EVALUATION OF STEEL BUILDING DESIGN METHODOLOGIES: TS 648, EUROCODE 3 AND LRFD

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submitted by **ALTAN ZERVENT** in partial fulfillment of the requirements for the degree of **Master of Science in Civil Engineering Department, Middle East Technical University** by,

Prof. Dr. Canan Özgen _____________________
Dean, Graduate School of **Natural and Applied Sciences**

Prof. Dr. Güney Özcebe _____________________
Head of Department, **Civil Engineering**

Prof. Dr. Çetin Yılmaz _____________________
Supervisor, **Civil Engineering Dept., METU**

Assist. Prof. Dr. Alp Caner _____________________
Co-Supervisor, **Civil Engineering Dept., METU**

Examining Committee Members:

Assoc. Prof. Dr. Uğur Polat _____________________
Civil Engineering Dept., METU

Prof. Dr. Çetin Yılmaz _____________________
Engineering Science Dept., METU

Assoc. Prof. Dr. Cem Topkaya _____________________
Civil Engineering Dept., METU

Assist. Prof. Dr. Alp Caner _____________________
Civil Engineering Dept., METU

M.Sc. Halit Levent Akbaş _____________________
Chairman of the Board, Atak Engineering

Date: May 12, 2009

I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

Name, Last name: Altan Zervent

Signature

ABSTRACT

EVALUATION OF STEEL BUILDING DESIGN METHODOLOGIES:
TS 648, LRFD, EUROCODE 3

Zervent, Altan
M.S., Department of Civil Engineering
Supervisor: Prof. Dr. Çetin Yılmaz
Co-Supervisor: Assist. Prof. Dr. Alp Caner

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The aim of this study is designing steel structures with the same geometry, material and soil conditions but in the different countries, and comparing these designs in terms of material savings. According to three steel building codes, namely TS 648, LRFD, Eurocode 3, same structures with various stories (2, 4, 6, 8, and 10) are analyzed and designed. To calculate the design loads, Turkish Earthquake Code 2007 and Turkish Standard 498 (Design Load for Buildings) are utilized when TS 648 is applied. When LRFD is concerned, ASCE Standard 7-05 (Minimum Design Loads for Buildings and Other Structures) and AISC Standard 341-05 (Seismic Provisions for Structural Steel Buildings) are used for calculation of the design loads and earthquake loads. When Eurocode 3 is applied, Eurocode 8 (Earthquake Resistance Code), Eurocode 1 (Actions of Structures) and Eurocode-EN 1990 (Basis of Structural Design) are used in order to determine the design and earthquake loads. Weight of steel used on 1 m^2 is almost the same for procedures of LRFD and EC3.

It is important to note that those procedures consider 20 % of material saving compared to TS648.

Keywords: TS 648, AISC-LRFD, Eurocode 3, Allowable Stress Design, Limit State Design

ÖZ

ÇELİK YAPILARIN TASARIM METODLARININ DEĞERLENDİRİLMESİ: TS 648, LRFD, EUROCODE 3

Zervent, Altan
Yüksek Lisans, İnşaat Mühendisliği Bölümü
Tez Yöneticisi: Prof. Dr. Çetin Yılmaz
Ortak Tez Yöneticisi: Yrd. Doç. Dr. Alp Caner

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Bu çalışmanın amacı farklı ülkelerde inşa edilmiş fakat aynı geometriye, malzemeye ve zemin koşullarına sahip olan çelik yapıları malzeme sarfiyatları bakımından karşılaştırmaktır. Aynı çelik yapı farklı kat varyasyonları (2, 4, 6, 8, 10) için TS 648, AISC-LRFD ve Eurocode 3`e göre dizayn edilmiştir. TS 648` e göre tasarım yapılırken Türk Deprem Şartnamesi (TDY 2007) ve Türk Yük Şartnamesi (TS 498) kullanılmıştır. AISC-LRFD` ye göre tasarım yapılırken Amerikan İnşaat Mühendisleri Topluluğu`nun Binalar ve Diğer Yapılar için Minimum Dizayn Yükleri Şartnamesi (AISC Standard 7-05) ile Çelik Yapılar için Sismik Önlemler Şartnamesi (AISC Standard 341-05) kullanılmıştır. Eurocode 3` e göre tasarım yapılırken Avrupa Deprem Şartnamesi (Eurocode 8), Avrupa Yük Şartnamesi (Eurocode 1) ve Yapı Dizaynının Temeli Şartnamesi (Eurocode - EN 1990) kullanılmıştır. LRFD ve Eurocode 3 yönetmeliklerine göre dizayn edilen yapılarda, 1m^2 alana düşen çelik ağırlığı yaklaşık olarak aynıdır. Aynı zamanda bu iki yönetmelik TS 648`e kıyasla %20 daha hafif sonuç vermiştir.

Anahtar Kelimeler: TS 648, AISC-LRFD, Eurocode 3, Emniyet Gerilmesi Tasarımı, Taşıma Gücü Tasarımı

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LIST OF SYMBOLS / ABBREVATIONS

TS 648	Turkish Standard 648
LRFD	Load and Resistance Factor Design
ASCE	American Society of Civil Engineers
AISC	American Institute of Steel Construction
TEC 2007	Turkish Earthquake Code
ASD	Allowable Stress Design
f_{calc}	Calculated Stress
F_{allow}	Allowable Stress
EY	Sum of the Main Loads
EIY	Sum of the Main Loads and Superimposed Loads
Q_i	Calculated load effect
γ_i	Load factor
R_n	Nominal strength
ϕ	Resistance factor
ULS	Ultimate Limit States
SLS	Serviceability Limit States
F_d	Design action
γ_f	Partial load factor
F_k	Value of characteristic action
R_d	Design resistance
R_k	Characteristic resistance
γ_M	Partial factor
D	Dead load

L	Live load
L_r	Roof live load
S	Snow load
R	Rain load
W	Wind load
E	Earthquake load
H	Load due to lateral earth pressure
Q	Variable action
A	Accidental action
A_d	Design value of accidental action
A_{Ed}	Design value of seismic action
A_{Ek}	Characteristic value of seismic action
P	Prestressing action
γ_G	Partial factor for permanent actions
γ_Q	Partial factor for variable actions
γ_P	Partial factor for prestressing actions
Ψ	Combination factor
"+"	'to be combined with '
$\sum$	'the combined effect of '
WLB	Web Local Buckling
FLB	Flange Local Buckling
F_y	Yield Stress
λ	Slenderness Parameter
λ_f	Slenderness parameter for flange
λw	Slenderness parameter for web
b_f	Flange width
t_f	Thickness of flange
h	Clear distance between flanges
t_w	Thickness of web

ε	Factor for modified the limiting width-thickness ratio
f_y	Yield strength
A_g	Gross area or total cross-sectional area
n	Number of holes
s	Distance between two adjacent holes in the direction of force
g	Distance between two adjacent holes in the direction normal to the direction of force
t	Thickness of element
m	Number of diagonal or zigzag lines
σ_a or σ_y	Yield limit stress
$\sigma_{çem}$	Allowable tensile stress
σ_d	Ultimate tensile strength
A_e	Effective net area of member
F_y	Specified minimum yield stress
F_u	Specified minimum tensile stress
ϕ_t	Resistance factor for tension
U	Shear lag factor
f_y	Specified minimum yield stress
f_u	Specified minimum tensile stress
I_c	Moment of inertia of columns
I_g	Moment of inertia of beams
L_c	Length of columns
L_g	Length of beams
λ_p	Critical slenderness ratio
E	Modulus of elasticity
n	Factor of safety
F_E	Elastic buckling stress
F_{cr}	Elastic critical buckling stress

ϕ_c	Resistance factor for compression
N_{Ed}	Design value of the compression force
$N_{c,Rd}$	Design resistance of the cross-section for uniform compression
A	Gross area of member
$N_{b,Rd}$	Design buckling resistance of the cross-section
X	Reduction factor for the relevant buckling mode
γ_{M1}	Partial factor for resistance of members
Φ	Value to determine the reduction factor X
$\bar{\lambda}$	Non dimensional slenderness
N_{cr}	Elastic critical force for the relevant buckling mode
I	Second moment of area
L_{cr}	Buckling length
σ_{max}	Maximum stress of the beam
M_{max}	Maximum bending moment at the section about bending axis
W_{el}	Elastic section modulus of cross section
V	External shear
Q	Statical moment of area
I	Moment of inertia
t	Lateral width of cross section
τ_{all}	Allowable shear stress
h	Height of cross section
s	Distance between the supports of the beam
i_{yc}	Radius of gyration of the compression flange and 1/3 of the compression web area about the symmetry axis
F_b	Cross-sectional area of the compression flange
d	Beam depth
C_b	Bending coefficient

σ_B	The allowable compressive stress considering lateral buckling
M_1	The smaller of the two end moments at the lateral supports of the beam
M_2	The larger of the two end moments at the lateral supports of the beam
M_u	Moment under factored loads (from structural analysis)
M_n	Nominal flexural strength of beam
V_u	Shear under factored loads (from structural analysis)
V_n	Nominal shear strength of beam
Φ_b	Resistance factor for flexure
Φ_v	Resistance factor for shear
M_p	Plastic bending moment
F_y	Yield stress of the type of steel being used
Z_x	Plastic section modulus about the x-axis
S_x	Elastic section modulus taken about the x-axis
L_p	Limiting laterally unbraced length for the limit state of yielding
L_r	Limiting laterally unbraced length for the limit state of inelastic lateral torsional buckling
M_{max}	Absolute value of maximum moment in the unbraced segment
M_A	Absolute value of moment at 1/4 point of the unbraced segment
M_B	Absolute value of moment at 1/2 point of the unbraced segment
M_C	Absolute value of moment at 3/4 point of the unbraced segment
R_m	Cross-section monosymmetry parameter

r_{ts}	Effective radius of gyration used in the determination of L_r
r_y	Radius of gyration about y-axis
h_0	Distance between the flange centroids
C_w	Warping constant
J	Torsional constant
Φ_v	Resistance factor for shear
V_n	Nominal shear strength
A_w	Web area
C_v	Web shear coefficient
W_{pl}	Plastic section modulus
$W_{el,min}$	Minimum elastic section modulus
$W_{eff,min}$	Minimum effective section modulus
γ_{M0}	Partial factor for resistance of cross section
h_w	Overall web depth
η	Shear area factor
f_{yr}	Reduced yield stress
ρ	Reduced factor to determine reduced design values of the resistance to bending moment
X_{LT}	Reduction factor for lateral torsional buckling
Φ_{LT}	Value to determine the reduction factor XLT
$\overline{\lambda}_{LT}$	Non dimensional slenderness for lateral torsional buckling
α_{LT}	Imperfection factor
Mcr	Elastic critical moment for lateral torsional buckling
G	Shear modulus
C_1	Value to determine the elastic critical moment Mcr
ψ	Ratio of the end moments
I_T	Torsion constant

I_w	Warping constant
σ_{Bx} σ_{By}	Allowable bending stresses for bending about major and minor axes
$\sigma_{e'x}$	Critical stresses about major axis
$\sigma_{e'y}$	Critical stresses about minor axis
C_{mx} and C_{my}	Coefficients reflecting moment gradient and lateral support condition
P_r	Required axial compressive strength using LRFD load combinations
P_c	Design axial compressive strength
M_r	Required flexural strength using LRFD load combinations
M_c	Design flexural strength
ϕ_b	Resistance factor for compression
ϕ_c	Resistance factor for flexure
M_{nt}	First-order moment using LRFD load combinations
M_{lt}	First-order moment using LRFD load combinations caused by lateral translation of the frame only
P_{nt}	First-order axial force using LRFD load combinations
P_{lt}	First-order axial force using LRFD load combinations caused by lateral translation of the frame only
$\sum P_{nt}$	Total vertical load supported by the story using LRFD load combinations
C_m	A coefficient assuming no lateral translation of the frame
P_{el}	Elastic critical buckling resistance of the member

$\sum P_{e2}$	Elastic critical buckling resistance for the story determined by sidesway buckling analysis
N_{Rk} , $M_{x,Rk}$,$M_{y,Rk}$	The characteristic values of the compression resistance of the cross section and the bending moment resistance of the cross section
X_x and X_y	The reduction factors due to flexural buckling
X_{LT}	The reduction factors due to lateral torsional buckling
k_{xx} , k_{yy} , k_{xy} , k_{yx}	The interaction factors
A_k	Shear area of cross-sections
N_d	Design axial force of link beam
e	Length of link beam
W_T	Total building weight
A(T)	Spectral acceleration coefficient
R_a	Seismic load reduction factor
A_0	Effective ground acceleration coefficient
I	Importance Factor
g_i	Total dead load at i'th storey of building
q_i	Total live load at i'th storey of building
n	Live load participation factor
F_i	Design seismic load acting at i'th storey in equivalent seismic load method
Δ_{FN}	Additional equivalent seismic load acting on the N'th storey (top) of building
N	Total number of stories of building from the foundation level
H_i	Height of i'th storey of building measured from the top foundation level
w_i	Weight of i'th storey of building by considering live load participation factor

V_t	Total equivalent seismic load
F_{fi}	Fictitious load acting at i'th storey in the determination of fundamental natural vibration period
d_{fi}	Displacement calculated at i'th storey of building under fictitious loads Ffi
Δ_i	Reduced storey drift
d_i	Displacement calculated at i'th storey of building under design seismic loads
R	Structural behavior factor
C_s	Seismic response coefficient
W	Effective seismic weight
S_{DS}	Design spectral response acceleration parameter in the short period range
R	Response modification coefficient
T	Fundamental period of structure
T_L	Long-period transition period (s) from a map
S_1	Mapped maximum considered earthquake spectral response acceleration parameter at a period of 1 s
S_{D1}	Design spectral response acceleration parameter at a period of 1,0 sec
S_{MS}	The MCE (Maximum considered earthquake) spectral response acceleration for short period
S_{M1}	The MCE (Maximum considered earthquake) spectral response acceleration at 1 s
S_S	The mapped MCE (Maximum considered earthquake) spectral response acceleration at short period
C_d	Deflection amplification factor
δ	Deflection
di	Displacement calculated at i'th storey of building under design seismic loads

T_1	Fundamental period of the building
m	Total mass of the building
$\Psi_{E,i}$	Combination coefficient
a_g	Design ground acceleration
S	Soil factor
β	Lower bound
q	Behavior factor
a_{gR}	Peak ground acceleration derived from zonation maps
γ_I	Importance factor
e_{ai}	The accidental eccentricity of storey mass
L_i	Floor dimension perpendicular to the direction of the seismic action
θ	Interstorey drift coefficient
P_{tot}	Total gravity load at and above the storey
V_{tot}	Total seismic storey shear
d_r	Design storey drift
h	Interstorey height
d_e	Storey drift

CHAPTER 1

INTRODUCTION

In civil engineering discipline, two methods are utilized in designing steel structures, namely methods based on elastic theories and methods based on plastic theories. In literature, many studies have been conducted to estimate and compare the methods in terms of the economical perspective. It has been already proven that designs based on the plastic theories lead to more economical solutions. However, in literature, evaluations are realized designing either only a specific frame or a member. Besides, those studies have been conducted for the same design loads obtained from various codes and earthquake codes. Consequently, more economical solutions have been obtained by making use of the design codes based on plastic theories. However in practice, it has been restricted that each steel design codes must be utilized together with the load and earthquake codes specific to the country. Therefore structures in different countries but with the same material and soil conditions cannot be designed identically since they are subjected to different loading conditions.

The aim of this thesis is designing structures with the same geometry, material and soil conditions but in the different countries, and comparing these designs in terms of material savings.

This thesis deals with the evaluation of steel building design methodologies including TS 648 (Turkish Standard 648) [1], LRFD (Load and Resistance Factor Design) [2] and Eurocode 3 [3] (Design of steel structures - prEN 1993-1-1). Moreover, TEC 2007 [4] (Turkish Earthquake Code 2007) and TS 498 [5] (Design Loads for Buildings) are made use of in calculating the design loads when TS 648 is applied. When LRFD is concerned, ASCE (American Society of Civil Engineers) Standard 7-05 [6] (Minimum Design Loads for Buildings and Other Structures) and AISC Standard 341-05 [7] (Seismic Provisions for Structural Steel Buildings) are used for calculation of the design loads and earthquake loads. When Eurocode 3 (Design of steel structures - prEN 1993-1-1) is applied, Eurocode 8 [8] (Design of Structures for Earthquake Resistance-prEN 1998-1), Eurocode 1 [9] (Actions of Structures-prEN 1991-1-1) and Eurocode [10] (Basis of Structural Design-EN 1990) are utilized in order to determine the design and earthquake loads.

To achieve the complete evaluation, structures with various stories (2, 4, 6, 8 and 10 stories) are formed and analyses are conducted with the outstanding abilities of LARSA-4D [11]. Afterwards, the structural members, namely beams, columns, and braces, are designed according to the formulae and restrictions of the previously mentioned codes by the help of MS Office Excel [12].

The thesis consists of eleven chapters. Aim and scope of the thesis are presented in the first chapter. Relevant studies and their evaluations are introduced in the second chapter.

In the third chapter, design fundamentals, loads which depend on the used code and load combinations are explained for TS 648, LRFD, and Eurocode 3. And this chapter also contains classification guide of the cross-sections.

In the fourth chapter, solution techniques for the members which are subjected to axial tension are presented. The fifth chapter introduces the solution techniques for the members which are exposed to pure compression. In the sixth and seventh chapters, flexural members (i.e. beams) and members which are subjected to either axial compression combined with flexure or tension combined with flexure are explained respectively.

In the eighth chapter, solutions techniques of the link beams which is required because of the usage of the eccentric braced frame is presented according to relevant codes which are TEC 2007, Seismic Provisions for Structural Steel Buildings and Eurocode 8.

The ninth chapter explains the basic principles of the earthquake codes particularly the parts of the codes where equivalent static forces are introduced since the earthquake forces are calculated according to the equivalent force methods. In the tenth chapter, dimensions, material properties, geometry, occupancy category and geotechnical properties of the structures are demonstrated. Besides, calculation procedure of the earthquake loads for each code is handled. Furthermore, design of the structural members is conducted such a way that the worst loading case is considered to find the stresses. Afterwards, the solution procedures are explained in a detailed way.

And finally, the last chapter includes conclusions of the thesis, emphasizing important findings and results of the study.

CHAPTER 2

LITERATURE REVIEW

The basic rule of thumb is that least weight meaning least cost for the steel building designer and the researchers has investigated which methodologies had been more economical and there have been a few academic dissertations and articles about this topic.

The thesis about comparison of the steel bending members according to TS 4561 (Turkish Design Standard for Plastic Design), LRFD and Eurocode 3 was published by Mehmet Ilgaz Büyüktaşkın [13] in 1998. In this master thesis, only one beam as a numerical example for same loading condition and same cross section is solved according to TS 4561, LRFD, Eurocode 3 and he has found out the section solved with LRFD was smaller than the sections solved with TS 4561 and Eurocode 3. Moreover, the sections solved with TS 4561 and Eurocode 3 has been same section.

The article about the economic design of low-rise building was published by D.Kirk Harman [14] in 1998. He has chosen three buildings, one of them had five stories and the rest had three stories. He has designed them using different steel yield strengths which are A36 (St37 in Turkish Standard) and Gr.50 (St52 in Turkish Standard) according to LRFD and ASD (Allowable Stress Design). His study shows that if the steel yield strength remains constant, the average savings of 11,8% are realized with LRFD over ASD for A36 and the average savings of 3,61% are realized with LRFD over ASD for Gr.50. Furthermore, his study pointed out that if the design method remains constant, the average savings of 14,7% are realized with Gr.50 steel grade over

A36 steel grade for the ASD designs. Using LRFD, the average savings are realized as 7,23%.

In 1999, Ali Onur Kuyucak [15] studied about comparison of TS 648, LRFD and Eurocode 3 for design fundamentals, members which are subjected to tension, compression and flexure under specific loads. In his thesis, the parameters are effective area reduction factor, the ratio of net area to total area and the ratio of dead load to total load for the numerical examples about members subjected tension. For numerical examples which are subjected to compression members, the parameters are the ratio of dead load to total load and slenderness ratio. Type of structural section, support condition, length of the structural member and the ratio of dead load to live load are used as the parameters for the numerical examples which are members subjected to flexure. His results have pointed out that LRFD and Eurocode 3 are more economical than TS 648.

Hakan Bayram [16] investigated that three steel structure codes, namely TS 648, LRFD and Eurocode 3 were compared with regard to structural steel connections in 2001.For the purpose of the comparison, four different types of connections, which are hinge type bolted connections, rigid type bolted connections, hinge type welded connections and rigid type welded connections, were used under the different ratio of dead loads and live loads. His results show that LRFD and Eurocode 3 give more economical solutions with respect to TS 648.

Erkal Albayrak [17] presented design principle and calculation of beam-columns, subjected to both axial load and moment, according to TS 4561, LRFD and Eurocode 3 in 2005. Three numerical examples were solved in this thesis. The results state that LRFD and Eurocode 3 are more complicated for design than TS 4561 but TS 4561 is simple and easier. TS 4561 and Eurocode 3 give same solutions, however LRFD leads to more economical solutions in his thesis.

A comparative study of LRFD and Eurocode 3 approaches to beam-column buckling resistance was published by Danny Yong, Aitziber Lopez and Miguel Serna [18] in 2006. For a rolled I-section, the paper is performed with different slenderness ratio. The results show that Eurocode 3 is more complicated than LRFD but LRFD gives more than enough to obtain reasonable solutions for most cases.

Eurocode 3 and NBE-EA-95 (Spanish Steel Building Code) were compared by M. A. Serna, E. Bayo, I. Castillo, L. Clemos and A. Loureiro [19] for standard steel construction. In this paper, a planar multistory frame and a pitched portal frame are analyzed and designed according to Eurocode 3 and NBE-EA-95. The results show that Eurocode 3 is more complicated than NBE-EA-95. In contrast, Eurocode 3 gives more economical solutions than NBE-EA-95.

Ioannides and Ruddy [20] presents rules and heuristics about several steel framing whose designs are based on LRFD. They come up with an approximation formula for the steel weight per square foot in a braced building using steel with St 52 which is as follows:

$$W_t \text{(psf)} = N/3 + 7 \tag{2.1}$$

where N represents the number of stories.

CHAPTER 3

DESIGN FUNDAMENTALS

3.1 Design Fundamentals

3.1.1 Design Fundamentals of TS 648

TS 648 is a code based on elastic design methodology (also called working stress design) and the material used in the structure is assumed to be linearly elastic and perfectly plastic. In Cem Topkaya`s lecture notes [21], the basic equation of TS 648 is:

$$f_{calc} \leq F_{allow} \qquad (3.1)$$

In the above equation, f_{calc} is a calculated stress in a structural component under service load, and F_{allow} calculated that the stress at failure is divided by factor of safety which is restricted from 1,67 to 2,5 by TS 648 [1] is allowable stress.

In TS 468 [1], some checks must be controlled for an acceptable design:

1) Stress Checks

2) Stability Checks

3) Overturning Checks

4) Deflection Checks

All of these checks must be carried out during the construction, transportation, assembly, and operation stages.

Stress checks must be controlled EY (Sum of the main loads) and EIY (Sum of the main loads and superimposed loads) loading conditions separately.

Stability checks must cover buckling, web crippling, and lateral buckling. Overturning checks must have safety factor of 2 against overturning for each element. In some special cases, this safety factor can be taken as 1,5. The safety factor against of supports shall not be less than 1,3. For the entire structure the safety factor shall not be less than 1,5.

The limits of deflection checks are tabulated as following table;

Table 3.1 Deflection Limits of TS 648

	Maximum Deflection
Beams and Purlings with span lengths more than 5 m	L/300
Cantilever Beams	L/250
Grillage Beams in foundations and supports	L/1000

When the structure is designed according to TS 648 [1], the material properties are pointed out below table;

Table 3.2 Material Properties of TS 648

Type of Steel	Tensile Strength, σ_d t/cm^2 (N/mm^2)	Yield Limit, σ_y or σ_a t/cm^2 (N/mm^2)	Modulus of Elasticity, E t/cm^2 (N/mm^2)	Shear Modulus, G t/cm^2 (N/mm^2)	Coefficient of Thermal Expansion, α_t
Fe 33	3,3-5,0 (324-490)	1,9 (186)	2100 (206182)	810 (79434)	0,000012
Fe 34	3,4-4,2 (333-412)	2,1 (206)			
Fe 37	3,7-4,5 (363-491)	2,4 (235)			
Fe 42	4,2-5,0 (412-490)	2,6 (255)			
Fe 46	4,4-5,4 (431-530)	2,9 (284)			
Fe 50	5,0-6,0 (490-588)	3,0 (294)			
Fe 52	5,2-6,2 (510-608)	3,6 (353)			
Fe 60	6,0-7,2 (588-706)	3,4 (333)			
Fe 70	7,0-8,5 (686-834)	3,7 (363)			

3.1.2 Design Fundamentals of LRFD

The new method of AISC Specification has been Load and Resistance Factor Design [2] (or called limit states design). During the past years, structural steel design has been progressed toward a more and probability-based design procedure which is called limit states design. In the LRFD, both overload and understrength are taken into account for provisions. The LRFD is based on a probability-based model, calibration with the Allowable Stress Design, and evolution using judgment and past experience.

There are two category of the LRFD; strength and serviceability. Strength limit states consist of plastic strengths, fracture, buckling, lateral buckling, cross sectional local buckling, fatigue, overturning. Serviceability limit states based on occupancy of structures mean excessive deflections and the performance of the structures mean excessive vibrations.

In Cem Topkaya`s lecture notes [21], the basic design equation of LRFD is:

$$\sum \gamma_i * Q_i \leq \phi * R_n \tag{3.2}$$

In the above equation, Q_i is the calculated load effect under service load i, γ_i is the load factor depends on load type and combination, R_n is the nominal strength, and ϕ is the resistance factor depends on type of resistance. The right side of the equation 3.2 represents sum of the load effects and the left side of the equation 3.2 represents the strength of the members or systems.

AISC-LRFD Specification actually does not include the limit of deflection of drift ratio for member or structure. When the structure is designed according to AISC-LRFD, the drift ratios are taken from ASCE Standard 7-05 [6] (Minimum Design Loads for Buildings and Other Structures).

3.1.3 Design Fundamentals of Eurocode 3

Eurocode 3 [3] is based on the concept of limit states. Limit states are defined as states beyond which the structure no longer satisfies the design performance requirements. Limit states include Ultimate Limit States (ULS) and Serviceability Limit States (SLS).

In Eurocode 3, the ultimate limit states consist of these requirements;

a) Equilibrium of the structure

b) Excessive deformations

c) Stability of the structure

d) Rupture

e) Fatigue

f) Time-depended effects

In Eurocode 3 [3], the serviceability limit states include of this requirements;

a) Deformations which affect the appearance or the comfort of occupants or mechanism of the structure

b) Vibrations which affect the comfort of users

The basic design equation of Eurocode 3 can be written as;

$$F_d = \gamma_f * F_k \leq R_d = R_k / \gamma_M \qquad (3.3)$$

In the above equation, F_d is the design action which means factored loads, γ_f is a partial load factor depends on the load type and combination, F_k is the value of characteristic action, R_d is the design resistance, R_k is the characteristic resistance, γ_M is a partial factor depends on material, geometric and modeling uncertainties.

The limits of vertical deflection and horizontal deflection are tabulated as following tables;

Table 3.3 Vertical Deflection Limits of Eurocode 3

	Vertical Deflection Limits
Beams carrying plaster or other brittle finish	Span/360
Cantilever Beams	Length/180
Purlins and sheeting rails	To suit cladding
Other Beams	Span/200

Table 3.4 Horizontal Deflection Limits of Eurocode 3 for Vertical Loads

	Horizontal Deflection Limits
Tops of Columns in single storey buildings, except portal frames	Height/300
In each storey of a building with more than one storey	Height of Storey/300
Columns in portal frame buildings, not supporting crane runways	To suit cladding

The below table show the material properties when the structure is designed according to Eurocode 3 [3];

Table 3.5 Material Properties of Eurocode 3

Steel Grade	Thickness Range, t (mm)	Yield Strength, f_y (N/mm^2)	Modulus of Elasticity, E (N/mm^2)	Shear Modulus, G (N/mm^2)	Coefficient of Thermal Expansion, α_t	Poisson`s Ratio, ν
S235	$t \leq 40$	235	210000	81000	0,000012	0,3
	$40 < t \leq 80$	215				
S275	$t \leq 40$	275				
	$40 < t \leq 80$	255				
S355	$t \leq 40$	355				
	$40 < t \leq 80$	335				

3.2 Loads and Load Factors

3.2.1 Loads and Load Factor for TS 648

When the structure is designed according to TS 648, TS 498 is applied because TS 498 shows the design load for buildings. In TS 498, the weights per unit volume of construction materials and live loads per unit area are given to be used in the estimation. The below table shows values of the live loads for TS 498 [5].

Table 3.6 Live Loads for TS 648

	Live Loads (kg/m^2)
Attick Rooms	150
Rooms of Residendal Builldings Rooms of Office Buildings Corridor for Residental Buildings Hospital Patient Rooms	200
Classrooms Examinations Rooms of Hospitals Amphitheatres Landings for Residental Buildings Dormitory Rooms	350
Mosques Theatres and Cinema Theatres Sport, Dancing and Exhibition Rooms Stands with Fixed Chairs Restaurants Libraries	500
Tribunes with Moveable Chairs	750
Garages for less than 2,5 tons	500

In TS 648, the loads are classified as main loads and superimposed loads. Main loads include self-weight, regular and additional live loads and snow loads. Superimposed loads consist of wind load, earthquake loads, break forces, temperature effects etc.

According the TS 648 [1], two loading cases are considered for the design. The first one is EY loading case. EY loading case includes all main loads and EIY loading case includes all main loads and superimposed loads acting on the structure.

The allowable stresses are different for the two loading cases. The allowable stresses for EIY loading are found by increasing the values given for EY by 15%.

3.2.2 Loads and Load Factor for LRFD

LRFD refers to ASCE Standard 7-05 [6] (Minimum Design Loads for Buildings and Other Structures) for design loads. In this standard, dead load, live load, soil load, wind load, snow load, rain load, flood load, and earthquake load are presented.

Live loads may change in position and magnitude but live loads are assumed to be uniformly distributed. When the structure is designed according to LRFD, the values of the uniformly distributed live loads are utilized in the below table.

Table 3.7 Live Loads for LRFD

	Live Loads (kN/m^2)
Residental Dwellings(one- and two- family):	
Unhabitable attics without storage	0,48
Unhabitable attics with storage	0,96
Habitable attics and sleeping areas	1,44
All other areas except stairs and balconies	1,92
Hotels and multifamily houses:	
Private rooms and corridors	1,92
Public rooms and corridors	4,79
Office Buildings:	
Lobbies and first-floor corridors	4,79
Offices	2,40
Corridors above first floor	3,83
Hospitals:	
Operating rooms, laboraties	2,87
Patient rooms	1,92
Corridors above first floor	3,83
Libraries:	
Reading rooms	2,87
Stack rooms	7,18
Corridors above first floor	3,83
Roofs:	
Ordinary flat,pitched and curved roofs	0,96
Roofs used promenade purposes	2,87
Schools:	
Classrooms	1,92
Corridors above first floor	3,83
First - floor corridors	4,79

The following list of load combinations from ASCE 7-05 [6] is used when the structures and their components are to be designed for LRFD design methodology.

1) 1,4 * D
2) 1,2 * D + 1,6 * L + 0,5 * (L_r or S or R)
3) 1,2 * D + 1,6 * (L_r or S or R) + (L or 0,8 * W)
4) 1,2 * D + 1,6 * W + L + 0,5 * (L_r or S or R)
5) 1,2 * D + 1,0 * E + L + 0,2 * S
6) 0,9 * D + 1,0 * E + 1,6 * H
7) 0,9 * D + 1,6 * W + 1,6 * H

The load factor on L in combinations 3, 4, 5 is permitted to equal 0,5 for all occupancies in which L is less than or equal to 4,79 kN/m^2.
For the above load combinations, the below abbreviations are used;

- D : Dead load
- L : Live load
- L_r : Roof live load
- S : Snow load
- R : Rain load
- W : Wind load
- E : Earthquake load
- H : Load due to lateral earth pressure, ground water pressure, or pressure of bulk materials

3.2.3 Loads and Load Factor for Eurocode 3

Eurocode 3 [3] refers to Eurocode 1 (prEN 1991-1-1) and Eurocode (Basis of Structural Design-EN 1990) for design action and their combinations. The actions are divided three groups; permanent actions, (e.g. self-weight), variable actions, (e.g. imposed loads, snow loads), accidental actions, (e.g. explosions).

In Eurocode 1 [9], areas of buildings are classified as a lot of categories but only four categories; residential areas, social areas, commercial areas, administration areas for imposed loads are examined. Other categories are storage and industrial activities areas, garages and vehicle traffic areas, helicopter areas, roofs. The below table presents the four categories of use for imposed loads.

Table 3.8 Category of Use for Eurocode 1

Category	Specific Use	Example
A	Areas for domestic and residential activities	Rooms in residential buildings and houses; bedrooms in hotels
B	Office areas	
C	Areas where people may congregate	C1 : Areas with tables eg. areas in schools, cafes, restaurants C2 : Areas with fixed seats eg. areas in churches, theatres or cinemas C3 : Areas without obstacles for moving people eg. areas in museums, exhibition rooms C4 : Areas with possible physical activities eg. dance halls, gymnastic rooms, stages C5 : Areas susceptible to large crowds eg. sport halls including stands, terraces
D	Shopping areas	D1 : Areas in general retail shops D2 : Areas in department stores

The values of the imposed loads depend on Table 3.8 are tabulated as the below table.

Table 3.9 Imposed Loads for Eurocode 1

Categories of Loaded Areas	q_k **(kN/m²)**
Category A	
Floors	1,5 to 2,0
Stairs	2,0 to 4,0
Balconies	2,5 to 4,0
Category B	2,0 to 3,0
Category C	
C1	2,0 to 3,0
C2	3,0 to 4,0,
C3	3,0 to 5,0
C4	4,5 to 5,0
C5	5,0 to 7,5
Category D	
D1	4,0 to 5,0
D2	4,0 to 5,0

The National annex may be different from the above table. Where a range is given in the table, the values of the imposed loads may be altered by the National annex.

There are three design situations for combinations of actions in the Eurocode (EN 1990) [10]; combinations of actions for persistent or transient design situations, combinations of actions for accidental design situations, and combinations of actions for seismic design situations.

In the Eurocode (EN 1990) [10], the basic format of the combinations of the actions for persistent or transient design situations can be written as;

$$\sum_{j\geq 1}\gamma_{G,j}G_{k,j}"+"\gamma_P P"+"\gamma_{Q,1}Q_{k,1}"+"\sum_{i>1}\gamma_{Q,i}\Psi_{0,i}Q_{k,i} \tag{3.4}$$

In the Eurocode (EN 1990), the basic format of the combinations of the actions for accidental design situations can be written as;

$$\sum_{j\geq 1}G_{k,j}"+"P"+"A_d"+"(\Psi_{1,1} or \Psi_{2,1})Q_{k,1}"+"\sum_{i>1}\Psi_{2,i}Q_{k,i} \tag{3.5}$$

In the Eurocode (EN 1990) [10], the basic format of the combinations of the actions for seismic design situations can be written as;

$$\sum_{j\geq 1}G_{k,j}"+"P"+"A_{Ed}"+"\sum_{i\geq 1}\Psi_{2,i}Q_{k,i} \tag{3.6}$$

For the equations which are (3.4), (3.5) and (3.6), the below abbreviations are used;

- G : Permanent action
- Q : Variable action
- A : Accidental action
- A_d : Design value of accidental action
- A_{Ed} : Design value of seismic action ($A_{Ed}=\gamma_I A_{Ek}$)
- A_{Ek} : Characteristic value of seismic action
- P : Prestressing action
- γ_G : Partial factor for permanent actions
- γ_Q : Partial factor for variable actions
- γ_P : Partial factor for prestressing actions
- Ψ : Combination factor
- "+" : means ‘ to be combined with ’
- $\sum$: means ‘ the combined effect of ’

In order to establish combinations of actions for buildings, methods, rules and Ψ factors are presented in Annex A1 of the Eurocode (EN 1990).

The below table presents design values of actions in persistent and transient design situations.

Table 3.10 Design Values of Actions for Use in Persistent and Transient Combinations of Actions

Persistent and Transient Design Situations	Permanent Actions		Leading Varibale Action	Accompanying Variable Actions	
	Unfavourable	Favourable		Main	Others
Equation	$\gamma_{Gj,sup}G_{kj,sup}$	$\gamma_{Gj,inf}G_{kj,inf}$	$\gamma_{Q,1}Q_{k,1}$		$\gamma_{Q,i}\Psi_{0,i}Q_{k,i}$
$\gamma_{Gj,sup}$=1,35 $\gamma_{Gj,inf}$=1,15 $\gamma_{Q,1}$=1,50 for unfavourable (0 for favourable) $\gamma_{Q,i}$=1,50 for unfavourable (0 for favourable)					

The below table presents design values of actions in accidental and seismic design situations.

Table 3.11 Design Values of Actions for Use in Accidental and Seismic Combinations of Actions

Design Situation	Permanent Actions		Leading Accidental or Seismic Action	Accompanying Variable Actions	
	Unfavourable	Favourable		Main	Others
Accidental	$G_{kj,sup}$	$G_{kj,inf}$	A_d	$\Psi_{1,1}$ or $\Psi_{2,1}Q_{k,1}$	$\Psi_{2,i}Q_{k,i}$
Seismic	$G_{kj,sup}$	$G_{kj,inf}$	$\gamma_I A_{Ek}$ or A_{Ed}	$\Psi_{2,i}Q_{k,i}$	

The recommended values of Ψ factors for used Table 3.10 and Table 3.11 are tabulated as following tables;

Table 3.12 The Values of Ψ Factors in the Eurocode (EN 1990)

Action	Ψ_0	Ψ_1	Ψ_2
Imposed Loads			
Category A: Domestic,residental area	0,7	0,5	0,3
Category B: Office areas	0,7	0,5	0,3
Category C: Congragation areas	0,7	0,7	0,6
Category D: Shopping areas	0,7	0,7	0,6
Category E: Storage areas	1,0	0,9	0,8
Category F: Traffic areas for vehicle weight≤30 kN	0,7	0,7	0,6
Category G: Traffic areas for 30kN≤vehicle weight≤160 kN	0,7	0,5	0,3
Category H: Roofs	0	0	0
Wind Loads on Buildings	0,6	0,2	0
Temperature in Buildings	0,6	0,5	0

3.3 Classification of Cross-Sections

3.3.1 Classification of Cross-Sections for TS 648

In TS 648, there are no rules for classification of cross-sections. However, there are limitations for width-thickness ratios of cross-sections in TEC 2007. This subject will be discussed in the ninth chapter.

3.3.2 Classification of Cross-Sections for LRFD

In the LRFD, the classification of cross-sections depend on local buckling which are web local buckling (WLB) and flange local buckling (FLB) and these local buckling depend on two variable; yield stress (F_y) of the type of steel being used and width-thickness ratio of web or flange.

There are three classes for the classification of cross-sections according to the LRFD. These are compact section, noncompact section and slender section. For this classification, "λ" is slenderness parameter used as a symbol for width-thickness ratio. "λ_f" is slenderness parameter for flange and "λ_w" is slenderness parameter for web in a wide-flange shape. λ_f and λ_w are expressed as;

$$\lambda_f = b_f / 2t_f \quad (3.7)$$

$$\lambda_w = h / t_w \quad (3.8)$$

where

b_f : Flange width

t_f : Thickness of flange

h : Clear distance between flanges less the fillet or corner radius for rolled shapes

t_w : Thickness of web

For I shape, b_f, t_f, h, t_w are shown in the below figure.

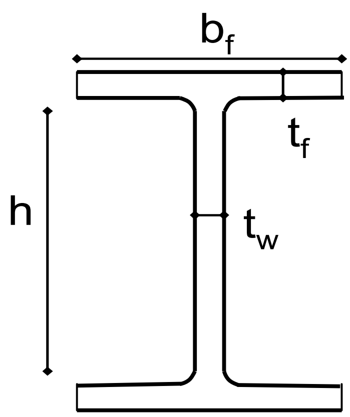


Figure 3.1 I Shape

Compact section is defined as capable of developing a fully plastic stress distribution before local buckling occurs. Noncompact section can develop the yield stress in compression before the onset of local buckling; however will not resist inelastic local buckling at strain levels required for a fully plastic stress. Slender section which cannot be loaded to cause yield at extreme fibers due to local buckling occurred early.

Compact section's width-thickness ratios must not be greater than λ_p (Limiting slenderness parameter for compact element). Noncompact section's width-thickness ratios must not be greater than λ_r (Limiting slenderness parameter for noncompact element) but must greater than λ_p. Slender section's width-thickness ratios must exceed λ_r .

In the LRFD [2], the limiting width-thickness ratios for compressions elements are tabulated as following tables;

Table 3.13 The Limiting Width-Thickness Ratios for Compressions Elements in the LRFD

Description of Element	Width Thickness Ratio	Limiting Width-Thickness Ratios	
		λ_p (Compact)	λ_r (Noncompact)
Flexure in flanges of rolled I-shaped sections and channels	b/t	$0,38*(E/F_y)^{0,5}$	$1,0*(E/F_y)^{0,5}$
Flexure in webs of doubly symmetric I-shaped sections and channels	h/t_w	$3,76*(E/Fy)^{0,5}$	$5,70*(E/Fy)^{0,5}$
Flexure in webs of rectengular hollow square sections	h/t	$2,42*(E/Fy)^{0,5}$	$5,70*(E/Fy)^{0,5}$

3.3.3 Classification of Cross-Sections for Eurocode 3

In Eurocode 3, there are four classes of cross-sections which depend on width-thickness ratios of the cross section.

According the Eurocode 3 [3], the four classes are defined as; Class 1 cross-sections can be named as plastic sections because they can develop a plastic hinge with the rotation capacity required from plastic analysis, Class 2 cross-sections can be named as compact sections for they have limited rotation capacity due to local buckling but can develop their plastic moment resistance, Class 3 cross-sections can be named as semi-compact sections because the elastically calculated stress in the extreme compression fiber of the steel component assuming an elastic distribution can reach the yield strength, however local buckling is apt to prevent development of the plastic moment resistance, Class 4 can be names as slender section because local buckling will occur before the extreme compression fiber of the steel component reach yielding stress.

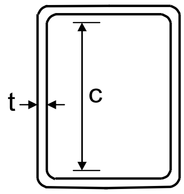
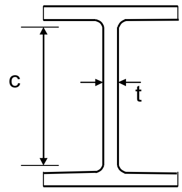
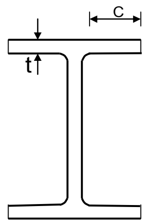
For this classification, "ε" is used as a factor for modified the limiting width-thickness ratio. ε is defined as;

$$\varepsilon = \sqrt{235/f_y} \tag{3.9}$$

where f_y is yield strength.

The below table shows maximum width-thickness ratios for compression parts. The limitation ratios can be obtained from below table for Class 1, Class 2 and Class 3 compression parts.

Table 3.14 The Limiting Width-Thickness Ratios for Compressions Parts in the Eurocode 3

	Internal Compression Part		Outstand Flanges
			
Class	Part Subject to Bending	Part Subject to Compression	Part Subject to Compression
1	c/t ≤ 72*ε	c/t ≤ 33*ε	c/t ≤ 9*ε
2	c/t ≤ 83*ε	c/t ≤ 38*ε	c/t ≤ 10*ε
3	c/t ≤ 124*ε	c/t ≤ 42*ε	c/t ≤ 14*ε

If a part exceeds the limitation of Class 3, the part is taken as Class 4. The overall classification of the cross-section is classified that taken as the highest class of its component parts.

CHAPTER 4

AXIAL TENSION IN MEMBERS

Tension members are structural elements that are subjected to axial tensile forces. They are usually used in different types of structures. Tension members are usually found as braces in buildings and bridges; truss members, and cables in suspended roof systems. Tension members frequently appear as tie rods.

4.1 Design of Tension Members for TS 648

4.1.1 Net Cross-Sectional Area

The net cross-sectional area is obtained by substracing the cross sectional area of holes on the most critical path from the overall cross-sectional area whenever a tension member is to be fastened by means of rivets or bolts.

A_{net} can be described as below formula for a tension member with bolt or rivet holes:

$$A_{net} = A_g - n * A_{hole} + m * \frac{s^2}{4 * g} * t \leq 0,85 * A_g \qquad (4.1)$$

where

A_g = Gross area or total cross-sectional area

n = Number of holes

s = Distance between two adjacent holes in the direction of force

g = Distance between two adjacent holes in the direction normal to the direction of force

t = Thickness of element

m = number of diagonal or zigzag lines

The below figure related to the above formula:

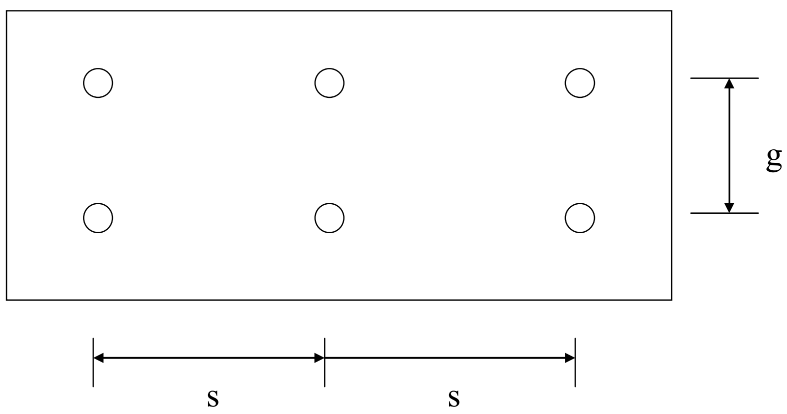


Figure 4.1 Tension Member

The net area must be lesser than 85% of the gross area.

On the other hand, if there is a tension member without bolt or rivet holes, net section area equals to total cross sectional area.

$$A_{net} = A_g \tag{4.2}$$

4.1.2 Allowable Stresses

For the tension members, the allowable tensile stress ($\sigma_{çem}$) on the net cross-sectional area:

$$\sigma_{çem} \leq 0,6 * \sigma_a \tag{4.3}$$

$\sigma_a (\sigma_y)$ is yield limit and allowable tensile stress ($\sigma_{çem}$) must not be greater than $0,5 * \sigma_d$ (Ultimate tensile strength).

Stability is not a criterion in the design of tension members according to Çetin Yılmaz` s Analysis and Design of Steel Structures [22].

However, it is necessary to limit the length of a tension member in order to prevent it from:

- Becoming too flexible
- Sagging excessively due to its own weight
- Vibrating excessively when subjected to wind forces

So, the slenderness ratio (λ) of these members should not exceed 250.

4.2 Design of Tension Members for LRFD

4.2.1 Tensile Strength

In LRFD [2], the below equations are used to calculate the design tensile strength ($\phi_t * P_n$) of tension members:

For tensile yielding in the gross section;

$$\phi_t * P_n = \phi_t * F_y * A_g \quad \text{with } \phi_t = 0,9 \tag{4.4}$$

For fracture in the net section;

$$\phi_t * P_n = \phi_t * F_u * A_e \quad \text{with } \phi_t = 0{,}75 \qquad (4.5)$$

where

A_g = Gross area of member

A_e = Effective net area of member

F_y = Specified minimum yield stress

F_u = Specified minimum tensile stress

ϕ_t = Resistance factor for tension

The design tensile strength is taken as the smaller value obtained from Eq. (4.4) and Eq. (4.5).

The slenderness ratio (λ) of the tensile members must not be greater than 300.

4.2.2 Area Determination

If there is a tension member without bolt or rivet holes for example welded connections, net section area equals to total cross sectional area.

If there is a tension member with bolt or rivet holes for example bolted connections, net section area can be described as reduced area of a section.

The net area formula is same for both LRFD and TS 648 as specified in Eq (4.1). Therefore Eq. (4.1) is valid for LRFD.

The effective area (A_e) of tension members can be computed as ;

$$A_e = U * A_n \qquad (4.6)$$

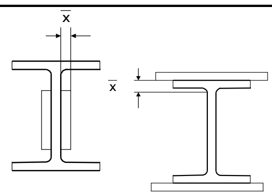
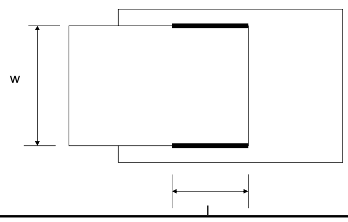
where

A_n = Net area of member

U = Shear lag factor

The shear lag factors are tabulated as following tables;

Table 4.1 Shear Lag Factor for LRFD

Description of Element	**Shear Lag Factor, U**	**Example**
All tension members where the tension load is transmitted directly to each of cross-sectional elements by fasteners or welds.	U = 1,0	
All tension members, except plates and HSS, where the tension load is transmitted to some but not all of the crosssectional elements by fasteners or longitudinal welds	$U = 1{,}0 - \bar{x} / l$	
All tension members where the tension load is transmitted by transverse welds to some but not all of the cross-sectional elements.	U=1,0 and An = area of directly connected elements	
Plates where the tension load is transmitted by longitudinal welds only	l ≥ 2*w ……… U = 1,0 2*w > l ≥ 1,5*w …..U = 0,87 1,5*w >l ≥ w…….U = 0,75	

4.3 Design of Tension Members for Eurocode 3

In Eurocode 3 [3], the below equations are utilized to calculate the design tensile strength of tension members:

For tensile yielding in the gross section;

$$N_{pl,Rd} = \frac{A * f_y}{\gamma_{M0}} \quad \text{with} \quad \gamma_{M0} = 1{,}0 \tag{4.7}$$

For the ultimate resistance of the net cross-section;

$$N_{u,Rd} = \frac{0{,}9 * A_{net} * f_y}{\gamma_{M2}} \qquad \text{with} \quad \gamma_{M2} = 1{,}25 \tag{4.8}$$

where

A = Gross area of member

A_{net} = Effective net area of member

f_y = Specified minimum yield stress

f_u = Specified minimum tensile stress

The design tensile strength must be selected as the lower value obtained from Eq. (4.7) and Eq. (4.8).

The net cross section area formula is same for both Eurocode 3 and TS 648 as specified in Eq. (4.1). Therefore Eq. (4.1) is valid for Eurocode 3 as well.

CHAPTER 5

AXIAL COMPRESSION IN MEMBERS

5.1 Design of Compression Members for TS 648

When a compression member is designed according to TS 648 [1], the first thing a designer should know is slenderness ratio (λ) which is given by the following formula:

$$\lambda = \frac{k * L}{i} \tag{5.1}$$

where

k = Effective length factor

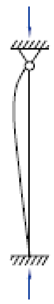
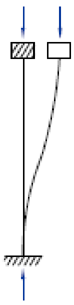
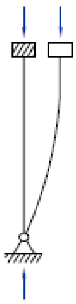
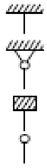
L = Length of member

i = Radius of gyration

Effective length factor (k) is given in the below Table 5.1 according to the support condition.

The effective length factor (k) for compression members in a frame system can be determined based on the Figure 5.1 and Figure 5.2 by deciding whether sidesway of the frame is prevented or permitted. The coefficient G used in the Figure 5.1 and Figure 5.2 is determined for two ends (A and B) of the compression members.

Table 5.1 Effective length factor (k)

	(a)	(b)	(c)	(d)	(e)	(f)
Buckled shape of column is shown by dashed line.						
Theoretical *K* value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when ideal conditions are approximated	0.65	0.80	1.2	1.0	2.10	2.0
End condition code	 *Rotation fixed and translation fixed* *Rotation free and translation fixed* *Rotation fixed and translation free* *Rotation free and translation free*					

The coefficient G can be calculated as using the following formula:

$$G = \frac{\sum \frac{I_c}{L_c}}{\sum \frac{I_g}{L_g}} \tag{5.2}$$

where

I_c = Moment of inertia of columns

I_g = Moment of inertia of beams

L_c = Length of columns

L_g = Length of beams

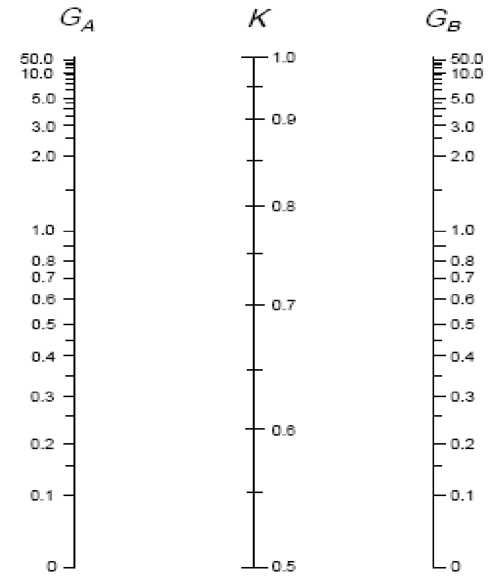


Figure 5.1 K Values For Sidesway Prevented

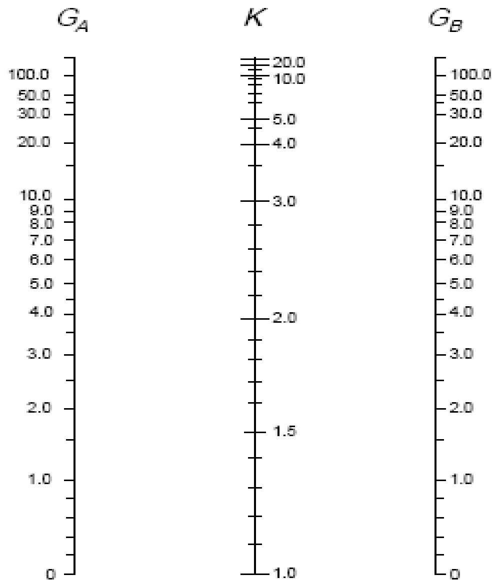


Figure 5.2 K Values For Sidesway Permitted

For the column ends supports, if the support is hinged, G theoretically becomes infinity however, it is taken as 10,0. If the support is a fixed end, G is taken as 1,0.

When conditions at the far end of any particular beam are known, refinements can be calculated for the beam stiffness I_g / L_g according to Çetin Yılmaz`s Analysis and Design of Steel Structures [22]. In this case, the beam stiffness is multiplied with the values in the below table:

Table 5.2 Values to Multiply the Beam Stiffness

Condition at Far End of Beam	Sidesway Prevented	Sidesway Permitted
Pinned	1,5	0,5
Fixed	2	0,67

For a column, two λ values which are λ_x and λ_y (buckling perpendicular to the principle axes x-x and y-y respectively) are calculated and selected as the greater value of either λ_x or λ_y. It can be explained as below formula:

$$\lambda = \max(\lambda_x; \lambda_y) \qquad (5.3)$$

When allowable compressive stress is calculated according to TS 648, the another thing a designer should know is critical slenderness ratio (λ_p) which is expressed as;

$$\lambda_p = \sqrt{\frac{2 * \pi^2 * E}{\sigma_y}} \qquad (5.4)$$

If the slenderness ratio of axially loaded (λ) member is less than critical slenderness ratio (λ_p), the allowable compressive stress must be calculated as;

$$\sigma_{\text{bem}} = \frac{\left[1 - 0,5 * (\lambda / \lambda_p)^2\right] * \sigma_y}{n} \qquad \text{for} \quad \lambda \leq \lambda_p \tag{5.5}$$

If the slenderness ratio of axially loaded (λ) member is greater than critical slenderness ratio (λ_p), the allowable compressive stress must be written as;

$$\sigma_{\text{bem}} = \frac{2 * \pi^2 * E}{5 * \lambda^2} \qquad \text{for} \quad \lambda \geq \lambda_p \tag{5.6}$$

where

E = Modulus of elasticity

σ_y = Yield stress of steel

λ_p = Critical slenderness ratio

λ = Slenderness ratio of axially loaded member

n = Factor of safety ≥ 1,67

n = 1,67 for $\lambda < 20$

$n = 1,5 + 1,2 * (\lambda / \lambda_p) - 0,2 * (\lambda / \lambda_p)^3$ for $20 \leq \lambda \leq \lambda_p$

n = 2,5 for $\lambda \geq \lambda_p$

The allowable compressive stress (σ_{bem}) is equal to the allowable tensile stress ($\sigma_{\text{çem}}$) in case the slenderness ratio is less than 20.

5.2 Design of Compression Members for LRFD

When a compression member is designed according to LRFD [2], slenderness ratio (λ), effective length factor (k), the coefficient G are same for both LRFD and TS 648 as specified in Eq (5.1), Eq (5.2), Eq (5.3), Figure 5.1, Figure 5.2, Table 5.1 and Table 5.2. Therefore, these common concepts are skipped in this part.

When allowable compressive strength is calculated according to LRFD, the another thing a designer should know is the elastic critical buckling stress (F_E) which is expressed as;

$$F_E = \frac{\pi^2 * E}{\lambda^2} \tag{5.7}$$

where

F_E = Elastic buckling stress

E = Modulus of elasticity of steel

λ = Slenderness ratio

If the slenderness ratio of axially loaded (λ) member is less than $4{,}71 * \sqrt{\frac{E}{F_y}}$, the flexural buckling stress can be written as;

$$F_{cr} = \left[0{,}658^{\left(Fy/F_E \right)} \right] * Fy \qquad \text{for} \qquad \lambda \leq 4{,}71 * \sqrt{\frac{E}{F_y}} \tag{5.8}$$

If the slenderness ratio of axially loaded (λ) member is greater than $4{,}71 * \sqrt{\frac{E}{F_y}}$, the flexural buckling stress is written as;

$$F_{cr} = 0{,}877 * F_E \qquad \text{for} \qquad \lambda > 4{,}71 * \sqrt{\frac{E}{F_y}} \tag{5.9}$$

where

F_{cr} = Elastic critical buckling stress

F_E = Elastic buckling stress

E = Modulus of elasticity of steel

λ = Slenderness ratio

The nominal compressive strength (P_n) can be computed as;

$$P_n = F_{cr} * A_g \tag{5.10}$$

The design compressive strength ($\phi_c * P_n$) can be computed as;

$$\phi_c * P_n = 0{,}9 * P_n \qquad \text{with } \phi_c = 0{,}9 \tag{5.11}$$

where

A_g = Gross area

F_{cr} = Elastic critical buckling stress

ϕ_c = Resistance factor for compression

5.3 DESIGN OF COMPRESSION MEMBERS FOR EUROCODE 3

When a member in axial compression is to be designed according to Eurocode 3, two criteria are controlled: Compression Resistance and Buckling Resistance.

- Compression Resistance :

The design resistance to normal forces of the cross-section for uniform compression must be greater than design value of the compression force.

It can be written as;

$N_{c,Rd} \geq N_{Ed}$ (5.12)

where

N_{Ed} = Design value of the compression force

$N_{c,Rd}$ = Design resistance of the cross-section for uniform compression

The design resistance to normal forces of the cross-section can be expressed as ;

$N_{c,Rd} = \frac{A * f_y}{\gamma_{M0}}$ with $\gamma_{M0} = 1{,}0$ for Class 1, 2 or 3 (5.13)

$N_{c,Rd} = \frac{A_{eff} * f_y}{\gamma_{M0}}$ with $\gamma_{M0} = 1{,}0$ for Class 4 (5.14)

where

A = Gross area of member

A_{eff} = Effective area of a cross section

f_y = Specified minimum yield stress

γ_{M0} = Partial factor for resistance of cross section

- Buckling Resistance :

The design buckling resistance of the cross-section for uniform compression must be greater than the design value of the compression force.

It can be written as

$N_{b,Rd} \geq N_{Ed}$ (5.15)

where

N_{Ed} = Design value of the compression force

$N_{b,Rd}$ = Design buckling resistance of the cross-section

The design buckling resistance of the compression member should be taken as ;

$$N_{b,Rd} = \frac{X * A * f_y}{\gamma_{M1}} \quad \text{with } \gamma_{M1} = 1{,}0 \quad \text{for Class 1, 2 and 3} \qquad (5.16)$$

$$N_{b,Rd} = \frac{X * A_{eff} * f_y}{\gamma_{M1}} \quad \text{with } \gamma_{M1} = 1{,}0 \quad \text{for Class 4} \qquad (5.17)$$

where

X = Reduction factor for the relevant buckling mode

A = Gross area of member

A_{eff} = Effective area of a cross section

f_y = Specified minimum yield stress

γ_{M1} = Partial factor for resistance of members

The reduction factor for the relevant buckling mode (X) is given as below;

$$X = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \leq 1{,}0 \tag{5.18}$$

where

Φ = Value to determine the reduction factor X

$\bar{\lambda}$ = Non dimensional slenderness

The value to determine the reduction factor X (Φ) is given as below;

$$\Phi = 0{,}5 * \left[1 + \alpha * (\bar{\lambda} - 0{,}2) + \bar{\lambda}^2\right] \tag{5.19}$$

where

$\bar{\lambda}$ = Non dimensional slenderness

α = Imperfection factor

Non dimensional slenderness ($\bar{\lambda}$) can be expressed as;

$$\bar{\lambda} = \sqrt{\frac{A * f_y}{N_{cr}}} \quad \text{for Class 1, 2 and 3} \tag{5.20}$$

$$\bar{\lambda} = \sqrt{\frac{A_{eff} * f_y}{N_{cr}}} \quad \text{for Class 4} \tag{5.21}$$

where

N_{cr} = Elastic critical force for the relevant buckling mode

The elastic critical force for the relevant buckling mode (N_{cr}) can be expressed as;

$$N_{cr} = \frac{\pi^2 * E * I}{L_{cr}^2} \qquad (5.22)$$

where

E = Modulus of elasticity

I = Second moment of area

L_{cr} = Buckling length

The buckling length of the compression member (L_{cr}) can be obtained from below figure;

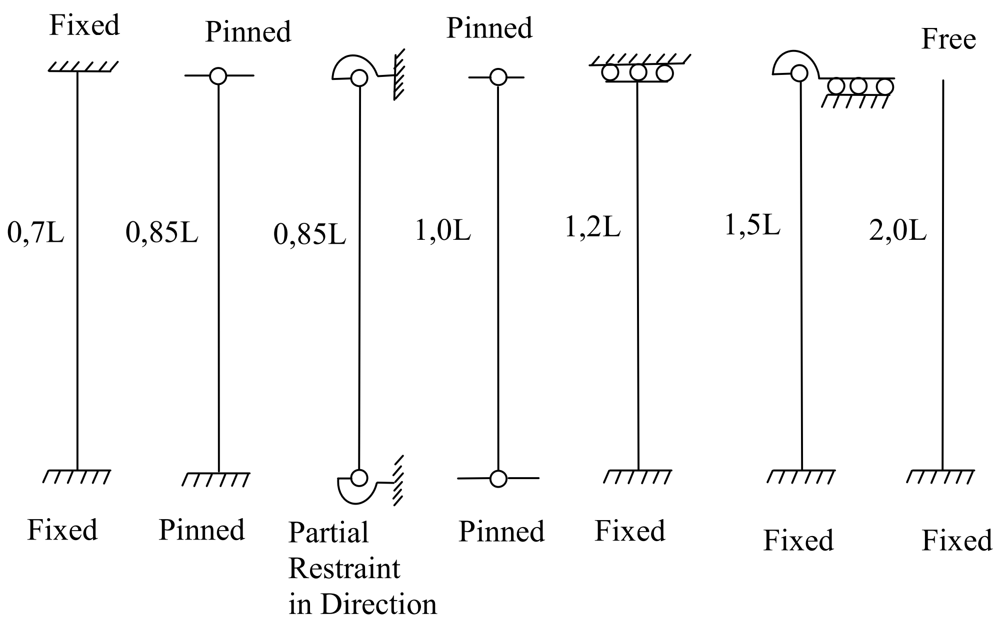


Figure 5.3 Buckling Lengths (L_{cr}) for compression members

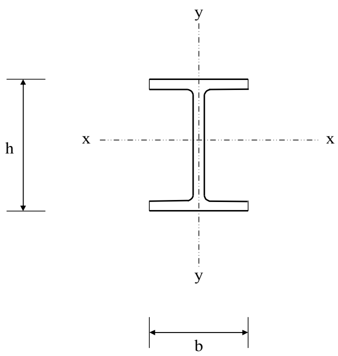
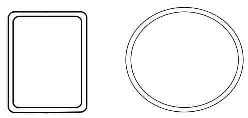
The imperfection factor for buckling curves (α) can be obtained from below table;

Table 5.3 Imperfection factors for buckling curves

Buckling Curve	a_0	a	b	c	d
Imperfection Factor (α)	0,13	0,21	0,34	0,49	0,76

The buckling curve for a cross section can be obtained from below table;

Table 5.4 Selection of buckling curve for a cross sections

Cross Section			Limits	Buckling about axis	Buckling Curve: S 235, S 275, S 355, S 420	Buckling Curve: S 460
Rolled Sections		h/b > 1,2	$t_f \leq 40$ mm	x - x	a	a_0
				y - y	b	a_0
			40 mm < $t_f \leq 100$ mm	x - x	b	a
				y - y	c	a
		h/b ≤ 1,2	$t_f \leq 100$ mm	x - x	b	a
				y - y	c	a
			$t_f > 100$ mm	x - x	d	c
				y - y	d	c
Hollow Sections		Hot Finished		any	a	a_0
		Cold Formed		any	c	c

CHAPTER 6

DESIGN OF MEMBERS SUBJECTED TO FLEXURE

6.1 Design of Members Subjected To Flexure for TS 648

If a beam is to be designed according to TS 648 [1], stress check, shear check, lateral buckling check must be controlled. Moreover, if the beam is subjected to biaxial bending, it must be controlled as well.

- Stress Check :

The bending stress of the beam must be less than the allowable compressive stress in bending. The bending stress can be expressed as follows;

$$\sigma_{max} = \frac{M_{max}}{W_{el}} \leq \sigma_{all} = 0,6 * \sigma_y \qquad (6.1)$$

where

σ_{max} = Maximum stress of the beam

M_{max} = Maximum bending moment at the section about bending axis

W_{el} = Elastic section modulus of cross section

σ_{all} = Allowable bending stress

σ_y = Yield stress of the material

- Shear Check :

The shear stress of the beam must be less than the allowable shear stress. For a beam having an open cross section (symmetrical section), the shear stress can be obtained from the shear formula as follows;

$$\tau = \mp \frac{V_y * Q_x}{I_x * t} \mp \frac{V_x * Q_y}{I_y * t} \leq \tau_{all} = \frac{\sigma_{çem}}{\sqrt{3}} \qquad (6.2)$$

where

V = External shear

Q = Statical moment of area

I = Moment of inertia

t = Lateral width of cross section

$\sigma_{çem}$ = Allowable tensile stress

τ_{all} = Allowable shear stress

For a beam having a rectangular section, the shear stress can be obtained from the shear formula as follows;

$$\tau = \frac{3}{2} * \frac{V}{h * t} \leq \tau_{all} = \frac{\sigma_{çem}}{\sqrt{3}} \qquad (6.3)$$

where

V = External shear

h = Height of cross section

t = Lateral width of cross section

For a beam having an I section, the shear stress can be obtained from the shear formula as follows;

$$\tau = \frac{V}{h*t} \leq \tau_{all} \tag{6.4}$$

where

V = External shear

h = Height of cross section

t = Lateral width of cross section

- Lateral Buckling Check :

In Çetin Yılmaz`s Analysis and Design of Steel Structures [22], beam buckling can be called a lateral-torsional buckling by reason of the fact that the buckling displacements are in the lateral direction and also twisting. If the compression flange of a beam is restrained against lateral buckling, lateral buckling does not occur. If the compression flange of a beam is not restrained against lateral buckling, the lateral stability of the beam must be checked.

According to Çetin Yılmaz`s Analysis and Design of Steel Structures [22], for lateral buckling the critical stress is affected by:

- Material properties
- Spacing
- Types of lateral support
- Types of end support
- Loading conditions

In TS 648, the allowable compressive stress considering lateral buckling can be obtained as follows;

If $\frac{s}{i_y} \leq \sqrt{\frac{30000 * C_b}{\sigma_y}}$;

$$\sigma_{B1} = \left[\frac{2}{3} - \frac{\sigma_y * (s/_{i_{yc}})^2}{90000 * C_b} \right] * \sigma_y \leq \sigma_{all} \tag{6.5}$$

If $\frac{s}{i_y} \geq \sqrt{\frac{30000 * C_b}{\sigma_y}}$;

$$\sigma_{B1} = \frac{10000 * C_b}{(s / i_y)^2} \leq \sigma_{all} \tag{6.6}$$

If the compression flange is almost rectangular and not smaller than tension flange;

$$\sigma_{B2} = \frac{840 * C_b}{s * \frac{d}{F_b}} \tag{6.7}$$

where

s = Distance between the supports of the beam

i_{yc} = Radius of gyration of the compression flange and 1/3 of the compression web area about the symmetry axis

F_b = Cross-sectional area of the compression flange

d = Beam depth

C_b = Bending coefficient

σ_B = The allowable compressive stress considering lateral buckling

σ_y = Yield stress of the compression flange material

The allowable compressive stress considering lateral buckling is the greater of Eq.(6.5) or of Eq.(6.6) and Eq.(6.7) however, this value must not be greater than $\sigma_{all} = 0,6 * \sigma_y$. This can be expressed as follow;

$$\sigma_B = \max(\sigma_{B1} ; \sigma_{B2}) \quad (6.8)$$

C_b is a bending coefficient which is determined as follow;

$$C_b = 1,75 + \left[1,05 * (\frac{M_1}{M_2}) \right] + 0,3 * \left[(\frac{M_1}{M_2})^2 \right] \leq 2,3 \quad (6.9)$$

in which

M_1 = The smaller of the two end moments at the lateral supports of the beam

$M_{\backslash 2}$ = The larger of the two end moments at the lateral supports of the beam

In the above equation, if M_1 and M_2 have the same sign which means the beam is double curvature, (M_1 / M_2) is positive and if M_1 and M_2 have the different sign which means the beam is single curvature, (M_1 / M_2) is negative.

If the internal moment at any point between the lateral supports of the beam is larger than the end moments, C_b must be as 1,0.

6.2 Design of Members Subjected To Flexure for LRFD

In Cem Topkaya`s lecture notes [21], the following equations must be satisfied at each point along the length of beam to design of beams subjected to flexure for LRFD:

$M_u \leq \Phi_b * M_n$ with $\Phi_b = 0,9$ (6.10)

$V_u \leq \Phi_v * V_n$ with $\Phi_v = 0,9$ or $1,0$ (6.11)

in which

M_u = Moment under factored loads (from structural analysis)

M_n = Nominal flexural strength of beam

V_u = Shear under factored loads (from structural analysis)

V_n = Nominal shear strength of beam

Φ_b = Resistance factor for flexure

Φ_v = Resistance factor for shear

The nominal flexural strength (M_n) is the lesser value of:

- Yielding
- Lateral torsional buckling
- Flange local buckling
- Web local buckling

In this thesis, only doubly symmetric compact I-shaped members subjected to flexure have been designed. Thus, only corresponding solution procedure has been explained.

The nominal flexural strength, M_n, must be lower value obtained according to the limit states of yielding (plastic moment) and lateral-torsional buckling for the doubly symmetric compact I-shaped members bent about their major axis.

- Yielding :

$Mn = Mp = F_y * Z_x$ (6.12)

where

M_p = Plastic bending moment

F_y = Yield stress of the type of steel being used

Z_x = Plastic section modulus about the x-axis

- Lateral Torsional Buckling :

If $L_b \leq L_p$;

The limit state of lateral torsional buckling does not apply.

where

L_b = Length between points that are either braced against lateral displacement of compression flange

L_p = Limiting laterally unbraced length for the limit state of yielding

If $L_P < L_b \leq L_r$;

$$M_n = C_b * \left[M_p - (M_p - 0{,}7 * F_y * S_x) * (\frac{L_b - L_p}{L_r - L_p}) \right] \leq M_p \quad (6.13)$$

where

C_b = Lateral torsional buckling modification factor

M_p = Plastic bending moment

F_y = Yield stress of the type of steel being used

S_x = Elastic section modulus taken about the x-axis

L_p = Limiting laterally unbraced length for the limit state of yielding

L_r = Limiting laterally unbraced length for the limit state of inelastic lateral torsional buckling

If $L_b > L_r$;

$$M_n = F_{cr} * S_x \leq M_p \quad (6.14)$$

where

S_x = Elastic section modulus taken about the x-axis

F_{cr} = Critical stress

For above equations, lateral torsional buckling modification factor (C_b) is determined based on the below equation;

$$C_b = \frac{12{,}5 * M_{\max}}{2{,}5 * M_{\max} + 3 * M_A + 4 * M_B + 3 * M_C} * R_m \leq 3{,}0 \quad (6.15)$$

where

M_{max} = Absolute value of maximum moment in the unbraced segment

M_A = Absolute value of moment at 1/4 point of the unbraced segment

M_B = Absolute value of moment at 1/2 point of the unbraced segment

M_C = Absolute value of moment at 3/4 point of the unbraced segment

R_m = Cross-section monosymmetry parameter (1,0 for doubly symmetric members)

C_b can be permitted to be conservatively taken as 1,0 for all cases. Moreover, C_b is taken as 1,0 for cantilevers or overhangs.

For above equations, the limiting lengths L_p and L_r can be expressed as;

$$L_p = 1{,}76 * r_y * \sqrt{\frac{E}{F_y}} \qquad (6.16)$$

$$L_r = 1{,}95 * r_{ts} * \frac{E}{0{,}7 * Fy} * \sqrt{\frac{J * c}{S_x * h_0}} * \sqrt{1 + \sqrt{1 + 6{,}76 * (\frac{0{,}7 * F_y}{E} * \frac{S_x * h_0}{J * c})^2}} \qquad (6.17)$$

where

r_y = Radius of gyration about y-axis

E = Modulus of elasticity

F_y = Yield stress of the type of steel being used

r_{ts} = Effective radius of gyration used in the determination of L_r

$$r_{ts} = \sqrt{\frac{\sqrt{I_y * C_w}}{S_x}} \qquad (6.18)$$

I_y = Moment of inertia

S_x = Elastic section modulus taken about the x-axis

C_w = Warping constant

J = Torsional constant

c = 1,0   (for a doubly symmetric I-shape)

h_0 = Distance between the flange centroids

For above equations, critical stress (F_{cr}) can be expressed as;

$$F_{cr} = \frac{C_b * \pi^2 * E}{(\frac{L_b}{r_{ts}})^2} * \sqrt{1 + 0{,}078 * \frac{J * c}{S_x * h_0} * (\frac{L_b}{r_{ts}})^2} \tag{6.19}$$

The below figure obtained from Cem Topkaya`s lecture notes [21] clearly shows the relationship between L_b and M_n.

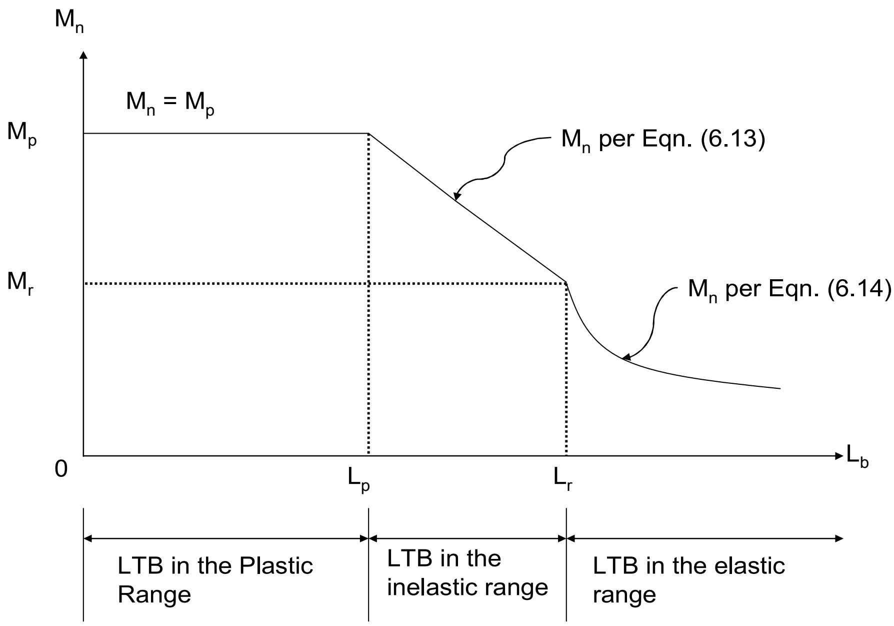


Figure 6.1 Variation of M_n with L_b

The nominal flexural strength, M_n, must be lower value obtained according to the limit states of yielding (plastic moment) and flange local buckling for the doubly symmetric compact I-shaped members bent about their minor axis.

- Yielding :

$$Mn = Mp = F_y * Z_y \leq 1,6 * F_y * S_y \qquad (6.20)$$

where

F_y = Yield stress of the type of steel being used

Z_y = Plastic section modulus about the y-axis

S_y = Elastic section modulus about the y-axis

- Flange Local Buckling :

The limit state of flange local buckling does not apply for compact flange.

$$Mn = Mp = F_y * Z_y \qquad (6.21)$$

For doubly symmetric members subject to shear in the plane of web, the design shear strength, Φv * Vn, can be expressed as below formula;

$$\Phi_v * V_n = \Phi v * (0,6 * F_y * A_w * C_v) \quad \text{for } \frac{h}{t_w} \leq 2,24 * \sqrt{\frac{E}{F_y}} \qquad (6.22)$$

Where

Φ_v = 1,0 (Resistance factor for shear)

V_n = Nominal shear strength

$A_w = h * t_w$ (Web area)

$C_v = 1,0$ (Web shear coefficient)

For doubly symmetric members subject to shear in the weak axis, the design shear strength, $\Phi_v * V_n$, can be expressed as below formula;

$$\Phi_v * V_n = \Phi v * (0,6 * F_y * A_w * C_v) \quad \text{for } \frac{h}{t_w} \leq 2,24 * \sqrt{\frac{E}{F_y}} \tag{6.23}$$

where

$\Phi_v = 0,9$ (Resistance factor for shear)

$A_w = b_f * t_f$ (Web area)

$C_v = 1,0$ (Web shear coefficient)

6.3 Design of Members Subjected To Flexure for Eurocode 3

If a beam is to be designed according to Eurocode 3, the following checks must be controlled:

- Bending
- Shear
- Bending and Shear
- Bending and Axial Force
- Biaxial Bending
- Buckling Resistance in Bending

In this thesis, only doubly symmetric compact I-shaped members subjected to flexure have been designed. Thus, only corresponding solution procedure has been explained.

- Bending Resistance :

The design value of the resistance to bending moment ($M_{c,Rd}$) must be greater than the design value of the bending moment (M_{Ed}).

It can be written as;

$$M_{c,Rd} \geq M_{Ed} \tag{6.24}$$

The design resistance expressions for bending of the cross-sections are given below formulae;

$$M_{c,Rd} = \frac{W_{pl} * f_y}{\gamma_{M0}} \quad \text{for Class 1 or 2} \tag{6.25}$$

$$M_{c,Rd} = \frac{W_{el,\min} * f_y}{\gamma_{M0}} \quad \text{for Class 3} \tag{6.26}$$

$$M_{c,Rd} = \frac{W_{eff,\min} * f_y}{\gamma_{M0}} \quad \text{for Class 4} \tag{6.27}$$

where

W_{pl} = Plastic section modulus

$W_{el,min}$ = Minimum elastic section modulus

$W_{eff,min}$ = Minimum effective section modulus

The subscript 'min' shows that the minimum value of W_{el} or W_{eff} must be used.

- Shear Resistance :

The design plastic shear resistance of the cross-section ($V_{pl,Rd}$) must be greater than design value of the shear force (V_{Ed}).

It can be expressed as;

$$V_{pl,Rd} \geq V_{Ed} \tag{6.28}$$

The yield stress of steel in shear is approximately $1/\sqrt{3}$ of its yield stress thus the design plastic shear resistance is given below;

$$V_{pl,Rd} = \frac{A_v * (f_y / \sqrt{3})}{\gamma_{M0}} \tag{6.29}$$

where

A_v = Shear area

f_y = Specified minimum yield stress

γ_{M0} = Partial factor for resistance of cross section

The shear area can be expressed as below formulae;

1. Rolled I Sections, load parallel to web :

$$A_v = A - 2 * b * t_f + (t_w + 2 * r) \geq \eta * h_w * t_w \tag{6.30}$$

2. Rolled I Sections, load parallel to flanges :

$$A_w = A - \sum (h_w * t_w) \tag{6.31}$$

3. Rolled rectangular hollow sections of uniform thickness, load parallel to depth :

$A_v = A * h / (b+h)$ (6.32)

4. Rolled rectangular hollow sections of uniform thickness, load parallel to depth :

$A_v = A * b / (b+h)$ (6.33)

where

A = Cross-sectional area

h_w = Overall web depth

η = Shear area factor (taken as 1,2)

The resistance of the web to shear buckling must be checked. If the below criterion is satisfied for unstiffened webs, shear buckling need not be checked.

$$\frac{h_w}{t_w} \leq 72 * \frac{\varepsilon}{\eta} \quad (6.34)$$

- Bending and Shear :

If the design value of the shear force is less than half the plastic shear resistance of the cross-section, its effect on the moment resistance can be neglected. For cases where the applied shear force is greater than half the plastic shear resistance of the cross section, the reduced moment resistance should be calculated using a reduced design strength calculated by below equation [3];

$f_{yr} = (1- \rho) * f_y$ (6.35)

where

f_{yr} = Reduced yield stress

f_y = Yield stress

ρ = Reduced factor to determine reduced design values of the resistance to bending moment

The reduced factor to determine reduced design values of the resistance to bending moment (ρ) can be calculated as below equation;

$$\rho = (\frac{2 * V_{Ed}}{V_{pl,Rd}})^2 \tag{6.36}$$

- Bending and Axial Force :

The design plastic moment resistance reduced due to the axial force ($M_{N,Rd}$) should be greater than design bending moment (M_{Ed}). It can be expressed as;

$$M_{Ed} \leq M_{N,Rd} \tag{6.37}$$

For doubly symmetric I sections subjected to axial force and major axis bending moment, no reduction in the major axis plastic moment resistance is provided if both of the below criteria are met ;

$$N_{Ed} \leq 0{,}25 * N_{pl,Rd} \tag{6.38}$$

$$N_{Ed} \leq \frac{0{,}5 * h_w * t_w * f_y}{\gamma_{M0}} \tag{6.39}$$

For doubly symmetric I sections subjected to axial force and minor axis bending moment, no reduction in the major axis plastic moment resistance is provided if both of the below criterion is met ;

$$N_{ed} \leq \frac{h_w * t_w * f_y}{\gamma_{M0}} \tag{6.40}$$

where

N_{Ed} = Design normal force

$N_{pl,Rd}$ = Design plastic resistance to normal forces of the gross section

h_w = Overall web depth

t_w = Web thickness

If the above criteria are not met, a reduced plastic moment resistance shall be calculated using by following formulae;

Major Axis:

$$M_{N,Rd} = M_{pl,Rd} * \frac{1-n}{1-0,5*a} \leq M_{pl,Rd} \tag{6.41}$$

Minor Axis:

For n ≤ a $\quad M_{N,Rd} = M_{pl,Rd}$ (6.42)

For n > a $\quad M_{N,Rd} = M_{pl,Rd} * \left[1-(\frac{n-a}{1-a})^2\right]$ (6.43)

in which

n = Ratio of applied load to plastic compression resistance of section

$$n = \frac{N_{Ed}}{N_{pl,Rd}} \tag{6.44}$$

a = Ratio of the area of the web to the total area

$$a = \frac{A - 2 * b * t_f}{A} \tag{6.45}$$

- Biaxial Bending :

For biaxial bending the below equation is used;

$$(\frac{M_{x,Ed}}{M_{N,x,Rd}})^{\alpha} + (\frac{M_{y,Ed}}{M_{N,y,Rd}})^{\beta} \leq 1,0 \tag{6.46}$$

where α and β are constants for I Sections, as defined below;

$$\alpha = 2 \quad \text{and} \quad \beta = 5 * n \geq 1,0 \tag{6.47}$$

- Buckling Resistance in Bending

The design buckling resistance moment ($M_{b,Rd}$) must be greater than the design value of the moment (M_{Ed}). It can be explained as;

$$M_{b,Rd} \geq M_{Ed} \tag{6.48}$$

The design buckling resistance moment of a laterally unrestrained beam is described through below equation;

$$M_{b,Rd} = X_{LT} * W_x * \frac{f_y}{\gamma_{M1}} \tag{6.49}$$

where

X_{LT} = Reduction factor for lateral torsional buckling

W_x = Appropriate section modulus

- $W_x = W_{pl.x}$ for Class 1 and 2 (6.50)

- $W_x = W_{el.x}$ for Class 3 (6.51)

- $W_x = W_{eff.x}$ for Class 4 (6.52)

f_y = Specified minimum yield stress

γ_{M1} = Partial factor for resistance of members

The reduction factor for lateral torsional buckling (X_{LT}) is explained as below equation;

$$X_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \bar{\lambda}_{LT}^2}} \qquad (6.53)$$

where

Φ_{LT} = Value to determine the reduction factor X_{LT}

$\bar{\lambda}_{LT}$ = Non dimensional slenderness for lateral torsional buckling

The value to determine the reduction factor X_{LT} (Φ_{LT}) is given as below;

$$\Phi_{LT} = 0{,}5 * \left[1 + \alpha_{LT} * (\bar{\lambda}_{LT} - 0{,}2) + \bar{\lambda}_{LT}^2\right] \qquad (6.54)$$

where

$\bar{\lambda}_{LT}$ = Non dimensional slenderness for lateral torsional buckling

α_{LT}= Imperfection factor

Non-dimensional slenderness for lateral torsional buckling ($\bar{\lambda}_{LT}$) can be expressed as;

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_x * f_y}{M_{cr}}} \qquad (6.55)$$

where

M_{cr} = Elastic critical moment for lateral torsional buckling

The elastic critical moment for lateral torsional buckling (M_{cr}) can be expressed as;

$$M_{cr} = C_1 * \frac{\pi^2 * E * I_y}{L_{cr}^{\;2}} * \left(\frac{I_w}{I_y} + \frac{L_{cr}^{\;2} * G * I_T}{\pi^2 * E * I_y}\right)^{0,5} \qquad (6.56)$$

where

E = Modulus of elasticity

I_y = Second moment of area about the minor axis

L_{cr} = Length of beam between points of lateral restraint

I_T = Torsion constant

I_w = Warping constant

G = Shear modulus

C_1 = Value to determine the elastic critical moment M_{cr}

The values of C_1 for end moment loading can be approximated by below equation;

$$C_1 = 1,88 - 1,4 * \psi + 0,52 * \psi^2 \leq 2,7 \qquad (6.57)$$

where

ψ = Ratio of the end moments

The ratio of the end moments (ψ) can be expressed as below;

$$\psi = \frac{M_1}{M_2} \tag{6.58}$$

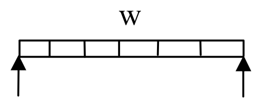
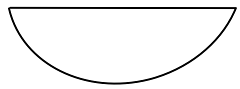
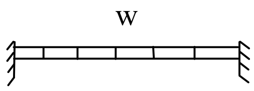
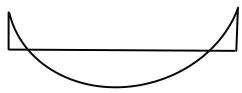
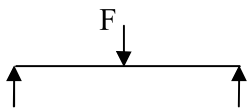
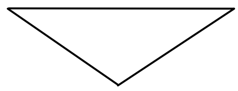
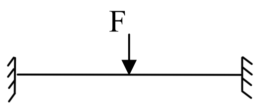
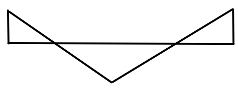
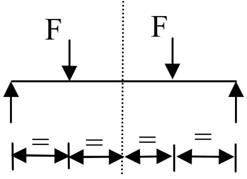
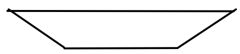
M_1 = The smaller of the two end moments at the lateral supports of the beam

M_2 = The larger of the two end moments at the lateral supports of the beam

In the above equation, if M_1 and M_2 have the same sign which means the beam is double curvature, (M_1 / M_2) is negative and if M_1 and M_2 have the different sign which means the beam is single curvature, (M_1 / M_2) is positive.

The values of C_1 for transverse loading can be obtained from below table;

Table 6.1 C_1 values for end moment loading

Loading and Support Conditions	Bending Moment Diagram	Value of C_1
		1,132
		1,285
		1,365
		1,565
		1,046

The imperfection factor (α_{LT}) can be obtained from below table;

Table 6.2 Imperfection factors

Buckling Curve	a	b	c	d
Imperfection Factor (α_{LT})	0,21	0,34	0,49	0,76

The lateral torsional buckling curve for a cross section can be obtained from below table;

Table 6.3 Selection of lateral torsional buckling curve for cross sections

Cross-Section	Limits	Buckling Curves
Rolled I Sections	$h/b \leq 2$ $h/b > 2$	a b
Welded I Sections	$h/b \leq 2$ $h/b > 2$	c d
Other Cross-Sections	——	d

CHAPTER 7

AXIAL COMPRESSION AND FLEXURE IN MEMBERS

7.1 Design of Beam-Column Members for TS 648

In Course 485 Class Notes by Yılmaz Çetin and Polat Uğur [23], treatment of beam-columns is complex due to:

- Presence of secondary moments
- Interaction between the instability caused by axial compression and flexure

In order to be able to design beam-columns according to TS 648, interaction equations are used.

If the axial force is tensile, the interaction equation can be formulated as below:

$$\frac{\sigma_{\text{eç}}}{\sigma_{\text{çem}}} + \frac{\sigma_{\text{bx}}}{\sigma_{\mathbf{B}\text{x}}} + \frac{\sigma_{\text{by}}}{\sigma_{\text{By}}} \leq 1{,}0 \qquad (7.1)$$

in which

$\sigma_{\text{eç}}$ = Computed tensile stress

$\sigma_{\text{çem}}$ = Allowable tensile stress

σ_{bx} and σ_{by} = Computed bending stresses about major and minor axes

$\sigma_{\mathbf{B}\text{x}}$ and $\sigma_{\mathbf{B}y}$ = Allowable bending stresses for bending about major and minor axes

If the axial force is compression, the interaction equations can be formulated as below;

$$\frac{\sigma_{eb}}{0,6*\sigma_y}+\frac{\sigma_{bx}}{\sigma_{Bx}}+\frac{\sigma_{by}}{\sigma_{By}}\leq 1,0 \tag{7.2}$$

$$\frac{\sigma_{eb}}{\sigma_{bem}}+\frac{C_{mx}}{(1-\frac{\sigma_{eb}}{\sigma_{e'x}})}*\frac{\sigma_{bx}}{\sigma_{Bx}}+\frac{C_{my}}{(1-\frac{\sigma_{eb}}{\sigma_{e'y}})}*\frac{\sigma_{by}}{\sigma_{By}}\leq 1,0 \tag{7.3}$$

If $\frac{\sigma_{eb}}{\sigma_{bem}}\leq 0,15$, moment magnification factors can be neglected and the following formula is only used;

$$\frac{\sigma_{eb}}{\sigma_{bem}}+\frac{\sigma_{bx}}{\sigma_{Bx}}+\frac{\sigma_{by}}{\sigma_{By}}\leq 1,0 \tag{7.4}$$

in which

σ_{eb} = Computed axial compression stress

σ_{bem} = Allowable compression stress under axial load only

$\sigma_{e'x}$ = Critical stresses about major axis

$\sigma_{e'y}$ = Critical stresses about minor axis

C_{mx} and C_{my} = Coefficients reflecting moment gradient and lateral support condition

σ_y = Yield stress of used steel

Critical stresses ($\sigma_{e'x}$ and $\sigma_{e'y}$) can be computed as below equations:

$$\sigma_{e'x} = \frac{2 * \pi^2 * E}{5 * \lambda_x^2} \quad \text{and} \quad \sigma_{e'y} = \frac{2 * \pi^2 * E}{5 * \lambda_y^2} \tag{7.5}$$

Coefficients reflecting moment gradient and lateral support condition (C_{mx} and C_{my}) can be expressed as below;

- In frames with sidesway permitted C_m = 0,85
- In frames with sidesway prevented and no span loads:

$$C_m = 0{,}6 - 0{,}4 * (\frac{M_1}{M_2}) \geq 0{,}4 \tag{7.6}$$

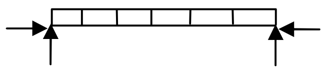
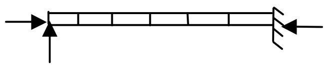
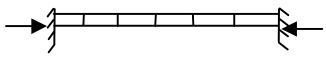
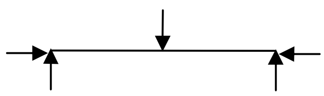
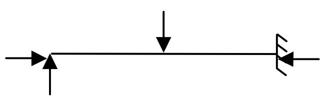
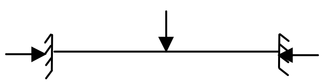
M_1 and M_2 are the small end moment and the large end moment. The ratio of M_1 / M_2 is taken as positive for double curvature, the ratio of M_1 / M_2 is taken as negative for single curvature [1].

- In frames with no sidesway and with span loading ;

$$C_m = 1 - \psi * \frac{\sigma_{eb}}{\sigma'_e} \tag{7.7}$$

ψ values can be obtained from below Table 7.1.

Table 7.1 ψ values and C_m

CONDITION	ψ	C_m
	0	1,0
	-0,3	$1-0,3*\frac{\sigma_{eb}}{\sigma'_e}$
	-0,4	$1-0,4*\frac{\sigma_{eb}}{\sigma'_e}$
	-0,2	$1-0,2*\frac{\sigma_{eb}}{\sigma'_e}$
	-0,4	$1-0,4*\frac{\sigma_{eb}}{\sigma'_e}$
	-0,6	$1-0,6*\frac{\sigma_{eb}}{\sigma'_e}$

7.2 Design of Beam-Column Members for LRFD

For doubly and singly symmetric members subjected to flexure and compression, below equations are used;

If $\frac{P_r}{P_c} \geq 0,2$

$$\frac{P_r}{P_c} + \frac{8}{9} * (\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}) \leq 1,0 \qquad (7.8)$$

If $\frac{P_r}{P_c} < 0,2$

$$\frac{P_r}{2 * P_c} + (\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}) \leq 1,0 \qquad (7.9)$$

in which

P_r = Required axial compressive strength using LRFD load combinations

$P_c = \phi_c * P_n$ = Design axial compressive strength (obtain from Chapter 5.2)

M_r = Required flexural strength using LRFD load combinations

$M_c = \phi_b * M_n$ = Design flexural strength (obtain from Chapter 6.2)

ϕ_b = Resistance factor for compression = 0,9

ϕ_c = Resistance factor for flexure = 0,9

For doubly and singly symmetric members subjected to flexure and tension, Equation (7.8) and Equation (7.9) are used in the same way.

To calculate the required flexural strength using LRFD load combinations (M_r) and the required axial compressive strength using LRFD load combinations (P_r), there are two options.

1. General Second-Order Elastic Analysis :

Any second-order elastic analysis method may be used.

2. Second-Order Analysis by Amplified First-Order Elastic Analysis :

The required flexural strength using LRFD load combinations (M_r) and the required axial compressive strength using LRFD load combinations (P_r) can be expressed as follows ;

$$M_r = \beta_1 * M_{nt} + \beta_2 * M_{lt} \tag{7.10}$$

$$P_r = P_{nt} + \beta_2 * P_{lt} \tag{7.11}$$

in which

$$\beta_1 = \frac{C_m}{1 - \alpha * P_r / P_{el}} \geq 1 \tag{7.12}$$

$$\beta_2 = \frac{1}{1 - \dfrac{\alpha * \sum P_{nt}}{\sum P_{e2}}} \geq 1 \tag{7.13}$$

M_{nt} = First-order moment using LRFD load combinations (assuming no lateral translation of the frame)

M_{lt} = First-order moment using LRFD load combinations caused by lateral translation of the frame only

P_{nt} = First-order axial force using LRFD load combinations (assuming no lateral translation of the frame)

P_{lt} = First-order axial force using LRFD load combinations caused by lateral translation of the frame only

$\sum P_{nt}$ = Total vertical load supported by the story using LRFD load combinations

C_m = A coefficient assuming no lateral translation of the frame

P_{el} = Elastic critical buckling resistance of the member

$\sum P_{e2}$ = Elastic critical buckling resistance for the story determined by sidesway buckling analysis

α = 1,0

The coefficient assuming no lateral translation of the frame (C_m) can be expressed as below;

- For beam-columns in frames with sidesway prevented and no span loads ;

$$C_m = 0,6 - 0,4 * (\frac{M_1}{M_2}) \tag{7.14}$$

M_1 and M_2 are the small end moment and the large end moment. The ratio of M_1 / M_2 is taken as positive for double curvature, the ratio of M_1 / M_2 is taken as negative for single curvature.

- For beam-columns subjected to transverse loading between supports

$$C_m = 1,0 \tag{7.15}$$

7.3 Design of Beam-Column Members for Eurocode 3

When members subject to combined bending and axial compression, both following equations are satisfied (omitting for Class 4 sections) according to Eurocode 3 [3].

$$\frac{N_{Ed}}{X_x * N_{Rk} / \gamma_{M1}} + k_{xx} * \frac{M_{x,Ed}}{X_{LT} * M_{x,Rk} / \gamma_{M1}} + k_{xy} * \frac{M_{y,Ed}}{M_{y,Rk} / \gamma_{M1}} \leq 1{,}0 \qquad (7.16)$$

$$\frac{N_{Ed}}{X_y * N_{Rk} / \gamma_{M1}} + k_{yx} * \frac{M_{x,Ed}}{X_{LT} * M_{x,Rk} / \gamma_{M1}} + k_{yy} * \frac{M_{y,Ed}}{M_{y,Rk} / \gamma_{M1}} \leq 1{,}0 \qquad (7.17)$$

in which

N_{Ed} , $M_{x,Ed}$ and $M_{y,Ed}$ = The design value of the compression force and the maximum moments

N_{Rk} , $M_{x,Rk}$ and $M_{y,Rk}$ = The characteristic values of the compression resistance of the cross section and the bending moment resistance of the cross section

X_x and X_y = The reduction factors due to flexural buckling

X_{LT} = The reduction factors due to lateral torsional buckling

k_{xx} , k_{yy} , k_{xy} , k_{yx} = The interaction factors

The interaction factors (k_{xx} , k_{yy} , k_{xy} , k_{yx}) are obtained from below tables ;

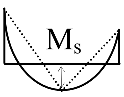
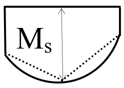
Table 7.2 Interaction factors for members not susceptible to torsional deformations

Interaction Factors	Type of Sections	Design Assumptions	
		Class 3 and Class 4	Class 1 and Class 2
k_{xx}	I Sections	$C_{mx} * \left[1 + (0,6 * \overline{\lambda}_x * \frac{N_{Ed}}{X_x * N_{Rk}/\gamma_{M1}}\right] \leq C_{mx} * \left[1 + 0,6 * \frac{N_{Ed}}{X_x * N_{Rk}/\gamma_{M1}}\right]$	$C_{mx} * \left[1 + (\overline{\lambda}_x - 0,2) * \frac{N_{Ed}}{X_x * N_{Rk}/\gamma_{M1}}\right] \leq C_{mx} * \left[1 + 0,8 * \frac{N_{Ed}}{X_x * N_{Rk}/\gamma_{M1}}\right]$
k_{xy}	I Sections	k_{yy}	$0,6 * k_{yy}$
k_{yx}	I Sections	$0,8 * k_{xx}$	$0,6 * k_{xx}$
k_{yy}	I Sections	$C_{my} * \left[1 + 0,6 * \overline{\lambda}_y * \frac{N_{Ed}}{X_y * N_{Rk}/\gamma_{M1}}\right] \leq C_{my} * \left[1 + 0,6 * \frac{N_{Ed}}{X_y * N_{Rk}/\gamma_{M1}}\right]$	$C_{my} * \left[1 + (2 * \overline{\lambda}_y - 0,6) * \frac{N_{Ed}}{X_y * N_{Rk}/\gamma_{M1}}\right] \leq C_{my} * \left[1 + 1,4 * \frac{N_{Ed}}{X_y * N_{Rk}/\gamma_{M1}}\right]$

Table 7.3 Interaction factors for member susceptible to torsional deformations

Interaction Factors	Type of Sections	Design Assumptions	
		Class 3 and Class 4	Class 1 and Class 2
k_{xx}	I Sections	$C_{mx} * \left[1 + (0,6 * \bar{\lambda}_x * \frac{N_{Ed}}{X_x * N_{Rk}/\gamma_{M1}}\right]$ $\leq C_{mx} * \left[1 + 0,6 * \frac{N_{Ed}}{X_x * N_{Rk}/\gamma_{M1}}\right]$	$C_{mx} * \left[1 + (\bar{\lambda}_x - 0,2) * \frac{N_{Ed}}{X_x * N_{Rk}/\gamma_{M1}}\right]$ $\leq C_{mx} * \left[1 + 0,8 * \frac{N_{Ed}}{X_x * N_{Rk}/\gamma_{M1}}\right]$
k_{xy}	I Sections	k_{yy}	$0,6 * k_{yy}$
k_{yx}	I Sections	$1 - \frac{0,05 * \bar{\lambda}_y}{C_{MLT} - 0,25} * \frac{N_{Ed}}{X_y * N_{Rk}/\gamma_{M1}}$ $\geq 1 - \frac{0,05}{C_{MLT} - 0,25} * \frac{N_{Ed}}{X_y * N_{Rk}/\gamma_{M1}}$	$for \bar{\lambda}_y \geq 0,4;$ $1 - \frac{0,1 * \bar{\lambda}_y}{C_{MLT} - 0,25} * \frac{N_{Ed}}{X_y * N_{Rk}/\gamma_{M1}}$ $\geq 1 - \frac{0,1}{C_{MLT} - 0,25} * \frac{N_{Ed}}{X_y * N_{Rk}/\gamma_{M1}}$ $for \bar{\lambda}_y < 0,4;$ $0,6 + \bar{\lambda}_y \leq 1 - \frac{0,1 * \bar{\lambda}_y}{C_{MLT} - 0,25} * \frac{N_{Ed}}{X_y * N_{Rk}/\gamma_{M1}}$
k_{yy}	I Sections	$C_{my} * \left[1 + 0,6 * \bar{\lambda}_y * \frac{N_{Ed}}{X_y * N_{Rk}/\gamma_{M1}}\right]$ $\leq C_{my} * \left[1 + 0,6 * \frac{N_{Ed}}{X_y * N_{Rk}/\gamma_{M1}}\right]$	$C_{my} * \left[1 + (2 * \bar{\lambda}_y - 0,6) * \frac{N_{Ed}}{X_y * N_{Rk}/\gamma_{M1}}\right]$ $\leq C_{my} * \left[1 + 1,4 * \frac{N_{Ed}}{X_y * N_{Rk}/\gamma_{M1}}\right]$

Table 7.4 Equivalent uniform moment factors C_m

Moment Diagram	Range		C_{mx} and C_{my} and C_{mLT}	
			Uniform Loading	Concentrated Load
M  ψM	$-1 \le \psi \le 1$		$0,6 + 0,4 * \psi \ge 0,4$	
M_h  ψM_h $\alpha_s = M_s / M_h$	$0 \le \alpha_s \le 1$	$-1 \le \psi \le 1$	$0,2 + 0,8*\alpha_s \ge 0,4$	$0,2 + 0,8*\alpha_s \ge 0,4$
	$-1 \le \alpha_s \le 0$	$0 \le \psi \le 1$	$0,1 - 0,8*\alpha_s \ge 0,4$	$- 0,8*\alpha_s \ge 0,4$
		$-1 \le \psi \le 0$	$0,1(1 - \psi) - 0,8*\alpha_s \ge 0,4$	$0,2(- \psi) - 0,8*\alpha_s \ge 0,4$
M_h  ψM_h $\alpha_h = M_h / M_s$	$0 \le \alpha_h \le 1$	$-1 \le \psi \le 1$	$0,95 + 0,05*\alpha_h$	$0,90 + 0,10*\alpha_h$
	$-1 \le \alpha_h \le 0$	$0 \le \psi \le 1$	$0,95 + 0,05*\alpha_h$	$0,90 + 0,10*\alpha_h$
		$-1 \le \psi \le 0$	$0,95 + 0,05*\alpha_h*(1+2*\psi)$	$0,90 - 0,10*\alpha_h*(1+2*\psi)$
For members with sway buckling mode the equivalent uniform moment factor should be taken C_{mx} = 0,9 or C_{my} = 0,9 respectively.				

CHAPTER 8

LINK BEAMS

8.1 Design of Link Beams for TS 648

When a link beam is designed according to TS 648, TS 648 refers to TEC 2007. In TEC 2007 [4], the length of the link (e_{link}) beam must be within the following limit.

$$e_{link,\min} = 1{,}0 * \frac{M_p}{V_p} \le e_{link} \le e_{link,\max} = 5{,}0 * \frac{M_p}{V_p} \tag{8.1}$$

where

M_p = Bending moment capacity

V_p = Shear force capacity

The bending moment capacity (M_p) and the shear force capacity (V_p) can be calculated as below formulae;

$$M_p = W_p * \sigma_y \tag{8.2}$$

$$V_p = 0{,}6 * \sigma_y * A_k \tag{8.3}$$

where

W_p = Section modulus of cross-sections

σ_y = Yield stress of the material

A_k = Shear area of cross-sections

$$A_k = (h - 2 * t_f) * t_w \tag{8.4}$$

The design shear force of the link beam (V_d) shall satisfy the both of below conditions;

$$V_d \leq V_p \tag{8.5}$$

$$V_d \leq \frac{2 * M_p}{e_{link}} \tag{8.6}$$

If $N_d / (\sigma_a * A) > 0{,}15$, reduced shear force capacity (V_{pn}) and reduced bending moment capacity (M_{pn}) are used in Equation (8.5) and (8.6) instead of V_p and M_p. The reduced shear force capacity (V_{pn}) and the reduced bending moment capacity (M_{pn}) can be expressed as below;

$$M_{pn} = 1{,}18 * M_p * \left[1 - \frac{N_d}{\sigma_y * A}\right] \tag{8.7}$$

$$V_{pn} = V_p * \sqrt{1 - (N_d / \sigma_y * A)^2} \tag{8.8}$$

where

N_d = Design axial force of link beam

A = Area of cross sections

8.2 Design of Link Beams for LRFD

When a link beam is designed according to LRFD, LRFD refers to AISC Standard 341-05 [7] (Seismic Provisions for Structural Steel Buildings).

The link design shear strength ($\Phi_v * V_n$) must be greater than the required shear strength based on LRFD load combinations (V_u or V_r). It can be expressed as;

$$V_u \leq \Phi_v * V_n \quad (8.9)$$

where

V_n = Nominal shear strength of the link beam

Φ_v = Resistance factor for shear (Φ_v = 0,9)

The nominal shear strength of the link beam (V_n) can be expressed as ;

$$V_n = \min (V_p ; 2*M_p/e) \quad (8.10)$$

where

$$V_p = 0,6 * F_y * A_w \quad (8.11)$$

$$M_p = F_y * Z \quad (8.12)$$

$$A_w = (d - 2 * t_f) * t_w \quad (8.13)$$

e = length of link beam

If $P_u \leq 0,15 * P_y$, the effect of axial force on the shear strength of the link beam can be neglected.

where

$P_u = P_r$ = required axial strength based on LRFD load combinations

$P_y = P_c$ = Nominal axial yield strength

$$P_y = A_g * F_y \tag{8.14}$$

If $P_u > 0,15 * P_y$, the following requirements must be satisfied;

- The available shear strength of the link beam (V_n) can be expressed as ;

$$\Phi_v * V_n = \min(\Phi_v * V_{pa} \,;\, 2 * \Phi_v * M_{pa} / e) \tag{8.15}$$

where

$$V_{pa} = V_p * \sqrt{1 - (P_r / P_c)^2} \tag{8.16}$$

$$M_{pa} = 1,18 * M_p * \left[1 - \frac{P_r}{P_c}\right] \tag{8.17}$$

- The length of the link beam should not exceed ;

If $\rho' * (A_w / A_g) \geq 0,3$;

$$e_{link,max} = [1,15 - 0,5 * \rho' * (A_w / A_g)] * 1,6 * M_p / V_p \tag{8.18}$$

If $\rho' * (A_w / A_g) < 0,3$;

$$e_{link,max} = 1,6 * M_p / V_p \tag{8.19}$$

where

$$\rho' = P_r / V_r \tag{8.20}$$

A_g = Gross area of the link beam

8.3 Design of Link Beams for Eurocode 3

When a link beam is designed according to Eurocode 3, Eurocode 3 refers to Eurocode 8 [8]. The bending moment capacity ($M_{p,link}$) and the shear force capacity ($V_{p,link}$) can be calculated as below formulae ;

$$M_{p,Link} = f_y * b * t_f * (h - t_f) \tag{8.21}$$

$$V_{p,Link} = (f_y / \sqrt{3}) * t_w * (h - t_f) \tag{8.22}$$

If $\frac{N_{Ed}}{N_{pl,Rd}} < 0{,}15$, the design resistance of the link beam shall satisfy the both of below conditions ;

$$V_{Ed} \leq V_{p,Link} \tag{8.23}$$

$$M_{Ed} \leq M_{p,Link} \tag{8.24}$$

in which

N_{Ed} , M_{Ed} and V_{Ed} = Design axial force, design bending moment and design shear

If $\frac{N_{Ed}}{N_{pl,Rd}} \geq 0{,}15$, reduced shear force capacity ($V_{p,Link,r}$) and reduced bending moment capacity ($M_{p,Link,r}$) are used in Equation (8.23) and (8.24) instead of $V_{p,Link}$ and $M_{p,Link}$. The reduced shear force capacity ($V_{p,Link,r}$) and the reduced bending moment capacity ($M_{p,Link,r}$) can be expressed as below ;

$$V_{p,Link,r} = V_{p,Link} * \sqrt{1 - (N_{Ed} / N_{pl,Rd})^2} \tag{8.25}$$

$$M_{p,Link,r} = M_{p,Link} * \left[1 - \frac{N_{Ed}}{N_{pl,Rd}}\right] \tag{8.26}$$

If $\frac{N_{Ed}}{N_{pl,Rd}} \geq 0{,}15$, the length of the link beam (e_{Link}) should not exceed ;

If $R < 0{,}3$;

$$e_{Link} \leq e_{Link,max} = 1{,}6 * M_{p,Link} / V_{p,Link} \tag{8.27}$$

If $R \geq 0{,}3$;

$$e_{Link} \leq e_{Link,max} = [1{,}15 - 0{,}5 * R] * 1{,}6 * M_{p,Link} / V_{p,Link} \tag{8.28}$$

in which

$$R = N_{Ed} * t_w * (d - 2*t_f) / (V_{Ed} * A) \tag{8.29}$$

A = Gross area of the link beam

CHAPTER 9

CALCULATION OF EARTHQUAKE LOADS

9.1 Calculation of Earthquake Loads for TS 648

When the earthquake loads are calculated according to TS 648, TS 648 refers to TEC 2007 [4]. In this thesis, equivalent seismic load method is used to calculate the earthquake loads. The applicability conditions are given in the below table for the equivalent seismic load method;

Table 9.1 Buildings for which equivalent seismic load method is applicable

Seismic Zone	Type of Building	Total Building Height Limit
1, 2	Buildings without extreme torsional irregularity depending on a criterion in TEC 2007	25 m
1, 2	Buildings without extreme torsional irregularity depending on two criteria in TEC 2007	40 m
3, 4	All buildings	40 m

Total Equivalent Seismic Load (base shear), V_t, can be expressed as below formula;

$$V_t = \frac{W_T * A(T)}{R_a(T)} \geq 0{,}1 * A_0 * I * W_T \qquad (9.1)$$

where

W_T = Total building weight

A(T) = Spectral acceleration coefficient

R_a = Seismic load reduction factor

A_0 = Effective ground acceleration coefficient

I = Importance Factor

Total building weight (W_T) to be used in Eq.(9.1) can be expressed below formula ;

$$W_T = \sum_{i=1}^{N} w_i \qquad (9.2)$$

$$w_i = g_i + n * q_i \qquad (9.3)$$

where

g_i = Total dead load at i'th storey of building

q_i = Total live load at i'th storey of building

n = Live load participation factor

Live load participation factor (n) is given in below table;

Table 9.2 Live load participation factor (n)

Purpose of Occupancy of Building	n
Depot, warehouse, etc.	0,8
School, dormitory, sport facility, cinema, theatre, concert hall, car park, restaurant, shop, etc.	0,6
Residence, office, hotel, hospital, etc.	0,3

The spectral acceleration coefficient, A(T), given in Eq.(9.1) can be written as ;

$$A(T) = A_0 * I * S(T) \tag{9.4}$$

where

S(T) = Spectrum coefficient

The spectrum coefficient, S(T), to be used in Eq.(9.4) can be expressed as below formulae depending on local site conditions and the building natural period ;

$$S(T) = 1 + 1,5 * \frac{T}{T_A} \qquad (0 \leq T \leq T_A) \tag{9.5}$$

$$S(T) = 2,5 \qquad (T_A < T \leq T_B) \tag{9.6}$$

$$S(T) = 2,5 * (\frac{T_B}{T})^{0,8} \qquad (T_B < T) \tag{9.7}$$

The below figure clearly shows the relationship between T (building natural period) and S(T).

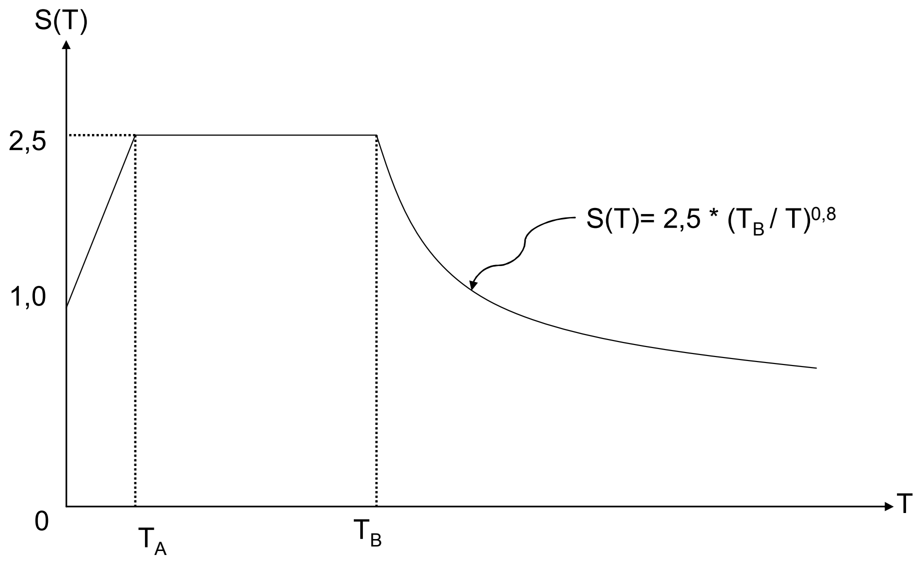


Figure 9.1 Shape of Design Spectrum

The effective ground acceleration coefficient (A_0) to be used in Eq. (9.1) and Eq. (9.4) which depending on seismic zone derived from zonation maps can be obtained from below table;

Table 9.3 Effective Ground Acceleration Coefficient (A_0)

Seismic Zone	A_0
1	0,40
2	0,30
3	0,20
4	0,10

The importance factor (I) to be used in Eq. (9.1) and Eq. (9.4) can be obtained from below table;

Table 9.4 Building Importance Factor (I)

Purpose of Occupancy or Type of Building	Importance Factor (I)
a) Buildings required to be utilised immediately after the earthquake (Hospitals, dispensaries, health wards, fire fighting buildings and facilities, PTT and other telecommunication facilities, transportation stations and terminals, power generation and distribution facilities; governorate, county and municipality administration buildings, firstaid and emergency planning stations) b) Buildings containing or storing toxic, explosive and flammable materials, etc.	1,50
2. Intensively and long-term occupied buildings and buildings preserving valuable goods : a) Schools, other educational buildings and facilities, dormitories and hostels, military barracks, prisons, etc. b) Museums	1,40
3. Intensively but short-term occupied buildings : Sport facilities, cinema, theatre and concert halls, etc.	1,20
4. Other buildings : Buildings other than above-defined buildings. (Residential and office buildings, hotels, building-like industrial structures, etc.)	1,00

Spectrum Characteristic Periods, T_A and T_B, shown in Eq. 9.5, Eq. 9.6 and Eq. 9.7 are specified in below table, depending on local site classes;

Table 9.5 Spectrum Characteristic Periods (T_A and T_B)

Local Site Class	T_A (Second)	T_B (Second)
Z1	0,10	0,30
Z2	0,15	0,40
Z3	0,15	0,60
Z4	0,20	0,90

The seismic load reduction factor, $R_a(T)$, shall be determined by below formulae in terms of structural behavior factor, R, for various structural systems, and the natural vibration period T.

$$R_a(T) = 1{,}5 + (R - 1{,}5) + \frac{T}{T_A} \qquad (0 \leq T \leq T_A) \qquad (9.8)$$

$$R_a(T) = R \qquad (T_A < T) \qquad (9.9)$$

The structural behavior factor, R, can be obtained from Table 9.6;

To determine the design seismic loads acting at storey levels, the below formulae are used;

$$V_t = \Delta F_N + \sum_{i=1}^{N} F_i \qquad (9.10)$$

$$\Delta F_N = 0{,}0075 * N * V_t \qquad (9.11)$$

$$F_i = (V_t - \Delta F_N) * \frac{w_i * H_i}{\sum_{j=1}^{N} w_j * H_j} \qquad (9.12)$$

Table 9.6 Structural Behavior Factors (R)

STRUCTURAL STEEL BUILDINGS	System of Nominal Ductility Level	System of High Ductility Level
Buildings in which seismic loads are fully resisted by frames	5	8
Buildings in which seismic loads are fully resisted by single-storey frames with columns hinged at top	.--------	4
Buildings in which seismic loads are fully resisted by braced frames or cast-in-situ reinforced concrete structural walls :		
(a) Concentrically braced frames......................................	4	5
(b) Eccentrically braced frames..	--------	7
(c) Reinforced concrete structural walls............................	4	6
Buildings in which seismic loads are jointly resisted by frames and braced frames or cast-in-situ reinforced concrete structural walls		
(a) Concentrically braced frames......................................	5	6
(b) Eccentrically braced frames..	--------	8
(c) Reinforced concrete structural walls............................	4	7

in which

F_i = Design seismic load acting at i'th storey in equivalent seismic load method

ΔF_N = Additional equivalent seismic load acting on the N'th storey (top) of building

N = Total number of stories of building from the foundation level

H_i = Height of i'th storey of building measured from the top foundation level

w_i = Weight of i'th storey of building by considering live load participation factor

V_t = Total equivalent seismic load (calculated by Eq. (9.1))

At each floor, equivalent seismic loads determined in accordance with Eq (9.12) shall be applied to the floor mass centre as well as to the points defined by shifting it +5% and −5% of the floor length in the perpendicular direction to the earthquake direction considered in order to account for the additional eccentricity effects [4].

To determine the first natural vibration period of the building to be used in Eq.(9.1), Eq.(9.4), Eq.(9.5), Eq.(9.6), Eq.(9.7), Eq.(9.8) and Eq.(9.9) the below formula is used ;

$$T_1 = 2 * \pi * \left(\frac{\sum_{i=1}^{N} m_i * d_{fi}^{\ 2}}{\sum_{i=1}^{N} F_{fi} * d_{fi}} \right)^{1/2} \tag{9.13}$$

in which

F_{fi} = Fictitious load acting at i'th storey in the determination of fundamental natural vibration period

d_{fi} = Displacement calculated at i'th storey of building under fictitious loads F_{fi}

Under the combined effects of independently acting x and y direction earthquakes to the structural system, internal forces in element principal axis a and b shall be obtained by below equations such that the most unfavorable results yield and the below figure can explain the case [4].

$$B_a = \mp B_{ax} \mp 0,3 * B_{ay} \quad \text{or} \quad B_a = \mp 0,3 * B_{ax} \mp B_{ay} \tag{9.14}$$

$$B_b = \mp B_{bx} \mp 0,3 * B_{by} \quad \text{or} \quad B_b = \mp 0,3 * B_{bx} \mp B_{by} \tag{9.15}$$

in which

B_a = Design internal force component of a structural element in the direction of its principal axis a

B_{ax} = Internal force component of a structural element in the direction of its principal axis a due to earthquake in x direction

B_{ay} = Internal force component of a structural element in the direction of its principal axis a due to earthquake in y direction perpendicular to x direction

B_b = Design internal force quantity of a structural element in principal direction b

B_{bx} = Internal force component of a structural element in the direction of principal axis b due to earthquake in x direction

B_{by} = Internal force component of a structural element in the direction of principal axis a due to earthquake in y direction perpendicular to x direction

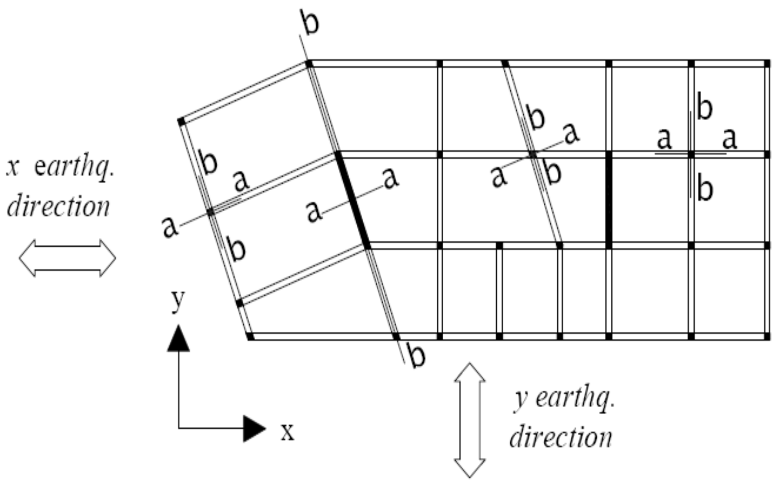


Figure 9.2 Acting x and y direction earthquakes to the structural system

To calculate effective storey drifts (δ_i) for buildings, the below formulae are used ;

$$\Delta_i = d_i - d_{i-1} \tag{9.16}$$

$$\delta_i = R * \Delta_i \tag{9.17}$$

in which

Δ_i = Reduced storey drift

d_i = Displacement calculated at i'th storey of building under design seismic loads

R = Structural behavior factor

The maximum value of effective storey drifts, $(\delta_i)_{max}$, obtained for each earthquake direction by Eq.(9.17) at columns or structural walls of a given i'th storey of a building shall satisfy the condition given by below equation ;

$$\frac{(\delta_i)_{\max}}{h_i} \leq 0,02 \qquad (9.18)$$

In TEC 2007 [4], there are some additional requirements for steel structures as follows;

- In the earthquake zone 1 and zone 2, columns shall satisfy an additional strength requirements for axial tension and compression forces (ignoring the bending moments) which are due to considering under the increased loading cases based on Eq.(9.21) and Eq.(9.22) while satisfying the conditions for axial forces and bending moments emerged by both earthquake effect and vertical load. Axial tension and compression capacities of the cross sections of the columns under the increased earthquake effect are determined according to the below Equations.

$$N_{bp} = 1,7 * \sigma_{bem} * A \qquad (9.19)$$

$$N_{çp} = \sigma_y * A_n \qquad (9.20)$$

where

N_{bp} and $N_{çp}$ = Axial compression capacity and axial tension capacity

A_n = Net cross sectional area

σ_{bem} = Allowable compressive stress

Increased earthquake effect shall be the maximum one of below equations;

$$G + Q \pm \Omega_0 * E \qquad (9.21)$$

$$0,9 * G \pm \Omega_0 * E \qquad (9.22)$$

The earthquake magnification coefficients (Ω_0) can be obtained from below table for the Eq. (9.21) and Eq. (9.22);

Table 9.7 Magnification Coefficients (Ω_0)

Structral System Type	Ω_0
Steel Frame with High Ductility	2,5
Steel Frame with Nominal Ductility	2,0
Steel Concentrically Braced Frame (with High or Nominal Ductility)	2,0
Steel Eccentrically Braced Frame	2,5

- Slenderness ratio of the all braces in steel eccentrically braced frame with high ductility shall not exceed $4{,}0 * \sqrt{E/\sigma_y}$. It can be formulated as below ;

$$\lambda_{brace} = \frac{k * L}{i_{\min}} < 4{,}0 * \sqrt{E/\sigma_y} \tag{9.23}$$

- Limiting width-thickness ratios for cross-sections of beams and columns of the system with high ductility frame are tabulated in below Table 9.8.

Table 9.8 Limiting Width-Thickness Ratios (TEC 2007)

Description of Element	Width Thickness Ratio	Limiting Width-Thickness Ratios	
		High Ductility System	Nominal Ductility System
Flexure and Uniform Compression in I-Shaped Sections	b/2t	$0,3*\sqrt{\frac{E}{\sigma_y}}$	$0,5*\sqrt{\frac{E}{\sigma_y}}$
Flexure in I-Shaped Sections	h/t_w	$3,2*\sqrt{\frac{E}{\sigma_y}}$	$5,0*\sqrt{\frac{E}{\sigma_y}}$
Flexure and Uniform Compression in I-Shaped Sections	h/t_w	For $\left\lvert\frac{N_d}{\sigma_y * A}\right\rvert \leq 0,10$ $3,2*\sqrt{\frac{E}{\sigma_y}}*(1-1,7*\left\lvert\frac{N_d}{\sigma_y * A}\right\rvert)$	For $\left\lvert\frac{N_d}{\sigma_y * A}\right\rvert \leq 0,10$ $5,0*\sqrt{\frac{E}{\sigma_y}}*(1-1,7*\left\lvert\frac{N_d}{\sigma_y * A}\right\rvert)$
		For $\left\lvert\frac{N_d}{\sigma_y * A}\right\rvert > 0,10$ $1,33*\sqrt{\frac{E}{\sigma_y}}*(2,1-\left\lvert\frac{N_d}{\sigma_y * A}\right\rvert)$	For $\left\lvert\frac{N_d}{\sigma_y * A}\right\rvert > 0,10$ $2,08*\sqrt{\frac{E}{\sigma_y}}*(2,1-\left\lvert\frac{N_d}{\sigma_y * A}\right\rvert)$
Flexure or Uniform Compression in Rectangular HSS	b/t or h/t_w	$0,7*\sqrt{\frac{E}{\sigma_y}}$	$1,2*\sqrt{\frac{E}{\sigma_y}}$

9.2 Calculation of Earthquake Loads for LRFD

When the earthquake loads are calculated according to LRFD, LRFD refers to ASCE 7-05 and AISC Standard 341-05. In this thesis, equivalent force analysis method is used to calculate the earthquake loads. The applicability conditions are given in the below table for the equivalent force analysis method;

Table 9.9 Permitted Analytical Procedures

Seismic Design Category	Structural Characteristics	Equivalent Force Analysis	Modal Response Spectrum Analysis	Seismic Response History Procedures
B, C	Occupancy Category I or II buildings of light-framed construction not exceeding 3 stories in height	P	P	P
	Other Occupancy Category I or II buildings not exceeding 2 stories in height	P	P	P
	All other structures	P	P	P
D, E, F	Occupancy Category I or II buildings of light-framed construction not exceeding 3 stories in height	P	P	P
	Other Occupancy Category I or II buildings not exceeding 2 stories in height	P	P	P
	Regular structures with $T<3,5*T_s$ and all structures of light frame construction	P	P	P
	Irregular structures with $T<3,5*T_s$ and having only special horizontal or vertical irregularities type	P	P	P
	All other structures	NP	P	P
P = Permitted; NP = Not Permitted				

The occupancy categories of buildings to be mentioned in Table 9.9 can be obtained from below table.

Table 9.10 Occupancy Category of Buildings

Nature of Occupancy	Occupancy Category
Buildings and other structures that represent a low hazard to human life in the event of failure, including but not limited to : • Agricultural facilities • Certain temporary facilities • Minor storage facilities	I
All buildings and other structures except those listed in Occupancy Categories I,III,IV	II
Buildings and other structures that represent a substantial hazard to human life in the event of failure, including but not limited to : • Buildings and other structures where more than 300 people congregate in one area • Buildings and other structures with daycare facilities with a capacity greater than 150 • Buildings and other structures with elementary or secondary school facilities with a capacity greater than 250 • Buildings and other structures with elementary or secondary school facilities with a capacity greater than 250 • Buildings and other structures with a capacity greater than 500 for collages or adult education facilities • Health care facilities with a capacity of 50 or more patients	III
Buildings and other structures designated as essential facilities ,including but not limited to : • Hospitals and other health care facilities • Fire, rescue, ambulance and police stations • Designated earthquake, hurricane or other emergency shelters • Designated emergency preparedness, communication, and operation centers • Aviation control towers, air traffic control centers	IV

The seismic base shear, V, can be expressed as below formula for the equivalent force method;

$$V = C_s * W \tag{9.24}$$

in which

C_s = Seismic response coefficient

W = Effective seismic weight

The effective seismic weight of a structure (W) is equal to the total dead load and the seismic response coefficient (C_s) can be written as below formula;

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)} \tag{9.25}$$

where

S_{DS} = Design spectral response acceleration parameter in the short period range

R = Response modification coefficient

I = Importance factors

The seismic response coefficient (C_s) computed in accordance with Eq. (9.25) need not exceed the following values:

$$Cs = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} \qquad \text{for} \quad T \leq T_L \tag{9.26}$$

$$Cs = \frac{S_{D1} * T_L}{T^2\left(\frac{R}{I}\right)} \qquad \text{for} \quad T > T_L \tag{9.27}$$

C_s shall not be less than;

$$C_s = 0{,}01 \tag{9.28}$$

Moreover, for structures where S_1 is equal to or greater than 0,6g, C_s must not less than ;

$$Cs = \frac{0{,}5 * S_1}{\left(\frac{R}{I}\right)} \tag{9.29}$$

in which

T = Fundamental period of structure

T_L = Long-period transition period (s) from a map

S_1 = Mapped maximum considered earthquake spectral response acceleration parameter at a period of 1 s

S_{D1} = Design spectral response acceleration parameter at a period of 1,0 sec

The fundamental period of structure (T) must be established using the structural properties and deformational characteristics of the resisting element in a properly substantiated analysis. The fundamental period of structure (T) shall not exceed the product of the coefficient for upper limit on calculated period (C_u) and the approximate fundamental period (T_a) [6]. It can be expressed as;

$$T \le T_a * C_u \tag{9.30}$$

The approximate fundamental period of structure (T_a) can be written as;

$T_a = C_t * (h_n)^x$ (h_n is the height of structure in metric) (9.31)

C_t and x values for the approximate fundamental period of structure and the coefficient for upper limit on calculated period, C_u, can be obtained from below Table 9.11 and Table 9.12.

Table 9.11 Values of approximate period parameters, C_t and x

Structure Type	C_t	x
Steel moment-resisting frame	0,0724	0,8
Concrete moment-resisting frame	0,0466	0,9
Eccentrically braced steel frame	0,0731	0,75
All other structural systems	0,0488	0,75

Table 9.12 Coefficient for upper limit on calculated period

Design Spectral Response Acceleration Parameter at 1s, S_{D1}	Coefficient C_u
$\geq$ 0,40	1,4
0,30	1,4
0,20	1,5
0,15	1,6
$\leq$ 0,1	1,7

The importance factors (I) and the response modification coefficient (R) to be used in Eq. (9.25), Eq. (9.26), Eq. (9.27) and Eq. (9.29) can be obtained from below Table 9.13 and Table 9.14.

Table 9.13 Importance Factors

Occupancy Category	I
I or II	1,0
III	1,25
IV	1,50

Table 9.14 Design Coefficients and Factors for Seismic Force-Resisting Systems

Seismic Force-Resisting System	Response Modification Coefficient, R	Deflection Amplification Factor, C_d
Steel Eccentrically Braced Frames, Moment Resisting, Connections at Columns away from Links	8	4
Steel Eccentrically Braced Frames, Non-Moment-Resisting, Connections at Columns away from Links	7	4
Special Steel Concentrically Braced Frames	6	5
Ordinary Steel Concentrically Braced Frames	3,25	3,25
Special Steel Moment Resisting Frames	8	5,5

Design earthquake spectral response acceleration parameter at short period (S_{DS}) and at 1 s period (S_{D1}) to be used in Eq. (9.25), Eq. (9.26) and Eq. (9.27) can be determined as;

$$S_{DS} = \frac{2}{3} * S_{MS} \tag{9.32}$$

$$S_{D1} = \frac{2}{3} * S_{M1} \tag{9.33}$$

where

S_{MS} = The MCE (Maximum considered earthquake) spectral response acceleration for short period

S_{M1} = The MCE (Maximum considered earthquake) spectral response acceleration at 1 s

The MCE spectral response acceleration for short period (S_{MS}) and at 1 s (S_{M1}), adjusted for site class effects, can be written as;

$$S_{MS} = F_a * S_S \tag{9.34}$$

$$S_{M1} = F_v * S_1 \tag{9.35}$$

in which

S_S = The mapped MCE (Maximum considered earthquake) spectral response acceleration at short period

S_1 = The MCE spectral response acceleration at a period of 1 s

F_a and F_v = Site coefficient

The site coefficients (F_a and F_v) are tabulated as below tables;

Table 9.15 Site Coefficient, F_a

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration Parameter at Short Period				
	$S_S \leq 0,25$	$S_S = 0,50$	$S_S = 0,75$	$S_S = 1,00$	$S_S \geq 1,25$
A	0,8	0,8	0,8	0,8	0,8
B	1,0	1,0	1,0	1,0	1,0
C	1,2	1,2	1,1	1,0	1,0
D	1,6	1,4	1,2	1,1	1,0
E	2,5	1,7	1,2	0,9	0,9

Table 9.16 Site Coefficient, F_v

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \leq 0{,}10$	$S_1 = 0{,}20$	$S_1 = 0{,}30$	$S_1 = 0{,}40$	$S_1 \geq 0{,}50$
A	0,8	0,8	0,8	0,8	0,8
B	1,0	1,0	1,0	1,0	1,0
C	1,7	1,6	1,5	1,4	1,3
D	2,4	2,0	1,8	1,6	1,5
E	3,5	3,2	2,8	2,4	2,4

T_s to be used in Table 9.9 shall be determined from the following equation;

$$T_s = \frac{S_{D1}}{S_{DS}} \qquad (9.36)$$

The seismic design category to be mentioned above Table 9.9 can be determined as follows;

- Occupancy category I, II, III structures located where the mapped spectral response acceleration parameter at 1 s period (S_1) is greater than or equal to 0,75 must be assigned to seismic design category E.
- Occupancy category IV structures located where the mapped spectral response acceleration parameter at 1 s period (S_1) is greater than or equal to 0,75 must be assigned to seismic design category F.
- All other structures can be assigned to a seismic design category based on their occupancy category and the design spectral response acceleration parameters (S_{DS} and S_{D1}) in accordance with below Table 9.17 or Table 9.18.

Table 9.17 Seismic Design Category Based On Short Period Response Acceleration Parameter

Value of S_{DS}	Occupancy Category		
	I or II	III	IV
$S_{DS} < 0{,}167$	A	A	A
$0{,}167 \leq S_{DS} < 0{,}33$	B	B	C
$0{,}33 \leq S_{DS} < 0{,}50$	C	C	D
$S_{DS} \leq 0{,}50$	D	D	D

Table 9.18 Seismic Design Category Based On 1-S Period Response Acceleration Parameter

Value of S_{D1}	Occupancy Category		
	I or II	III	IV
$S_{D1} < 0{,}067$	A	A	A
$0{,}067 \leq S_{D1} < 0{,}133$	B	B	C
$0{,}133 \leq S_{D1} < 0{,}20$	C	C	D
$S_{D1} \leq 0{,}20$	D	D	D

Moreover, the following seismic load combinations for structures not subject to flood or atmospheric ice loads shall be used instead of load combination 5 and 6 in the list of load combinations in Part 3.2.2.

5. $(1{,}2 + 0{,}2 * S_{DS}) * D + \rho * E + L + 0{,}2 * S$

6. $(0{,}9 - 0{,}2 * S_{DS}) * D + \rho * E + 1{,}6 * H$

The load factor on L in combinations 5 is permitted to equal 0,5 for all occupancies in which L is less than or equal to 4,79 kN/m^2.

ρ in the above load combinations is redundancy factor and shall be assigned to the seismic force-resisting system in each of two orthogonal directions. The value of ρ is equal to 1,0 for the structures assigned to seismic design category B or C. The value of ρ is equal to 1,3 for the structures assigned to seismic design category D, E or F unless one of following two conditions is met, whereby ρ is to be taken as 1,0 [6];

1. Each story resisting more than 35 percent of the base shear in the direction of interest shall comply with below Table 9.19.
2. Structures that are regular in plan at all levels provided that the seismic force-resisting systems consist of at least two bays of seismic force-resisting perimeter framing on each side of the structure in each orthogonal direction at each story resisting more than 35 percent of the base shear. The number of bays for a shear wall shall be calculated as the length of shear wall divided by the story height or two times the length of the shear wall divided by the story height for light-framed construction.

Table 9.19 Requirements for Each Story Resisting More Than 35 % of the Base Shear

Lateral Force-Resisting Element	Requirement
Braced Frames	Removal of an individual brace, or connection thereto, would not result in more than a 33 % reduction in story strength, nor does the resulting system have an extreme torsional irregularity
Moment Frames	Loss of moment resistance at the beam-to-column connections at both ends of a single beam would not result in more than a 33 % reduction in story strength, nor does the resulting system have an extreme torsional irregularity
Shear Walls or Wall Pier with a Height-to-Length Ratio of Greater than 1,0	Removal of shear walls or wall pier with a height-to-length ratio of greater than 1,0 within any story , or collector connections thereto, would not result in more than a 33 % reduction in story strength, nor does the resulting system have an extreme torsional irregularity
Cantilever Columns	Loss of moment resistance at the base connections of any single cantilever column would not result in more than a 33 % reduction in story strength, nor does the resulting system have an extreme torsional irregularity
Other	No requirements

In ASCE 7-05 [6], the design must include the accidental torsional moments caused by assumed displacement of the center of mass each way from its actual location by a distance equal to 5 percent of the dimension of the structure perpendicular to the direction of the applied force and the following combination for the lateral loads: 100 percent of the forces for one direction plus 30 percent of the forces for the perpendicular direction.

To calculate storey drifts (Δ_i) for buildings, the below formula is used;

$$\Delta = \frac{C_d * \delta}{I} \tag{9.37}$$

where

C_d = Deflection amplification factor obtained from Table 9.14

δ = Deflection (be obtained from below formula)

$$\delta = d_i - d_{i-1} \tag{9.38}$$

d_i = Displacement calculated at i'th storey of building under design seismic loads

The story drift must not exceed allowable story drift defined in below table;

Table 9.20 Allowable Story Drift (Δ_a)

Structure	Occupancy Category		
	I or II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drift	$0,025*h_{sx}$	$0,020*h_{sx}$	$0,015*h_{sx}$
Masonry cantilever shear wall structures	$0,010*h_{sx}$	$0,010*h_{sx}$	$0,010*h_{sx}$
Other masonry shear wall structures	$0,007*h_{sx}$	$0,007*h_{sx}$	$0,007*h_{sx}$
All other structures	$0,020*h_{sx}$	$0,015*h_{sx}$	$0,010*h_{sx}$
h_{sx} = Story height			

If the response modification coefficient (R) is greater than 3, regardless of the seismic design category, AISC Standard 341-05 (Seismic Provisions for Structural Steel Buildings) must be applied. According to this standard, width-thickness ratios for cross-sections of members of the system must not exceed limiting width-thickness ratios (λ_{ps}) obtained from below Table 9.21.

Table 9.21 Limiting Width-Thickness Ratios (AISC 341-05)

Description of Element	Width Thickness Ratio	Limiting Width-Thickness Ratios λ_{ps} (seismically compact)
Flexure in Flanges of I-Shaped Sections	b/t	$0{,}3 * \sqrt{\frac{E}{F_y}}$
Uniform Compression in Flanges of I-Shaped Sections	b/t	$0{,}3 * \sqrt{\frac{E}{F_y}}$
Webs in Flexural Compression or Combined Flexure and Axial Compression	h/t_w	For Ca $\leq 0{,}125$ $3{,}14 * \sqrt{\frac{E}{f_y}}(1 - 1{,}54 * C_a)$ For Ca $> 0{,}125$ $1{,}12 * \sqrt{\frac{E}{f_y}}(2{,}33 - C_a) \geq 1{,}49 * \sqrt{\frac{E}{f_y}}$
Rectangular HSS in Axial and/or Flexural Compression	b/t or h/t_w	$0{,}64 * \sqrt{\frac{E}{F_y}}$

Columns in EBF, it is permitted to use the following for λ_{ps}:

For Ca $\leq 0{,}125$; $3{,}76 * \sqrt{\frac{E}{f_y}}(1 - 2{,}75 * C_a)$

For Ca $> 0{,}125$; $1{,}12 * \sqrt{\frac{E}{f_y}}(2{,}33 - C_a) \geq 1{,}49 * \sqrt{\frac{E}{f_y}}$

$C_a = \frac{P_u}{\Phi_b * P_y}$ where P_u = Required Compressive Strength , P_y = Axial Yield Strength , $\Phi_b = 0{,}9$

9.3 Calculation of Earthquake Loads for Eurocode 3

When the earthquake loads are calculated according to Eurocode 3, Eurocode 3 refers to Eurocode 8 [8]. In this thesis, lateral force method of analysis is used to calculate the earthquake loads. The applicability conditions are given in the below for the lateral force method of analysis;

1. The fundamental periods of buildings must be smaller than the following two values in the two main direction

$$T_1 \leq \begin{cases} 4 * T_C \\ 2{,}0s \end{cases} \tag{9.39}$$

2. The criteria for regularity in elevation must be satisfied.

The seismic base shear force, F_b, can be expressed as below formula;

$$F_b = S_d\,(T_1) * m * \lambda \tag{9.40}$$

in which

λ = Correction factor

T_1 = Fundamental period of the building

m = Total mass of the building

$S_d\,(T_1)$ = The ordinate of the design spectrum

The value of the correction factor (λ) to be used in Eq. 9.40 can be taken as 0,85 if $T_1 \leq 2 * T_C$ and the building has more than two stories. Otherwise, the value of the correction factor (λ) can be taken as 1,00.

The fundamental period of the building (T_1) to be used in Eq. 9.40 can be calculated with methods of structural dynamics (exp. Rayleigh method).

The total mass of the building (m) to be used in Eq. 9.40 can be expressed as;

$$m = \sum G_{k,j} "+" \sum \Psi_{E,i} * Q_{k,i} \qquad (9.41)$$

where

G = Permanent action

Q = Variable action

$\Psi_{E,i}$ = Combination coefficient

The combination coefficient ($\Psi_{E,i}$) to be used in Eq. 9.41 can be determined in accordance with below equation;

$$\Psi_{E,i} = \varphi * \psi_{2,i} \qquad (9.42)$$

where

$\psi_{2,i}$ = The values of Ψ factors obtained from Table 3.12

The values of φ are defined in below table;

Table 9.22 Values of φ for calculating ψ_{Ei}

Type of variable action	Storey	φ
Categories A-C	Roof Storey with correlated occupancies Independently occupied storey	1,0 0,8 0,5
Categories D-F		1,0
Categories defined in Table 3.8		

The design spectrum, S_d (T_1), can be defined by following equations;

$$0 \leq T \leq T_B \quad : \quad S_d(T) = a_g * S * \left[\frac{2}{3} + \frac{T}{T_B} * \left(\frac{2,5}{q} - \frac{2}{3} \right) \right] \qquad (9.43)$$

$$T_B \leq T \leq T_C \quad : \quad S_d(T) = a_g * S * \frac{2,5}{q} \qquad (9.44)$$

$$T_C \leq T \leq T_D \quad : \quad S_d(T) \begin{cases} = a_g * S * \frac{2,5}{q} * \left(\frac{T_C}{T} \right) \\ \geq \beta * a_g \end{cases} \qquad (9.45)$$

$$T_D \leq T \quad : \quad S_d(T) \begin{cases} = a_g * S * \frac{2,5}{q} * \left(\frac{T_C * T_D}{T^2} \right) \\ \geq \beta * a_g \end{cases} \qquad (9.46)$$

where

a_g = Design ground acceleration

T_B , T_C , T_D = Limit of the period of the spectrum

S = Soil factor

β = Lower bound factor (equal to 0,2)

q = Behavior factor

The design ground acceleration (a_g) to be used the design spectrum equations can be expressed as below;

$$a_g = \gamma_I * a_{gR} \qquad (9.47)$$

where

a_{gR} = Peak ground acceleration derived from zonation maps found in its National Annex

γ_I = Importance factor

The importance factors (γ_I) are listed in below table;

Table 9.23 Importance classes and importance factors for buildings

Importance Class	Buildings	Importance Factor (γ_I)
I	Buildings of minor importance for public safety, e.g. Agricultural buildings	0,8
II	Ordinary buildings, not belonging in the other categories	1,0
III	Buildings whose seismic resistance is of importance in view of the consequences with a collapse, e.g. schools, assembly halls	1,2
IV	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations	1,4

There are two types of the spectra to obtain the values of T_B, T_C, T_D and S: Type 1 and Type 2. If the earthquakes that contribute most to the seismic hazard defined for the site for the purpose of probabilistic hazard assessment have a surface wave magnitude, M_s, not greater than 5,5, it is recommended that Type 2 spectrum is adopted. Otherwise, Type 1.

The values of T_B, T_C, T_D and S introduced in below tables for Type 1 and Type 2.

Table 9.24 Values of parameters the recommended Type 1

Ground Type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,15	0,4	2,0
B	1,2	0,15	0,5	2,0
C	1,15	0,20	0,6	2,0
D	1,35	0,20	0,8	2,0
E	1,4	0,15	0,5	2,0

Table 9.25 Values of parameters the recommended Type 2

Ground Type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,05	0,25	1,2
B	1,35	0,05	0,25	1,2
C	1,5	0,10	0,25	1,2
D	1,8	0,10	0,30	1,2
E	1,6	0,05	0,25	1,2

The behavior factor (q) to be used the design spectrum equations can be obtained from below table;

Table 9.26 Values of behavior factors for systems regular in elevation

Structural Type For Steel Buildings	Ductility Class	
	DCM	DCH
Moment resisting frames	4	$5*\alpha_u / \alpha_1$
Frame with concentric bracing Diagonal bracings V-bracing	 4 2	 4 2,5
Frame with eccentric bracing	4	$5*\alpha_u / \alpha_1$
DCM : Ductility Class Medium DCH : Ductility Class High		

The values of α_u / α_1 to be used in Table 9.26 can be showed in the below figures;

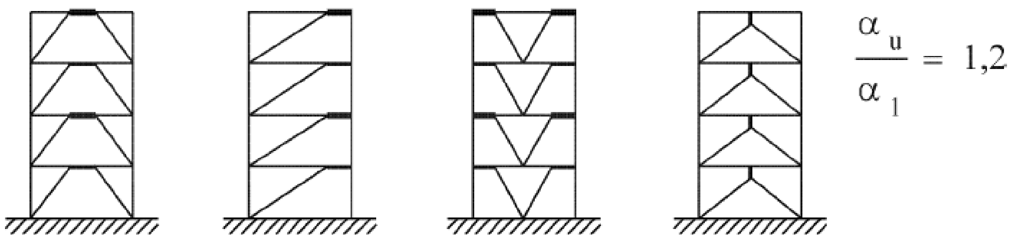


Figure 9.3 Frames with eccentric bracing

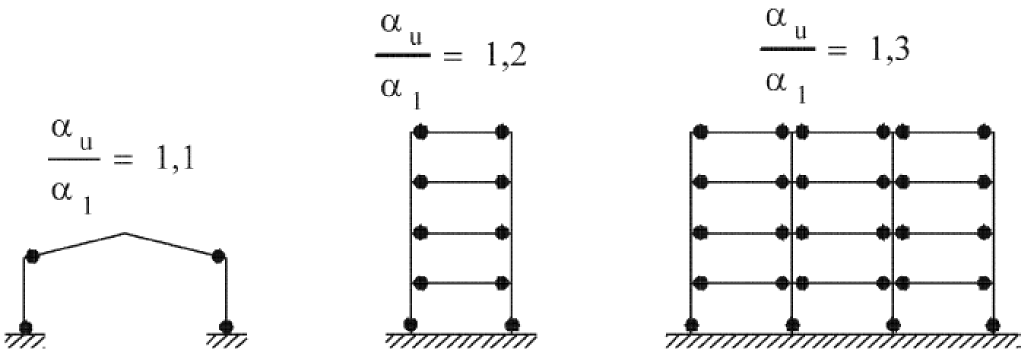


Figure 9.4 Moment resisting frames

For effect of the seismic action, the accidental torsional effects to account for uncertainties in the location of masses shall be determined from the following equation [8];

$e_{ai} = \pm 0,05 * L_i$ (9.48)

where

e_{ai} = The accidental eccentricity of storey mass

L_i = Floor dimension perpendicular to the direction of the seismic action

The effects due to the combination of the horizontal components of the seismic action can be computed below equations;

E_{Edx} "+" 0,30 * E_{Edy} (9.49)

E_{Edy} "+" 0,30 * E_{Edx} (9.50)

in which

E_{Edx} and E_{Edy} = The effects due to the application of the same seismic action along the chosen horizontal axis x and y of the structure respectively.

Moreover, the requirements on cross-sectional class of members depending on ductility class and behavior factor are defined in the below table;

Table 9.27 Requirements on cross-sectional depending on ductility class and behavior factor

Ductility Class	Reference value of behavior factor q	Required cross-sectional class
DCM	$1,5 < q \leq 2$	Class 1, 2 or 3
	$2 < q \leq 4$	Class 1 or 2
DCH	$q > 4$	Class 1
DCM : Ductility Class Medium DCH : Ductility Class High		

To calculate storey drifts (θ_i) for buildings, the below formula is used;

$$\theta = \frac{P_{tot} * d_r}{V_{tot} * h} \qquad (9.51)$$

where

θ = Interstorey drift coefficient

P_{tot} = Total gravity load at and above the storey

V_{tot} = Total seismic storey shear

d_r = Design storey drift

h = Interstorey height

Design storey drift (d_r) to be used in the Equation 9.51 can be calculated as below formula;

$$d_r = q * d_e \quad (9.52)$$

in which

q = Behavior factor

d_e = Storey drift

$$d_{ie} = d_i - d_{i-1} \quad (9.53)$$

d_i = Displacement calculated at i'th storey of building under design seismic loads

The value of the coefficient θ shall not exceed 0,3.

CHAPTER 10

CASE STUDIES

This chapter includes building properties and detailed calculations.

The building is located in Bakırköy-İstanbul, on a soil class of Z2 as classified in TEC 2007. This soil profile is equivalent to Class C in LRFD procedure and Ground Type B in Eurocode 8. Moreover, the building is found to be in Earthquake Zone 1. Effective ground acceleration coefficient A_0=0,4 in TEC 2007 and the equivalent term design ground acceleration a_g=0,4 in Eurocode 8. However, while calculating earthquake load using ASCE 7-05, S_s (the mapped MCE spectral response acceleration at short period) and S_1 (the mapped MCE spectral response acceleration at a period of 1 s) are required. From Marmaray Project conducted in Istanbul, the values are found to be 0,775 for S_s and 0,35 for S_1.

The geometrical properties of the building is taken from İrtem et. al. [24], but some modifications are introduced. For the same plan, buildings having 2, 4, 6, 8, 10 stories are generated and designed using three different procedures as mentioned above. In all buildings first story is 3,5 m in height, whereas the rest is 3 m. The structural system of the steel building in both x and y directions are considered as eccentrically braced frames with high ductility level. Considering this frame type, both "the structural behavior factor (R)" in TEC 2007 and "response modification coefficient (R)" in ASCE-7-05 correspond to 7, and "behavior factor (q)" in Eurocode 8 is calculated as 6.

All main beams connected to the columns are assumed to take no moment, that is they are all pin connected. Besides, secondary beams are also pin connected to the main beams and placed in every 2 m parallel to x-direction. Furthermore, all slabs are 10 cm thick, and there are only exterior walls with 20 cm.

All earthquake forces are evaluated and assigned using “Equivalent Seismic Force Method”. Also, section tables can be found in appendix

The building plan and 3D views of LARSA model are given below.

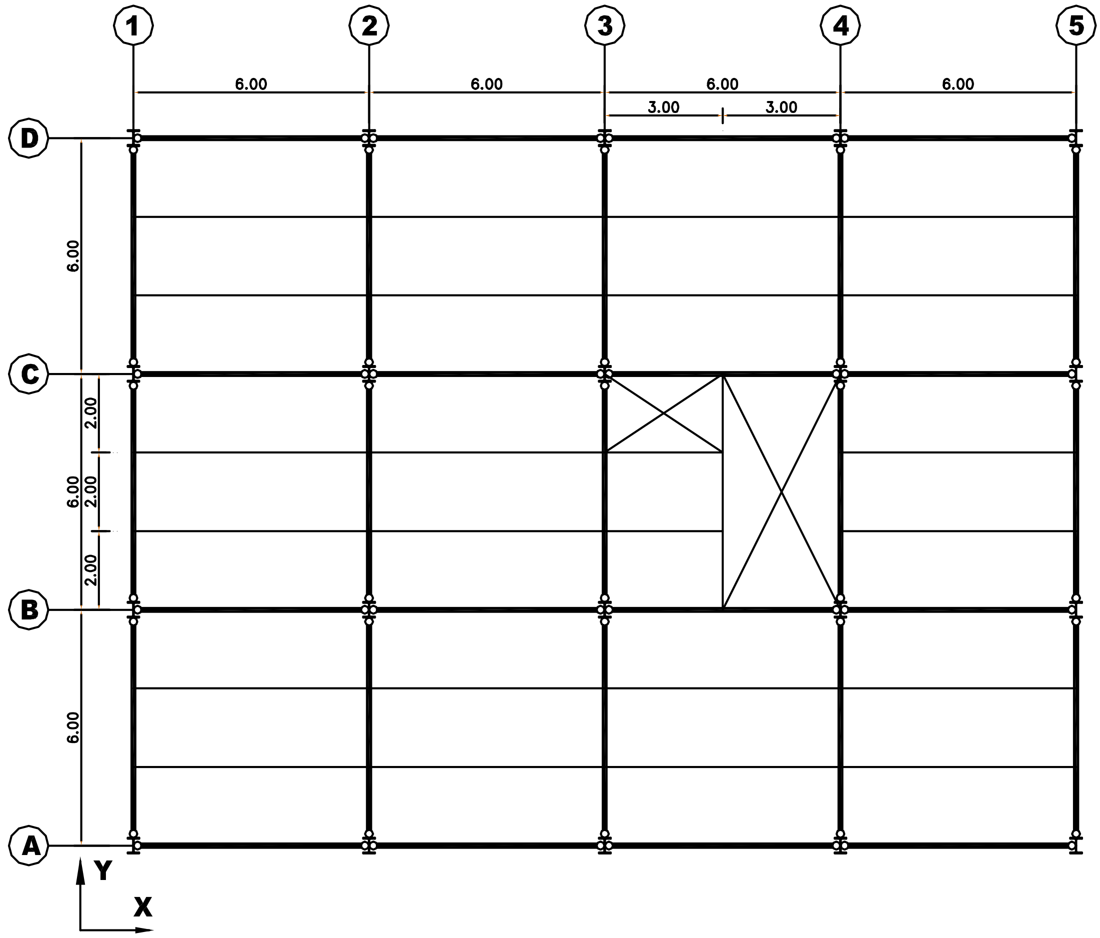


Figure 10.1 Plan of the building

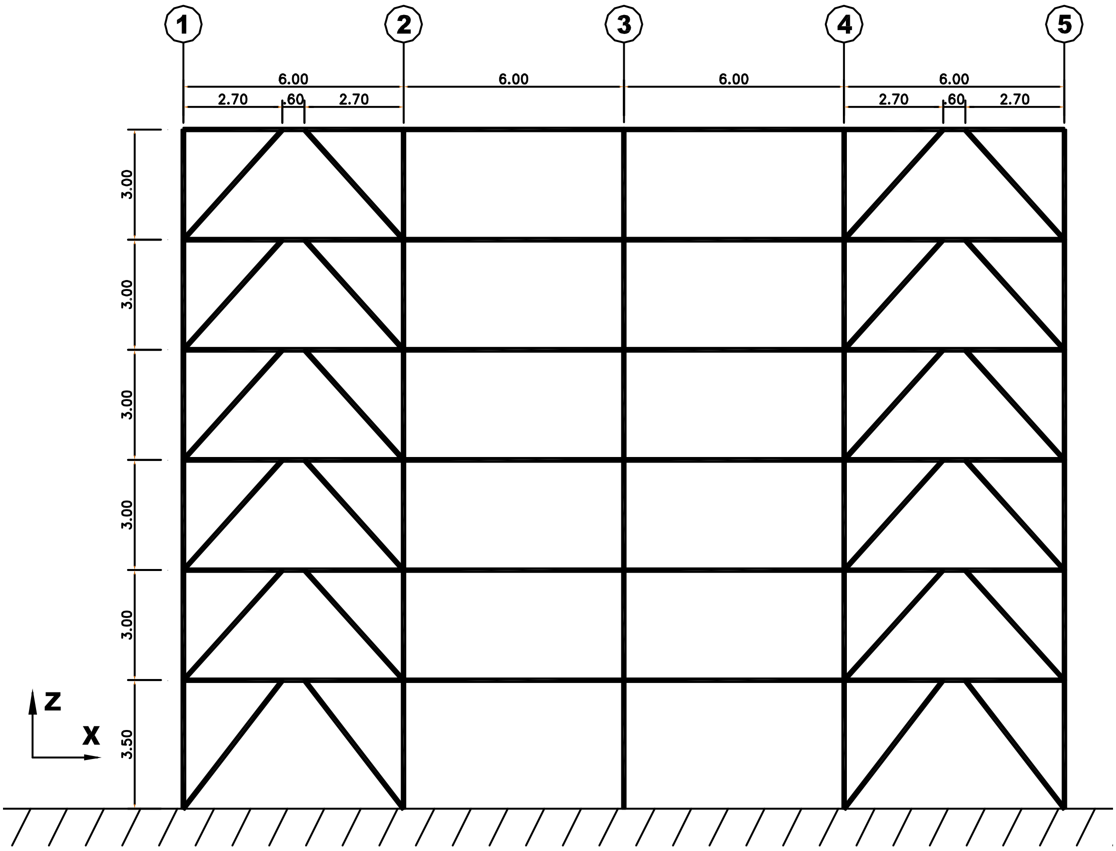


Figure 10.2 Building X-Z view

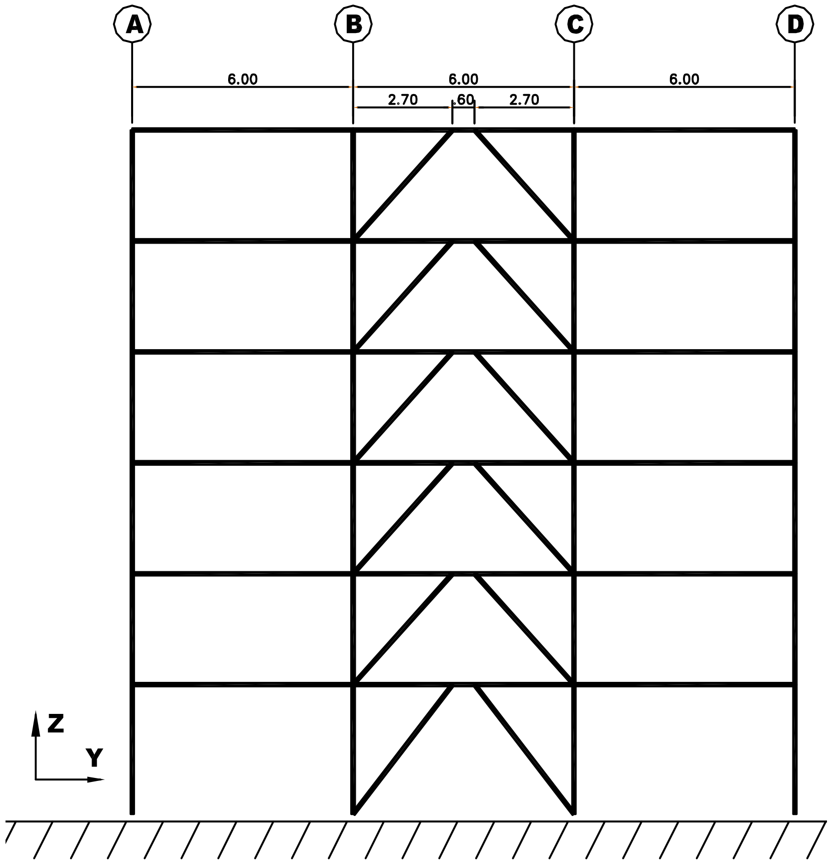


Figure 10.3 Building Y-Z view

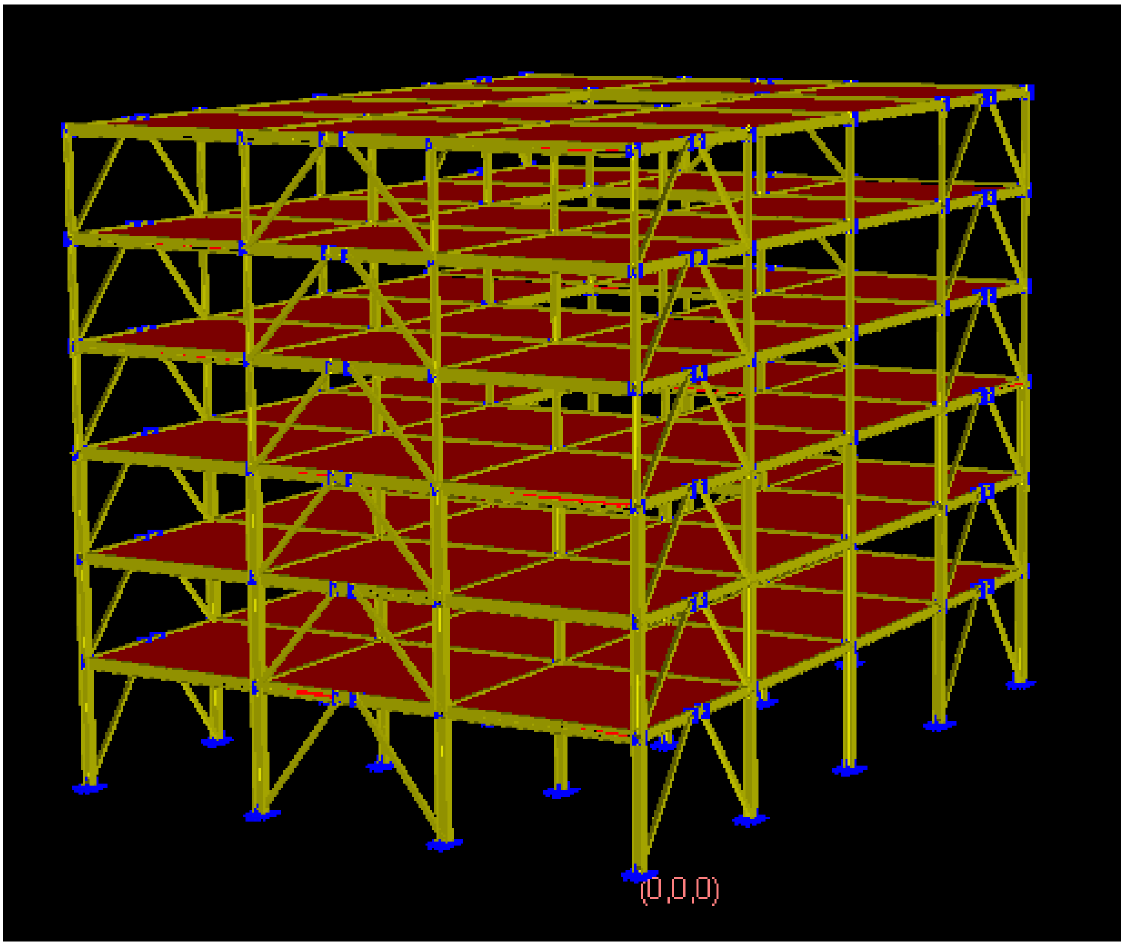


Figure 10.4 3D View of LARSA Model

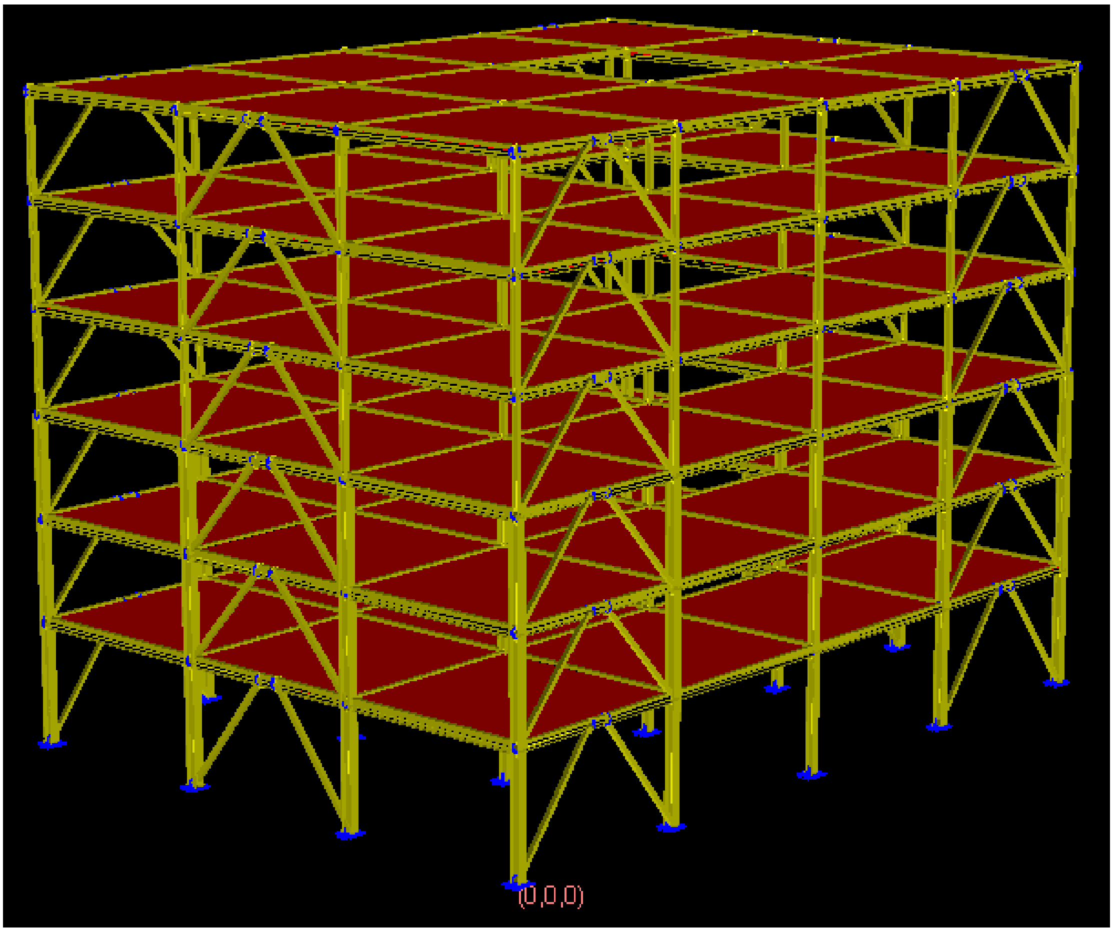


Figure 10.5 3D View of LARSA Model

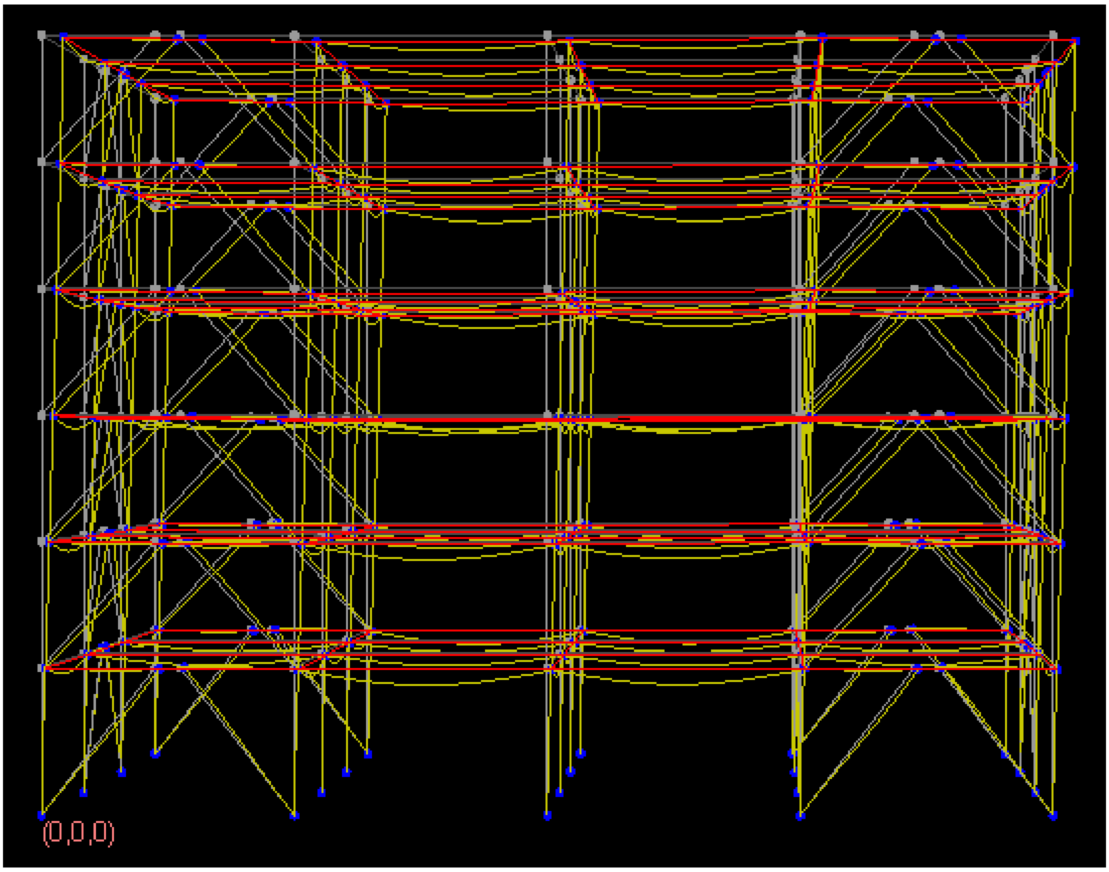


Figure 10.6 Deformed Shape of LARSA Model

To show the difference between the three procedures described above, a case study for 2 story building is performed and for the preliminary design all beams are taken as IPN 340, all columns are HE 140 B and all braces are TUBE 140 x 140 x 8.

For each procedure, calculation steps including earthquake load calculation and design of all members (columns, beams, braces and link beams) are presented separately.

10.1 Earthquake Calculation

10.1.1 Earthquake Design According to TEC 2007

Member sections for 2 storey building:

All Beams	: IPN 340
All Columns	: HE 140 B
All Braces	: Tube 140 x 140 x 8

The each storey weight of the building is calculated utilizing Eq. 9.3 and Table 9.2.

$w_1 = g_1 + 0{,}3 * q_1 = 3038{,}9 + 0{,}3*869 = 3298{,}1$ kN

$w_2 = g_2 + 0{,}3 * q_2 = 2754{,}2 + 0{,}3 * 869 = 3013{,}9$ kN

<u>Fictitious Loads</u>

In order to calculate fundamental periods of the building, following loads are introduced to structural model

1. Storey: 37 kN

2. Storey: 63 kN

<u>Fictitious Displacements</u>

Fictitious displacements are obtained from outputs of LARSA.

<u>X-direction</u>

$$d_{f1x} = 0.257 * 10^{-3}\, m$$
$$d_{f2x} = 0.414 * 10^{-3}\, m$$

<u>Y-direction</u>

$d_{f1y} = 0.5 * 10^{-3} m$
$d_{f2y} = 0.821 * 10^{-3} m$

<u>Fundamental Periods of the Buildings</u>

Making use of Rayleigh method in Eq. 9.13

T_x = 0,29 sec

T_y = 0,40 sec

A_{0} = 0,4 (obtained from Table 9.3)

I = 1,0 (obtained from Table 9.4)

Soil class = Z2

T_A = 0,15 and T_B = 0,4 (obtained from Table 9.5)

$S(T_x)$ = 2,5 (Making use of Eq. 9.6)

$A(T) = A_0 * I * S(T_x) = 0,4 * 1 * 2,5 = 1,0$ (Utilizing Eq. 9.4)

$T > T_A \rightarrow R_{ax}(T) = 7$ (Making use of Eq. 9.9 and Table 9.6)

$W_T = w_1 + w_2 = 3298,1 + 3013,4 = 6311,5$ kN (Using Eq. 9.2)

Total Equivalent Seismic Load (base shear), V_t, is calculated utilizing Eq. 9.1.

$$V_{tx} = \frac{W_T * A(T)}{R_{ax}} = \frac{6311,5 * 1}{7} = 901,6 \text{ kN} > 0,1 * A_0 * I * W = 0,1 * 0,4 * 1,0 * 6311,5 = 252,5 \text{ kN}$$

$\Delta F_{Nx} = 0,0075 * N * V_{tx} = 0,0075 * 2 * 901,6 = 13,52$ kN (Making use of Eq. 9.11)

V_{tx} = 901,6 kN

V_{ty} = 901,6 kN

10.1.2 Earthquake Design According to ASCE 7-05

Member sections for 2 storey building:

All Beams	: IPN 340
All Columns	: HE 140B
All Braces	: Tube 140 x 140 x 8

The effective seismic weight of a structure (W) is equal to the total dead load.

$w_1 = g_1$ = 3038,9 kN

$w_2 = g_2$ = 2754,2 kN

<u>Fictitious Loads</u>

In order to calculate fundamental periods of the building, following loads are introduced to structural model

1. Storey: 37 kN

2. Storey: 63 kN

<u>Fictitious Displacements</u>

Fictitious displacements are obtained from outputs of LARSA

X-direction

$d_{f1x} = 0.257 * 10^{-3} m$
$d_{f2x} = 0.414 * 10^{-3} m$

Y-direction

$d_{f1y} = 0.5 * 10^{-3} m$
$d_{f2y} = 0.821 * 10^{-3} m$

The fundamental period of structure (T) shall not exceed the product of the coefficient for upper limit on calculated period (C_u) and the approximate fundamental period (T_a).

$T \leq T_a * C_u$ (Utilizing Eq. 9.30)

$C_u = 1,4$ (Obtained from Table 9.11)

$T_a = C_t * (h_n)^x = 0,0713 * (6,5)^{0,75} = 0,29$ sec

$T = Cu * T_a = 1,4 * 0,29 = 0,41$ sec

Fundamental Periods of the Buildings

Making use of Rayleigh method in Eq. 9.13

$T_x = 0,27$ sec $\leq$ T = 0,41 sec

$T_y = 0,38$ sec $\leq$ T = 0,41 sec

Site Class: C

$S_s = 0,775 \rightarrow F_a = 1,09$ (Obtained from Table 9.15)

$S_1 = 0,35 \rightarrow F_v = 1,45$ (Obtained from Table 9.16)

$S_{MS} = F_a * S_S = 1{,}09*0{,}775 = 0{,}845$ (Utilizing Eq. 9.34)

$S_{M1} = F_v * S_1 = 1{,}45*0{,}35 = 0{,}51$ (Utilizing Eq. 9.35)

$S_{DS} = \frac{2}{3} * S_{MS} = \frac{2}{3} * 0.805 = 0.564$ (Utilizing Eq. 9.32)

$S_{D1} = \frac{2}{3} * S_{M1} = \frac{2}{3} * 0.51 = 0.34$ (Utilizing Eq. 9.33)

$T_s = \frac{S_{D1}}{S_{DS}} = \frac{0{,}34}{0{,}564} = 0.6$ (Utilizing Eq. 9.36)

Seismic Design Category = D (Obtained from Table 9.17 or Table 9.18)

The seismic base shear, V, is calculated as;

V = Cs * W (Utilizing Eq. 9.24)

$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)} = \frac{0{,}564}{\left(\frac{7}{1}\right)} = 0{,}0806$ (Utilizing Eq. 9.25)

Effective seismic weight;

$W = w_1 + w_2 = 3038{,}9 + 2754{,}2 = 5793{,}1$ kN

Response Modification Coefficient: R = 7 (Obtained from Table 9.14)

Utilizing Eq. 9.25:

$T \leq T_L = 4 \text{ sec} \rightarrow C_{smax} = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} = \frac{0{,}34}{0{,}27 * \left(\frac{7}{1}\right)} = 0{,}18 \geq Cs = 0{,}0806 \quad \therefore \text{ OK}$

$V = C_s * W = 0,0806 * 5793,1 = 466,9$ kN (Utilizing Eq. 9.24)

The value of $0,2 * S_{DS}$ is used for seismic load combinations in Part 9.2.

$0,2 * S_{DS} = 0,2 * 0,564 = 0,11$

$V_{tx} = 466,9$ kN

$V_{ty} = 466,9$ kN

10.1.3 Earthquake Design According to Eurocode 8

Member sections for 2 storey building:

All Beams : IPN 340
All Columns : HE 140 B
All Braces : Tube 140 x 140 x 8

The each storey weight of the building is calculated according to Eq. 9.41, Eq. 9.42 and Table 9.22.

$w_1 = g_1 + 0,24 * q_1 = 3038,9 + 0,3 * 864 = 3246,3$ kN

$w_2 = g_2 + 0,3 * q_2 = 2754,2 + 0,3 * 869 = 3013,9$ kN

Fictitious Loads

In order to calculate fundamental periods of the building, following loads are introduced to structural model

1. Storey: 37 kN

2. Storey: 63 kN

Fictitious Displacements

Fictitious displacements are obtained from outputs of LARSA.

X-direction

$d_{f1x} = 0.257 * 10^{-3} m$

$d_{f2x} = 0.414 * 10^{-3} m$

Y-direction

$d_{f1y} = 0.5 * 10^{-3} m$

$d_{f2y} = 0.821 * 10^{-3} m$

Fundamental Periods of the Buildings

Making use of Rayleigh method in Eq. 9.13

T_x = 0,28 sec

T_y = 0,40 sec

Ground Type: B

$W_T = w_1 + w_2$ = 3246,3 + 3013,4 = 6259,7 kN = m * g (Utilizing Eq. 9.41)

λ = 1 (Due to the building has two stories)

q = 6 (Obtained from Table 9.26 and Figure 9.3)

$a_g = \gamma_I * a_{gR}$ = 1,0 * 0,4 = 0,4

Type 1 spectra is used for a_g = 0,4 due to M_s > 5,5 :

The below values are obtained from Table 9.24 for ground type B.

S = 1,2 sec

$T_B = 0,15$ sec

$T_C = 0,50$ sec

$T_D = 2,00$ sec

<u>$T_B < T < T_C$</u>

$S_d(T) = a_g * S * 2,5 / q = 0.4 * 1.2 * \frac{2.5}{6} = 0.2$ (Using Eq. 9.44)

$F_b = S_d (T_1) * m * \lambda = 0,2 * 6259,7 * 1 = 1251,9$ kN (Using Eq. 9.40)

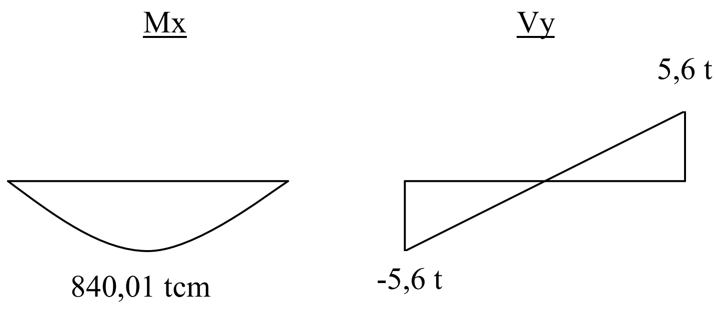
$F_{bx} = 1251,9$ kN

$F_{by} = 1251,9$ kN

10.2 Beam Design

Following design steps are for beam between A2 and A3 axis in x-direction. This beam is designed for the unfavorable loading case for the three methodologies as defined and explained previously.

10.2.1 Beam Design According to TS 648



Moment and shear diagrams of the beam in the x direction which are subjected to the worst loading condition are determined, after that the beam will be designed based on these diagrams

Loading Type: EY

Material: St 37

$\sigma_y = 2{,}4 \text{ t/cm}^2$

$\sigma_{all} = 0{,}6 * 2{,}4 = 1{,}44 \text{ t/cm}^2$

$$\tau_{all} = \frac{\sigma_{all}}{\sqrt{3}} = \frac{1.44}{\sqrt{3}} = 0.831 \text{ t/cm}^2$$

Selected Section: IPN 280

h	**t_w**	**t_f**	**b**	**r**	**W_{elx}**
28 cm	1,01 cm	1,52 cm	11,9 cm	0,61 cm	542 cm^3

Sectional Properties of the selected section must satisfy the following conditions specified in TEC 2007.

$$\frac{b}{2 * t_f} \leq 0.3 * \sqrt{\frac{E}{\sigma_y}} \quad \text{(Obtained from Table 9.8)}$$

$$\frac{b}{2 * t_f} = \frac{11{,}9}{2 * 1{,}52} = 3{,}91 \leq 0.3 * \sqrt{\frac{E}{\sigma_y}} = 0.3 * \sqrt{\frac{2100}{2{,}4}} = 8{,}87 \quad \therefore \text{ OK}$$

$$\frac{h}{t_w} \leq 3.2 * \sqrt{\frac{E}{\sigma_y}} \quad \text{(Obtained from Table 9.8)}$$

$$\frac{h}{t_w} = \frac{h - 2*t_f - 2*r}{t_w} = \frac{28 - 2*1{,}52 - 2*0{,}61}{1.01} = 23{,}5 \le 3.2*\sqrt{\frac{E}{\sigma_y}} = 3.2*\sqrt{\frac{2100}{2.4}} = 94{,}7$$

$\therefore$ OK

- Stress Check:

M_{max} = 840,01 tcm

$\sigma_{max} = \frac{M_{max}}{W_{elx}} \le \sigma_{all}$ (Using Eq. 6.1)

$\sigma_{max} = \frac{M_{max}}{W_{elx}} = \frac{840.01}{542} = 1.55 t/cm^2 \ge \sigma_{all} = 1.44 t/cm^2$ $\therefore$ Not OK

Take Greater Section!

Selected Section: IPN 300

h	**t_w**	**t_f**	**b**	**r**	**W_{elx}**
30 cm	1,08 cm	1,62 cm	12,5 cm	0,65 cm	653 cm^3

Sectional Properties of the selected section must satisfy the following conditions specified in TEC 2007.

$\frac{b}{2*t_f} = \frac{12.5}{2*1.62} = 3.86 \le 0.3*\sqrt{\frac{E}{\sigma_y}} = 0.3*\sqrt{\frac{2100}{2.4}} = 8.87$ $\therefore$ OK (Obtained from Table 9.8)

$$\frac{h}{t_w} = \frac{h - 2*t_f - 2*r}{t_w} = \frac{30 - 2*1.62 - 2*0.65}{1.08} = 23.57 \le 3.2*\sqrt{\frac{E}{\sigma_y}} = 3.2*\sqrt{\frac{2100}{2.4}} = 94.7$$

$\therefore$ OK

(Obtained from Table 9.8)

- Stress Check :

M_{max} = 806,4 tcm

$$\sigma_{max} = \frac{M_{max}}{W_{elx}} \leq \sigma_{all} \quad \text{(Using Eq. 6.1)}$$

$$\sigma_{max} = \frac{M_{max}}{W_{elx}} = \frac{840.01}{653} = 1.29\ t/cm^2 \leq \sigma_{all} = 1.44\ t/cm^2 \ \therefore \ \text{OK}$$

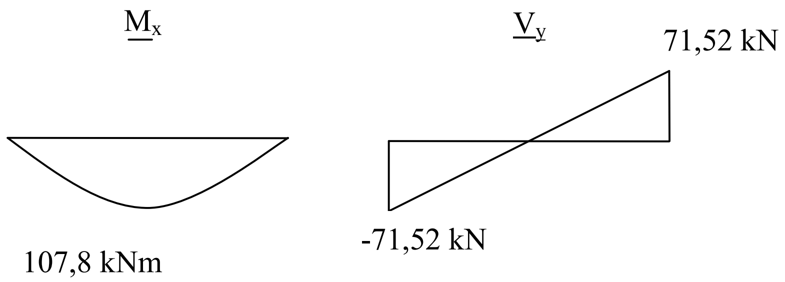
- Shear Check :

V_{max} = 5,6 t

$$\tau_{max} = \frac{V_{max}}{h * t_w} = \frac{5.6}{30 * 1.08} = 0.173\ t/cm^2 \leq \tau_{all} = 0.831\ t/cm^2 \ \therefore \ \text{OK (Using Eq. 6.4)}$$

Stability control of the beams will not be conducted since the compression flanges of the beams are restrained against lateral buckling due to the effect of the slab. Besides, the end conditions of the beams are free. At any point in the compression flanges only positive moments are present.

10.2.2 Beam Design According to LRFD



M_A = 80,46 kNm
M_B = 107,29 kNm
M_C = 80,46 kNm

E = 200000 N/mm^2
f_y = 235 N/mm^2

Selected Section: IPN 240

h	240 mm	**I_x**	42,5 x 10^6 mm^4	**I_y**	2,21 x 10^6 mm^4
b	106 mm	**W_{elx}**	354 x 10^3 mm^3	**W_{ely}**	41,7 x 10^3 mm^3
t_w	8,7 mm	**W_{plx}**	412 x 10^3 mm^3	**W_{ply}**	70 x10^3 mm^3
t_f	13,1 mm	**i_x**	95,9 mm	**i_y**	22 mm
r	5,2 mm	**I_t**	250 x 10^3 mm^4		
A	4610 mm^2	**I_w**	28700 x $10^6 mm^6$		

- Width-Thickness Ratio :

$$\lambda_f = \frac{b}{2*t_f} = \frac{106}{2*13,1} = 4,05 \text{ (flange) (Using Eq. 3.7)}$$

$$\lambda_w = \frac{h-2*t_f-2*r}{t_w} = \frac{200-2*13,1-2*5,2}{8,7} = 23,38 \text{ (web) (Using Eq. 3.8)}$$

λ_f and λ_w values cannot exceed the limiting width-thickness ratio values obtained from Table 9.21 in Seismic Provisions for Structural Steel Buildings.

For Flanges

$$\lambda_{fps} = 0.30*\sqrt{\frac{E}{f_y}} = 0,30*\sqrt{\frac{200000}{235}} = 8,75$$

For Web

$$C_a = \frac{Pu}{\Phi * Py} \text{ (Obtained from Table 9.21)}$$

We assume that Pu is zero for beams so C_a can be taken as zero.

$Ca = 0 \leq 0,125$

$$\lambda_{wps} = 3.14 * \sqrt{\frac{E}{f_y}}(1 - 1.54 * C_a) = 3.14 * \sqrt{\frac{200000}{235}} * (1 - 1,54 * 0) = 91.6$$

$\lambda_f = 4.05 \quad < \quad \lambda_{fps} = 8.75 \;\therefore\; \text{OK}$

$\lambda_w = 23,38 < \lambda_{wps} = 91.6 \;\therefore\; \text{OK}$

- Cross Section Classification :

The below values are obtained from Table 3.13.

For Flanges

$\lambda_{pf} = 0,38 * \sqrt{\frac{E}{f_y}} = 0.38 * \sqrt{\frac{200000}{235}} = 11,09$ (Compact)

$\lambda_{rf} = 1,0 * \sqrt{\frac{E}{f_y}} = 1,0 * \sqrt{\frac{200000}{235}} = 29,17$ (Noncompact)

For Web

$\lambda_{pw} = 3,76 * \sqrt{\frac{E}{f_y}} = 3,76 * \sqrt{\frac{200000}{235}} = 109,69$ (Compact)

$\lambda_{rw} = 5,70 * \sqrt{\frac{E}{f_y}} = 5,70 * \sqrt{\frac{200000}{235}} = 166,29$ (Noncompact)

$\lambda_f < \lambda_{pf}$ and $\lambda_w < \lambda_{pw}$

$\therefore$ Section is compact flange and compact web.

So this beam has been solved according to Doubly Symmetric Compact I-Shaped Members in LRFD.

- Yielding :

Mn = Mp = $F_y * Z_x$ = 235 * 412000 = 96820000 Nmm = 96,82 kNm (Using Eq. 6.12)

(Z_x=Wplx=412000 mm^3)

Φ * Mn = 0,9 *96,82 = 87,318 kNm < Mmax = 107,28 kNm $\therefore$ Not OK (Using Eq. 6.10)

Take a greater section!

Selected Section: IPN 260

h	260 mm	**I_x**	57,4 x 10^6 mm^4	**I_y**	2,88 x 10^6 mm^4
b	113 mm	**W_{elx}**	442 x 10^3 mm^3	**W_{ely}**	51 x 10^3 mm^3
t_w	9,4 mm	**W_{plx}**	514 x 10^3 mm^3	**W_{ply}**	85,9 x 10^3 mm^3
t_f	14,1 mm	**i_x**	104 mm	**i_y**	23,2 mm
r	5,6 mm	**I_t**	335 x 10^3 mm^4		
A	5330 mm^2	**I_w**	44100 x $10^6$$mm^6$		

- Width-Thickness Ratio :

$$\lambda_f = \frac{b}{2*t_f} = \frac{113}{2*14,1} = 4,01 \text{ (flange) (Using Eq. 3.7)}$$

$$\lambda_w = \frac{h-2*t_f-2*r}{t_w} = \frac{260-2*14,1-2*5,6}{9,4} = 23,48 \text{ (web) (Using Eq. 3.8)}$$

λ_f and λ_w values cannot exceed the limiting width-thickness ratio values obtained from Table 9.21 in Seismic Provisions for Structural Steel Buildings.

For Flanges

$$\lambda_{fps} = 0.30 * \sqrt{\frac{E}{f_y}} = 0,30 * \sqrt{\frac{200000}{235}} = 8,75$$

For Web

$C_a = \frac{Pu}{\Phi * Py}$ (Obtained from Table 9.21)

We assume that P_u is zero for beams so C_a can be taken as zero.

$Ca = 0 \leq 0,125$

$$\lambda_{wps} = 3,14 * \sqrt{\frac{E}{f_y}}(1 - 1,54 * C_a) = 3,14 * \sqrt{\frac{200000}{235}} * (1 - 1,54 * 0) = 91,6$$

$\lambda_f = 4,01 < \lambda_{fps} = 8,75 \therefore$ OK

$\lambda_w = 23,48 < \lambda_{wps} = 91,6 \therefore$ OK

- Cross Section Classification :

The below values are obtained from Table 3.13.

For Flanges

$\lambda_{pf} = 0,38 * \sqrt{\frac{E}{f_y}} = 0.38 * \sqrt{\frac{200000}{235}} = 11,09$ (Compact)

$\lambda_{rf} = 1{,}0 * \sqrt{\dfrac{E}{f_y}} = 1{,}0 * \sqrt{\dfrac{200000}{235}} = 29{,}17$ (Noncompact)

For Web

$\lambda_{pw} = 3{,}76 * \sqrt{\dfrac{E}{f_y}} = 3{,}76 * \sqrt{\dfrac{200000}{235}} = 109{,}69$ (Compact)

$\lambda_{rw} = 5{,}70 * \sqrt{\dfrac{E}{f_y}} = 5{,}70 * \sqrt{\dfrac{200000}{235}} = 166{,}29$ (Noncompact)

$\lambda_f < \lambda_{pf}$ and $\lambda_w < \lambda_{pw}$

$\therefore$ Section is compact flange and compact web.

So this beam has been solved according to Doubly Symmetric Compact I-Shaped Members in LRFD.

- Yielding :

Mn= Mp = $F_y * Z_x$ = 235 * 514000 = 120790000 Nmm = 120,79 kNm (Using Eq. 6.12)

(Z_x=Wplx=514000 mm^3)

Φ_b * Mn= 0,9 *120,79 = 108,71 kNm > Mmax= 107,28 kNm $\therefore$ OK (Using Eq. 6.10)

- Lateral Torsional Buckling :

We have not checked the stability control of the beams because of effect of slab and the beams` end conditions which are free.

- Shear :

Vn = 0,6 * F_y * Aw * Cv (Utilizing Eq. 6.22)

For web of rolled I shaped members with

$$\frac{h}{t_w} = 23,48 \leq 2,24 * \sqrt{\frac{E}{F_y}} = 2,24 * \sqrt{\frac{200000}{235}} = 65,35 \quad \therefore \text{ OK}$$

Take

Cv=1,0 and Φv=1,0

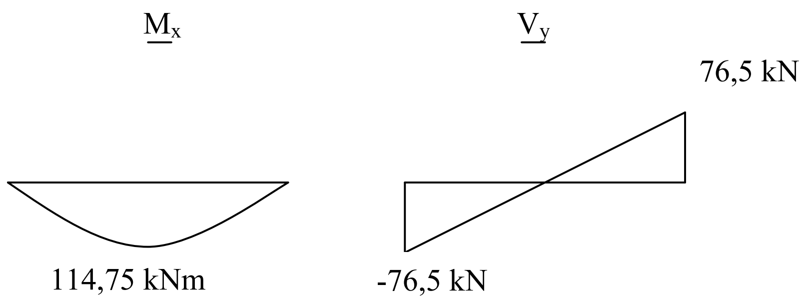
$A_w = h * t_w = 260 * 9,4 = 2444 \text{ mm}^2$

$\Phi v * Vn = \Phi v * (0,6 * F_y * A_w * C_v)$

$\Phi v * Vn = 1,0 * (0,6 * 235 * 2444 * 1,0\,) = 344604 \text{ N} = 344,604 \text{ kN} > Vy = 71,52 \text{ kN}$

$\therefore$ OK

10.2.3 Beam Design According to Eurocode 3



Material: S235 = St37

$E = 210000 \text{ N/mm}^2$
$f_y = 235 \text{ N/mm}^2$

Selected Section: IPN 240

h	240 mm	**I_x**	$42{,}5 \times 10^6$ mm^4	**I_y**	$2{,}21 \times 10^6$ mm^4
b	106 mm	**W_{elx}**	354×10^3 mm^3	**W_{ely}**	$41{,}7 \times 10^3$ mm^3
t_w	8,7 mm	**W_{plx}**	412×10^3 mm^3	**W_{ply}**	70×10^3 mm^3
t_f	13,1 mm	**i_x**	95,9 mm	**i_y**	22 mm
r	5,2 mm	**I_t**	250×10^3 mm^4		
A	4610 mm^2	**I_w**	28700×10^6mm^6		

- Cross Section Classification :

$$\varepsilon = \sqrt{\frac{235}{F_y}} = \sqrt{\frac{235}{235}} = 1{,}0 \text{ (Utilizing Eq. 3.9)}$$

Outstand Flanges in Compression:

$$\frac{c_f}{t_f} = \frac{(b - t_w - 2 * r)/2}{t_f} = \frac{(106 - 8{,}7 - 2 * 5{,}2)/2}{13{,}1} = 3{,}32$$

Limit for Class 1 flange = 9 * ε = 9 *1 = 9,0 (Obtained from Table 3.14)

9,0 > 3,32 ∴ Flanges are Class 1.

Web – internal part in bending:

$$\frac{c_w}{t_w} = \frac{h - 2 * t_f - 2 * r}{t_w} = \frac{240 - 2 * 13{,}1 - 2 * 5{,}2}{8{,}7} = 23{,}38$$

Limit for Class 1 web = 72 * ε = 72 * 1= 72 (Obtained from Table 3.14)

72 > 23,38 ∴ Web is Class 1.

Web and flanges are Class 1, hence the overall cross-section classification Class 1.

In Eurocode 8, if q (value of behavior factor) is greater than 4 with ductility high frame; the cross sectional Class 1 must be used for all elements in the steel structure.(Table 9.27)

This section is suitable for this requirement.

- Bending Resistance :

$M_{c,y,Rd} = \dfrac{W_{pl.x} * f_y}{\gamma_{M0}}$ for Class 1 (Using Eq. 6.25)

$M_{c,y,Rd} = \dfrac{412000 * 235}{1}$ = 96820000 Nmm = 96,82 kNm < M_{ed}=114,75 kNm

∴ NOT OK

Take a greater section!

Selected Section: IPN 260

h	260 mm	$\mathbf{I_x}$	$57{,}4 \times 10^6$ mm^4	$\mathbf{I_y}$	$2{,}88 \times 10^6$ mm^4
b	113 mm	$\mathbf{W_{elx}}$	442×10^3 mm^3	$\mathbf{W_{ely}}$	51×10^3 mm^3
$\mathbf{t_w}$	9,4 mm	$\mathbf{W_{plx}}$	514×10^3 mm^3	$\mathbf{W_{ply}}$	$85{,}9 \times 10^3$ mm^3
$\mathbf{t_f}$	14,1 mm	$\mathbf{i_x}$	104 mm	$\mathbf{i_y}$	23,2 mm
r	5,6 mm	$\mathbf{I_t}$	335×10^3 mm^4		
A	5330 mm^2	$\mathbf{I_w}$	44100×10^6mm^6		

- Cross Section Classification :

$$\varepsilon = \sqrt{\frac{235}{F_y}} = \sqrt{\frac{235}{235}} = 1{,}0 \text{ (Utilizing Eq. 3.9)}$$

Outstand Flanges in Compression:

$$\frac{c_f}{t_f} = \frac{(b - t_w - 2 * r)/2}{t_f} = \frac{(113 - 9{,}4 - 2 * 5{,}6)/2}{14{,}1} = 3{,}28$$

Limit for Class 1 flange = 9 * ε = 9 *1 = 9,0 (Obtained from Table 3.14)

9,0 > 3,28 $\therefore$ Flanges are Class 1.

Web – internal part in bending:

$$\frac{c_w}{t_w} = \frac{h - 2 * t_f - 2 * r}{t_w} = \frac{260 - 2 * 14{,}1 - 2 * 5{,}6}{9{,}4} = 23{,}5$$

Limit for Class 1 web = 72 * ε = 72 * 1= 72 (Obtained from Table 3.14)

72 > 23,5 $\therefore$ Web is Class 1.

Web and flanges are Class 1, hence the overall cross-section classification Class 1.

In Eurocode 8, if q (value of behavior factor) is greater than 4 with ductility high frame; the cross sectional Class 1 must be used for all elements in the steel structure.

This section is suitable for this requirement.

- Bending Resistance :

$$M_{c,x,Rd} = \frac{W_{pl.x} * f_y}{\gamma_{M0}} \quad \text{for Class 1 (Using Eq. 6.25)}$$

$M_{c,x,Rd} = \dfrac{514000 * 235}{1}$ = 120790000 Nmm = 120,79 kNm > M_{ed}=114,75 kNm

$\therefore$ OK

- Shear Resistance :

$V_{pl,Rd} = \dfrac{A_v * (f_y / \sqrt{3})}{\gamma_{M0}}$ (Using Eq. 6.29)

A_v = Shear Area; η = Shear area factor =1,2

A_v= A – 2 * b * t_f + (t_w + 2* r) ≥ η * h_w * t_w (Using Eq. 6.30)

h_w = h – 2 * t_f = 260 – 2 * 14,1 = 231,8 mm

A_v = 5330 – 2 * 113 *14,1 + (9,4 + 2 * 5,6) * 14,1 = 2433,9 mm^2 < 1,2 * 231,8 * 9,4 = 2614,7 mm^2

A_v should not less than 2614,7 mm^2.Take A_v = 2614,7 mm^2

$V_{pl,Rd} = \dfrac{2614{,}7 * (235/\sqrt{3})}{1}$ = 354755 N = 354,755 kN > V_{Ed} = 76,5 kN $\therefore$ OK

- Shear Buckling :

$\dfrac{h_w}{t_w} \leq 72 * \dfrac{\varepsilon}{\eta}$ (Using Eq. 6.34)

$\dfrac{h_w}{t_w} = \dfrac{231{,}8}{9{,}4} = 24{,}7 \leq 72 * \dfrac{\varepsilon}{\eta} = 72 * \dfrac{1}{1{,}2} = 60$ $\therefore$ NO Shear Buckling - OK

- Combined Bending and Shear :

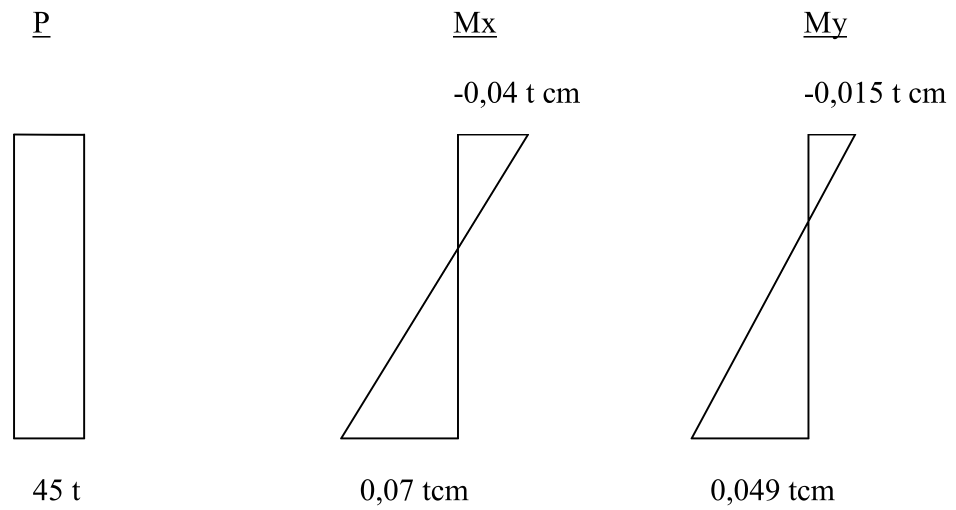
If the shear force (V_{ed}) isn't greater half the plastic shear resistance ($V_{pl,Rd}$) , $M_{y,v,Rd}$ which is reduced moment resistance due to applied shear force can be neglected.

$$V_{ed} = 76{,}5 \text{ kN} < \frac{V_{pl,Rd}}{2} = \frac{344{,}755}{2} = 172{,}38 \text{ kN} \;\; \therefore \;\; \text{OK}$$

10.3 Column Design

For column design the column at axis B2 on ground story is chosen as the case study. This column is designed for the three different methodologies to evaluate the difference. Since these solution steps are for the demonstration of the general design philosophy, only gravity load combinations are taken into account for simplifying the calculations.

10.3.1 Column Design According to TS 648



Vxmax = 0,0002 t No Sidesway in X and Y Direction
Vymax = 0,0003 t No Span Loading

Loading Type: EY

Material: St 37

$\sigma_y = 2,4$ t/cm^2

$\sigma_{all} = 0,6 * 2,4 = 1,44$ t/cm^2

$$\tau_{all} = \frac{\sigma_{all}}{\sqrt{3}} = \frac{1.44}{\sqrt{3}} = 0.831 \ \text{t/cm}^2$$

Selected Section: HE 140 B

h	14 cm	**I_x**	1509 cm^4	**I_y**	594 cm^4
b	14 cm	**W_{elx}**	215,6 cm^3	**W_{ely}**	78,52 cm^3
t_w	0,7 cm	**i_x**	5,93 cm	**i_y**	3,58 cm^3
t_f	1,2 cm				
r	1,2 cm				
A	42,96 cm^2				

L = 350 cm (Length of Column)

$k_x = k_y = 0,8$ (Obtained from Table 5.1)

$L_x = L_y = k_x * L = k_y * L = 0,8 * 350 = 280$ cm

This section must not be greater than the section limitation in TEC 2007.

$$\frac{b}{2 * t_f} \leq 0.3 * \sqrt{\frac{E}{\sigma_y}} \text{ (Obtained from Table 9.8)}$$

$$\frac{b}{2 * t_f} = \frac{14}{2 * 1,2} = 5,83 \leq 0.3 * \sqrt{\frac{E}{\sigma_y}} = 0.3 * \sqrt{\frac{2100}{2,4}} = 8,87 \quad \therefore \text{ OK}$$

If $\left|\frac{N_d}{\sigma_y * A}\right| \geq 0{,}10$, the below formula obtained from Table 9.8 is used.

$$\left|\frac{N_d}{\sigma_y * A}\right| = \left|\frac{45}{2{,}4 * 42{,}96}\right| = 0{,}44 \geq 0{,}10 \;\therefore\; \text{OK}$$

$$\frac{h}{t_w} = \frac{h - 2 * t_f - 2 * r}{t_w} \leq 1{,}33 * \sqrt{\frac{E}{\sigma_y}} * (2{,}1 - \left|\frac{N_d}{\sigma_y * A}\right|) \text{ (Obtained from Table 9.8)}$$

$$\frac{h}{t_w} = \frac{14 - 2 * 1{,}2 - 2 * 1{,}2}{0{,}7} = 13{,}14 \leq 1{,}33 * \sqrt{\frac{2100}{2{,}4}} * (2{,}1 - \left|\frac{45}{2{,}4 * 42{,}96}\right|) = 65{,}44 \;\therefore\; \text{OK}$$

$$\lambda_x = \frac{k_x * L}{i_x} = \frac{280}{5{,}93} = 47{,}22 \text{ (Utilizing Eq. 5.1)}$$

$$\lambda_y = \frac{k_y * L}{i_y} = \frac{280}{3{,}58} = 78{,}21 \text{ (Utilizing Eq. 5.1)}$$

$\lambda_=$ max (λ_x ; λ_y) = max (47,22; 78,21) = 78,21 (Utilizing Eq. 5.3)

$$\lambda_p = \sqrt{\frac{2 * \pi^2 * E}{\sigma_y}} = \sqrt{\frac{2 * 3{,}14^2 * 2100}{2{,}4}} = 131{,}4 \text{ (Utilizing Eq. 5.4)}$$

If $\lambda_{max} \leq \lambda_p$, the below formula is used.

$$\sigma_{bem} = \frac{\left[1 - 0{,}5 * (\lambda / \lambda_p)^2\right] * \sigma_y}{n} \text{ (Utilizing Eq. 5.5)}$$

and

$$n = 1{,}5 + 1{,}2 * (\lambda / \lambda_p) - 0{,}2 * (\lambda / \lambda_p)^3 \geq 1{,}67$$

$$n = 1{,}5 + 1{,}2 * (78{,}21/131{,}4) - 0{,}2 * (78{,}21/131{,}4)^3 = 2{,}17 \geq 1{,}67 \;\therefore\; \text{OK}$$

$$\sigma_{bem} = \frac{\left[1 - 0{,}5 * (78{,}21/131{,}4)^2\right] * 2{,}4}{2{,}17} = 0{,}91 \text{ t/cm}^2$$

$$\sigma_{eb} = \frac{P_{max}}{A} = \frac{45}{42{,}96} = 1{,}047 \text{ t/cm}^2 > \sigma_{bem} = 0{,}91 \text{ t/cm}^2 \quad \therefore\; \text{NOT OK}$$

Take a greater section!

Selected Section: HE 160 B

h	16 cm	**I_x**	2492 cm^4	**I_y**	889,2 cm^4
b	16 cm	**W_{elx}**	311,5 cm^3	**W_{ely}**	111,2 cm^3
t_w	0,8 cm	**i_x**	6,78 cm	**i_y**	4,05 cm
t_f	1,3 cm				
r	1,5 cm				
A	54,25 cm^2				

L = 350 cm (Length of Column)

$k_x = k_y = 0{,}8$ (Obtained from Table 5.1)

$L_x = L_y = k_x * L = k_y * L = 0{,}8 * 350 = 280$ cm

This section must not be greater than the section limitation in TEC 2007.

$$\frac{b}{2 * t_f} \leq 0.3 * \sqrt{\frac{E}{\sigma_y}}$$ (Obtained from Table 9.8)

$$\frac{b}{2*t_f} = \frac{16}{2*1,3} = 6,15 \le 0.3*\sqrt{\frac{E}{\sigma_y}} = 0.3*\sqrt{\frac{2100}{2,4}} = 8,87 \quad \therefore \text{ OK}$$

If $\left|\frac{N_d}{\sigma_y * A}\right| > 0,10$, the below formula obtained from Table 9.8 is used.

$$\left|\frac{N_d}{\sigma_y * A}\right| = \left|\frac{45}{2,4*54,25}\right| = 0,346 > 0,10 \;\therefore \text{ OK}$$

$$\frac{h}{t_w} = \frac{h-2*t_f-2*r}{t_w} \le 1,33*\sqrt{\frac{E}{\sigma_y}}*(2,1-\left|\frac{N_d}{\sigma_y * A}\right|) \text{ (Obtained from Table 9.8)}$$

$$\frac{h}{t_w} = \frac{16-2*1,3-2*1,5}{0,8} = 13 \le 1,33*\sqrt{\frac{2100}{2,4}}*(2,1-\left|\frac{45}{2,4*54,25}\right|) = 65,44 \;\therefore \text{ OK}$$

$$\lambda_x = \frac{k_x * L}{i_x} = \frac{280}{6,78} = 41,29 \text{ (Utilizing Eq. 5.1)}$$

$$\lambda_y = \frac{k_y * L}{i_y} = \frac{280}{4,05} = 69,14 \text{ (Utilizing Eq. 5.1)}$$

$\lambda = \max(\lambda_x ; \lambda_y) = \max(41,29 ; 69,14) = 69,14$ (Utilizing Eq. 5.3)

$$\lambda_p = \sqrt{\frac{2*\pi^2*E}{\sigma_y}} = \sqrt{\frac{2*3,14^2*2100}{2,4}} = 131,4 \text{ (Utilizing Eq. 5.4)}$$

If $\lambda_{max} \le \lambda_p$, the below formula is used.

$$\sigma_{bem} = \frac{\left[1-0,5*(\lambda/\lambda_p)^2\right]*\sigma_y}{n} \text{ (Utilizing Eq. 5.5)}$$

and

$$n = 1,5 + 1,2 * (\lambda / \lambda_p) - 0,2 * (\lambda / \lambda_p)^3 \geq 1,67$$

$$n = 1,5 + 1,2 * (69,14 / 131,4) - 0,2 * (69,14 / 131,4)^3 = 2,1 \geq 1,67 \quad \therefore \text{ OK}$$

$$\sigma_{\text{bem}} = \frac{\left[1 - 0,5 * (69,14 / 131,4)^2\right] * 2,4}{2,1} = 0,98 \text{ t/cm}^2$$

$$\sigma_{\text{eb}} = \frac{P_{\max}}{A} = \frac{45}{54,25} = 0,829 \text{ t/cm}^2 > \sigma_{\text{bem}} = 0,98 \text{ t/cm}^2 \quad \therefore \text{ OK}$$

- Stress Check :

$(M_x)_{max}$ = 0,07 tcm

$$(\sigma_x)_{max} = \frac{(M_x)_{max}}{W_{elx}} \leq \sigma_{all} \quad \text{(Using Eq. 6.1)}$$

$$(\sigma_x)_{max} = \frac{(M_x)_{max}}{W_{elx}} = \frac{0,07}{311,5} = 0,00023 t/cm^2 \geq \sigma_{all} = 1.44 t/cm^2 \quad \therefore \text{ OK}$$

$(M_y)_{max}$ = 0,049 tcm

$$(\sigma_y)_{max} = \frac{(M_y)_{max}}{W_{ely}} = \frac{0,049}{111,2} = 0,00044 t/cm^2 \geq \sigma_{all} = 1.44 t/cm^2 \quad \therefore \text{ OK}$$

- Shear Check :

$(V_x)_{max}$ = 0,0002 t

$(V_y)_{max}$ = 0,0003 t

$$\tau_x = \frac{V_y}{h * t_w} \leq \tau_{all} \quad \text{(Using Eq. 6.4)}$$

$$\tau_x = \frac{(V_y)_{max}}{h * t_w} = \frac{0,0003}{16 * 0,8} = 0,000023 t/cm^2 \leq \tau_{all} = 0,831 t/cm^2 \quad \therefore \text{ OK}$$

$$\tau_y = \frac{V_y * \left[\frac{b}{2} * t_f * (\frac{h - t_f}{2})\right]}{I_x * t_f} + \frac{V_x * \left[\frac{b}{2} * t_f * (\frac{b}{4})\right]}{I_y * t_f} \leq \tau_{all} \quad \text{(Using Eq. 6.2)}$$

$$\tau_y = \frac{0,0003 * \left[\frac{16}{2} * 1,3 * (\frac{16 - 1,3}{2})\right]}{2491 * 1,3} + \frac{0,0002 * \left[\frac{16}{2} * 1,3 * (\frac{16}{4})\right]}{889,2 * 1,3} = 0,0000142 t/cm^2$$

$$\tau_y = 0,0000142 t/cm^2 \leq \tau_{all} = 0,831 t/cm^2 \quad \therefore \text{ OK}$$

- Biaxial Bending Control :

$$\frac{M_{1x}}{W_{elx}} + \frac{M_{1y}}{W_{ely}} \leq \sigma_{all} \quad \text{(Using Eq. 6.1)}$$

$$\frac{0,07}{311,5} + \frac{0,049}{111,2} = 0,00067 t/cm^2 \leq \sigma_{all} = 1,44 t/cm^2 \quad \therefore \text{ OK}$$

$$\frac{M_{2x}}{W_{elx}} + \frac{M_{2y}}{W_{ely}} \leq \sigma_{all} \quad \text{(Using Eq. 6.1)}$$

$$\frac{0,04}{311,5} + \frac{0,015}{111,2} = 0,00026 t/cm^2 \leq \sigma_{all} = 1,44 t/cm^2 \quad \therefore \text{ OK}$$

- Axial Loading and Bending Check :

σ_{bx} = 0,00023 t/cm^2

σ_{by} = 0,00044 t/cm^2

σ_{bem} = 0,98 t/cm^2

σ_{eb} = 0,829 t/cm^2

If $\frac{\sigma_{eb}}{\sigma_{bem}} > 0,15$, below formulae are used.

$$\frac{\sigma_{eb}}{0,6 * \sigma_y} + \frac{\sigma_{bx}}{\sigma_{Bx}} + \frac{\sigma_{by}}{\sigma_{By}} \leq 1,0$$ (Strength Requirement) (Using Eq. 7.2)

$$\frac{\sigma_{eb}}{\sigma_{bem}} + \frac{C_{mx}}{(1 - \frac{\sigma_{eb}}{\sigma_{e'x}})} * \frac{\sigma_{bx}}{\sigma_{Bx}} + \frac{C_{my}}{(1 - \frac{\sigma_{eb}}{\sigma_{e'y}})} * \frac{\sigma_{by}}{\sigma_{By}} \leq 1,0$$ (Stability Requirement) (Using Eq. 7.3)

$F_b = b * t_f = 16 * 1,3 = 20,8$ cm^2 (Cross-sectional area of the compression flange)

$F_c = t_f * b + (h – 2 * t_f) * t_w / 6 = 1,3 * 16 + (16 – 2 * 1,3) * 0,8 / 6 = 22,59$ cm^2

$I_{yc} = t_f * (b)^3 / 12 + (h – 2 * t_f) / 6 * (t_w)^3 / 12$

$I_{yc} = 1,3 * (16)^3 / 12 + (16 – 2 * 1,3) / 6 * (0,8)^3 / 12 = 443,83$ cm^4

$i_{yc} = \sqrt{\frac{I_{yc}}{F_c}} = \sqrt{\frac{443,8}{22,59}} = 4,43 cm$ (Radius of gyration of the compression flange and 1/3 of the compression web area about the symmetry axis)

s = 350 cm (Distance between the supports of the column)

$$C_{bx} = 1,75 + \left[1,05 * (\frac{M_{1x}}{M_{2x}}) \right] + 0,3 * \left[(\frac{M_{1x}}{M_{2x}})^2 \right] \leq 2,3$$ (Using Eq. 6.9)

$$C_{bx} = 1,75 + \left[1,05 * (\frac{0,04}{0,07}) \right] + 0,3 * \left[(\frac{0,04}{0,07})^2 \right] = 2,45$$

But this value must not greater than 2,3. $\therefore$ Take C_{bx} = 2,3 !

$$\frac{s}{i_{yc}} = \frac{350}{4,43} = 79 \leq \sqrt{\frac{30000 * C_{bx}}{\sigma_y}} = \sqrt{\frac{30000 * 2,3}{2,4}} = 169,6$$

$$\sigma_{B1x} = \left[\frac{2}{3} - \frac{\sigma_y * (s/i_{yc})^2}{90000 * C_{bx}} \right] * \sigma_y \leq \sigma_{all} \text{ (Using Eq. 6.5)}$$

$$\sigma_{B1x} = \left[\frac{2}{3} - \frac{2,4 * (79)^2}{90000 * 2,3} \right] * 2,4 = 1,43 t/cm^2 \leq \sigma_{all} = 1,44 t/cm^2$$

$$\sigma_{B2x} = \frac{840 * C_{bx}}{s * \frac{d}{F_b}} \text{ (Using Eq. 6.7)}$$

$$\sigma_{B2x} = \frac{840 * 2,3}{350 * \frac{16}{20,8}} = 7,18 t/cm^2$$

σ_{Bx} = max (σ_{B1x} ; σ_{B2x}) = max (1,43 ; 7,18) = 7,18 t/cm^2 (Utilizing Eq. 6.8)

But this value must be smaller or equal to σ_{all} = 1,44 t/cm^2. So Take σ_{Bx} = 1,44 t/cm^2

σ_{By} = σ_{all} = 1,44 t/cm^2 (for I Section)

$$C_{mx} = 0,6 - 0,4 * (\frac{M_{1x}}{M_{2x}}) \geq 0,4 \text{ (Using Eq. 7.6)}$$

$$C_{mx} = 0,6 - 0,4 * (\frac{0,04}{0,07}) = 0,37$$

But this value must not be smaller than 0,4. So Take $C_{mx} = 0,4$

$$C_{my} = 0,6 - 0,4 * (\frac{M_{1y}}{M_{2y}}) \geq 0,4 \text{ (Using Eq. 7.6)}$$

$$C_{my} = 0,6 - 0,4 * (\frac{0,015}{0,049}) = 0,477 \geq 0,4$$

$$\sigma_{e'x} = \frac{2 * \pi^2 * E}{5 * \lambda_x^2} \text{ (Using Eq. 7.5)}$$

$$\sigma_{e'x} = \frac{2 * 3,14^2 * 2100}{5 * 41,29^2} = 4,86 t / cm^2$$

$$\sigma_{e'y} = \frac{2 * \pi^2 * E}{5 * \lambda_y^2} \text{ (Using Eq. 7.5)}$$

$$\sigma_{e'y} = \frac{2 * 3,14^2 * 2100}{5 * 69,14^2} = 1,733 t / cm^2$$

$$\frac{\sigma_{eb}}{0,6 * \sigma_y} + \frac{\sigma_{bx}}{\sigma_{Bx}} + \frac{\sigma_{by}}{\sigma_{By}} = \frac{0,829}{0,6 * 2,4} + \frac{0,00023}{1,44} + \frac{0,00044}{1,44} = 0,58 \leq 1,0 \therefore \text{ OK}$$

$$\frac{\sigma_{eb}}{\sigma_{bem}} + \frac{C_{mx}}{(1 - \frac{\sigma_{eb}}{\sigma_{e'x}})} * \frac{\sigma_{bx}}{\sigma_{Bx}} + \frac{C_{my}}{(1 - \frac{\sigma_{eb}}{\sigma_{e'y}})} * \frac{\sigma_{by}}{\sigma_{By}} =$$

$$= \frac{0,829}{0,98} + \frac{0,4}{(1 - \frac{0,829}{4,86})} * \frac{0,00023}{1,44} + \frac{0,477}{(1 - \frac{0,829}{1,733})} * \frac{0,00044}{1,44} = 0,846 \leq 1,0$$

∴ OK

- G + Q + 2,5 * E (Utilizing Eq. 9.21 and Table 9.7)

P_{max} = 58,2 t (Compression) and P_{min} = 4,3 t (Tension)

N_{cp}= 1,7 * σ_{bem} * A > Pmax (Using Eq. 9.19)

N_{tp}= σ_a *A_n > Pmin (Using Eq. 9.20)

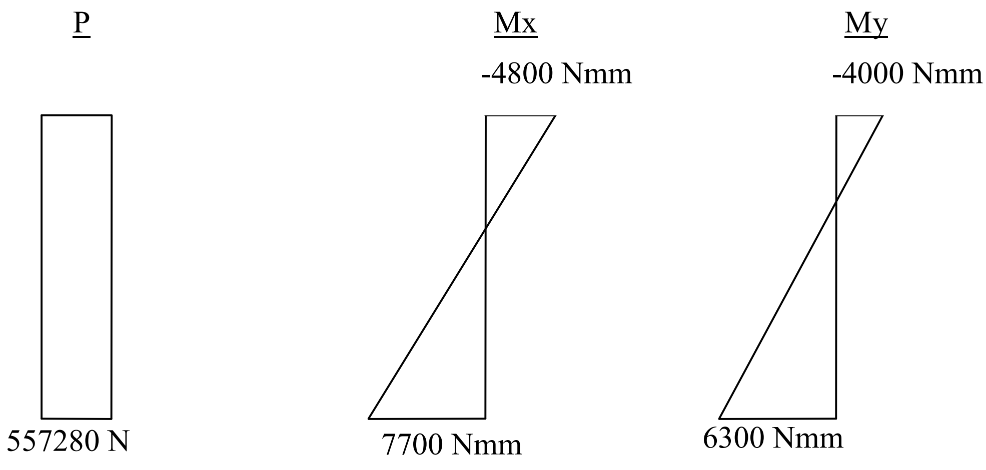
σ_{bem} is different from above value. Because EIY Loading the allowable stresses are found by increasing the values given for loading EY by 15 %.

σ_{bem} = 1,15 * 0,98 = 1,13 t/cm^2

N_{cp} = 1,7 * σ_{bem} * A = 1,7 * 1,13 * 54,25 = 104,21 t > P_{max} = 58,2 t ∴ OK

N_{tp}= σ_a *A_n = 2,4 * 54,25=130,2 t > P_{min} = 4,3 t ∴ OK

10.3.2 Column Design According to LRFD



Vxmax = 2,9 N
Vymax = 3,6 N
M_{xmax}= 7700 Nmm

M_A = 1700 Nmm
M_B = 800 Nmm
M_C = 4600 Nmm

Material: A36 = St 37

$F_y = 235$ N/mm^2

$E = 200000$ N/mm^2

Selected Section: HE 120 B

h	120 mm	**I_x**	$8,644 \times 10^6$ mm^4	**I_y**	$3,175 \times 10^6$ mm^4
b	120 mm	**W_{elx}**	$144,1 \times 10^3$ mm^3	**W_{ely}**	$52,92 \times 10^3$ mm^3
t_w	6,5 mm	**W_{plx}**	$165,2 \times 10^3$ mm^3	**W_{ply}**	$80,97 \times 10^3$ mm^3
t_f	11 mm	**i_x**	50,4 mm	**i_y**	30,6 mm
r	12 mm	**I_t**	$138,4 \times 10^3$ mm^4		
A	3401 mm^2	**I_w**	$9,41 \times 10^9$mm^6		

L = 3500 mm (Length of Column)

$k_x = k_y = 0,8$ (Obtained from Table 5.1)

$L_x = L_y = k_x * L = k_y * L = 0,8 * 3500 = 2800$ mm

- Width-Thickness Ratio :

$$\lambda_f = \frac{b}{2 * t_f} = \frac{120}{2 * 11} = 5,45 \text{ (flange) (Using Eq. 3.7)}$$

$$\lambda_w = \frac{h - 2 * t_f - 2 * r}{t_w} = \frac{120 - 2 * 11 - 2 * 12}{6,5} = 11,38 \text{ (web) (Using Eq. 3.8)}$$

λ_f and λ_w values cannot exceed the limiting width-thickness ratio values obtained from Table 9.21 in Seismic Provisions for Structural Steel Buildings.

For Flanges

$$\lambda_{fps} = 0.30 * \sqrt{\frac{E}{f_y}} = 0,30 * \sqrt{\frac{200000}{235}} = 8,75 \text{ (Obtained from Table 9.21)}$$

For Web

$$C_a = \frac{Pu}{\Phi * Py} \quad \text{(Obtained from Table 9.21)}$$

$P_u = 557280$ N

$P_y = F_y * A = 235 * 3401 = 799235$ N

$$C_a = \frac{P_u}{\Phi * Py} = \frac{557280}{0,9 * 799235} = 0,77 > 0,125$$

$$\lambda_{wps} = 1,12 * \sqrt{\frac{E}{f_y}}(2,33 - C_a) \geq 1,49 * \sqrt{\frac{E}{f_y}} \text{ (Obtained from Table 9.21)}$$

$$\lambda_{wps} = 1,12 * \sqrt{\frac{200000}{235}}(2,33 - 0,77) = 50,97 \geq 1,49 * \sqrt{\frac{E}{f_y}} = 1,49 * \sqrt{\frac{200000}{235}} = 43,7$$

$$\lambda_f = 5,45 \quad < \lambda_{fps} = 8,75 \;\therefore\; \text{OK}$$

$$\lambda_w = 11,38 < \lambda_{wps} = 50,97 \;\therefore\; \text{OK}$$

- Compressive Strength for Flexural Buckling :

$$\lambda_x = \frac{k_x * L}{i_x} = \frac{0,8 * 3500}{50,4} = 55,56 \text{ (Utilizing Eq. 5.1)}$$

$$\lambda_y = \frac{k_y * L}{i_y} = \frac{0,8 * 350}{30,6} = 91,5 \text{ (Utilizing Eq. 5.1)}$$

$\lambda = \max(\lambda_x ; \lambda_y) = \max(55,56 ; 91,5) = 91,5$ (Utilizing Eq. 5.3)

$$\text{If } \lambda \leq 4,71 * \sqrt{\frac{E}{F_y}} \text{ (Utilizing Eq. 5.8)}$$

$$\lambda = 91,5 < 4,71 * \sqrt{\frac{E}{F_y}} = 4,71 * \sqrt{\frac{200000}{235}} = 137,4$$

$$F_E = \frac{\pi^2 * E}{\lambda^2} \text{ (Utilizing Eq. 5.7)}$$

$$F_E = \frac{\pi^2 * E}{\lambda^2} = \frac{3,14^2 * 200000}{(91,5)^2} = 235,8 N/mm^2$$

$$F_{cr} = \left[0,658^{\left(Fy/F_E\right)}\right] * Fy \text{ (Utilizing Eq. 5.8)}$$

$$F_{cr} = \left[0,658^{\left(Fy/F_E\right)}\right] * Fy = \left[0,658^{\left(235/235,8\right)}\right] * 235 = 154,84 N/mm^2$$

$$P_n = F_{cr} * A_g \text{ (Utilizing Eq. 5.10)}$$

$$P_n = F_{cr} * A_g = 154,84 * 3401 = 526610,86 N$$

$\phi_c * P_n = 0,9 * P_n = 0,9 * 526610,84 = 473949,8N$ (Utilizing Eq. 5.11)

$\phi * P_n = 473949,8N < P\max = 557280N$ $\therefore$ NOT OK

Take a greater section!

Selected Section: HE 140 B

h	140 mm	$\mathbf{I_x}$	15,09 x 10^6 mm^4	$\mathbf{I_y}$	5,497 x 10^6 mm^4
b	140 mm	$\mathbf{W_{elx}}$	215,6 x 10^3 mm^3	$\mathbf{W_{ely}}$	78,52 x 10^3 mm^3
$\mathbf{t_w}$	7 mm	$\mathbf{W_{plx}}$	245,4 x 10^3 mm^3	$\mathbf{W_{ply}}$	119,8 x10^3 mm^3
$\mathbf{t_f}$	12 mm	$\mathbf{i_x}$	59,3 mm	$\mathbf{i_y}$	35,8 mm
r	12 mm	$\mathbf{I_t}$	200,6 x 10^3 mm^4		
A	4296 mm^2	$\mathbf{I_w}$	22,48 x $10^9$$mm^6$		

- Width-Thickness Ratio :

$$\lambda_f = \frac{b}{2 * t_f} = \frac{140}{2*12} = 5,83 \text{ (flange) (Using Eq. 3.7)}$$

$$\lambda_w = \frac{h - 2 * t_f - 2 * r}{t_w} = \frac{140 - 2*12 - 2*12}{7} = 13,14 \text{ (web) (Using Eq. 3.8)}$$

λ_f and λ_w values cannot exceed the limiting width-thickness ratio values obtained from Table 9.21 in Seismic Provisions for Structural Steel Buildings.

<u>For Flanges</u>

$$\lambda_{fps} = 0.30 * \sqrt{\frac{E}{f_y}} = 0,30 * \sqrt{\frac{200000}{235}} = 8,75 \text{ (Obtained from Table 9.21)}$$

For Web

$C_a = \frac{Pu}{\Phi * Py}$ (Obtained from Table 9.21)

$P_u = 557280$ N

$P_y = F_y * A = 235 * 4296 = 1009560$ N

$$C_a = \frac{P_u}{\Phi * Py} = \frac{557280}{0{,}9 * 1009560} = 0{,}61 > 0{,}125$$

$$\lambda_{wps} = 1{,}12 * \sqrt{\frac{E}{f_y}}(2{,}33 - C_a) \geq 1{,}49 * \sqrt{\frac{E}{f_y}} \text{ (Obtained from Table 9.21)}$$

$$\lambda_{wps} = 1{,}12 * \sqrt{\frac{200000}{235}}(2{,}33 - 0{,}61) = 56{,}1 \geq 1{,}49 * \sqrt{\frac{E}{f_y}} = 1{,}49 * \sqrt{\frac{200000}{235}} = 43{,}7$$

$\lambda_f = 5{,}83 < \lambda_{fps} = 8{,}75 \;\therefore$ OK

$\lambda_w = 13{,}14 < \lambda_{wps} = 56{,}1 \;\therefore$ OK

- Compressive Strength for Flexural Buckling :

$\lambda_x = \frac{k_x * L}{i_x} = \frac{0{,}8 * 3500}{59{,}3} = 47{,}22$ (Utilizing Eq. 5.1)

$\lambda_y = \frac{k_y * L}{i_y} = \frac{0{,}8 * 350}{35{,}8} = 78{,}21$ (Utilizing Eq. 5.1)

$\lambda = \max(\lambda_x ; \lambda_y) = \max(47{,}22 ; 78{,}21) = 78{,}21$ (Utilizing Eq. 5.3)

If $\lambda < 4{,}71 * \sqrt{\frac{E}{F_y}}$ (Utilizing Eq. 5.8)

$$\lambda = 78{,}21 < 4{,}71 * \sqrt{\frac{E}{F_y}} = 4{,}71 * \sqrt{\frac{200000}{235}} = 137{,}4$$

$$F_E = \frac{\pi^2 * E}{\lambda^2}$$ (Utilizing Eq. 5.7)

$$F_E = \frac{\pi^2 * E}{\lambda^2} = \frac{3{,}14^2 * 200000}{(78{,}21)^2} = 322{,}7 N / mm^2$$

$$F_{cr} = \left[0{,}658^{\left(Fy / F_E \right)} \right] * Fy$$ (Utilizing Eq. 5.8)

$$F_{cr} = \left[0{,}658^{\left(Fy / F_E \right)} \right] * Fy = \left[0{,}658^{\left(235 / 322{,}7 \right)} \right] * 235 = 173{,}26 N / mm^2$$

$P_n = F_{cr} * A_g$ (Utilizing Eq. 5.10)

$$P_n = F_{cr} * A_g = 173{,}26 * 4296 = 744324{,}96 N$$

$\phi * P_n = 0{,}9 * P_n = 0{,}9 * 744324{,}96 = 669892{,}5 N$ (Utilizing Eq. 5.11)

$\phi * P_n = 669892{,}5 N > P\max = 557280 N$ $\therefore$ OK

- Cross Section Classification :

The below values are obtained from Table 3.13.

For Flanges

$\lambda_{pf} = 0,38 * \sqrt{\frac{E}{f_y}} = 0.38 * \sqrt{\frac{200000}{235}} = 11,09$ (Compact)

$\lambda_{rf} = 1,0 * \sqrt{\frac{E}{f_y}} = 1,0 * \sqrt{\frac{200000}{235}} = 29,17$ (Noncompact)

For Web

$\lambda_{pw} = 3,76 * \sqrt{\frac{E}{f_y}} = 3,76 * \sqrt{\frac{200000}{235}} = 109,69$ (Compact)

$\lambda_{rw} = 5,70 * \sqrt{\frac{E}{f_y}} = 5,70 * \sqrt{\frac{200000}{235}} = 166,29$ (Noncompact)

$\lambda_f < \lambda_{pf}$ and $\lambda_w < \lambda_{pw}$

$\therefore$ Section is compact flange and compact web.

So this beam has been solved according to Doubly Symmetric Compact I-Shaped Members in LRFD.

- Yielding (Major Axis):

Mn = Mp = $F_y * Z_x$ = 235 * 245400 = 57669000 Nmm (Using Eq. 6.12)

(Z_x=Wplx=245400 mm^3)

Φ * Mn = 0,9 *57669000= 51902100 Nmm > (Mx)max = 7700 Nmm $\therefore$ OK

- Lateral Torsional Buckling (Major Axis) :

C_b = Lateral Torsional Modification Factor

$$C_b = \frac{12{,}5 * M_{max}}{2{,}5 * M_{max} + 3 * M_A + 4 * M_B + 3 * M_C} * R_M \leq 3{,}0 \text{ (Using Eq. 6.15)}$$

R_m = 1,0 for Doubly Symmetric Members

$$C_b = \frac{12{,}5 * 7700}{2{,}5 * 7700 + 3 * 4600 + 4 * 800 + 3 * 1700} * 1{,}0 = 2{,}33 \leq 3{,}0$$

L_b = 3500 mm

$r_y = i_y$ = 35,8 mm

$S_x = W_{el.x}$ = 215600 mm^3 (Elastic Section Modulus)

$J_c = I_t$ = 200600 mm^4 (Torsional Constant)

$C_w = I_w$ = 22,48 * 10^9 mm^6 (Warping Constant)

$h_0 = h - t_f$ = 140 – 12 = 128 mm (Distance between the flange centroids)

$$L_p = 1{,}76 * r_y * \sqrt{\frac{E}{F_y}} \text{ (Using Eq. 6.16)}$$

$$L_p = 1{,}76 * r_y * \sqrt{\frac{E}{F_y}} = 1{,}76 * 35{,}8 * \sqrt{\frac{200000}{235}} = 1838{,}1mm$$

$$r_{ts} = \sqrt{\frac{\sqrt{I_y * C_w}}{S_x}}$$ and c = 1,0 (for a doubly symmetric I-shape) (Using Eq. 6.18)

$$r_{ts} = \sqrt{\frac{\sqrt{I_y * C_w}}{S_x}} = \sqrt{\frac{\sqrt{5497000 * 22{,}48 * 10^9}}{215600}} = 40{,}38mm$$

L_r = $1{,}95 * r_{ts} * \frac{E}{0{,}7 * Fy} * \sqrt{\frac{J * c}{S_x * h_0}} * \sqrt{1 + \sqrt{1 + 6{,}76 * (\frac{0{,}7 * F_y}{E} * \frac{S_x * h_0}{J * c})^2}}$ (Using Eq.6.17)

$$= 1{,}95 * 40{,}38 * \frac{200000}{0{,}7 * 235} * \sqrt{\frac{200600 * 1{,}0}{215600 * 128}} * \sqrt{1 + \sqrt{1 + 6{,}76 * (\frac{0{,}7 * 235}{200000} * \frac{215600 * 128}{200600 * 1{,}0})^2}}$$

L_r = 11644,3 mm

L_p = 1838,1 mm < L_b = 3500 mm ≤ L_r = 11644,3 mm

$$M_n = C_b * \left[M_p - (M_p - 0{,}7 * F_y * S_x) * (\frac{L_b - L_p}{L_r - L_p}) \right] \leq M_p$$ (Using Eq. 6.13)

$$M_n = 2{,}33 * \left[57669000 - (57669000 - 0{,}7 * 235 * 215600) * (\frac{3500 - 1838{,}1}{11664{,}3 - 1838{,}1}) \right]$$

$M_n = 125619275{,}6 Nmm > Mp = 57669000 Nmm$ (Not greater than M_p)

Take M_n = 576690000Nmm

The nominal flexural strength, M_n, shall be selected as the lowest value obtained from limit states of yielding and lateral torsional buckling.

M_n = min [Mn (from yielding); M_n (from lateral torsional buckling)]

M_n = min [57669000; 57669000] = 57669000 Nmm

Φ * M_n = 0,9 * M_n = 0,9 * 57669000 = 51902100 Nmm (Using Eq. 6.10)

Φ * M_n =51902100 Nmm > $(Mx)_{max}$ = 7700 Nmm ∴ OK

- Yielding (Minor Axis) :

$M_n = M_p = F_y * Z_y \leq 1,6 * F_y * S_y$ (Using Eq. 6.20)

$M_n = M_p = 235 * 119800 = 28153000$ Nmm $\leq 1,6 * 235 * 78520 = 29523520$ Nmm

($Z_y = W_{ply} = 119800$ mm^3 and $S_y = W_{ely} = 78520$ mm^3)

- Flange Local Buckling (Minor Axis) :

$M_n = M_p$ (for compact flange)

$\therefore$ $M_n = 28153000$ Nmm (Using Eq. 6.21)

The nominal flexural strength, M_n, shall be selected as the lowest value obtained from limit states of yielding and flange local buckling.

M_n = min [Mn (from yielding); M_n (from flange local buckling)]

M_n = min [28153000; 28153000] = 28153000 Nmm

$\Phi * M_n = 0,9 * M_n = 0,9 * 28153000 = 25337700$ Nmm (Using Eq. 6.10)

$\Phi * M_n = 25337700$ Nmm $> (My)_{max} = 6300$ Nmm $\therefore$ OK

- Shear (Major Axis) :

$Vn = 0,6 * F_y * Aw * Cv$ (Using Eq. 6.22)

For web of rolled I shaped members with

$$\frac{h}{t_w} = 13,14 \leq 2,24 * \sqrt{\frac{E}{F_y}} = 2,24 * \sqrt{\frac{200000}{235}} = 65,35 \quad \therefore \text{ OK}$$

Take

$C_v = 1,0$ and $\Phi_v = 1,0$

$A_w = h * t_w = 140 * 7 = 980\ mm^2$

$\Phi v * V_n = \Phi v * (0,6 * F_y * Aw * C_v)$

$\Phi v * V_n = 1,0 * (0,6 * 235 * 980 * 1,0) = 138180\ N > V_y = 3,6\ N \quad \therefore$ OK

- Shear (Minor Axis) :

$Vn = 0,6 * F_y * A_w * C_v$ (Using Eq. 6.23)

Take

$C_v = 1,0$ and $\Phi v = 0,9$

$A_w = b_f * t_f = 140 * 12 = 1680\ mm^2$

$\Phi v * V_n = \Phi v * (0,6 * F_y * A_w * C_v)$

$\Phi v * V_n = 0,9 * (0,6 * 235 * 1680 * 1,0) = 213192\ N > V_x = 2,9\ N \quad \therefore$ OK

- Flexure and Axial Force :

$P_r = 557280$ N (from second-order elastic analysis)

$M_{rx} = 7700$ Nmm (from second-order elastic analysis)

$M_{ry} = 6300$ Nmm (from second-order elastic analysis)

$M_{cx} = 51902100$ Nmm

$M_{cy} = 25337700$ Nmm

$$P_c = \phi_c * P_n$$

$$P_c = \phi_c * P_n = 669892{,}5N$$

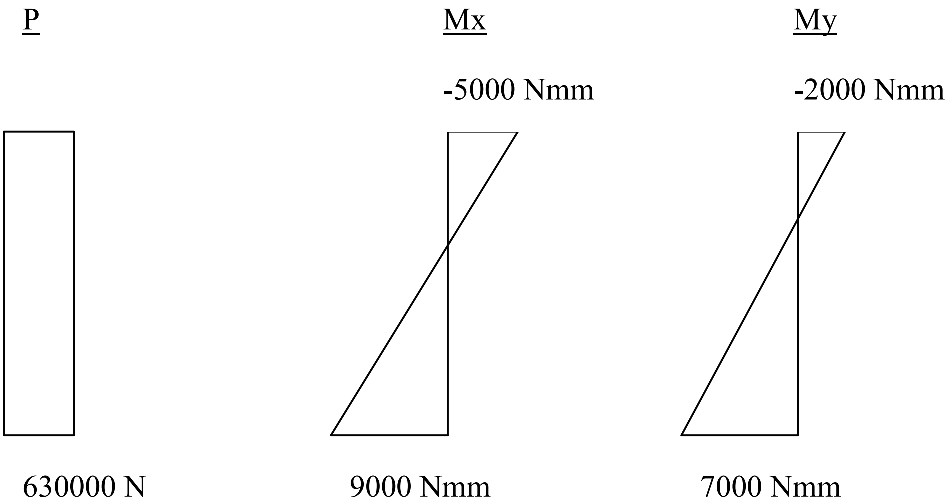
If $\frac{P_r}{P_c} \geq 0{,}2$

$$\frac{P_r}{P_c} + \frac{8}{9} * (\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}) \leq 1{,}0 \text{ (Using Eq. 7.8)}$$

$$\frac{P_r}{P_c} = \frac{557280}{669892{,}5} = 0{,}831 \geq 0{,}2$$

$$\frac{P_r}{P_c} + \frac{8}{9} * (\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}) = \frac{557280}{669892{,}5} + \frac{8}{9} * (\frac{7700}{51902100} + \frac{6300}{25337700}) = 0{,}832 \leq 1{,}0 \quad \therefore \text{ OK}$$

10.3.3 Column Design According to Eurocode 3



$V_{Ed,x}$ = 2,4 N
$V_{Ed,y}$ = 4 N

N_{Ed} = 630000 N

Material: S235 = St 37

F_y = 235 N/mm^2 E = 210000 N/mm^2 G = 81000 N/mm^2

L_{cr} = 3500 mm (Length of Column) $k_x = k_y$ = 0,85 (Obtained from Figure 5.3)

Selected Section: HE 140 B

h	140 mm	$\mathbf{I_x}$	15,09 x 10^6 mm^4	$\mathbf{I_y}$	5,497 x 10^6 mm^4
b	140 mm	$\mathbf{W_{elx}}$	215,6 x 10^3 mm^3	$\mathbf{W_{ely}}$	78,52 x 10^3 mm^3
$\mathbf{t_w}$	7 mm	$\mathbf{W_{plx}}$	245,4 x 10^3 mm^3	$\mathbf{W_{ply}}$	119,8 x10^3 mm^3
$\mathbf{t_f}$	12 mm	$\mathbf{i_x}$	59,3 mm	$\mathbf{i_y}$	35,8 mm
r	12 mm	$\mathbf{I_t}$	200,6 x 10^3 mm^4		
A	4296 mm^2	$\mathbf{I_w}$	22,48 x 10^{9}mm^6		

- Cross Section Classification :

$$\varepsilon = \sqrt{\frac{235}{F_y}} = \sqrt{\frac{235}{235}} = 1{,}0 \text{ (Using Eq. 3.9)}$$

Outstand Flanges in Compression:

$$\frac{c_f}{t_f} = \frac{(b - t_w - 2*r)/2}{t_f} = \frac{(140 - 7 - 2*12)/2}{12} = 4{,}54$$

Limit for Class 1 flange = 9 * ε = 9 *1 = 9,0 (Obtained from Table 3.14)

9,0 > 4,54 $\therefore$ Flanges are Class 1.

Web – internal part in bending:

$$\frac{c_w}{t_w} = \frac{h - 2 * t_f - 2 * r}{t_w} = \frac{140 - 2 * 12 - 2 * 12}{7} = 13{,}14$$

Limit for Class 1 web = 33 * ε = 33 * 1= 33 (Obtained from Table 3.14)

33 > 13,14 ∴ Web is Class 1.

Web and flanges are Class 1, hence the overall cross-section classification Class 1.

In Eurocode 8, if q (value of behavior factor) is greater than 4 with ductility high frame; the cross sectional Class 1 must be used for all elements in the steel structure. (Table 9.27)

- Compression Resistance :

$N_{c,Rd} = \frac{A * f_y}{\gamma_{M0}}$ for Class 1 (Utilizing Eq. 5.13)

$N_{c,Rd} = \frac{A * f_y}{\gamma_{M0}} = \frac{4296 * 235}{1} = 1009560N > N_{Ed} = 630000$ N ∴ OK

- Buckling Resistance :

$N_{b,Rd} = \frac{X * A * f_y}{\gamma_{M1}}$ for Class 1 (Using Eq. 5.16)

$X = \frac{1}{\Phi + \sqrt{\Phi^2 - \overline{\lambda}^2}} \leq 1{,}0$ (Using Eq. 5.18)

$\Phi = 0{,}5 * \left[1 + \alpha * (\overline{\lambda} - 0{,}2) + \overline{\lambda}^2\right]$ (Using Eq. 5.19)

$\overline{\lambda} = \sqrt{\frac{A * f_y}{N_{cr}}}$ for Class 1 (Using Eq. 5.20)

$$N_{cr} = \frac{\pi^2 * E * I}{L_{cr}^{\ 2}} \text{ (Using Eq. 5.22)}$$

Minor Axis (Buckling About Y-Y Axis):

$L_{cr.y} = k_y * L_{cr} = 0,85 * 3500 = 2975$ mm

$$N_{cr,y} = \frac{\pi^2 * E * I_y}{L_{cr,y}^{\ 2}} = \frac{3,14^2 * 2100000 * 5497000}{2975^2} = 1285968N$$

$$\overline{\lambda}_y = \sqrt{\frac{A * f_y}{N_{cr,y}}} = \sqrt{\frac{4296 * 235}{1285968}} = 0,89$$

Buckling Curve:

$$\frac{h}{b} = \frac{140}{140} = 1,0 < 1,2$$

$\therefore$ Buckling Curve = c (Obtained from Table 5.4)

Imperfection Factor:

$\alpha_y = 0,49$ (Obtained from Table 5.3)

$$\Phi_y = 0,5 * \left[1 + \alpha_y * (\overline{\lambda}_y - 0,2) + \overline{\lambda}_y^{\ 2}\right] = 0,5 * \left[1 + 0,49 * (0,89 - 0,2) + 0,89^2\right]$$

$$\Phi_y = 1,06$$

$$X_y = \frac{1}{\Phi_y + \sqrt{\Phi_y^{\ 2} - \overline{\lambda}_y^{\ 2}}} = \frac{1}{1,06 + \sqrt{1,06^2 - 0,89^2}} = 0,61 \leq 1,0$$

$$N_{b,y,Rd} = \frac{X_y * A * f_y}{\gamma_{M1}} = \frac{0,61 * 4296 * 235}{1,0} = 615831N < N_{Ed} = 630000N \quad \therefore \text{ NOT OK}$$

Take a greater section!

Selected Section: HE 160 B

h	160 mm	**I_x**	24,92 x 10^6 mm^4	**I_y**	8,892 x 10^6 mm^4
b	160 mm	**W_{elx}**	311,5 x 10^3 mm^3	**W_{ely}**	111,2 x 10^3 mm3
t_w	8 mm	**W_{plx}**	354 x 10^3 mm^3	**W_{ply}**	170 x 10^3 mm^3
t_f	13 mm	**i_x**	67,8 mm	**i_y**	40,5 mm
r	15 m	**I_t**	312,4 x 10^3 mm^4		
A	5425 mm^2	**I_w**	47,9 x 10^9 mm^6		

- Cross Section Classification :

$$\varepsilon = \sqrt{\frac{235}{F_y}} = \sqrt{\frac{235}{235}} = 1,0 \text{ (Using Eq. 3.9)}$$

Outstand Flanges in Compression:

$$\frac{c_f}{t_f} = \frac{(b - t_w - 2 * r)/2}{t_f} = \frac{(160 - 8 - 2*15)/2}{13} = 4,69$$

Limit for Class 1 flange = 9 * ε = 9 *1 = 9,0 (Obtained from Table 3.14)

9,0 > 4,69 ∴ Flanges are Class 1.

Web – internal part in bending:

$$\frac{c_w}{t_w} = \frac{h - 2 * t_f - 2 * r}{t_w} = \frac{160 - 2*13 - 2*15}{8} = 13$$

Limit for Class 1 web = 33 * ε = 33 * 1= 33 (Obtained from Table 3.14)

33 > 13 ∴ Web is Class 1.

Web and flanges are Class 1, hence the overall cross-section classification Class 1.

In Eurocode 8, if q (value of behavior factor) is greater than 4 with ductility high frame; the cross sectional Class 1 must be used for all elements in the steel structure. (Table 9.27)

- Compression Resistance :

$N_{c,Rd} = \frac{A * f_y}{\gamma_{M0}}$ for Class 1 (Utilizing Eq. 5.13)

$N_{c,Rd} = \frac{A * f_y}{\gamma_{M0}} = \frac{5425 * 235}{1} = 1274875N > N_{Ed} = 630000$ N ∴ OK

- Buckling Resistance :

$N_{b,Rd} = \frac{X * A * f_y}{\gamma_{M1}}$ for Class 1 (Using Eq. 5.16)

$X = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \leq 1{,}0$ (Using Eq. 5.18)

$\Phi = 0{,}5 * \left[1 + \alpha * (\bar{\lambda} - 0{,}2) + \bar{\lambda}^2\right]$ (Using Eq. 5.19)

$\bar{\lambda} = \sqrt{\frac{A * f_y}{N_{cr}}}$ for Class 1 (Using Eq. 5.20)

$N_{cr} = \frac{\pi^2 * E * I}{L_{cr}^2}$ (Using Eq. 5.22)

Minor Axis (Buckling About Y-Y Axis):

$L_{cr.y} = k_y * L_{cr} = 0,85 * 3500 = 2975$ mm

$$N_{cr,y} = \frac{\pi^2 * E * I_y}{L_{cr,y}^{\ 2}} = \frac{3,14^2 * 2100000 * 8892000}{2975^2} = 2082302N$$

$$\overline{\lambda}_y = \sqrt{\frac{A * f_y}{N_{cr,y}}} = \sqrt{\frac{5425 * 235}{2082302}} = 0,78$$

Buckling Curve:

$$\frac{h}{b} = \frac{160}{160} = 1,0 < 1,2$$

$\therefore$ Buckling Curve = c   (Obtained from Table 5.4)

Imperfection Factor:

$\alpha_y = 0,49$ (Obtained from Table 5.3)

$$\Phi_y = 0,5 * \left[1 + \alpha_y * (\overline{\lambda}_y - 0,2) + \overline{\lambda}_y^{\ 2}\right] = 0,5 * \left[1 + 0,49 * (0,78 - 0,2) + 0,78^2\right]$$

$$\Phi_y = 0,95$$

$$X_y = \frac{1}{\Phi_y + \sqrt{\Phi_y^{\ 2} - \overline{\lambda}_y^{\ 2}}} = \frac{1}{0,95 + \sqrt{0,95^2 - 0,78^2}} = 0,67 \leq 1,0$$

$$N_{b,y,Rd} = \frac{X_y * A * f_y}{\gamma_{M1}} = \frac{0,67 * 5425 * 235}{1,0} = 854166N > N_{Ed} = 630000N \quad \therefore \text{ OK}$$

Major Axis (Buckling About X-X Axis) :

$L_{cr.x} = k_x * L_{cr} = 0,85 * 3500 = 2975$ mm

$$N_{cr,x} = \frac{\pi^2 * E * I_x}{L_{cr,x}^2} = \frac{3,14^2 * 2100000 * 24920000}{2975^2} = 5829787N$$

$$\bar{\lambda}_x = \sqrt{\frac{A * f_y}{N_{cr,x}}} = \sqrt{\frac{5425 * 235}{5829787}} = 0,47$$

Buckling Curve:

$$\frac{h}{b} = \frac{160}{160} = 1,0 < 1,2$$

$\therefore$ Buckling Curve = b (Obtained from Table 5.4)

Imperfection Factor:

$\alpha_y = 0,34$ (Obtained from Table 5.3)

$$\Phi_x = 0,5 * \left[1 + \alpha_x * (\bar{\lambda}_x - 0,2) + \bar{\lambda}_x^2\right] = 0,5 * \left[1 + 0,34 * (0,47 - 0,2) + 0,47^2\right]$$

$$\Phi_y = 0,65$$

$$X_x = \frac{1}{\Phi_x + \sqrt{\Phi_x^2 - \bar{\lambda}_x^2}} = \frac{1}{0,65 + \sqrt{0,65^2 - 0,47^2}} = 0,90 \leq 1,0$$

$$N_{b,x,Rd} = \frac{X_x * A * f_y}{\gamma_{M1}} = \frac{0,90 * 5425 * 235}{1,0} = 1147387N > N_{Ed} = 630000N \quad \therefore \text{ OK}$$

- Bending Resistance :

Major Axis (X-X):

$$M_{c,x,Rd} = \frac{W_{pl.x} * f_y}{\gamma_{M0}}$$ for Class 1 (Using Eq. 6.25)

$$M_{c,x,Rd} = \frac{354000 * 235}{1} = 83190000 \text{ Nmm} > M_{Ed,x} = 9000 \text{ Nmm} \therefore \text{ OK}$$

Minor Axis (Y-Y):

$$M_{c,y,Rd} = \frac{W_{pl.y} * f_y}{\gamma_{M0}}$$ for Class 1 (Using Eq. 6.25)

$$M_{c,x,Rd} = \frac{170000 * 235}{1} = 39950000 \text{ Nmm} > M_{Ed,y} = 7000 \text{ Nmm} \therefore \text{ OK}$$

- Shear Resistance:

$$V_{pl,Rd} = \frac{A_v * (f_y / \sqrt{3})}{\gamma_{M0}}$$ (Using Eq. 6.29)

A_v = Shear Area; η = Shear area factor =1,2

$h_w = h - 2 * t_f = 160 - 2 * 13 = 134$ mm (Overall web depth)

Load Parallel to Web:

$V_{Ed,y} = 4$ N

$A_v = A - 2 * b * t_f + (t_w + 2 * r) \geq \eta * h_w * t_w$ (Using Eq. 6.30)

$A_v = 5425 - 2 * 160 * 13 + (8 + 2 * 15) * 13 = 1759 \text{ mm}^2 > 1{,}2 * 134 * 8 = 1286{,}4 \text{ mm}^2$

$$V_{pl,y,Rd} = \frac{A_v * (f_y / \sqrt{3})}{\gamma_{M0}} = \frac{1759 * (235 / \sqrt{3})}{1} = 238656{,}8 \text{ N} > V_{Ed,y} = 4 \text{ N} \therefore \text{ OK}$$

Load Parallel to Flanges:

$V_{Ed,x} = 2,4$ N

$A_w = A - \sum(h_w * t_w) = 5425 - 134*8 = 4353 mm^2$ (Using Eq. 6.31)

$V_{pl,x,Rd} = \frac{A_w * (f_y / \sqrt{3})}{\gamma_{M0}} = \frac{4353*(235/\sqrt{3})}{1} = 590603,3 \text{ N} > V_{Ed,x} = 2,4 \text{ N} \therefore$ OK

- Shear Buckling:

$\frac{h_w}{t_w} \leq 72 * \frac{\varepsilon}{\eta}$ (Using Eq. 6.34)

$\frac{h_w}{t_w} = \frac{134}{8} = 16,75 \leq 72 * \frac{\varepsilon}{\eta} = 72 * \frac{1}{1,2} = 60 \therefore$ NO Shear Buckling - OK

- Cross-Section Resistance Under Bending, Shear and Axial Force:

Bending and Shear:

If the shear forces ($V_{ed,y}$ and $V_{ed,x}$) aren`t greater half the plastic shear resistances ($V_{pl,Rd}$) , $M_{v,y,Rd}$ and $M_{v,x,Rd}$ which are reduced moment resistance due to applied shear force can be neglected.

$V_{ed,x} = 2,4 \text{ N} < \frac{V_{pl,x,Rd}}{2} = \frac{590603}{2} = 295301,5 \text{ N} \therefore$ OK

$V_{ed,y} = 4 \text{ N} < \frac{V_{pl,y,Rd}}{2} = \frac{238656,4}{2} = 119328,2 \text{ N} \therefore$ OK

Bending and Axial Force:

If the following criteria are satisfied, there is no reduction plastic resistance moment for major and minor axis.

Major Axis:

$N_{Ed} \leq 0{,}25 * N_{pl,Rd}$ (Using Eq. 6.38)

$N_{Ed} \leq \dfrac{0{,}5 * h_w * t_w * f_y}{\gamma_{M0}}$ (Using Eq. 6.39)

$N_{Ed} = 630000N > 0{,}25 * N_{pl,Rd} = 0{,}25 * 1274875 = 318718{,}8N$ $\therefore$ Not Satisfied

$N_{Ed} = 630000N \leq \dfrac{0{,}5 * h_w * t_w * f_y}{\gamma_{M0}} = \dfrac{0{,}5 * 134 * 8 * 235}{1{,}0} = 125960N$ $\therefore$ Not Satisfied

$M_{N,x,Rd} = M_{pl,x,Rd} * \dfrac{1-n}{1-0{,}5*a} \leq M_{pl,x,Rd}$ (Utilizing 6.41)

$n = \dfrac{N_{Ed}}{N_{pl,Rd}}$ (Utilizing 6.45)

$n = \dfrac{N_{Ed}}{N_{pl,Rd}} = \dfrac{630000}{1274875} = 0{,}49$

$a = \dfrac{A - 2 * b * t_f}{A}$ (Utilizing 6.44)

$a = \dfrac{A - 2 * b * t_f}{A} = \dfrac{5425 - 2 * 160 * 13}{5425} = 0{,}23$

$$M_{N,x,Rd} = M_{pl,x,Rd} * \frac{1-n}{1-0,5*a} = 83190000 * \frac{1-0,49}{1-0,5*0,23} = 47940000 Nmm$$
$$\leq M_{pl,x,Rd} = 83190000 Nmm$$

$M_{N,x,Rd} = 47940000 Nmm > M_{x,Ed} = 9000 Nmm$ $\therefore$ OK

Minor Axis:

$N_{ed} \leq \frac{h_w * t_w * f_y}{\gamma_{M0}}$ (Utilizing 6.40)

$N_{ed} = 630000N > \frac{h_w * t_w * f_y}{\gamma_{M0}} = \frac{134*8*235}{1,0} = 259440N$ $\therefore$ Not Satisfied

For n > a $M_{N,y,Rd} = M_{pl,y,Rd} * \left[1-(\frac{n-a}{1-a})^2\right]$ (Utilizing 6.43)

n = 0,49 > a = 0,23

$$M_{N,y,Rd} = M_{pl,y,Rd} * \left[1-(\frac{n-a}{1-a})^2\right] = 39950000 * \left[1-(\frac{0,49-0,23}{1-0,23})^2\right] = 35395066 Nmm$$

$M_{N,y,Rd} = 35395066 Nmm > M_{y,Ed} = 7000 Nmm$ $\therefore$ OK

- Biaxial Bending :

α = 2 and β = 5 * n = 5 * 0,49 = 2,45 > 1,0

$(\frac{M_{x,Ed}}{M_{N,x,Rd}})^{\alpha} + (\frac{M_{y,Ed}}{M_{N,y,Rd}})^{\beta} \leq 1,0$ (Utilizing 6.46)

$(\frac{9000}{34790000})^2 + (\frac{7000}{35395066})^{2,4} \cong 0 \leq 1,0$ $\therefore$ OK

- Buckling Resistance in Bending :

$M_{Ed,x}$ = 9000 Nmm

G = 81000 N/mm^2

L_{cr} = 3500 mm (Length of column between points of lateral restraint)

$M_{b,Rd} = X_{LT} * W_x * \frac{f_y}{\gamma_{M1}}$ (Utilizing 6.49)

$W_x = W_{pl,x}$ for Class 1 and 2

Determine M_{cr} :

$$M_{cr} = C_1 * \frac{\pi^2 * E * I_y}{L_{cr}^2} * (\frac{I_w}{I_y} + \frac{L_{cr}^2 * G * I_T}{\pi^2 * E * I_y})^{0,5}$$ (Using Eq. 6.56)

$$\psi = \frac{M_{1x}}{M_{2x}} = \frac{5000}{9000} = -0,56$$ (Double Curvature) (Using Eq. 6.58)

$C_1 = 1,88 - 1,4 * \psi + 0,52 * \psi^2 \leq 2,7$ (Using Eq. 6.57)

$C_1 = 1,88 - 1,4 * (-0,56) + 0,52 * (-0,56)^2 = 2,82 > 2,7$

This value must not be greater than 2,7. Take $C_1 = 2,7$

$$M_{cr} = 2,7 * \frac{3,14^2 * 2100000 * 8892000}{3500^2} * (\frac{47,9 * 10^9}{8892000} + \frac{3500^2 * 81000 * 312400}{3,14^2 * 2100000 * 8892000})^{0,5}$$

$$M_{cr} = 604938865 Nmm$$

Buckling Curve:

$$\frac{h}{b} = \frac{160}{160} = 1 < 2$$

$\therefore$ Buckling Curve = a (Obtained from Table 6.3)

Imperfection Factor:

$\alpha_{LT} = 0,21$ (Obtained from Table 6.2)

$$\overline{\lambda}_{LT} = \sqrt{\frac{W_{pl,x} * f_y}{M_{cr}}} \quad \text{(Using Eq. 6.55)}$$

$$\overline{\lambda}_{LT} = \sqrt{\frac{W_{pl,x} * f_y}{M_{cr}}} = \sqrt{\frac{354000 * 235}{6049388865}} = 0,37$$

$$\Phi_{LT} = 0,5 * \left[1 + \alpha_{LT} * (\overline{\lambda}_{LT} - 0,2) + \overline{\lambda}_{LT}^{\,2}\right] \text{ (Utilizing Eq. 6.54)}$$

$$\Phi_{LT} = 0,5 * \left[1 + 0,21 * (0,37 - 0,2) + 0,37^2\right] = 0,59$$

$$X_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^{\,2} - \overline{\lambda}_{LT}^{\,2}}} = \frac{1}{0,59 + \sqrt{0,59^2 - 0,37^2}} = 0,95 \leq 1,0 \text{ (Utilizing Eq. 6.53)}$$

$$M_{b,Rd} = X_{LT} * W_{pl,x} * \frac{f_y}{\gamma_{M1}} \quad \text{(Utilizing Eq. 6.49)}$$

$$M_{b,Rd} = X_{LT} * W_{pl,x} * \frac{f_y}{\gamma_{M1}} = 0,95 * 354000 * \frac{235}{1} = 79030500 Nmm > M_{Ed,x} = 9000 \text{ Nmm}$$

$\therefore$ OK

- Buckling Resistance in Combined Bending and Axial Compression :

The column is laterally and torsionally unrestrained so it is susceptible to torsional deformations. When members are subject to combined bending and axial compression, both following equations are satisfied.

$$\frac{N_{Ed}}{X_x * N_{Rk}/\gamma_{M1}} + k_{xx} * \frac{M_{x,Ed}}{X_{LT} * M_{x,Rk}/\gamma_{M1}} + k_{xy} * \frac{M_{y,Ed}}{M_{y,Rk}/\gamma_{M1}} \leq 1{,}0 \text{ (Utilizing Eq. 7.16)}$$

$$\frac{N_{Ed}}{X_y * N_{Rk}/\gamma_{M1}} + k_{yx} * \frac{M_{x,Ed}}{X_{LT} * M_{x,Rk}/\gamma_{M1}} + k_{yy} * \frac{M_{y,Ed}}{M_{y,Rk}/\gamma_{M1}} \leq 1{,}0 \text{ (Utilizing Eq. 7.17)}$$

$N_{Rk} = N_{c,Rd} = 1274875$ N

$M_{x,Rk} = M_{c,x,Rd} = 83190000$ Nmm

$M_{y,Rk} = M_{c,y,Rd} = 39950000$ Nmm

Equivalent Uniform Moment Factors:

$C_{Mi} = 0{,}6 + 0{,}4 * \psi \geq 0{,}4$ (Obtained from Table 7.4)

X-X Bending and in plane support:

$$\psi_x = \frac{M_{1x}}{M_{2x}} = \frac{5000}{9000} = -0{,}56 \text{ (Double Curvature) (Using Eq. 6.58)}$$

$C_{Mx} = 0{,}6 + 0{,}4 * \psi_x = 0{,}6 + 0{,}4 * (-0{,}56) = 0{,}376 < 0{,}4$ (Obtained from Table 7.4)

Take $C_{Mx} = 0{,}4$

Y-Y Bending and in plane support:

$$\psi_x = \frac{M_{1x}}{M_{2x}} = \frac{2000}{7000} = -0{,}286 \text{ (Double Curvature) (Using Eq. 6.58)}$$

C_{My} = 0,6 + 0,4 * ψ_y = 0,6 + 0,4 * (-0,286) = 0,49 > 0,4 (Obtained from Table 7.4)

X-X Bending and out-of- plane support:

$$\psi_{LT} = \frac{M_{1x}}{M_{2x}} = \frac{5000}{9000} = -0{,}56 \text{ (Double Curvature) (Using Eq. 6.58)}$$

C_{mLT} = 0,6 + 0,4 * ψ_{LT} = 0,6 + 0,4 * (-0,56) = 0,376 < 0,4 (Obtained from Table 7.4)

Take C_{mLT} = 0,4

Interaction Factors (k_{ij}) (Obtained from Table 7.13 due to member susceptible to torsional deformations):

For Class 1 and I Sections:

$$k_{xx} = C_{mx} * \left[1 + (\overline{\lambda}_x - 0{,}2) * \frac{N_{Ed}}{X_x * N_{Rk} / \gamma_{M1}}\right] \le C_{mx} * \left[1 + 0{,}8 * \frac{N_{Ed}}{X_x * N_{Rk} / \gamma_{M1}}\right]$$

$$k_{xx} = 0{,}4 * \left[1 + (0{,}47 - 0{,}2) * \frac{630000}{0{,}9 * 1274875/1{,}0}\right] \le 0{,}4 * \left[1 + 0{,}8 * \frac{630000}{0{,}9 * 1274875/1{,}0}\right]$$

$$k_{xx} = 0{,}46 < 0{,}575$$

$$k_{yy} = C_{my} * \left[1 + (2\overline{\lambda}_y - 0{,}6) * \frac{N_{Ed}}{X_y * N_{Rk} / \gamma_{M1}}\right] \le C_{my} * \left[1 + 1{,}4 * \frac{N_{Ed}}{X_y * N_{Rk} / \gamma_{M1}}\right]$$

$$k_{yy} = 0{,}49 * \left[1 + (2 * 0{,}78 - 0{,}6) * \frac{630000}{0{,}67 * 1274875/1{,}0}\right] \leq 0{,}4 * \left[1 + 1{,}4 * \frac{630000}{0{,}67 * 1274875/1{,}0}\right]$$

$$k_{yy} = 0{,}83 < 0{,}966$$

$$k_{xy} = 0{,}6 * k_{yy}$$

$$k_{xy} = 0{,}6 * 0{,}83 = 0{,}498$$

For $\overline{\lambda}_y \geq 0{,}4$

$$k_{yx} = 1 - \frac{0{,}1 * \overline{\lambda}_y}{C_{MLT} - 0{,}25} * \frac{N_{Ed}}{X_y * N_{Rk}/\gamma_{M1}} \geq 1 - \frac{0{,}1}{C_{MLT} - 0{,}25} * \frac{N_{Ed}}{X_y * N_{Rk}/\gamma_{M1}}$$

$$k_{yx} = 1 - \frac{0{,}1 * 0{,}78}{0{,}4 - 0{,}25} * \frac{630000}{0{,}67 * 1274875/1{,}0} \geq 1 - \frac{0{,}1}{0{,}4 - 0{,}25} * \frac{630000}{0{,}67 * 1274875/1{,}0}$$

$$k_{yx} = 0{,}62 > 0{,}5$$

$$\frac{N_{Ed}}{X_x * N_{Rk}/\gamma_{M1}} + k_{xx} * \frac{M_{x,Ed}}{X_{LT} * M_{x,Rk}/\gamma_{M1}} + k_{xy} * \frac{M_{y,Ed}}{M_{y,Rk}/\gamma_{M1}} \leq 1{,}0 \text{ (Utilizing Eq. 7.16)}$$

$$\frac{630000}{0{,}9 * 1274875/1{,}0} + 0{,}46 * \frac{9000}{0{,}95 * 83190000/1{,}0} + 0{,}498 * \frac{7000}{39950000/1{,}0} = 0{,}55 \leq 1{,}0$$

$\therefore$ OK

$$\frac{N_{Ed}}{X_y * N_{Rk}/\gamma_{M1}} + k_{yx} * \frac{M_{x,Ed}}{X_{LT} * M_{x,Rk}/\gamma_{M1}} + k_{yy} * \frac{M_{y,Ed}}{M_{y,Rk}/\gamma_{M1}} \leq 1{,}0 \text{ (Utilizing Eq. 7.17)}$$

$$\frac{630000}{0{,}67 * 1274875/1{,}0} + 0{,}62 * \frac{9000}{0{,}95 * 83190000/1{,}0} + 0{,}83 * \frac{7000}{39950000/1{,}0} = 0{,}74 \leq 1{,}0$$

$\therefore$ OK

10.4 Brace Design

Designing braces, the brace having the most unfavorable load is taken as the case study considering all load combinations. The brace in the x-direction on the ground story is designed for again three design methodologies.

10.4.1 Brace Design According to TS 648

Loading Type: EIY

Material: St 37

$\sigma_y = 2,4$ t/cm^2

$\sigma_{all} = 1,15 * 0,6 * 2,4 = 1,656$ t/cm^2

$$\tau_{all} = \frac{\sigma_{all}}{\sqrt{3}} = \frac{1,656}{\sqrt{3}} = 0,956 \text{ t/cm}^2$$

Compression = 23,7 t

Tension = 14 t

L = 442 cm (Length of Brace)

Selected Section: Tube 100 x 100 x 10

A	27,2 cm^2	**b**	10 cm
i_{min}	3,67 cm	**t**	0,8 cm

This section must not be greater than the section limitation in TEC 2007.

$$\frac{b}{t} \le 0,7 * \sqrt{\frac{E}{\sigma_y}}$$ (Obtained from Table 9.8)

$$\frac{b}{t} = \frac{10}{0,8} = 12,5 \le 0,7 * \sqrt{\frac{E}{\sigma_y}} = 0,7 * \sqrt{\frac{2100}{2,4}} = 20,7 \quad \therefore \text{ OK}$$

For Brace: k = 1

$$\frac{k * L}{i_{\min}} < 4 * \sqrt{\frac{E}{\sigma_y}} \quad \text{(Using Eq. 9.23)}$$

$$\frac{k * L}{i_{\min}} = \frac{1 * 442}{3,67} = 120,44 > 4 * \sqrt{\frac{E}{\sigma_y}} = 4 * \sqrt{\frac{2100}{2,4}} = 118,32 \quad \therefore \text{ NOT OK}$$

Take a greater section!

Selected Section: Tube 120 x 120 x 6

A	27,3 cm^2	**b**	12 cm
$\mathbf{i_{min}}$	4,58 cm	**t**	0,63 cm

This section must not be greater than the section limitation in TEC 2007.

$$\frac{b}{t} \le 0,7 * \sqrt{\frac{E}{\sigma_y}} \quad \text{(Obtained from Table 9.8)}$$

$$\frac{b}{t} = \frac{12}{0,63} = 19 \le 0,7 * \sqrt{\frac{E}{\sigma_y}} = 0,7 * \sqrt{\frac{2100}{2,4}} = 20,7 \quad \therefore \text{ OK}$$

For Brace: k = 1

$$\frac{k * L}{i_{\min}} < 4 * \sqrt{\frac{E}{\sigma_y}} \quad \text{(Using Eq. 9.23)}$$

$$\frac{k*L}{i_{\min}} = \frac{1*442}{4{,}49} = 96{,}51 < 4*\sqrt{\frac{E}{\sigma_y}} = 4*\sqrt{\frac{2100}{2{,}4}} = 118{,}32 \;\therefore\; \text{OK}$$

- Axial Tension :

$P_{all} = \sigma_{all} * A$

$P_{all} = 1{,}656 * 27{,}3 = 45{,}2$ t > $P_{Tension} = 14$ t ∴ OK

- Axial Compression :

k = 1 L = 442 cm (Length of Brace) $i_{min} = 4{,}49$ cm

$$\lambda_{max} = \frac{k*L}{i_{\min}} = \frac{1*442}{4{,}49} = 96{,}51 \text{ (Utilizing Eq. 5.1)}$$

$$\lambda_p = \sqrt{\frac{2*\pi^2*E}{\sigma_y}} = \sqrt{\frac{2*3{,}14^2*2100}{2{,}4}} = 131{,}4 \text{ (Utilizing Eq. 5.4)}$$

If $\lambda max \leq \lambda_p$, the below formulae is used.

$$\sigma_{bem} = 1{,}15*\frac{\left[1-0{,}5*(\lambda/\lambda_p)^2\right]*\sigma_y}{n} \text{ (Utilizing Eq. 5.5)}$$

and

$$n = 1{,}5 + 1{,}2*(\lambda/\lambda_p) - 0{,}2*(\lambda/\lambda_p)^3 \geq 1{,}67$$

$$n = 1{,}5 + 1{,}2*(96{,}51/131{,}4) - 0{,}2*(96{,}51/131{,}4)^3 = 2{,}27 \geq 1{,}67 \;\therefore\; \text{OK}$$

$$\sigma_{bem} = 1{,}15 * \frac{\left[1 - 0{,}5 * (96{,}51 / 131{,}4)^2\right] * 2{,}4}{2{,}27} = 0{,}89 \ \text{t/cm}^2$$

$P_{all} = A * \sigma_{bem} = 27{,}3 * 0{,}89 = 24{,}3$ t $> P_{Compression} = 23{,}7$ t $\therefore$ OK

10.4.2 Brace Design According to LRFD

Material: A36 = St 37

$F_y = 235$ N/mm^2

$E = 200000$ N/mm^2

Compression = 188600 N

Tension = 93810 N

L = 4420 mm

For Brace: k = 1

Selected Section: Tube 80 x 80 x 6

A	1720 mm^2	**b**	80 mm
i_{min}	29,4 mm	**t**	6,3 mm
r_0 (external)	15,75 mm		

For Rectangular HSS in axial and flexural compression:

$$\frac{b}{t} \, or \, \frac{h}{t_w} \leq \lambda_{ps} = 0{,}64 * \sqrt{\frac{E}{F_y}} \quad \text{(Obtained from Table 9.21)}$$

$\lambda_w = \lambda_f = \frac{b}{t} = \frac{b - 2*r}{t} = \frac{80 - 2*15{,}75}{6{,}3} = 7{,}7 \le \lambda_{ps} = 0{,}64*\sqrt{\frac{E}{F_y}} = 0{,}64*\sqrt{\frac{200000}{235}} = 18{,}7$

$\therefore$ OK

- Axial Tension :

Φ_t = 0,9 (Using Eq. 4.4)

$P_N = A_g * F_y$ (Using Eq. 4.4)

$\Phi_t * P_N$ = 0,9 * (1720 * 235) = 363780 N > $P_{Tension}$ = 93810 N $\therefore$ OK

- Axial Compression :

$\lambda = \frac{k*L}{i_{min}} = \frac{1*4420}{29{,}4} = 150{,}34$ (Utilizing Eq. 5.1)

$\lambda > 4{,}71*\sqrt{\frac{E}{F_y}}$ (Utilizing Eq. 5.9)

$\lambda = 150{,}34 > 4{,}71*\sqrt{\frac{E}{F_y}} = 4{,}71*\sqrt{\frac{200000}{235}} = 137{,}4$

$F_E = \frac{\pi^2 * E}{\lambda^2}$ (Utilizing Eq. 5.7)

$F_E = \frac{\pi^2 * E}{\lambda^2} = \frac{3{,}14^2 * 200000}{(150{,}34)^2} = 87{,}3 N/mm^2$

$F_{cr} = 0{,}877 * F_E$ (Utilizing Eq. 5.9)

$F_{cr} = 0{,}877 * F_E = 0{,}877 * 87{,}3 = 76{,}59 N/mm^2$

$P_n = F_{cr} * A_g$ (Utilizing Eq. 5.10)

$P_n = F_{cr} * A_g = 76,59 * 1720 = 131734,8N$

$\phi * P_n = 0,9 * P_n = 0,9 * 131734,8 = 118561,32N$ (Utilizing Eq. 5.11)

$\phi * P_n = 118561,32N < P_{Compression} = 188600N$ $\therefore$ NOT OK

Take a greater section!

Selected Section: Tube 100 x 100 x 5

A	1840 mm^2	**b**	100 mm
i_{min}	38,4 mm	**t**	5 mm
r_0 (external)	10 mm		

$$\lambda_w = \lambda_f = \frac{b}{t} = \frac{b - 2 * r}{t} = \frac{100 - 2 * 10}{5} = 16 \leq \lambda_{ps} = 0,64 * \sqrt{\frac{E}{F_y}} = 0,64 * \sqrt{\frac{200000}{235}} = 18,7$$

$\therefore$ OK (Obtained from Table 9.21)

- Axial Tension :

$\Phi_t = 0,9$ (Using Eq. 4.4)

$P_N = A_g * F_y$ (Using Eq. 4.4)

$\Phi_t * P_N = 0,9 * (1840 * 235) = 389160$ N $> P_{Tension} = 93810$ N $\therefore$ OK

- Axial Compression :

$$\lambda = \frac{k * L}{i_{min}} = \frac{1 * 4420}{38,4} = 115,1$$ (Utilizing Eq. 5.1)

$$\lambda < 4{,}71 * \sqrt{\frac{E}{F_y}}$$ (Utilizing Eq. 5.8)

$$\lambda = 115{,}1 < 4{,}71 * \sqrt{\frac{E}{F_y}} = 4{,}71 * \sqrt{\frac{200000}{235}} = 137{,}4$$

$$F_E = \frac{\pi^2 * E}{\lambda^2}$$ (Utilizing Eq. 5.7)

$$F_E = \frac{\pi^2 * E}{\lambda^2} = \frac{3{,}14^2 * 200000}{(115{,}1)^2} = 148{,}8 N / mm^2$$

$$F_{cr} = \left[0{,}658^{\left(Fy/_{F_E} \right)} \right] * Fy$$ (Utilizing Eq. 5.8)

$$F_{cr} = \left[0{,}658^{\left(Fy/_{F_E} \right)} \right] * Fy = \left[0{,}658^{\left(235/_{148{,}8} \right)} \right] * 235 = 121{,}3 N / mm^2$$

$$P_n = F_{cr} * A_g$$ (Utilizing Eq. 5.10)

$$P_n = F_{cr} * A_g = 121{,}3 * 1840 = 223192 N$$

$$\phi * P_n = 0{,}9 * P_n = 0{,}9 * 223192 = 200872{,}8 N$$ (Utilizing Eq. 5.11)

$$\phi * P_n = 200872{,}8 N < P_{Compression} = 188600 N \quad \therefore \quad \text{OK}$$

10.4.3 Brace Design According to Eurocode 3

Material: S235 = St 37

F_y = 235 N/mm^2 E = 210000 N/mm^2 G = 81000 N/mm^2

L_{cr} = 4420 mm (Length of Brace)

For Brace: k = 1

Compression = 310400 N

Tension = 2140000 N

Selected Section: Tube 100 x 100 x 5

A	1840 mm^2	**b**	100 mm
I	2710000 mm^4	**t**	5 mm

- Cross Section Classification :

$$\varepsilon = \sqrt{\frac{235}{F_y}} = \sqrt{\frac{235}{235}} = 1{,}0 \text{ (Using Eq. 3.9)}$$

For RHS:

$$\frac{c}{t} = \frac{b - 3*t}{t} = \frac{100 - 3*5}{5} = 17$$

Limit for Class 1 = 33 * ε =33 * 1 = 33,0 (Obtained from Table 3.14)

33,0 > 17 $\therefore$ Overall Classification of Cross Section = Class 1

In Eurocode 8, if q (value of behavior factor) is greater than 4 with ductility high frame; the cross sectional Class 1 must be used for all elements in the steel structure. (Table 9.27)

- Tension Resistance :

$N_{Ed,t}$ = 214000 N

$$N_{t,Rd} = \frac{A * f_y}{\gamma_{M0}} \quad \text{(Using Eq. 4.7)}$$

$$N_{t,Rd} = \frac{A * f_y}{\gamma_{M0}} = \frac{1840 * 235}{1} = 432400N > N_{Ed,t} = 214000N \quad \therefore \quad \text{OK}$$

- Compression Resistance :

$$N_{c,Rd} = \frac{A * f_y}{\gamma_{M0}} \quad \text{for Class 1 (Using Eq. 5.13)}$$

$$N_{c,Rd} = \frac{A * f_y}{\gamma_{M0}} = \frac{1840 * 235}{1} = 432400N > N_{Ed,c} = 310400\ \text{N} \quad \therefore \ \text{OK}$$

- Buckling Resistance :

$$N_{b,Rd} = \frac{X * A * f_y}{\gamma_{M1}} \quad \text{for Class 1 (Using Eq. 5.16)}$$

$$X = \frac{1}{\Phi + \sqrt{\Phi^2 - \overline{\lambda}^2}} \leq 1{,}0 \quad \text{(Using Eq. 5.18)}$$

$$\Phi = 0{,}5 * \left[1 + \alpha * (\overline{\lambda} - 0{,}2) + \overline{\lambda}^2\right] \quad \text{(Using Eq. 5.19)}$$

$$\overline{\lambda} = \sqrt{\frac{A * f_y}{N_{cr}}} \quad \text{for Class 1 (Using Eq. 5.20)}$$

$$N_{cr} = \frac{\pi^2 * E * I}{L_{cr}^{\ 2}} \quad \text{(Using Eq. 5.22)}$$

$L_{cr} = k * L_{cr} = 1 * 4420 = 4420$ mm

$$N_{cr} = \frac{\pi^2 * E * I}{L_{cr}^{\ 2}} = \frac{3{,}14^2 * 2100000 * 2710000}{4420^2} = 287212{,}5N$$

$$\overline{\lambda} = \sqrt{\frac{A * f_y}{N_{cr}}} = \sqrt{\frac{1840 * 235}{287212{,}5}} = 1{,}23$$

Buckling Curve:

Buckling Curve = a (Obtained from Table 5.4)

Imperfection Factor:

$\alpha_y = 0{,}21$ (Obtained from Table 5.3)

$$\Phi = 0{,}5 * \left[1 + \alpha * (\overline{\lambda} - 0{,}2) + \overline{\lambda}^2\right] = 0{,}5 * \left[1 + 0{,}21 * (1{,}23 - 0{,}2) + 1{,}23^2\right]$$

$$\Phi_y = 1{,}36$$

$$X = \frac{1}{\Phi + \sqrt{\Phi^2 - \overline{\lambda}^2}} = \frac{1}{1{,}36 + \sqrt{1{,}36^2 - 1{,}23^2}} = 0{,}51 \leq 1{,}0$$

$$N_{b,Rd} = \frac{X * A * f_y}{\gamma_{M1}} = \frac{0{,}51 * 1840 * 235}{1{,}0} = 220524N < N_{Ed,c} = 310400N \quad \therefore \text{ NOT OK}$$

Take a greater section!

Selected Section: Tube 120 x 120 x 5

A	2240 mm^2	**b**	120 mm
I	4850000 mm^4	**t**	5 mm

- Cross Section Classification :

$$\varepsilon = \sqrt{\frac{235}{F_y}} = \sqrt{\frac{235}{235}} = 1{,}0 \text{ (Using Eq. 3.9)}$$

For RHS:

$$\frac{c}{t} = \frac{b - 3 * t}{t} = \frac{120 - 3 * 5}{5} = 21$$

Limit for Class 1 = 33 * ε =33 * 1 = 33,0 (Obtained from Table 3.14)

33,0 > 21 ∴ Overall Classification of Cross Section = Class 1

In Eurocode 8, if q (value of behavior factor) is greater than 4 with ductility high frame; the cross sectional Class 1 must be used for all elements in the steel structure.

- Tension Resistance :

$N_{Ed,t}$ = 214000 N

$$N_{t,Rd} = \frac{A * f_y}{\gamma_{M0}} \quad \text{(Using Eq. 4.7)}$$

$$N_{t,Rd} = \frac{A * f_y}{\gamma_{M0}} = \frac{2240 * 235}{1} = 526400N > N_{Ed,t} = 214000N \quad \therefore \quad \text{OK}$$

- Compression Resistance :

$$N_{c,Rd} = \frac{A * f_y}{\gamma_{M0}} \qquad \text{for Class 1 (Using Eq. 5.13)}$$

$N_{c,Rd} = \dfrac{A * f_y}{\gamma_{M0}} = \dfrac{2240 * 235}{1} = 526400N > N_{Ed,c} = 310400$ N $\therefore$ OK

- Buckling Resistance :

$N_{b,Rd} = \dfrac{X * A * f_y}{\gamma_{M1}}$ for Class 1 (Using Eq. 5.16)

$X = \dfrac{1}{\Phi + \sqrt{\Phi^2 - \overline{\lambda}^2}} \leq 1{,}0$ (Using Eq. 5.18)

$\Phi = 0{,}5 * \left[1 + \alpha * (\overline{\lambda} - 0{,}2) + \overline{\lambda}^2\right]$ (Using Eq. 5.19)

$\overline{\lambda} = \sqrt{\dfrac{A * f_y}{N_{cr}}}$ for Class 1 (Using Eq. 5.20)

$N_{cr} = \dfrac{\pi^2 * E * I}{{L_{cr}}^2}$ (Using Eq. 5.22)

$L_{cr} = k * L_{cr} = 1 * 4420 = 4420$ mm

$$N_{cr} = \frac{\pi^2 * E * I}{{L_{cr}}^2} = \frac{3{,}14^2 * 2100000 * 4850000}{4420^2} = 514015N$$

$$\overline{\lambda} = \sqrt{\frac{A * f_y}{N_{cr}}} = \sqrt{\frac{2240 * 235}{514015}} = 1{,}01$$

Buckling Curve:

Buckling Curve = a (Obtained from Table 5.4)

Imperfection Factor:

$\alpha_y = 0{,}21$ (Obtained from Table 5.4)

$$\Phi = 0{,}5 * \left[1 + \alpha * (\overline{\lambda} - 0{,}2) + \overline{\lambda}^2\right] = 0{,}5 * \left[1 + 0{,}21 * (1{,}01 - 0{,}2) + 1{,}01^2\right]$$

$$\Phi_y = 1{,}1$$

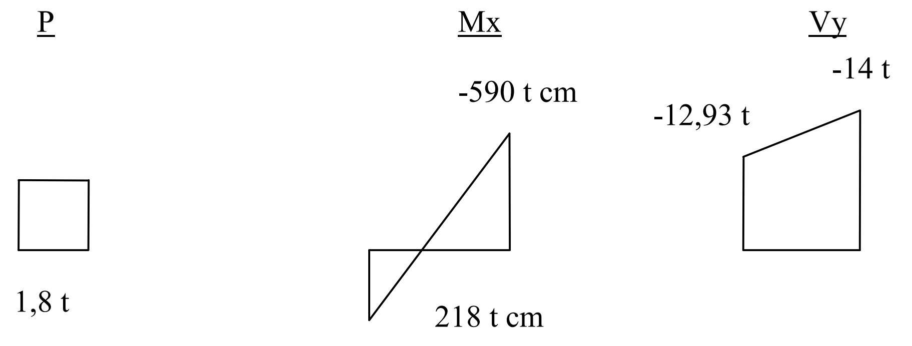
$$X = \frac{1}{\Phi + \sqrt{\Phi^2 - \overline{\lambda}^2}} = \frac{1}{1{,}1 + \sqrt{1{,}1^2 - 1{,}01^2}} = 0{,}65 \leq 1{,}0$$

$$N_{b,Rd} = \frac{X * A * f_y}{\gamma_{M1}} = \frac{0{,}65 * 2240 * 235}{1{,}0} = 342160N < N_{Ed,c} = 310400N \quad \therefore \text{ OK}$$

10.5 Link-Beam Design

Link beams are designed for the unfavorable loading case. The link beam between axis A1 and A2 is taken as the demonstration of the design methodologies.

10.5.1 Link-Beam Design According to TS 648



$N_d = 1{,}8$ t $M_d = 590$ tcm $V_d = 14$ t

Link Beam Section: IPN 300

h	t_w	t_f	b	r	W_{eIx}	W_{pIx}
30 cm	1,08 cm	1,62 cm	12,5 cm	0,65 cm	653 cm^3	762 cm^3

e = 60 cm (length of link beam)

$$e_{link,\min} = 1{,}0 * \frac{M_p}{V_p} \leq e_{link} \leq e_{link,\max} = 5{,}0 * \frac{M_p}{V_p} \text{ (Using Eq. 8.1)}$$

For $\frac{N_d}{\sigma_y * A} < 0{,}15$

$M_p = W_p * \sigma_y$ and $V_p = 0{,}6 * \sigma_y * A_k$ (Using Eq. 8.2 and Eq. 8.3) ($W_{pl,x} = W_p$)

$A_k = (h - 2 * t_f) * t_w$ (Utilizing Eq. 8.4)

$$\frac{N_d}{\sigma_y * A} = \frac{1{,}8}{2{,}4 * 69} = 0{,}01 < 0{,}15$$

$$M_p = W_p * \sigma_y = 762 * 2{,}4 = 1828{,}8tcm$$

$$A_k = (h - 2 * t_f) * t_w = (30 - 2 * 1{,}62) * 1{,}08 = 26{,}76cm^2$$

$$V_p = 0{,}6 * \sigma_y * A_k = 0{,}6 * 2{,}4 * 26{,}76 = 38{,}53t$$

$$e_{link,\min} = 1{,}0 * \frac{M_p}{V_p} \leq e_{link} \leq e_{link,\max} = 5{,}0 * \frac{M_p}{V_p} \text{ (Using Eq. 8.1)}$$

$$1{,}0 * \frac{M_p}{V_p} = 1{,}0 * \frac{1828{,}8}{38{,}53} = 47{,}46cm$$

$$5{,}0*\frac{M_p}{V_p}=5{,}0*\frac{1828{,}8}{38{,}53}=237{,}3cm$$

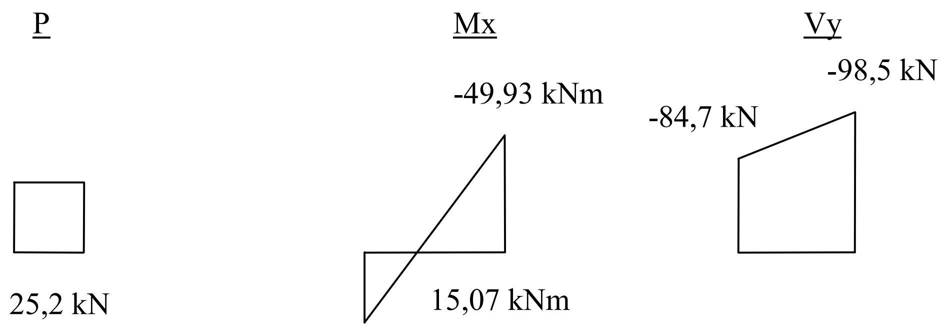
$$e_{link,\min}=47{,}46cm\le e_{link}=60cm\le e_{link,\max}=237{,}3cm \quad \therefore \text{ OK}$$

$V_d \le V_p$ 14t ≤ 38,53t $\therefore$ OK

$$V_d \le \frac{2*M_p}{e_{link}} \qquad V_d=14t\le\frac{2*M_p}{e_{link}}=\frac{2*1828{,}8}{60}=60{,}9t \qquad \therefore \text{ OK}$$

$M_d \le M_p$ 590tcm ≤ 1828,8 tcm $\therefore$ OK

10.5.2 Link-Beam Design According to LRFD



$P_{r,link}=P_u$ = 25,2 kN $M_{r,link}$ = 49,93 kNm $V_{r,link}$ = 98,5 kN

Link Beam Section: IPN 260

h	260 mm	**I_x**	57,4 x 10^6 mm^4
b	113 mm	**W_{elx}**	442 x 10^3 mm^3
t_w	9,4 mm	**W_{plx}**	514 x 10^3 mm^3
t_f	14,1 mm	**i_x**	104 mm
r	5,6 mm		
A	5330 mm^2		

Limiting Width-Thickness Ratios are same with beam (IPN 260). ∴ OK

For $P_u \leq 0{,}15 * P_y$

P_u = 25200 N

$P_y = A_g * F_y = 5330 * 235 = 1252550N \geq P_u = 25200N$ (Using Eq. 8.14)

∴ Not be considered axial force effect.

V_n (Nominal Shear Strength of the Link) must be lesser than V_p or $2*M_p/e$.

d = h and Z = $W_{pl,x}$

$A_w = (d - 2 * t_f) * t_w$ (Using Eq. 8.13)

A_w = (260 – 2 * 14,1) * 9,4 = 2178,9 mm^2

$M_p = F_y * Z$ = 235 * 514000 = 120790000 Nmm (Using Eq. 8.12)

$2*M_p/e$ = 2 *120790000 / 60 = 402633 N

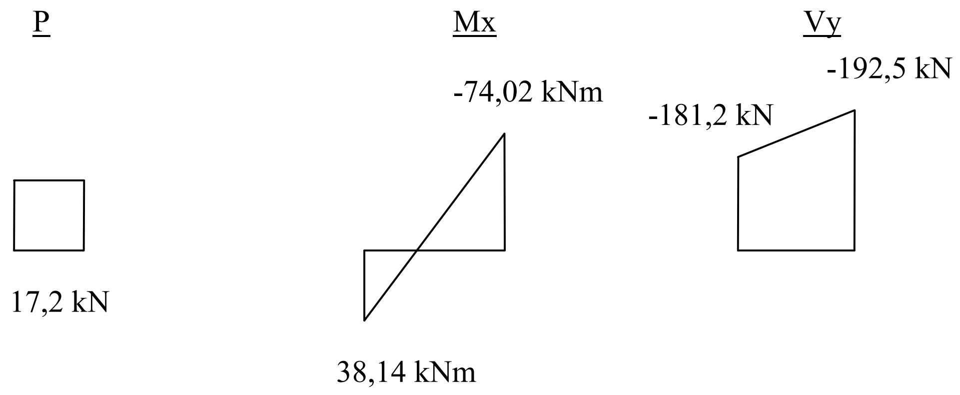
$V_p = 0{,}6 * F_y * A_w$ = 0,6 * 235 * 2178,9 = 307224,9 N (Using Eq. 8.11)

$V_n = \min (V_p ; 2*M_p/e)$ = min (307224,9 ; 402633) = 307224,9 N (Using Eq. 8.10)

$\phi_v * V_n = 0{,}9 * 307224{,}9 = 276502{,}4N > V_{r,Link} = 98500N$ ∴ OK (Using Eq. 8.9)

Φ * Mn = 0,9 *120790000 = 108711000 Nmm > $M_{r,link}$ = 49930000 Nmm ∴ OK

10.5.3 Link-Beam Design According to Eurocode 3



N_{Ed} = 17,2 kN = 17200 N
M_{Ed}= 74,02 kNm = 74020000 Nmm
V_{Ed} = 192,5 kN = 192500 N

Link Beam Section: IPN 260

h	260 mm	**I_x**	$57{,}4 \times 10^6$ mm^4
b	113 mm	**W_{elx}**	442×10^3 mm^3
t_w	9,4 mm	**W_{plx}**	514×10^3 mm^3
t_f	14,1 mm	**i_x**	104 mm
r	5,6 mm		
A	5330 mm^2		

$N_{pl,Rd} = N_{c,Rd} = \dfrac{A * f_y}{\gamma_{M0}}$ for Class 1 (Using Eq. 5.13)

$$N_{c,Rd} = \frac{A * f_y}{\gamma_{M0}} = \frac{5330 * 235}{1} = 1252550N$$

If $\dfrac{N_{Ed}}{N_{pl,Rd}} < 0{,}15$

$V_{Ed} \leq V_{p,Link}$ and $M_{Ed} \leq M_{p,Link}$ (Using Eq. 8.23 and Eq. 8.24)

$V_{p,Link} = (f_y / \sqrt{3}) * t_w * (h - t_f)$ (Utilizing Eq. 8.22)

$M_{p,Link} = f_y * b * t_f * (h - t_f)$ (Utilizing Eq. 8.21)

$$\frac{N_{Ed}}{N_{pl,Rd}} = \frac{17200}{1252550} = 0,013 < 0,15$$

$$V_{p,Link} = (235/\sqrt{3}) * 9,4 * (260 - 14,1) = 313612,7N$$

$$M_{p,Link} = 235 * 113 * 14,1 * (260 - 14,1) = 92071230,5Nmm$$

V_{Ed} = 192500 N $\leq$ $V_{p,Link}$ = 313612,7 N $\therefore$ OK

M_{Ed} = 74020000 Nmm $\leq$ $M_{p,Link}$ = 92071230,5 Nmm $\therefore$ OK

10.6 Story Drift Control

Drift control calculations for x-direction are shown below.

10.6.1 Story Drift Control According to TS 648

d_{0x} = 0 mm

d_{1x} = 2,33 mm (First storey drift)

d_{2x} = 3,76 mm (Second storey drift)

$\Delta_i = d_i - d_{i-1}$ (Using Eq. 9.16)

$\Delta_1 = d_1 - d_0$ = 2,33 – 0 = 2,33 mm

$\Delta_2 = d_2 - d_1$ = 3,76 – 2,33 = 1,43 mm

$\delta_i = R * \Delta_i$ (Utilizing Eq. 9.17)

R = 7 (Obtained from Table 9.6)

$\delta_1 = R * \Delta_1 = 7 * 2{,}33 = 16{,}31$ mm

$\delta_2 = R * \Delta_2 = 7 * 1{,}43 = 10{,}01$ mm

$$\frac{(\delta_i)_{max}}{h_i} \leq 0{,}02$$ (Utilizing Eq. 9.18)

$$\frac{(\delta_i)_{max}}{h_i} = \frac{16{,}31}{3500} = 0{,}0047 \leq 0{,}02 \;\therefore\; \text{OK}$$

$$\frac{(\delta_i)_{max}}{h_i} = \frac{10{,}01}{3000} = 0{,}0034 \leq 0{,}02 \;\therefore\; \text{OK}$$

10.6.2 Story Drift Control According to LRFD

$d_{0x} = 0$ mm

$d_{1x} = 1{,}57$ mm (First storey drift)

$d_{2x} = 2{,}52$ mm (Second storey drift)

$\delta_i = d_i - d_{i-1}$ (Using Eq. 9.38)

$\delta_1 = d_1 - d_0 = 1{,}57 - 0 = 1{,}57$ mm

$\delta_2 = d_2 - d_1 = 2{,}52 - 1{,}57 = 0{,}95$ mm

$$\Delta = \frac{C_d * \delta}{I}$$ (Using Eq. 9.37)

$C_d = 4$ (Obtained from Table 9.14)

I = 1 (Obtained from Table 9.13)

$\Delta_a = 0{,}020*h_{sx}$ (Obtained from Table 9.20)

$$\Delta_1 = \frac{C_d * \delta_1}{I} = \frac{4*1{,}57}{1} = 6{,}28mm \leq \Delta_a = 0{,}020*\text{h}_{\text{sx}} = 0{,}02*3500 = 70\text{mm} \therefore \text{ OK}$$

$$\Delta_2 = \frac{C_d * \delta_2}{I} = \frac{4*0{,}95}{1} = 3{,}8mm \leq \Delta_a = 0{,}020*\text{h}_{\text{sx}} = 0{,}02*3000 = 60\text{mm} \therefore \text{ OK}$$

10.6.3 Story Drift Control According to Eurocode 3

d_{0x} = 0 mm

d_{1x} = 3,22 mm (First storey drift)

d_{2x} = 5,19 mm (Second storey drift)

$d_{ie} = d_i - d_{i-1}$ (Using Eq. 9.53)

$d_{1e} = d_1 - d_0 = 3{,}22 - 0 = 3{,}22$ mm

$d_{2e} = d_2 - d_1 = 5{,}19 - 3{,}22 = 1{,}97$ mm

$d_{1r} = q * d_{1e} = 6 * 3{,}22 = 19{,}32$ mm (Utilizing Eq. 9.52)

$d_{2r} = q * d_{2e} = 6 * 1{,}97 = 11{,}82$ mm (Utilizing Eq. 9.52)

$$\theta = \frac{P_{tot} * d_r}{V_{tot} * h}$$ (Using Eq. 9.51)

$$\theta_1 = \frac{P_{1tot} * d_{1r}}{V_{1tot} * h_1} = \frac{6259{,}7*19{,}32}{1251{,}9*3500} = 0{,}027 \leq 0{,}3 \therefore \text{ OK}$$

$$\theta_2 = \frac{P_{2tot} * d_{2r}}{V_{2tot} * h_2} = \frac{792{,}3*11{,}82}{3013{,}4*3000} = 0{,}00104 \leq 0{,}3 \therefore \text{ OK}$$

CHAPTER 11

CONCLUSION

In this study design concept is as follows:

- All main beams in x direction are the same at all stories and replaced with the greatest section which carries the given load safely.
- All main beams in y direction are the same at all stories and replaced with the greatest section which carries the given load safely.
- All braces in x direction are the same and replaced with the greatest section which carries the given load safely.
- All braces in y direction are the same and replaced with the greatest section which carries the given load safely.
- Secondary beams are all the same in whole building.
- All beams are assumed to be buckling restrained beams.
- While performing analysis on 4 story building, columns connected to braces are found to take grater forces, so those members are assigned greater sections when compared to the rest. For this reason, columns with braces in x-direction are designed using the same section in whole building, same principle is applied for the columns with braces in y-direction also. Besides, columns without braces are also assigned same section other than the ones mentioned previously (Figure 11.1 and 11.2).
- As for buildings having 6, 8 and 10 stories, the building is divided into two in terms of number of stories and the each half is assigned different sections while

considering the principles mentioned above (for four stories) (Figure 11.1 and 11.2).

- Beams connected to the braces are designed according to the procedures applied for beam columns. (Figure 11.3 and 11.4).
- In design, all beams are of IPN section, braces are of Square Hollow Section (SHS) and columns are of HEB section for buildings having 2, 4 and 6 stories, HEM for 8 stories and as for 10 stories columns are of HEM section except that HD sections are assigned to columns with braces in y-direction for only first five stories.
- In this study, for three codes section checking based on the modified earthquake design forces (for capacity protection of columns and brace) emerged by application of the overstrength ratio is not considered.

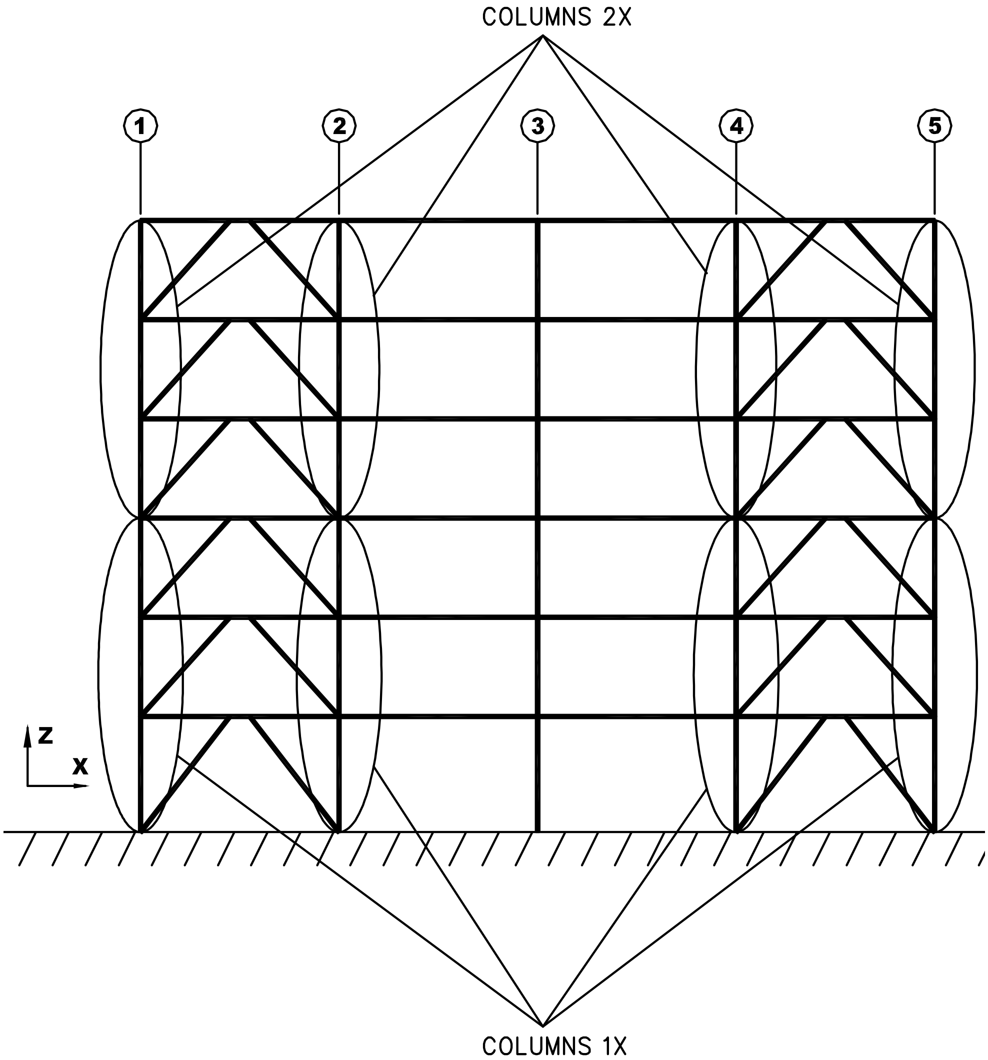


Figure 11.1 Columns 1X and 2X view

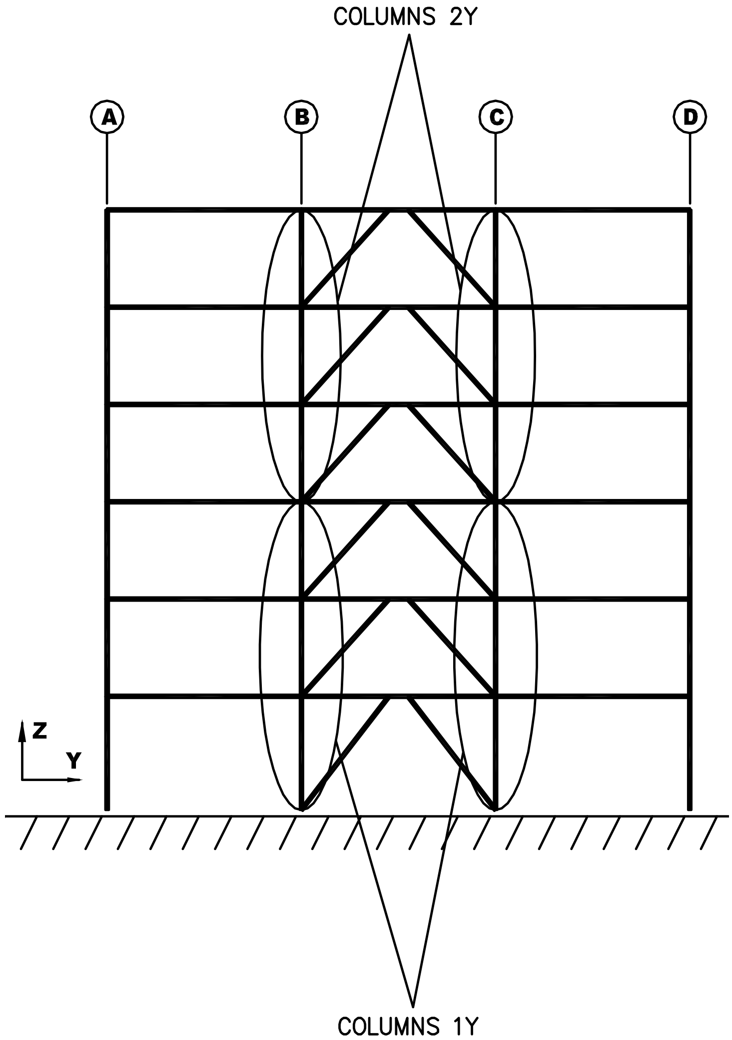


Figure 11.2 Columns 1Y and 2Y view

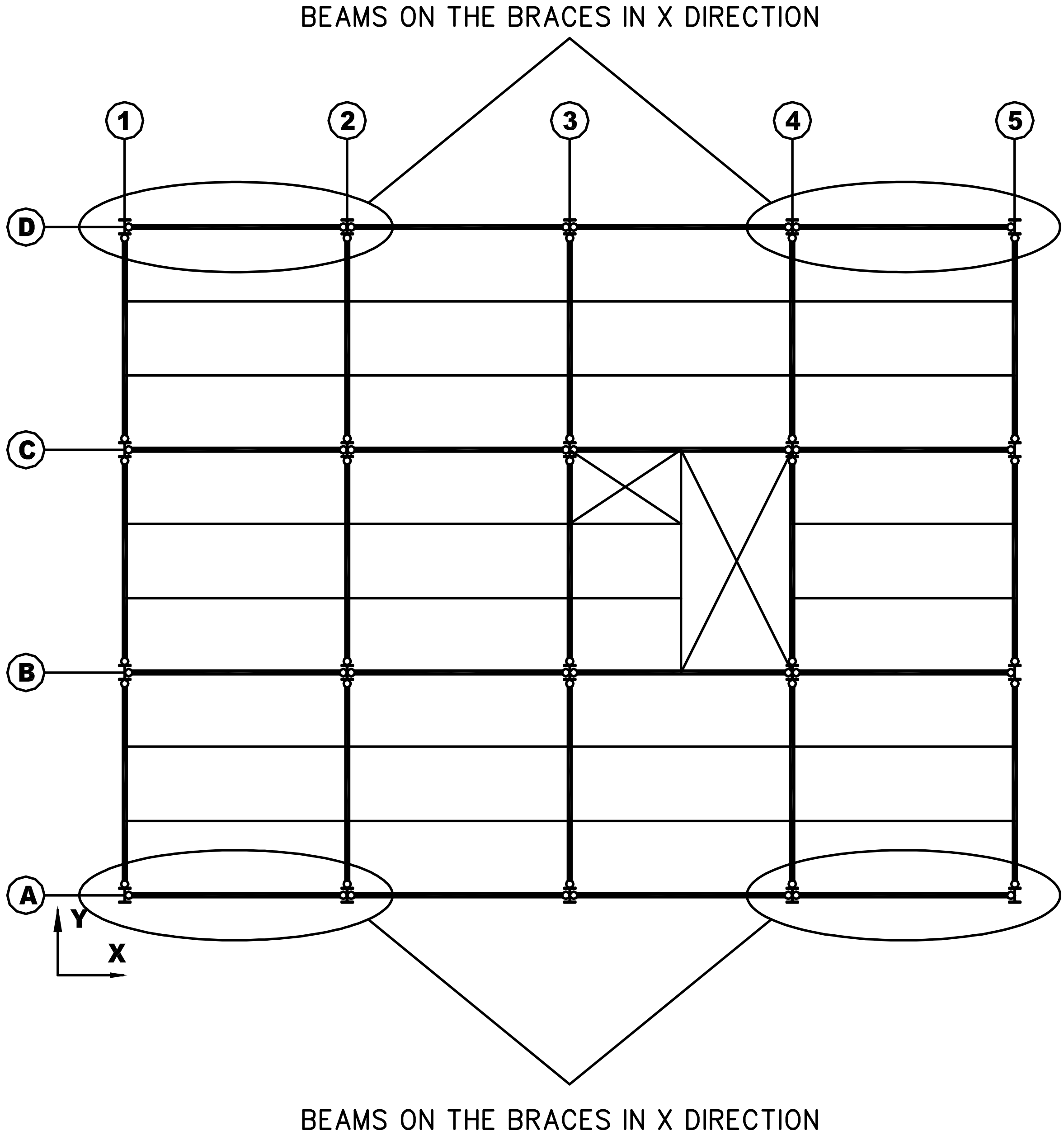


Figure 11.3 Beams connected to the braces in X direction view

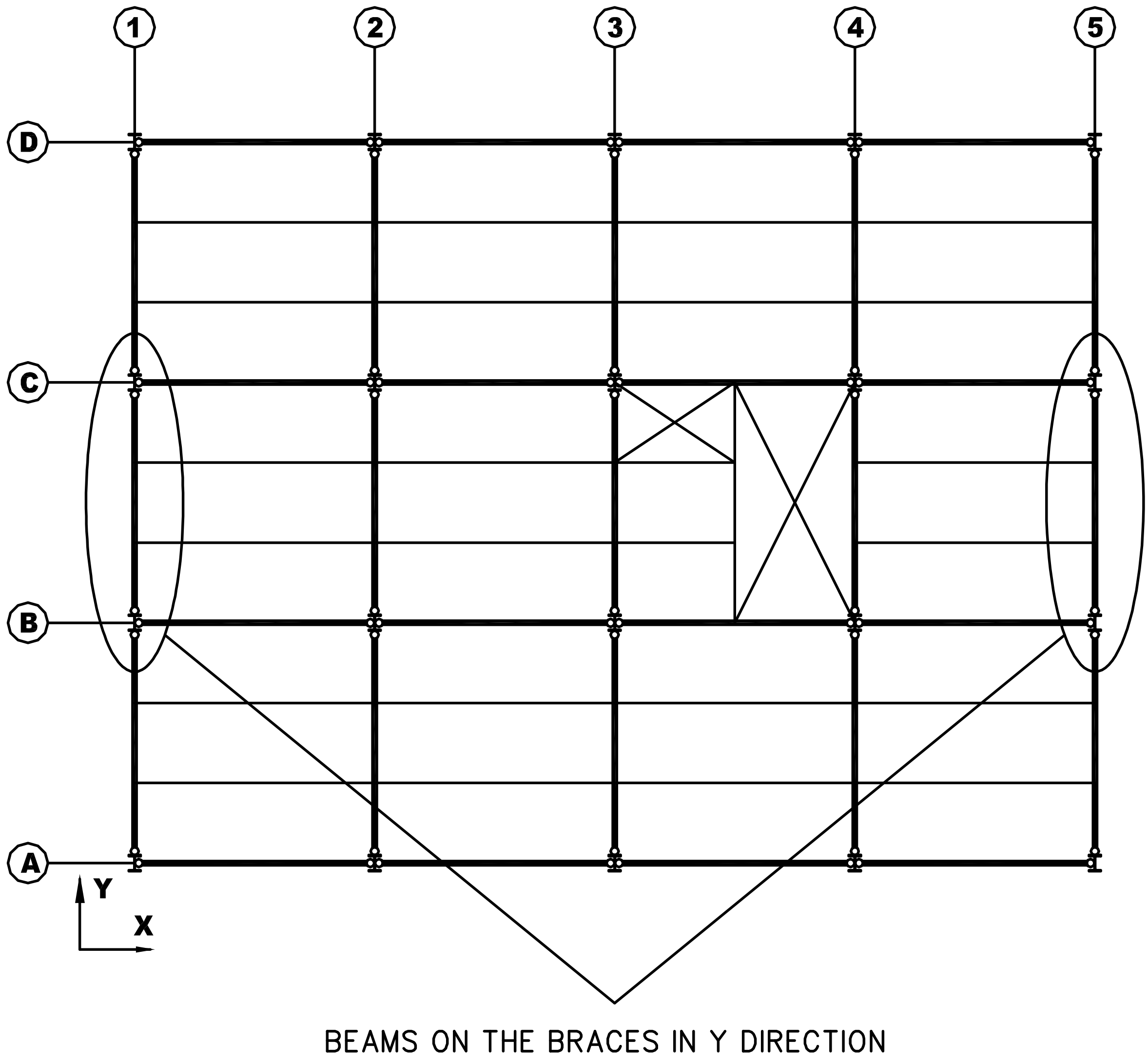


Figure 11.4 Beams connected to the braces in Y direction view

While performing analysis using ASCE-7-05 procedure, the ρ values (redundancy factor) for the specified load combinations are given below for each building in both x and y directions.

Table 11.1 ρ Values For LRFD

ρ values for LRFD	2 Storey Building	4 Storey Building	6 Storey Buildings	8 Storey Buildings	10 Storey Buildings
X direction	1,30	1,30	1,00	1,00	1,00
Y direction	1,30	1,30	1,30	1,30	1,30

Results and conclusions drawn from this study are tabulated below.

Table 11.2 Fundamental Periods of the Buildings

Fundamental Periods of The Buildings	2 Storey Building		4 Storey Building		6 Storey Buildings		8 Storey Buildings		10 Storey Buildings	
	T_x (sec)	T_y (sec)	T_x (sec)	T_y (sec)	T_x (sec)	T_y (sec)	T_x (sec)	T_y (sec)	T_x (sec)	T_y (sec)
TS 648	0,34	0,37	0,55	0,64	0,82	0,96	1,10	1,23	1,40	1,58
LRFD	0,40	0,43	0,64	0,70	0,97	1,04	1,31	1,38	1,60	1,71
Eurocode 3	0,39	0,43	0,67	0,76	1,00	1,12	1,29	1,39	1,56	1,69

Table 11.3 Value of Base Shear

Base Shear	TS 648		LRFD		Eurocode 3	
	V_{tx} (kN)	V_{ty} (kN)	V_x (kN)	V_y (kN)	F_{bx} (kN)	F_{by} (kN)
2 Storey Building	900,60	900,60	461,40	461,40	1239,40	1239,40
4 Storey Building	1418,51	1256,55	878,54	826,86	1585,29	1397,56
6 Storey Buildings	1550,84	1367,10	930,23	930,23	1595,98	1661,61
8 Storey Buildings	1651,01	1509,87	1005,03	1005,03	2019,73	2019,73
10 Storey Buildings	1711,47	1553,62	1076,90	1076,90	2540,78	2540,78

Table 11.4 Sections of Members for 2 Storey Buildings

Sections of Members	2 Storey Buildings		
	TS 648	LRFD	Eurocode 3
Columns	HE 160 B	HE 140 B	HE 160 B
Beams X	IPN 300	IPN 260	IPN 260
Beams Y	IPN 360	IPN 340	IPN 320
Braces X	Tube 120 x 120 x 6	Tube 100 x 100 x 5	Tube 120 x 120 x 5
Braces Y	Tube 160 x 160 x 8	Tube 120 x 120 x 6	Tube 140 x 140 x 6
Secondary Beams	IPN 260	IPN 240	IPN 240

Table 11.5 Sections of Members for 4 Storey Buildings

Sections of Members	4 Storey Buildings		
	TS 648	LRFD	Eurocode 3
Columns	HE 220 B	HE 200 B	HE 160 B
Columns X	HE 240 B	HE 180 B	HE 160 B
Columns Y	HE 260 B	HE 220 B	HE 160 B
Beams X	IPN 300	IPN 260	IPN 260
Beams Y	IPN 360	IPN 340	IPN 320
Braces X	Tube 140 x 140 x 8	Tube 120 x 120 x 6	Tube 140 x 140 x 5
Braces Y	Tube 160 x 160 x 10	Tube 140 x 140 x 8	Tube 160 x 160 x 6
Secondary Beams	IPN 260	IPN 240	IPN 240

Table 11.6 Sections of Members for 6 Storey Buildings

Sections of Members	6 Storey Buildings		
	TS 648	LRFD	Eurocode 3
Columns 1	HE 260 B	HE 240 B	HE 240 B
Columns 2	HE 200 B	HE 180 B	HE 180 B
Columns 1X	HE 300 B	HE 220 B	HE 240 B
Columns 1Y	HE 360 B	HE 300 B	HE 300 B
Columns 2X	HE 180 B	HE 140 B	HE 140 B
Columns 2Y	HE 180 B	HE 160 B	HE 160 B
Beams X	IPN 300	IPN 260	IPN 260
Beams Y	IPN 360	IPN 340	IPN 320
Braces X	Tube 140 x 140 x 8	Tube 120 x 120 x 6	Tube 140 x 140 x 5
Braces Y	Tube 160 x 160 x 10	Tube 140 x 140 x 8	Tube 160 x 160 x 6
Secondary Beams	IPN 260	IPN 240	IPN 240

Table 11.7 Sections of Members for 8 Storey Buildings

Sections of Members	8 Storey Buildings		
	TS 648	LRFD	Eurocode 3
Columns 1	HE 240 M	HE 220 M	HE 220 M
Columns 2	HE 160 M	HE 140 M	HE 160 M
Columns 1X	HE 260 M	HE 200 M	HE 240 M
Columns 1Y	HE 300 M	HE 260 M	HE 300 M
Columns 2X	HE 160 M	HE 120 M	HE 120 M
Columns 2Y	HE 180 M	HE 140 M	HE 140 M
Beams X	IPN 300	IPN 260	IPN 260
Beams Y	IPN 360	IPN 340	IPN 320
Braces X	Tube 140 x 140 x 8	Tube 120 x 120 x 6	Tube 140 x 140 x 5
Braces Y	Tube 180 x 180 x 10	Tube 160 x 160 x 8	Tube 180 x 180 x 6
Secondary Beams	IPN 260	IPN 240	IPN 240

Table 11.8 Sections of Members for 10 Storey Buildings

Sections of Members	10 Storey Buildings		
	TS 648	LRFD	Eurocode 3
Columns 1	HE 240 M	HE 240 M	HE 240 M
Columns 2	HE 200 M	HE 160 M	HE 180 M
Columns 1X	HE 300 M	HE 240 M	HE 260 M
Columns 1Y	HD 400 x 314	HD 400 x 216	HD 400 x 262
Columns 2X	HE 180 M	HE 140 M	HE 140 M
Columns 2Y	HE 200 M	HE 160 M	HE 180 M
Beams X	IPN 300	IPN 260	IPN 260
Beams Y	IPN 360	IPN 340	IPN 320
Braces X	Tube 140 x 140 x 8	Tube 120 x 120 x 6	Tube 140 x 140 x 6
Braces Y	Tube 180 x 180 x 10	Tube 160 x 160 x 8	Tube 180 x 180 x 8
Secondary Beams	IPN 260	IPN 240	IPN 240

Since yielding occurs at beams connected to the braces in only some stories of 8 and 10 storey buildings whose designs are based on Eurocode 3, following tables are given ;

Table 11.9 Sections of Beams Connected to the Braces for 6 Storey Building (Eurocode 3)

8 Storey Building (Eurocode 3)		
Beams connected to the braces in X direction	1. Storey	IPN 280
Beams connected to the braces in Y direction	1-2-3. Storey	IPN 340

Table 11.10 Sections of Beams Connected To the Braces for 10 Storey Building (Eurocode 3)

10 Storey Building (Eurocode 3)		
Beams connected to the braces in X direction	1-2. Storey	IPN 300
	3-4-5. Storey	IPN 280
Beams connected to the braces in Y direction	1-2-3. Storey	IPN 380
	4-5. Storey	IPN 360
	6. Storey	IPN 340

Story drift controls are conducted and the results are given below tables;

Table 11.11 Story Drift Control for 2 Storey Buildings

Story Drift Control	2 Storey Buildings					
Number of Storey	TS 648 (δi/hi, Ratios)		LRFD (Δi, mm)		Eurocode 3 (θi, Ratios)	
	X Direction	Y Direction	X Direction	Y Direction	X Direction	Y Direction
1. Storey	0,0068	0,0078	6,27	12,22	0,054	0,064
2. Storey	0,0045	0,0590	3,82	7,81	0,026	0,036

Table 11.12 Story Drift Control for 4 Storey Buildings

Story Drift Control	4 Storey Buildings					
Number of Storey	TS 648 (δi/hi, Ratios)		LRFD (Δi, mm)		Eurocode 3 (θi, Ratios)	
	X Direction	Y Direction	X Direction	Y Direction	X Direction	Y Direction
1. Storey	0,0079	0,0086	20,37	20,13	0,092	0,100
2. Storey	0,0081	0,0100	17,27	19,97	0,077	0,100
3. Storey	0,0070	0,0094	14,59	17,33	0,056	0,076
4. Storey	0,0051	0,0074	10,13	13,05	0,034	0,050

Table 11.13 Story Drift Control for 6 Storey Buildings

Story Drift Control	6 Storey Buildings					
Number of Storey	TS 648 (δi/hi, Ratios)		LRFD (Δi, mm)		Eurocode 3 (θi, Ratios)	
	X Direction	Y Direction	X Direction	Y Direction	X Direction	Y Direction
1. Storey	0,0083	0,0083	15,58	21,12	0,129	0,124
2. Storey	0,0097	0,0119	15,50	24,20	0,134	0,123
3. Storey	0,0097	0,0123	15,56	24,20	0,135	0,107
4. Storey	0,0092	0,0119	14,74	23,05	0,077	0,089
5. Storey	0,0086	0,0115	13,40	21,54	0,041	0,073
6. Storey	0,0070	0,0099	10,69	17,70	0,014	0,053

Table 11.14 Story Drift Control for 8 Storey Buildings

Story Drift Control	8 Storey Buildings					
Number of Storey	TS 648 (δi/hi, Ratios)		LRFD (Δi, mm)		Eurocode 3 (θi, Ratios)	
	X Direction	Y Direction	X Direction	Y Direction	X Direction	Y Direction
1. Storey	0,0085	0,0079	21,66	20,28	0,154	0,150
2. Storey	0,0105	0,0119	22,05	25,02	0,175	0,189
3. Storey	0,0108	0,0127	22,62	26,28	0,159	0,186
4. Storey	0,0112	0,0131	22,96	27,05	0,146	0,176
5. Storey	0,0108	0,0130	21,93	26,37	0,126	0,156
6. Storey	0,0106	0,0131	21,05	26,33	0,111	0,143
7. Storey	0,0097	0,0124	18,79	24,39	0,089	0,121
8. Storey	0,0083	0,0110	15,49	21,01	0,067	0,095

Table 11.15 Story Drift Control for 10 Storey Buildings

Story Drift Control	10 Storey Buildings					
Number of Storey	TS 648 (δi/hi, Ratios)		LRFD (Δi, mm)		Eurocode 3 (θi, Ratios)	
	X Direction	Y Direction	X Direction	Y Direction	X Direction	Y Direction
1. Storey	0,0083	0,0073	21,76	19,59	0,151	0,145
2. Storey	0,0109	0,0121	24,17	27,09	0,182	0,198
3. Storey	0,0114	0,0134	25,11	29,74	0,187	0,194
4. Storey	0,0121	0,0142	26,20	31,63	0,183	0,196
5. Storey	0,0122	0,0147	26,36	32,42	0,174	0,199
6. Storey	0,0121	0,0148	25,56	32,19	0,167	0,198
7. Storey	0,0123	0,0154	25,39	33,13	0,158	0,197
8. Storey	0,0119	0,0155	24,08	32,51	0,143	0,183
9. Storey	0,0111	0,0147	21,77	30,34	0,123	0,161
10. Storey	0,0099	0,0136	18,76	27,24	0,100	0,136

Weight of steel used on 1 m^2 for all buildings is introduced in below table;

Table 11.16 Weight of Steel Used on 1 m^2 (kg/m^2)

Weight of Steel Used on 1 m^2 (kg/m^2)			
Number of Stories	TS 648	LRFD	EC3
2	51,38	42,58	42,86
4	57,61	46,96	45,96
6	58,01	47,56	46,32
8	64,59	51,48	52,84
10	68,59	55,58	56,77

The below figure clearly shows the relationship between number of stories and the steel weight for per square meter.

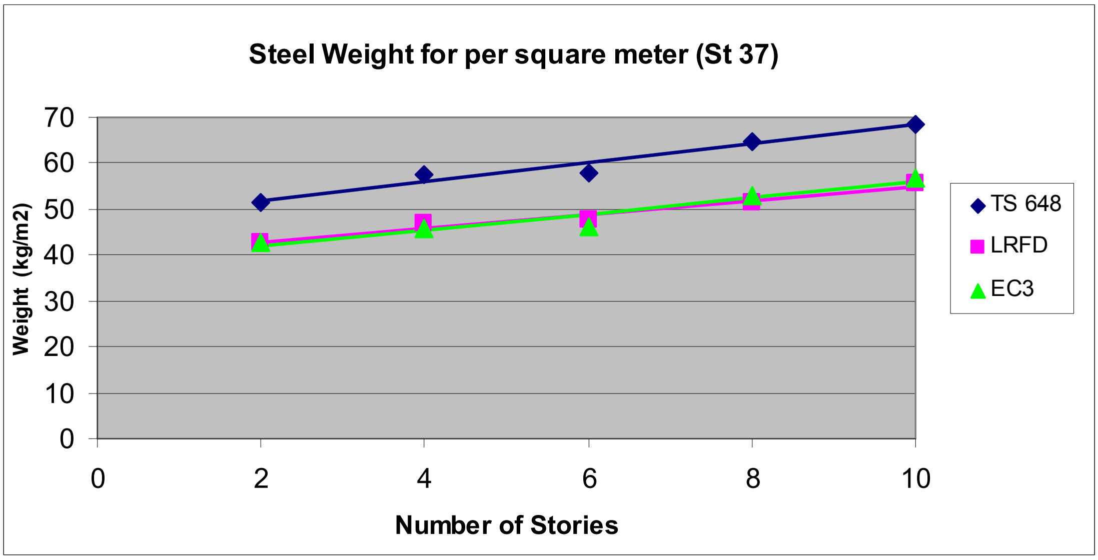


Figure 11.5 Weight of steel used on 1 m^2

The formulae of the linear lines in Figure 11.4 can be computed from the following equations;

2,07 * N + 47,62 for TS 648 (kg/m^2) (11.1)

1,53 * N + 39,68 for LRFD (kg/m^2) (11.2)

1,74 * N + 38,54 for Eurocode 3 (kg/m^2) (11.3)

where

N = Number of stories

For 2-8-10 story-buildings, in LRFD procedure steel used on 1 m² is found to be less when compared to EC3. The reason for 2 story-building is that the design principle that all columns should have the same section provides greater sections but not as great as the other procedures because of the load combinations (the combination including EQ loads are not critical because of height, and also the other combination including 1,2D+1,6L+0,5L_r means less axial load on columns in this building).

The main reason for having less steel in LRFD procedure for taller buildings (8-10 story buildings) is that earthquake loads become less critical when compared to the other two procedures.

Weight of steel used on 1 m² is almost the same for procedures of LRFD and EC3. It is important to note that those procedures consider 20 % of material saving compared to TS648.

Moreover, the result based on LRFD of this study is compared to the result of the rule of thumb which is introduced by Ioannides and Ruddy [20]. However units the approximation formula is not consistent with SI units, therefore Eq. 2.1 is converted as follows:

$$W_t(\text{kg/m}^2) = 1.627 * N + 34.18 \tag{11.1}$$

where N is the number of stories.

Furthermore, this formula is derived for St 52. Therefore, it must be calibrated to St 37 by making use of Harman's study [14]. The below figure clearly shows the comparison.

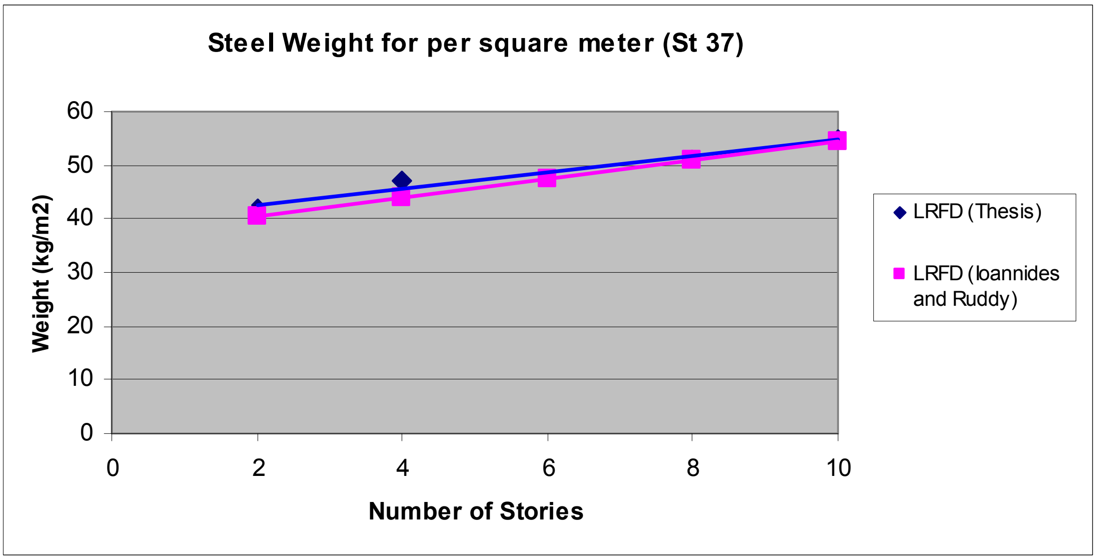


Figure 11.6 Comparison of the results

It is obvious that results of this study perfectly overlap with the results obtained from Eq 11.1. In other words, findings of this study are consistent with the article which is conducted by Ioannides and Ruddy [20].

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APPENDIX A

SECTIONAL PROPERTIES

Table A1 Sectional Properties of Tube Section

TUBE	kg/m	mm	mm	mm	mm	cm^2	cm^4	cm^3	cm^3	cm	cm^4
Section	G	b	t	ro(ext)	ri(int)	A	I	Wel	Wpl	i	I_t
TUBE 40x40x2	2,82	40	2,5	5	2,5	3,59	8,22	4,11	4,97	1,51	13,6
TUBE 40x40x3	3,3	40	3	6	3	4,21	9,32	4,66	5,72	1,49	15,8
TUBE 40x40x4	4,2	40	4	8	4	5,35	11,1	5,54	7,01	1,44	19,4
TUBE 50x50x3	4,25	50	3	6	3	5,41	19,5	7,79	9,39	1,9	32,1
TUBE 60x60x3	5,19	60	3	6	3	6,61	35,1	11,7	14	2,31	57,1
TUBE 50x50x4	5,45	50	4	8	4	6,95	23,7	9,49	11,7	1,85	40,4
TUBE 50x50x5	6,56	50	5	10	5	8,36	27	10,8	13,7	1,8	47,5
TUBE 60x60x4	6,71	60	4	8	4	8,55	43,6	14,5	17,6	2,26	72,6
TUBE 60x60x5	8,13	60	5	10	5	10,4	50,5	16,8	20,9	2,21	86,4
TUBE 80x80x3	8,37	80	3,6	7,2	3,6	10,7	102	25,5	30,2	3,09	165
TUBE 80x80x4	9,22	80	4	8	4	11,7	111	27,8	33,1	3,07	180
TUBE 80x80x5	11,3	80	5	10	5	14,4	131	32,9	39,7	3,03	218
TUBE 100x100x4	11,7	100	4	8	4	14,9	226	45,3	53,3	3,89	362
TUBE 80x80x6	13,5	80	6,3	15,75	9,45	17,2	149	37,1	46,1	2,94	261
TUBE 100x100x5	14,4	100	5	10	5	18,4	271	54,2	64,6	3,84	441
TUBE 100x100x6	17,5	100	6,3	15,75	9,45	22,2	314	62,8	76,4	3,76	536
TUBE 120x120x5	17,5	120	5	10	5	22,4	485	80,9	95,4	4,66	778
TUBE 140x140x5	20,7	140	5	10	5	26,4	791	113	132	5,48	1256
TUBE 100x100x8	21,4	100	8	20	12	27,2	366	73,2	91,1	3,67	645
TUBE 120x120x6	21,4	120	6,3	15,75	9,45	27,3	572	95,3	114	4,58	955
TUBE 140x140x6	25,4	140	6,3	15,75	9,45	32,3	941	134	160	5,39	1550
TUBE 100x100x10	25,6	100	10	25	15	32,6	411	82,2	105	3,55	750
TUBE 120x120x8	26,4	120	8	20	12	33,6	677	113	138	4,49	1163
TUBE 160x160x6	29,3	160	6,3	15,75	9,45	37,4	1442	180	213	6,21	2349
TUBE 140x140x8	31,4	140	8	20	12	40	1127	161	194	5,3	1901
TUBE 120x120x10	31,8	120	10	25	15	40,6	777	129	162	4,38	1376
TUBE 180x180x6	33,3	180	6,3	15,75	9,45	42,4	2096	233	273	7,03	3383
TUBE 160x160x8	36,5	160	8	20	12	46,4	1741	218	260	6,12	2897
TUBE 200x200x6	37,2	200	6,3	15,75	9,45	47,4	2922	292	341	7,85	4682
TUBE 180x180x8	41,5	180	8	20	12	52,8	2546	283	336	6,94	4189
TUBE 160x160x10	44,4	160	10	25	15	56,6	2048	256	311	6,02	3490
TUBE 200x200x8	46,5	200	8	20	12	59,2	3566	357	421	7,76	5815
TUBE 250x250x6	47,1	250	6,3	15,75	9,45	60	5873	470	544	9,89	9290
TUBE 180x180x10	50,7	180	10	25	15	64,6	3017	335	404	6,84	5074
TUBE 160x160x12	52,6	160	12,5	37,5	25	67	2275	284	356	5,83	4114
TUBE 200x200x10	57	200	10	25	15	72,6	4251	425	508	7,65	7072
TUBE 250x250x8	59,1	250	8	20	12	75,2	7229	578	676	9,8	11598
TUBE 200x200x12	68,3	200	12,5	37,5	25	87	4859	486	594	7,47	8502
TUBE 250x250x10	72,7	250	10	25	15	92,6	8707	697	822	9,7	14197
TUBE 250x250x12	84,8	250	12	36	24	108	9859	789	944	9,55	16691
TUBE 300x300x10	88,4	300	10	25	15	113	15519	1035	1211	11,7	24966
TUBE 350x350x10	104	350	10	25	15	133	25189	1439	1675	13,8	40127
TUBE 300x300x12	108	300	12,5	37,5	25	137	18348	1223	1451	11,6	30601
TUBE 250x250x16	109	250	16	48	32	139	12047	964	1180	9,32	21146
TUBE 350x350x12	127	350	12,5	37,5	25	162	30045	1717	2020	13,6	49393
TUBE 300x300x16	134	300	16	48	32	171	22076	1472	1774	11,4	37837
TUBE 350x350x16	159	350	16	48	32	203	36511	2086	2488	13,4	61481

Table A2 Sectional Properties of HEB Section

								Strong Axis				Weak Axis					
HEB	kg/m	mm	mm	mm	mm	mm	cm^2	cm^4	cm^3	cm^3	cm	cm^4	cm^3	cm^3	cm	cm^4	cm^6*10^{-3}
Section	G	h	b	tw	tf	r	A	Ix	Wel.x	Wpl.x	ix	Iy	Wel.y	Wpl.y	iy	I_t	I_w
HE 100 B	20,4	100	100	6,0	10,0	12	26,04	449,5	89,91	104,20	4,16	167,3	33,45	51,42	2,53	9,25	3,38
HE 120 B	26,7	120	120	6,5	11,0	12	34,01	864,4	144,1	165,2	5,04	317,5	52,92	80,97	3,06	13,84	9,41
HE 140 B	33,7	140	140	7,0	12,0	12	42,96	1509,0	215,6	245,4	5,93	549,7	78,52	119,80	3,58	20,06	22,48
HE 160 B	42,6	160	160	8,0	13,0	15	54,25	2492,0	311,5	354,0	6,78	889,2	111,2	170,0	4,05	31,24	47,94
HE 180 B	51,2	180	180	8,5	14,0	15	65,25	3831,0	425,7	481,4	7,66	1363,0	151,4	231,0	4,57	42,16	93,75
HE 200 B	61,3	200	200	9,0	15,0	18	78,08	5696,0	569,6	642,5	8,54	2003,0	200,3	305,8	5,07	59,28	171,1
HE 220 B	71,5	220	220	9,5	16,0	18	91,04	8091,0	735,5	827,0	9,43	2843,0	258,5	393,9	5,59	76,57	295,4
HE 240 B	83,2	240	240	10,0	17,0	21	106,00	11260,0	938,3	1053,0	10,31	3923	326,9	498,4	6,08	102,70	486,9
HE 260 B	93,0	260	260	10,0	17,5	24	118,40	14920,0	1148,0	1283,0	11,22	5135	395,0	602,2	6,58	123,80	753,7
HE 280 B	103	280	280	10,5	18,0	24	131,40	19270,0	1376,0	1534	12,11	6595	471,0	717,6	7,09	143,70	1130,0
HE 300 B	117,0	300	300	11,0	19,0	27	149,10	25170,0	1678,0	1869,0	12,99	8563	570,9	870,1	7,58	185,00	1688
HE 320 B	127,0	320	300	11,5	20,5	27	161,30	30820,0	1926	2149	13,82	9239	615,9	939,1	7,57	225,10	2069
HE 340 B	134	340	300	12,0	21,5	27	170,90	36660,0	2156	2408	14,65	9690	646,0	985,7	7,53	257,20	2454
HE 360 B	142	360	300	12,5	22,5	27	180,60	43190,0	2400	2683	15,46	10140	676,1	1032,0	7,49	292,50	2883
HE 400 B	155,0	400	300	13,5	24,0	27	197,80	57680,0	2884	3232	17,08	10820	721,3	1104,0	7,40	355,70	3817
HE 450 B	171	450	300	14,0	26,0	27	218,00	79890,0	3551	3982	19,14	11720	781,4	1198,0	7,33	440,50	5258
HE 500 B	187	500	300	14,5	28,0	27	238,60	107200,0	4287	4815	21,19	12620	841,6	1292,0	7,27	538,40	7018
HE 550 B	199	550	300	15,0	29,0	27	254,10	136700,0	4971	5591	23,20	13080	871,8	1341,0	7,17	600,30	8856
HE 600 B	212	600	300	15,5	30,0	27	270,00	171000,0	5701	6425	25,17	13530	902,0	1391,0	7,08	667,20	10970
HE 650 B	225	650	300	16,0	31,0	27	286,30	210600,0	6480	7320	27,12	13980	932,3	1441,0	6,99	739,20	13360
HE 700 B	241	700	300	17,0	32,0	27	306,40	256900,0	7340	8327	28,96	14440	962,7	1495,0	6,87	830,90	16060
HE 800 B	262	800	300	17,5	33,0	30	334,20	359100,0	8977	10230	32,78	14900	993,6	1553,0	6,68	946,00	21840
HE 900 B	291	900	300	18,5	35,0	30	371,30	494100,0	10980	12580	36,48	15820	1054,0	1658,0	6,53	1137,00	29460
HE 1000 B	314	1000	300	19,0	36,0	30	400,00	644700,0	12890	14860	40,15	16280	1085,0	1716,0	6,38	1254,00	37640

Table A3 Sectional Properties of HD Section

								Strong Axis				Weak Axis					
HD	kg/m	mm	mm	mm	mm	mm	cm^2	cm^4	cm^3	cm^3	cm	cm^4	cm^3	cm^3	cm	cm^4	cm^6*10^{-3}
Section	G	h	b	tw	tf	r	A	Ix	Wel.x	Wpl.x	ix	Iy	Wel.y	Wpl.y	iy	I_t	I_w
HD 260 x 93.0	93,0	260	260	10,0	17,5	24	118,4	14920	1148	1283	11,22	5135	395,0	602,2	6,58	123,8	753,7
HD 260 x 114	114	268	262	12,5	21,5	24	145,7	18910	1411	1600	11,39	6456	492,8	752,5	6,66	222,4	979
HD 320 x 127	127	320	300	11,5	20,5	27	161,3	30820	1926	2149	13,82	9239	615,9	939,1	7,57	225,1	2069
HD 260 x 142	142	278	265	15,5	26,5	24	180,3	24330	1750	2015	11,62	8236	621,6	950,5	6,76	406,8	1300
HD 320 x 158	158	330	303	14,5	25,5	27	201,2	39640	2403	2718	14,04	11840	781,7	1194	7,67	420,5	2741
HD 360 x 162	162	364	371	13,3	21,8	15	206,3	51540	2832	3139	15,81	18560	1001	1516	9,49	295,5	5432
HD 260 x 172	172	290	268	18,0	32,5	24	219,6	31310	2159	2524	11,94	10450	779,7	1192	6,90	719	1728
HD 360 x 179	179	368	373	15,0	23,9	15	228,3	57440	3122	3482	15,86	20680	1109	1683	9,52	393,8	6119
HD 400 x 187	187	368	391	15,0	24,0	15	237,6	60180	3271	3642	15,91	23920	1224	1855	10,03	414,6	7074
HD 360 x 196	196	372	374	16,4	26,2	15	250,3	63630	3421	3837	15,94	22860	1222	1856	9,56	517,1	6829
HD 320 x 198	198	343	306	18,0	32,0	27	252,3	51900	3026	3479	14,34	15310	1001	1530	7,79	805,3	3695
HD 400 x 216	216	375	394	17,3	27,7	15	275,5	71140	3794	4262	16,07	28250	1434	2176	10,13	637,3	8515
HD 400 x 237	237	380	395	18,9	30,2	15	300,9	78780	4146	4686	16,18	31040	1572	2387	10,16	825,5	9489
HD 320 x 245	245	359	309	21,0	40,0	27	312,0	68130	3796	4435	14,78	19710	1276	1951	7,95	1501	5004
HD 400 x 262	262	387	398	21,1	33,3	15	334,6	89410	4620	5260	16,35	35020	1760	2676	10,23	1116	10940
HD 400 x 287	287	393	399	22,6	36,6	15	366,3	99710	5074	5813	16,50	38780	1944	2957	10,29	1464	12300
HD 320 x 300	300	375	313	27,0	48,0	27	382,1	86900	4635	5522	15,08	24600	1572	2414	8,02	2650	6558
HD 400 x 314	314	399	401	24,9	39,6	15	399,2	110200	5525	6374	16,62	42600	2125	3236	10,33	1870	13740
HD 400 x 347	347	407	404	27,2	43,7	15	442,0	124900	6140	7139	16,81	48090	2380	3629	10,43	2510	15850
HD 400 x 382	382	416	406	29,8	48,0	15	487,1	141300	6794	7965	17,03	53620	2641	4031	10,49	3326	18130
HD 400 x 421	421	425	409	32,8	52,6	15	537,1	159600	7510	8880	17,24	60080	2938	4489	10,58	4398	20800
HD 400 x 463	463	435	412	35,8	57,4	15	589,5	180200	8283	9878	17,48	67040	3254	4978	10,66	5735	23850
HD 400 x 509	509	446	416	39,1	62,7	15	649,0	204500	9172	11030	17,75	75400	3625	5552	10,78	7513	27630
HD 400 x 551	551	455	418	42,0	67,6	15	701,4	226100	9939	12050	17,95	82490	3947	6051	10,85	9410	30870
HD 400 x 592	592	465	421	45,0	72,3	15	754,9	250200	10760	13140	18,20	90170	4284	6574	10,93	11560	34670
HD 400 x 634	634	474	424	47,6	77,1	15	808,0	274200	11570	14220	18,42	98250	4634	7117	11,03	14020	38570
HD 400 x 677	677	483	428	51,2	81,5	15	863,4	299500	12400	15350	18,62	106900	4994	7680	11,13	16790	42920
HD 400 x 744	744	498	432	55,6	88,9	15	948,1	342100	13740	17170	19,00	119900	5552	8549	11,25	21840	49980
HD 400 x 818	818	514	437	60,5	97,0	15	1043	392200	15260	19260	19,39	135500	6203	9561	11,40	28510	58650
HD 400 x 900	900	531	442	65,9	106	15	1149	450200	16960	21620	19,79	153300	6938	10710	11,55	37350	68890
HD 400 x 990	990	550	448	71,9	115	15	1262	518900	18870	24280	20,27	173400	7739	11960	11,72	48210	81530
HD 400 x 1086	1086	569	454	78,0	125	15	1386	595700	20940	27210	20,73	196200	8645	13380	11,90	62290	96080

Table A4 Sectional Properties of HEM Section

								Strong Axis				Weak Axis					
HEM	kg/m	mm	mm	mm	mm	mm	cm^2	cm^4	cm^3	cm^3	cm	cm^4	cm^3	cm^3	cm	cm^4	cm^6*10^{-3}
Section	G	h	b	tw	tf	r	A	Ix	Wel.x	Wpl.x	ix	Iy	Wel.y	Wpl.y	iy	I_t	I_w
HE 100 M	41,8	120	106	12,0	20,0	12	53,24	1143,0	190,4	235,80	4,63	399,2	75,31	116,30	2,74	68,21	9,93
HE 120 M	52,1	140	126	12,5	21,0	12	66,41	2018,0	288,2	350,6	5,51	702,8	111,60	171,60	3,25	91,66	24,79
HE 140 M	63,2	160	146	13,0	22,0	12	80,56	3291,0	411,4	493,8	6,39	1144,0	156,80	240,50	3,77	120,00	54,33
HE 160 M	76,2	180	166	14,0	23,0	15	97,05	5098,0	566,5	674,6	7,25	1759,0	211,9	325,5	4,26	162,40	108,10
HE 180 M	88,9	200	186	14,5	24,0	15	113,30	7483,0	748,3	883,4	8,13	2580	277,4	425,2	4,77	203,30	199,30
HE 200 M	103,0	220	206	15,0	25,0	18	131,30	10640,0	967,4	1135,0	9,00	3651,0	354,5	543,2	5,27	259,40	346,3
HE 220 M	117,0	240	226	15,5	26,0	18	149,40	14600,0	1217,0	1419,0	9,89	5012,0	443,5	678,6	5,79	315,30	572,7
HE 240 M	157,0	270	248	18,0	32,0	21	199,60	24290,0	1799,0	2117,0	11,03	8153	657,5	1006,0	6,39	627,90	1152,0
HE 260 M	172	290	268	18,0	32,5	24	219,60	31310,0	2159,0	2524,0	11,94	10450	779,7	1192,0	6,90	719,00	1728,0
HE 280 M	189,0	310	288	18,5	33,0	24	240,20	39550,0	2551,0	2966,0	12,83	13160	914,1	1397,0	7,40	807,30	2520,0
HE 300 M	238,0	340	310	21,0	39,0	27	303,10	59200,0	3482	4078	13,98	19400	1252,0	1913,0	8,00	1408,00	4386
HE 320 M	245	359	309	21,0	40,0	27	312,00	68130,0	3796	4435	14,78	19710	1276,0	1951,0	7,95	1501,00	5004
HE 340 M	248	377	309	21,0	40,0	27	315,80	76370,0	4052	4718	15,55	19710	1276,0	1953,0	7,90	1506,00	5584
HE 360 M	250,0	395	308	21,0	40,0	27	318,80	84870,0	4297	4989	16,32	19520	1268,0	1942,0	7,83	1507,00	6137
HE 400 M	256	432	307	21,0	40,0	27	325,80	104100,0	4820	5571	17,88	19340	1260,0	1934,0	7,70	1515,00	7410
HE 450 M	263	478	307	21,0	40,0	27	335,40	131500,0	5501	6331	19,80	19340	1260,0	1939,0	7,59	1529,00	9251
HE 500 M	270	524	306	21,0	40,0	27	344,30	161900,0	6180	7094	21,69	19150	1252,0	1932,0	7,46	1539,00	11190
HE 550 M	278	572	306	21,0	40,0	27	354,40	198000,0	6923	7933	23,64	19160	1252,0	1937,0	7,35	1554,00	13520
HE 600 M	285	620	305	21,0	40,0	27	363,70	237400,0	7660	8772	25,55	18980	1244,0	1930,0	7,22	1564,00	15910
HE 650 M	293	668	305	21,0	40,0	27	373,70	281700,0	8433	9657	27,45	18980	1245,0	1936,0	7,13	1579,00	18650
HE 700 M	301	716	304	21,0	40,0	27	383,00	329300,0	9198	10540	29,32	18800	1237,0	1929,0	7,01	1589,00	21400
HE 800 M	317	814	303	21,0	40,0	30	404,30	442600,0	10870	12490	33,09	18630	1230,0	1930,0	6,79	1646,00	27780
HE 900 M	333	910	302	21,0	40,0	30	423,60	570400,0	12540	14440	36,70	18450	1222,0	1929,0	6,60	1671,00	34750
HE 1000 M	349	1008	302	21,0	40,0	30	444,20	722300,0	14330	16570	40,32	18460	1222,0	1940,0	6,45	1701,00	43020

Table A5 Sectional Properties of IPN Section

								Strong Axis				Weak Axis					
IPN	kg/m	mm	mm	mm	mm	mm	cm^2	cm^4	cm^3	cm^3	cm	cm^4	cm^3	cm^3	cm	cm^4	$cm^6 * 10^{-3}$
Section	G	h	b	tw	tf	r	A	Ix	Wel.x	Wpl.x	ix	Iy	Wel.y	Wpl.y	iy	I_t	I_w
IPN 120	11,1	120	58	5,1	7,7	3,1	14,2	328	54,7	63,6	4,81	21,5	7,41	12,4	1,23	2,71	0,69
IPN 140	14,3	140	66	5,7	8,6	3,4	18,3	573	81,9	95,4	5,61	35,2	10,7	17,9	1,40	4,32	1,54
IPN 160	17,9	160	74	6,3	9,5	3,8	22,8	935	117	136	6,4	54,7	14,8	24,9	1,55	6,57	3,14
IPN 180	21,9	180	82	6,9	10,4	4,1	27,9	1450	161	187	7,2	81,3	19,8	33,2	1,71	9,58	5,92
IPN 200	26,2	200	90	7,5	11,3	4,5	33,4	2140	214	250	8,0	117	26,0	43,5	1,87	13,5	10,5
IPN 220	31,1	220	98	8,1	12,2	4,9	39,5	3060	278	324	8,8	162	33,1	55,7	2,02	18,6	17,8
IPN 240	36,2	240	106	8,7	13,1	5,2	46,1	4250	354	412	9,59	221	41,7	70,0	2,20	25,0	28,7
IPN 260	41,9	260	113	9,4	14,1	5,6	53,3	5740	442	514	10,4	288	51,0	85,9	2,32	33,5	44,1
IPN 280	47,9	280	119	10,1	15,2	6,1	61,0	7590	542	632	11,1	364	61,2	103	2,45	44,2	64,6
IPN 300	54,2	300	125	10,8	16,2	6,5	69,0	9800	653	762	11,9	451	72,2	121	2,56	56,8	91,8
IPN 320	61,0	320	131	11,5	17,3	6,9	77,7	12510	782	914	12,7	555	84,7	143	2,67	72,5	129
IPN 340	68,0	340	137	12,2	18,3	7,3	86,7	15700	923	1080	13,5	674	98,4	166	2,80	90,4	176
IPN 360	76,1	360	143	13	19,5	7,8	97,0	19610	1090	1276	14,2	818	114,0	194	2,90	115	240
IPN 380	84,0	380	149	13,7	20,5	8,2	107	24010	1260	1482	15,0	975	131,0	221	3,02	141	319
IPN 400	92,4	400	155	14,4	21,6	8,6	118	29210	1460	1714	15,7	1160	149	253	3,13	170	420
IPN 450	115	450	170	16,2	24,3	9,7	147	45850	2040	2400	17,7	1730	203	345	3,43	267	791
IPN 500	141	500	185	18	27	10,8	179	68740	2750	3240	19,6	2480	268	456	3,72	402	1400
IPN 550	166	550	200	19	30	11,9	212	99180	3610	4240	21,6	3490	349	592	4,02	544	2390