CODE COMPARISON OF TS 648, EUROCODE 3 : PART 1.1 AND LOAD AND RESISTANCE FACTOR DESIGN - AISC FOR STEEL CONNECTIONS

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ABSTRACT

CODE COMPARISON OF TS 648, EUROCODE 3: PART 1.1 AND LOAD AND RESISTANCE FACTOR DESIGN-AISC FOR STEEL CONNECTIONS

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In this thesis, three building codes for steel structures, namely TS 648, AISC-LRFD and EUROCODE 3: PART 1.1 are compared with regard to structural steel connections.

First, the philosophies of the three different codes are discussed and then the major differences for connection designs are stated. The comparison is based on the live load to dead load ratio, which is the

ÖZ

TS 648, EUROCODE 3: BÖLÜM 1.1 VE YÜK VE DAYANIM KATSAYILARI TASARIMI-AISC ŞARTNAMELERİNİN ÇELİK YAPI BAĞLANTILARI İÇİN KARŞILAŞTIRILMASI

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Bu tezde, TS 648, AISC-LRFD ve EUROCODE 3: Bölüm 1.1 şartnameleri çelik yapılarda bağlantılar açısından karşılaştırılmıştır.

Öncelikle üç şartnamenin bağlantı tasarım kriterleri tartışılmış, daha sonrada en önemli farklılıkları belirtilmiştir. Karşılaştırmada esas faktör hareketli yük- zati yük oranı olarak alınmıştır. Dört farklı bağlantı tipi araştırılmıştır. Sonuç olarak, taşıma gücüne göre tasarım

To My Family

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CHAPTER 1

INTRODUCTION

For the design of steel structures, the Allowable Stress Design (ASD) method has been used for all types of steel structural members in Turkey, in the United States, in Europe, and many other countries around the world. Codes like TS648[1], AISC-ASD, DIN 18800, etc. use Allowable Stress Design. In ASD approach, the load effects such as bending moments, axial forces and shear forces and environmental effects such as earthquake forces and thermal effects on structural members are computed according to accepted methods of structural analysis for the working loads specified by the appropriate codes and the elastic stress caused by these effects on the structural members is compared with the elastic capacities of these members. Elastic capacities that are divided by a factor of safety are called the allowable stresses. In allowable stress design approach, the allowable stresses are increased for short term loading combinations such as the earthquake force. However, only a single factor of safety is applied to all load combinations, and this leads to a non-uniform safety margin.

The load and resistance factor design is a probabilistic approach for the design of steel structures. It involves explicit consideration of limit state, multiple load factors, and implicit probabilistic determination of reliability. The designation LRFD-AISC[2] and Eurocodes reflects the concept of factoring both the loads and material resistances. Factoring both loads and resistances differentiate this approach from the Part 1 of the allowable stress design 1978 AISC Specification and the TS 648, where only resistances are divided by a common load factor. In LRFD approach, the load and resistance factors exist to account for the uncertainties and variabilities inherent in loads, analysis, limit-state model, material properties, geometry, fabrication, erection and so on. Different than the allowable stress method, which defines only one uniform factor of safety for a limit state, LRFD employs multiple load factors which provide a refinement in the design that can account for the different degrees of uncertainties and variability of the design parameters, and this offers the designer a greater flexibility, more rationality, consistency and possible overall economy, especially for varying load conditions.

Eurocode 3 Part 1.1[3] applies to the design of buildings and civil engineering works in steel. The design rules are only concerned with the requirements for resistance, serviceability and durability of such structures.

Eurocodes are the European standards for structural design. Why are Eurocodes necessary?[7] The establishment of international standards for structural design is not a new idea. The process of drawing up of the European design standards has been given a fresh impetus with the drive towards the political and economic unification of the EC. There are several advantages to be gained from having design standards which are accepted by all member states. The first and foremost reason is that the provisions of Eurocodes and the associated European standards for construction products will help lower trade barriers between the member states. This will allow contractors and consultants from all member states to compete fairly for work within Europe. This may lead to a pooling of resources and the sharing of expertise, thereby lowering production costs. It is further believed that such standards will boost the international standing of European engineers which should help in increasing their chances of winning contracts abroad. A further benefit is that these codes make it easier for engineers to practise within all EC countries.

The main objective of this search is to examine the specification Eurocode 3 Part 1.1 and AISC-LRFD code and compare them with the code in use in Turkey, TS648. The comparison is made for the design of connections.

CHAPTER 2

INTRODUCTION TO TS 648

TS648 is the specification for steel structures used in Turkey. This code dictates the use of allowable stress method in the design of members and connections. The allowable stress method (ASD) was the first design method that was developed for the design of steel structures. The criterion for acceptable design strength is that the calculated maximum stress, assuming elastic behavior up to the anticipated maximum loads, is kept lower than a specified allowable stress. The allowable stress is intended to be less than the calculated stress at buckling or yielding, by a factor of safety. Typical values of this factor are 1.65 to 2.50[17]. By keeping the actual stress levels under a fraction of some specified stress, such as yield stress, the allowable stress method typically provides conservative results in some cases and liberal results in some others.

2.1 Introductory Comments on TS648

Ordinary structural steel and high strength steels are all included in this code. Ordinary structural steel is designated by Fe37. The physical and chemical properties of ordinary structural steel are provided in TS908, TS909, TS910, TS911, TS912, TS913 and TS 2162[17].

TS648[1] covers all structures with load carrying components having a minimum thickness of 4 mm and their connections. Temporary and re-locatable structures are also included. Special purpose structures such as highways and railroad bridges, cranes and transmission towers are not covered by TS648.

TS648 permits the designer to select his/or her method of analysis provided that results are not contrary to the code regulations and that the material is assumed to be linearly elastic. In case the designer uses uncommon formulas, reference must be given. Otherwise it becomes necessary to explain the formulas in sufficient detail.

2.2 Design Loads

TS648 refers to TS498 for the design loads. According to TS648, the loads that a structure is to resist are divided into two groups:

- Main Loads (EY): They include dead load, live load (snow load included but not together with the wind load), inertial forces of machines.
- Superimposed Loads (EIY): Wind load, earthquake load, break force, horizontal force component (i.e., cranes), loads of cranes that are seldomly used in mounting and repairing jobs (the dead

load and repetitive load of such cranes belong to the first group), operational and atmospheric effects.

There are two types of loading cases to be considered for stress analysis and stress checks:

- Loading case EY: This case consists of all main loads acting on the structure.
- Loading case EIY: This case consists of all main loads and superimposed loads acting on the structure.

The allowable stresses for these two cases are generally different. If, apart from its own dead weight, a structure is subjected to superimposed loads only, the largest one of these superimposed loads is treated as a main load. The loading case which results in the largest cross section shall be used for design. In general, stresses and support reactions shall be determined separately for each loading case. The sections are selected for the governing case. After selection is completed, maximum stresses are determined and they are compared with the allowable stresses. Allowable stresses for EIY are 15% higher than the EY case.

2.3 Basis of Design

For an acceptable design, it is necessary to go through the following checks:

- Stress checks,
- Stability checks,

- Deflection checks.

These checks shall be made during the construction, shipping, assembly, and operation stages. Stress analysis is made for each one of the EY and EIY loading cases separately.

In this thesis work, the primary concern is the connection design. So the text will go on with this subject, hereafter.

2.4 Connections

The purpose of the connectors is to transmit the stresses from a structural member to another safely, and thereby make them to act as a unit. Bolts and the welds are the common type of connectors used in today's design media. In the selection of the specific type of connector, the designer should consider the following:

- Connection strength required,
- Space limitations of the connection,
- Available work force to fabricate and erect the structure,
- Service conditions and
- Total cost of the installation.

2.4.1 Bolted Connections

Bolts are short pieces of round steel bars with generally a hexagonal head at one end and a threaded portion at the other. Nuts are used to secure the bolts in place. For a better distribution of pressure and keeping the threaded part outside the grip, washers are used between the nut and the plate.

2.4.1.1 Types of Bolts

There are several type of bolts which can be used for connecting structural steel members. These include unfinished bolts, turned bolts, ribbed bolts, interference bolts and high-strength bolts.

High-strength bolts are the most popular types used in today's steel construction area. These bolts made from 10K and 8G steel. In the US, they are designated by ASTM as A325 and A490 bolts. 10K high-strength bolts have an ultimate tensile strength of 10000 kg/cm^2 and a yield strength of 9000 kg/cm^2. These bolts are heavy hexagon-head bolts used with heavy semi-finished hexagon nuts. They may be tightened to develop high tensile stress in them which results in a predictable clamping force on the connection. This permits the loads to be transferred by friction. For the design of this type of bolt, TS648 refers to DIN 1050.

The bolts can be classified on the basis of the mode of the load transmission. They may be subjected to shear or tension. If the load is transmitted through the bearing between the plate and the shank, producing shear in the connector, connector is said to be in shear. If the load is transmitted through the bearing between the plate and the head of the fastener, the fastener will undergo tension.

In bolted type of structures, two type of joint can be mentioned. When the load is transmitted by shear in only one section in a bolt, the connection is called a "Lap Joint". The bolt is in single shear in this type of joint. If the bolts connect three members, then the joint is called a "Butt Joint" and the bolts are in double shear.

2.4.1.2 Modes of Failure

The following modes of failure are observed in the simple (lap or butt) type of bolted connections:

- Tension failure of the connected member,
- Shear failure of the bolt,
- Bearing failure of the bolt,
- Bearing failure of the connected members,
- Shear-out failure of the connected members,
- Tension failure of the bolts.

Normally, minimum edge distance limitation prevents shear-out failure, that is also called Block-Shear Failure, so the shear-out stresses need not to be checked. Tension failure of the connected members is prevented by using net cross-sectional area in the design of the member. Therefore, only shear, bearing and tensile stresses should be checked.

2.4.1.3 Spacing Limitations

Different spacing limitations are given for various type of fasteners. In general there are two types of fasteners: load carrying fasteners and stitch fasteners. The notations used in the Turkish Code will be explained next.

The fastener hole diameter and the minimum plate thickness are denoted by d_1 and t_{min}, respectively. "e" is the center-to-center spacing of fasteners in a direction parallel to the applies load. "g" is the center-to center spacing of fasteners in a transverse direction to the appliedload and usually taken as in between $3d_1$ and $3.5d_1$ in Turkish practice. "e1" is the edge distance in a direction parallel to the applied load and "e2" is the edge distance in a transverse direction to the applied load.

The specified distances are presented as Table 14 in TS648. For load carrying and stitch fasteners (do not carry load), the minimum and maximum spacings are tabulated as follows;

Table 2.1 Minimum and Maximum Spacings for Load Carrying Fasteners

Spacing	Minimum	Maximum
e	$3d_1$	$8d_1$ or $15t_{min}$
e_1	$2d_1$	$3d_1$ or $6t_{min}$
e_2	$1.5d_1$	$3d_1$ or $6t_{min}$

Table 2.2 Maximum Spacings for Stitch Fasteners

Spacing	Compression Members	Tension Members
e	$8d_1$ or $15t_{min}$	$12d_1$ or $25t_{min}$
e_1	$3d_1$ or $6t_{min}$	$3d_1$ or $6t_{min}$
e_2	$3d_1$ or $6t_{min}$	$3d_1$ or $6t_{min}$

As a reminder, the bolt hole diameter is taken 1mm larger than the diameter of the bolt.

2.4.1.4 Design Considerations and Allowable Stresses

The factors determining the strength of a fastener are its diameter and the thickness and the arrangement of the pieces being connected.

For lap joints, the allowable shear load per bolt is given by,

$$N_{em} = (\pi / 4)\, d^2\, \tau_{em} \qquad (2.1)$$

and the allowable bearing load per bolt is given by,

$$N_{ez} = d\, t_{min}\, \sigma_{ez} \qquad (2.2)$$

For butt joints, the allowable shear load per bolt is given by,

$$N_{em} = 2\, (\pi / 4)\, d^2\, \tau_{em} \qquad (2.3)$$

and the allowable bearing load per bolt is given by,

$$N_{ez} = d\, t_{min}\, \sigma_{ez} \qquad (2.4)$$

where, d = diameter of the bolt,

t_{min} = minimum plate thickness,

τ_{em} = allowable shearing stress,

σ_{ez} = allowable bearing stress.

Allowable Stresses:

The nominal stresses in the fasteners are defined as the total load coming to the fastener divided by the area involved in a particular type of failure.

The allowable stresses for rivets as specified by TS648 are given in the Table 2.3. However, nowadays, rivets are no longer as popular as bolts.

Table 2.3 Allowable Stresses for Rivets According to TS648 (kgf/cm^2)

	Rivet Steel St 34		Rivet Steel St 44	
Loading	EY	EIY	EY	EIY
Shear, τ_{em}	1400	1600	2100	2400
Bearing, σ_{ez}	2800	3200	4200	4800
Tension, $\sigma_{çem}$	480	540	720	810

Allowable stresses that are given by TS 648 for unfinished (black) bolts, turned bolts and anchorage bolts subjected to different loading conditions are given as follows:

Table 2.4 Allowable Stresses for Bolts According to TS648 (kgf/cm^2)

	St 38 or 4D Unfinished Bolts		St 38 or 4D Turned Bolts		St 52 or 5D Turned Bolts		Anchorage Bolts	
Loading	EY	EIY	EY	EIY	EY	EIY	EY	EIY
Shear, τ_{em}	1120	1260	1400	1600	2100	2400	-	-
Bearing, σ_{ez}	2400	2700	2800	3200	4200	4800	-	-
Tension, $\sigma_{\text{çem}}$	1120	1120	1120	1120	1500	1500	1120	1120
			For St 37		For St 52			

The allowable stresses for high-strength bolts are not included in this table. The allowable stresses either refer to American or German specifications.

In a high-strength bolted connection, loads are carried by either friction or shear or bearing. Therefore in the specifications these type of bolts are classified as friction type or bearing type bolts. For the friction type bolts, there is a high factor of safety against slippage, and for bearing type this factor of safety is reduced. Thus, the allowable shearing stresses for bearing type are equal to or larger than those for friction type showing that bearing type gives better economy due to requirement of less number of bolts in a connection.

The allowable and classification according to German specification for high strength bolts are given in Tables 2.5, 2.6 and 2.7.

Table 2.5 Allowable Shear Stresses, τ_{em} for SL and SLP Bolts (kgf/cm^2)

	SL		SLP	
Loading	EY	EIY	EY	EIY
Shear, τ_{em}	2400	2700	2800	3200

Table 2.6 Allowable Bearing Stresses, σ_{ez} for SL and SLP Bolts (kgf/cm^2)

		St 37		St 52	
Type	Preload	EY	EIY	EY	EIY
SL	0	2800	3200	4200	4700
	≥ 0.5 Py	3800	4300	5700	6400
SLP	0	3200	3600	4800	5400
	≥ 0.5 Py	4200	4700	6300	7100

Table 2.7 Allowable Bearing Stresses, σ_{ez} for GV and GVP Bolts (kgf/cm^2)

σ_{ez}	Connected Material			
	St 37		St 52	
	EY	EIY	EY	EIY
	4800	5400	7200	8100

SL and SLP type high-strength bolts are designed for shear and bearing. Shank and threaded parts for these bolts are specified in DIN 7968. If the difference between the hole diameter and shank diameter is less than or equal to 1.0 mm, this bolt is defined as SL type. If the difference between the hole diameter and shank diameter is less than or equal to 0.3 mm, this bolt is defined as SLP type.

GV and GVP type high-strength bolts are used for the friction type of connections. If the difference between the hole diameter and shank diameter is less than or equal to 1.0 mm, this bolt is defined as GV type. If the difference between the hole diameter and shank diameter is less than or equal to 0.3 mm, this bolt is defined as GVP type. GV bolts carry loads only by friction. GVP bolts carry loads not only by friction but also by shear and bearing.

2.4.2 Welded Connections

Welding is a process of joining metal parts by means of heat and pressure which causes fusion of the parts (resistance welding), or by heating the metal to the fusion temperature, with or without the addition of weld metal (fusion welding). Fusion welding usually employs either an electric arc or an oxyacetylene flame to heat the metal to the fusion temperature. The electric arc is used for most structural welding.

Welds are classified according to their types, their position and types of joints.

2.4.2.1 Classification of Welds

Types of Welds :

A groove or butt weld is made in the opening, that is called a groove, between two parts being joined. To use groove welds in every situation would mean that the members would have to fit almost perfectly which is most generally not the case. For this reason, groove welds constitute roughly 15 percent of the total weld usage. Fillet weld, which is triangular in shape, joins surfaces which are at an angle with one another. Fillet welds are more economical and fabricated more easily. They generally require less precision in the “fitting up” since the plates do not need special preparation. A plug weld is made by depositing weld metal in a circular hole in one of two lapped pieces. The hole must be filled completely. A slot weld is similar, the only difference being that the hole is elongated. Holes and slots can also be fillet-welded around the circumference, but these are not plug or slot welds. The two latter ones are occasionally used in lap joints when the desired length of fillet welds can not be obtained. They are also useful in preventing overlapping parts from buckling.

Position of Weld :

According to the positions, welds can be classified as being flat, horizontal, vertical and overhead. The flat welds are the most economical welds. Welding in the flat position is executed from above, the weld face being approximately horizontal. The horizontal position is similar, but the weld is harder to make. Work in the overhead position is from the underside of the joint; this is the most difficult weld to make. The longitudinal axis of the weld is vertical in vertical-position welding.

Type of Joint :

Welded joints are classified as butt, lap, tee, corner and edge. The butt joint is groove-welded, while the lap joint is fillet-welded. The tee joint can be groove-welded or it can be fillet-welded with one fillet on each side. Edge joints are generally not used for structural purposes. They are for keeping two or more plates in a given plane or maintaining initial alignment.

Many types of welding processes are used in arc welding. They are *manual* or *automatic*[17]. Other terminology, such as shielded-metal-arc welding (SMAW) or submerged-arc welding (SAW), is also used.

Most manual welding (stick welding) is performed with the shielded-metal arc process. In this process the electrode is placed in an electrode holder to establish electrical contact and positioned by the welder. Shielding is obtained by the use of electrodes heavily coated with a material of such composition that large quantities of gas are produced in the heat of the arc.

Automatic arc-welding processes produce high quality welds at very high welding speeds. They are commonly used in construction and fabrication.

2.4.2.2 Design Requirements and Allowable Stresses

Turkish specifications on welding are given in TS 3357 which covers all welded steel constructions subject to, generally, stationary loads.

General Provisions :

TS 3357 is valid only for St 37 and St 52 structural steels. If St 33 steel is used then the allowable stresses shall be half of those given below. St 37 plates thicker than 30 mm and St 52 plates thicker than 25 mm require special welding tests for bending check before they are used in welded structures.

According to TS 3357, for the following cases, there is no need to check the resultant stress σ_v:

i) In fillet welds where $\sigma=0$ or $\tau=0$

ii) In a connection which is subject to moment, shear force and axial force, if the following assumptions are made:

   1. Moment is carried by the flange welds
   2. Shear force is carried by the web welds.
   3. Axial force is carried by all welds.

iii) The values of σ and $\sqrt{\tau^2+\tau_{11}^2}$ are those given in Table 2.8.

Table 2.8 Allowable Stresses (kg/cm^2)

	Type of Steel	
	St 37	St 52
σ and $\sqrt{\tau^2+\tau_{11}^2}$	750	1200
$\sigma+\sqrt{\tau^2+\tau_{11}^2}$	1100	1700

Table 2.9 Allowable Stresses in Welds (kg/cm^2)

Type Of Weld	Type of Stress	Type of Steel			
		St 37		St 52	
		EY	EIY	EY	EIY
1- Butt Welds, K-Welds	Compression	1400	1600	2400	2700
2- Fillet Welds	Compression, Tension, Resultant	1100	1250	1700	1900
3- All Welds	Shear	1100	1250	1700	1900

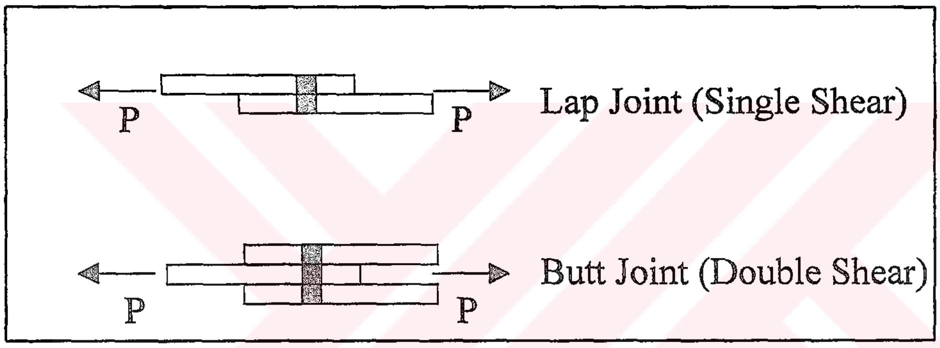

Figure 2.1 Types of Joints for Bolted Connections

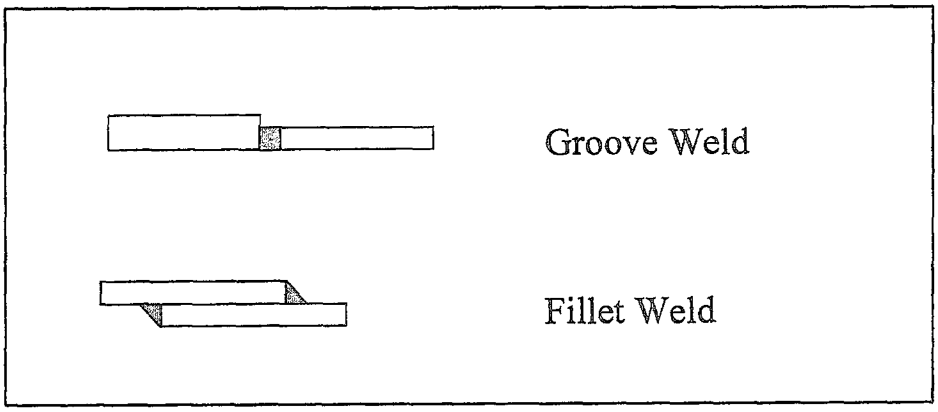

Figure 2.2 Types of Welds

CHAPTER 3

INTRODUCTION TO EUROCODE 3 PART 1.1

This chapter describes the contents of Part 1.1 of Eurocode 3[3], the new European standard for the design of buildings in steel. First, a general information is given in introduction part and then the connection design is explained in detail in the connection part.

3.1 Introduction

3.1.1 Scope of Eurocode 3 Part 1.1

Eurocode 3 (EC3) applies to the design of buildings and civil engineering works in steel. It is based on limit state principles and comes in several parts as shown in Table 3.1. The Eurocode is only concerned with the requirements for resistance, serviceability and durability of structures. Other requirements, e.g. concerning thermal and sound insulation are not considered.

Eurocode 3 does not cover the special requirements of seismic design. Rules related to such requirements are provided in ENV 1998

Eurocode 8 " Design of structures for earthquake resistance" which adapts the rules of Eurocode 3 specifically for this purpose.

Table 3.1 Overall Scope of Eurocode 3

Part	Subject
1.1	General rules and rules for buildings
1.2	Fire resistance
1.3	Cold formed thin gauge members and sheeting
2	Bridges and plated structures
3	Towers, masts and chimneys
4	Tanks, silos and pipelines
5	Piling
6	Crane Structures
7	Marine and maritime structures
8	Agriculture

Part 1.1 of Eurocode 3 gives a general basis for the design of buildings and civil engineering works in steel. Part 1.1 of Eurocode 3 is hereafter referred to as EC3. It was first published in draft form in 1984 and then a European pre-standard, reference no. DD-ENV 1993-1-1: 1992, in September 1992.

EC3 is published in two volumes and deals with the following subjects:

Volume 1:

Chapter 1:	Introduction (covers layout of code, important conventions and assumptions.
Chapter 2:	Basis of Design

Chapter 3: Materials
Chapter 4: Serviceability Limit State
Chapter 5: Ultimate Limit State
Chapter 6: Connection Subjected to Static Loading
Chapter 7: Fabrication and Erection
Chapter 8: Design assisted by Testing
Chapter 9: Fatigue

Volume 2:
Annex B: Reference Standards
Annex C: Design against Brittle Structure
Annex E: Buckling Length of a Compression Member
Annex F: Lateral-torsion Buckling
Annex J: Beam to Column Connections
Annex K: Hollow Section Lattice Girder Connections
Annex L: Column Bases
Annex M: Alternative Method for Fillet Welds
Annex Y: Guidelines for Loading tests

There are two types of Annex in EC3: normative and informative. Normative Annexes have the same status as the main body of the text, while the Informative Annexes provide additional information. The Annexes generally contain more detailed material or material which is used less frequently.

3.1.2 Distinction between Principles and Application Rules

In order to produce a document which is concise, describes the overall aims of the design and gives specific guidance as to how these aims can be achieved in practice, the material in EC3 is divided into ‘principles’ and ‘application rules’.

Principles are general statements, definitions, analytical methods, etc. for which no alternative is permitted. The principles are printed in roman type.

Application rules are generally recognized rules which follow the principles and satisfy their requirements.

Application rules each contain one suggested method for satisfying the corresponding principle. It is permissible to use alternative design rules provided that it can be shown that they satisfy the relevant principles and do not negate the other aspects, e.g. serviceability, durability, of the structure.

3.1.3 Assumptions

In EC3, the following assumptions are specified at the beginning.

- Structures are designed by appropriately qualified and experienced personnel.
- Adequate supervision and quality control is provided in factories, in plants and on site.
- Construction is carried out by personnel having the appropriate skill and experience.

- The construction materials and product are used as specified in EC3 or in the relevant material and product specifications.
- The structure will be adequately maintained.
- The structure will be used in accordance with the design brief.

3.1.4 Units

S.I. units shall be used. (kN, m and kg/m^3, etc.)

3.1.5 Basis of Design

In EC3, there are some fundamental requirements specified for a sound design. A structure shall be designed and constructed in such a way that two basic conditions should be fulfilled.

First, with acceptable probability, it will remain fit for the use which it is required, having due to regard to its intended life and its cost.

Secondly, with appropriate degrees of reliability, it will sustain all actions (loads) and influences likely to occur during construction period and service period and adequate durability in relation to maintenance costs.

These two basic items in design stage can be satisfied by the choice of materials, by appropriate design and detailing and by specifying control procedures for production, construction and use.

EC3 is a limit state code in which the principles and rules are given for the verification of Serviceability Limit State (SLS) and Ultimate Limit State (ULS)[7].

The limit states are the states beyond which the structure no longer satisfies the design performance requirement.

The principal serviceability limit states in Eurocode 3 are listed below.

- Deformations or deflections which affect the appearance or effective use of the structure;
- Vibrations, oscillation or sway (i.e. dynamic effects) which causes discomfort to the occupants of a building or damage to its contents;
- Damage to finishes or non-structural elements due to deformations, deflections or dynamic effect.

The principal ultimate limit states for steel structures and components are also listed below.

- Static equilibrium of the structure,
- Rupture or excessive deformation of a member,
- Transformation of the structure into a mechanism,
- Instability induced by the second-order effects, e.g. lack of fit, thermal effects, sway;
- Fatigue,
- Accidental damage (may include fire resistance).

In the context of elemental design, the designer is principally concerned with the ultimate limit states which affect the strength of the member, e.g. yield, buckling and rupture and the serviceability limit state of deflection.

3.1.6 Actions

Before proceeding with load cases considered in Eurocode 3, the term 'action' is defined. All the details about actions are introduced in Eurocode 1 : Basis of Design and Actions on Structures.

An action means two things. Firstly, it may be a force or a load that is applied to a structure. This one is also called as direct action. Secondly, it may be an imposed deformation, e.g. temperature effects or differential settlement. This one is also termed as indirect action. The letter "F" stands for the action in Eurocode terminology.

Actions are classified in three groups. First one is the permanent actions. The letter "G" stands for permanent actions in Eurocode terminology. It comprises such loads as self-weight of structures, fittings, ancillaries and fixed equipment, etc. The second group of actions are called as variable actions. The letter "Q" represents this type of actions. Imposed loads, wind load and snow load can be thought as this type of loading. The letter "q" is used for imposed loads in Eurocode terminology, and the letters "w" and "s" are used for wind loads and snow loads, respectively.

The term 'characteristic value of an action' means the real naked values and not factored by the partial safety factors, yet. F_k stands for the characteristic value of an action. The characteristic values of actions are either specified in Eurocode 1 or other relevant loading codes, or by client or the designer in consultation with the client, provided that the minimum provisions specified in the relevant loading codes or by the competent authority are observed.

The ‘design values of actions’ are obtained by multiplying the characteristic actions by the appropriate partial safety factor for actions. It can be expressed in general terms as follows;

$$F_d = \gamma_f * F_k \tag{3.1}$$

where	F_k	is the characteristic, i.e. unfactored value of action.
	γ_f	is the partial safety factor for the action considered by taking into account of the possibility of unfavorable deviations of the actions, the possibility of inaccurate modeling of the actions and uncertainties in the assessment of the limit state considered.

Another term used in Eurocode 3 is ‘the effects of actions’ that is denoted by letter “E”. The effects of actions are responses of the structure to the actions. Examples of these responses are internal forces and moments, stresses, strains, deflections and rotations. Design values of the effects of actions are determined from the design values of actions, geometrical data and material properties. ”E_d” is the letter used to denote the design values of actions.

3.1.7 Material Properties

Table 3.2 shows the characteristic yield and ultimate strength of structural steelwork recommended for use by Eurocode 3.

Table 3.2 Nominal Values of Yield Strength, f_y, and Ultimate Tensile Strength, f_u, for Structural Steel to EN 10025

Nominal steel grade	Thickness, t (mm)			
	$t \leq 40$ mm		40 mm$< t \leq$ 100 mm	
	f_y (N/mm^2)	f_u (N/mm^2)	f_y (N/mm^2)	f_u (N/mm^2)
Fe 360	235	360	215	340
Fe 430	275	430	255	410
Fe 510	355	510	335	490

t is the nominal thickness of the element
- of the flange of rolled sections
- of the particular elements of the welded sections.

fy = yield strength
fu = ultimate tensile strength

The table above is arranged in accordance with BS EN 10025:1990. The latest version, BS EN 10025:1993, uses "S" instead of "Fe". For example, the designation S275 is used in stead of Fe 430 and S355 is used in stead of Fe 510. For the new designation style, numeric character shows the yield strength of the steel grade used.

The below table shows the typical material coefficients used in the design.

Table 3.3 Design Values of Material Coefficients

Modulus of elasticity	E= 210000 Mpa
Shear modulus	G= 81000 Mpa
Coefficient of thermal expansion	$\alpha = 12*10^{-6}$ 1/C
Density	$\rho = 7850$ kg/m^3

In Eurocode 3, the material properties for steel structures are generally represented by the nominal values used as characteristic values, i.e. unfactored. X_k is used to denote this feature.

For steel structures, the design resistance, R_d, is generally determined directly from the characteristic values of the material properties and geometrical data as follows:

$$R_d = R(X_k, a_k, \ldots) / \gamma_M \tag{3.2}$$

where γ_M is the partial safety factor for the resistance that are introduced in Table 3.4.

X_k is the characteristic values of the material properties.

a_k is the characteristic values of the geometrical data.

Table 3.4 Partial Safety Factors for the Resistance

- at Ultimate Limit States:	
- resistance of class 1,2 or 3 cross-sections	$\gamma_{M0} = 1.1$
- resistance of class 4 cross-section	$\gamma_{M1} = 1.1$
- resistance of members to buckling	$\gamma_{M1} = 1.1$
- resistance of net sections at bolt holes	$\gamma_{M2} = 1.25$
- resistance of bolted connections	$\gamma_{Mb} = 1.25$
- resistance of welded connections	$\gamma_{Mw} = 1.25$
- at Serviceability Limit States:	
- slip resistance of preloaded bolts	$\gamma_{Ms.ser} = 1.1$

3.1.8 Classification of Cross-sections

For a designer the usual procedure is to choose a cross-section in such a way that the maximum capacity is not controlled by local buckling but is associated with the bearing load of a particular member of the structure. Therefore, the local buckling plays an important role in the design of structural steel. The critical level over which local buckling appears is defined by the classification of cross-sections.

For the check of cross-sections and members at Ultimate Limit State, the steel cross-sections shall be classified. The classification of cross-sections allows to evaluate beforehand their behavior, their ultimate resistance and their deformation capacity, taking into account the possible limits on the resistance due to local buckling of the compression elements of cross-sections.

The classification of cross-section permits to guide the selection of the global analysis of the structure, i.e. elastic or plastic global analysis. It also allows to determine the criteria to be used for ultimate limit state checks of the cross-sections and members, e.g. utilization of the partial safety factors.

Four classes of cross-section are defined according to the slenderness of its compression elements (width-over-thickness ratios of web or flange), the yield strength of steel and the applied loading.

‘Class 1 cross-sections’ are those which can form a plastic hinge with the rotation capacity required for the 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 stresses in the extreme compression fiber 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 it is necessary to make explicit allowances for the effects of local buckling when determining their moment resistance or compression resistance.

The ultimate resistance of cross-sections and of members submitted to bending and/or compression, depends on class of cross-sections and is based on the properties tabulated below.

Table 3.5 Properties of Cross-sections

	Distribution of stresses across the section	Cross-section properties for ULS check formulas	ULS partial safety factors
Class 1 or 2	- full plastic distribution - at the level of yield str.	Plastic prop. (W_{pl})	γ_{M0}
Class 3	- elastic distribution - with yield str. reached in the extreme fiber	Elastic prop. (W_{el})	γ_{M0}
Class 4	- elastic distribution across the effective sect. considering local buckling - with yield strength reached in the extreme fiber.	Effective prop.(A_{eff}, W_{eff})	γ_{M1}

3.1.9 Design Requirements

In order to achieve a proper design in accordance with the Eurocode 3, it shall be verified that no relevant limit state is exceeded. All relevant design situations and load cases shall be considered. Possible deviations from the assumed directions or positions of actions shall be considered. Calculations shall be performed using appropriate design models involving all relevant variables. The models shall be sufficiently precise to predict the structural behavior, commensurate with the standard of workmanship likely to be achieved, and with the reliability of the information on which the design is based.

For serviceability limit state, it shall be verified that :

$$E_d \leq C_d \tag{3.3}$$

where E_d is the design effects of actions determined on the basis of one of the load combinations given in Table 3.6

C_d is a nominal value or a function of certain properties of materials related to design effect of actions considered.

For example, E_d may be mid-span deflection of a beam, and the C_d will be the limiting value for deflection, say L/200.

Table 3.6 Combinations of Actions for Serviceability Limit State

<u>Load combinations to be considered:</u>	G_k- permanent actions, e.g. self-weight
1. $\Sigma G_k + Q_{k.max}$	Q_k –variable actions, e.g. imposed loads, snow loads, wind loads
2. $\Sigma G_k + 0.9\Sigma Q_k$	$Q_{k.max}$ – the variable action which causes the largest effect

For the ultimate limit state, considering a limit state of rupture or excessive deformation of a section, member or connection, it shall be verified that:

$$S_d \leq R_d \tag{3.4}$$

where S_d is the design value of an internal force or moment determined on the basis of one of the load combinations given in Table 3.7

R_d is corresponding design resistance.

For example, $M_{Sd} \leq M_{Rd}$ should be satisfied. M_{Sd} is the design moment and M_{Rd} is the design resisting moment.

For the ultimate limit state, the load combinations are given in Table 3.7.

Table 3.7 Combination of Actions for Ultimate Limit State

Load combinations to be considered:	G_k- permanent actions, e.g. self-weight Q_k –variable actions, e.g. imposed loads, snow loads, wind loads $Q_{k.max}$ – the variable action which causes the largest effect γ_G – partial safety factors for permanent loads γ_Q – partial safety factors for variable loads
$\gamma_G * \Sigma G_k + \gamma_Q * Q_{k.max}$ 1. $1.35 * \Sigma G_k + 1.50 * Q_{k.max}$	
$\gamma_G * \Sigma G_k + 0.9 \gamma_Q * Q_k$ 2. $1.35 * \Sigma G_k + 1.35 * Q_k$	
If the dead load G counteracts the Variable action Q, e.g. wind load, $\gamma_G = 1.00$	
If the variable load Q counteracts the dominant loading, $\gamma_Q = 0.00$	
The load combination which gives the largest effect (i.e. internal forces or moment) is decisive.	

3.2 Design of Connections

In this part, design of connections subject to static loading in accordance with Eurocode 3 Part 1.1 is emphasized in more details. Firstly, some basic design ideas about connections are given and then general points are presented such as applied forces and moments to connections, how they resist to these forces, design assumptions which are fundamental in all aspects and finally remarks on fabrication and erection of steel structures.

3.2.1 Basis of Design

All connections shall have a design resistance such that the structure remains effective and is capable of satisfying all the basic design requirements.

The partial safety factors concerning the resistances of bolts, welds and slip resistance of preloaded bolts are given in Table 3.4.

In order to get the forces and moments applied to the connections, global analysis of the structural system shall be carried out in accordance with Eurocode 3. These applied forces and moments shall include second order effects, the effects of imperfections and effects of connection flexibility in the case of semi-rigid connections. Having obtained these force resultants, the connections may be designed by distributing the internal forces and moments in whatever rational way is best. In order to achieve this, there are some fundamental points to be taken under consideration. First one is that the assumed internal forces and moments are in equilibrium with the applied forces and moments. Second one is that each element in the connection is capable of resisting the forces or stresses assumed in the analysis. Third one is that the deformation implied by this distribution are within the deformation capacity of the fasteners or welds and the connected parts. The last point is that the deformation assumed in any design model based on the yield lines are based on rigid body rotations and in-plane deformations which are physically possible. In addition, the assumed distribution of internal forces shall be realistic with regard to relative stiffnesses within the joint. The internal forces seek to follow the path with the greatest rigidity. This path shall be clearly identified and are consistently followed throughout the design of the connection. Also another point is that the residual stresses and stresses due to ordinary accuracy of fit-up need not normally be allowed for.

The resistance of a connection is determined on the basis of resistances of the individual fasteners or welds. Linear elastic analysis is generally used in the design of connections. As an alternative to this procedure, non-linear analysis of the connection can be employed provided that it takes into account of the load deformation characteristics of all the components of the connection.

In the design of connections, there are points to be considered with regard to ease of fabrication and erection. Attention should be paid to the clearances necessary for safe erection and for tightening fasteners, the need for access for welding. Moreover, all inspections, surface treatment and maintenance of the connections should be followed closely.

Another point while making a connection design is to arrange the meeting of members at a joint coincident with their centroidal axes intersecting at a point. In case of an eccentricity, this should be taken into account. In the case of joints with angles or tees connected by at least two bolts at each connection, the setting out lines for the bolts in the angles or tees may be substituted for the centroidal axes for the purpose of intersection at the joints.

3.2.2 Classification of Connections

Connections may be classified by considering two criteria. One is rigidity and the other one is strength.

In terms of rigidity, three types of connections are defined. These are nominally pinned connections, rigid connections and semi-rigid connections. A nominally pinned connection is designed in such a way that it can not develop significant moments which may adversely affect members of structure. They should be capable of transmitting the forces and free of rotation. A rigid connection, on the other hand, is designed on the basis that its deformation has no significant influence on the distribution of internal forces and moments in the structure. The deformations of rigid connections should be such that they do not reduce the resistance of the structure by more than 5 %. The semi-rigid connections are the ones which do not meet any of the criteria mentioned for both pinned and rigid connections. This type of connections should provide a predictable degree of interaction between the members based on the moment-rotation characteristics of the joint.

Another criteria for the classification of joints is strength. There are again three types defined in terms of strength. These are nominally pinned connections, full-strength connections and partial strength connections. Pinned connections do not allow the development of the moment. They have enough rotation capacity to enable all the necessary plastic hinges to form under the design loads. Full strength connections should provide design resistance not less than that of the member connected. The rigidity of a full-strength connection should be such that, under the design loads, the rotations at the necessary plastic hinges do not exceed their rotation capacities. The last type of connection is partial strength connections. The design resistance of this type of connections shall not be less that that necessary to transmit the calculated design forces and moments, but may be less than that of the member connected. Moreover, the rotation capacity of a partial strength connection which occurs at a plastic hinge location shall not be less than that needed to enable all the necessary plastic hinges to develop under the design loads.

3.2.3 Bolted Connections

The nominal values of the yield strength, f_{yb}, and ultimate tensile strength, f_{ub}, of bolts are shown in Table 3.8.

Table 3.8 Nominal Bolt Strengths

Bolt grade	f_{yb} ($N.mm^{-2}$)	f_{ub} ($N.mm^{-2}$)
4.6	240	400
4.8	320	400
5.6	300	500
5.8	400	500
6.8	480	600
8.8	640	800
10.9	900	1000

The nominal clearances in standard holes for bolted connections and the nominal clearances in oversize holes for slip-resistant connections should be as shown in Table 3.9.

Table 3.9 Clearances in Holes for Fasteners

Standard holes for bolted connections		Oversized holes for slip-resistant connections	
Bolt diameter (mm)	Clearance (mm)	Bolt diameter (mm)	Clearance (mm)
M12-M14	1	M12	3
M16-M24	2	M14-M22	4
M27 and larger	3	M24	6
		M27 and larger	8

3.2.3.1 Positioning of Bolts

The next subject is the positioning of the bolts. It should be such that no corrosion and local buckling take place. Also it should ease the installation of the bolts.

Minimum End Distance:

The end distance, e_1, from the center of a fastener hole to the adjacent end of any part, measured in the direction of load transfer, should not be less than $1.2d_0$, where d_0 is the hole diameter. The end distance should be increased to provide adequate bearing resistance.

Minimum Edge Distance:

The edge distance, e_2, from the center of a fastener hole to the adjacent edge of any part, measured at right angle to the direction of load transfer should not be less than $1.5d_o$.

Maximum End and Edge Distances:

Under normal conditions, the end and edge distances should not exceed 12t or 150 mm, whichever is larger, where t is the thickness of the thinner outer connected part. Also, edge distance should not also exceed the maximum to satisfy the local buckling requirements for an element.

Minimum Spacing:

The spacing between, p_1, the centers of fasteners in the direction of the load transfer should not be less than $2.2d_o$.

The spacing, p_2, between the rows of fasteners, measured perpendicular to the direction of load transfer should not be less than $3.0d_o$.

Maximum Spacing in Compression Members:

The spacing, p_1, of the fasteners in each row and spacing, p_2, between row of fasteners should not exceed the lesser of 14t or 200 mm. Adjacent rows of fasteners may be symmetrically staggered.

Moreover, the center-to-center spacing of fasteners should not exceed the maximum width which satisfies the local buckling requirements for an internal element.

Slotted Holes:

The minimum distance, e_3, from the axis of a slotted hole to the adjacent end or edge of any part should not be less than $1.5d_0$.

The minimum distance, e_4, from the center of the end radius of a slotted hole to the adjacent end or edge of any part should not be less than $1.5d_0$.

3.2.3.2 Categories of Bolted Connections

Eurocode 3 categorizes the bolted connections as shear carrying connections and tension connections. Category A, B and C include the shear connections. Category D and E include the tension connections.

Category A consists of bearing type of bolted connections. Ordinary bolts and high strength bolts of all grades shall be used as this type. No pre-loading and special provisions for contact surfaces are required. The design ultimate shear load shall not exceed design shear resistance nor the design bearing resistance.

$$F_{v.Sd} \leq F_{v.Rd} \tag{3.5}$$

$$F_{v.Sd} \leq F_{b.Rd} \tag{3.6}$$

where $F_{v.Sd}$ = design shear force per bolt for the ultimate limit state.

$F_{v.Rd}$ = design shear resistance per bolt.

$F_{b.Rd}$ = design bearing resistance per bolt.

In Category B, the bolted connections are required to be slip-resistant at serviceability limit state. Pre-loaded high strength bolts with controlled tightening shall be used. Slip shall not occur at the serviceability limit state. The design serviceability shear load shall not exceed the design slip resistance. In addition to this check, the design ultimate shear load shall not exceed design shear resistance nor the design bearing resistance.

$$F_{v.Sd.ser} \leq F_{v.Rd.ser} \tag{3.7}$$

$$F_{v.Sd} \leq F_{v.Rd} \tag{3.5}$$

$$F_{v.Sd} \leq F_{b.Rd} \tag{3.6}$$

where $F_{v.Sd.ser}$ = design shear force per bolt for the serviceability limit state.

$F_{v.Rd.ser}$ = design slip resistance per bolt for the serviceability limit state.

In Category C, the bolted connections are required to be slip-resistant at ultimate limit state. Pre-loaded high strength bolts with controlled tightening shall be used. Slip shall not occur at the ultimate limit state. The design ultimate shear load shall not exceed the design slip resistance nor the design bearing resistance.

$F_{v.Sd} \leq F_{s.Rd}$ (3.8)

$F_{v.Sd} \leq F_{b.Rd}$ (3.6)

where $F_{s.Rd.}$ = design shear force per bolt for the ultimate limit state.

Category D consists of tension connection with no pre-loading. Ordinary bolts and high strength bolts of all grades shall be used as this type. This category shall not be used where the connections are frequently subjected to variations of tensile loading. The design ultimate tensile load shall not exceed design tension resistance.

$F_{t.Sd} \leq F_{t.Rd}$ (3.9)

where $F_{t.Sd}$ = design tensile force per bolt for the ultimate limit state.

$F_{t.Rd}$ = design tension resistance per bolt.

Category E consists of tension connection with pre-loading. Only high strength bolts with controlled tightening shall be used as this type. If this type of connection is subject to both tension and shear, special surface treatment of the contact area may be needed.

The design ultimate tensile load shall not exceed design tension resistance.

$F_{t.Sd} \leq F_{t.Rd}$ (3.9)

3.2.3.3 Distribution of Forces between Fasteners

The distribution of internal forces between fasteners at the ultimate limit state shall be proportional to the distance from the center of rotation, for Category C slip resistant connections and other shear connections where the design shear resistance of the connection is less than the design bearing resistance.

In other cases the distribution of internal force between bolts at the ultimate state may be either as stated above or plastic.

3.2.3.4 Design Resistance of Bolts

At the ultimate limit state the design shear force $F_{v.Sd}$ on a bolt shall not exceed the lesser of :

- the design shear resistance $F_{b.Rd}$
- the design bearing resistance $F_{v.Rd}$

At the ultimate limit state the design tensile force $F_{t.Sd}$, inclusive of any force due to prying action, shall not exceed the lesser of :

- the design tension resistance $F_{t.Rd}$
- the design punching shear resistance $F_{p.Rd}$

3.2.3.5 Design Shear Resistance

If the shear plane passes through the threaded portion of the bolt, the design shear resistance per shear plane, $F_{v.Rd}$, for strength grades 4.6, 5.6 and 8.8 bolts is given by

$$F_{v.Rd} = 0.6\, f_{ub}\, A_s / \gamma_{Mb} \qquad (3.10)$$

and for strength grades 4.8, 5.8, 6.8 and 10.9 bolts is given by

$$F_{v.Rd} = 0.5\ f_{ub} A_s / \gamma_{Mb} \tag{3.11}$$

If the shear plane passes through the unthreaded portion of the bolt, the design shear resistance is given by

$$F_{v.Rd} = 0.6\ f_{ub} A / \gamma_{Mb} \tag{3.12}$$

where A = the gross cross-section area of the bolt,

A_s = tensile stress area of the bolt,

d = bolt diameter,

d_o = the hole diameter.

Note that these values for design shear resistance apply only where the bolts are used in holes with nominal clearances specified previously.

As a special note for "long joints", where the distance L_j between the centers of the end bolts in a joint is more than 15d, the design shear resistance, $F_{v.Rd}$, of all the bolts shall be reduced by a reduction factor, β_{Lf}, given by

$$\beta_{Lf} = 1 - (L_j - 15d) / 200d \tag{3.13}$$

where $0.75 \leq \beta_{Lf} \leq 1.00$ should be followed.

3.2.3.6 Design Bearing Resistance

The design bearing resistance, $F_{b.Rd}$, is given by

$$F_{b.Rd} = 2.5\ \alpha\ f_u\ d\ t / \gamma_{Mb} \qquad (3.14)$$

where α is the smallest of:

$(e_1/3d_o)$; $(p_1/3d_o)$-1/4; f_{ub}/f_u or 1

and, e_1 = end distance in the direction of load,
p_1 = spacing of bolts in the direction of load,
f_{ub} = tensile strength of a bolt,
f_u = tensile strength of the connected material.

Note that the values of the design bearing resistance only apply where the edge distance e_2 is not less than $1.5d_o$ and the spacing p_2 is not less than $3.0d_o$, where e_2 is the edge distance perpendicular to the direction of load, p_2 is spacing of bolts in the transverse direction of load. If e_2 is reduced to $1.2d_0$ and/or p_2 is reduced to $2.4d_0$, then the bearing resistance $F_{b.Rd}$ should be reduced to 2/3 of the original value.

3.2.3.7 Design Tension Resistance

The design tension resistance of a bolt is given by

$$F_{b.Rd} = 0.9\ f_{ub}\ A_s / \gamma_{Mb} \qquad (3.15)$$

When the plate thickness t_p is smaller than 0.5d, the design punching resistance of the bolt head and the nut shall be checked and evaluated as follows:

$$B_{p.Rd} = (\,0.6\,\pi\,d_m\,t_p\,f_u\,) / \gamma_{Mb} \qquad (3.16)$$

3.2.3.8 Interaction of Shear and Tension

Bolts subject to both shear and tension shall in addition satisfy the following criterion;

$$(\,F_{v.Sd} / F_{v.Rd}\,) + (\,F_{t.Sd} / 1.4F_{t.Rd}\,) \leq 1.0 \qquad (3.17)$$

3.2.3.9 High Strength Bolts in Slip-resistant Connections

When the slip-resistance is needed in the serviceability limit state, the design for a preloaded high-strength bolt shall be carried out.

The design serviceability shear load should not exceed the design slip-resistance of a preloaded bolt, $F_{s.Rd}$, that is given by

$$F_{s.Rd} = k_s\,n\,\mu\,F_{p.Cd} / \gamma_{Ms.ser} \qquad (3.18)$$

where k_s = 1.0 where the holes in all plies have standard nominal clearances as outlined before, k_s = 0.85 for oversize or short slotted holes and k_s = 0.70 for long slotted holes.

n = number of friction interfaces,

μ = slip factor,

$F_{p.Cd}$ = design pre-loading force. It is given by

$$F_{p.Cd} = 0.7\ f_{ub}\ A_s \tag{3.19}$$

The design slip resistance of a pre-loaded high strength bolt at the ultimate state can be taken as given in formulae (3.18) with $\gamma_{Ms.ult}$ substitutes $\gamma_{Ms.ser}$.

The slip factor, μ, is dependent on the specified class of surface treatment. $\mu = 0.5$ for class A surfaces where surfaces are blasted with shot or grit, with any loose rust removed, or spray-metallized with aluminum. $\mu = 0.4$ for class B surfaces where surfaces are blasted with shot or grit, and painted with alkali-zinc silicate paint to produce a coating thickness of 50-80 µm. $\mu = 0.3$ for class C surfaces where surfaces are cleaned by wire brushing or flame cleaning with any loose rust removed. $\mu = 0.2$ for class D surfaces where surfaces are not treated.

3.2.3.10 Block Shear Check

Near the end of a member with a group of fastener holes in the webs the block shear failure shall be prevented by using appropriate hole spacing. The design value of shear force applied to the beam web shall be less than, $V_{eff.Rd}$, given by

$$V_{eff.Rd} = 0.6\ f_{ub}\ A_{v.net} / \gamma_{M2} \tag{3.20}$$

where $V_{eff.Rd}$ = the design value of the effective resistance to block shear,

$A_{v.net}$ = effective shear area,

f_u = ultimate tensile strength.

3.2.3.11 Net Section Resistance for Tension Members

For members in axial tension the design value of the tensile force $N_{x.Sd}$, at each cross-section shall be checked for the net section rupture at holes for fasteners, as given by

$$N_{u.Rd} = 0.9\, f_u A_{net} / \gamma_{M2} \tag{3.20}$$

where $N_{u.Rd}$ = the design ultimate resistance of the net cross-section,

A_{net} = net area of a member or element cross-section with appropriate deductions for holes and other openings,

f_u = ultimate tensile strength.

3.2.4 Welded Connections

Eurocode 3 also classifies the weld types in two types, namely, fillet welds and butt welds. Butt welds can be further divided into two classes as full penetration butt welds and partial penetration butt welds.

3.2.4.1 Fillet Welds

Fillet welds may be used for connecting parts where fusion faces form an angle of 60° and 120°. Smaller angles than 60° are also permitted. However, the fillet weld in such cases acts as a partial penetration butt weld. For angles over 120°, fillet welds are not safe in terms of transmitting forces.

The throat thickness, a, of a fillet weld is defined as the largest triangle which can be inscribed within the fusion faces and weld surface, measured perpendicular to the outer side of this triangle. The minimum throat thickness is specified as 3 mm.

In order to check the capacity of the fillet weld, the design forces transmitted through the weld is determined first. This design force is calculated as a resultant force of all three components, two of which are the shears in parallel and perpendicular directions to weld direction, and the third component is the out-of-plane normal force. This resultant force is given as shown below;

$$F_{w.Sd} = \sqrt{N^2_{\perp.Sd} + V^2_{\perp.Sd} + V^2_{//.Sd}} \qquad (3.21)$$

The design resistance of a fillet weld ,$F_{w.Sd}$, is given by

$$F_{w.Sd} = a\, L\, f_u / (\sqrt{3}\ \beta_w \qquad (3.22)$$

where a = throat thickness,

L = weld length,

f_u = ultimate tensile strength of the weaker part joined,

β_w = correlation factor. 0.80 for S235 steel, 0.85 for S275, 0.90 for S355, 0.95 for S420 and 1.0 for S460.

3.2.4.2 Butt Welds

As mentioned previously, there are two types of butt welds, namely, full penetration butt welds and partial penetration butt welds. A full penetration butt weld is defined as a butt weld that has complete penetration and fusion of weld and parent metal throughout the thickness of the joint. On the other hand, a partial penetration butt weld is defined as a butt weld that has a penetration thickness less than the thickness of the parent metal.

The throat thickness for a full penetration butt weld is taken as the thickness of the thinner parent metal. For partial penetration butt welds, throat thickness is the depth of penetration that can be achieved for a particular joint.

The design resistance of a full penetration butt weld is taken as equal to the design of the weaker of the parts joined.

The design resistance of a partial penetration butt weld is determined as for a deep penetration fillet weld.

CHAPTER 4

INTRODUCTION TO LRFD-AISC

This chapter describes the contents of Load and Resistance Factor Design (LRFD) released by American Institute of Steel Construction (AISC)[2]. The acronym LRFD and AISC will be used throughout the text. First, a general information is given in introduction part and then the connection design is explained in detail in the connection part.

4.1 Introduction

4.1.1 Scope of LRFD-AISC

LRFD-AISC applies to the design, fabrication and erection of steel-framed buildings. As an alternative, ASD-AISC, Allowable Stress Design, is also permitted.

Seismic design of buildings, design of single-angle members, design of hollow structural members and the design of nuclear structures are not included in this specification.

LRFD is a method of proportioning the structures so that neither of the limit states is exceeded when the structure is subjected to all appropriate factored load combinations.

The limit states that should be satisfied are the Ultimate Limit State, also called Strength Limit State, and Serviceability Limit State. Strength limit states are related to the safety and concern maximum load carrying capacity. Serviceability limit states are related to the performance under normal service conditions.

4.1.2 LRFD Philosophy

Today, there are two widely recognized philosophies that are being used in the design of steel structure. These are Allowable Stress Design method and the Load and Resistance Factor Design method.

The most common method for designing steel structures until the late 1980s was the allowable stress design because of its simplicity and so much of experience gained for years for safe and reliable design. The allowable stress design philosophy is based on keeping the stresses in a member below some fraction of a specified stress in the steel. By keeping the actual stress levels under a fraction of some specified stress, such as the yield stress, this method typically provides conservative results. It has been the primary design philosophy used since the introduction of steel as a building material. However, with the introduction of LRFD by AISC in 1986, the trend changes toward this alternative design method.

The LRFD method approaches structural steel design using a probabilistic approach, that incorporates the variability of loads and member resistance, to arrive at a more rational and cost effective solution. A simplified view of this method states that when members are loaded they respond by resisting that load. The failure of a member can occur because either the loads on a member are larger than those for which it was designed or because the resistance of the member is smaller than what was anticipated. In this respect, it is understood that the loads and member resistances are indeed variable over the life of the structure.

The loads on structures may consists of dead loads, live loads, wind loads, earthquake loads and snow loads. These loads are very different in terms of predictability. The uncertainty of a dead load should be relatively small as compared to the variability associated with the wind loads or earthquake loads. As a designer of a structure, one should have a good handle on the major components of dead load such as self weight, but possibly be less certain of the live load requirements of the structure. Therefore, one must logically states that live loads are more unpredictable than dead loads.

The resistance of a member to applied loads is also variable based on manufacturing, age and dimensions. When designer specify St 37 steel, this means that it is supposed to have a minimum yields strength of 2400 kg/cm^2. But there is small percentages of St 37 steel members with a yield strength less than 2400 kg/cm^2. The resistance of some steel members may also decrease with age because of processes such as corrosion which may reduce the effective cross-sectional area needed for resistance.

All these variability in loads and resistances are taken into account in LRFD by using a probabilistic design approach.

4.1.3 Limit States

When a member no longer functions, it is said to have exceeded a limit state. In the LRFD approach, as in ASD, there are two types of limit states which are discussed; strength limit states and serviceability limit states. Designers are primarily concerned with strength limit states such as moment capacity and shear capacity. Serviceability limit states are primarily related to deflection, vibration and drift. The focus of the most codes is on the strength limit states because of the great concern for public safety.

The limit state violations in allowable stress design are expressed in terms of a factor of safety against failure. From a design perspective, the major shortcoming with this philosophy is that the factors of safety for various behaviors, such as bending, shear, etc., all have different values. These values emerged from years of practical design experience and not from a rational basis. This has led to designs in which identical members experiencing different conditions, such as different loading types, would actually be very different in terms of a safety margin against failure. This is not to say that the allowable stress method worked poorly; on the contrary, it has worked well. But the philosophy, while working well in terms of public safety, is providing non-uniform reliability to all members.

In the LRFD method, the reliability index β is used in place of factor of safety. The reliability index ,β, is the number of standard deviations where the mean value of a combined resistance and loading probability lies away from the point of failure. The concept of probabilistic approach of LRFD is best explained in a thesis work called "Comparison of Load and Resistance Factor Design and TS648 for Structural Members" by Çağatay Nuhoğlu, 1996. One of the main goals of the LRFD philosophy is the establishment of uniform reliability between all types of members subjected to all types and combination of loads.

LRFD is unique as a philosophy because it provides a uniform reliability for all members in a structure based on a probabilistic approach to load factors and member resistances. This leads to the designs of more cost effective and reliable structures.

4.1.4 Basis of Design

The design method for LRFD can be expressed in one general equation:

Design Strength ≥ Required Resistance

or

$$\phi R_n \geq \Sigma \gamma_i Q_i \qquad (4.1)$$

where ϕ = resistance factor ($\phi < 1$),

R_n = nominal resistance of the member,

γ_i = load factors ($\gamma_i > 1$)

Q_i = different load effects.

The required resistance will be the factored moment, shear or axial load which are applied to a particular member. The required resistance is computed by considering all load combinations.

4.1.5 Load Combinations for LRFD

The load factors used for the required resistance by the LRFD were developed by the American National Standards Institute (ANSI A58.1, now ASCE 7-88) for the purpose of specifying minimum acceptable load criteria for buildings. LRFD uses the basic load combinations suggested by this standard as the minimum requirement for load carrying structure. As already mentioned, the LRFD specification states that the design strength of a member has to be greater than or equal to the required resistance which is the effect, such as moment, shear, etc., caused by the highest factored load combination as stated in ASCE. These load combinations with their respective load factors are given in Table 4.1.

Table 4.1 Load combinations for LRFD

1- 1.4 D	given by ASCE
2- 1.2 D + 1.6 L + 0.5 (L_r or S or R)	
3- 1.2 D + (0.5 L or 0.8 W) + 1.6 (L_r or S or R)	
4- 1.2 D + 0.5 L + 1.3 W + 0.5 (L_r or S or R)	
5- 1.2 D ± 1.0 E + 0.5 L + 0.2 S	
6- 0.9 D ± (1.3 W or 1.0 E)	
With D = gravity dead loads, L = gravity live loads, L_r = roof live loads, W = wind load, E = earthquake load, S = snow load, R = load to initial rainwater of ice.	

4.1.6 Resistance Factors for LRFD

The resistance factors, i.e. partial safety factors as used in Eurocode terminology, to be used in member design for LRFD specification is given in Table 4.2.

Table 4.2 Resistance (Partial Safety) Factors for LRFD

1- Axial compression	$\phi = 0.85$
2- Shear	$\phi = 0.90$
3- Bending Moment	$\phi = 0.90$
4- Yielding in a tension member	$\phi = 0.90$
5- Fracture in a tension member	$\phi = 0.75$
6- Bolts	$\phi = 0.75$
7- Welds in - shear	$\phi = 0.75$
- tension or compression	$\phi = 0.80$
9- Slip-Critical Connections - Standard holes - Over-sized short slotted - Long slotted ⊥ to F - Long slotted // to F	 $\phi = 1.00$ $\phi = 0.85$ $\phi = 0.70$ $\phi = 0.60$

4.1.7 Units

In LRFD-AISC 1999[2], the values and equations are given in both U.S. customary and metric, i.e. SI, units. This is a new feature in LRFD-AISC which was lacked in the previous versions. This shows the trend of U.S. designers toward using the same units with the rest of the world as a result of overseas works that they are intending to get.

4.1.8 Material used in LRFD-AISC

The designation of the steel used in buildings and bridges in United States are somewhat different than the Europeans. Some typical types of steel used are given in Table 4.3.

Table 4.3 Steels Used in Buildings and Bridges in U.S.A.

Steel Type	ASTM Designation	f_y (ksi)	f_u (ksi)	Common Usage
Carbon	A36	32	58-80	General Buildings
High-strength Low Alloy	A441	40	60	Welded Construction
Corrosion Resistant	A242	42	63	Weathering Steel
Quenched and Tempered low alloy	A514	90	100-130	Plates for Welding

4.2 Design of Connections

In this part, design of connections subject to static loading in accordance with LRFD-AISC is emphasized in more details. Firstly, some basic design ideas about connections are given and then general points are presented such as applied forces and moments to connections, and how they resist to these forces, etc.

4.2.1 General Remarks

Connections consists of elements of connected members, such as beam webs, connecting elements, such as gussets, angles, brackets, and connectors, such as welds, bolts and rivets. These components are proportioned so that their design strength equals or exceeds the required strength determined by the structural analysis for factored loads acting on the structure.

Three types of connections are defined in LRFD[2]. First one is the hinge type of joints. These are the joints where the rotational restraint at the ends of the member is as little as practicable. For beams, only shear transfer is intended. It is usually assumed to exist when the original angle between the intersecting members may change approximately 80% or more of the amount it theoretically would change if frictional hinged connections could be used. In LRFD, 'partially restrained' term is used for hinge type of connections and it is designated by "Type PR".

The second type of connection is defined as rigid connections. It is the situation where full continuity is provided at the connection so that the original angle between the intersecting members are maintained essentially constant during the loading of the structure. In LRFD, 'fully restrained' term is used for this type of connection and it is designated as "Type FR".

The last type of connection defined in LRFD is the semi-rigid joints where the rotational restrained is approximately between 20% and 90% of that necessary to prevent relative angle change. This means that the moment transmitted across the joint is neither zero nor the full moment as in the case of rigid joints. In LRFD, "Type PR" is also designates these types of connections. Semi-rigid connections are not used in structures when plastic analysis is used in design, and are not commonly used in Allowable Stress Design because of the difficulty in obtaining the moment-rotation relationship for a given connection.

In LRFD, minimum strength criterion is provided in connection design. All the connections should resist a factored load of 44 kN except for lacing, sag rods and girts.

Another point is the placement of the welds and bolts. The center of gravity of the group of welds or bolts at the ends of a member should be coincident with the center of gravity of the member itself.

In such cases where the bolts and welds are used together, one should care about the loads shared by the connectors. LRFD says that the bearing type of bolted connections do not share any load in combination with welds. Welds are assumed to take all the loads in that particular connection.

4.2.2 Bolted Connections

The need to join steel members together has existed since the introduction of steel as a building material in the latter half of the nineteenth century. Steel structures of every type require fastening of individual members to achieve their ultimate structural form. Two of the most common methods of connecting steel structures[11] in US are high-strength bolting and riveting.

An early method of connecting steel members that was widely accepted was riveting. They have virtually been replaced in today's world with high-strength bolts due primarily to economic considerations.

High-strength bolting is one of the most common procedures used today in the connection of structural steel members. The two most common ones are the A325 bolt and the A490 bolt. The A325 bolt is the most common high-strength bolt used today and is made from heat-treated medium carbon steel, while A490 bolt is a higher strength bolt manufactured from an alloy steel.

The proper installation of bolts in some connections requires that the bolts be "fully tensioned", while in other connections the bolts need only be "snug-tight". The snug-tight condition brings the plates being joined into firm contact, and is achieved by a few impacts of an impact wrench or the complete effort of a worker using a spud wrench. A fully tensioned connection is based on achieving adequate clamping forces between the members being joined. This clamping force is provided by attaining the proper tension in the bolts, which is required to prevent slippage of the plates being connected and to prevent the bolts becoming loose. This is especially critical in structures that are subject to repeated stresses or stress reversals. Minimum bolt tensions for fully tensioned A325 and A490 bolts given by LRFD[2] are shown in Table 4.4.

Table 4.4 Minimum Bolt Pretension, kN

<table>
<tr><td>Bolt Size, mm</td><td>A325 Bolts</td><td>A490 Bolts</td></tr>
<tr><td>M16</td><td>91</td><td>114</td></tr>
<tr><td>M20</td><td>142</td><td>179</td></tr>
<tr><td>M22</td><td>176</td><td>221</td></tr>
<tr><td>M24</td><td>205</td><td>257</td></tr>
<tr><td>M27</td><td>267</td><td>334</td></tr>
<tr><td>M30</td><td>326</td><td>408</td></tr>
<tr><td>M36</td><td>475</td><td>595</td></tr>
<tr><td colspan="3">Minimum bolt tensions are equal to 0.70 of minimum tensile strength of the bolt.</td></tr>
</table>

4.2.2.1 Design Requirements

4.2.2.2 Tensile Strength of Fasteners

In accordance with the fracture limit state in tension, the nominal strength R_n of one fastener in tension is given by

$$R_n = F_u^b A_n \tag{4.2}$$

where F_u^b is the tensile strength of the bolt material. The net area A_n should be the area through the threaded portion of the bolt, known as the "tensile stress area".

$$A_n = 0.7854 \left[D - (0.9743 / n) \right]^2 \tag{4.3}$$

D is the diameter of the bolts in inches, and n is the number of threads per inch. The ratio of tensile stress area to the gross area A_b ranges from 0.75 and 0.79. Thus, in terms of the gross area A_b of one bolt Eq. 4.3 becomes

$$R_n = F_u^b (0.75 A_b) \tag{4.3}$$

The design strength ϕR_n based on the tension strength of the fastener is given by,

$$\phi R_n = \phi F_u^b (0.75 A_b) \tag{4.4}$$

where ϕ is resistance factor for fracture in tension and equals to 0.75. Then, Eq. 4.4 becomes

$$\phi R_n = 0.75 F_u^b (0.75 A_b) \tag{4.5}$$

4.2.2.3 Bearing Strength of Fasteners

The bearing limit state relates to deformation around a bolt hole. The bearing strength R_n is the force applied against the side of the hole to split or tear the plate. The larger the end distance, L, measured from the center of the hole to the edge, the less the possibility of having a splitting failure.

The design strength ϕR_n based on the bearing strength at bolt holes is prescribed in several categories:

1. For usual conditions (standard holes or short-slotted holes, end distance not less than 1.5d, bolt center-to-center spacing not less than 3d, and with the two or more bolts in the line force) ,

$$\phi R_n = \phi (2.4 \, d \, t \, F_u) \tag{4.6}$$

where $\phi = 0.75$

d = nominal diameter of bolt (not at threads)
t = thickness of the connected part
F_u = tensile strength of steel comprising connected part

2. For long-slotted holes perpendicular to the direction of the applied load, end distance not less than 1.5d, bolt center-to-center spacing not less than 3d, and with the two or more bolts in the line force,

$$\phi R_n = \phi (2.0\ d\ t\ F_u) \tag{4.7}$$

where $\phi = 0.75$.

3. For the bolt closest to the edge when conditions of Eq. 4.6 and Eq. 4.7 are not satisfied,

$$\phi R_n = \phi (L\ t\ F_u) \tag{4.8}$$

where $\phi = 0.75$

L = end distance in line of force, from the center of a standard hole or oversized hole, or from the mid-width of a slotted hole, to an edge of a connected part.

4. When hole elongation greater than 0.25 in. and hole "ovalization" can be tolerated, then

$$\phi R_n = \phi (3.0\ d\ t\ F_u) \tag{4.9}$$

where $\phi = 0.75$.

4.2.2.4 Shear Strength of Fasteners

In accordance with the fracture limit state as the basis for fastener strength, the nominal strength R_n for one fastener will be the ultimate shear stress τ_u across the gross area A_b of the bolt times the number, m, of shear planes: thus,

$$R_n = m\,A_b\,\tau_u = m\,A_b\,(0.60\,F_u^b) \tag{4.10}$$

Note that the ultimate shear strength was found experimentally to be about 62% of ultimate tensile strength: about the same ratio as for the yield strengths. The practical coefficient 0.60 is used instead of 0.62.

Equation 4.10 assumes no threads to be in the shear planes. If threads are in the shear planes, the area at the root of the threads should be used in place of A_b. Since the area at the root of the threads is somewhat smaller than the tensile stress area, the root area is taken as 0.70 of the gross area. As a result, Equation 4.10 becomes

$$R_n = m\,(0.70\,A_b)\,(0.60\,F_u^b) \tag{4.11}$$

$$R_n = 0.45\,m\,A_b\,F_u^b \tag{4.12}$$

The design shear strength ϕR_n, for no threads in the shear planes, based on the shear strength of the fastener, according to LRFD is,

$$\phi R_n = \phi \ (0.60 F_u^b) m A_b \tag{4.13}$$

$$\phi R_n = 0.75 (0.60 F_u^b) m A_b \tag{4.14}$$

where $\phi = 0.75$,

F_u^b = tensile strength of bolt material, (See Table 1.1),

m = the number of shear planes participating

m = 1 for single shear

m = 2 for double shear

A_b = gross cross-sectional area across the unthreaded shank of the bolt.

The design shear strength ϕR_n, for threads in the shear planes, based on the shear strength of the fastener is

$$\phi R_n = \phi \ (0.45 F_u^b) m A_b \tag{4.15}$$

$$\phi R_n = 0.75 (0.45 F_u^b) m A_b \tag{4.16}$$

4.2.2.5 Slip-Critical Fasteners

When slip resistance at service load is desired, the connection is referred to as slip-critical connection or friction-type connection. All tensioned high-strength bolted connections actually resist load by friction. The applied pretension force, T, at the bolts equals the clamping force between the pieces being fastened. The resistance to shear is provided by means of frictional force μT, where μ is the coefficient of friction depending on the surface condition.

Slip is an action when the friction bond between the pieces being jointed is totally broken and the surfaces between the pieces slip with respect to each other with a considerable amount. The AISC specifications use the "shear stress" approach to provide adequate slip resistance in joints where slip at service load can not be tolerated. The limit state of slip in the joint is a serviceability requirement. Since the resistance to slip in slip critical connections is a limit state to be analyzed under service loads, the limiting "shear stresses" to be used are in concept the same for LRFD and ASD.

The design of slip-critical connections requires full consideration of strength limit states, namely, the strength of fasteners in shear, bearing and direct tension. These strength requirements should be fulfilled to resist factored loads. Moreover, the service load that must be transferred by friction without slip should not exceed the allowable.

Accordingly, the service load capacity R_n per bolt based on shear in a slip-critical connection is

$$R_n = F_v \ m A_b \tag{4.17}$$

where F_v is the maximum acceptable unfactored service load shear per bolt divided by the nominal area.

4.2.2.6 Minimum Spacing of Bolts in Line of Transmitted Force

The minimum spacing of bolts in a line is three bolt diameters and shall not be less than 8/3 bolt diameters.

Under all conditions, the following equation gives the minimum center-to-center spacing of bolts.

$$\text{Spacing} \geq (P / \phi F_u t) + d_h / 2 \qquad (4.18)$$

where $\phi = 0.75$

P = factored load acting on one bolt

F_u = tensile strength of plate material

t = thickness of plate material

d_h = diameter of the bolt hole

4.2.2.7 Minimum End Distance in Direction of Transmitted Force

If the generally accepted strengths given by Eq. 4.6 and Eq. 4.7 are used, the minimum end distances must be at least 3/2 bolt diameters. When higher bearing strengths are used, the minimum end distance, L, is given by LRFD as follows;

$$L \geq (P / \phi F_u t) \qquad (4.19)$$

where $\phi = 0.75$

P = factored load acting on one bolt

F_u = tensile strength of plate material

t = thickness of plate material

The end distance actually must be larger of that computed from Eq. 4.19 and the minimum prescribed by Table 4.5.

Table 4.5 Minimum Edge Distance, mm.

Bolt Diameter, mm	At Sheared Edges	At Rolled Edges
16	28	22
20	34	26
22	38	28
24	42	30
27	48	34
30	52	38
36	64	46

4.2.2.8 Nominal Hole Dimensions

There are several types of holes allowed to use in LRFD. The standard type of holes are the general hole type used in member to member connections. The oversized holes are allowed to be used in slip critical connections, but they shall not be used in bearing type of connections. The short-slotted holes are allowed for both slip critical and bearing type of connections. The direction of the slot is not important in slip-critical connection, but its length shall be normal to the direction of the load in bearing type of connections. Long-slotted holes are allowed in only one of the connected parts of either slip-critical or bearing type of connection at an individual joined surface. Its position with respect to the direction of loading is the same as described for short-slotted holes.

The maximum size of holes are given in Table 4.6.

Table 4.6 Nominal Hole Dimensions, mm

Bolt Diameter	Hole Dimensions			
	Standard	Oversize	Short-slotted	Long-slotted
M16	18	20	18 x 22	18 x 40
M20	22	24	22 x 26	22 x 50
M22	24	28	24 x 30	24 x 55
M24	27	30	27 x 32	27 x 60
M27	30	35	30 x 37	30 x 67
M30	33	38	33 x 40	33 x 75

4.2.2.9 Combined Shear and Tension

In case of connections having shear and tension simultaneously, the design strength of a bolt is given by,

$$\phi R_n = \phi \ F_t \ A_b \tag{4.20}$$

where $\phi = 0.75$

F_t = nominal tension stress, in MPa, as a function of f_v, the required shear stresses produced by the factored loads.

F_t for A325 bolts and where the threads included in the shear plane, is given by,

$$F_t = 807 - 2.5 \ f_v \leq 621 \tag{4.21}$$

F_t for A325 bolts and where the threads not included in the shear plane, is given by,

$$F_t = 807 - 2.0 \ f_v \leq 621 \tag{4.22}$$

F_t for A490 bolts and where the threads included in the shear plane, is given by,

$$F_t = 1010 - 2.5 \ f_v \leq 779 \tag{4.23}$$

F_t for A490 bolts and where the threads not included in the shear plane, is given by,

$$F_t = 1010 - 2.0\ f_v \leq 779 \qquad (4.24)$$

4.2.3 Welded Connections

Welding can be thought of as the fusing two pieces of metal together to form a continuous, rigid plate. Groove weld and fillet weld are common types used.

4.2.3.1 Groove Welds

The effective area of groove welds shall be considered as the effective length of the weld times the effective throat thickness. The effective length of a groove weld shall be the width of the part joined. The effective throat thickness of a complete-joint-penetration groove weld shall be the thickness of the thinner part joined. For the partial penetration groove weld, it equals to the depth of the chamber. Minimum throat thickness is 3 mm, where the thicker part is less than 6 mm, 5 mm for t in between 6 and 13 mm, 6 mm for t in between 13 and 19 mm, etc.

4.2.3.2 Fillet Welds

The effective throat thickness of a fillet weld shall be the shortest distance from the root of the joint to the face of the weld.

The minimum size of the fillet welds, defined as the fillet leg angle, a, is 3 mm for thickness up to 6 mm (thicker part), 5 mm for t in between 6 and 13 mm, 6 mm for t in between 13 and 19 mm, and 8 mm for t > 19 mm.

The minimum size of the fillet weld shall not be less than the required to transmit the calculated forces. The maximum size of the fillet weld shall not be greater than the thickness of the material along the edges of the material less than 6 mm thick, and it shall not be greater than the thickness of the material –2 mm along the edges of the material more than 6 mm thick.

4.2.3.3 Design Requirements

Groove Welds:

The design strength per unit length of the groove welds depends on the type of stress.

1. Tension and compression normal to the effective area and tension and compression parallel to the axis of weld:

$$\phi \, R_{nw} = 0.90 \;\; F_y \; t_e \quad \text{base material} \qquad (4.25)$$

$$\phi \, R_{nw} = 0.90 \;\; F_{yw} \; t_e \quad \text{weld metal} \qquad (4.26)$$

2. Shear on effective area:

$$\phi\, R_{nw} = 0.90\ \tau_y\ t_e \quad \text{base material} \qquad (4.27)$$

$$\phi\, R_{nw} = 0.80\ \tau_{yw}\ t_e \quad \text{weld metal} \qquad (4.28)$$

where

$\tau_y = 0.60\, F_y$

$\tau_{yw} = 0.60\, F_{EEX}$

where F_{EEX} is electrode tensile strength.

Fillet Welds:

The design strength per unit length of a fillet weld is based on the shear resistance through the throat of the weld as follows;

$$\phi\, R_{nw} = 0.75\ (\,0.60\, F_{EEX}\,)\ t_e \quad \text{fillet weld} \qquad (4.31)$$

but not less than the shear rupture strength of the adjacent base metal.

$$\phi\, R_{nw} = 0.75\ (\,0.60\, F_u\,)\ t \quad \text{base metal} \qquad (4.32)$$

t_e = effective throat thickness = 0.707a, t is the thickness of the plate.

CHAPTER 5

COMPARISON AMONG CODES TS 648, LRFD-AISC AND EUROCODE 3 PART 1.1 DESIGN AND PRESENTATION OF RESULTS

5.1 General

In this section, numerical examples of connections subjected to varying magnitude of load effects, such as shear and/or moment, are analyzed and designed according to the stipulations of the three codes, which are TS 648, LRFD-AISC, and Eurocode 3 Part 1.1 in order to make comparisons among these three codes.

The analysis is performed for four types of beam to column connections. They are,

- Hinge type bolted connections
- Rigid type bolted connections
- Hinge type welded connections
- Rigid type welded connections

The connections are analyzed under the effects of dead loads and live loads. The load combination considered in the design of connections by TS 648 is DL+LL, by LRFD-AISC is 1.2DL+1.6LL, and by Eurocode 3 is 1.35DL+1.50LL, where the notations DL and LL represent the dead load and live load, respectively. The analyses are performed considering different live load to dead load ratios starting from 1 with 25% increments up to live load to dead load ratio of 10.

Only elastic analysis is performed since TS 648 does not accept plastic design. The calculations are utilized by means of spread sheets to ease the work.

5.2 Hinge Typed Bolted Connections

This is a simple end beam connection where beam is connected from only its web to column, that means, bolts carry shear only (no or negligible moment). To obtain the connection, an equal angle is used which can be double or single. Due to the eccentricity of the shear force at the end of the beam with the neutral axis of bolt group nominal small moment occurs which is used for bolt design. This eccentricity causes eccentric shear in angle leg to beam web. Angle to column flange changes the behavior whether it is single or double such that if double angle is used, concentric shear occurs due to symmetry and if single angle is used, eccentric shear occurs due to eccentricity as mentioned above.

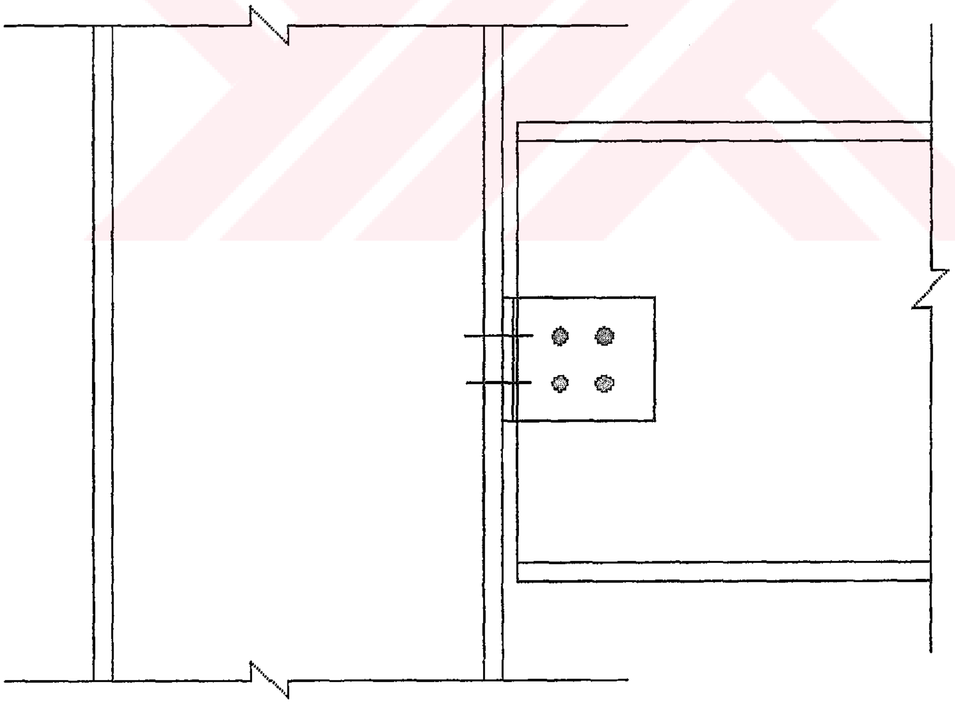

Figure-5.1

5.2.1 Definition of the Design Parameters

The following parameters are used in the design of hinge type of bolted connection.

Steel type : St 37, $f_u = 370\ \text{N/mm}^2$, $f_y = 240\ \text{N/mm}^2$,
Beam: I 400 $t_w = 14.4$ mm.
Column: HEA 400 $t_f = 19.0$ mm.
Double Angle: L 100.100.10 $t_a = 10.0$ mm.
Bolt: M16 Grade 10.9 with no pretension (SL type),
$f_{ub} = 1000\ \text{N/mm}^2$, $f_{yb} = 900\ \text{N/mm}^2$,
$\tau_{em} = 240\ \text{N/mm}^2$, $\sigma_{ez} = 280\ \text{N/mm}^2$,
$A = 201.1\ \text{mm}^2$, $A_s = 157\ \text{mm}^2$,
Loading : EY (Dead load + Live Load)
Assumptions: 1- shear in unthreaded portion of the bolt.
2- edge distance in the direction of loading, $e_1=2d_0$,
spacing in the direction of loading, $p_1=3d_0$.

Design Requirement:

Determination of number of bolts required for beam web-to-angle connection and for column flange-to-angle connection.

In the next part, the solutions procedures given by each code will be presented.

Solution by TS 648:

Beam web- to – angle connection;

Design shear resistance per bolt (Double Shear);

$F_{vb} = 2*A*\tau_{em} = 2*201.1*240 = 96528$ N = 96.5 kN.

Design bearing resistance per bolt ($t = t_{min} = 14.4$ mm.);

$F_{bb} = d*t_{min}\ \tau_{em} = 16*14.4*280 = 64512$ N = 64.5 kN.

Number of Bolts Required, n ;

$n = V_d / F_{bb}$

Column flange- to – angle connection;

Design shear resistance per two bolt, due to double angle arrangement (Single Shear);

$F_{vb} = 1*A*\tau_{em}*2 = 1*201.1*240*2 = 96528$ N = 96.5 kN.

Design bearing resistance per two bolt, due to double angle arrangement ($t = t_{min} = 10.0$ mm.);

$F_{bb} = d*t_{min}\ \tau_{em}*2 = 16*10.0*280*2 = 89600$ N = 89.6 kN.

Number of Bolts Required, n ;

$n = V_d / F_{bb}$

Solution by Eurocode 3:

Beam web- to – angle connection;

Design shear resistance per bolt (Double Shear);

$F_{vb} = 2*0.60*f_{ub}*A / \gamma_{Mb} = 2*0.6*1000*201.1/1.25 = 193.0$ kN.

Design bearing resistance per bolt ($t = t_{min} = 14.4$ mm.);

$\alpha = 0.75$ for $e_1 = 2d_0$

$F_{bb} = 2.5*\alpha*f_u*d*t / \gamma_{Mb} = 2.5*0.75*370*16*14.4/1.25 = 127.8$ kN.

Number of Bolts Required, n ;

$n = V_d / F_{bb}$

Column flange- to – angle connection;

Design shear resistance per two bolt, due to double angle arrangement (Single Shear);

$F_{vb} = 2*0.60*f_{ub}*A / \gamma_{Mb} = 2*0.6*1000*201.1/1.25 = 193.0$ kN.

Design bearing resistance per two bolt, due to double angle arrangement ($t = t_{min} = 10.0$ mm.);

$\alpha = 0.75$ for $e_1 = 2d_0$

$F_{bb} = 2*2.5*\alpha*f_u*d*t / \gamma_{Mb} = 2*2.5*0.75*370*16*10.0/1.25 = 177.6$ kN.

Number of Bolts Required, n ;

$n = V_d / F_{bb}$

Solution by LRFD-AISC:

Beam web- to – angle connection;

Design shear resistance per bolt (Double Shear);

$F_{vb} = 2*\phi*0.60*f_{ub}*A = 2*0.75*0.6*1000*201.1 = 181.0$ kN.

Design bearing resistance per bolt ($t = t_{min} = 14.4$ mm.);

$F_{bb} = \phi*2.4*f_u*d*t = 0.75*2.4*370*16*14.4 = 153.4$ kN.

Number of Bolts Required, n ;

$n = V_d / F_{bb}$

Column flange- to – angle connection;

Design shear resistance per two bolt, due to double angle arrangement (Single Shear);

$F_{vb} = 2*\phi*0.60*f_{ub}*A = 2*0.75*0.6*1000*201.1 = 181.0$ kN.

Design bearing resistance per two bolt, due to double angle arrangement ($t = t_{min}$ =10.0 mm.);

$F_{bb} = 2*\phi*2.4*f_u*d*t = 2*0.75*2.4*370*16*10.0 = 213.1$ kN.

Number of Bolts Required, n ;

$n = V_d / F_{bb}$

Solution Tables:

Solutions according to TS 648 are given in Table 5.1.

Solutions according to Eurocode 3 are given in Table 5.2.

Solutions according to LRFD are given in Table 5.3.

Summary of designs for each of codes are given in Table 5.4.

Notations used in Tables:

V_{DL} = unfactored dead load shear,

V_{LL} = unfactored live load shear,

LL/DL = live load to dead load ratio,

$V_{d,DL}$ = factored dead load shear,

$V_{d,LL}$ = factored live load shear,

V_d = total factored design shear.

Table 5.1 Hinged Type Bolted Connection Design According to TS648.

Load	V_{DL}	V_{LL}	LL/DD	$V_{d,DL}$	$V_{d,LL}$	$1.0V_{d,DL}+1.0V_{d,LL}$	Beam web-to Angle		Column Flange to Angle		No's of Bolts-Final Results	
Case	kN	kN		kN	kN	kN	F_{vb} (kN)	F_{bb} (kN)	F_{vc} (kN)	F_{bc} (kN)	Beam Web	Column Flange
							96.5	**64.5**	**96.5**	**89.6**	**n**	**2xn**
1	40	40	1	40	40	80	*2 (0.8)	2 (1.2)	*2 (0.8)	*2 (0.9)	2	2
2	40	40	1.25	40	50	90	*2 (0.9)	2 (1.4)	*2 (0.9)	2 (1.0)	2	2
3	40	40	1.5	40	60	100	2 (1.0)	2 (1.6)	2 (1.0)	2 (1.1)	2	2
4	40	40	1.75	40	70	110	2 (1.1)	2 (1.7)	2 (1.1)	2 (1.2)	2	2
5	40	40	2	40	80	120	2 (1.2)	2 (1.9)	2 (1.2)	2 (1.3)	2	2
6	40	40	2.25	40	90	130	2 (1.3)	3 (2.1)	2 (1.3)	2 (1.5)	3	2
7	40	40	2.5	40	100	140	2 (1.5)	3 (2.2)	2 (1.5)	2 (1.6)	3	2
8	40	40	2.75	40	110	150	2 (1.6)	3 (2.3)	2 (1.6)	2 (1.7)	3	2
9	40	40	3	40	120	160	2 (1.7)	3 (2.5)	2 (1.7)	2 (1.8)	3	2
10	40	40	3.25	40	130	170	2 (1.8)	3 (2.6)	2 (1.8)	2 (1.9)	3	2
11	40	40	3.5	40	140	180	2 (1.9)	3 (2.8)	2 (1.9)	3 (2.1)	3	3
12	40	40	3.75	40	150	190	2 (2.0)	3 (2.9)	2 (2.0)	3 (2.2)	3	3
13	40	40	4	40	160	200	3 (2.1)	4 (3.1)	3 (2.1)	3 (2.5)	4	3
14	40	40	4.5	40	180	220	3 (2.3)	4 (3.4)	3 (2.3)	3 (2.7)	4	3
15	40	40	5	40	200	240	3 (2.5)	4 (3.7)	3 (2.5)	3 (2.8)	4	3
16	40	40	5.5	40	220	260	3 (2.7)	5 (4.1)	3 (2.7)	3 (2.9)	5	3
17	40	40	6	40	240	280	3 (2.9)	5 (4.3)	3 (2.9)	4 (3.1)	5	4
18	40	40	7	40	280	320	4 (3.3)	5 (5.0)	4 (3.3)	4 (3.6)	5	4
19	40	40	8	40	320	360	4 (3.7)	6 (5.6)	4 (3.7)	5 (4.1)	6	5
20	40	40	9	40	360	400	5 (4.1)	7 (6.2)	5 (4.1)	5 (4.5)	7	5
21	40	40	10	40	400	440	5 (4.6)	7 (6.8)	5 (4.6)	5 (4.9)	7	5

Note : 1- The values in paranthesis show the theoretical values of bolt numbers resulting from the calculations.
2- "*2" means that the minimum bolt number, 2, has to be used in the connections.

Table 5.2 Hinged Type Bolted Connection Design According to Eurocode 3 Part 1.1.

Load	V_{DL}	V_{LL}	LL/DD	$V_{d,DL}$	$V_{d,LL}$	$1.35V_{d,DL}+1.5V_{d,LL}$	Beam web-to Angle		Column Flange to Angle		No's of Bolts-Final Results	
Case	kN	kN		kN	kN	kN	F_{vb} (kN)	F_{bb} (kN)	F_{vc} (kN)	F_{bc} (kN)	Beam Web	Column Flange
							193	127.8	193	177.6	n	2xn
1	40	40	1	54	60	114	*2 (0.6)	*2 (0.9)	*2 (0.6)	*2 (0.6)	2	2
2	40	40	1.25	54	75	129	*2 (0.7)	2 (1.0)	*2 (0.7)	*2 (0.7)	2	2
3	40	40	1.5	54	90	144	*2 (0.7)	2 (1.1)	*2 (0.7)	*2 (0.8)	2	2
4	40	40	1.75	54	105	159	*2 (0.8)	2 (1.2)	*2 (0.8)	*2 (0.9)	2	2
5	40	40	2	54	120	174	*2 (0.9)	2 (1.4)	*2 (0.9)	*2 (1.0)	2	2
6	40	40	2.25	54	135	189	*2 (1.0)	2 (1.5)	*2 (1.0)	2 (1.1)	2	2
7	40	40	2.5	54	150	204	2 (1.1)	2 (1.6)	2 (1.1)	2 (1.1)	2	2
8	40	40	2.75	54	165	219	2 (1.1)	2 (1.7)	2 (1.1)	2 (1.2)	2	2
9	40	40	3	54	180	234	2 (1.2)	2 (1.8)	2 (1.2)	2 (1.3)	2	2
10	40	40	3.25	54	195	249	2 (1.3)	2 (1.9)	2 (1.3)	2 (1.4)	2	2
11	40	40	3.5	54	210	264	2 (1.4)	3 (2.1)	2 (1.4)	2 (1.5)	3	2
12	40	40	3.75	54	225	279	2 (1.4)	3 (2.2)	2 (1.4)	2 (1.6)	3	2
13	40	40	4	54	240	294	2 (1.5)	3 (2.3)	2 (1.5)	2 (1.7)	3	2
14	40	40	4.5	54	270	324	2 (1.7)	3 (2.5)	2 (1.7)	2 (1.8)	3	2
15	40	40	5	54	300	354	2 (1.8)	3 (2.8)	2 (1.8)	2 (2.0)	3	2
16	40	40	5.5	54	330	384	2 (2.0)	4 (3.0)	2 (2.0)	3 (2.2)	4	3
17	40	40	6	54	360	414	3 (2.1)	4 (3.2)	3 (2.1)	3 (2.3)	4	3
18	40	40	7	54	420	474	3 (2.5)	4 (3.7)	3 (2.5)	3 (2.7)	4	3
19	40	40	8	54	480	534	3 (2.8)	5 (4.2)	3 (2.8)	4 (3.0)	5	4
20	40	40	9	54	540	594	4 (3.1)	5 (4.6)	4 (3.1)	4 (3.3)	5	4
21	40	40	10	54	600	654	4 (3.4)	6 (5.1)	4 (3.4)	4 (3.7)	6	4

Note : 1- The values in paranthesis show the theoretical values of bolt numbers resulting from the calculations.
2- "*2" means that the minimum bolt number, 2, has to be used in the connections.

Table 5.3 Hinged Type Bolted Connection Design According to LRFD.

Load Case	V_{DL} kN	V_{LL} kN	LL/DD	$V_{d,DL}$ kN	$V_{d,LL}$ kN	$1.2V_{d,DL}+1.6V_{d,LL}$ kN	Beam web-to Angle F_{vb} (kN)	Beam web-to Angle F_{bb} (kN)	Column Flange to Angle F_{vc} (kN)	Column Flange to Angle F_{bc} (kN)	No's of Bolts-Final Results Beam Web	No's of Bolts-Final Results Column Flange
							181	153.4	181	213.1	n	2xn
1	40	40	1	48	64	112	*2 (0.6)	*2 (0.6)	*2 (0.6)	*2 (0.5)	2	2
2	40	40	1.25	48	80	128	*2 (0.7)	*2 (0.7)	*2 (0.7)	*2 (0.6)	2	2
3	40	40	1.5	48	96	144	*2 (0.8)	*2 (0.8)	*2 (0.8)	*2 (0.7)	2	2
4	40	40	1.75	48	112	160	*2 (0.9)	2 (1.1)	*2 (0.9)	*2 (0.8)	2	2
5	40	40	2	48	128	176	*2 (1.0)	2 (1.1)	*2 (1.0)	*2 (0.8)	2	2
6	40	40	2.25	48	144	192	2 (1.1)	2 (1.3)	2 (1.1)	*2 (0.9)	2	2
7	40	40	2.5	48	160	208	2 (1.1)	2 (1.4)	2 (1.1)	*2 (1.0)	2	2
8	40	40	2.75	48	176	224	2 (1.2)	2 (1.5)	2 (1.2)	2 (1.1)	2	2
9	40	40	3	48	192	240	2 (1.3)	2 (1.6)	2 (1.3)	2 (1.1)	2	2
10	40	40	3.25	48	208	256	2 (1.4)	2 (1.7)	2 (1.4)	2 (1.2)	2	2
11	40	40	3.5	48	224	272	2 (1.5)	2 (1.8)	2 (1.5)	2 (1.3)	2	2
12	40	40	3.75	48	240	288	2 (1.6)	2 (1.9)	2 (1.6)	2 (1.4)	2	2
13	40	40	4	48	256	304	2 (1.7)	2 (2.0)	2 (1.7)	2 (1.4)	2	2
14	40	40	4.5	48	288	336	2 (1.9)	3 (2.2)	2 (1.9)	2 (1.6)	3	2
15	40	40	5	48	320	368	3 (2.0)	3 (2.4)	3 (2.0)	2 (1.7)	3	3
16	40	40	5.5	48	352	400	3 (2.2)	3 (2.6)	3 (2.2)	2 (1.9)	3	3
17	40	40	6	48	384	432	3 (2.4)	3 (2.8)	3 (2.4)	3 (2.0)	3	3
18	40	40	7	48	448	496	3 (2.7)	4 (3.2)	3 (2.7)	3 (2.3)	4	3
19	40	40	8	48	512	560	4 (3.1)	4 (3.7)	4 (3.1)	3 (2.6)	4	4
20	40	40	9	48	576	624	4 (3.4)	5 (4.1)	4 (3.4)	3 (2.9)	5	4
21	40	40	10	48	640	688	4 (3.8)	5 (4.5)	4 (3.8)	4 (3.2)	5	4

Note : 1- The values in paranthesis show the theoretical values of bolt numbers resulting from the calculations.
2- "*2" means that the minimum bolt number, 2, has to be used in the connections.

Table 5.4 Hinged Type Bolted Connection Design_SUMMARY.

Load	V_{DL}	V_{LL}	LL/DD	TS648			Eurocode 3			LRFD		
Case	kN	kN		V_d (kN)	Bolt No's Beam	Bolt No's Column	V_d (kN)	Bolt No's Beam	Bolt No's Column	V_d (kN)	Bolt No's Beam	Bolt No's Column
1	40	40	1	80	2	2	114	2	2	112	2	2
2	40	40	1.25	90	2	2	129	2	2	128	2	2
3	40	40	1.5	100	2	2	144	2	2	144	2	2
4	40	40	1.75	110	2	2	159	2	2	160	2	2
5	40	40	2	120	2	2	174	2	2	176	2	2
6	40	40	2.25	130	3	2	189	2	2	192	2	2
7	40	40	2.5	140	3	2	204	2	2	208	2	2
8	40	40	2.75	150	3	2	219	2	2	224	2	2
9	40	40	3	160	3	2	234	2	2	240	2	2
10	40	40	3.25	170	3	2	249	2	2	256	2	2
11	40	40	3.5	180	3	3	264	3	2	272	2	2
12	40	40	3.75	190	3	3	279	3	2	288	2	2
13	40	40	4	200	4	3	294	3	2	304	2	2
14	40	40	4.5	220	4	3	324	3	2	336	3	2
15	40	40	5	240	4	3	354	3	2	368	3	3
16	40	40	5.5	260	5	3	384	4	3	400	3	3
17	40	40	6	280	5	4	414	4	3	432	3	3
18	40	40	7	320	5	4	474	4	3	496	4	3
19	40	40	8	360	6	5	534	5	4	560	4	4
20	40	40	9	400	7	5	594	5	4	624	5	4
21	40	40	10	440	7	5	654	6	4	688	5	4

5.3 Rigid Type Bolted Connection

This is a rigid end beam connection where beam is connected from both its web and flange to column, that means, bolts carry not only shear but also moment together with axial load. For this connection as discussed at hinge type an equal angle for web connection and a T-shape angle which can be either half of a rolled section (1/2I, 1/2IPE, 1/2IPBv etc.) or T-angle. Web connection is totally same as the hinge connection, therefore, the web design will not be repeated here. Flange connection also has two different behavior. One is at the angle to beam flange which is direct or concentric shear and the other is at the angle leg to column flange which is direct tension.

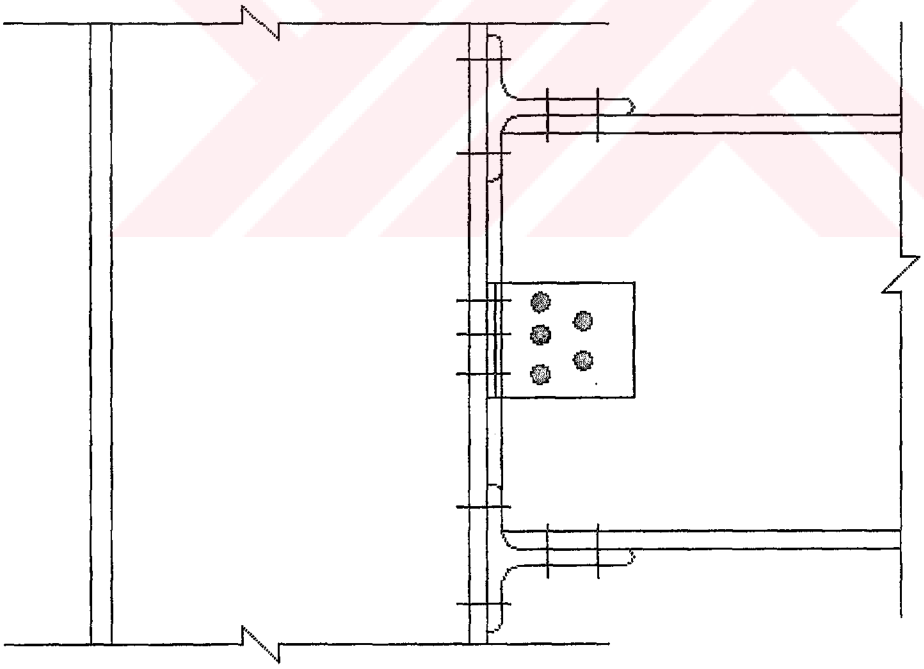

Figure-5.2

5.3.1 Definition of the Design Parameters

The following parameters are used in the design of rigid type of bolted connection.

Steel type : St 37, $f_u = 370$ N/mm^2, $f_y = 240$ N/mm^2,
Beam: I 400 $t_f = 21.6$ mm.
Column: HEA 400 $t_f = 19.0$ mm.
Tee Angle: ½ I 240 $t_f = 13.1$ mm. $t_w = 8.7$ mm.
Bolt: M16 Grade 10.9 with no pretension (SL type),
$f_{ub} = 1000$ N/mm^2, $f_{yb} = 900$ N/mm^2,
$\tau_{em} = 240$ N/mm^2, $\sigma_{ez} = 280$ N/mm^2, $F_{t,all} = 56.5$ kN,
$A = 201.1$ mm^2, $A_s = 157$ mm^2,
Loading : EY (Dead load + Live Load)
Assumptions: 1- shear in unthreaded portion of the bolt.
2- edge distance in the direction of loading, $e_1=2d_0$,
spacing in the direction of loading, $p_1=3d_0$.

Design Requirement:

Determination of number of bolts required for beam flange-to-angle connection and for column flange-to-angle connection due to tension resulting from moment, $T_d = M_d / (0.4087)$.

In the next part, the solutions procedures given by each code will be presented.

Solution by TS 648:

Beam flange- to – angle connection;

Design shear resistance per bolt (Single Shear);

$F_{vb} = 1*A*\tau_{em} = 1*201.1*240 = 48.2$ kN.

Design bearing resistance per bolt ($t = t_w = 8.7$ mm.);

$F_{bb} = d*t_{min}\ \tau_{em} = 16*8.7*280 = 38.9$ kN.

Number of Bolts Required, n ;

$n = T_d / F_{bb}$

Column flange- to – angle connection;

Design tension resistance per bolt;

$F_{t,all} = 56.5$ kN for M16 bolts.

Number of Bolts Required, n ;

$n = T_d / F_{t,all}$

Solution by Eurocode 3:

Beam web- to – angle connection;

Design shear resistance per bolt (Single Shear);

$F_{vb} = 1*0.60*f_{ub}*A / \gamma_{Mb} = 1*0.6*1000*201.1/1.25 = 96.5$ kN.

Design bearing resistance per bolt ($t = t_w = 8.7$ mm.);

$\alpha = 0.75$ for $e_1 = 2d_0$

$F_{bb} = 2.5*\alpha*f_u*d*t / \gamma_{Mb} = 2.5*0.75*370*16*8.7/1.25 = 77.2$ kN.

Number of Bolts Required, n ;

$n = T_d / F_{bb}$

Column flange- to – angle connection;

Design tension resistance per bolt;

$F_{t,all} = 0.90 * f_{ub} * A_s / \gamma_{Mb} = 0.9*1000*157/1.25 = 113.0$ kN.

Number of Bolts Required, n ;

$n = T_d / F_{t,all}$

Solution by LRFD-AISC:

Beam web- to – angle connection;

Design shear resistance per bolt (Single Shear);

$F_{vb} = 1*\phi*0.60*f_{ub}*A = 1*0.75*0.6*1000*201.1 = 90.4$ kN.

Design bearing resistance per bolt ($t = t_w = 8.7$ mm.);

$F_{bb} = \phi*2.4*f_u*d*t = 0.75*2.4*370*16*8.7 = 92.7$ kN.

Number of Bolts Required, n ;

$n = V_d / F_{vb}$

Column flange- to – angle connection;

Design tension resistance per bolt;

$F_{t,all} = \phi*f_{ub}*A_s = 0.75*1000*157 = 117.8$ kN.

Number of Bolts Required, n ;

$n = T_d / F_{t,all}$

Solution Tables:

Solutions according to TS 648 are given in Table 5.5.

Solutions according to Eurocode 3 are given in Table 5.6.

Solutions according to LRFD are given in Table 5.7.

Summary of designs for each of codes are given in Table 5.8.

Table 5.5 Rigid Type Bolted Connection Design According to TS648.

$1.0V_{d,DL}+1.0V_{d,LL}$	$1.0M_{d,DL}+1.0M_{d,LL}$		Column Flange			Number of Bolts		Beam Flange			Number of Bolts
V_d	M_d		T_d	$F_{t,all}$	n	Provided		T_d	F_{bb}	n	Provided
kN	kN.m		kN	kN		top/bottom		kN	kN		left/right
80	20	Bolts Reuired for Column to T-Angle Connection	48.9	56.5	*2 (0.9)	2X1	Bolts Reuired for Beam to T-Angle Connection	48.9	38.9	2 (1.3)	2x1
90	22.5		55.1	56.5	*2 (1.0)	2X1		55.1	38.9	2 (1.4)	2x1
100	25		61.2	56.5	2 (1.1)	2X1		61.2	38.9	2 (1.6)	2x1
110	27.5		67.3	56.5	2 (1.2)	2X1		67.3	38.9	2 (1.7)	2x1
120	30		73.4	56.5	2 (1.3)	2X1		73.4	38.9	2 (1.9)	2x1
130	32.5		79.5	56.5	2 (1.4)	2X1		79.5	38.9	3 (2.1)	2x2
140	35		85.6	56.5	2 (1.5)	2X1		85.6	38.9	3 (2.2)	2x2
150	37.5		91.8	56.5	2 (1.6)	2X1		91.8	38.9	3 (2.4)	2x2
160	40		97.9	56.5	2 (1.7)	2X1		97.9	38.9	3 (2.5)	2x2
170	42.5		104.0	56.5	2 (1.8)	2X1		104.0	38.9	3 (2.7)	2x2
180	45		110.1	56.5	2 (1.9)	2X1		110.1	38.9	3 (2.8)	2x2
190	47.5		116.2	56.5	3 (2.1)	2X2		116.2	38.9	3 (3.0)	2x2
200	50		122.3	56.5	3 (2.2)	2X2		122.3	38.9	4 (3.1)	2x2
220	55		134.6	56.5	3 (2.4)	2X2		134.6	38.9	4 (3.5)	2x2
240	60		146.8	56.5	3 (2.6)	2X2		146.8	38.9	4 (3.8)	2x2
260	65		159.0	56.5	3 (2.8)	2X2		159.0	38.9	5 (4.1)	3x2
280	70		171.3	56.5	4 (3.1)	2X2		171.3	38.9	5 (4.4)	3x2
320	80		195.7	56.5	4 (3.5)	2X2		195.7	38.9	6 (5.0)	3x2
360	90		220.2	56.5	4 (3.9)	2X2		220.2	38.9	6 (5.7)	3x2
400	100		244.7	56.5	5 (4.3)	2X3		244.7	38.9	7 (6.3)	4x2
440	110		269.1	56.5	5 (4.8)	2X3		269.1	38.9	7 (6.9)	4x2

Note : 1- The values in paranthesis show the theoretical values of bolt numbers resulting from the calculations.
2- "*2" means that the minimum bolt number, 2, has to be used in the connections.

Table 5.6 Rigid Type Bolted Connection Design According to Eurocode 3 Part 1.1.

$1.35V_{d,DL}+1.5V_{d,LL}$	$1.35M_{d,DL}+1.5M_{d,LL}$		Column Flange			Number of Bolts		Beam Flange			Number of Bolts
V_d	M_d		T_d	$F_{t,all}$	n	Provided		T_d	F_{bb}	n	Provided
kN	kN.m		kN	kN		top/bottom		kN	kN		left/right
114	28.5	Bolts Reuired for Column to T-Angle Connection	69.7	113	***2 (0.6)**	2X1	Bolts Reuired for Beam to T-Angle Connection	69.7	77.2	***2 (0.9)**	2x1
129	32.25		78.9	113	***2 (0.7)**	2X1		78.9	77.2	**2 (1.0)**	2x1
144	36		88.1	113	***2 (0.8)**	2X1		88.1	77.2	**2 (1.1)**	2x1
159	39.75		97.3	113	***2 (0.9)**	2X1		97.3	77.2	**2 (1.3)**	2x1
174	43.5		106.4	113	***2 (0.9)**	2X1		106.4	77.2	**2 (1.4)**	2x1
189	47.25		115.6	113	**2 (1.0)**	2X1		115.6	77.2	**2 (1.5)**	2x1
204	51		124.8	113	**2 (1.1)**	2X1		124.8	77.2	**2 (1.6)**	2x1
219	54.75		134.0	113	**2 (1.2)**	2X1		134.0	77.2	**2 (1.7)**	2x1
234	58.5		143.1	113	**2 (1.3)**	2X1		143.1	77.2	**2 (1.9)**	2x1
249	62.25		152.3	113	**2 (1.3)**	2X1		152.3	77.2	**2 (2.0)**	2x2
264	66		161.5	113	**2 (1.4)**	2X1		161.5	77.2	**3 (2.1)**	2x2
279	69.75		170.7	113	**2 (1.5)**	2X1		170.7	77.2	**3 (2.2)**	2x2
294	73.5		179.8	113	**2 (1.6)**	2X1		179.8	77.2	**3 (2.3)**	2x2
324	81		198.2	113	**2 (1.8)**	2X1		198.2	77.2	**3 (2.6)**	2x2
354	88.5		216.5	113	**2 (1.9)**	2X1		216.5	77.2	**3 (2.8)**	2x2
384	96		234.9	113	**3 (2.1)**	2X2		234.9	77.2	**4 (3.1)**	2x2
414	103.5		253.2	113	**3 (2.2)**	2X2		253.2	77.2	**4 (3.3)**	2x2
474	118.5		289.9	113	**3 (2.6)**	2X2		289.9	77.2	**4 (3.8)**	2x2
534	133.5		326.6	113	**3 (2.9)**	2X2		326.6	77.2	**5 (4.2)**	3x2
594	148.5		363.3	113	**4 (3.2)**	2X2		363.3	77.2	**5 (4.7)**	3x2
654	163.5		400.0	113	**4 (3.5)**	2X2		400.0	77.2	**6 (5.2)**	3x2

Note : 1- The values in paranthesis show the theoretical values of bolt numbers resulting from the calculations.
2- "*2" means that the minimum bolt number, 2, has to be used in the connections.

Table 5.7 Rigid Type Bolted Connection Design According to LRFD.

$1.2V_{d,DL}+1.6V_{d,LL}$	$1.2M_{d,DL}+1.6M_{d,LL}$		Column Flange			Number of Bolts		Beam Flange			Number of Bolts
V_d	M_d		T_d	$F_{t,all}$	n	Provided		T_d	F_{vb}	n	Provided
kN	kN.m		kN	kN		top/bottom		kN	kN		left/right
112	28	Bolts Reuired for Column to T-Angle Connection	68.5	117.8	***2 (0.6)**	2X1	Bolts Reuired for Beam to T-Angle Connection	68.5	90.5	***2 (0.8)**	2x1
128	32		78.3	117.8	***2 (0.7)**	2X1		78.3	90.5	***2 (0.9)**	2x1
144	36		88.1	117.8	***2 (0.7)**	2X1		88.1	90.5	***2 (1.0)**	2x1
160	40		97.9	117.8	***2 (0.8)**	2X1		97.9	90.5	**2 (1.1)**	2x1
176	44		107.7	117.8	***2 (0.9)**	2X1		107.7	90.5	**2 (1.2)**	2x1
192	48		117.4	117.8	***2 (1.0)**	2X1		117.4	90.5	**2 (1.3)**	2x1
208	52		127.2	117.8	**2 (1.1)**	2X1		127.2	90.5	**2 (1.4)**	2x1
224	56		137.0	117.8	**2 (1.2)**	2X1		137.0	90.5	**2 (1.5)**	2x1
240	60		146.8	117.8	**2 (1.2)**	2X1		146.8	90.5	**2 (1.6)**	2x1
256	64		156.6	117.8	**2 (1.3)**	2X1		156.6	90.5	**2 (1.7)**	2x2
272	68		166.4	117.8	**2 (1.4)**	2X1		166.4	90.5	**2 (1.8)**	2x2
288	72		176.2	117.8	**2 (1.5)**	2X1		176.2	90.5	**2 (1.9)**	2x2
304	76		186.0	117.8	**2 (1.6)**	2X1		186.0	90.5	**3 (2.1)**	2x2
336	84		205.5	117.8	**2 (1.7)**	2X1		205.5	90.5	**3 (2.3)**	2x2
368	92		225.1	117.8	**2 (1.9)**	2X1		225.1	90.5	**3 (2.5)**	2x2
400	100		244.7	117.8	**3 (2.1)**	2X2		244.7	90.5	**3 (2.7)**	2x2
432	108		264.3	117.8	**3 (2.2)**	2X2		264.3	90.5	**3 (2.9)**	2x2
496	124		303.4	117.8	**3 (2.6)**	2X2		303.4	90.5	**4 (3.4)**	2x2
560	140		342.5	117.8	**3 (2.9)**	2X2		342.5	90.5	**4 (3.8)**	2X2
624	156		381.7	117.8	**4 (3.2)**	2X2		381.7	90.5	**5 (4.2)**	3x2
688	172		420.8	117.8	**4 (3.6)**	2X2		420.8	90.5	**5 (4.7)**	3x2

Note : 1- The values in paranthesis show the theoretical values of bolt numbers resulting from the calculations.
2- "*2" means that the minimum bolt number, 2, has to be used in the connections.

Table 5.8 Rigid Type Bolted Connection Design_SUMMARY

	TS648		Eurocode 3		LRFD		Number of M16 Bolts Required					
	1.0DL+1.0LL	1.0DL+1.0LL	1.35DL+1.5LL	1.35DL+1.5LL	1.2DL+1.6LL	1.2DL+1.6LL	TS648		Eurocode		LRFD	
	V_d (kN)	M_d (kN.m)	V_d (kN)	M_d (kN.m)	V_d (kN)	M_d (kN.m)	beam	colmn.	beam	colmn.	beam	colmn.
1	80	20	114	28.5	112	28	2	2	2	2	2	2
2	90	22.5	129	32.25	128	32	2	2	2	2	2	2
3	100	25	144	36	144	36	2	2	2	2	2	2
4	110	27.5	159	39.75	160	40	2	2	2	2	2	2
5	120	30	174	43.5	176	44	2	2	2	2	2	2
6	130	32.5	189	47.25	192	48	3	2	2	2	2	2
7	140	35	204	51	208	52	3	2	2	2	2	2
8	150	37.5	219	54.75	224	56	3	2	2	2	2	2
9	160	40	234	58.5	240	60	3	2	2	2	2	2
10	170	42.5	249	62.25	256	64	3	2	2	2	2	2
11	180	45	264	66	272	68	3	2	3	2	2	2
12	190	47.5	279	69.75	288	72	3	3	3	2	2	2
13	200	50	294	73.5	304	76	4	3	3	2	3	2
14	220	55	324	81	336	84	4	3	3	2	3	2
15	240	60	354	88.5	368	92	4	3	3	2	3	2
16	260	65	384	96	400	100	5	3	4	3	3	3
17	280	70	414	103.5	432	108	5	4	4	3	3	3
18	320	80	474	118.5	496	124	6	4	4	3	4	3
19	360	90	534	133.5	560	140	6	4	5	3	4	3
20	400	100	594	148.5	624	156	7	5	5	4	5	4
21	440	110	654	163.5	688	172	7	5	6	4	5	4

5.4 Hinge Type Welded Connection

As it was discussed for the hinge type bolted connection, connection of this type only carries shear, no moment. Web connection is provided with fillet welds. Equal angle also has the option to be double or single. In the examples, the double angle is selected.

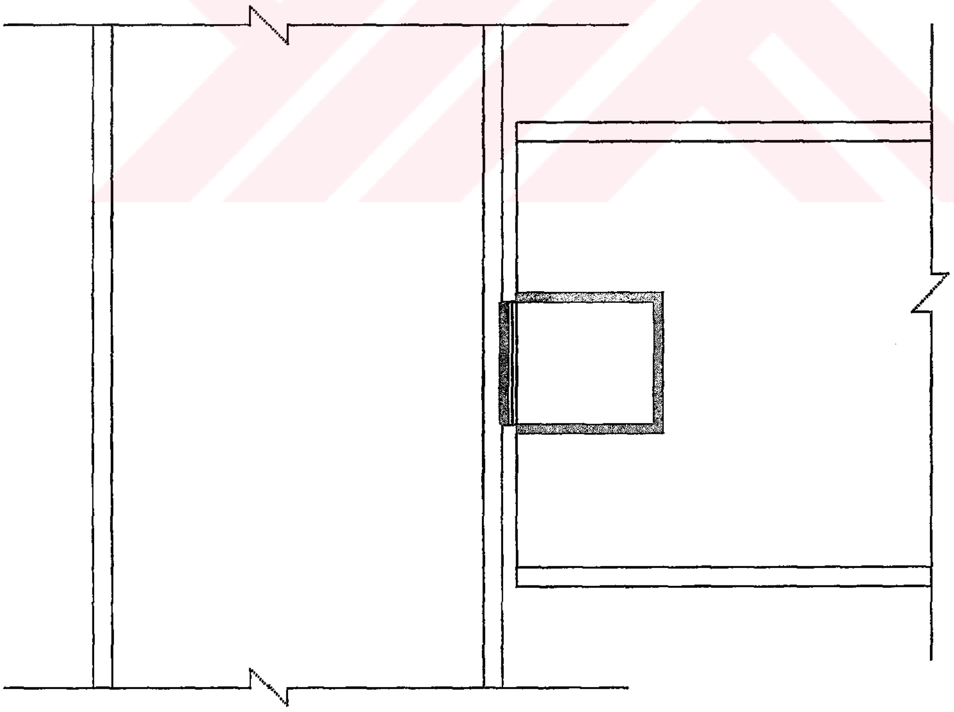

Figure-5.3

5.4.1 Definition of the Design Parameters

The following parameters are used in the design of hinge type of welded connection.

Steel type : St 37, $f_u = 370$ N/mm^2, $f_y = 240$ N/mm^2,

Beam: I 400 $t_w = 14.4$ mm.

Column: HEA 400 $t_f = 19.0$ mm.

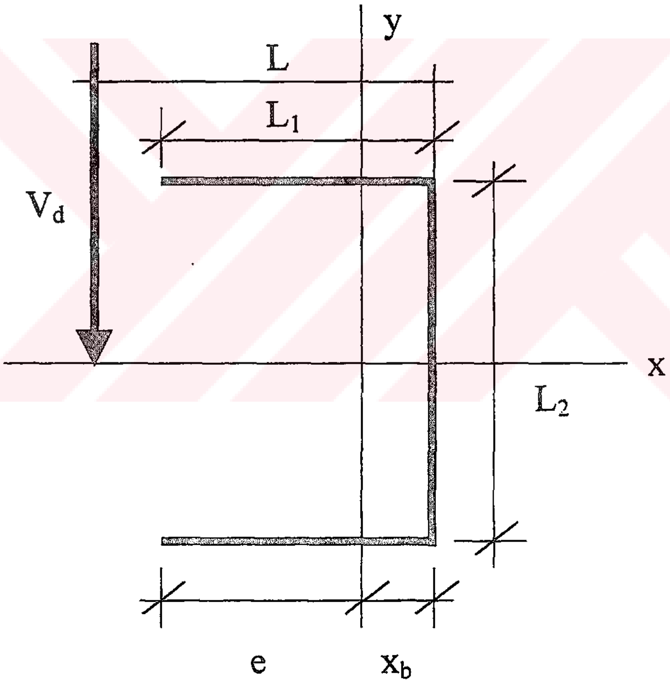

Figure- 5.4

Solution Procedure:

According to Figure-5.4, the calculation steps are as follows;

$x_b = L_1^2 / (2*L_1+L_2)$

$e = L - x_b$

$V= V_d / 2 \quad M = V.e$

$A_1= a_w*L_1 \qquad A_2 = a_w*L_2 \quad a_w$ = throat thickness.

$I_x = 2*A_1*(L_2/2)^2 + (1/12)*a_w*L_2^3$

$I_y = 2*(1/12)*a_w*L_1^3+2*a_w*L_1*(L_1/2-x_b)^2+a_w*L_2*x_b^2$

$I_z = I_x + I_y$

$\tau' = V / (2*A_1+A_2)$

$\tau''_{11} = M*y / I_z = M*(L_2/2) / I_z$

$\tau'' = M*x / I_z = M*(L_1-e) / I_z$

$\tau_{11} = \tau''_{11}$

$\tau = \tau' + \tau''$

$\sigma_v = (\tau_{11}^2 + \tau^2)^{0.5}$

Design Requirement:

Determination of weld length required to carry the applied shear load.

In the next part, the weld resistances given by each code will be presented.

Solution by TS 648:

Design weld resistance;

$\tau_{all} = 1.1 \text{ t/cm}^2$

Solution by Eurocode 3:

Design weld resistance per unit weld length;

$\tau_{all} = a_w * f_u / (\sqrt{3} * \beta\omega * \gamma_{Mw}) = 213.6\, a_w$ N/mm

for $a_w = 5$mm, $\tau_{all} = 1068$ N/mm

for $a_w = 6$mm, $\tau_{all} = 1281$ N/mm

for $a_w = 7$mm, $\tau_{all} = 1495$ N/mm

Solution by LRFD:

Design weld resistance per unit weld length;

$\tau_{all} = \phi * a_w * 0.60 * f_u = 0.75 * a_w * 0.60 * 370 = 166.5\, a_w$ N/mm

for $a_w = 5$mm, $\tau_{all} = 832.5$ N/mm

Solution Tables:

Solutions according to TS 648 are given in Table 5.9.

Solutions according to Eurocode 3 are given in Table 5.10.

Solutions according to LRFD are given in Table 5.11.

Summary of designs for each of codes are given in Table 5.12.

Notations used in Tables:

V_d = total factored design shear.

a_w = throat thickness

L_T = total length of weld required.

σ_v = design shear acting on the weld group.

Table 5.9 Hinged Type Welded Connection Design According to TS648.

	V_d (kN)	aw (cm)	L_1 (cm)	L_2 (cm)	L_T (cm)	σ_v (t/cm2)	τ_{all} (t/cm2)	ANGLE TYPE (2x)
1	80	0.3	4	5	**26**	1.03	1.1	**50.5**
2	90	0.3	4	6	**28**	1.08	1.1	**50.5**
3	100	0.3	4	8	**32**	1.06	1.1	**50.5**
4	110	0.3	4	9	**34**	1.09	1.1	**50.5**
5	120	0.3	5	9	**38**	107	1.1	**60.6**
6	130	0.5	4	5	**26**	1.02	1.1	**80.8**
7	140	0.5	4.5	5	**28**	1.03	1.1	**80.8**
8	150	0.5	5	5	**30**	1.03	1.1	**80.8**
9	160	0.5	5.5	5	**32**	1.03	1.1	**80.8**
10	170	0.5	5.5	5.5	**33**	1.07	1.1	**80.8**
11	180	0.5	6	6	**36**	1.04	1.1	**80.8**
12	190	0.5	6	6.5	**37**	1.07	1.1	**80.8**
13	200	0.5	6	7.5	**39**	1.07	1.1	**80.8**
14	220	0.5	7	8	**44**	1.07	1.1	**80.8**
15	240	0.5	7	9.5	**47**	1.06	1.1	**80.8**
16	260	0.5	8	10	**52**	1.09	1.1	**90.9**
17	280	0.5	8	12.5	**57**	1.09	1.1	**90.9**
18	320	0.6	9	10	**56**	1.09	1.1	**100.10**
19	360	0.6	10	15	**70**	1.08	1.1	**120.10**
20	400	0.7	10	15	**70**	1.08	1.1	**120.10**
21	440	0.8	10	15	**70**	1.08	1.1	**120.10**

Table 5.10 Hinged Type Welded Connection Design According to Eurocode 3.

	V_d (kN)	aw (cm)	L_1 (cm)	L_2 (cm)	L_T (cm)	σ_v (N/mm)	τ_{all} (N/mm)	ANGLE TYPE (2x)
1	114	0.5	4	6.5	**29**	1034	1068	**50.5**
2	129	0.5	4	7	**30**	1068	1068	**50.5**
3	144	0.5	4	8	**32**	1063	1068	**50.5**
4	159	0.5	4	9	**34**	1038	1068	**50.5**
5	174	0.5	4	10	**36**	1017	1068	**50.5**
6	189	0.5	5	10.5	**41**	1028	1068	**60.6**
7	204	0.5	6.5	10.5	**47**	1068	1068	**60.6**
8	219	0.5	6.5	11.5	**49**	1052	1068	**60.6**
9	234	0.5	6.5	12.5	**51**	1036	1068	**60.6**
10	249	0.5	6.5	13	**52**	1060	1068	**60.6**
11	264	0.5	7	14	**56**	1036	1068	**80.8**
12	279	0.5	7	14.5	**57**	1057	1068	**80.8**
13	294	0.5	8	15	**62**	1063	1068	**90.9**
14	324	0.5	8	16.5	**65**	1066	1068	**90.9**
15	354	0.5	9	18	**72**	1057	1068	**100.10**
16	384	0.5	9	19.5	**75**	1059	1068	**100.10**
17	414	0.5	9	21	**78**	1060	1068	**100.10**
18	474	0.6	9	20	**76**	1275	1281	**100.10**
19	534	0.6	11	22	**88**	1281	1281	**120.10**
20	594	0.7	11	22	**88**	1431	1495	**120.10**
21	654	0.7	11	23.5	**91**	1476	1495	**120.10**

Table 5.11 Hinged Type Welded Connection Design According to LRFD.

	V_d (kN)	aw (cm)	L_1 (cm)	L_2 (cm)	L_T (cm)	σ_v (N/mm)	τ_{all} (N/mm)	ANGLE TYPE (2x)
1	112	0.5	4	8	**32**	826	833	**50.5**
2	128	0.5	4	9	**34**	833	833	**50.5**
3	144	0.5	5	10	**40**	822	833	**60.6**
4	160	0.5	5	11	**42**	829	833	**60.6**
5	176	0.5	5	12	**44**	833	833	**60.6**
6	192	0.5	7	13	**54**	810	833	**80.8**
7	208	0.5	7	14	**56**	817	833	**80.8**
8	224	0.5	7	15	**58**	821	833	**80.8**
9	240	0.5	8	16	**64**	814	833	**90.9**
10	256	0.5	8	17	**66**	818	833	**90.9**
11	272	0.5	8	18	**68**	820	833	**90.9**
12	288	0.5	9	19	**74**	815	833	**100.10**
13	304	0.5	9	20	**76**	818	833	**100.10**
14	336	0.5	9	22	**80**	821	833	**100.10**
15	368	0.5	11	23.5	**91**	831	833	**120.10**
16	400	0.6	11	22	**88**	963	999	**120.10**
17	432	0.6	11	23	**90**	996	999	**120.10**
18	496	0.7	11	23	**90**	1144	1165	**120.10**
19	560	0.8	11	23	**90**	1292	1332	**120.10**
20	624	0.9	11	23	**90**	1439	1999	**120.10**
21	688	0.9	11	23	**90**	1590	1665	**120.10**

Table 5.12 Hinged Type Welded Connection Design_SUMMARY.

	V_{DL}	V_{LL}	LL/DD	TS648			Eurocode 3			LRFD		
	kN	kN		a (cm)	L_T (cm)	Angle Type	a (cm)	L_T (cm)	Angle Type	a (cm)	L_T (cm)	Angle Type
1	40	40	1	0.3	26.0	50.5	0.5	29.0	50.5	0.5	32.0	50.5
2	40	40	1.25	0.3	28.0	50.5	0.5	30.0	50.5	0.5	34.0	50.5
3	40	40	1.5	0.3	32.0	50.5	0.5	32.0	50.5	0.5	40.0	60.6
4	40	40	1.75	0.3	34.0	50.5	0.5	34.0	50.5	0.5	42.0	60.6
5	40	40	2	0.3	38.0	60.6	0.5	36.0	50.5	0.5	44.0	60.6
6	40	40	2.25	0.5	26.0	80.8	0.5	41.0	60.6	0.5	54.0	80.8
7	40	40	2.5	0.5	28.0	80.8	0.5	47.0	60.6	0.5	56.0	80.8
8	40	40	2.75	0.5	30.0	80.8	0.5	49.0	60.6	0.5	58.0	80.8
9	40	40	3	0.5	32.0	80.8	0.5	51.0	60.6	0.5	64.0	90.9
10	40	40	3.25	0.5	33.0	80.8	0.5	52.0	60.6	0.5	66.0	90.9
11	40	40	3.5	0.5	36.0	80.8	0.5	56.0	80.8	0.5	68.0	90.9
12	40	40	3.75	0.5	37.0	80.8	0.5	57.0	80.8	0.5	74.0	100.10
13	40	40	4	0.5	39.0	80.8	0.5	62.0	90.9	0.5	76.0	100.10
14	40	40	4.5	0.5	44.0	80.8	0.5	65.0	90.9	0.5	80.0	100.10
15	40	40	5	0.5	47.0	80.8	0.5	72.0	100.10	0.5	91.0	120.10
16	40	40	5.5	0.5	52.0	90.9	0.5	75.0	100.10	0.6	88.0	120.10
17	40	40	6	0.5	57.0	90.9	0.5	78.0	100.10	0.6	90.0	120.10
18	40	40	7	0.6	56.0	100.10	0.6	76.0	100.10	0.7	90.0	120.10
19	40	40	8	0.6	70.0	120.10	0.6	88.0	120.10	0.8	90.0	120.10
20	40	40	9	0.7	70.0	120.10	0.7	88.0	120.10	0.9	90.0	120.10
21	40	40	10	0.8	70.0	120.10	0.7	91.0	120.10	0.9	90.0	120.10

5.5 Rigid Type Welded Connection

Beam is welded to column directly both form its web and flanges. In this case, the web is assumed to carry both moment, shear and axial loads at the same time. In the examples, only shear and moment force resultants acting on the connection. In order to satisfy the weld resistance requirements, the beam section is selected accordingly.

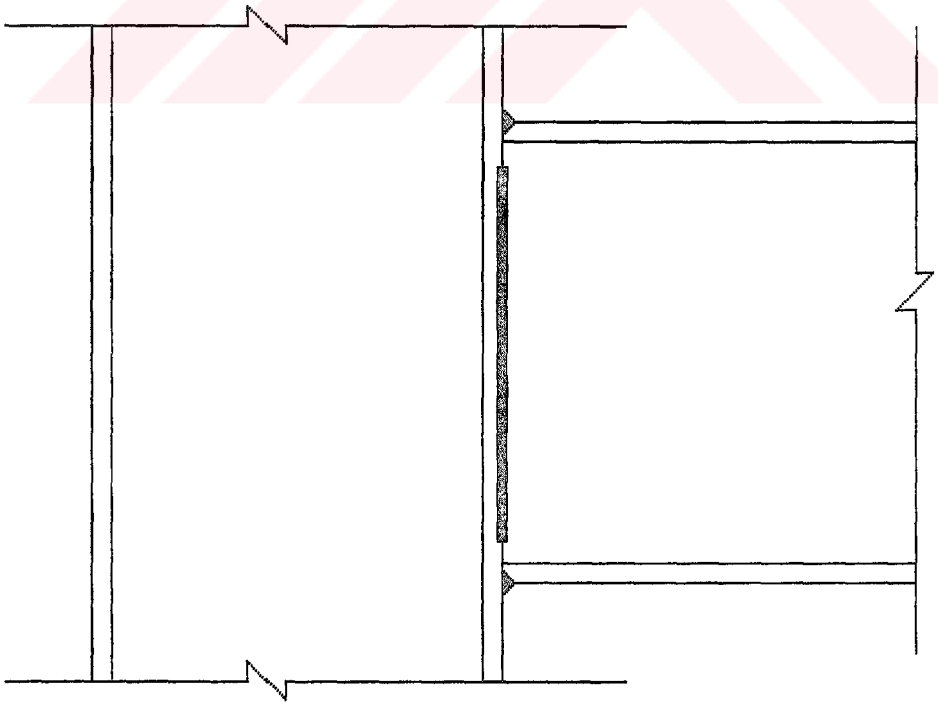

Figure-5.5

5.5.1 Definition of the Design Parameters

The following parameters are used in the design of rigid type welded connection.

<u>Steel type</u> : St 37, $f_u = 370\ N/mm^2$, $f_y = 240\ N/mm^2$,

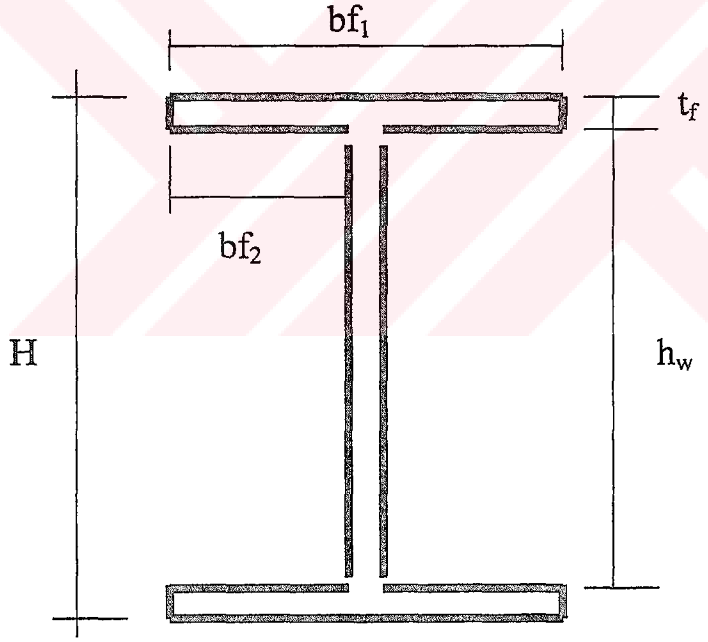

Figure 5.6

Solution Procedure:

According to Figure-5.6, the calculation steps are as follows;

$A_1=2*a_w*h_w$ a_w = throat thickness for web.

$A_2=2*a_f*bf_1$ a_f = throat thickness for flange.

$A_3=4*a_f*bf_2$

$A_4=4*a_f*t_f$

$A_T = A_1 + A_2 + A_3 + A_4$

$J = 2*(1/12)*a_w*h_w^3 + A_2*d_2^2 + A_3*d_3^2 + A_4*d_4^2$

Stress Check at welding in web:

$\tau_1 = V / (A_1) < \tau_{all}$

$\sigma_1 = M*c / J$

$\sigma_v = (\tau_1^2 + \sigma_1^2)^{0.5} < \sigma_{all,w}$

Stress Check at welding in top flange:

$\sigma_1 = M*c / J < \sigma_{all,f}$

Design Requirement:

Determination of weld length and resulting beam section required to carry the applied shear load and moment.

In the next part, the weld resistances given by each code will be presented.

Solution by TS 648:

Design weld resistance;

$\tau_{all} = 0.75\ t/cm^2$

$\sigma_{all,w} = 1.10\ t/cm^2$

$\sigma_{all,f} = 0.75\ t/cm^2$

Solution by Eurocode 3:

Design weld resistance per unit weld length;

$\tau_{all} = a_w * f_u\ /\ (\sqrt{3} * \beta\omega * \gamma_{Mw}) = 213.6\ a_w$ N/mm

for $a_w = 5mm$, $\tau_{all} = 1068$ N/mm

Solution by LRFD:

Design weld resistance per unit weld length (shear) for web;

$\tau_{all} = \phi * a_w * 0.60 * f_u = 0.75 * a_w * 0.60 * 370 = 166.5\ a_w$ N/mm

for $a_w = 5mm$, $\tau_{all} = 832.5$ N/mm

Design weld resistance per unit weld length (tension) for web;

$\sigma_{all,w} = \phi * a_w * 0.60 * f_u = 0.80 * a_w * 0.60 * 370 = 177.6\ a_w$ N/mm

for $a_w = 5mm$, $\sigma_{all,w} = 888.0$ N/mm

Design weld resistance per unit weld length (tension) for flange;

$\sigma_{all,f} = \phi * a_w * 0.60 * f_u = 0.80 * a_w * 0.60 * 370 = 177.6\ a_w$ N/mm

for $a_w = 7mm$, $\sigma_{all,f} = 1243$ N/mm

Solution Tables:

Solutions according to TS 648 are given in Table 5.13.

Solutions according to Eurocode 3 are given in Table 5.14.

Solutions according to LRFD are given in Table 5.15.

Summary of designs for each of codes are given in Table 5.16.

Notations used in Tables:

V_d = total factored design shear.

M_d = total factored design moment.

a_w = throat thickness of web

a_w = throat thickness of flange

Table 5.13 Rigid Type Welded Connection Design According to TS648.

	V_d (kN)	M_d (kN.m)	aw (cm)	af (cm)	hw (cm)	bf1 (cm)	bf2 (cm)	tf (cm)	H (cm)	τ_1 (t/cm2)	σ_v (t/cm2)	σ_1 (t/cm2)	BEAM
1	80	20	0.5	0.7	17.5	9	4.1	1.1	19.7	0.46	0.77	0.74	**I200**
2	90	22.5	0.5	0.7	18.5	9.5	4.4	1.1	20.7	0.48	0.79	0.74	**I220**
3	100	25	0.5	0.7	20	10	4.6	1.1	22.2	0.5	0.8	0.73	**I240**
4	110	27.5	0.5	0.7	20.5	10.5	4.9	1.1	22.7	0.54	0.84	0.75	**I240**
5	120	30	0.5	0.7	21	11	5.1	1.3	23.6	0.57	0.85	0.75	**I240**
6	130	32.5	0.5	0.7	22	11.5	5.3	1.3	24.6	0.59	0.87	0.75	**I260**
7	140	35	0.5	0.7	23	11.8	5.5	1.3	25.6	0.6	0.88	0.75	**I260**
8	150	37.5	0.5	0.7	23.8	12.2	5.7	1.3	26.4	0.63	0.9	0.75	**I280**
9	160	40	0.5	0.7	24.5	12.6	5.85	1.4	27.3	0.65	0.91	0.75	**I280**
10	170	42.5	0.5	0.7	25.2	13	6	1.5	28.2	0.67	0.93	0.75	**I300**
11	180	45	0.5	0.7	26	13.5	6.25	1.5	29	0.69	0.94	0.75	**I300**
12	190	47.5	0.5	0.7	26.5	13.6	6.25	1.75	30	0.72	0.96	0.74	**I300**
13	200	50	0.5	0.7	27.2	14	6.45	1.75	30.7	0.74	0.97	0.75	**I320**
14	220	55	0.5	0.7	29.5	14.2	6.5	1.75	33	0.74	0.98	0.75	**I340**
15	240	60	0.5	0.7	32	14.3	6.5	1.95	35.9	0.75	0.97	0.74	**I360**
16	260	65	0.5	0.7	34.7	15	6.8	2.05	38.8	0.75	0.95	0.68	**I400**
17	280	70	0.5	0.7	37.5	15.5	7	2.2	41.9	0.75	0.94	0.65	**I425**
18	320	80	0.5	0.7	42.7	17.8	8.05	2.56	47.82	0.75	0.89	0.57	**I500**
19	360	90	0.5	0.7	48	20	9.05	3	54	0.75	0.87	0.5	**I550**
20	400	100	0.5	0.7	53.5	21.5	9.67	3.24	59.98	0.75	0.85	0.46	**I600**
21	440	110	0.5	0.7	59	22	9.8	3.5	66	0.75	0.84	0.44	**I650+**

Table 5.14 Rigid Type Welded Connection Design According to Eurocode 3.

	V_d (kN)	M_d (kN.m)	aw (cm)	af (cm)	hw (cm)	bf1 (cm)	bf2 (cm)	tf (cm)	H (cm)	τ_1 (t/cm2)	σ_v (t/cm2)	σ_1 (t/cm2)	BEAM
1	114	28.5	0.5	0.7	13	7.4	3.4	0.95	14.9	0.44	1.04	1.17	**I160**
2	129	32.25	0.5	0.7	14.2	7.4	3.4	0.95	16.1	0.45	1.07	1.19	**I180**
3	144	36	0.5	0.7	14.7	8.2	3.7	1.04	16.78	0.48	1.07	1.17	**I180**
4	159	39.75	0.5	0.7	16.2	8.2	3.7	1.04	18.28	0.49	1.06	1.15	**I200**
5	174	43.5	0.5	0.7	16.7	9	4.1	1.1	18.9	0.52	1.06	1.13	**I200**
6	189	47.25	0.5	0.7	18	9	4.1	1.1	20.2	0.53	1.06	1.11	**I220**
7	204	51	0.5	0.7	18.3	9.8	4.5	1.22	20.74	0.56	1.06	1.09	**I220**
8	219	54.75	0.5	0.7	19.6	9.8	4.5	1.22	22.04	0.56	1.06	1.08	**I220**
9	234	58.5	0.5	0.7	20	10.6	4.9	1.31	22.62	0.59	1.06	1.06	**I240**
10	249	62.25	0.5	0.7	21	10.6	4.9	1.31	23.62	0.59	1.06	1.06	**I240**
11	264	66	0.5	0.7	21.5	11.3	5.2	1.41	24.32	0.61	1.06	1.03	**I260**
12	279	69.75	0.5	0.7	22.5	11.3	5.2	1.41	25.32	0.62	1.06	1.03	**I260**
13	294	73.5	0.5	0.7	23.2	11.9	5.4	1.52	26.24	0.63	1.06	1	**I280**
14	324	81	0.5	0.7	25.1	11.9	5.4	1.52	28.14	0.64	1.06	1	**I280**
15	354	88.5	0.5	0.7	26.8	12.5	5.7	1.62	30.04	0.66	1.06	0.97	**I300**
16	384	96	0.5	0.7	28	13.1	6	1.73	31.46	0.68	1.06	0.96	**I320**
17	414	103.5	0.5	0.7	30	13.7	6.2	1.83	33.66	0.69	1.06	0.91	**I340**
18	474	118.5	0.5	0.7	32.4	14.9	6.8	2.05	36.5	0.73	1.06	0.89	**I380**
19	534	133.5	0.5	0.7	35.5	15.5	7	2.16	39.82	0.75	1.06	0.86	**I400**
20	594	148.5	0.5	0.7	38	16.3	7.4	2.3	42.6	0.78	1.06	0.85	**I425**
21	654	163.5	0.5	0.7	40.5	17.8	8	2.56	45.62	0.8	1.06	0.8	**I475**

Table 5.15 Rigid Type Welded Connection Design According to LRFD.

	V_d (kN)	M_d (kN.m)	aw (cm)	af (cm)	hw (cm)	bf1 (cm)	bf2 (cm)	tf (cm)	H (cm)	τ_1 (t/cm2)	σ_v (t/cm2)	σ_1 (t/cm2)	BEAM
1	112	28	0.5	0.7	14	8.2	3.8	1.04	16.08	0.4	0.87	0.97	**I160**
2	128	32	0.5	0.7	15.8	8.2	3.8	1.04	17.88	0.4	0.88	0.95	**I180**
3	144	36	0.5	0.7	16.5	9	4.1	1.13	18.76	0.44	0.88	0.94	**I200**
4	160	40	0.5	0.7	17.5	9.8	4.5	1.22	19.94	0.46	0.87	0.91	**I200**
5	176	44	0.5	0.7	19.2	9.8	4.5	1.22	21.64	0.46	0.87	0.89	**I220**
6	192	48	0.5	0.7	20	10.6	4.9	1.31	22.62	0.48	0.87	0.87	**I240**
7	208	52	0.5	0.7	21.3	10.6	4.9	1.31	23.92	0.49	0.87	0.85	**I240**
8	224	56	0.5	0.7	22	11.3	5.2	1.41	24.82	0.51	0.87	0.85	**I260**
9	240	60	0.5	0.7	23.2	11.3	5.2	1.41	26.02	0.52	0.87	0.83	**I260**
10	256	64	0.5	0.7	24.1	11.9	5.4	1.52	27.14	0.53	0.87	0.81	**I280**
11	272	68	0.5	0.7	25	12.5	5.7	1.62	28.24	0.54	0.87	0.81	**I300**
12	288	72	0.5	0.7	26.2	12.5	5.7	1.62	29.44	0.55	0.87	0.79	**I300**
13	304	76	0.5	0.7	27	13.1	6	1.73	30.46	0.56	0.87	0.77	**I320**
14	336	84	0.5	0.7	29	13.7	6.2	1.83	32.66	0.58	0.87	0.75	**I340**
15	368	92	0.5	0.7	31	14.3	6.5	1.95	34.9	0.59	0.87	0.75	**I360**
16	400	100	0.5	0.7	32.5	14.9	6.8	2.05	36.6	0.62	0.87	0.73	**I380**
17	432	108	0.5	0.7	34.4	15.5	7	2.16	38.72	0.63	0.87	0.69	**I400**
18	496	124	0.5	0.7	37.9	17	7.7	2.43	42.76	0.65	0.87	0.66	**I425**
19	560	140	0.5	0.7	42	17	8.8	2.56	47.12	0.67	0.87	0.61	**I475**
20	624	156	0.5	0.7	45	20	9.1	3	51	0.69	0.87	0.6	**I550**
21	688	172	0.5	0.7	48	21.5	9.7	3.2	54.4	0.72	0.87	0.6	**I550**

Table 5.16 Rigid Type Welded Connection Design_SUMMARY

	TS648		Eurocode 3		LRFD		SELECTED I BEAM		
	1.0DL+1.0LL	1.0DL+1.0LL	1.35DL+1.5LL	1.35DL+1.5LL	1.2DL+1.6LL	1.2DL+1.6LL	TS648	Eurocode	LRFD
	V_d (kN)	M_d (kN.m)	V_d (kN)	M_d (kN.m)	V_d (kN)	M_d (kN.m)			
1	kN	kN.m	kN	kN.m	kN	kN.m	I200	I160	I160
2	80	20	114	28.5	112	28	I220	I180	I180
3	90	22.5	129	32.25	128	32	I240	I180	I200
4	100	25	144	36	144	36	I240	I200	I200
5	110	27.5	159	39.75	160	40	I240	I200	I220
6	120	30	174	43.5	176	44	I260	I220	I240
7	130	32.5	189	47.25	192	48	I260	I220	I240
8	140	35	204	51	208	52	I280	I220	I260
9	150	37.5	219	54.75	224	56	I280	I240	I260
10	160	40	234	58.5	240	60	I300	I240	I280
11	170	42.5	249	62.25	256	64	I300	I260	I300
12	180	45	264	66	272	68	I300	I260	I300
13	190	47.5	279	69.75	288	72	I320	I280	I320
14	200	50	294	73.5	304	76	I340	I280	I340
15	220	55	324	81	336	84	I360	I300	I360
16	240	60	354	88.5	368	92	I400	I320	I380
17	260	65	384	96	400	100	I425	I340	I400
18	280	70	414	103.5	432	108	I500	I380	I425
19	320	80	474	118.5	496	124	I550	I400	I475
20	360	90	534	133.5	560	140	I600	I425	I550
21	400	100	594	148.5	624	156	I650+	I475	I550

CHAPTER 6

CONCLUSIONS

LRFD and EC3 are based on probabilistic concepts. They promise consistent reliability over elastic design especially for cases where loads and load effects can be reasonably determined.

6.1 Conclusions

This study covers four type of connections as mentioned in Chapter 5. Since the codes are adopted for steel buildings in general, the lower values of LL/DL ratios (LL/DL may be less than 1 for bridges, etc.) are not considered in the comparison of codes. Only high-strength bolts with no pretension are used for bolted-type of connections. The load combinations involve dead and live loads. Within this scope, the following conclusions can be stated;

1. Generally, for the intermediate values of LL/DL ratios, the results are very close to each other, especially for LL/DL ratios up to 3 or 4.

2. For the extreme values of LL/DL ratios, LRFD and EC3 give more economical results compared to TS648.
3. For hinge type of welded connections studied in this thesis, TS648 produces more economical solutions over LRFD and Eurocode 3.
4. For rigid type of welded connection studied in this thesis, the use of the limit state codes, LRFD and Eurocode 3, is more advantageous compared to TS648. Especially, Eurocode 3 gives very economical solutions.
5. For bolted type of connections studied in this thesis, when comparing the results of two limit states codes, LRFD results in less number of bolts with respect to Eurocode 3.
6. For welded type of connections studied in this thesis, when comparing the results of two limit state codes, Eurocode 3 produces more economical designs compared to LRFD.
7. LRFD and Eurocode 3 allow various combinations of different load types by applying different load factors.
8. LRFD and Eurocode 3 allow to use larger bolt holes and slotted holes which is very good from the erection point of view. On the other hand, the bolt holes are standard in TS648.

In general, TS648, LRFD and Eurocode 3 produce similar designs for the ratios of LL/DL that are frequently encountered in the design of common types of buildings.

It will be very beneficial to start to use LRFD or Eurocode 3 in Turkey because they insure the consistency in reliability due to its probabilistic nature. The same thing can not be said for the allowable stress design. Moreover, the results are more rational from the design point of view in LRFD and EC3. In addition, in case of acceptance of Turkey to European Union, it will be mandatory to use Eurocodes. So it is better to experience now.

6.2 Suggestions For Further Research

1. The study may be extended to cover other types of connections, such as semi-rigid, end plate, etc.
2. Other load combinations including earthquake, wind, etc. can also be studied.
3. Lower values of LL/DL ratios (less than 1) may be studied.
4. High-strength bolts with pre-tensioning may be included in the study.

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