless steels usually form the cathode in a bimetallic couple and therefore do not suffer corrosion. Contact between austenitic stainless steels and zinc or aluminium may result in some additional corrosion of the latter two metals. This is unlikely to be significant structurally, but the resulting white/grey powder may be deemed unsightly. The couple with copper should generally be avoided except under benign conditions.

The general behaviour of metals in bimetallic contact in rural, urban, industrial and coastal environments is fully documented in BS PD 6484 Commentary on corrosion at bimetallic contacts and its alleviation.

2.2.20 Stress corrosion cracking

The development of stress corrosion cracking (SCC) requires the simultaneous presence of tensile stresses and specific environmental factors unlikely to be encountered in normal building atmospheres. The stresses do not need to be very high in relation to the proof stress of the material and may be due to loading, residual effects from manufacturing processes such as welding or bending. Duplex stainless steels usually have superior resistance to stress corrosion cracking than the austenitic stainless steels. Higher alloy austenitic stainless steels such as grades 1.4539, 1.4529, 1.4547 and 1.4565 have been developed for applications where SCC is a corrosion hazard.

Caution should be exercised when stainless steel members containing high residual stresses (e.g. due to cold working) are used in chloride rich environments (e.g. indoor swimming pools, marine, offshore). EN 1993-1-4 advises that for load-bearing members in atmospheres containing chlorides that cannot be cleaned regularly (e.g. in suspended ceilings above swimming pools), only grades 1.4529, 1.4547, 1.4565 should be used, unless the concentration of chloride ions in the pool water is (unusually) ≤ 250 mg/l, in which case grade 1.4539 is also suitable. Alternative grades that have been shown to have equivalent resistance to stress corrosion cracking in these atmospheres may also be used.

2.2.21 General (uniform) corrosion

Under normal conditions typically encountered in structural applications, stainless steels do not suffer from the general loss of section that is characteristic of rusting in non-alloyed irons and steels.

Stainless steel is resistant to many chemicals; indeed, it is sometimes used for their containment. However, reference should be made to tables in manufacturers' literature, or the advice of a competent corrosion engineer should be sought, if the stainless steel is exposed to chemicals.

2.2.22 Intergranular corrosion (sensitisation) and weld decay

When austenitic stainless steels are subject to prolonged heating in the range 450°C to 850°C, the carbon in the steel diffuses to the grain boundaries and precipitates chromium carbide. This removes chromium from the solid solution and leaves a lower chromium content adjacent to the grain boundaries. Steel in this condition is termed sensitized. The grain boundaries become prone to preferential attack on subsequent exposure to a corrosive environment. This phenomenon is known as weld decay when it occurs in the heat-affected zone of a weldment.

There are three ways to avoid intergranular corrosion:

- use steel having a low carbon content
- use steel stabilised with titanium or niobium, because these elements combine preferentially with carbon to form stable particles, thereby reducing the risk of forming chromium carbide
- use heat treatment, however this method is rarely used in practice. Grades of stainless steel with a low carbon content (0, 03% maximum) up to 20 mm thick should not suffer from intergranular corrosion after arc welding.

2.2.23 Corrosion in selected environments

2.2.23.1 Air

Atmospheric environments vary, as do their effect on stainless steels. Rural atmospheres, uncontaminated by industrial fumes or coastal salt, are very mild in terms of corrosivity, even in areas of high humidity. Industrial and marine atmospheres are

considerably more severe. Table 2.5 should be referred to for guidance on selecting suitable types of stainless steel.

The most common causes of atmospheric corrosion are metallic iron particles, arising from fabrication operations either in the workshop or at site, and chlorides originating from the sea, industrial processes or from calcium chloride used to make cement. Some deposited particles, although inert, are able to absorb weak acid solutions of sulfur dioxide from the atmosphere, which may locally break down the passive film. General appearance of exposed stainless steel is affected by surface finish (the smoother the better) and whether or not regular washing down is carried out (either intentionally or by rain).

2.2.23.2 Seawater

Seawater, including brackish water, contains high levels of chloride and hence is very corrosive, particularly when the water current is low (under about 1.5 m/s). At low current flows, severe pitting of grades 1.4301 and 1.4401 can occur. In addition, these grades can suffer attack at crevices, whether these result from design details or from fouling organisms such as barnacles.

Salt spray may cause as much attack as complete immersion because the chloride concentration is raised by the evaporation of water or because of salt crystal deposits. The possibility of severe bimetallic corrosion must be considered if stainless steel is used with other metals in the presence of seawater.

2.2.23.3 Other waters

Austenitic stainless steels usually perform satisfactorily in distilled, tap and boiler waters. Where acidity is high, grade 1.4401 is to be preferred, otherwise 1.4301 will usually suffice. Grade 1.4401 is also suggested as being more suitable where there are minor amounts of chloride present to avoid possible pitting and crevice corrosion problems. River water needs special consideration; biological and microbiological activity can cause pitting in austenitic stainless steels within a comparatively short time.

The possibility of erosion-corrosion should be considered for waters containing abrasive particles.

2.2.23.4 Soils

External corrosion of buried stainless steel is dependent on soil chemistry and resistivity. Soils differ in their corrosiveness depending on moisture level, pH, and aeration, presence of chemical contamination, microbiological activity and surface drainage. Stainless steels generally perform well in a variety of soils and especially well in soils with high resistivity, although some pitting has occurred in low resistivity, moist soils. The presence of aggressive chemical species such as chloride ions as well as types of bacteria and stray current (caused by local direct current electric transportation systems such as railways or tram systems) can cause localized corrosion. The development of stray current can be suppressed with a proper electrical insulation of the pipe (coatings or wrappings) and/or cathodic protection.

For grade selection purposes, it is recommended to consider the corrosion resistance of buried stainless steel firstly in relation to the presence of chloride ions and secondly according to the soil resistivity and pH, assuming poorly drained soils in all cases. Table 2.6 recommends suitable grades for different soil conditions.

Table 2.6 : Stainless steel grades for use in different soil conditions [1]

Typical location	Soil condition	Grade of stainless steel
Inland	Cl < 500 ppm	1.4301
	Resistivity > 1000 ohm.cm	1.4404
	pH > 4.5	
Marine – non-tidal zone	Cl < 1500 ppm	
	Resistivity > 1000 ohm.cm	1.4404
	pH > 4.5	
Marine – tidal zone	Cl < 6000 ppm	1.4410
	Resistivity > 500 ohm.cm	1.4547
	pH > 4.5	
Note: 1.4410 is a duplex grade and 1.4547 is a super-austenitic grade. These grades are not generally used in construction applications and fall outside the scope of this Design Manual.		

2.2.23.5 Design for corrosion control

The most important step in preventing corrosion problems is selecting an appropriate grade of stainless steel with suitable fabrication procedures for the given environment. However, after specifying a particular steel, much can be achieved in realising the full potential of the steel's resistance by careful attention to detailing. Anti-corrosion actions should ideally be considered at the planning stage and during detailed design.

Table 2.7 gives a checklist for consideration. Not all points would give the best detail from a structural strength point of view and neither are the points intended to be applied to all environments. In particular, in environments of low corrosivity or where regular maintenance is carried out, many would not be required. Figure 2.3 illustrates poor and good design features for durability.

Table 2.7 : Design for corrosion control [1]

Avoid dirt entrapment • orientate angle and channel profiles to minimise the likelihood of dirt retention • provide drainage holes, ensuring they are of sufficient size to prevent blockage • avoid horizontal surfaces • specify a small slope on gusset stiffeners which nominally lie in a horizontal plane • use tubular and bar sections (seal tubes with dry gas or air where there is a risk of harmful condensates forming) • specify smooth finishes.
Avoid crevices • use welded rather than bolted connections • use closing welds or mastic fillers • Preferably, dress/profile welds • prevent biofouling.
Reduce the likelihood of stress corrosion cracking in those specific environments where it may occur : • minimise fabrication stresses by careful choice of welding sequence • shot peen (but avoid the use of iron/steel shot).
Reduce likelihood of pitting : • remove weld spatter • pickle stainless steel to remove unwanted welding products. Strongly oxidising chloride-containing reagents such as ferric chloride should be avoided; rather a pickling bath or a pickling paste, both containing a mixture of nitric acid and hydrofluoric acid should be used. Welds should always be cleaned up to restore corrosion resistance.

Table 2.7 : (Continued) Design for corrosion control [1]

• avoid pick-up of carbon steel particles (e.g. use workshop area and tools dedicated to stainless steel) • follow a suitable maintenance programme.
Reduce likelihood of bimetallic corrosion : • provide electrical insulation • use paints appropriately • minimise periods of wetness • use metals that are close to each other in electrical potential.

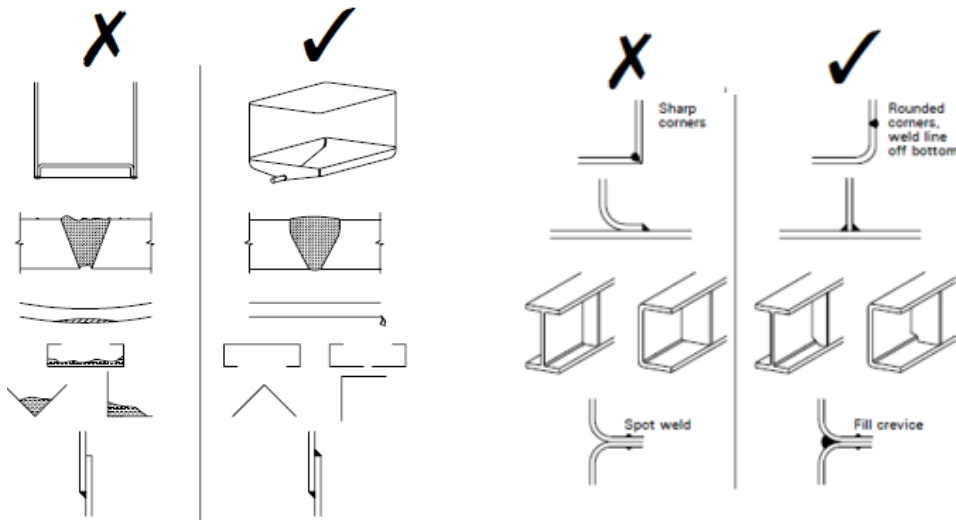


Figure 2.3 : Poor and good design features for durability [1]

2.2.24 Comparison of structural behaviour of stainless and carbon steel

The main reasons for the difference in structural behaviour between carbon and stainless steel members are:

- The stress-strain curve for stainless steel departs from linearity at a much lower stress than that for carbon steels
- Stainless steels have greater ductility and a greater capacity for work hardening than carbon steels
- The material modulus of stainless steels reduces with increasing stress, unlike that of carbon steels which is constant
- The residual stresses arising from fabrication are higher in stainless steel than in carbon steels.

As a result of this, different buckling curves are required from those of carbon steel. This applies to:

- local (plate) buckling for elements in compression
- flexural, torsional, torsional-flexural buckling for members subject to axial compression.
- lateral-torsional buckling for beams with unrestrained compression flanges.

2.2.25 Estimation of deflections

As the stiffness of stainless steel decreases as the stress level increases, deflections are greater than those for carbon steel members. It is therefore necessary to use a reduced modulus to predict the behaviour of members in which high stresses occur. This is explained in the article Calculating the deflections of stainless steel beams.

2.2.26 Properties of sections

The width-to-thickness ratios of elements that are partly or wholly in compression determine whether they are subject to local buckling, with a consequential reduction in the resistance of the cross-section. Elements and cross-sections are classified as Class 1, 2, 3 or 4 depending on the susceptibility to local buckling and their rotation capacity (Class 1 and 2).

The reduced resistance of Class 4 cross-sections may be allowed for in design by the use of effective widths of elements.

Mid-line dimensions may only be used for calculating section properties of cold formed members and sheeting. For other sections, the overall dimensions should be used. EN 1993-1-3 and EN 1993-1-5 permit mid-line dimensions to be used in calculating resistances. EN 1993-1-1 also allows the use of mid-line dimensions in calculating resistances.

2.2.27 Maximum width-to-thickness ratios

Table 2.8 gives maximum width-to-thickness ratios for stainless steel elements.

2.2.28 Classification of cross-sections

In principle, stainless steel cross-sections may be classified in the same way as those of carbon steel. Four classes of cross-section are defined as follows:

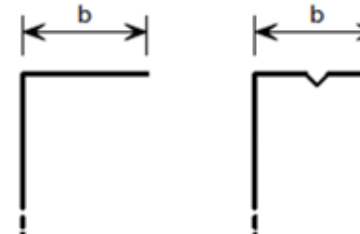
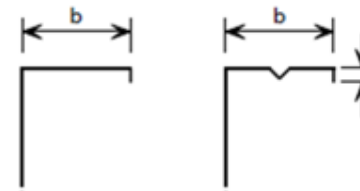
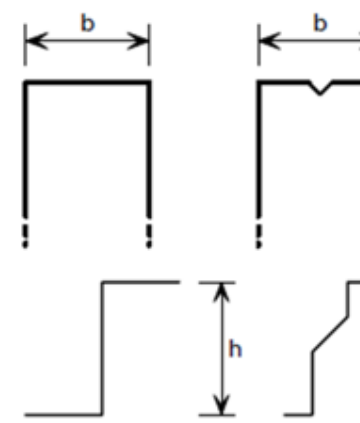
- | | |
|---------|--|
| Class 1 | cross-sections are those which can form a plastic hinge with the rotation capacity required from plastic analysis. |
| Class 2 | cross-sections are those which can develop their plastic moment |

resistance, but have limited rotation capacity.

Class 3 cross-sections are those in which the calculated stress in the extreme compression fibre of the steel member can reach its yield strength, but local buckling is liable to prevent development of the plastic moment resistance.

Class 4 cross-sections are those in which local buckling will occur before the attainment of yield stress in one or more parts of the cross-section.

Table 2.8 : Maximum width-to-thickness ratios [1]

a) Flat element or intermediately stiffened element connected to a web along one edge with the other edge unsupported:	$b/t \leq 50$ 
b) Flat element or intermediately stiffened element connected to a web along one edge and provided with a small simple lip along the other edge	$b/t \leq 60$ $c/t \leq 50$ 
c) Flat element or intermediately stiffened element connected along both edges to webs or flanges:	$b/t \leq 400$ $h/t \leq 400$ 
Note: Flat elements supported as in a) above with b/t ratios greater than approximately 30 and flat elements supported otherwise with b/t ratios greater than approximately 75 are likely to develop visual distortion at serviceability design loads.	

The classification of a cross-section depends on the highest (least favourable) class of its constituent parts that are partially or wholly in compression. It should be noted that the cross-section classification can vary according to the proportion of moment or axial load present and thus can vary along the length of a member.

2.2.29 Classification limits for parts of cross-sections

Sections are classified as Class 1, 2, or 3 depending on the limits set out in Table 2.9. Those sections, which do not meet the criteria for Class 3 sections, are classified as Class 4.

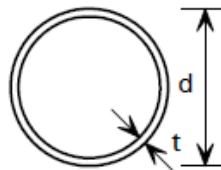
Table 2.9 : Maximum width-to-thickness ratios for compression parts [1]

Internal compression parts			
Class	Part subject to bending	Part subject to compression	Part subject to bending and compression
1	$c/t \leq 56,0\epsilon$	$c/t \leq 25,7\epsilon$	when $\alpha > 0,5$: $c/t \leq 308\epsilon/(13\alpha - 1)$ when $\alpha \leq 0,5$: $c/t \leq 28\epsilon/\alpha$
2	$c/t \leq 58,2\epsilon$	$c/t \leq 26,7\epsilon$	when $\alpha > 0,5$: $c/t \leq 320\epsilon/(13\alpha - 1)$ when $\alpha \leq 0,5$: $c/t \leq 29,1\epsilon/\alpha$
3	$c/t \leq 74,8\epsilon$	$c/t \leq 30,7\epsilon$	$c/t \leq 15,3\epsilon\sqrt{k_\sigma}$ For k_σ see 4.4.1
$\epsilon = \left[\frac{235}{f_y} \frac{E}{210\,000} \right]^{0,5}$	Grade	1.4301	1.4401 1.4462
	f_y (N/mm ²)	210	220 460
	ϵ	1,03	1,01 0,698
Notes: For hollow sections, c may conservatively be taken as $(h-2t)$ or $(b-2t)$ $E = 200\,000$ N/mm ² $\alpha = \frac{1}{2} \left(1 + \frac{N_{Ed}}{f_y c \sum t_w} \right)$ for sections which are symmetrical about the major axis			

Table 2.10 : Maximum width-to-thickness ratios for compression parts [1]

<u>Outstand flanges</u>				
Class	Type of section	Part subject to compression	Part subject to bending and compression	
			Tip in compression	Tip in tension
1	Cold formed	$c/t \leq 10,0s$	$c/t \leq \frac{10s}{\alpha}$	$c/t \leq \frac{10s}{\alpha\sqrt{\alpha}}$
	Welded	$c/t \leq 9,0s$	$c/t \leq \frac{9s}{\alpha}$	$c/t \leq \frac{9s}{\alpha\sqrt{\alpha}}$
2	Cold formed	$c/t \leq 10,4s$	$c/t \leq \frac{10,4s}{\alpha}$	$c/t \leq \frac{10,4s}{\alpha\sqrt{\alpha}}$
	Welded	$c/t \leq 9,4s$	$c/t \leq \frac{9,4s}{\alpha}$	$c/t \leq \frac{9,4s}{\alpha\sqrt{\alpha}}$
3	Cold formed	$c/t \leq 11,9s$	$c/t \leq 18,1s\sqrt{k_\sigma}$	For k_σ see 4.4.1.
	Welded	$c/t \leq 11,0s$	$c/t \leq 16,7s\sqrt{k_\sigma}$	For k_σ see 4.4.1.
<u>Angles</u> Refer also to Outstand flanges Does not apply to angles in continuous contact with other components				
Class	Section in compression			
3	$h/t \leq 11,9s$; $(b+h)/2t \leq 9,1s$			
$\alpha = \left[\frac{235}{f_y} \frac{E}{210\,000} \right]^{0,5}$	Grade	1.4301	1.4401	1.4462
	f_y (N/mm ²)	210	220	460
	s	1,03	1,01	0,698
Note: $E = 200\,000$ N/mm² $\alpha = \frac{1}{2} \left(1 + \frac{N_{Ed}}{f_y c \sum t_w} \right)$ for sections which are symmetrical about the major axis				

Table 2.11 : Maximum width-to-thickness ratios for compression parts[1]

<u>Tubular sections</u> 				
Class	Section in bending		Section in compression	
1	$d/t \leq 50\varepsilon^2$		$d/t \leq 50\varepsilon^2$	
2	$d/t \leq 70\varepsilon^2$		$d/t \leq 70\varepsilon^2$	
3	$d/t \leq 280\varepsilon^2$ Note for $d/t > 280\varepsilon^2$ see EN 1993-1-6		$d/t \leq 90\varepsilon^2$ Note for $d/t > 90\varepsilon^2$ see EN1993-1-6	
$\varepsilon = \left[\frac{235}{f_y} \frac{E}{210\,000} \right]^{0.5}$	Grade	1.4301	1.4401	1.4462
	f_y (N/mm ²)	210	220	460
	ε	1,03	1,01	0,698
Note: $E = 200\,000$ N/mm ²				

2.2.30 Effective widths

The properties of Class 4 cross-sections may be established by calculation using the effective widths of the component parts in full or partial compression.

The effective area of a Class 4 cross-section in full or partial compression, A_{eff} , is the gross area of the cross-section minus the sum of the ineffective areas of each slender element making up the cross-section. The effective area of each Class 4 element is the effective breadth b_{eff} calculated below multiplied by the element thickness. When the cross-section is subject to bending, an effective moment of inertia I_{eff} and effective section modulus W_{eff} also need to be calculated.

The effective widths of elements in full or partial compression may be obtained from Table 2.12 for internal elements, and from Table 2.13 for outstand elements.

The effective widths of flange elements in compression may be based on the stress ratio ψ determined for the gross cross-section. The effective width of a web element

should be based on the stress ratio ψ determined for a cross-section comprising the effective area of the compression flange but the gross area of the web and tension flange.

The reduction factor ρ may be calculated as follows:

Welded internal elements:

$$\rho = \frac{0,772}{\bar{\lambda}_p} - \frac{0,125}{\bar{\lambda}_p^2} \quad \text{but} \leq 1 \quad (2.1)$$

Welded outstand elements

$$\rho = \frac{1}{\bar{\lambda}_p} - \frac{0,242}{\bar{\lambda}_p^2} \quad \text{but} \leq 1 \quad (2.2)$$

Where $\bar{\lambda}_p$ is the element slenderness defined as:

$$\bar{\lambda}_p = \frac{\bar{b}/t}{28,4\epsilon\sqrt{k_\sigma}} \quad (2.3)$$

in which:

t is the relevant thickness

k_σ is the buckling factor corresponding to the stress ratio

$\bar{b}$ is the relevant width as follows:

$\bar{b} = d$ for webs (except RHS)

$\bar{b}$ = flat element width for webs of RHS, which can conservatively be taken as $h-2t$

$\bar{b} = b$ for internal flange elements (except RHS)

$\bar{b}$ = flat element width for flanges of RHS, which can conservatively be taken as $b-2t$

$\bar{b} = c$ for outstand flanges

$\bar{b} = h$ for equal leg angles and unequal leg angles

ϵ is the material factor

Generally, the neutral axis of the effective section will shift by a dimension e compared to the neutral axis of the gross section, see Figures 2.4 and 2.5. This should be taken into account when calculating the properties of the effective cross-section.

When the cross-section is subject to axial compression, the recommendations take account of the additional moment ΔM_{Ed} given by:

$$\Delta M_{Ed} = N_{Ed} e_N$$

where e_N is the shift of the neutral axis when the cross-section is subject to uniform compression.

Table 2.12 : Internal compression elements [1]

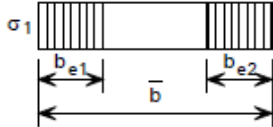
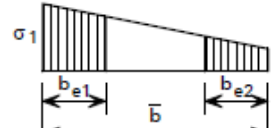
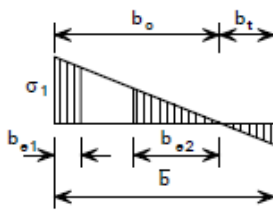
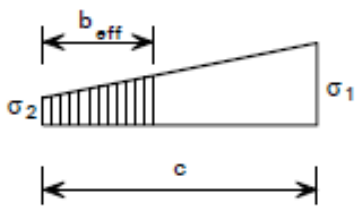
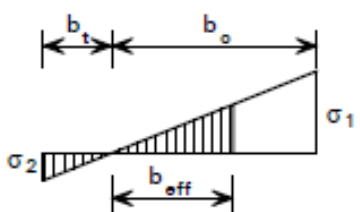
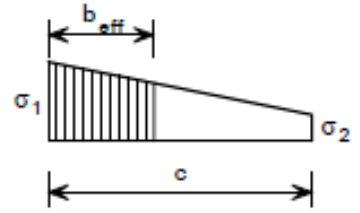
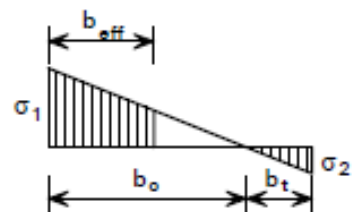
Stress distribution (compression positive)				Effective width b_{eff}		
				$\psi = -1$: $b_{eff} = \rho \bar{b}$ $b_{e1} = 0,5 b_{eff}$ $b_{e2} = 0,5 b_{eff}$		
				$1 > \psi \geq 0$: $b_{eff} = \rho \bar{b}$ $b_{e1} = \frac{2 b_{eff}}{5 - \psi}$ $b_{e2} = b_{eff} - b_{e1}$		
				$\psi < 0$: $b_{eff} = \rho b_o = \rho \bar{b} / (1 - \psi)$ $b_{e1} = 0,4 b_{eff}$ $b_{e2} = 0,6 b_{eff}$		
$\psi = \sigma_2 / \sigma_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1	$-1 > \psi > -3$
Buckling factor k_σ	4,0	$8,2 / (1,05 + \psi)$	7,81	$7,81 - 6,29\psi + 9,78\psi^2$	23,9	$5,98 (1 - \psi)^2$

Table 2.13 : Outstand compression elements [1]

Stress distribution (compression positive)			Effective width b_{eff}		
			$1 > \psi \geq 0:$ $b_{eff} = \rho c$		
			$\psi < 0:$ $b_{eff} = \rho b_c = \rho c / (1 - \psi)$		
$\psi = \sigma_2 / \sigma_1$	1	0	-1	$+1 \geq \psi \geq -3$	
Buckling factor k_σ	0,43	0,57	0,85	$0,57 - 0,21 \psi + 0,07 \psi^2$	
			$1 > \psi \geq 0:$ $b_{eff} = \rho c$		
			$\psi < 0:$ $b_{eff} = \rho b_c = \rho c / (1 - \psi)$		
$\psi = \sigma_2 / \sigma_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1
Buckling factor k_σ	0,43	$0,578 / (\psi + 0,34)$	1,70	$1,7 - 5 \psi + 17,1 \psi^2$	23,8

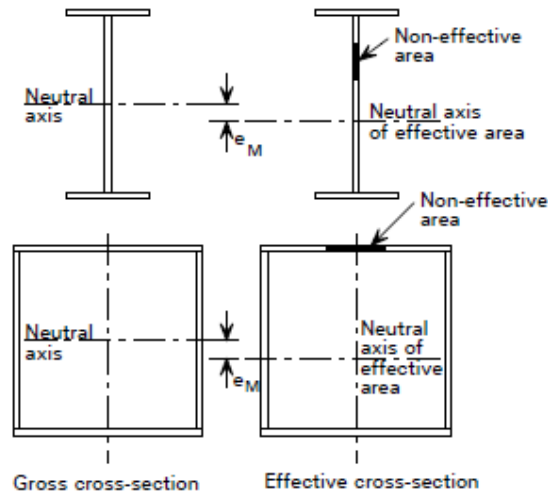


Figure 2.4 : Class 4 cross-section subject to bending moment [1]

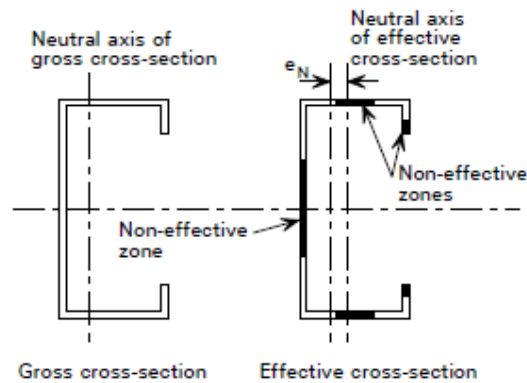


Figure 2.5 : Class 4 cross-section subject to compression [1]

2.2.31 Effects of shear lag

Shear lag in flanges may be neglected if $b_0 < L_e / 50$, where b_0 is taken as the flange outstand or half the width of an internal element and L_e is the length between points of zero bending moment. Where this limit for b_0 is exceeded, the effects of shear lag in flanges should be considered; the guidance for carbon steel in EN 1993-1-5 is applicable. Note that EN 1993-1-5 requires that shear lag to be taken into account at both the ultimate and the serviceability limit states.

2.2.32 Resistances of cross-sections

This Section relates to the resistance of cross-sections only; a check on possible buckling modes is also required to establish member resistance.

2.2.32.1 Cross-sections subject to tension

The resistance of cross-sections subject to uniform tensile stresses only, $N_{t,Rd}$, may be taken as the smaller of:

a) the design plastic resistance of the gross cross-section.

$$N_{pl,Rd} = \frac{A_g f_y}{\gamma_{M0}} \quad (2.4)$$

where:

A_g is the gross area

f_y is the characteristic yield strength (generally taken as the minimum specified 0,2% proof strength)

2.2.32.2 Cross-sections subject to compression

The resistance of a cross-section subject to compression, $N_{c,Rd}$, with a resultant acting through the centroid of the gross section (for Class 1, 2 and 3 cross-sections) or the effective section (Class 4 cross-sections) may be taken as:

for Class 1, 2 or 3 cross-sections

$$N_{c,Rd} = A_g f_y / \gamma_{M0} \quad (2.5)$$

for Class 4 cross-sections

$$N_{c,Rd} = A_{eff} f_y / \gamma_{M0} \quad (2.6)$$

2.2.32.3 Cross-sections subject to bending moment

In the absence of shear and axial forces, the design moment resistance of a cross-section subject to a uniaxial moment, $M_{c,Rd}$, should be taken as:

for Class 1 or 2 cross-sections

$$M_{c,Rd} = W_{pl} f_y / \gamma_{M0} \quad (2.7)$$

for Class 3 cross-sections

$$M_{c,Rd} = W_{el,min} f_y / \gamma_{M0} \quad (2.8)$$

for Class 4 cross-sections

$$M_{c,Rd} = W_{eff,min} f_y / \gamma_{M0} \quad (2.9)$$

where:

W_{pl} is the plastic section modulus
 $W_{el,min}$ is the elastic section modulus corresponding to the fibre with the maximum elastic stress
 $W_{eff,min}$ is the elastic modulus of effective section corresponding to the fibre with the maximum elastic stress.

2.2.32.4 Cross-sections subject to shear

The plastic shear resistance of a cross-section, $V_{pl,Rd}$ may generally be taken as:

$$V_{pl,Rd} = A_v (f_y / \sqrt{3}) / \gamma_{M0} \quad (2.10)$$

where:

A_v is the shear area, which may be taken as follows:

- welded I, H and box sections, load parallel to web $\eta \Sigma(h_w t_w)$
- welded I, H, channel and box section, load parallel to flanges $A - \Sigma(h_w t_w)$

h is the overall depth

h_w is the depth of the web

η see EN 1993-1-5. (EN 1993-1-4 recommends $\eta = 1,20$.)

2.2.32.5 Cross-sections subject to combination of loads

When an axial force is present, allowance should be made for its effect on the plastic moment resistance. For Class 1 and 2 cross sections, the following criterion should be satisfied:

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

where:

$M_{N,Rd}$ is the design plastic moment resistance reduced due to the axial force N_{Ed} .

For doubly symmetrical I- and H-sections or other flange sections, allowance need not be made for the effect of the axial force on the plastic resistance moment about the y-y axis when both the following criteria are satisfied:

$$N_{Ed} \leq 0,25 N_{pl,Rd} \text{ and}$$

$$N_{Ed} \leq 0,5 h_w t_w f_y / \gamma_{M0}$$

In the absence of shear force, for Class 3 and Class 4 cross-sections the maximum longitudinal stress should satisfy the criterion:

$$\sigma_{x,Ed} \leq f_y / \gamma_{M0}$$

where:

$\sigma_{x,Ed}$ is the design value of the local longitudinal stress due to moment and axial force, taking account of fastener holes where relevant.

For Class 4 cross-sections, the following criterion should also be met:

$$\frac{N_{Ed}}{A_{eff} f_y / \gamma_{M0}} + \frac{M_{y,Ed} + N_{Ed} e_{Ny}}{W_{eff,y,min} f_y / \gamma_{M0}} + \frac{M_{z,Ed} + N_{Ed} e_{Nz}}{W_{eff,z,min} f_y / \gamma_{M0}} \leq 1 \quad (2.11)$$

where:

A_{eff} is the effective area of the cross-section when subjected to uniform compression

$W_{eff,min}$ is the effective section modulus of the cross-section when subjected only to moment about the relevant axis

e_N is the shift of the relevant centroidal axis when the cross-section is subjected to compression only.

Note that for angles, the y and z axes in the above should be taken as the u and v axes respectively.

When V_{Ed} exceeds 50% of $V_{pl,Rd}$, the design resistance of the cross-section to combinations of moment and axial force should be calculated using a reduced yield strength $(1 - \rho) f_y$ for the shear area, where

$$\rho = (2V_{Ed} / V_{pl,Rd} - 1)^2. \quad (2.12)$$

2.2.32.6 Utilisation of strain hardening behaviour

In certain circumstances, such as beams under short term accidental loads, it is permissible to recognise the benefits of the strain hardening properties of stainless steel more fully in design.

This may be achieved by utilising an enhanced proof stress, σ_0 , in place of the 0.2% proof stress f_y in all calculations. It is recommended that in the absence of a more detailed appraisal, perhaps involving non-linear finite element analysis, the following restrictions should be observed:

- The cross-section should be of Class 1 or Class 2 as calculated by using σ_0 in place of f_y .
- The cross-section is subject to major axis bending only.

- The member concerned is not subject to instability caused by any form of buckling (flexural, torsional, lateral-torsional or distortional – see Section 5), again using σ_0 in all calculations.
 - Connections to adjacent members and the members themselves are sufficiently strong to realise the enhanced resistance of the member.
- Particular attention should be given to assessing connection strength requirements.

2.3 Member Design

The design checks required for stainless steel members are similar to those required for carbon steel members. It is recommended that the forces and moments in the members are derived from an elastic global analysis.

A possible design approach for checking against buckling in stainless steel members is to use the tangent modulus corresponding to the buckling stress instead of the initial modulus as used in carbon steel rules. Assuming similar levels of geometric and residual stress imperfections in carbon steel and stainless steel members, this generally leads to satisfactory results when it is based on validated carbon steel rules. This approach is therefore available to the designer. However, it requires iterative solution techniques and therefore has been avoided in this study except in some cases when it has been used to derive effective design curves for use with the initial modulus. Instead, emphasis has been given to calibrating against available experimental data.

The following subsections are intended for use with singly, doubly or point-symmetric uniform sections. The resistance of members not possessing any axis of symmetry should be verified by appropriate tests.

2.3.1 Tension members

Members subject to tension only do not suffer any instability due to buckling.

2.3.2 Compression members

Members in compression are susceptible to a number of possible buckling modes including:

- Plate buckling
- Flexural buckling
- Torsional buckling

- Torsional-flexural buckling.

2.3.3 Flexural buckling

The resistance to flexural buckling should be determined from:

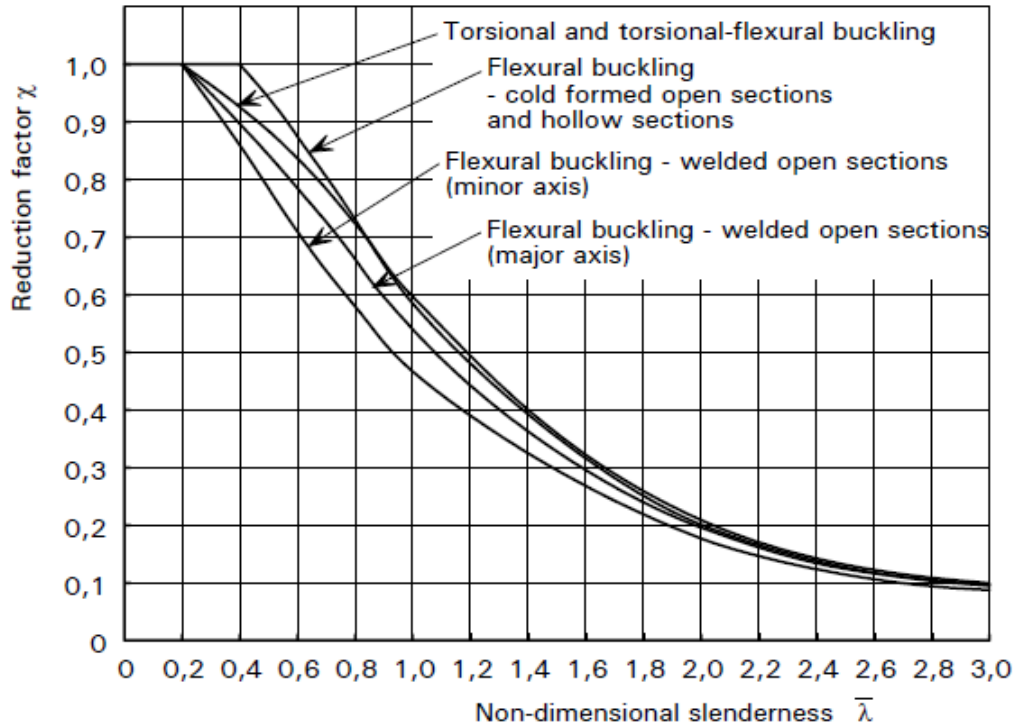


Figure 2.6 : Buckling curves for flexural, torsional and torsional-flexural buckling

The determination of the buckling length should be based upon structural mechanics principles, considering boundary conditions.

Table 2.14 : Values of α for buckling [1]

Buckling mode	Type of member	α	$\bar{\lambda}_0$
Flexural	Cold formed open sections	0,49	0,40
	Hollow sections (welded and seamless)	0,49	0,40
	Welded open sections (major axis)	0,49	0,20
	Welded open sections (minor axis)	0,76	0,20
Torsional and torsional-flexural	All members	0,34	0,20

2.3.4 Torsional and torsional-flexural buckling

The resistance to these buckling modes should be determined as:

(Taking $\alpha = 0,34$ and $\bar{\lambda}_0 = 0,2$)

for Class 1, 2 and 3 cross-sections;

$$\bar{\lambda}_T = \sqrt{\frac{Af_y}{N_{cr}}} \quad (2.13)$$

for Class 4 cross-sections.

$$\bar{\lambda}_T = \sqrt{\frac{A_{eff} f_y}{N_{cr}}} \quad (2.14)$$

in which:

$$N_{cr} = N_{cr,TF} \text{ and } N_{cr} < N_{cr,T} \quad (2.15)$$

Where:

$N_{cr,T}$ is the elastic critical torsional buckling force

$$N_{cr,T} = \frac{1}{i_o^2} \left(GI_t + \frac{\pi^2 EI_w}{l_T^2} \right) \quad (2.16)$$

$N_{cr,T}$ is the elastic critical torsional-flexural buckling force.

For cross-sections that are symmetrical about the y-y axis (e.g. $z_o = 0$)

$$N_{cr,TF} = \frac{N_{cr,y}}{2\beta} \left[1 + \frac{N_{cr,T}}{N_{cr,y}} - \sqrt{\left(1 - \frac{N_{cr,T}}{N_{cr,y}} \right)^2 + 4 \left(\frac{y_o}{i_o} \right)^2 \frac{N_{cr,T}}{N_{cr,y}}} \right] \quad (2.17)$$

in which:

$$i_o^2 = i_y^2 + i_z^2 + y_o^2 + z_o^2 \quad (2.18)$$

i_y and i_z are the radii of gyration of the gross cross-section about the y and z axes respectively

y_o and z_o are the shear centre co-ordinates with respect to the centroid of the gross cross-section

G is the shear modulus

l_T is the buckling length of the member for torsional buckling(see EN 1993-1-3)

I_T is the torsional constant of the gross cross-section

I_w is the warping constant of the gross cross-section

$$\beta = 1 - \left(\frac{y_o}{i_o} \right)^2 \quad (2.19)$$

$N_{cr,y}$ and $N_{cr,z}$ are the elastic critical axial force for flexural buckling about the y-y and z-z axes respectively.

For a doubly symmetric cross-section, the shear centre coincides with the centroid, therefore $y_o = 0$ and $z_o = 0$ and

$$N_{cr,TF} = N_{cr,T} \text{ provided } N_{cr,T} < N_{cr,y} \text{ and } N_{cr,T} < N_{cr,z} . \quad (2.20)$$

Note that for angles, the y and z axes in the above should be taken as the u and v axes respectively.

2.3.5 Flexural members

A member is in simple bending under loads acting normal to the longitudinal axis if it is connected in such a way as to eliminate twisting or tensile or compressive end loading.

The following criteria should be considered for establishing the moment resistance of a beam:

- Yielding of the cross-section
- Plate buckling
- Lateral-torsional buckling
- Shear buckling
- Local strength at points of loading or reaction

The effects of shear lag and flange curling may have to be accounted for flexural members in design.

Biaxial bending should be treated.

2.3.6 Lateral-torsional buckling

The possibility of lateral-torsional buckling may be discounted for the following classes of member:

- beams subject to bending only about the minor axis
- beams laterally restrained throughout their length by adequate bracing

For all other classes of member, the resistance to lateral torsional buckling should be determined from:

$$M_{b,Rd} = \chi_{LT} W_y f_y / \gamma_{M1} \quad (2.21)$$

The buckling curves for lateral torsional buckling are shown in Figure 2.8.

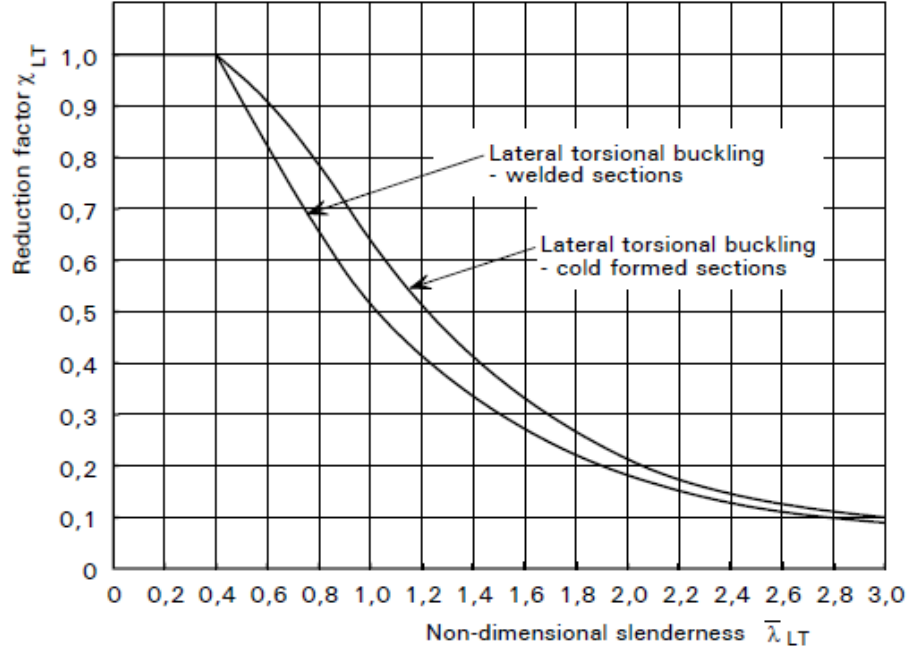


Figure 2.7 : Buckling curves for lateral-torsional buckling [1]

2.3.7 Shear resistance

The shear resistance is limited by either the plastic shear resistance or the shear buckling resistance.

The shear buckling resistance only requires checking when

$$h_w / t \geq 52 \epsilon \eta \text{ for an unstiffened web or } h_w / t \geq 23 \epsilon \sqrt{k_\tau} / \eta \text{ for a stiffened web.}$$

The shear buckling resistance for a beam should be obtained from:

$$V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \leq \frac{\eta f_{yw} h_w t}{\sqrt{3} \gamma_{M1}} \quad (2.22)$$

in which the contribution from the web is given by:

$$V_{bw,Rd} = \frac{\chi_w f_{yw} h_w t}{\sqrt{3} \gamma_{M1}} \quad (2.23)$$

where:

h_w is the clear web depth between flanges

k_τ is the shear buckling coefficient

$V_{bw,Rd}$ is the contribution to the shear buckling resistance from the web

$V_{bf,Rd}$ is the contribution to the shear buckling resistance from the flanges

f_{yw} is the characteristic yield strength of web

η see EN 1993-1-5 (EN 1993-1-4 recommends $\eta = 1,20$.)

Note: The same value of η should be used for calculating the plastic shear resistance as is used for calculating the shear buckling resistance.

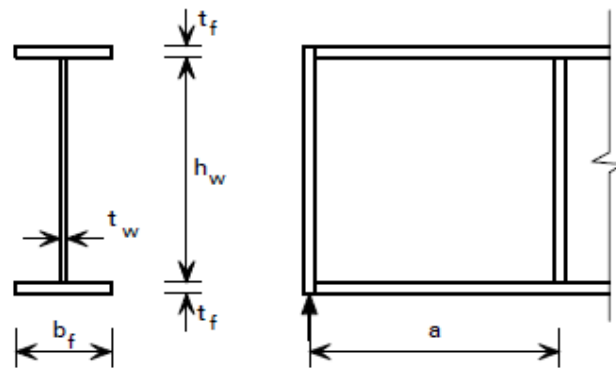


Figure 2.8 : Notation for geometrical dimensions [1]

For plates with one or two longitudinal stiffeners, if the aspect ratio $a/h_w \geq 3$. For plates with one or two longitudinal stiffeners and an aspect ratio $a/h_w < 3$, reference should be made to EN 1993-1-5 Annex A3.

For simplicity, the contribution from the flanges χ_f may be neglected. However, if the flange resistance is not completely utilized in withstanding the bending moment ($M_{Ed} < M_{f,Rd}$) then the contribution from the flanges may be obtained as follows:

$$V_{bf,Rd} = \frac{b_f t_f^2 f_{yf}}{c \gamma_{M1}} \left[1 - \left[\frac{M_{Ed}}{M_{f,Rd}} \right]^2 \right] \quad (2.24)$$

where:

b_f and t_f are taken for the flange which provides the least axial resistance, b_f being taken as not larger than $15\epsilon t_f$ on each side of the web

$$\eta_1 = \frac{N_{Ed}}{f_y A_{eff} / \gamma_{M0}} + \frac{M_{y,Ed} + N_{Ed} e_{y,N}}{f_y W_{y,eff} / \gamma_{M0}} + \frac{M_{z,Ed} + N_{Ed} e_{z,N}}{f_y W_{z,eff} / \gamma_{M0}} \leq 1,0 \quad (2.25)$$

where:

A_{eff} is the effective cross-section area

$e_{y,N}$ is the shift in the position of the neutral axis with respect to the y axis

$e_{z,N}$ is the shift in the position of the neutral axis with respect to the z axis

$M_{y,Ed}$ is the design bending moment with respect to the y axis

$M_{z,Ed}$ is the design bending moment with respect to the z axis

N_{Ed} is the design axial force

$W_{y,eff}$ is the effective section modulus with respect to the y axis

$W_{z,eff}$ is the effective section modulus with respect to the z axis

Action effects M_{Ed} and N_{Ed} should include global second order effects where relevant.

The plate buckling verification of the panel should be carried out for the stress resultants at a distance 0,4a or 0,5b, whichever is the smallest, from the panel end where the stresses are the greater.

Provided that $\bar{\eta}_3$ does not exceed 0,5, the design resistance to bending moment and axial force need not be reduced to allow for the shear force. If $\bar{\eta}_3$ is more than 0,5 the combined effects of bending and shear in the web of an I or box girder should satisfy:

$$\bar{\eta}_1 + \left(1 - \frac{M_{f,Rd}}{M_{pl,Rd}} \right) (2\bar{\eta}_3 - 1)^2 \leq 1,0 \quad \text{for} \quad \bar{\eta}_1 \geq \frac{M_{f,Rd}}{M_{pl,Rd}} \quad (2.26)$$

where:

$M_{f,Rd}$ is the design plastic moment of resistance of the section consisting of the effective area of the flanges

$M_{pl,Rd}$ is the design plastic moment of resistance of the cross section

Stresses are taken as positive. M_{Ed} and V_{Ed} should include second order effects where relevant.

If an axial force N_{Ed} is present, then $M_{pl,Rd}$ should be replaced by the reduced plastic resistance moment $M_{N,Rd}$ according to 6.2.9 of EN 1993-1-1 and $M_{f,Rd}$ should be reduced according to equation (1.27).

$$\left[1 - \frac{N_{Ed}}{\frac{(A_{f1} + A_{f2})f_{yf}}{\gamma_{M0}}} \right] \quad (2.27)$$

Reference should be made to EN 1993-1-5 if the axial force is so large that the whole web is in compression.

2.3.8 Web crushing, crippling and buckling

If the flanges are laterally restrained, one of three possible failure modes will govern the resistance of an unstiffened web to forces from concentrated loads or support reactions:

- crushing of the web close to the flange, accompanied by plastic deformation of the flange
- crippling of the web in the form of localised buckling and crushing of the web close to the flange, accompanied by plastic deformation of the flange
- buckling of the web over most of the depth of the member.

For unstiffened or stiffened webs the design resistance to local buckling under transverse forces should be taken as:

$$F_{Rd} = f_{yw} L_{eff} t_w / \gamma_{M1} \quad (2.28)$$

where:

t_w is the thickness of the web

f_{yw} is the yield strength of the web

L_{eff} is the effective length for resistance to transverse forces, which should be determined from

$$L_{eff} = \chi_F l_y \quad (2.29)$$

where:

l_y is the effective loaded length appropriate to the length of stiff bearing S_s

χ_F is the reduction factor due to local buckling

In addition the effect of the transverse force on the moment resistance of the member should be considered.

To determine L_{eff} , a distinction should be made between three types of force application, as follows:

- Forces applied through one flange and resisted by shear forces in the web
- Forces applied to one flange and transferred through the web directly to the other flange
- Forces applied through one flange close to an unstiffened end.

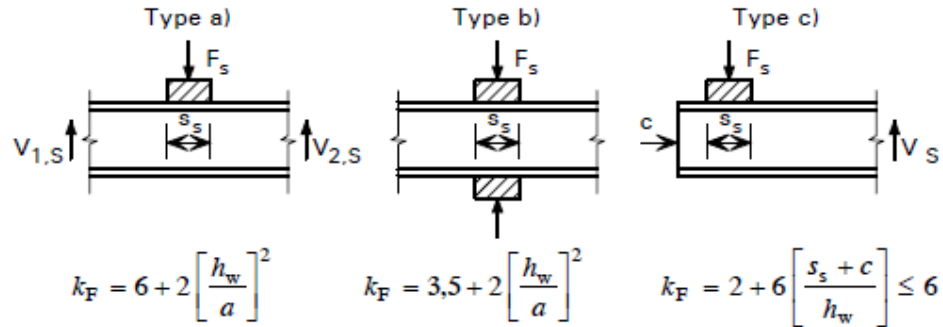


Figure 2.9 : Buckling coefficients for different types of load application [1]

2.3.9 Length of stiff bearing

The length of stiff bearing, s_s , on the flange is the distance over which the applied force is effectively distributed and it may be determined by dispersion of load through solid steel material at a slope of 1:1, see Figure 2.11. However, s_s should not be taken as larger than the depth of the web, h_w .

If several concentrated loads are closely spaced, the resistance should be checked for each individual load as well as for the total load, with s_s as the centre-to-centre distance between outer loads.

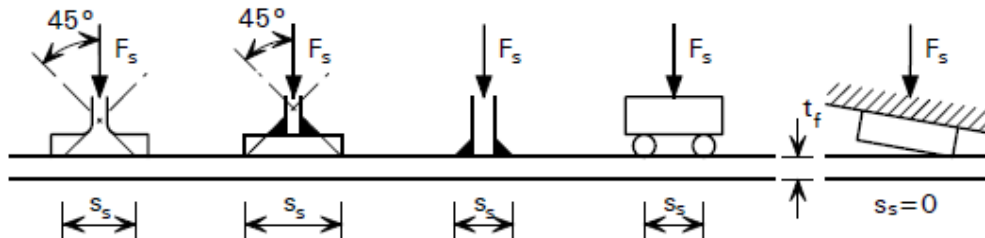


Figure 2.10 : Length of stiff bearing [1]

Transverse stiffeners

Transverse stiffeners at supports and at other positions where significant external forces are applied should preferably be double-sided and symmetric about the

centreline of the web. These stiffeners should be checked for cross-section crushing and buckling. Intermediate stiffeners not subject to external forces need only be checked for buckling.

The effective cross-section to use in the buckling check should include a width of web plate as shown in Figure 2.12. At the end of a member (or at the openings in the web) the width of web included in the cross-section should not exceed that available.

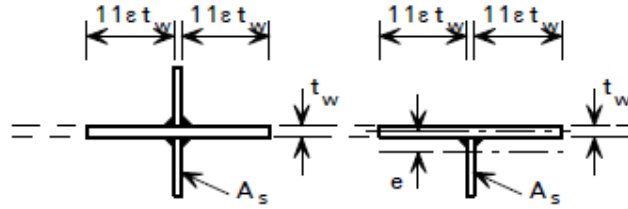


Figure 2.11 : Effective cross-section of stiffeners for buckling [1]

The out-of-plane buckling resistance $N_{b,Rd}$ of the stiffener should be determined as $\alpha = 0,49$ and $\lambda = 0,2$. The buckling length l of the stiffener should be appropriate for the conditions of restraint, but not less than $0,75 h_w$, where both ends are fixed laterally. A larger value of l should be used for conditions that provide less end restraint. The torsional buckling resistance of the cruciform section should also be checked.

At supports or at intermediate positions where significant loads are applied, the buckling resistance should exceed the reaction or load. At other intermediate positions, the compression force N_{Ed} in the stiffener may be obtained from:

$$N_{Ed} = V_{Ed} - \frac{1}{\bar{\lambda}_w^2} \frac{f_{yw} h_w t_w}{\sqrt{3} \gamma_{M1}} \quad (2.30)$$

where:

V_{Ed} is the design shear force in the member.

The above expression should be calculated assuming the stiffener under consideration is removed.

2.3.10 Determination of deflections

Deflections should be determined for the load combination at the relevant Serviceability Limit State.

The deflection of elastic beams (i.e. those not containing a plastic hinge) may be estimated by standard structural theory, except that the secant modulus of elasticity should be used instead of Young's modulus. The secant modulus varies with the stress level in the beam and values which are accurate for predicting deflections when the secant modulus is based on the maximum stress in the member and this maximum stress does not exceed 65% of the 0,2% proof strength. At higher levels of stress, the method becomes very conservative and a more accurate method (e.g. one that involves integrating along the length of the member) should be used.

In the case of Class 4 cross-sections and/or members subject to shear lag, an effective section should be used in the calculations. As a first estimate, it is appropriate to use the effective section based on the effective widths. As a refinement, it is possible to use an effective section based on the effective buckling widths determined for the actual stress in the elements by taking as:

$$\varepsilon = \left[\frac{235}{\sigma} \frac{E}{210\,000} \right]^{1/2} \quad (2.31)$$

where:

σ is the actual stress in the element in the associated effective cross-section

E is Young's modulus.

2.3.11 Members subject to combinations of axial load and bending moments

2.3.11.1 Axial tension and bending

Tension members with moments should be checked for resistance to lateral torsional buckling under the moment alone.

They should also be checked for capacity under the combined effects of axial load and moment at the points of maximum bending moment and axial load.

The following relationship should be satisfied:

$$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed}}{M_{y,Rd}} + \frac{M_{z,Ed}}{M_{z,Rd}} \leq 1 \quad (2.32)$$

where:

N_{Ed} is the design axial tensile load in the member at the critical location

N_{Rd} is the design resistance of the member in tension

$M_{y,Ed}$ is the design moment about the major axis at the critical section

$M_{z,Ed}$ is the design moment about the minor axis at the critical section

$M_{y,Rd}$ is the design moment resistance about the major axis in the absence of axial load and include any reduction that may be caused by shear effects

$M_{z,Rd}$ is the design moment about the minor axis in the absence of axial load and includes any reduction that may be caused by shear effects

2.4 Composite Bridge Design

2.4.1 Global analysis of composite bridges

The global analysis [2] is the calculation of the whole bridge for determining the internal forces and moments and the corresponding stresses in all its cross-sections. This is calculated by respecting the construction phases and by considering two peculiar dates in the bridge life – at traffic opening (short term situation) and at infinite time (long term situation or 100 years old).

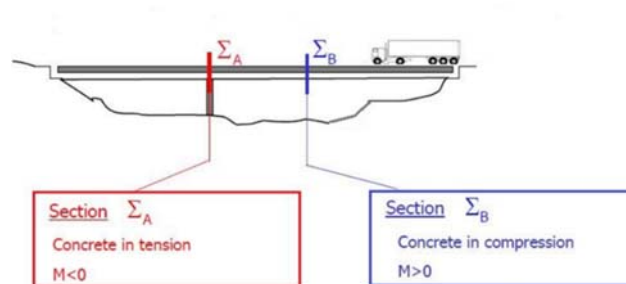


Figure 2.12 : Analysis of two different cross-sections

The deformed geometry has no influence on the internal forces and moments in case of a girder bridge. The analysis is therefore a **first order analysis**. EN1994-2, 5.2.1

The lateral torsional buckling is justified by using specific verification formulae. It could also be justified by defining a deformed initial geometry of the structure, followed by a second order analysis. EN1994-2, 5.3.2(1)

2.4.2 Influence of the material non-linearities

The composite cross-section resistance in sagging bending moment region is generally a plastic resistance calculation which takes into account the material non-linearities.

The internal forces and moments (and then the stress distribution) are nevertheless calculated with a **linear elastic analysis**. EN1994-2, 5.4.1.1(1)

This analysis should take the cracking of concrete, its shrinkage and its creep into account as well as the construction phases. EN1994-2, 5.4.2.1(1)

2.4.3 Taking the concrete cracking into account

This is normally achieved by two successive global analysis :

- In a first global analysis - called « **uncracked analysis** » - the concrete strength is considered for calculating the mechanical properties of all the crosssections in the modeled main girder;
- In a given cross-section if the longitudinal upper fibre tensile stress σ_c in the concrete slab is lower than $-2.f_{ctm}$ for characteristic SLS combination of actions, then the concrete of this crosssection should be considered as cracked in the second global analysis. This criterion thus defines *cracked zones* on both sides of the intermediate supports;
- In a second global analysis - called « **cracked analysis** » - the concrete slab stiffness in the cracked zones is reduced to the stiffness of its reinforcing steel. The internal forces and moments - as well as the corresponding stress distributions - of this cracked analysis are used in the following chapters to justify all the transverse cross-sections of the deck.

On condition that (EN 1994-2, 5.4.2.3 (3)):

- the ratio between two adjacent span lengths is always higher than 0.6 and,
- no differences in level are used at internal supports,

the cracked analysis may be performed directly by using cracked zones which are defined by considering 15% of the span lengths on both sides at each internal support.

2.4.4 Taking shear lag into account in the concrete slab

The shear lag in the concrete slab is taken into account by reducing the actual slab width to an “effective” width. It thus influences the mechanical properties of the cross-sections which are used in the calculations of the global analysis.

2.4.5 Internal forces and moments – Stresses

To analyse the global longitudinal bending, the deck is modeled as a continuous line of bar elements which corresponds to the neutral fibre of the modeled main girder and which is simply supported at piles and abutments. With respect to a fixed reference this neutral fibre changes throughout the calculation according to the mechanical properties (areas and second moments of area) allocated to the bar elements in the model. This is due to the different modular ratios to be considered and to the fact that a given cross-section could be composite or not, with a cracked concrete or not, following the phases of the global analysis.

In addition to the cross-sections at internal and end supports and at mid-spans, some peculiar crosssections are worthy of being at the bar element ends:

- at the quarter and three-quarters of each span (to define the effective widths of the slab to calculate the stress distribution,
- at the ends of every slab concreting segment,
- at the thickness changes in the structural steel distribution.

2.4.6 Effective width of the concrete slab

In a given cross-section of one of the main girder, the effective width of the concrete slab is the sum of 3 terms in accordance with EN1994-2, 5.4.1.2 (5) (see Figure 2.14):

$$b_{eff} = b_0 + \beta_1 b_{e1} + \beta_2 b_{e2} \quad ; \quad (2.33)$$

- b_0 (= 200 mm), the centre-to-centre distance between the outside stud rows;

$$\bullet \quad b_{ei} = \min \{ L_e/8 ; b_i \} \quad (2.34)$$

where

L_e is the equivalent span length in the considered cross-section and where b_i is the actual geometric width of the slab associated to the main girder;

- $\beta_1 = \beta_2 = 1$ except for the cross-sections at end supports C0 and C3

Where

$$\beta_i = 0,55 + 0.025 \cdot L_e/b_{ei} < 1.0 \quad (2.35)$$

with b_{ei} taken as equal to the effective width at mid-end span in accordance with EN1994-2, 5.4.1.2 (6).

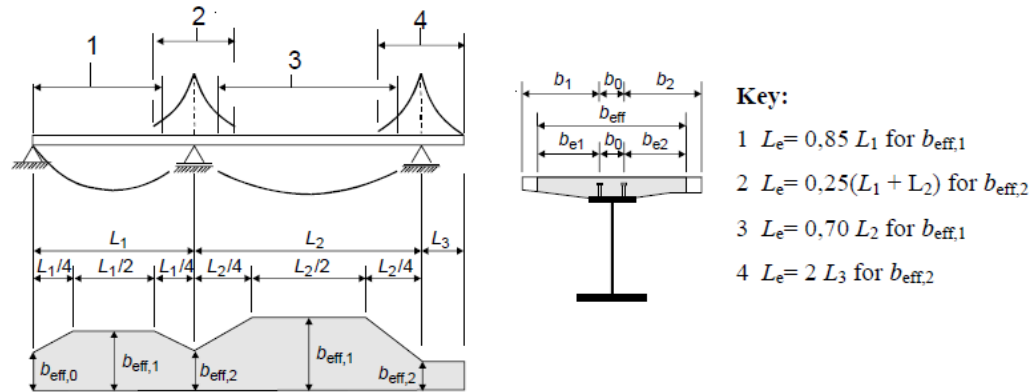


Figure 2.13 : Effectives width of concrete slab (ULS and SLS)

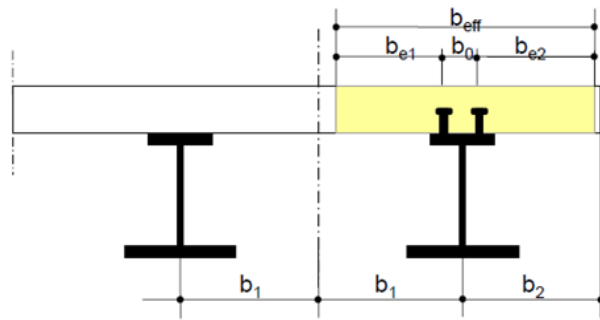


Figure 2.14 : Effective slab width for a main girder in a given cross-section

To calculate the internal forces and moments with a linear elastic global analysis, constant widths may be used for each span by considering the values at mid-span. This means that the calculation can be performed with the actual slab width over the entire bridge length, i.e. the shear lag in the concrete slab has no influence on the internal forces and moments. EN1994-2, 5.4.1.2(4).

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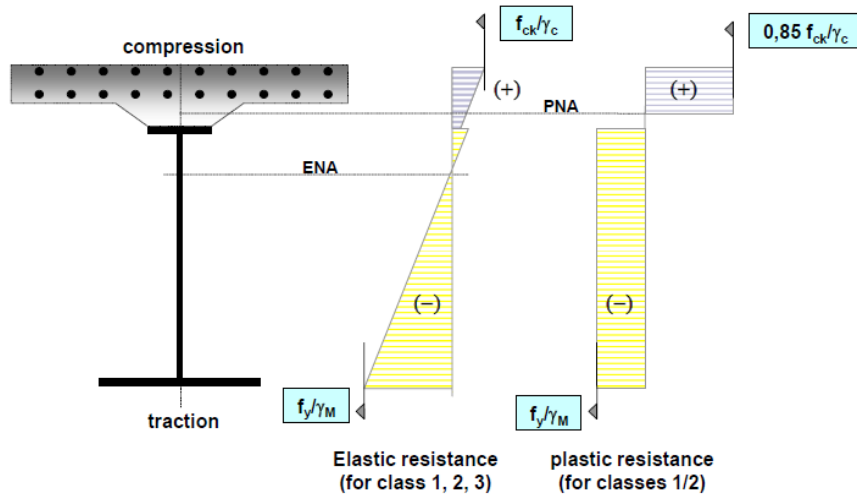


Figure 2.15 : Composite cross-section verification at ULS

To calculate the internal forces and moments with a linear elastic global analysis, constant widths may be used for each span by considering the values at mid-span. This means that the calculation can be performed with the actual slab width over the entire bridge length, i.e. the shear lag in the concrete slab has no influence on the internal forces and moments. EN1994-2, 5.4.1.2(4)

2.4.7 Justification of the composite cross-sections at ULS other than fatigue

According to EN1994-2, 6.1.1, a composite cross-section should be checked at ULS in terms of:

- resistance of cross-section: EN1994-2, 6.2.1 and 6.2.2 ;
- resistance to shear buckling: EN1994-2, 6.2.2 ;
- resistance to transverse load during launching: EN1994-2, 6.5
- resistance to slip between concrete and steel (connection): EN1994-2, 6.6
- fatigue resistance: EN1994-2, 6.8
- resistance to lateral torsional buckling (EN1994-2, 6.4)

2.4.8 Determining a composite cross-section Class in practice

The classification system established for steel beams also applies to composite beams. The Class of a composite cross-section is the highest Class of the compressed elements making it up. EN1994-2, 5.5.1(1) and EN1994-2, 5.5.1(2)

Three preliminary comments are possible:

- Local buckling can only be induced by compressive stresses. Any element subjected only to tensile stress must be classified in Class 1, irrespective of its slenderness;
- If an element is a Class n element under uniform compression then it always is in Class $m \leq n$ under any other stress distribution which can only reduce the compressive stresses;
- If the shear connectors fulfil the spacing required in EN 1994-2, 6.6.5.5, then a steel flange in compression connected to a concrete slab is in Class 1.

To classify an internal compression element (i.e. an element bordered to opposite edges by two other perpendicular elements) as an I-girder web or an outstand compression element (i.e. an element bordered to only one edge) Table 3 in EN1993-1-1 should be used. These tables provide the limit slenderness between Classes. To determine the class of an element in a given cross-section this element is first assumed to be in Class 1 or 2 and then calculated with its plastic resistance. The Plastic Neutral Axis (PNA) location in the section is used to determine the limit slenderness of this element (between Class 2 and Class 3) and to justify the plastic assumption. If not, the elastic stress distribution at ULS (coming from the global cracked analysis and taking the construction phases of the structure into account) is used to determine the limit slenderness between Class 3 and Class 4. If the actual slenderness of the element exceeds this limit, this element is in Class 4.

EN1994-2 allows that a cross-section with Class 3 web and Class 1 or 2 flanges may be treated as an effective Class 2 cross-section. The cross-section is then justified according to its plastic resistance. The plastic resistance moment is calculated by assuming that the effective compressive parts of the web are limited to $20t_w$ i.e. by suppressing the web zone likely to buckle.

In a composite bridge the in-span cross-sections under sagging (positive) bending moment are usually in Class 1 or 2 (the compressive part of the web is very small due to a very high location of the PNA and the upper steel flange connected to a compressed concrete slab is in Class 1). On the other side the cross-section located in internal support regions under hogging (negative) bending moment are usually in Class 3 or 4 (fairly important part of the web in compression) EN1994-2, 5.5.2(3).

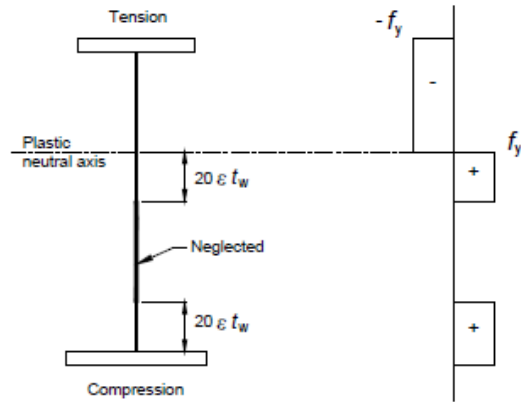


Figure 2.16 : Effective cross-section in Class 2

2.4.9 Cross-section justification principles

2.4.9.1 Bending resistance

Class 1 or 2 cross-sections can be checked by using the plastic or elastic bending resistance. Class 3 cross-sections are checked with the elastic bending resistance, or possibly reclassified as effective Class 2 cross-section and then checked with the plastic bending resistance. Class 4 cross-sections are also checked with the elastic bending resistance but by using the effective cross-section, reduced to take account of buckling.

2.4.9.2 Plastic verification

The location of the Plastic Neutral Axis (PNA) as well as the plastic resistance moment M are calculated by using the following design yield strengths for the materials: EN1994-2, 6.2.1.2

- structural steel (tension or compression):

$$f_{yd} = f_{yk} / \gamma_{M0} \quad (2.36)$$

- reinforcing steel (tension) :

$$f_{sd} = f_{sk} / \gamma_s \quad (2.37)$$

- concrete (compression) :

$$0.85 \cdot f_{cd} = 0.85 \cdot f_{ck} / \gamma_c \quad (2.38)$$

The strength of the concrete in tension and of the reinforcing steel bars in compression is neglected in the cross-section resistance.

Figures 2.18 illustrate very generally the plastic stress distribution used for an I-girder under sagging bending moment ,

For a High Strength Steel the concrete could be cracked because of too much compression. The subsequent reduced cross-section resistance is modeled by a reduction factor β which depends on the location of the PNA and is directly applied to $M_{pl,Rd}^+$.

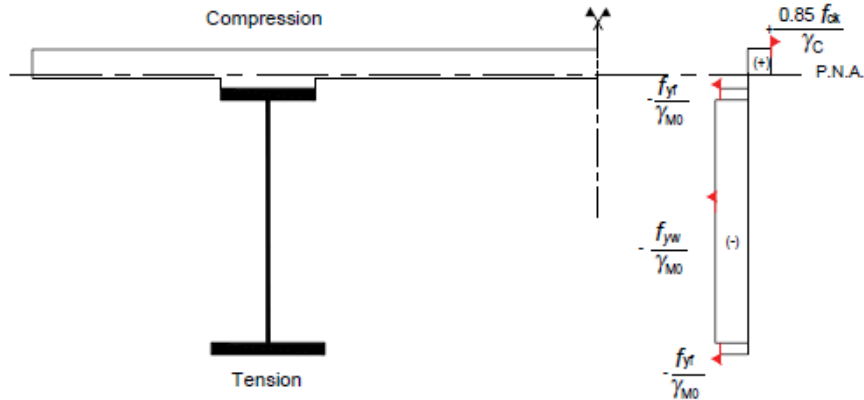


Figure 2.17 : Calculation of the design value of the positive plastic resistance moment $M_{pl,Rd}^+$

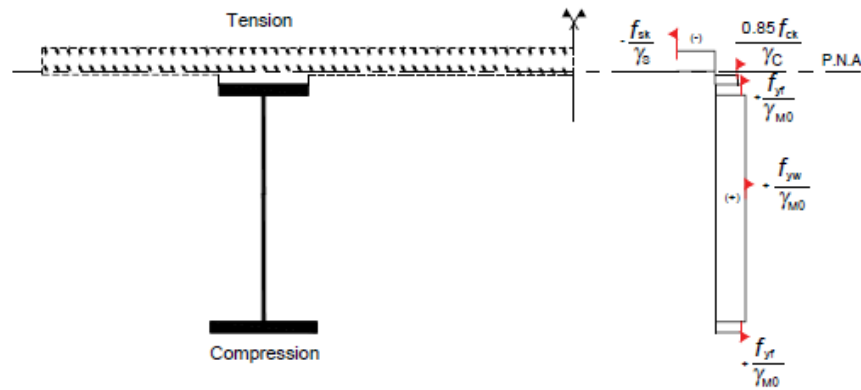


Figure 2.18 : Calculation of the design value of the negative plastic resistance moment

For Class 1 or 2 cross-sections, i.e. generally under sagging bending moment in the mid-span region, the ULS bending moment should be checked against the plastic resistance moment: $M_{Ed} \leq M_{pl,Rd}$.

In addition, M_{Ed} is calculated by a cracked elastic global analysis takes no account of the influence of a possible Class 1 or 2 cross-section yielding in the mid-span region

on the longitudinal M_{Ed} distribution. If the cross-section located at or near the adjacent internal support is in Class 3 or 4, and if the ratio of lengths of the spans adjacent to that support (shorter/ longer) is less than 0.6, the loading case leading to the maximum bending moment in span is close to that leading to the minimum bending moment at support. EN 1994-2 limits M_{Ed} to $0.9.M_{pl,Rd}$ in the Class 1 or 2 cross-section in span to avoid any moment redistribution which could be harmful.

2.4.10 Elastic verification

The limiting stresses at ULS are given per material in accordance with EN1994-2, 6.2.1.5(2) :

- f_{yd} for structural steel,
- f_{sd} for reinforcing steel bars,
- f_{cd} for concrete in compression.

The strength of concrete in tension is neglected.

Also, the elastic check could be performed with the stresses calculated in the mid-plane of the steel flanges instead of in the extreme fibres. EN1993-1-1, 6.2.1(9)

2.4.11 Effective cross-section for Class 4 section

For a Class 4 cross-section the stresses at ULS coming from the global analysis (and calculated with the gross area possibly reduced due to shear lag effect) are used to calculate the initial area A_c of the compressed part of the structural steel cross-section, and then the effective area $A_{c,eff} = \rho A_c$ of this compressed part (with a reduction factor $\rho < 1$) EN1994-2, 6.2.1.5(7).

The area A_c can be made up of several Class 4 elements (flanges and webs) and the calculation of $A_{c,eff}$ is thus iterative. Based on the initial stresses at ULS, a first calculation gives the reduction factor and the effective area for the first element. The ULS stresses are recalculated with the mechanical properties from this first effective cross-section and then used to determine the reduction factor and the effective area of the second element and so on EN1993-1-5, 4.4(4) note 1.

For a given Class 4 element ρ is calculated according to EN1993-1-5, section 4.5 when the element stiffened web of an I-girder.

Figure 2.20 suggests a method for an unstiffened I-girder cross-section (where only the web is in Class 4) under the bending moment $M_{Ed} < 0$ alone (most common situation). M_a is the part of the bending moment M_{Ed} resisted by the structural steel area alone and M_c is the part of M_{Ed} resisted by the composite area ($M_{Ed} = M_a + M_c$). The effective stresses to be checked are recalculated with the effective mechanical properties and the moments M_a and M_c .

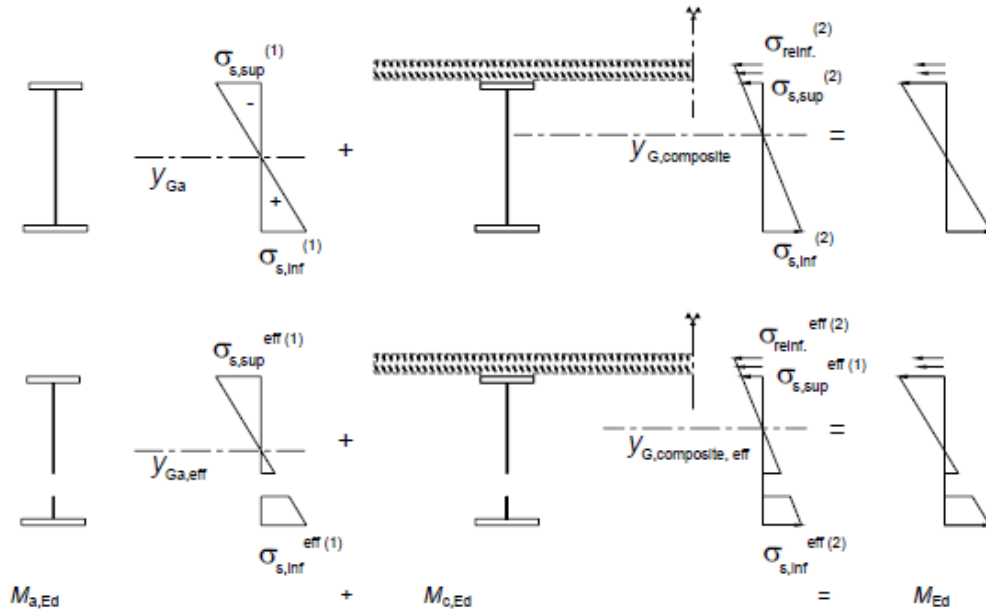


Figure 2.19 : Principle for recalculating the stresses in an effective cross-section

2.4.12 Shear resistance

Whatever the Class of the composite cross-section the criterion $V_{Ed} \leq V_{pl,a,Rd}$ should be checked where $V_{pl,a,Rd}$ is the plastic design shear resistance of the structural steel. If no torsion is applied to the cross-section, $V_{pl,a,Rd}$ is given by EN1994-2, 6.2.2 :

$$V_{pl,a,Rd} = A_v \frac{f_y}{\gamma_{Mc} \sqrt{3}} \quad (2.39)$$

where A_v is the structural steel shear area under V_{Ed} (normally the girder web area multiplied by a factor η which depends on the steel grade).

When the web becomes too slender it could buckle under V_{Ed} . Then the following criterion should be also checked:

$$V_{Ed} \leq V_{b,Rd} \quad (2.40)$$

where $V_{b,Rd}$ is the design resistance for shear buckling EN1993-1-5, 5.5(1) :

$$V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \leq \frac{\eta f_{yw}}{\gamma_{M1} \sqrt{3}} h_w t_w \quad (2.41)$$

$V_{bf,Rd}$ corresponds to the contribution from the flanges in the design shear buckling resistance. Although EN1993-1-5 suggests a method for calculating this contribution it is negligible compared to the contribution from the web in case of traditional bridge girders. In addition, if it is taken into account, it is also to be checked that the welds between web and flanges could transfer the shear force [EN1993-1-5, 5.2(2) and 9.3.5(1)].

$V_{bw,Rd}$ corresponds to the contribution from the web in the design shear buckling resistance.

The contribution from the reinforced concrete slab is neglected in the design plastic and design buckling shear resistances of a composite cross-section [EN1994-2, 6.2.2.2(1) and 6.2.2.3(2)].

When V_{Ed} is greater than half of $V_{Rd} = \min(V_{b,Rd} ; V_{pl,a,Rd})$, V_{Ed} reduces the bending resistance of the cross-section. The reduction to be taken into account depends on the cross-section Class [EN1994-2, 6.2.2.4(1)].

- For Class 1 or 2 cross-sections of an I-girder, the yield strength of the structural steel shear area A_v is reduced before calculating the design value of the plastic bending resistance $M_{pl,Rd}$ [EN1994-2, Figure 6.7].

When calculating $M_{pl,Rd}$ the shift in the position of the Plastic Neutral Axis (which is due to the change in the yield strength of the shear area A_v) is not taken into account [EN1994-2, 6.2.2.4(4)].

- For Class 3 or 4 cross-sections of an I-girder, EN1993-1-5 defines an interaction criterion:

The calculation of $M_{pl,Rd}$ for a cross-section with Class 4 elements only takes account of the effective area of the composite and/or steel flanges (due to shear lag effect as well as local buckling if the flange is in Class 4). Even if the web is in Class 4, its gross area is considered for evaluating $M_{pl,Rd}$.

$M_{f,Rd}$ is calculated with the same assumptions as $M_{pl,Rd}$ but neglecting the web area totally [EN1994-2, 6.2.2.5(2)].

The interaction criterion needs not to be checked for the cross-sections located less than $h_w/2$ from a support with a vertical stiffener [EN1993-1-5, 7.1(2)].

The bending and shear interaction can be checked under concomitant internal forces and moments.

2.4.13 Resistance of the headed stud shear connector

There are some limit dimensions for stud of beam;

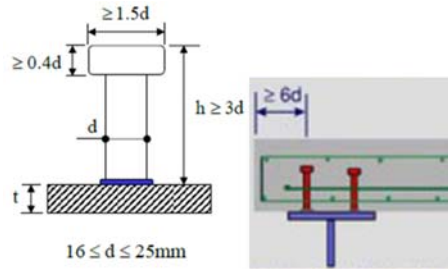


Figure 2.20 : Limit values for stud

As limit conditions some view of stud on the superstructure can be viewed on Figure 2.22 and 2.23.,



Limit State	Design resistance	National Annex
U.L.S.	$P_{Rd} = \frac{P_{Rk}}{\gamma_V}$	$\gamma_V = 1.25$
S.L.S.	$k_s P_{Rd}$	$k_s = 0.75$

$$P_{Rk} = \min [P_{Rk}^{(1)}; P_{Rk}^{(2)}] \quad (2.42)$$

Shank shear resistance

$$P_{Rk}^{(1)} = 0.8f_u \left\{ \frac{\pi d^2}{4} \right\} \quad (2.43)$$

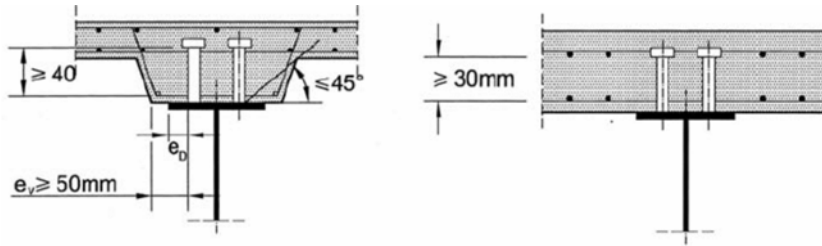
Concrete crushing :

$$P_{Rk}^{(2)} = 0.29\alpha d^2 \sqrt{f_{ck} E_{cm}} \quad (2.44)$$

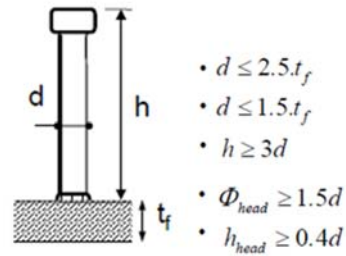
$$\text{If } 3 \leq \frac{h}{d} \leq 4 \text{ then } \alpha = 0.2 \left(\frac{h}{d} + 1 \right) \text{ else } \alpha = 1 \quad (2.45)$$

2.4.14 Detailing for shear connectors

for solid slabs,



to allow a correct welding of the connector : $25 \text{ mm} \leq e_D$



for a structural steel flange in tension, subjected to fatigue.

2.4.15 Longitudinal spacing between shear connectors rows

- to insure the composite behavior in all cross-sections :

$$e_{\max} = \min (800 \text{ mm}; 4 h)$$

where h is the concrete slab thickness

- if the structural steel flange in compression which is connected to the concrete slab, is a class 3 or 4 element :
- to avoid buckling of the flange between two studs rows :
- to avoid buckling of the cantilever e_D -long part of the flange :
- and if the used shear connectors are studs :

$$e_{trans,min} \geq 2.5.d \quad \text{for solid slabs}$$

$$e_{trans,min} \geq 4.d \quad \text{in other cases}$$



Figure 2.21 : View and detail of an upper flange connection [10]



Figure 2.22 : View of the lower flange connection of a steel girder [10]

2.5 Fatigue (connection and reinforcement)

In a composite bridge, fatigue verifications shall be performed for :

the structural steel details of the main girder (see EN1993-2 and EN1993-1-9)

the slab concrete (see EN1992-2)

the slab reinforcement (see EN1994-2)

the shear connection (see EN1994-2)

Two assessment methods in the Eurocodes which differ in the partial factor γ_{Mf} for fatigue strength in the structural steel :

Assessment method (National Choice)	Consequence of detail failure for the bridge	
	Low consequence	High consequence
Damage tolerant Required regular inspections and maintenance for detecting and repairing fatigue damage during the bridge life	$\gamma_{Mf}=1.0$	$\gamma_{Mf}=1.15$
Safe life No requirement for regular in-service inspection for fatigue damage	$\gamma_{Mf}=1.15$	$\gamma_{Mf}=1.35$

2.5.1 Damage equivalent stress range $\Delta\sigma_E$

In a given structural detail of the bridge which is subjected to repeated fluctuations of stresses due to traffic loads, a fatigue crack could initiate and propagate. The detail fails when the damage D in it reaches 1.0:

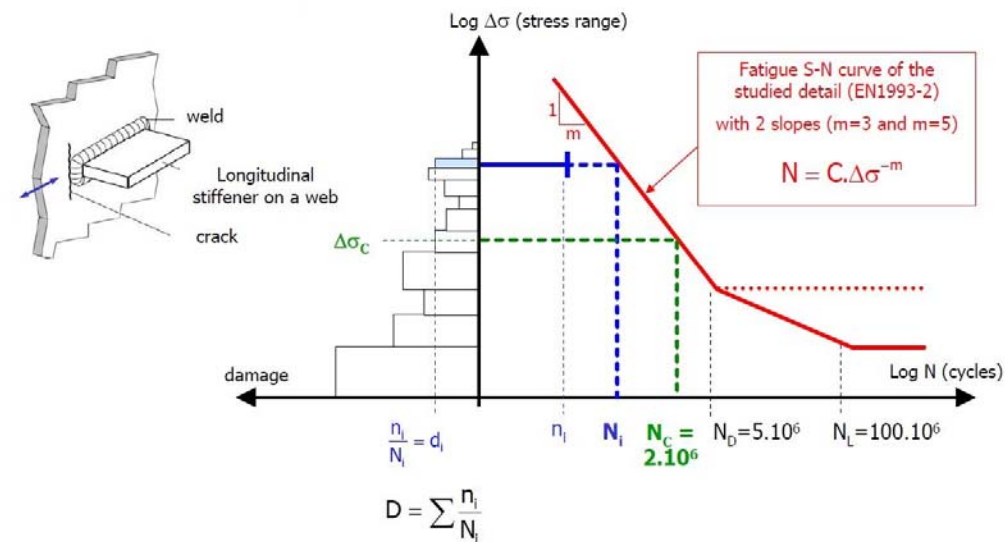


Figure 2.23 : Total damage in the detail

In term of D, the actual traffic $(n_i, \Delta\sigma_i)_i$ is equivalent to $n_E = \sum n_i$ cycles of the unique equivalent stress range $\Delta\sigma_E$.

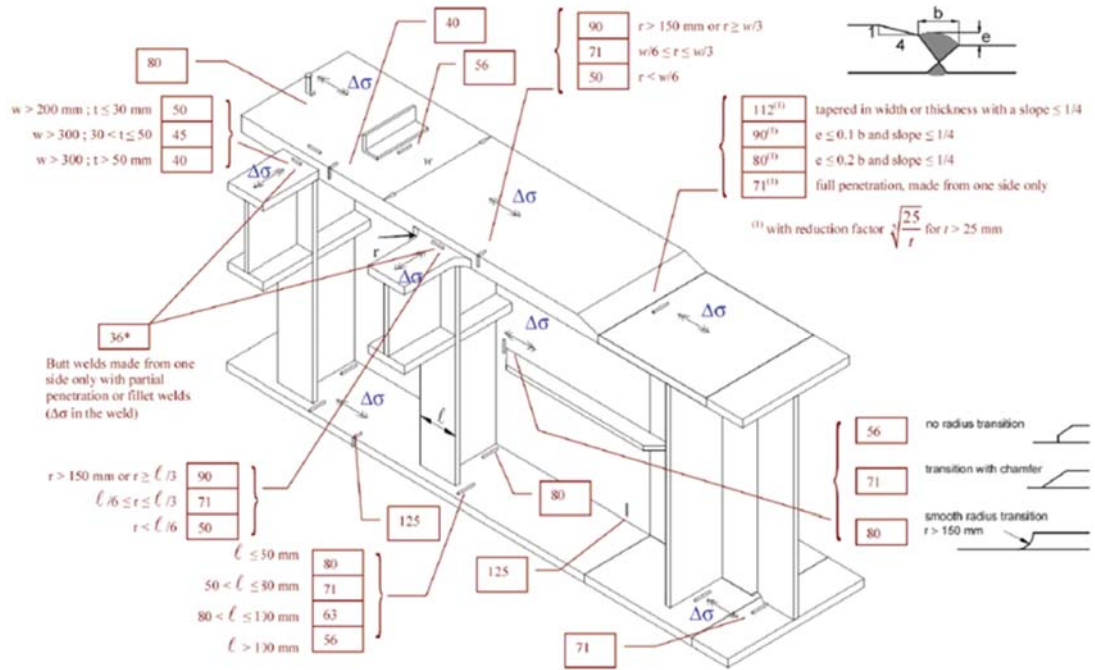


Figure 2.24 : Classification of typical structural details

2.5.2 Fatigue load model 3 «equivalent lorry» (FLM3)

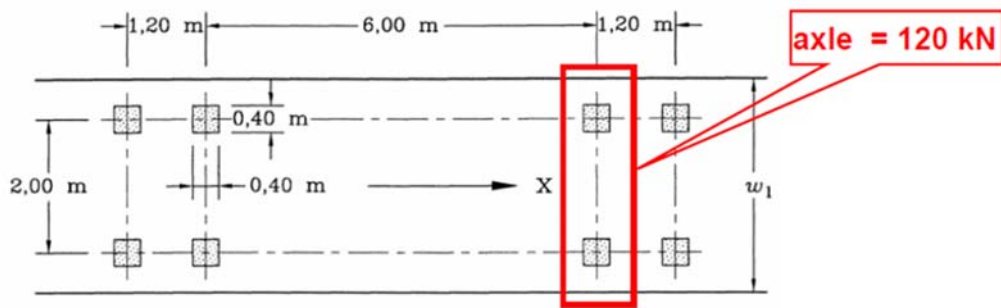


Figure 2.25 : Load detail of FLM-3 lorries

FLM-3 lorries are assumed to cross the bridge per year and per slow lane defined in the project

- every crossing induces a stress range $\Delta\sigma_p = |\sigma_{\max,f} - \sigma_{\min,f}|$ in a given structural detail
- the equivalent stress range $\Delta\sigma_E$ in this detail is obtained as follows :

where :

λ is the damage equivalence factor

Φ is the damage equivalent impact factor (= 1.0 as the dynamic effect is already included in the characteristic value of the axle load)

2.5.3 Damage equivalence factor λ

In a structural steel detail (in EN 1993-2) which represents the following parameters :

- λ_1 : influence of the loaded lengths, defined in function of the bridges spans (< 80 m) and the shape of the influence line for the internal forces and moments
- λ_2 : influence of the traffic volume
- λ_3 : life time of the bridge ($\lambda_3=1$ for 100 years)
- λ_4 : influence of the number of loaded lanes
- $\lambda_{\max}$: influence of the constant amplitude fatigue limit $\Delta\sigma_D$ at 5.10^6 cycles

For shear connection (in EN1994-2):

$$\lambda_v = \lambda_{v,1} \cdot \lambda_{v,2} \cdot \lambda_{v,3} \cdot \lambda_{v,4} \quad (2.46)$$

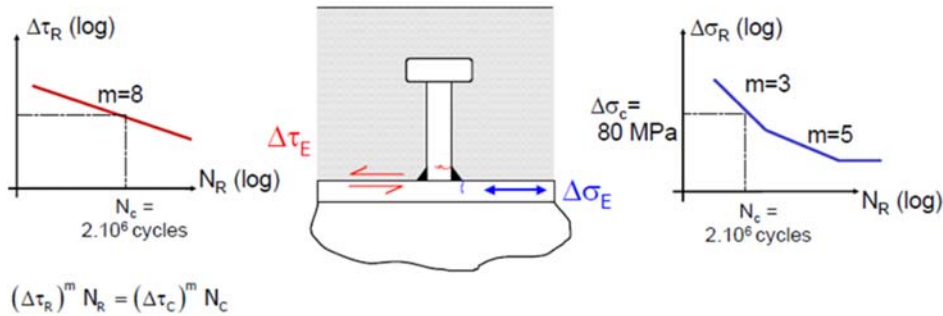
For reinforcement (in EN1992-2):

$$\lambda_s = \varphi_{\text{fat}} \cdot \lambda_{s,1} \cdot \lambda_{s,2} \cdot \lambda_{s,3} \cdot \lambda_{s,4} \quad (2.47)$$

For concrete in compression (in EN1992-2 and only defined for railway bridges):

$$\lambda_c = \lambda_{c,0} \cdot \lambda_{c,1} \cdot \lambda_{c,2,3} \cdot \lambda_{c,4} \quad (2.48)$$

Fatigue verifications for shear connectors,



2.5.4 Lateral torsional buckling of members in compression

It should be checked according to EN1993-1-5, 9.2.1(8) that the vertical web stiffeners do not buckle under torsion (which can occur for vertical T-shaped open stiffeners). The following criterion applies:

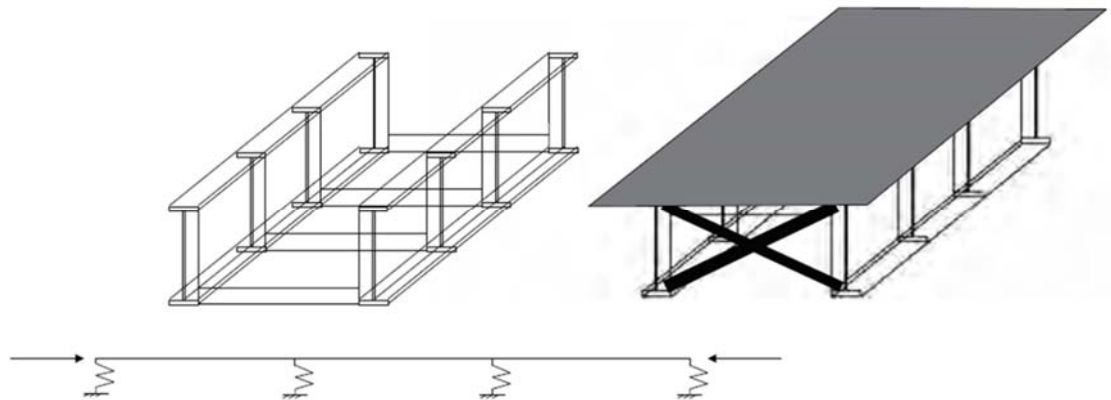
$$\frac{I_T}{I_P} \geq 5.3 \frac{f_y}{E_a} \quad (2.49)$$

Which,

I_T : The St. Venant torsional constant for the stiffener alone

I_P : The polar second moment of area of the stiffener alone around its edge welded to the web plate.

The LTB verification of the lower flange in a two-girder composite bridge under traffic loads is studied as the lateral column buckling of the isolated lower flange which is in compression around supports. This lower flange is then assumed to be laterally simply supported at piles and abutments (which means the design of a very rigid bracing transverse frame at any support, as it is usually provided due to the wind action transmission from the deck to the supports). The lower flange is also assumed to be laterally elastically supported at the bracing frames. The lateral stability of the flange is thus linked to the frame rigidity which is first calculated.



Rigidity C_d of bracing transverse frames,

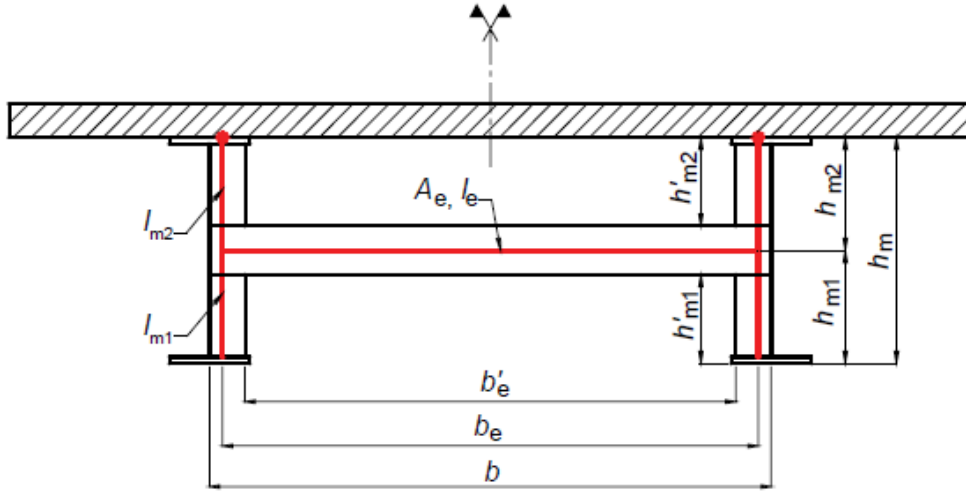


Figure 2.26 : Notations defining the modeled transverse frame [2]

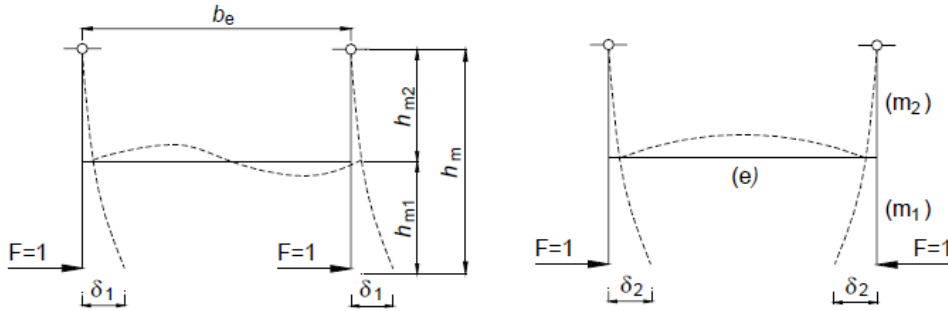


Figure 2.27 : Same direction forces (load case 1) & Opposite direction forces (load case 2) Load cases for the rigidity C_d calculation [2]

Three new flexibility terms linked to the shear resistant areas of the transverse brace girder Σ_e and of the vertical frame posts Σ_m are calculated for lateral displacements δ_1 and δ_2 . The stiffness C_d of the bracing frame is then given by

$$C_d = \min \left[\frac{1}{\delta_1}; \frac{1}{\delta_2} \right]. \quad (2.50)$$

3. PRELIMINARY DESIGN

The following is considered that according to EN 1991 :

- Selfweight
- Seismic actions
- Traffic loads

There are also other actions described in EN 1991, such as fire and snow loads, thermal actions (Simply-supports for steel beams does not have axial load as a reaction by variations in temperatures) which are usually irrelevant. Additional actions are foreseen in other EN Eurocodes, namely:

- Concrete creep and shrinkage (EN 1992)
- Settlements and earth pressures (EN 1997)

Theoretically, all possible combinations should be considered, but in most cases the effect of settlements and earth pressure are not critical for a bridge of that type.

For calculations, the following units are preferred:

- forces and loads kN, kN/m, kN/m²
- unit mass kg/m³
- unit weight kN/m³
- stresses and strengths N/mm² (= MN/m² or MPa)
- bending moments kN.m.

3.1 Actions and Material

3.1.1 Traffic loads analysis

According to EN 1991-2, Traffic loads were subjected to bridge. The sections are as follows:

- Forward
- Section 1 – General
- Section 2 – Classification of actions
- Section 3 – Design situations

- Section 4 – Road traffic actions and other actions specifically for road bridges
- Section 5 – Actions on footways, cycle tracks and footbridges
- Section 6 – Traffic actions and other actions specifically for railway bridges
- Annex A (informative) – Models of special vehicles for road bridges
- Annex B (informative) – Fatigue life assessment for road bridges assessment method based on recorded traffic
- Annex C (normative) – Dynamic factors $1 + \phi$ for real trains
- Annex D (normative) – Basis for the fatigue assessment of railway structures
- Annex E (informative) – Limits of validity of load model HSLM and the selection of the critical universal train from HSLM-A
- Annex F (informative) – Criteria to be satisfied if a dynamic analysis is not required
- Annex G (informative) – Method for determining the combined response of a structure and track to variable actions
- Annex F (informative) – Load models for rail traffic loads in transient design situations

3.2 General organisation for road bridges

3.2.1 Traffic load models

- Vertical forces: LM1, LM2, LM3, LM4

Load Model-1 (LM-1): Concentrated and distributed loads (main model – general local verifications)

Load Model-2 (LM-2): Single axle load (semi-local and local verifications)

Load Model-3 (LM-3): Set of special vehicles (general and local verifications)

Load Model-4 (LM-4): Crowd loading (general verifications)

(LM-1 and 3 which govern were subjected to calculation model)

Table 3.1 : Bases for the calibration of the main load models

Traffic Load Models	Characteristic values	Frequent values	Quasi-permanent values
Road bridges			
LM1	1000 year return period (or probability of	1 week return period for	Calibration in accordance with

Table 3.1 : (Continued) Bases for the calibration of the main load models

	exceedance of 5% in 50 years) for traffic on the main roads in Europe (factors equal to 1)	traffic on the main roads in Europe (factors equal to 1)	definition given in EN 1990.
LM2	1000 year return period (or probability of exceedance of 5% in 50 years) for traffic on the main roads in Europe (factor equal to 1.)	1 week return period for traffic on the main roads in Europe (factor equal to 1.)	Not relevant
LM3	Set of nominal values. Basic values defined in annex A are derived from a synthesis based on various national regulations.	Not relevant	Not relevant
LM4	Nominal value deemed to represent the effects of a crowd. Defined with reference to existing national standards.	Not relevant	Not relevant

3.2.1 Division of the carriageway into notional lanes**Table 3.2 :** Number and width of national lanes

Carriageway width w	Number of notional lanes	Width of a notional lane w_1	Width of the remaining area
$w < 5,4 \text{ m}$	$n_1 = 1$	3 m	$w - 3 \text{ m}$
$5,4 \text{ m} \leq w < 6 \text{ m}$	$n_1 = 2$	$\frac{w}{2}$	0
$6 \text{ m} \leq w$	$n_1 = \text{Int}\left(\frac{w}{3}\right)$	3 m	$w - 3 \times n_1$

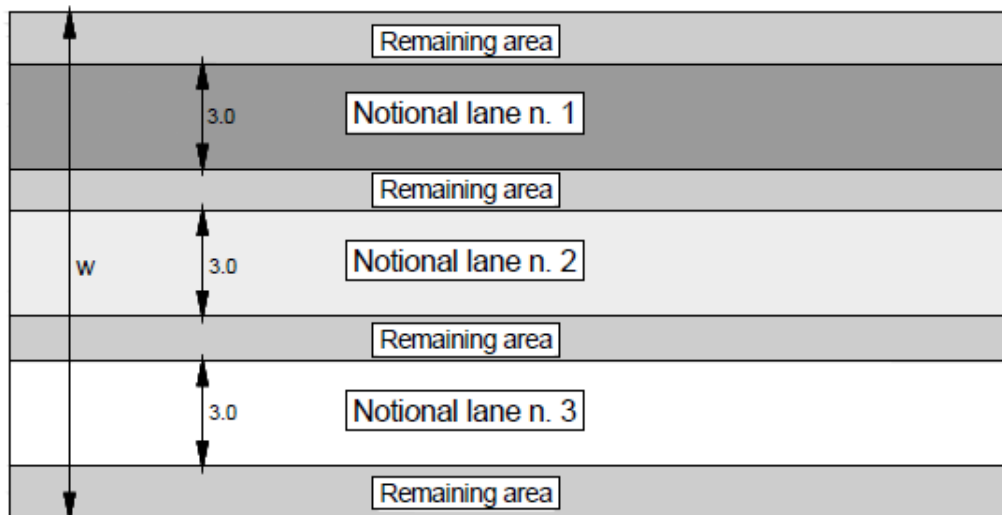


Figure 3.1 : Lane Numbering in the most general case

- 1 – Lane n-1 (3m)
- 2- Lane n-2 (3m)
- 3- Lane n-3 (3m)
- 4- Remaining area

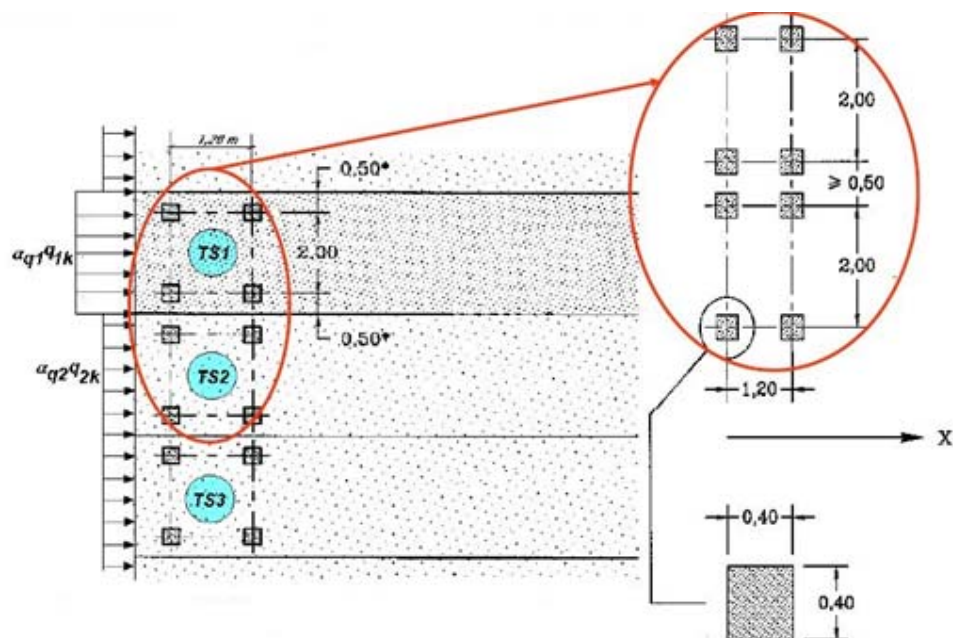


Figure 3.2 : The main load model (LM1)

TS: Tanden System

UDL: Uniformly distributed load

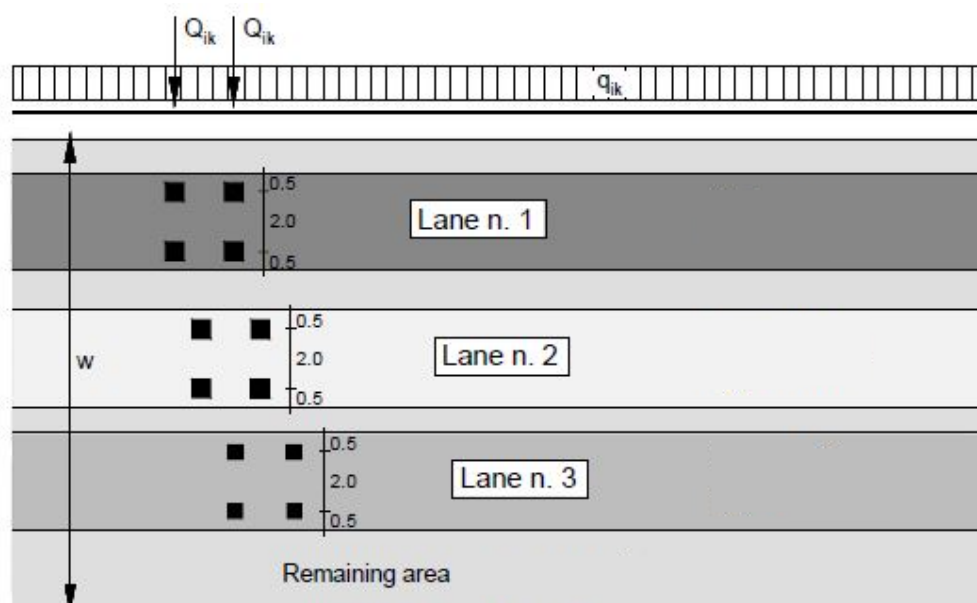


Figure 3.3 : The main load model for road bridges (LM-1)

For the determination of general effects, the tandems travel along the axis of the notional lanes.

For verifications, the heaviest tandem is positioned to get the most favourable effect. Where two tandems are located in two adjacent notional lanes, they are brought closer, the distance between axles being not less than 0,50.

Table 3.3 : Load Model 1 : Characteristic Values

Location	Tandem system TS	UDL system
	Axle loads Q_{ik} (kN)	q_{ik} (kN/m ²)
Lane Number 1	0	0.825
Lane Number 2	160	0.825
Lane Number 3	240	2.97
Lane Number 4	80	0.825
Lane Number 5	0	0.825
Remaining area (q_{rk})	0	0.825

The main load model (LM1):

Concentrated and uniformly distributed loads, covers most of the effects of the traffic of lorries and cars.

1st class : international heavy vehicle traffic

2nd class : « normal » heavy vehicle traffic

3rd class : « light » heavy vehicle traffic

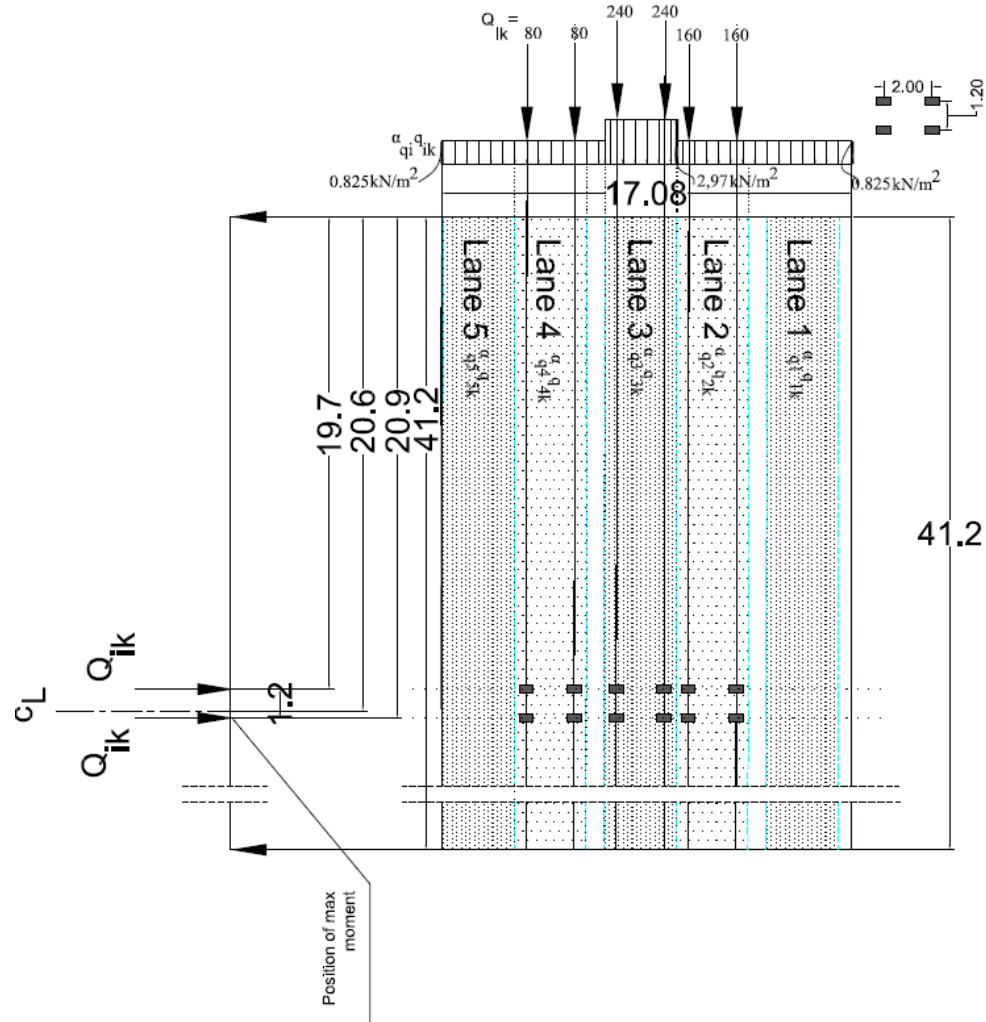


Figure 3.5 : Detail-2 of TS and UDL (in kN and m)

3.2.3 Detail of LM3 Loading (in kN and m)

Danish Road Directorate in “Vejledning til belastnings-og beregningsgrundlag” page 11 describes the configurations for Special Vehicles for LM3 loading.

For this study, it is checked for the load effects of Class 80 for Bridge Group II.

All the axles will fit on the deck with any of the three options for the centre axle spacing (1.4, 3.2 or 6.0m). The worst mid span moment will be produced with the highest concentration of load at the centre of the deck therefore, by inspection, the Class 80 with a centre axle spacing (10.16 m) will produce the worst mid span moment.

The maximum moment for a simply supported span occurs under the inner axle when the vehicle is positioned such that the mid span bisects the distance between the centroid of the load and the nearest axle.

With a 41.2m span and the Class 80 vehicle with equal axle loads, the inner axle is placed at 0,22m from the mid span.

The way of bridge belongs to Bridge group II (Clause 3) [12] which entails that a Class 80 standard vehicle for LM-3 according to EN 1991-2 Annex A is used.

Classes are listed as below,

Table 3.4 : List of axle configurations.

Classes	Axle Configuration (t and m)	Track width (m)
10		2,6
20		2,6
30		2,6
40		2,6
50		2,6
60		2,6
70		2,6
80		2,6
90		2,6
100		2,6

Class-80 is subjected to calculation model,

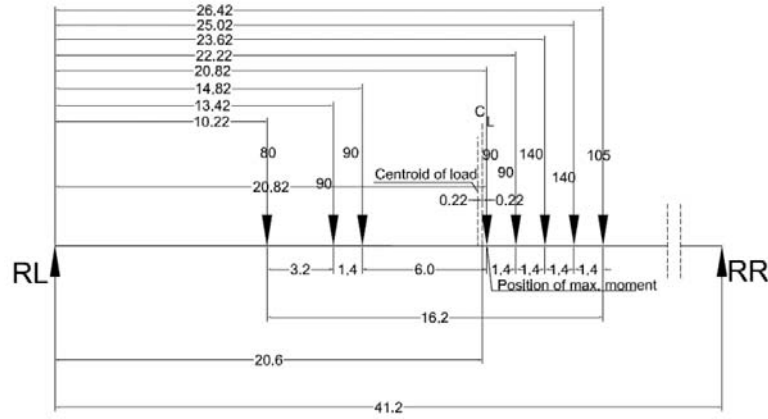


Figure 3.6 : Vehicle Load Class 80(in kN)

3.2.4 Horizontal forces: braking and acceleration

A characteristic braking force in accordance with EN 1991-2:2003, Q_{lk} , is a longitudinal force acting at the surfacing level of the carriageway. Q_{lk} , limited to 900 kN for the total width of the bridge, is calculated as a fraction of the total maximum vertical loads corresponding to Load Model 1 and applied on Lane Number 1. Figure 3.7 gives a graph of longitudinal force acting at the surfacing level of the carriageway.

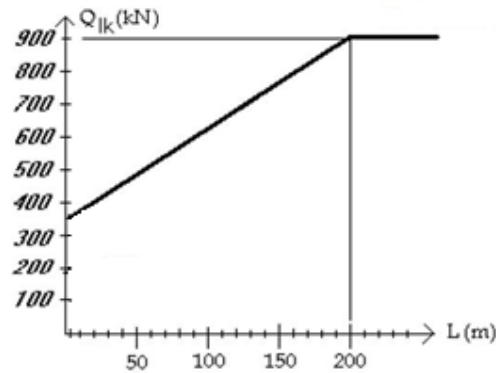


Figure 3.7 : longitudinal force acting at the surfacing level of the carriageway.

$$Q_{lk} = 0,6\alpha_{Ql}(2Q_{1k}) + 0,10\alpha_{q1}q_{1k}w_1L \quad (3.1)$$

$$180\alpha_{Ql} \text{ kN} \leq Q_{lk} \leq 900 \text{ kN} \quad (3.2)$$

L = length of the deck or of the part of it under consideration

3.3 Fatigue

3.3.1 Fatigue load models for road bridges

- Load Model Nr. 1 (FLM1) : Similar to characteristic Load Model Nr. 1

$$0,7 \times Q_{ik} - 0,3 \times q_{ik} - 0,3 \times q_{rk} \quad (3.3)$$

- Load Model Nr. 2 (FLM2) : Set of « fequent » lorries
- Load Model Nr. 3 (FLM3) : Single vehicle
- Load Model Nr. 4 (FLM4) : Set of « equivalent » lorries
- Load Model Nr. 5 (FLM5) : Recorded traffic

Models 1-2: just check whether max stress range $S < \text{fatigue limit}$

Models 3-4: damage assessment

Model 5 - general (additional assumptions might be necessary)

3.3.1.1 Fatigue LM1

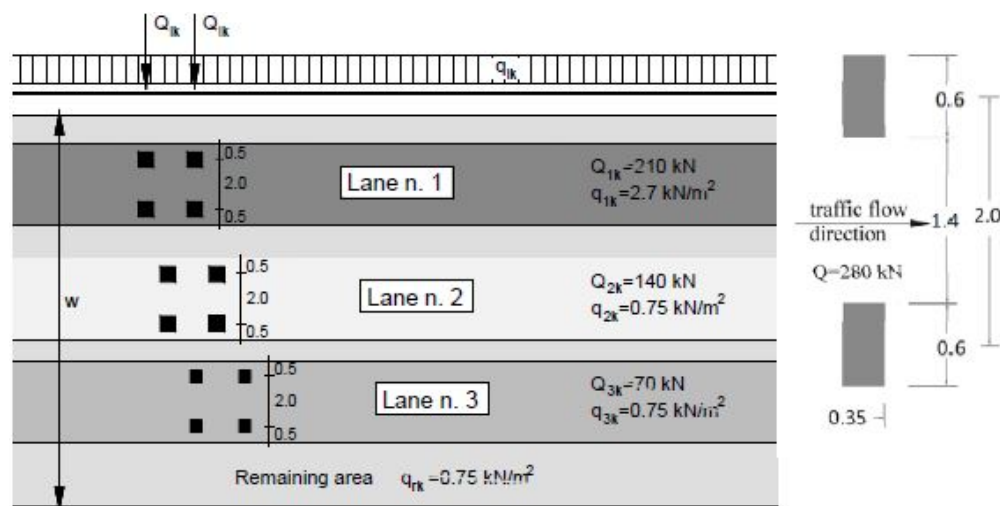
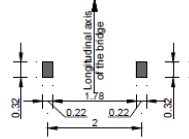
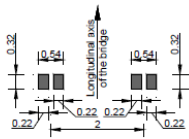
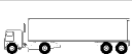
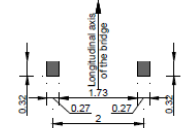


Figure 3.8 : Fatigue load model 1 and its local verification.

3.3.1.2 Fatigue LM 2

Loads of Fatigue LM 2 is listed below appendix A1.

Table 3.5 : Fatigue load model n. 2 – frequent set of lorries

LORRY SILHOUETTE	Interaxles [m]	Frequent axle loads [kN]	Wheel type (see table 3)	Geometrical definition
	4.5	90 190	A B	
	4.20 1.30	80 140 140	A B B	
	3.20 5.20 1.30 1.30	90 180 120 120 120	A B C C C	
	3.40 6.00 1.80	90 190 140 140	A B B B	
	4.80 3.60 4.40 1.30	90 180 120 110 110	A B C C C	

3.3.1.3 Fatigue LM 3

The fatigue load model FLM3, according to EN1991-2, 4.6.4, is used to calculate the longitudinal internal forces and moments in the bridge. This is a single-vehicle model made up of 4 axles (120 kN per axle). It moves in the middle of the slow lanes defined in the design. The contact surface of each wheel is a square with sides of 0.40 m (see Figure 3.9).

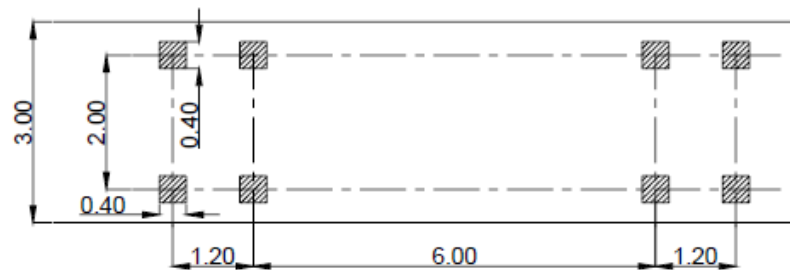
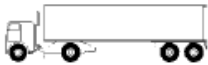


Figure 3.9 : Fatigue Vehicle of the fatigue load model FLM3

3.3.1.4 Fatigue LM 4

Loads of Fatigue LM 4 is listed below table 3.6. Fatigue Load Model Nr.3 (FLM3) is subjected to the calculation model.

Table 3.6 : Fatigue load model n. 4 – equivalent set of lorries

LORRY SILHOUETTE			TRAFFIC TYPE		
			Long distance	Medium distance	Local traffic
LORRY	Axle spacing [m]	Equivalent Axle loads [kN]	Lorry percentage	Lorry percentage	Lorry percentage
	4.5	70 130	20.0	40.0	80.0
	4.20 1.30	70 120 120	5.0	10.0	5.0
	3.20 5.20 1.30 1.30	70 150 90 90 90	50.0	30.0	5.0
	3.40 6.00 1.80	70 140 90 90	15.0	15.0	5.0
	4.80 3.60 4.40 1.30	70 130 90 80 80	10.0	5.0	5.0

Damage equivalent factor λ

The damage equivalent factor is given by:

$$\lambda = \prod_{i=1}^4 \lambda_i \leq \lambda_{\max} \quad (3.4)$$

Factor λ_1

The factor λ_1 takes into account the damage effects induced by the traffic volume following the length L of the influence line of the considered longitudinal internal force or moment in the deck. It also includes a « transit factor » from N_0 cycles per year to $2 \cdot 10^6$ cycles within 100 years.

Factor λ_2

λ_2 accounts for the traffic composition:

$$\lambda_2 = \frac{Q_{m1}}{Q_0} \left(\frac{N_{obs}}{N_0} \right)^{\frac{1}{5}} \quad (3.5)$$

Factor λ_3

The value of λ_3 according to EN1993-2, 9.5.2(5), follows the required design life of the structure.

For a bridge this is generally 100 years and thus $\lambda_3 = 1.00$.

Factor λ_4

λ_4 in accordance with EN1993-2, 9.5.2(6), takes into account the effects of the heavy traffic on the other additional slow lanes defined in this study. In the case of a single slow lane, $\lambda_4 = 1.0$. In the present case, the factor depends on the transverse influence of each slow lane on the internal forces and moments in the main girders:

$$\lambda_4 = 1.0$$

3.4 Characteristic ULS combinations

In the permanent design situation for design justifications of structural members, the following fundamental ULS combinations in the appendix A1 were considered.

3.5 Characteristic SLS combinations

In the permanent design situation for design justifications of structural members, the following fundamental SLS combinations in the table appendix A1 were considered.

3.5.1 Used material

- Shear connectors : stainless steel headed studs with $f_u = 483$ Mpa
 $f_y = 206$ Mpa
- Reinforcement :Dublex S31803 high bond bars with characteristic value of the yield strength of reinforcing steel $f_{sk} = 480$ Mpa
- Specified ultimate tensile strength $f_u = 680$ Mpa
- Concrete C35/45 defined in EN1992 : $f_{ck,cyl}$ (at 28 days) = 35 MPa
 $f_{ck,cube}$ (at 28 days) = 45 MPa
 $f_{ctm} = -3.2$ Mpa

3.5.2 For ultimate limit state (ULS):

Design situation	γ_c (concrete)	γ_s (reinforcement)	γ_M (structural steel)		γ_v (studs)
Persistent Transient	1.5	1.15	$\gamma_{M0} = 1.0$ $\gamma_{M1} = 1.0$ $\gamma_{M2} = 1.25$	Yielding, local instability Resistance of members to instability Resistance of joints	1.25
Reference	EN1992-1-1, 2.4.2.4		EN1993-2, 6.1 and Table 6.2		EN1994-2, 2.4.1.2 + National Annex

3.5.3 For fatigue ultimate limit state:

$\gamma_{C,fat}$ (concrete)	$\gamma_{S,fat}$ (reinforcement)	γ_{Mf} (structural steel)			$\gamma_{Mf,s}$ (studs)
1.5	1.15	Assessment method	Low consequence of failure	High consequence of failure	1.25
		Damage tolerant	1.0	1.15	
		Safe life	1.15	1.35	
EN1992-1-1, 2.4.2.4		EN1993-1-9, Table 3.1			EN1994-2, 6.8.2 + National Annex

In bridge design the danish national annexes have adopted the safe life concept (100 years).

For Serviceability Limit State (SLS):

γ_c (concrete)	γ_s (reinforcement)	$\gamma_{M,ser}$ (structural steel)	γ_v (studs)
1.0	1.0	1.0	1.25
EN1992-1-1, 2.4.2.4		EN1993-2, 7.3 (1)	Note: The stud resistance P_{Rk} is modified between SLS and ULS, and not the value of γ_v . EN1994-2, 6.8.1 (3)

For concrete and reinforcement the values of γ_c and γ_s are not used in practice during design verifications.

3.6 Seismic design of bridges with Eurocode 8

Eurocode 8 - Design of structures for earthquake resistance

- EN1998-1: General rules, seismic actions and rules for buildings
- EN1998-2: Bridges
- EN1998-3: Assessment and retrofitting of buildings
- EN1998-4: Silos, tanks and pipelines
- EN1998-5: Foundations, retaining structures and geotechnical aspects
- EN1998-6: Towers, masts and chimneys

3.6.1 Scope of EN 1998

EN 1998 applies to the design and construction of buildings and civil engineering works in seismic regions. Its purpose is to ensure that in the event of earthquakes: human lives are protected; damage is limited; and structures important for civil protection remain operational.

3.6.2 Performance requirements and compliance criteria

3.6.2.1 Fundamental requirements

Structures in seismic regions shall be designed and constructed in such a way that the following requirements are met, each with an adequate degree of reliability.

3.6.2.2 No-collapse requirement

The structure shall be designed and constructed to withstand the design seismic action without local or global collapse, thus retaining its structural integrity and a residual load bearing capacity after the seismic events (even with considerable damage).

For ordinary structures this requirement should be met for a reference seismic action with 10 % probability of exceedance in 50 years (recommended value) i.e. with 475 years Return Period.

3.6.2.3 Damage limitation requirement

The structure shall be designed and constructed to withstand a seismic action having a larger probability of occurrence than the design seismic action, without the occurrence of damage and the associated limitations of use, the costs of which would be disproportionately high in comparison with the costs of the structure P_{DLR} , in 10 years and a return period, T_{DLR} .

3.6.2.4 Intended seismic behaviour

- Ductile (D)
- Limited ductile/essentially elastic (LD)

3.6.2 Earthquake Resistance of designed Structure

for 2% damping

a_g : 0.1 g ground acceleration

S_d : 0.1992 g

M_{eqx} :2.94 kN.m applied moment on the pier transverse direction of bridge

M_{eqy} :342.1175 kN.m applied moment longitudinal direction of bridge

η : 1.2 correction factor (for 2% damping)

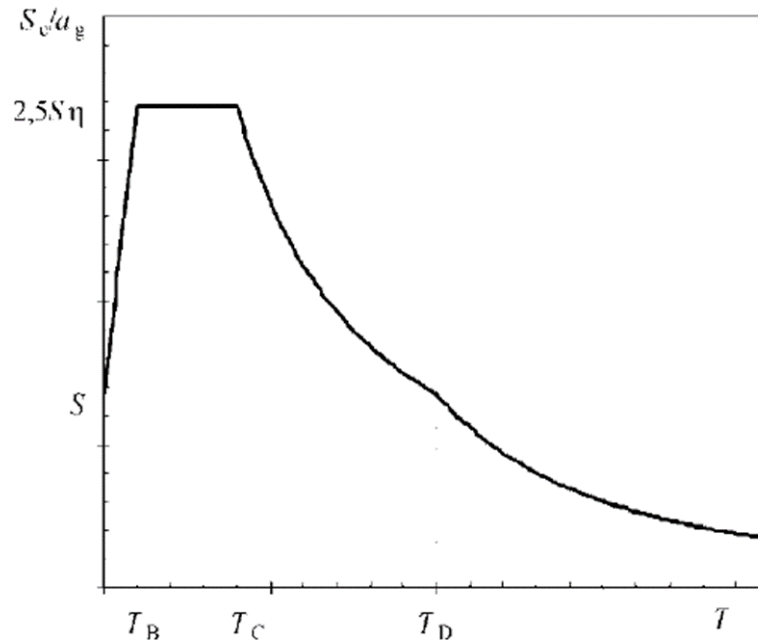


Figure 3.10 : Basic shape of the elastic response spectrum in EN 1998-1

$$S_d = a_g \cdot S \cdot \eta_d \cdot 2.5/q \quad (3.6)$$

It is determined that Earthquake effect does not govern live loads.

3.7 Description of the Bridge

In this study, a design of composite with stainless steel of the bridge “Egebækvej” will be compared with a danish traditional post-tensioned bridge on Helsingør motorway, Denmark..

3.7.1 The geometry of the bridge “Egebækvej”

The bridge “Egebækvej” which has one center support and the deck of spans are 31.2 m & 41.2 m for cost. In addition to that, 17,0 m between the crash barriers and min. vertical clearance is 4.63 m



Figure 3.11 : Plan of Bridge “Egebækvej”

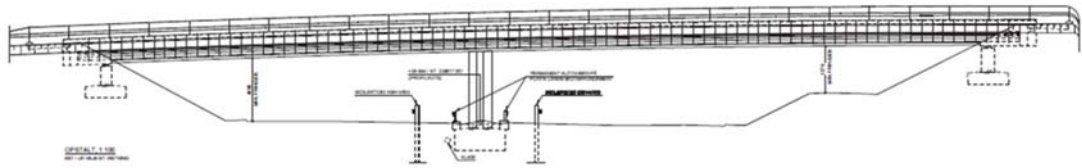


Figure 3.12 : The deck of spans: 31.2 m & 41.2 m longitudinal section of bridge

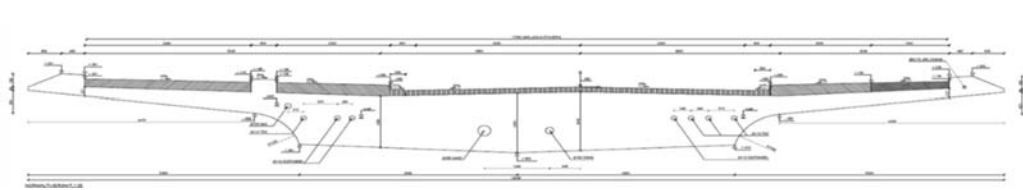


Figure 3.13 : Cross Section Of Bridge “Egebækvej”

4. CALCULATIONS & CHECK OF CROSS-SECTION AT MID-SPAN

4.1 Geometry and Stresses

At mid-span P1-P2 at ULS the concrete slab is in compression over its whole height. Its contribution is therefore taken into account in the cross-section resistance.

The stresses are subsequently calculated with the composite mechanical properties and obtained by summing the various steps whilst respecting the construction phases.

4.2 Cross-section of Composite Beam

Properties of section:

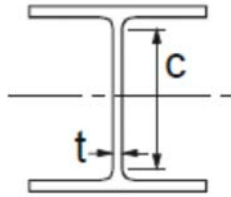


Figure 4.1 : Cross-section of beam

Section Properties:

f_y : f_{yw} : f_{yf} : 450 Mpa the yield strength

E : 200 Gpa and E_s : 200 Gpa

E_c : 34 Gpa

f_{ck} : 35 Mpa

f_{sk} : 480 Mpa yield strength

The internal forces and moments in this cross-section are:

Table 4.1 : Short list of moments on superstructure

ULS		
Equation& Load Combinations	Table Nr.	Moment kN.m
6.10a	3.6	7306.7

Table 4.1 : (Continued) Short list of moments on superstructure

Set B: 6.10b-1	3.6	14727.128
Set C: 6.10-3	3.6	15073.89
SLS		
Equation& Load Combinations	Table Nr.	Moment kN.m
6.14b-1	3.8	12189.48
6.15b-1	3.9	9260.81

4.3 Detail of Composite Cross-section

V_{ED} : 980 kN design shear force

M_{ED} :16000 kNm the design bending moment

for concrete at ULS

N_{ED} : 471.24 kN Design axial force Breaking force

F_{ED} :1960 kN Vehicle Class 80 + selfweight of composite section

To classify the steel web, the position of the Neutral Axis (NA) is determined as follows:

First reduced the concrete to an equivalent with of steel.

4.3.1 Effectives width of concrete slab (ULS and SLS) EN 1994-2

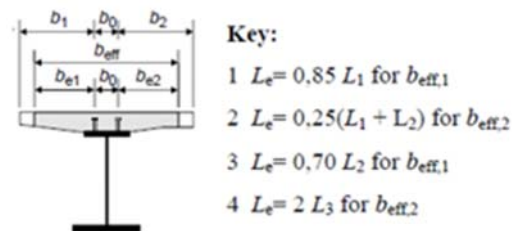


Figure 4.2 : Effectives width of concrete slab

$L_1 = 31.3$ m

$L_2 = 41.3$ m

$L_3 = 0$ m

$L = 72.6$ m

$b_{eff} : 2660$ mm

$$B : b_{eff} \frac{E_c}{E_s} : 452 \text{ mm}$$

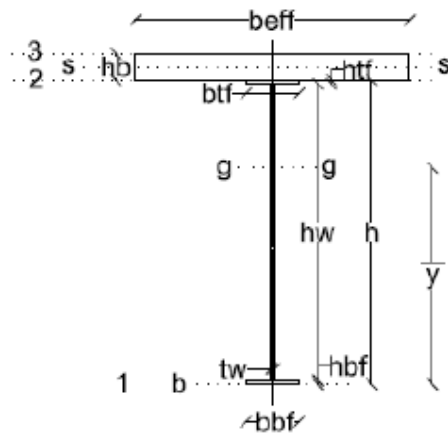


Figure 4.3 : Cross Section Dimensions

$h : 2300 \text{ mm}$
 $h_w : 2240 \text{ mm}$
 $b_{tf} : 400 \text{ mm}$
 $t_w : 18 \text{ mm}$
 $h_{tf} : 30 \text{ mm}$
 $b_{bf} : 400 \text{ mm}$
 $h_{bf} : 30 \text{ mm}$
 $h_b : 300 \text{ mm}$

Table 4.2 : The position of the centroid y

	Area mm^2	$\bar{y} \text{ mm}$	$A \bar{y} \text{ mm}^3$
A1 Concrete	135660	2450	3.3237×10^8
A2 top flange	12000	2285	2.742×10^7
A3 web	40320	1150	4.6368×10^7
A4 bottom flange	12000	15	1.8×10^5
Total	19998		4.0634×10^8

$\bar{y} : 2031.8782 \text{ mm}$ The position of the centroid.

Next finding the second moment of area about the horizontal centroid axis.

Used the parallel axis theorem.

h: distance from horizontal axis to centre of the part.

Table 4.3 : Table of second moment of area

	$BD^3/12 \text{ mm}^4$	h (mm)	$A h^2 (\text{mm}^4)$	$I \text{ mm}^4$
A1 Concrete	1.0175×10^9	418.1218	2.3717×10^{10}	2.4734×10^{10}
A2 top flange	9×10^5	253.1218	7.6885×10^8	7.6975×10^8
A3 web	1.6859×10^{10}	881.8782	3.1357×10^{10}	4.8216×10^{10}
A4 bottom flange	9×10^5	2016.8782	4.8814×10^{10}	4.8814×10^{10}
Total				1.2253×10^{11}

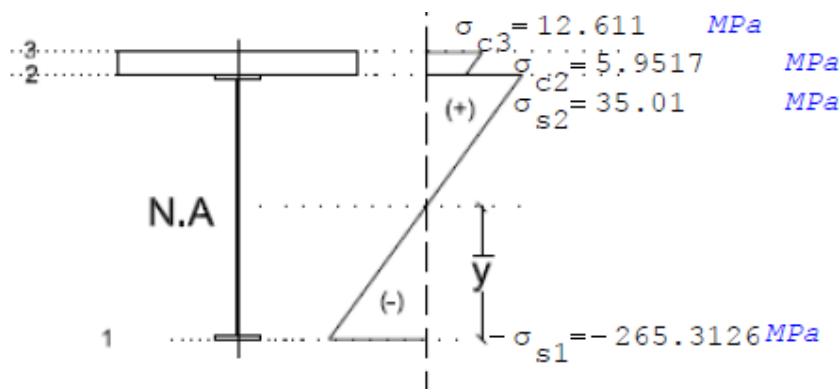


Figure 4.4 : Stresses at ULS in cross-section at mid-span P1-P2

4.3.2 Lower flange in tension therefore in class 1

The upper flange is composite and connected following the recommendations of EN1994-2, 6.6, therefore in Class 1.

To classify the steel web, the position of the Plastic Neutral Axis (PNA) is determined as follows:

Design plastic resistance of the concrete in compression:

$$F_c : A_c \frac{f_{ck}}{\gamma_c} \quad F_c: 18.62 \text{ MN} \quad (4.1)$$

The reinforcing steel bars in compression are neglected.

Design plastic resistance of the structural steel upper flange:

$$F_{fs} : A_{fs} \frac{f_{yf}}{\gamma_{M0}} \quad F_{fs}:5.4 \text{ MN} \quad (4.2)$$

Design plastic resistance of the structural steel web:

$$F_w : A_w \frac{f_{yw}}{\gamma_{M0}} \quad F_w:18.1 \text{ MN} \quad (4.3)$$

Design plastic resistance of the structural steel lower flange:

$$F_{fi} : A_{fi} \frac{f_{yf}}{\gamma_{M0}} \quad F_{fi}:5.4 \text{ MN} \quad (4.4)$$

From $F_c \leq F_{fs} + F_w + F_{fi}$ and $F_c + F_{fs} \geq F_w + F_{fi}$ the PNA is deduced to be located in the structural steel upper flange at a distance x from the extreme upper fibre of this flange.

Writing the forces equilibrium around the PNA deduces:

$$x: 28.6778 \text{ mm} \quad (4.5)$$

As the PNA is located in the upper flange the whole web is in tension and therefore in Class 1. The cross-section at mid-span P1-P2 is in Class 1 and is checked by a plastic section analysis.

4.3.3 Bending resistance check

The design plastic resistance moment is calculated from the position of the EN1994 - 2, 6.2.1.2(1). According to the clause, location of PNA is determines as 28.68 mm.

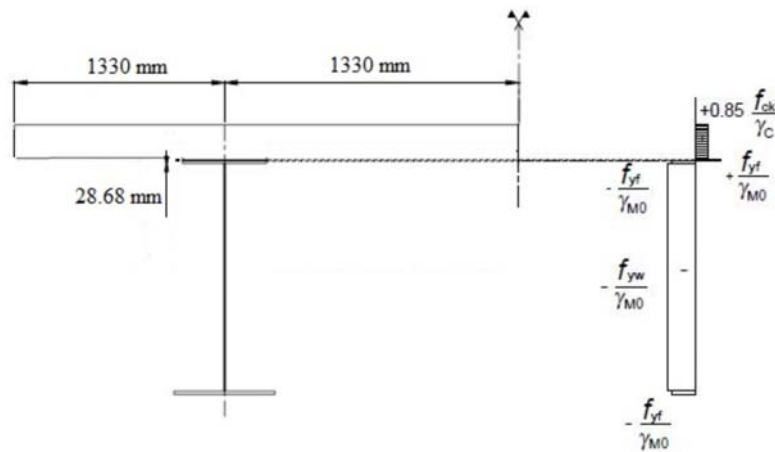


Figure 4.5 : Location of PNA

4.3.4 Design plastic resistance moment at mid-span P1-P2

$M_{ED}: 16000 \text{ kNm} < M_{plRd}: 24276.9 \text{ kNm}$ is verified.

4.1.1 Shear resistance check, as EN1993-1-5, 5.1(2) ,

$$\frac{t_w}{t_w} : 124.4 > \frac{31\varepsilon}{\eta} \sqrt{k_\tau} : 43.32 \quad (4.6)$$

The web (stiffened by the vertical stiffeners) should be checked in terms of shear buckling (EN1993-1-5, 5.1(2))

The maximum design shear resistance is given by

$$V_{Rd} : \min(V_{b,Rd}; V_{pl,Rd}) \text{ EN1994-2, 6.2.2}$$

Where

$$V_{pl,Rd} : 12.57 \text{ MN EN1994-2, 6.2.2}$$

$\nu: 0.3$ Poisson's ratio

$$\sigma_E : 11.67 \text{ MPa}$$

$$\tau_{cr} : 66 \text{ Mpa}$$

$$\overline{\lambda_w} : 1.98 > 1.08$$

$$\chi_w : 0.51 \text{ EN1993-1-5, Table 5.1}$$

$$V_{bwRd} : \min(5346; 12570) : 5346 \text{ kN}$$

$$V_{ED} : 980 \text{ kN} < V_{bwRd} : 5346.6058 \text{ kN}$$

4.3.5 M, V interaction check

$$V_{ED} : 980 \text{ kN} < 0.5 V_{bwRd} : 2673.3029 \text{ kN}$$

There is no need to check the M, V interaction.

4.3.6 Alternative: elastic section analysis

Whatever the Class a cross-section has, it can be justified by an elastic section analysis. Compared with the previous plastic section analysis, only the bending resistance check has to be performed again (EN1994-2, 6.2.1.1(2)).

This gives successively :

$$\sigma_{s,\text{sup}} : \sigma_{s1} : 265.3126 \text{ MPa} \leq \frac{f_{yf}}{\gamma_{M0}} : 450 \text{ MPa (upper flange)} \quad (4.7)$$

$$\sigma_{c,\max} : \sigma_{c3} : 12.611 \text{ MPa} \leq f_{cd} \frac{f_{ck}}{\gamma_c} : 23.33 \text{ MPa} \text{ (concrete in compression)} \quad (4.8)$$

4.4 Fatigue Assessment

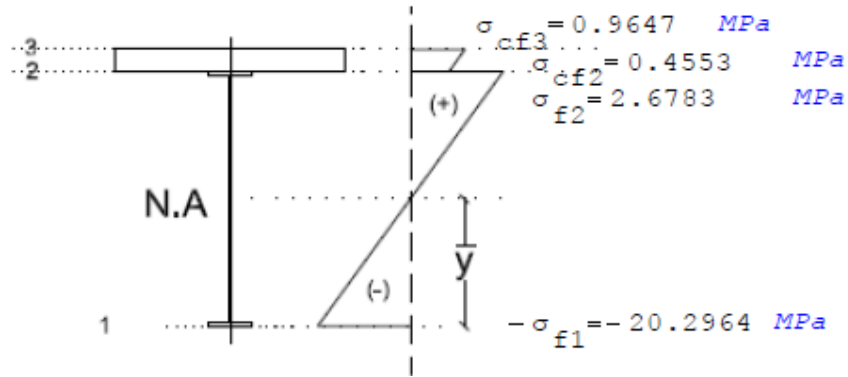


Figure 4.6 : Moment of FLM3

Maximum stress in flange due to passage of FLM3 vehicle in lane 1: 3 N/mm² (compression)

Minimum stress in flange due to passage of FLM3 vehicle in lane 1: -21 N/mm² (tension)

4.4.1 Design value of fatigue strength

Design value,

$$\frac{\delta \sigma_c}{\gamma_{Mf}} \quad (4.9)$$

The value of $\delta \sigma_c$ depends on the detail category. The design value of the fatigue strength is:

For a welding bearing plate: $\frac{36}{\gamma_{Mf}} : 32.73 \text{ MPa}$

For the attachment of bearing stiffeners: $\frac{80}{\gamma_{Mf}} : 72.73 \text{ MPa}$

Stress of web, 24.3 MPa < $\frac{36}{\gamma_{Mf}} : 32.73 \text{ MPa}$

4.5 Shear Connectors

Stud shear connectors 25 mm diameter 200 mm height (type SD1 to EN ISO 13918) are assumed, with $f_u = 450 \text{ N/mm}^2$

The resistance of a single stud is $f_{us}: 482.6 \text{ Mpa}$

$$\frac{P_{rk}}{\gamma_v} : 151.6133 \text{ kN} \quad (4.10)$$

$$n = \frac{V_{L1} + N_{ED}}{\frac{P_{rk}}{\gamma_v}} : \frac{16298 \text{ kN}}{151.6133 \text{ kN}} : 107 \text{ units} \quad (4.11)$$

space of stud (s): 0.25 m

Length of beam (L): 41.2 m

number of line of stud (nl): 2

used number of stud (ns): 330

Shear force per a stud: 50 kN

$\Delta\sigma : 100.7357 \text{ MPa}$

full loading on the bridge for fatigue of stud for DC of stud is 90;

$$2 \times 10^6 \times \left(\frac{90}{\Delta\sigma}\right)^3 : 1.4263 \times 10^6 \quad (4.12)$$

fatigue does not govern stud.

in flange, Dc of stud welding is 80;

$$2 \times 10^6 \times \left(\frac{80}{\Delta\sigma_2}\right)^3 : 4.8317 \times 10^6 \quad (4.13)$$

fatigue does not govern welding of stud.

4.6 Resistant Design of Earthquake

For checking of resistance of Earthquake below values were calculated;

For 2% damping, $\eta_d: 1.19$

peak of elastic response spectrum $2.5 \times S_x \eta_d: 2.9881$

$a_g : 0.1 \text{ g}$ ground acceleration

$S_d : 0.1992 \text{ g}$

for road bridge $\Psi_2 : 0.2$

$F_{pier}: 17.4229 \text{ kN}$

$F_{cap}: 27.6337 \text{ kN}$

Fgirder: 252.74 kN

Fvehicle: 147.3009 for 4 vehicle Class-80

M_{eqx} : $2.9352 \cdot 10^5$ kN.m applied moment (x) on the pier

P_{eq} : 5192.1547 kN vertical load

$1.4 \times P_{eq}$: 7269.0166 kN vertical load with factor

M_{eqy} : 342.1175 applied moment (y) on the pier

4.7 LTB Check of Structure

The determination of the critical bending moment is based on the frame model. The model takes into account the lateral displacement of the bottom flange causing bending of the steel web and rotation of the top flange that is resisted by bending of concrete slab.

4.7.1 Non- composite Section and deflection of non-composite section

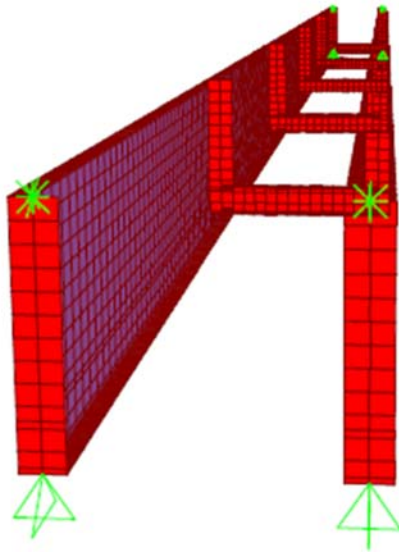


Figure 4.7 : 3-d Non-composite section of mathematics model

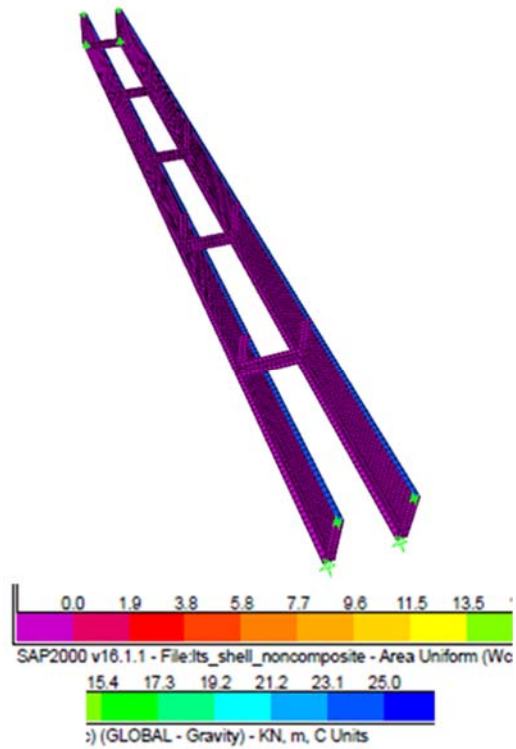
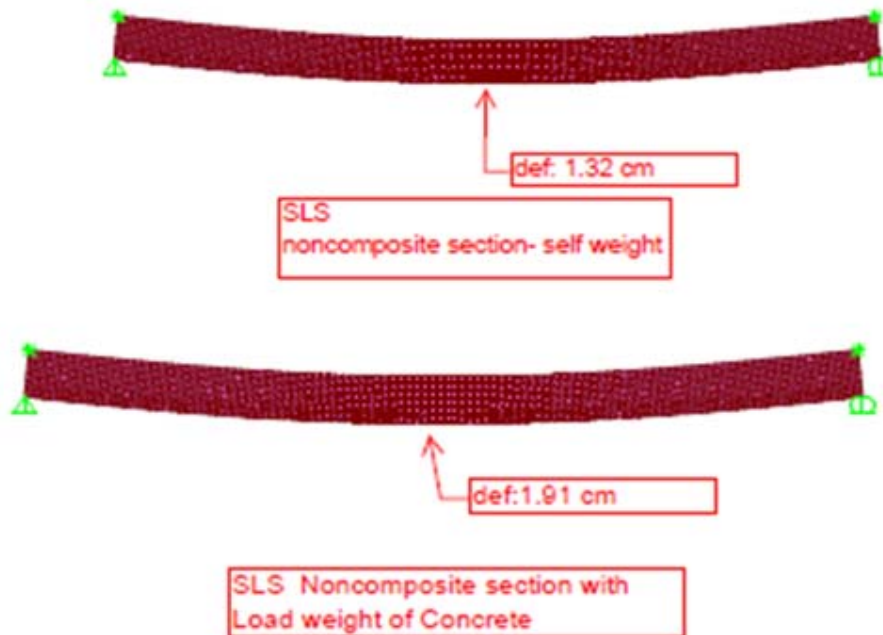


Figure 4.8 : Live Load of non-composite section

4.7.1 1 Deflections of non-composite section

Deflections of non-composite section at the ULS and SLS can be seen below.



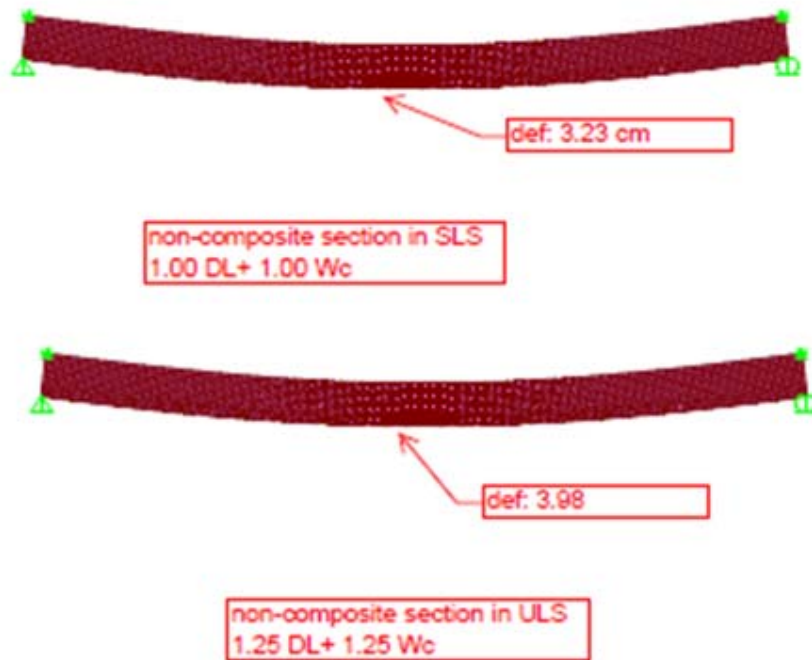


Figure 4.9 : Non-Composite section details

4.7.2 Composite Section and deflection of composite section

To determinate of the elastic critical bending momend and deflections, sap2000 is used with the inverted U-frame model (see figures below).

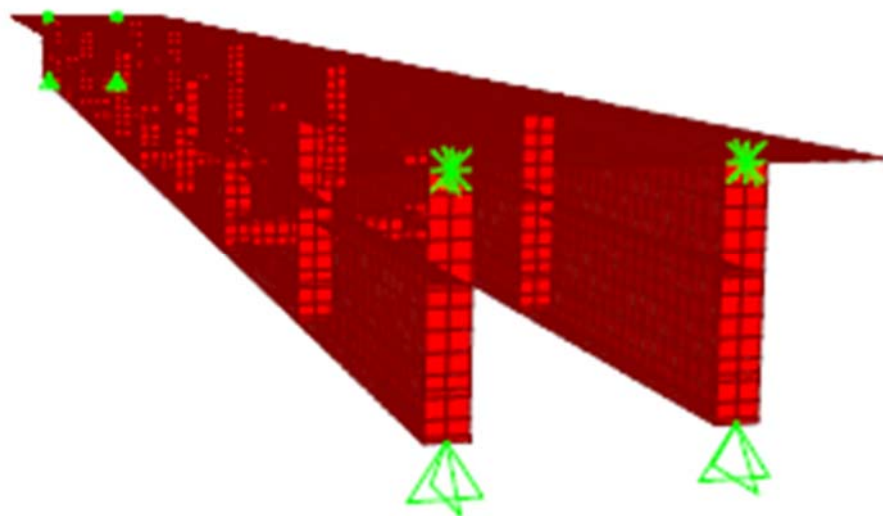


Figure 4.10 : 3-d composite section

For Live Load 25 kN/m is subjected to the calculation model.

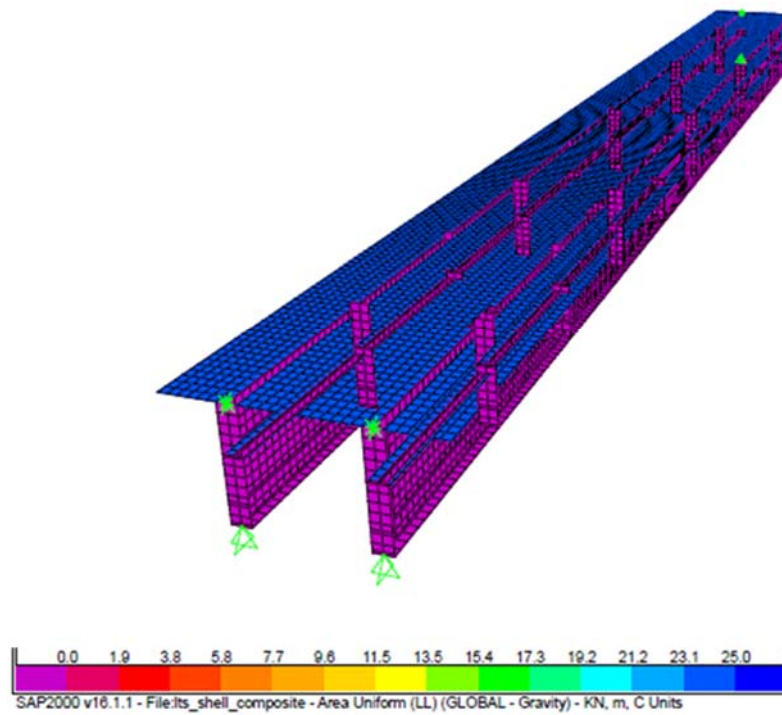
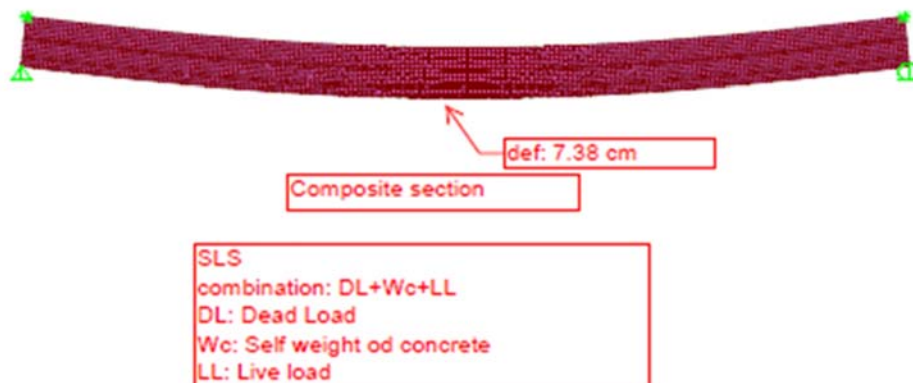
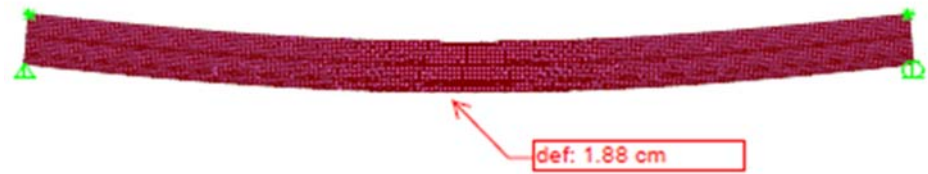


Figure 4.11 : Live Load of composite section

4.7.2.1 Deflections of composite section

All the deflections are obtained in the safety limits.





DL: Dead Load of shell composite section



SLS
LL (Live Load)



ULS of shell composite section
Combination: 1.25 DL+1.25 Wc+ 1.4 LL

4.7.3 Results of Buckling Analysis in Sap2000

To calculate buckling Analysis, sap2000 is used. As a result of mode-1 factor is determined 2.61 in Sap2000.

Combination: 1.00 Dead Load+1.00 Wc+1.00 LL

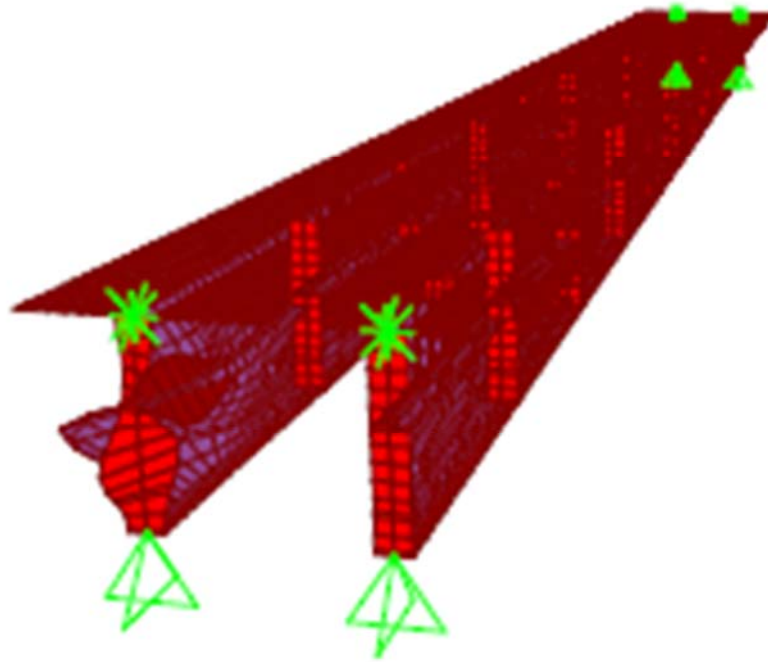


Figure 4.12 : Mode-1 of composite section

Mode-2 factor is determined 3.97 as the results of buckling analysis in Sap2000.
Combination: 1.00 Dead Load+1.00 Wc

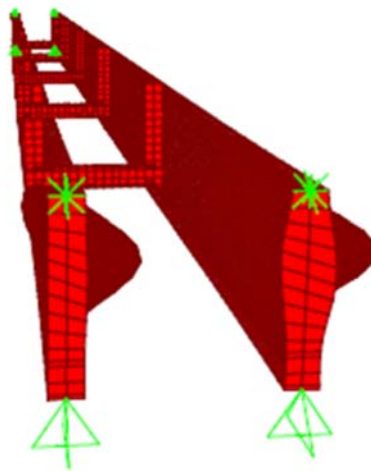


Figure 4.13 : Mode-2 of composite section

5. COST-EFFECTIVENESS ANALYSIS

After assignation the cost and benefits associated with each of the two alternative material types stainless steel and carbon steel, this study of thesis proceeds to analysis the cost-effectiveness on the basis of the costs and benefits. Cost effectiveness is best measured across the entire life cycle. This allow decision makers to answer questions such as which alternative yields the lowest total cost or maximum benefit over the project life and to address issues associated with the level of detail investigated in alternatives, and the impacts of different user costs in the long term.

According to the result of cost comparison of the composite bridge versus a traditional post-tensioned bridge in Denmark, cost estimates of the total construction are listed below. According to results, the bridge ‘ Egebækvej’(RC) with danish construction unit prices is about 3 million euro, the bridge ‘ Egebækvej’ (RC) with turkish construction unit prices is about 1.750 million euro, in addition to that, the bridge with stainless steel with turkish construction unit prices is about 2.4533 million euro and the bridge with carbon steel turkish construction unit prices is about 2.875 million euro as given table 5.1 shows that a bridge design with stainless steel does not need repainting like the material carbon steel. This implies that stainless steel is more cost-effective, over the life cycle, than the traditional material, carbon steel and the economic superiority of stainless steel increases further when user costs are considered. Result of a comparison of the composite bridge versus a traditional post-tensioned bridge in Denmark can be seen in table 5.1.

Cost estimations for bridge with stainless and carbon steel,

Table 5.1 : Cost analysis

Price estimation of bridge with stainless steel					
Member of Structure	Unit	Material	Unit Price (TL)	Year	Total (TL)
Pier	278.1 m ³	Concrete	450	2013	161309.6
Abutment	120.8 m ³	Concrete	450	2013	140110.6

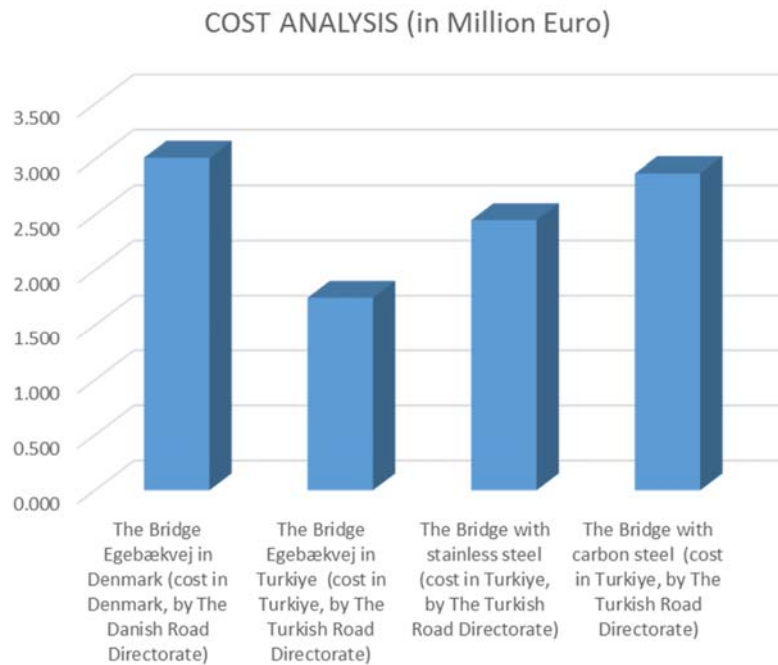
Table 5.1 : (Continued) Cost analysis

Reinforcement	78285.1 kg	Carbon Steel	1.6	2013	125256.17
Beam (with studs)	36200 kg	Stainless steel	25	2013	6335700
stud		Stainless steel	6.00	2013	3480
Slab	369.75 kg	Concrete	450	2013	214455
Elastomeric Bearing		Neopren with steel plates	510	2013	28560
				SUM	7008871
Source: the book " General Highways Directorate After the 2012 "					
Price estimation of bridge with stainless steel					
Member of Structure	Unit	Material	Unit Price (TL)	Year	Total (TL)
Pier	278.1 m ³	Concrete	450	2013	161309.6
Abutment	120.8 m ³	Concrete	450	2013	140110.6
Reinforcement	78285.1 kg	Carbon steel	1.6	2013	125256.17
Beam (with studs)	36200 kg	Carbon steel+painting	12	2013	3040850
Painting	3000 m ²	painting	1500	2013	4500000
stud		Carbon steel	4.25	2013	2465
Slab	369.8 m ³	Concrete	450	2013	214455
Elastomeric Bearing		Neopren with steel plates	510	2013	28560
				SUM	8213006.37
Source: the book " General Highways Directorate After the 2012 "					
Assumed that the bridge service life is 100 years and painted 20 times in service-life					

As a result of these estimations,

Table 5.1 : (Continued) Cost analysis

COST ANALYSIS	in Million TL	in Million Euro
The Bridge (RC) Egebækvej in Denmark (cost in Denmark, by The Danish Road Directorate)	8.6	3.013
The Bridge (RC) Egebækvej in Turkiye (cost in Turkiye, by The Turkish Road Directorate)	5.0	1.750
The Bridge with stainless steel (cost in Turkiye, by The Turkish Road Directorate)	7.0	2.453
The Bridge with carbon steel (cost in Turkiye, by The Turkish Road Directorate)	8.2	2.875

**Figure 5.1 : Cost analysis**

Consistently applied across all scenarios for life cycle costs with 60 year service life, all maintenance and inspection discount rates from UK government guidance costs of stainless steel with neutral / conservative assumptions are lower than carbon steel (see figure 5.2).

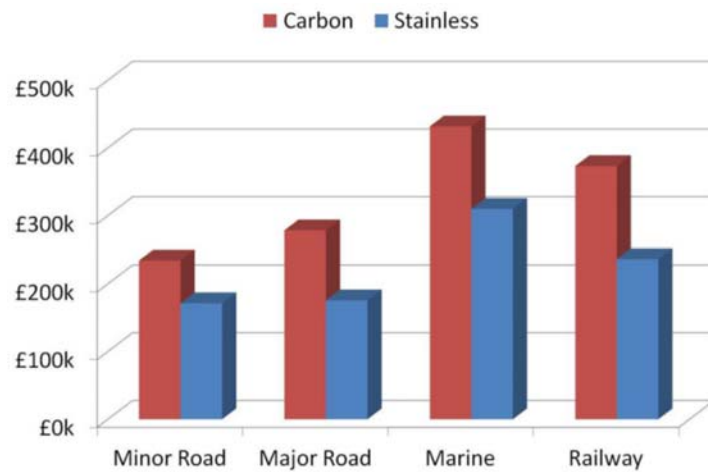


Figure 5.2 : Life Cycle Costs – Discounted

The analysis was further extended for different interest rates. As Figure 5.3 suggests, higher interest rates stainless steel becomes an even more attractive alternative.

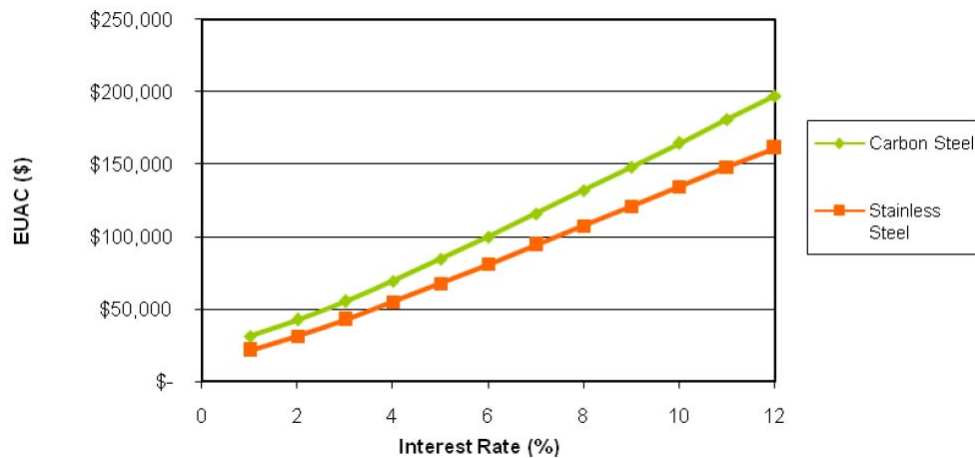


Figure 5.3 : EUAC for Variable Interest Rates

6. CONCLUSIONS AND RECOMMENDATIONS

As society and governments are becoming more conscious of environmental and economic factors, the growth in the use of stainless steel has been the highest of any material in the world. This helps highlight a growing amount of awareness in regards to how to make responsible decisions when it comes to material selection. Ideally, the stainless steel industry, together with the green industry, will continue to educate designers, architects and consumers on the benefits of stainless steel, so as to ensure that this trend continues.

6.1 Differences Stainless Steel and Carbon Steel

- The main difference between the two
 - the components that are added to the steel to make it useful for its intended purposes.
 - ❖ Carbon steel
 - ✓ higher carbon content which gives the steel a lower melting point,
 - ✓ more malleability and durability,
 - ✓ better heat distribution.
 - ❖ Stainless steel
 - ✓ high chromium content that forms an invisible layer on the steel to prevent corrosion and staining.
 - ✓ lower maintenance than carbon steel

6.2 Results

Carbon steel and stainless steel are easy to distinguish.

- ❖ Carbon steel is dull, with a matte finish that is comparable to a cast iron pot or wrought iron fencing.
- ❖ Stainless steel is lustrous and comes in various grades that can increase the chromium in the alloy until the steel finish is as reflective as a mirror.

Economic and environmental benefits which stainless steels offer the civil engineering

Stainless steels offer lightweight, durable design solutions to the the civil engineering. The resulting economic benefits create global market growth opportunities for the stainless steel industry.

Stainless steels offer:

- 1) Excellent corrosion resistance;
- 2) Tolerance of high flow rates;
- 3) Good strength and ductility;
- 4) Ease of fabrication;
- 5) Ready availability in a wide range of forms;
- 6) Very low leaching levels into water;
- 7) Excellent durability; and
- 8) Full recyclability.

Economic benefits

The excellent corrosion resistance of correctly applied stainless steels is the basic characteristic from which all the other advantages flow. There is no general corrosion, and the risk of localised corrosion can be minimised by material selection and good practice. As a result, there is no need for a corrosion allowance or any protective coating. This allows smaller sections with stainless steel components to be used, which may be lighter and simpler to fabricate than equivalent components in traditional materials.

Stainless steel systems do not require either a corrosion prevention system or modification of the water chemistry in order to prevent corrosion, so bringing further operating cost reductions.

The durability of stainless steel means components may still be in good condition and so have value at the end of the useful life of the structure.

Life cycle cost (LCC) analysis

The economic benefits of choosing stainless steel can be revealed by an analysis of life cycle costs (LCC) ' the total costs involved with installing, operating and maintaining over its complete life.

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APPENDICES

APPENDIX A.1

Table A.1 :Combinations of the ultimate limit state for road bridges [13]

Limit State	STR/GEO (Set B) ¹⁾ and (Set C) ²⁾										EQU, UPL, HYD (Set A) ³⁾	Fatigue
Equation	6.10a	Set B: 6.10b, Set C: 6.10									6.10	
Load Combinations		1	2	3	4	5	6	7	8	9	1 – 9	
Permanent Loads												
Self weight of structural members, equipments and additional ^{1) 2) 3)}												
unfavorable, $\gamma_{Gj,sup}$	1,25	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,10	1,00
favorable, $\gamma_{Gj,inf}$	1,00	0,90	0,90	0,90	0,90	0,90	0,90	0,90	0,90	0,90	0,90	1,00
Weight of soil and groundwater ¹⁾²⁾³⁾												
unfavorable, $\gamma_{Gj,sup}$	1,00 (1,25)6)	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,10	1,00
favorable, $\gamma_{Gj,inf}$	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	0,90	1,00
Settlements ⁹⁾	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
Prestress	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
Variable load⁵⁾												
Traffic loads ⁸⁾											⁴⁾	1,10
gr1a	LM1 + loads on footways and cycle track											
	Tandem load, TS	-	1,40	-	1,05	-	-	1,05	1,05	1,05	1,05	
	Uniform distributed load, UDL	-	1,40	-	0,56	-	-	0,56	0,56	0,56	0,56	
	loads on footways and cycle track (reduced value)	-	1,40	-	0,56	-	-	0,56	0,56	0,56	0,56	
gr1b	Single axle load	-	-	1,40	-	-	-	-	-	-	-	
gr2	Braking, acceleration, centrifugal forces	-	-	-	1,40	-	-	-	-	-	-	
gr3	loads on footways and cycle track ⁷⁾	-	-	-	1,40	-	-	-	-	-	-	
gr4	LM4 – single and 'crowd' loading	-	-	-	-	1,40	-	-	-	-	-	
Wind load, F_{wk}	-	0,90	-	0,90	0,90	0,90	1,50	0,90	1,12	0,90	⁴⁾	1,30
Ice load	-	0,90	-	0,90	0,90	0,90	0,90	1,50	0,90	0,90	⁴⁾	-
Wave load	-	0,90	-	0,90	0,90	0,90	1,12	0,90	1,50	0,90	⁴⁾	1,30
Temperature, T_k	-	0,90	-	0,90	0,90	0,90	0,90	0,90	0,90	1,50	⁴⁾	1,00

Table A.2 :Combinations of the accident load cases and seismic load cases [13]

Limit State		Accidental actions		Seismic actions	
Equation		6.11b		6.12b	
Load Combination		1	2	1	2
Permanent Loads					
Self weight of structural members, equipments and additionals					
unfavorable, $\gamma_{Gj,sup}$		1,00	1,00	1,00	1,00
favorable, $\gamma_{Gj,inf}$		1,00	1,00	1,00	1,00
Weight of soil and grounwater					
unfavorable, $\gamma_{Gj,sup}$		1,00	1,00	1,00	1,00
favorable, $\gamma_{Gj,inf}$		1,00	1,00	1,00	1,00
Settlements		1,00	1,00	1,00	1,00
Prestress		1,00	1,00	1,00	1,00
Variable load					
Traffic loads					
gr1a	LM1 + loads on footways and cycle track				
	Tandem load, TS	0,75	-	0,30	-
	Uniform distributed load, UDL	0,40	-	0,30	-
	loads on footways and cycle track (reduced value)	0,40	-	0,30	-
gr1b	Single axle load	-	-	-	-
gr2	Braking, acceleration, centrifugal forces	-	-	-	-
gr3	loads on footways and cycle track	-	-	-	-
gr4	LM4 – single and 'crowd' loading	-	-	-	-
Wind load, F_{wk}		-	0,20	-	0,20
Ice load		-	-	-	-
Wave load		-	0,20	-	0,20
Temperature, T_k		0,50	0,50	0,50	0,50
Accidental actions A_d		1,00	1,00		
Seismic actions A_{Ed}				1,00	1,00

Table A.3 :Characteristic load combinations for road bridges [13]

Limit State		Serviceability limit state Characteristic load combinations								
Equation		6.14b								
Load Combinations		1	2	3	4	5	6	7	8	9
Permanent loads										
Self weight of structural members, equipments and additional										
unfavorable, $\gamma_{Gj,sup}$		1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
favorable, $\gamma_{Gj,inf}$		1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
Weight of soil and groundwater										
unfavorable, $\gamma_{Gj,sup}$		1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
favorable, $\gamma_{Gj,inf}$		1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
Settlements		1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
Prestress		1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
Variable loads										
Traffic loads										
gr1a	LM1 + loads on footways and cycle track									
	Tandem load, TS	1,00	-	0,75	-	-	0,75	0,75	0,75	0,75
	Uniform distributed load, UDL	1,00	-	0,40	-	-	0,40	0,40	0,40	0,40
	loads on footways and cycle track (reduced value)	1,00	-	-	-	-	0,40	0,40	0,40	0,40
gr1b	Single axle load	-	1,00	-	-	-	-	-	-	-
gr2	Braking, acceleration, centrifugal forces	-	-	1,00	-	-	-	-	-	-
gr3	loads on footways and cycle track ²⁾	-	-	-	1,00	-	-	-	-	-
gr4	LM4 – single and ‘crowd’ loading	-	-	-	-	1,00	-	-	-	-
Wind load, F_{wk}		0,60	-	0,60	0,60	0,60	1,00	0,60	0,60	0,60
Ice load		0,60	-	0,60	0,60	0,60	0,60	1,00	0,60	0,60
Wave load		0,60	-	0,60	0,60	0,60	0,60	0,60	1,00	0,60
Temperature, T_k		0,60	-	0,60	0,60	0,60	0,60	0,60	0,60	1,00

Table A.4 :Frequent and quasi-permanent load combinations [13]

Limit State	Serviceability limit state							
	Frequent load combinations							Quasi-permanent
Equation	6.15b							6.16b
Load Combination	1	2	3	4	5	6	7	1
Permanent Loads								
Self weight of structural members, equipments and additional								
unfavorable, $\gamma_{Gj,sup}$	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
favorable, $\gamma_{Gj,inf}$	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
Weight of soil and grounwater								
unfavorable, $\gamma_{Gj,sup}$	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
favorable, $\gamma_{Gj,inf}$	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
Settlements	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
Prestress	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
Variable load								
Traffic loads								
gr1a	LM1 + loads on footways and cycle track							
	Tandem load, TS	0,75	-	-	-	-	-	-
	Uniform distributed load, UDL	0,40	-	-	-	-	-	-
	loads on footways and cycle track (reduced value)	0,40	-	-	-	-	-	-
gr1b	Single axle load	-	0,75	-	-	-	-	-
gr2	Braking, acceleration, centrifugal forces	-	-	-	-	-	-	-
gr3	loads on footways and cycle track	-	-	0,40	-	-	-	-
gr4	LM4 – single and ‘crowd’ loading	-	-	-	-	-	-	-
Wind load, F_{wk}	-	-	-	0,20	-	-	-	-
Ice load	-	-	-	-	0,20	-	-	-
Wave load	-	-	-	-	-	0,20	-	-
Temperature, T_k	0,50	0,50	0,50	0,50	0,50	0,50	0,60	0,50

Table A.5 :Combinations of permanent and transient load conditions [13]

Limit State			STR/GEO (Set B) ¹⁾ and (Set C) ²⁾								EQU, UPL, HYD (Set A) ³⁾	Fatigue	
Equation			6.10a	Set B: 6.10b, Set C: 6.10								6.10	
Load Combination				1	2	3	4	5	6	7	8	1 – 8	
Permanent Loads													
Self weight of structural members, equipments and additional ^{1) 2) 3)}													
unfavorable, $\gamma_{Gj,sup}$			1,25	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,10	1,00
favorable, $\gamma_{Gj,inf}$			1,00	0,90	0,90	0,90	0,90	0,90	0,90	0,90	0,90	0,90	1,00
Weight of soil and grounwater ¹⁾²⁾³⁾													
unfavorable, $\gamma_{Gj,sup}$			1,00 (1,25) ⁶⁾	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,10	1,00
favorable, $\gamma_{Gj,inf}$			1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	0,90	1,00
Settlements ⁸⁾			1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
Prestress			1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
Variable load ⁵⁾													
Traffic loads												⁴⁾	1,10
	Standard-Vehicles	Standard Vehicle A	-	1,40	-	-	1,05	1,05	1,05	1,05	1,05		
		Standard Vehicle B	-	1,05	-	-	1,05	1,05	1,05	1,05	1,05		
		Surface Load	-	0,56	-	-	0,56	0,56	0,56	0,56	0,56		
	Surface Load (Large bridges and spans)	-	-	1,40	-	0,56	0,56	0,56	0,56	0,56			
	Current Vehicle (direct effect)	-	-	-	1,20 (1,15) ⁹⁾	1,05	1,05	1,05	1,05	1,05			
	Horizontal loads (braking, etc.)	-	-	-	-	1,40	-	-	-	-			
Windload, F _{wk}			-	0,90	0,90	0,90	0,90	1,50	0,90	0,90	0,90	⁴⁾	1,30
Ice load			-	0,90	0,90	0,90	0,90	0,90	1,50	0,90	0,90	⁴⁾	-
Wave loads			-	0,90	0,90	0,90	0,90	0,90	0,90	1,50	0,90	⁴⁾	1,30
Temperature, T _k			-	0,90	0,90	0,90	0,90	0,90	0,90	0,90	1,50	⁴⁾	1,00

Table A.6 :Calculation of capacity and classification [13]

Limit State			Serviceability limit state Characteristic combinations							
Equation			6.14b							
Load Combination			1	2	3	4	5	6	7	8
Permanent Loads										
Self-weight of structural members, equipment and additional										
unfavorable, $\gamma_{Gj,sup}$			1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
favorable, $\gamma_{Gj,inf}$			1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
Weight of soil and groundwater ¹⁾²⁾³⁾										
unfavorable, $\gamma_{Gj,sup}$			1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
favorable, $\gamma_{Gj,inf}$			1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
Settlements			1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
Prestress			1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
Variable load										
Traffic loads										
	Standard-Vehicles	Standard Vehicle A	1,00	-	-	1,00	1,00	1,00	1,00	1,00
		Standard Vehicle B	1,00	-	-	0,75	0,75	0,75	0,75	0,75
		Surface Load	0,40	-	-	0,40	0,40	0,40	0,40	0,40
	Surface Load (Large bridges and spans)			1,00	-	0,40	0,40	0,40	0,40	0,40
	Current Vehicle (direct effect)		-	-	1,00	1,00	1,00	1,00	1,00	1,00
	Horizontal loads (Braking, etc.)		-	-	-	1,00	-	-	-	-
Wind load, F_{wk}			0,60	0,60	0,60	0,60	1,00	0,60	0,60	0,60
Ice load			0,60	0,60	0,60	0,60	0,60	1,00	0,60	0,60
Wave loads			0,60	0,60	0,60	0,60	0,60	0,60	1,00	0,60
Temperature, T_k			0,60	0,60	0,60	0,60	0,60	0,60	0,60	1,00

Table A.7 :Frequent and quasi-permanent load combinations [13]

Limit State			Serviceability limit state							
			Frequent combinations							Quasi-permanant
Equation			6.15b							6.16b
Load Combinations			1	2	3	4	5	6	7	
Permanent Loads										
Self-weight of structural members, equipment and additional										
unfavorable, $\gamma_{Gj,sup}$			1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
favorable, $\gamma_{Gj,inf}$			1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
Weight of soil and groundwater ¹⁾²⁾³⁾										
unfavorable, $\gamma_{Gj,sup}$			1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
favorable, $\gamma_{Gj,inf}$			1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
Settlements			1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
Prestress			1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
Variable load										
Traffic loads										
	Standard-Vehicles	Standard Vehicle A	1,00	-	-	-	-	-	-	-
		Standard Vehicle B	0,75	-	-	-	-	-	-	-
		Surface Load	-	-	-	-	-	-	-	-
	Surface Load (Large bridges and spans)		-	0,40	-	-	-	-	-	-
	Current Vehicle (direct effect)		-	-	1,00	-	-	-	-	-
	Horizontal loads (Braking, etc.)		-	-	-	-	-	-	-	-
Wind load, F_{wk}			-	-	-	0,20	-	-	-	-
Ice load			-	-	-	-	0,20	-	-	-
Wave loads			-	-	-	-	-	0,20	-	-
Temperature, T_k			0,50	0,50	0,50	0,50	0,50	0,50	0,60	0,50

Tables of mechanical properties of some reinforcing bar& studs& duplex stainless steels

Table A.8 : Mechanical Properties of Some Reinforcing Bar Alloys [9]

Alloy	Tensile Strength MPa (ksi)	Yield Strength as rolled, MPa (ksi)	Elongation % in 2 in. typical minimum values	Brinell Hardness
Carbon Steel	540 (78)	400 (58)min	25	200
Carbon Steel epoxy coated	540 (78)	400 (58)min	25	200
S30400 stainless steel	750 (108) ~10% cold work	600 (87)	37	200
S30453 stainless steel	730 (106) ~10% cold work	500 (73)	37	200
S31600 stainless steel	810 (117) ~10% cold work	520 (75)	40	220
S31653 stainless steel	690 (100) ¹⁵ ~10% cold work	523 (76)	16	250
Duplex S31803	680 (99) no cold work	480 (70)	25	290

Table A.9 : Physical Properties of Some Reinforcing Bar Alloys [9]

Alloy	Coefficient of Thermal Expansion 20-1000C $\mu\text{m}/\text{mm}/^{\circ}\text{C}$ ($\mu\text{in.}/\text{in.}/^{\circ}\text{F}$)	Modulus of Elasticity X 10^3	Magnetic	Corrosion Resistance General/Pitting	PREN Pitting Resistance Equivalent Number	Approx. Cost Ratio 1996
Carbon Steel	11.7 (6.6)	205MPa (30ksi)	yes	poor/poor	- - -	1.0
Carbon Steel epoxy coated	11.7 (6.6)	205MPa (30ksi)	yes	poor/poor	- - -	1.7 - 2.0
S30400 stainless steel	17.0 (9.5)	200MPa (29ksi)	no	good/fair	18	3.8
S30453 stainless steel	16.8 (9.4)	200MPa (29ksi)	no	good/fair	21	4.3
S31600 stainless steel	16.5 (9.2)	200MPa (29ksi)	no	good/better	25	4.4
S31653 stainless steel	16.5 (9.2)	200MPa (29ksi)	no	good/better	27	4.7
Duplex S31803	13 (7.3)	190MPa (28ksi)	yes	very good/very good	36	4.4

Table A.10 : Carbon and stainless steel reinforcing bar test results after 2 years

Test Program	Alloys					Environment			
	Carbon Steel	S30400 Stainless Steel	S31600 Stainless Steel	Duplex S31803	S24000	Temp. $^{\circ}\text{C}$	Relative Humidity	Time of Test	Test Results
Test #1: Temp. Cycling	0%Cl ⁻ 0.5%Cl ⁻ 2.0%Cl ⁻	0%Cl ⁻ 0.5%Cl ⁻ 2.0%Cl ⁻	0%Cl ⁻ 0.5%Cl ⁻ 2.0%Cl ⁻	0%Cl ⁻ 0.5%Cl ⁻ 2.0%Cl ⁻	0%Cl ⁻ 0.5%Cl ⁻ 2.0%Cl ⁻	Daily cycling between 25 $^{\circ}\text{C}$ and 65 $^{\circ}\text{C}$	80% RH	Since August 1996	Carbon steel samples in 2% Cl ⁻ cracked; minor corrosion on carbon steel in 0.5%Cl ⁻ ; no corrosion on stainless steel.
Test #2: Constant Temp.	0%Cl ⁻ 0.5%Cl ⁻ 2.0%Cl ⁻	0%Cl ⁻ 0.5%Cl ⁻ 2.0%Cl ⁻	0%Cl ⁻ 0.5%Cl ⁻ 2.0%Cl ⁻	0%Cl ⁻ 0.5%Cl ⁻ 2.0%Cl ⁻	0%Cl ⁻ 0.5%Cl ⁻ 2.0%Cl ⁻	Constant Temp. at 35 $^{\circ}\text{C}$	95% RH	Since February 1997	Some samples opened in August 1998. No corrosion.
Test #3: Immersion	0%Cl ⁻	0%Cl ⁻	0%Cl ⁻	0%Cl ⁻	0%Cl ⁻	Ambient	Immersed in 3.4% NaCl	Since February 1997	Electrochemical test no corrosion at this stage.
Test #4: 3.4% NaCl Ponding ASTM G-109	0%Cl ⁻	0%Cl ⁻	0%Cl ⁻	0%Cl ⁻	0%Cl ⁻	Ambient	3.4% NaCl solution - ponding	Since February 1997	Electrochemical test no corrosion at this early stage.

As a table with 80% relative humidity; temperature cycled between 25 and 650c; examination after two years; no effect from thermal expansion can be seen Table A.11.

Table A.11 : Concrete sample condition with carbon and stainless steel [9]

	Concrete Sample Condition	Rebar Condition
Carbon Steel		
0.5% Chloride	no cracking observed	no significant corrosion observed
2% Chloride	severe cracks, all samples broken after 4 months	severe corrosion on entire rebar
304 Stainless Steel		
0.5% Chloride	no cracking observed	no corrosion observed on rebar
2% Chloride	no cracking observed	no corrosion observed on rebar
316 Stainless Steel		
0.5% Chloride	no cracking observed	no corrosion observed on rebar
2% Chloride	no cracking observed	no corrosion observed on rebar

Table A.12 : Mechanical properties of stainless steel-studs

Minimum Values	Mechanical Properties - Metric				
	Mild Steel Shear and Concrete Anchors	Standard Mild Steel Studs	Mild Steel Deformed Bar Anchors	Stainless Steel Studs, as formed	Stainless Steel Studs, as formed, post-annealed
Yield, 0.2% offset (MPa), R_e	350	340	485	345	206
Ultimate Tensile (MPa), R_m	450	420	552	517	483
% Elongation, A_5 in 5 x diameter	20	17	N/A	40	40
% Area Reduction	N/A	N/A	N/A	50	50

Table A.13 : General filler recommendation for Duplex stainless steels [11]

Outokumpu	EN	ASTM / UNS	Welding consumables	
			Covered electrodes ISO 3581 / ISO 14172	Wires ISO 14343 / ISO 18274
LDX 2101®	1.4162	S32101	23 7 NL or 22 9 3 NL	23 7 NL or 22 9 3 NL
2304	1.4362	S32304	23 7 NL or 22 9 3 NL	23 7 NL or 22 9 3 NL
LDX 2404™	1.4662	S82441	22 9 3 NL	22 9 3 NL
2205	1.4462	S2205/S31803	22 9 3 NL	22 9 3 NL
2507	1.4410	S32750	25 9 4 NL	25 9 4 NL

Table A.14 : Chemical composition, % (Typical values) [11]

Outokumpu	EN	ASME/UNS	C	Cr	Ni	Mo	N	Others	PRE*
4307	1.4307	304L	0.02	18.1	8.1	-	-	-	18
4404	1.4404	316L	0.02	17.2	10.1	2.1	-	-	24
LDX 2101®	1.4162	S32101	0.03	21.5	1.5	0.3	0.22	5Mn	26
2304	1.4362	S32304	0.02	23	4.8	0.3	0.10	-	26
LDX 2404™	1.4662	S82441	0.02	24	3.6	1.6	0.27	3Mn	33
2205	1.4462	S32205**	0.02	22	5.7	3.1	0.17	-	35
2507	1.4410	S32750	0.02	25	7	4	0.27	-	43

Table A.15 : General filler recommendation for Duplex stainless steels [11]

Outokumpu	EN	ASTM / UNS	Welding consumables	
			Covered electrodes ISO 3581 / ISO 14172	Wires ISO 14343 / ISO 18274
LDX 2101®	1.4162	S32101	23 7 NL or 22 9 3 NL	23 7 NL or 22 9 3 NL
2304	1.4362	S32304	23 7 NL or 22 9 3 NL	23 7 NL or 22 9 3 NL
LDX 2404™	1.4662	S82441	22 9 3 NL	22 9 3 NL
2205	1.4462	S2205 / S31803	22 9 3 NL	22 9 3 NL
2507	1.4410	S32750	25 9 4 NL	25 9 4 NL

Table A.16 : Mechanical properties (At room temperature) [11]

Outokumpu	Min values acc. to EN 10028-7:2007									Min values according to ASTM A240-11		
	R _{0.2} , MPa			R _m , MPa			A ₅₀ , %			R _{0.2} , MPa	R _m , MPa	A ₅₀ , %
	P	H	C	P	H	C	P	H	C			
4307	200	200	220	500	520	520	45	45	45	170	485	40
4404	220	220	240	520	530	530	45	40	40	170	485	40
LDX 2101®	450*	480*	530*	650*	680*	700*	30*	30*	20*	530 (t ≤ 5.0 mm) / 450 (t > 5.0 mm)	700 (t ≤ 5.0 mm) / 650 (t > 5.0 mm)	30
2304	385/400	385/400	405/420	630	600	600	25	20	20	400	600	25
LDX 2404™										540 (t < 10 mm) / 480 (t ≥ 10 mm)	740 (t < 10 mm) / 680 (t ≥ 10 mm)	25
2205	445/460	445/460	485/500	640	700	700	25	25	20	450	655	25
2507	515/530	515/530	535/550	730	750	750	20	20	20	550	795	15

* Min values according to EN 10088-4:2009

P = Hot rolled plate H = Hot rolled strip C = Cold rolled coil and strip

For hot rolled plate (P) with T = 4.76 – 75 mm the impact toughness in transverse direction for LDX 2101® = 60 Joule at 20°C and 27 Joule at -40°C (values from internal standard AM 611E). For hot rolled strip (H) and cold rolled coil and strip (C) with T ≤ 10 mm the impact toughness in transverse direction for LDX 2101® = 80 Joule at 20°C and 50 Joule at -40°C (values from internal standard AM 611E).

Table A.17 : Physical properties [11]

Outokumpu	Density, g/cm ³	Modulus of elasticity, GPa	Poisson's ratio $\nu = -\epsilon_{trans} / \epsilon_{longitudinal}$	Average linear expansion at RT → 100°C x10 ⁻⁴ / °C
4307	7.9	200	0.3	16.0
4404	8.0	200	0.3	16.0
LDX 2101®	7.8	200	0.3	13.0
2304	7.8	200	0.3	13.0
LDX 2404™	7.7	205	0.3*	13.0
2205	7.8	200	0.3	13.0
2507	7.8	200	0.3	13.0

SYMBOLS

A_a	: Cross-sectional area of the structural steel section
A_b	: Cross-sectional area of concrete
A_c	: Cross-sectional area of the compression zone of a section
$A_{c,eff}$	: Effective cross-sectional area of the compression zone of a section
A_s	: Cross-sectional area of reinforcement
A_v	: Structural steel shear area
C_d	: Rigidity of bracing transverse frame
E_{cm}	: Secant modulus of elasticity of concrete
E_a	: Modulus of elasticity of structural steel
E_s	: Modulus of elasticity of reinforcement steel
F	: Applied force
F_{wk}	: Characteristic value of resultant wind force
G_k	: Characteristic (nominal) value of the effect of permanent actions
I	: Second moment of area
L_e	: Equivalent span
L	: Span; length
M_{Ed}	: Design bending moment
$M_{a,Ed}$	: Design bending moment applied to the structural steel section
$M_{c,Ed}$	: Design bending moment acting on the composite section
$M_{el,Rd}$	: Design value of the elastic resistance moment of the composite section
$M_{f,Rd}$	: Design value of the plastic resistance moment of a cross-section consisting of the flanges only
$M_{pl,Rd}$	: Design value of the plastic resistance moment
N_{Ed}	: Design axial force
P_{Rk}	: Characteristic value of the shear resistance of a single connector
V_{Ed}	: Design shear force
V_{Rd}	: Design resistance for shear
$V_{pl,Rd}$	: Plastic design shear resistance
$V_{pl,a,Rd}$	: Plastic design shear resistance applied to the structural steel section

Small Latin letters

A	: Length of a web plate between adjacent vertical stiffeners
B	: Width of a structural steel element
b_{eff}	: Effective width (concrete slab; steel bottom flange of a box girder)
b_0	: Centre-to-centre distance between outside rows of shear connectors
f_{cd}	: Design value of concrete compressive strength
f_{ck}	: Characteristic compressive cylinder strength of concrete at 28 days
f_{cm}	: Mean value of concrete cylinder compressive strength
f_{ctm}	: Mean value of axial tensile strength of concrete
$f_{ctk,0,05}$	: 5% fractile of the characteristic axial tensile strength of concrete
$f_{ctk,0,95}$	: 95% fractile of the characteristic axial tensile strength of concrete
f_{sk}	: Characteristic yield strength of reinforcement
f_y	: Yield strength of the structural steel
f_u	: Ultimate strength of the structural steel
f_{yk}	: Characteristic value of the yield strength of the structural steel
h	: Height; overall depth
h_0	: Notional size of the concrete slab
σ_k	: Plate buckling coefficient for normal stresses

- τ_k : Plate buckling coefficient for shear stresses
 n_0 : Structural steel / concrete modular ratio for short-term loading
 n_{ba} : Structural steel / reinforcement modular ratio
 n_L : Structural steel / concrete modular ratio for long-term loading

Capital Greek letters

- $\Delta\sigma_c$: Reference value of the fatigue strength at $NC = 2.106$ cycles (direct stresses)
 $\Delta\sigma_p$: Stress range from load p (fatigue in structural steel)
 $\Delta\sigma_{E,2}$: Equivalent constant amplitude stress range related to 2 million cycles (direct stress)
 $\Delta\sigma_{s,p}$: Stress range from load p (fatigue in reinforcing bars)
 $\Delta\tau_c$: Reference value of the fatigue strength at $NC = 2.106$ cycles (shear stresses)
 $\Delta\tau_{E,2}$: Equivalent constant amplitude stress range related to 2 million cycles (shear stress)
 Φ : Damage equivalent impact factor (structural steel)
 Φ_s : Diameter of a steel reinforcing bar

Small Greek letters

- α : Factor; angle; compressed height percentage
 α_{cc} : Factor on the concrete compressive strength
 χ : Reduction factor (≤ 1) for instability (lower indices: c, p, w, LT, op)
 ε : Strain; factor
 ε_r : Shrinkage strain
 ε_{cd} : Drying shrinkage strain
 ε_{th} : Thermal shrinkage strain
 ϕ : Creep function
 ϕ_{fat} : Damage equivalent impact factor (reinforcing steel)
 $\gamma_{F,f}$ (or $\gamma_{F,fat}$) : Partial factor for equivalent constant amplitude range $\Delta\sigma_E, E \Delta\tau$
 γ_M : Partial factor for resistance of structural steel
 $\gamma_{M,f}$: Partial factor for resistance of structural steel for fatigue loading
 $\gamma_{Mf,s}$: Partial factor for resistance of a stud connector for fatigue loading
 $\gamma_{M,ser}$: Partial factor for resistance of structural steel at Serviceability Limit State (SLS)
 γ_S : Partial factor for resistance of reinforcing steel
 $\gamma_{S,fat}$: Partial factor for resistance of reinforcing steel for fatigue loading
 γ_V : Partial factor for resistance of a stud connector
 η : Coefficient on the yield strength of structural steel
 λ : Reduced slenderness (possible lower indices: c, p, w, LT, op)
 λ : Damage equivalent factor (structural steel)
 λ_s : Damage equivalent factor (reinforcing steel)
 λ_V : Damage equivalent factor (shear connectors)
 μ : Moment of area
 ν : Poisson's ratio
 ρ : Reduction factor ($\leq 1,0$) for effective area of a structural steel cross-section
 ρ_c : Reduction factor for effective width
 P_s : Reinforcement ratio in a concrete cross-section
 σ_{cr} : Elastic critical plate buckling stress
 σ_E : Elastic critical Euler's stress
 σ_{Ed} : Design value of a direct stress in a cross-section
 τ_{cr} : Elastic critical shear buckling stress

τ_{Ed}	: Design value of a shear stress in a cross-section
Ψ	: Stress ratio between opposite edges of a structural steel plate
Ψ_L	: Creep multiplier for modular ratio
Ψ_0	: Factor for the combination value of a variable action
Ψ_1	: Factor for the frequent value of a variable action
Ψ_2	: Factor for the quasi-permanent value of a variable action

CURRICULUM VITAE

Born: 1986

Education

- B.Sc., Civil Engineering, University of Kocaeli, 2009.

Experience

- 2013 - to date: Grontmij A/S, Denmark&Turkey.
- 2012-2013: Ypu, Turkey
- 2010-2012: Epro Co., Turkey
- 2008: Cowi A/S, Denmark

Introduction

- Tijen has broad experience in design of bridge projects. She has also involved in port projects.

Areas of expertise

- Knowledge and experience within many aspects of civil engineering including steel and concrete structures, geotechnics, design of piles, sheet piles, bridges, viaducts, box culverts, cut and cover tunnel, underpass, flyover, retaining walls etc.
- Structural analysis of port related constructions.
- Earthquake-resistant design of composite, steel/ concrete structures.
- CAD, including 3D modelling