COMPARISON AND EVALUATION OF THE CURRENT BUILDING CODES FOR EARTHQUAKE RESISTANT STEEL STRUCTURES

by

Mustafa Kürşat BAŞARAN

B.S., in C.E., Boğaziçi University, 2009

Submitted to the Institute for Graduate Studies in
Science and Engineering in partial fulfillment of
the requirements for the degree of
Master of Science

Graduate Program in Civil Engineering

Boğaziçi University

2009

ACKNOWLEDGEMENTS

I would like to express my deep gratitude to my thesis advisor Prof. Dr. Gülay Altay for her guidance, help and support in the preparation of this thesis.

I would also like to thank my dear friend; Alper Fırat Delice; and my colleagues; Nurten Ateş, Rami Akyüz; for their continuous supports and generous helps.

Finally, I would like to express to thank my family who has never deprived me from their support and encouragement during my academic life and I am grateful to Sevil Kaytan who adds meaning and value to my life for her encouragement in the preparation of this thesis.

ABSTRACT

COMPARISON AND EVALUATION OF THE CURRENT BUILDING CODES FOR EARTHQUAKE RESISTANT STEEL STRUCTURES

In the last years, earthquake design of structures becomes an important phenomenon due to the disastrous earthquakes. Accurate modeling of the seismic action is really important to observe the real behaviour of the structure under earthquake forces. "Specification for Buildings to be Built in Seismic Zone" is used for the earthquake resistant design of structures which is published at 2007 by The Ministry of Public Works and Settlement. The code used in Turkey is compared theoretically to Eurocode 8 which is published at 2003 by European Committee for Standardization in this study.

The national building codes used in Turkey for design of steel structures are described to be insufficient by academicians and structural engineers. Eurocode 3 will be a good alternative for the design of steel structures in Turkey in the entry process to European Union. The differences and similarities of Eurocode 3 and TSE 648 are presented in this thesis.

During the theoretical compression of the codes, the ground conditions, the definition of seismic action, the design conditions for earthquake resistant structures, calculation methods for the seismic action and the main design rules of the steel structures are mentioned by comparison of the codes in the thesis.

This study ends with a comparative example using the "Specification for Buildings to be Built in Seismic Zone" for the calculation of seismic action and Eurocode 3 and TSE 648 for the design of steel structure.

ÖZET

DEPREME DAYANIKLI ÇELİK YAPILAR İÇİN MEVCUT YÖNETMELİKLERİN KARŞILAŞTIRILMALI DEĞERLENDİRİLMESİ

Son yıllarda yıkıcı depremlerden dolayı depreme dayanıklı yapı tasarımı önemli bir olgu olmuştur. Deprem hareketinin doğru modellenmesi yapının deprem yükleri altında gerçek davranışının incelenmesi açısından önemlidir. Depreme dayanaklı yapı tasarımı için 2007 yılında Bayındırlık ve İskân Bakanlığı tarafından yayınlanan Deprem Bölgelerinde Yapılacak Binalar Hakkında Yönetmelik kullanılmaktadır. Bu çalışmada Türkiye ‘de kullanılan bu yönetmelik CEN tarafından 2003 yılında yayınlanan Eurocode 8 ile teorik olarak karşılaştırılmaktadır.

Türkiye ‘de çelik yapıların boyutlandırılmasında yararlanılan şartnameler akademisyenler ve proje mühendisleri tarafından yetersiz olarak nitelendirilmektedir. Avrupa Biriliği ‘ne giriş sürecinde, Eurocode 3 Türkiye ‘ de çelik yapıların tasarımı için güzel bir alternatif olacaktır. Bu tezde Eurocode 3 ile TSE 648 arasındaki farklılıklar ve benzerlikler sunulmuştur.

Bu tezde teorik olarak şartnamelerin karşılaştırılması yapılırken, zemin koşulları, deprem hareketinin tanımı, depreme dayanıklı tasarım kriterleri, yapının deprem analizi için kullanılacak hesap yöntemleri ve çelik yapıların temel tasarım kurallarından bahsedilmiştir.

Bu çalışma deprem hareketinin hesabı için Deprem Bölgelerinde Yapılacak Binalar Hakkında Yönetmelik ve çelik yapının boyutlandırılması için Eurocode 3 ve TSE 648 kullanılarak yapılan karşılaştırılmalı örnek ile tamamlanmıştır.

TABLE OF CONTENTS

ACKNOWLEDGEMENTS iii
ABSTRACT iv
ÖZET v
LIST OF FIGURES xi
LIST OF TABLES xivv
LIST OF SYMBOLS/ABBREVIATIONS xvii
1. INTRODUCTION 1
2. GENERAL RULES OF EARTHQUAKE DESIGN 2
2.1. General Rules 2
2.1.1. General Rules of EC8 2
2.1.2. General Rules of TEC2007 3
2.2. Site Conditions 3
2.2.1. Site Conditions According to EC8 3
2.2.2. Site Conditions According to TEC2007 4
2.3. Seismic Design 6
2.3.1. Definition of Seismic Action According to EC8 and TEC2007 6
2.3.2. Definition of Load Combinations According to EC8 and TEC2007 15
3. GENERAL RULES FOR BUILDING STRUCTURAL SYSTEM 18
3.1. Criteria for Structural Regularity According to EC8 18
3.1.1. Criteria for Regularity in Plan 19
3.1.2. Criteria for Regularity in Elevation 19
3.2. Criteria for Structural Regularity According to TEC2007 21
3.2.1. Criteria for Regularity in Plan 21
3.2.2. Criteria for Regularity in Elevation 22
3.2.3. Conditions for Irregular Buildings 23
3.3. Selection of Analysis Method 24
3.3.1. Structural Modeling & Analysis According to EC8 24
3.3.2. Structural Modeling & Analysis According to TEC2007 32
3.3.3. Serviceability Conditions 40
3.3.3.1 Limitation of Displacements 40

3.3.3.2 Second Order Effects 42
3.3.3.3 Seismic Joints 43
4. SPECIFIC RULES AND GUIDELINES FOR STEEL BUILDINGS 45
4.1. Rules According to EC8 45
4.1.1. Design Concept 45
4.1.2. Material Conditions and Material Safety Factors 45
4.1.3. Classification of Steel Structural Systems 46
4.1.4. Design Criteria & Detailing Rules for Dissipative Structural Behaviour Common to All Structural Types 51
4.1.5. Design and Detailing Rules for Moment resisting Frames 53
4.1.6. Design and Detailing Rules for Frames with Concentric Bracing 57
4.1.7. Design and Detailing Rules for Frames with Eccentric Bracing 60
4.1.8. Design Rules for Inverted Pendulum Structures 66
4.1.9. Design Rules for Moment Resisting Frames Combined with Concentric Bracings 66
4.2. Rules According to TEC2007 66
4.2.1. Design Concept 66
4.2.2. Material Conditions and Allowable Stresses 67
4.2.3. Classification of Steel Structural Systems 69
4.2.4. Frames of High Ductility Level 71
4.2.4.1 Cross Section Requirement 71
4.2.4.2 Requirement of Having Columns Stronger Than Beams 72
4.2.4.3 The Case Where Some Columns Cannot Satisfy the Requirement of Having Columns Stronger than Beams 73
4.2.4.4 Beam-Column Connections 74
4.2.4.5 Splices of Column and Beam 77
4.2.4.6 Supporting of Beam Flanges in the Lateral Direction 77
4.2.5. Frames of Nominal Ductility Level 78
4.2.5.1 Cross Section Requirement 78
4.2.5.2 Beam-Column Connections 78
4.2.5.3 Splices of Column and Beam 79
4.2.6. Concentric and Eccentric Steel Bracings 79
4.2.7. Steel Concentrically Braced Frames of High Ductility Level 80

4.2.7.1 Cross Section Requirement 80
4.2.7.2 Distribution of Horizontal Forces 80
4.2.7.3 Connections of Bracings 80
4.2.7.4 Additional Conditions for Special Bracing Arrangements 81
4.2.8. Steel Concentrically Braced Frames of Nominal Ductility Level 81
4.2.8.1 Cross Section Requirement 82
4.2.8.2 Connections of Bracings 82
4.2.8.3 Additional Conditions for Special Bracing Arrangements 83
4.2.9. Steel Eccentrically Braced Frames of High Ductility Level 83
4.2.9.1 Cross Section Requirement 83
4.2.9.2 Seismic Links 83
4.2.9.3 Supporting of Seismic Link in the Lateral Direction 84
4.2.9.4 Rotation Angle of Seismic Link 84
4.2.9.5 Stiffener Plates 85
4.2.9.6 Beams, Columns and Bracings 86
4.2.9.7 Connection of Seismic Link and Column 87
5. DESIGN OF STEEL STRUCTURES ACCORDING TO EC3 88
5.1. Scope of EC3 88
5.2. Material Properties 88
5.3. Serviceability Limit State 90
5.4. Ultimate Limit State 92
5.5. Classification of Cross Sections 93
5.6. Analysis of Cross Sections 97
5.6.1. Connections 97
5.6.1.1 Bolted Connections 98
5.6.1.2 Welded Connections 100
5.6.2. Tension Members 102
5.6.3. Compression Members 103
5.6.3.1 Buckling Resistance 103
5.6.3.2 Flexural Buckling 104
5.6.4. Flexural Members 107
5.6.5. Shear Resistance of Structural Members 107
5.6.6. Bending and Shear 109

5.6.7. Bending and Axial Force 109

5.6.7.1 Class 1 and 2 Cross-Sections 109

5.6.7.2 Class 3 Cross-Sections 111

5.6.7.3 Class 4 Cross-Sections 111

5.6.8. Bending, Shear and Axial Force 112

6. DESIGN OF STEEL STRUCTURES ACCORDING TO TSE 648 113

6.1. Scope of TSE 648 113

6.2. Material Properties 113

6.3. Load Patterns and Loading Conditions 113

6.3.1. Cross Sections Considering in Design of Steel Structural Elements... 114

6.4. Analysis of Cross Sections 115

6.4.1. Tension Members 116

6.4.2. Compression Members 116

6.4.3. Bending and Axial Compression Force 121

6.4.4. Stability Checks 124

6.4.4.1 Lateral Buckling 124

6.4.4.2 Allowable Stress of Lateral Buckling 124

7. CASE STUDY 126

7.1. Introduction 126

7.2. Loads Acting On The Structure 128

7.2.1. Earthquake Loads 129

7.3. Irregularity Checks 131

7.3.1. Irregularities In Plan 131

7.3.2. Irregularities In Elevation 132

7.4. Used Structural Materials 134

7.5. Design of Structural Members 134

7.5.1. Design of Columns 134

7.5.1.1 Section Design of Columns According to TS 648 134

7.5.1.2 Section Design of Columns According to Eurocode 3 137

7.5.2. Design of Beams 140

7.5.2.1 Section Design of Frame Beams According to TSE 648 140

7.5.2.2 Section Design of Frame Beams According to Eurocode 3 ... 143

7.5.2.3 Section Design of Secondary Beams According to TSE 648. 144

7.5.2.4 Section Design of Secondary Beams According to Eurocode 3 146

7.5.3. Design of Braces 148

7.5.3.1 Section Design of Braces According to TSE 648 148

7.5.3.2 Section Design of Braces According to Eurocode 3 149

8. CONCLUSIONS AND RECOMMENDATIONS 151

APPENDIX : Drawings 154

REFERENCES 155

LIST OF FIGURES

Figure 2.1. Elastic Response Spectrum 8

Figure 2.2. Elastic Response Spectrum for Ground Types A to E with 5% damping.... 9

Figure 2.3. Design Acceleration Spectrum 14

Figure 3.1. Criteria for regularity of buildings in elevation 20

Figure 3.2. Design Seismic Loads Acting at Storey Levels 34

Figure 3.3. Accidental Eccentricity 35

Figure 3.4. Accidental Eccentricity with A2 irregularity 35

Figure 3.5. Combined Effects of Independently Acting x and y Direction Earthquakes 37

Figure 4.1. Moment Resisting Frames with Default Values of α_u/α_1 46

Figure 4.2. Frames with Concentric Bracings 47

Figure 4.3. Frames with Eccentric Bracings with Default Values of α_u/α_1 48

Figure 4.4. Inverted Pendulum with Default Values of α_u/α_1 48

Figure 4.5. Structures with Concrete Cores or Concrete Walls 48

Figure 4.6. Moment Resisting Frames Combined with Concentric Bracings with Default Values of α_u/α_1 49

Figure 4.7. Moment Resisting Frames Combined with Infills 49

Figure 4.8. Web Panel Framed by Flanges and Stiffener 55

Figure 4.9. Beam Deflection for the Calculation of θ_p 56

Figure 4.10. Concentric Braced Frames 58

Figure 4.11. Equal and Unequal Moments at Link Ends 63

Figure 4.12. Beam-Column Joint 73

Figure 4.13. Slip Zone at the Beam-Column Joint 75

Figure 4.14. Reinforcing Plates on the Column Web 76

Figure 4.15. Concentric Bracings 79

Figure 4.16. Eccentric Bracings 80

Figure 4.17. Rotation Angle of Seismic Link 85

Figure 4.18. Seismic Link and Stiffeners 86

Figure 4.19. Seismic Link and Stiffeners 87

Figure 5.1. Vertical Deflections 91

Figure 5.2. Definitions of Geometric Properties 96

Figure 6.1. Ratio of M1/M2 122

Figure 6.2. Ψ Numbers 123

Figure 7.1. 3D Model of Structure 127

LIST OF TABLES

Table 2.1. Soil groups ... 5

Table 2.2. Local Site Classes ... 6

Table 2.3. Elastic Response Spectra Parameters ... 9

Table 2.4. Important Classes for Buildings ... 11

Table 2.5. Effective Ground Acceleration ... 12

Table 2.6. Building Importance Factor ... 13

Table 2.7. Spectrum Characteristic Periods ... 14

Table 2.8. Live Load Participation Factors ... 17

Table 3.1. Structural Regularity ... 18

Table 3.2. Values of φ for calculating $\Psi_{E,i}$ ... 27

Table 3.3. Application Limits of Equivalent Seismic Load Method ... 32

Table 4.1. Upper Limit of reference values of behaviour factors for systems regular in elevation ... 50

Table 4.2. Reference Behaviour Factor Depending on Ductility Class ... 52

Table 4.3. Da Increase Factors ... 68

Table 4.4. Magnification Factors ... 68

Table 4.5. Structural Behaviour Factor (R) 70

Table 4.6. Cross Section Conditions 72

Table 5.1. Nominal Values of the Yield Strength and the Ultimate Strength for Hot Rolled Structural Steel 89

Table 5.2. Nominal Values of the Yield Strength and the Ultimate Strength for Bolts 90

Table 5.3. Recommended Limits for Vertical Deflections 91

Table 5.4. Limiting Width-Thickness Ratio for Classification of Cross Sections (Compression and Bending) 94

Table 5.5. Limiting Width-Thickness Ratio for Classification of Cross Sections (Bending) 95

Table 5.6. Categories of Bolted Connections 98

Table 5.7. Common Types of Welded Joints 101

Table 5.8. Correlation Factors 102

Table 5.9. Imperfection Factors 104

Table 5.10. Reduction Factors χ 105

Table 5.11. Selection of a Buckling Curve for a Cross Section 106

Table 6.1. Material Properties of Steel 113

Table 6.2. Cross Sections Considering in Design of Steel Structural Members 114

Table 6.3. Variable Buckling Lengths 117

Table 6.4. The Buckling Coefficients 119

Table 7.1. Earthquake Forces Acting Each Storey 130

Table 7.2. Torsional Irregularity 131

Table 7.3. Interstorey Stiffness Irregularity 132

Table 7.4. Effective Storey Drift 133

Table 7.5. Column Section According to TSE 648 135

Table 7.6. Column Section According to EC 3 138

Table 7.7. Beam Section According to TSE 648 140

Table 7.8. Secondary Beam Section According to TSE 648 144

Table 7.9. Bracing Section According to TSE 648 148

Table 7.10. Bracing Section According to EC 3 149

Table 8.1. Design Sections 152

LIST OF SYMBOLS/ABBREVIATIONS

A_{Ed}	Design value of seismic action
A_{Ek}	Characteristic value of the seismic action for the reference return period
E_{d}	Design value of action effects
$S_{\mathrm{e}(T)}$	Elastic horizontal ground acceleration response spectrum
$S_{\mathrm{De}(T)}$	Elastic displacement response spectrum
$S_{\mathrm{d}(T)}$	Design spectrum for elastic analysis
S	Soil factor
T	Vibration period of a linear single degree of freedom system
a_{g}	Design ground acceleration on type A ground
d_{g}	Design ground displacement
g	Acceleration of gravity
q	Behaviour factor
E_{E}	Effect of the seismic action
E_{Edx}, E_{Edy}	Design values of the action effects due to the horizontal components (x and y) of the seismic action
E_{Edz}	Design value of the action effects due to the vertical component of the seismic action
F_{i}	Horizontal seismic force at storey i
F_{a}	Horizontal seismic force acting on a non-structural element (appendage)
F_{b}	Base shear force
H	Building height from the foundation or from the top of a rigid basement
L_{max}, L_{min}	Larger and smaller in plan dimension of the building measured in orthogonal directions
R_{d}	Design value of resistance
T_1	Fundamental period of vibration of a building
d	Displacement
d_r	Design interstorey drift
e_{a}	Accidental eccentricity of the mass of one storey from its nominal location
h	Interstorey height

m_i	Mass of storey *i*
n	Number of storeys above the foundation or the top of a rigid basement
z_i	Height of mass *m*i above the level of application of the seismic action
L	Beam span
M_{Ed}	Design bending moment from the analysis for the seismic design situation
$M_{pl,RdA}$	Design value of plastic moment resistance at end A of a member
$M_{pl,RdB}$	Design value of plastic moment resistance at end B of a member
N_{Ed}	Design axial force from the analysis for the seismic design situation
V_{Ed}	Design shear force from the analysis for the seismic design situation
$N_{Ed,E}$	Axial force from the analysis due to the design seismic action alone
$N_{Ed,G}$	Axial force due to the non-seismic actions included in the combination of actions for the seismic design situation
$N_{pl,Rd}$	Design value of yield resistance in tension of the gross cross-section of a member in accordance with Eurocode 3
$V_{pl,Rd}$	Design value of shear resistance of a member in accordance with Eurocode 3
$N_{Rd}(M_{Ed}, V_{Ed})$	Design value of axial resistance of column or diagonal in accordance with Eurocode 3, taking into account the interaction with the bending moment M_{Ed} and the shear V_{Ed} in the seismic situation
R_d	Resistance of connection in accordance with Eurocode 3
R_{fy}	Plastic resistance of connected dissipative member based on the design yield stress of material as defined in Eurocode 3
V_{Ed}	Design shear force from the analysis for the seismic design situation
$V_{Ed,G}$	Shear force due to the non seismic actions included in the combination of actions for the seismic design situation
$V_{Ed,M}$	Shear force due to the application of the plastic moments of resistance at the two ends of a beam
$V_{wp,Ed}$	Design shear force in web panel due to the design seismic action effects
$V_{wp,Rd}$	Design shear resistance of the web panel in accordance with Eurocode 3
e	Length of seismic link
f_y	Nominal yield strength of steel
$f_{y,max}$	Maximum permissible yield stress of steel
t_w	Web thickness of a seismic link

t_f	Flange thickness of a seismic link
Ω	Multiplicative factor on axial force $N_{Ed,E}$ from the analysis due to the design seismic action, for the design of the non-dissipative members in concentric or eccentric braced frames
α	Ratio of the smaller design bending moment $M_{Ed,A}$ at one end of a seismic link to the greater bending moments $M_{Ed,B}$ at the end where plastic hinge forms, both moments taken in absolute value
γ_M	Partial factor for material property
γ_{ov}	Material overstrength factor
δ	Beam deflection at midspan relative to tangent to beam axis at beam end
γ_{pb}	Multiplicative factor on design value $N_{pl,Rd}$ of yield resistance in tension of compression brace in a V bracing, for the estimation of the unbalanced seismic action effect on the beam to which the bracing is connected
γ_s	Partial factor for steel
θ_p	Rotation capacity of the plastic hinge region
λ	Non-dimensional slenderness of a member as defined in Eurocode 3
α	Ratio of the design ground acceleration to the acceleration of gravity
θ	Interstorey drift sensitivity coefficient
γ_I	Importance factor
η	Damping correction factor
ξ	Viscous damping ratio (in percent)
$\psi_{2,i}$	Combination coefficient for the quasi-permanent value of a variable action
$\psi_{E,i}$	Combination coefficient for a variable action *i*, to be used when determining the effects of the design seismic action
EC 3	Design of Steel Structures
EC 8	Design Provisions for Earthquake Resistant Design
TS-498	Design Loads for Buildings
TS-500	Requirements for Design and Construction of Reinforced Concrete Structures

TEC 2007 Specification for Buildings to be Built in Seismic Zones 2007

1. INTRODUCTION

In the last years earthquake design of structures becomes an important phenomena due to the disastrous earthquakes which cause a big human tragedy all around the world. These earthquakes show that the buildings have low seismic performance due to the usage of low quality material, low quality of workmanship and inadequacy of the design codes. Since then, many new codes detailing requirements have been introduced to ensure seismic resistance. For example, the provisions have been added to the Turkish Earthquake Code after the Marmara and Bolu-Düzce Earthquakes.

Earthquake resistant design of steel structures has been developing in the last years by means of analytical and experimental results. Although structural steel is in many ways an ideal material for earthquake resistance, care should be taken in design and detailing of framing systems and connections. In addition different structural systems are used in order to absorb dissipated energy during earthquake which are, concentrically braced frames, moment resisting frames and eccentrically braced frames.

In the design of steel structures, both of the earthquake codes and steel design codes should be followed simultaneously. Since the steel is a widely used structural material in European countries, there is a developed and sophisticated structural steel design code prepared by European Committee for Standardization (CEN) which is called Eurocode 3 and 8. These structural codes are frequently used between countries member of European Union. The main objective of this thesis is to present the difference and similarities between the TS648-TEC2007 and Eurocode 3-Eurocode 8 which are likely to be used in Turkey in the European Union entry process in the future.

2. GENERAL RULES OF EARTHQUAKE DESIGN

2.1. General Rules

In the following sections, the definition of seismic loads and analysis requirements to be applied to earthquake resistant design according to EC8 and TEC2007 are explained in details.

2.1.1. General Rules of EC8

Structures in seismic regions shall be design and constructed in such a way that the following requirements are met.

- No-Collapse Requirement: The structure shall be designed and constructed to withstand the design seismic action without local or global collapse, thus retaining its structural integrity and residual load bearing capacity after seismic events.
- Damage Limitation Requirement: The structure shall be designed and constructed to withstand a seismic action having a larger probability of occurrence than the design seismic action, without the occurrence of damage and the associated limitations of use, the costs of which would be disproportionately high in comparison with the costs of the structure itself. [1]

In order to satisfy the fundamental requirements of seismic design stated in EC8, the ultimate limit stated and damage limitation state should be checked. It shall be verified that the structural system has the resistance and energy dissipation capacity. The balance between resistance and energy dissipation capacity is characterized by the values of the behaviour factor “q” and the associated ductility classification. As a limiting case, for the design of structures classified as non-dissipative, no account is taken of any hysteretic energy dissipation and the behaviour factor may not be taken.

An adequate degree of reliability against unacceptable damage shall be ensured by satisfying the deformation limits or other relevant limits. In structures important for civil protection the structural system shall be verified to ensure that it has sufficient resistance and stiffness to maintain the function of the vital services in the facilities for a seismic event associated with an appropriate return period.

2.1.2. General Rules of TEC2007

The general principal of earthquake resistant design to this Specification is to prevent structural and non-structural elements of buildings from any damage in low intensity earthquakes, to limit the damage in structural and non-structural elements to repairable levels in medium-intensity earthquakes, and to prevent the overall or partial collapse of buildings in high intensity earthquakes in order to avoid the loss of life.

The design earthquake considered in this Specification corresponds to high intensity earthquake defined above. For buildings with Building Importance Factor of I = 1 the probability of exceedance of the design earthquake within a period of 50 years is 10%.

The building structural system resisting seismic loads as a whole as well as each structural element of the system shall be provided with sufficient stiffness, stability and strength to ensure an uninterrupted and safe transfer of seismic loads down to the foundation soil. In order to dissipate a significant part of the seismic energy fed into the structural system, ductile design principals should be followed. [2]

2.2. Site Conditions

2.2.1. Site Conditions According to EC8

Appropriate soil investigations should be carried out to identify the ground conditions in accordance with the local site class. The supporting ground at the construction site should be excluded from ground rapture, slope instability and permanent settlements due to the liquefaction or densification. The soil investigations and geological studies can be omitted for the structures with a low building importance factor and low earthquake hazard.

Ground Types:

Five kind of ground types are described by taking into account the influence of local ground conditions on the seismic action.

- Ground Type A: Rock or other rock like geological formation with shear wave velocity greater than 800 m/sec. Weaker material can be acceptable up to 5 m from the surface.
- Ground Type B: Deposits of very dense sand, gravel, or very stiff clay with shear wave velocity between 360 m/sec and 800 m/sec. The thickness of deposits varies at least several tens of meters.
- Ground Type C: Deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of meters. The shear wave velocity is between 180 m/sec and 360 m/sec.
- Ground Type D: Deposits of loose to medium cohesionless soil or predominantly soft to firm cohesive soil with shear wave velocity lower than 180 m/sec.
- Ground Type E: A soil profile consisting of a surface alluvium layer with shear wave velocity values of type C and type D. The thickness of deposits varies between about 5 m and 20 m. [1]

2.2.2. Site Conditions According to TEC2007

Soil groups and local site classes to be considered as the bases of determination of local soil conditions are given in Table 2.1 and Table 2.2, respectively. Values of soil parameters in Table 2.1 should be considered as standard values given for the guidance in determining the soil groups.

Soil investigation based on appropriate site and laboratory tests are mandatory to be conducted for below given buildings with related reports prepared and attached to design documents. Soil groups and local site classes to be defined in accordance with Table 2.1 and Table 2.2 should be clearly indicated in reports.

- All buildings with total height exceeding 60 m in the first and second seismic zones,
- Irrespective of building height, buildings in all seismic zones with Building Importance Factor of I = 1.5 and I = 1.4.

Table 2.1. Soil groups

Soil Group	*Description of Soil Group*	*Stand Penetr. (N/30)*	*Relative Density (%)*	*Unconf. Compres. Strength (kPa)*	*Shear Wave Velocity (m/s)*
(A)	**1.** Massive volcanic rocks, unweathered sound metamorphics rocks, stiff cemented sedimentary rocks	—	—	> 1000	> 1000
	2. Very dense sand, gravel.......	> 50	85-100	—	> 700
	3. Hard clay, silty lay...............	> 32	—	> 400	> 700
(B)	**1.** Soft volcanic rocks such as tuff and agglomerate, weathered cemented sedimentary rocks with planes of discontinuity.............	—	—	500 - 1000	700 - 1000
	2. Dense sand, gravel.............	30 - 50	65 - 85	—	400 - 700
	3. Very stiff clay, silty clay........	16 - 32	—	200 - 400	300 - 700
(C)	**1.** Highly weathered soft metamorphic rocks and cemented sedimentary rocks with planes of discontinuity	—	—	< 500	400 - 700
	2. Medium dense sand and gravel	10 - 30	35 - 65	—	200 - 400
	3. Stiff clay, silty clay...............	8 - 16	—	100 - 200	200 - 300
(D)	**1.** Soft, deep alluvial layers with high water table..............	—	—	—	< 200
	2. Loose sand........................	< 10	< 35	—	< 200
	3. Soft clay, silty clay.............	< 8	—	< 100	< 200

Table 2.2. Local Site Classes

Local Site Class	*Soil Group according to Table 12.1 and Topmost Layer Thickness (h_1)*
Z1	Group **(A)** soils Group **(B)** soils with $\mathbf{h_1 \leq 15}$ **m**
Z2	Group **(B)** soils with $\mathbf{h_1 > 15}$ **m** Group **(C)** soils with $\mathbf{h_1 \leq 15}$ **m**
Z3	Group **(C)** soils with $\mathbf{15\ m < h_1 \leq 50\ m}$ Group **(D)** soils with $\mathbf{h_1 \leq 10}$ **m**
Z4	Group **(C)** soils with $\mathbf{h_1 > 50}$ **m** Group **(D)** soils with $\mathbf{h_1 > 10}$ **m**

Regarding the buildings outside the scope of above given, in the first and second seismic zones, available local information or observation results should be included or published references should be noted in the seismic analysis reports to identify the soil groups and local site classes in accordance with Table 2.1 and Table 2.2. Also, in the first and second seismic zones, horizontal bedding parameters as well as horizontal and vertical load carrying capacities of piles under seismic loads in Group (C) and (D) soils according to Table 2.1 should be determined on the basis of soil investigation including in-situ and laboratory tests. [2]

2.3. Seismic Design

2.3.1. Definition of Seismic Action According to EC8 and TEC2007

Representation of Seismic Action in EC8: National territories shall be subdivided by National Authorities into seismic zones depending on the local hazard. The hazard within the each zone is assumed to be constant.

For most of the applications of EC, the hazard is described in terms of a single parameter, the value of the reference peak ground acceleration on type A ground, a_{gR}. The reference peak ground acceleration corresponds to the reference return period of the seismic action for the no-collapse requirement or the reference probability of exceedance in 50 years. An importance factor γ_I equal to 1,0 is assigned to this reference return period.

In cases of low seismicity where the design ground acceleration is not greater than 0,1 g , reduced or simplified seismic design procedures for certain types or categories of structures may be used. In cases of very low seismicity where the design ground acceleration is not greater than 0,04 g , the provisions of EC need not be observed.

Basic Representation of the Seismic Design: The earthquake motion at a given point on the surface is represented by an elastic ground acceleration response spectrum which is called *"elastic response spectrum"*. The horizontal seismic action is described by two orthogonal components as being independent and represented by the same response spectrum. When the earthquakes affecting a site are generated by widely differing sources, the possibility of using more than one shape of spectra should be considered to represent the design seismic action adequately. Topographic amplification effects should be taken into account for important structures at high seismicity zones.

Horizontal Elastic Response Spectrum: As seen in Figure 2.1, the elastic response spectrum $S_e(T)$ is defined by the following expressions for the horizontal components of seismic action.

$$0 \leq T \leq T_B \ : S_e(T) = a_g \, . S \, . \left[1 + \frac{T}{T_B} \, . \ (\eta \, . 2{,}5 - 1)\right] \tag{2.1}$$

$$T_B \ \leq T \leq T_C \ : S_e(T) = a_g \, . S \, . \, \eta \, . 2{,}5 \tag{2.2}$$

$$T_C \ \leq T \leq T_D \ : S_e(T) = a_g \, . S \, . \, \eta \, . 2{,}5 \, . \left[\frac{T_C}{T}\right] \tag{2.3}$$

$$T_D \ \leq T \leq 4 \ s : S_e(T) = a_g \, . S \, . \, \eta \, . 2{,}5 \, . \left[\frac{T_C T_D}{T^2}\right] \tag{2.4}$$

$S_e(T)$: Elastic response spectrum

T : Vibration period of a linear single degree of freedom system

a_g : Design ground acceleration on type A ground

T_B : Lower limit of period of the constant spectral acceleration branch

T_C : Upper limit of period of the constant spectral acceleration branch

T_D : Value defining the beginning of the constant displacement response range of spectrum

S : Soil factor

η : Damping correction factor with a reference value of $\eta = 1$ for 5% viscous damping

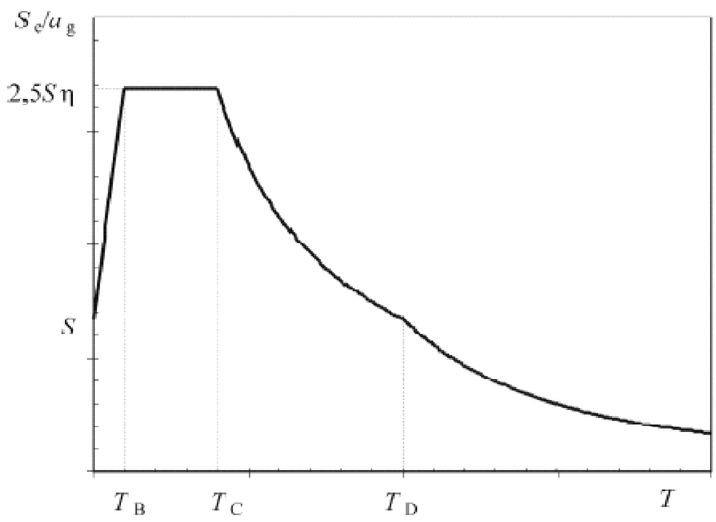


Figure 2.1. Elastic Response Spectrum

The values of the periods T_B, T_C, T_D and the soil factor S describing the shape of the elastic response spectrum depend on the ground type. The values of parameters are stated in Table 2.3 below.

Table 2.3. Elastic Response Spectra Parameters

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

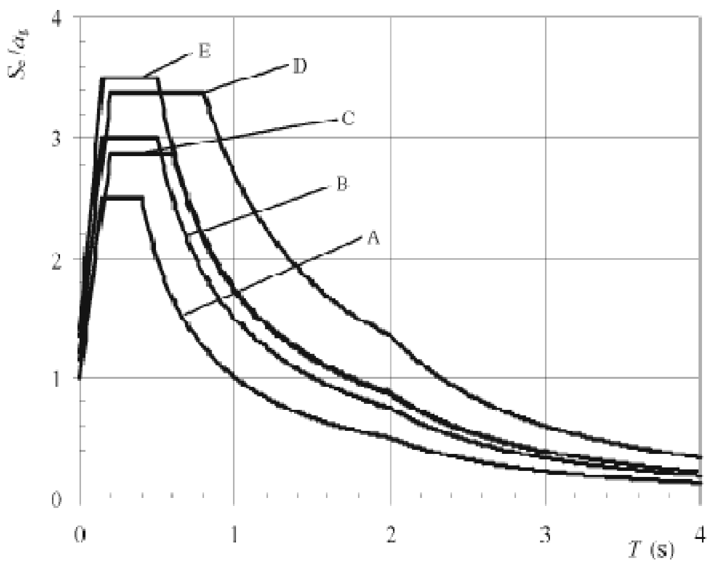


Figure 2.2. Elastic Response Spectrum for Ground Types A to E with 5% damping

The value of damping correction factor η is determined by the following expression.

$$\eta = \sqrt{10/(5+\xi)} \geq 0{,}55 \tag{2.5}$$

ξ : viscous damping ratio of the structure expressed as a percentage. A viscous damping ratio may be used different from 5% for special cases.

Design Spectrum for Elastic Analysis: The capacity of structural systems to resist seismic actions in the non-linear range generally permits their design for resistance to seismic forces smaller than those corresponding to a linear elastic response.

Instead of inelastic structural analysis in design, the energy dissipation capacity of the structure through mainly ductile behaviour of its elements is taken into account by performing an elastic analysis based on a reduced response spectrum which is called *"design spectrum"*. This reduction is fulfilled by the *behaviour factor q*. The behaviour factor q is an approximation of the ratio of the seismic forces that the structure would experience if its response was completely elastic with 5% viscous damping, to the seismic forces that may be used in the design with a elastic analysis model. The values of a behaviour factor q, which also account for the influence of the viscous damping different from 5%, are given for various materials and structural systems according to the relevant ductility classes. [1]

The design spectrum $S_d(T)$ for the horizontal components of the seismic action shall be defined by the following expressions.

$$0 \leq T \leq T_B \ : S_d(T) = a_g \, . S \, . \left[\frac{2}{3} + \frac{T}{T_B} \, . \, \left(\frac{2{,}5}{q} - \frac{2}{3} \right) \right] \tag{2.6}$$

$$T_B \leq T \leq T_C \ : S_d(T) = a_g \, . S \, . \, \frac{2{,}5}{q} \tag{2.7}$$

$$T_C \leq T \leq T_D \ : S_d(T) = \begin{cases} = a_g \, . S \, . \, \frac{2{,}5}{q} \, . \, \left[\frac{T_C}{T} \right] \\ \geq \beta \, . \, a_g \end{cases} \tag{2.8}$$

$$T_D \leq T : S_d(T) = \begin{cases} = a_g \, . S \, . \, \frac{2{,}5}{q} \, . \, \left[\frac{T_C T_D}{T^2} \right] \\ \geq \beta \, . \, a_g \end{cases} \tag{2.9}$$

$S_d(T)$: Design spectrum

q : Behaviour factor

β : Lower bound factor for the horizontal design spectrum may be used as 0,2

a_g : Design ground acceleration on type A ground

T : Vibration period of a linear single degree of freedom system

T_B : Lower limit of period of the constant spectral acceleration branch

T_C : Upper limit of period of the constant spectral acceleration branch

T_D : Value defining the beginning of the constant displacement response range of spectrum

S : Soil factor

Building Importance Factor: Buildings are classified in 4 importance classes, depending on the consequences of collapse for human life, on their importance for public safety and civil protection in the immediate post earthquake period, and on the social and economic consequences of collapse. The definitions of the important classes are given in Table 2.4.

Table 2.4. Important Classes for Buildings

Important Class	**Buildings**	**Importance Factor**
I	Building of minor importance for public safety, e.g. agricultural buildings, etc.	**0.8**
II	Ordinary buildings, not belonging in the other categories.	**1**
III	Building whose seismic resistance is of importance in view of the consequences associated with collapse, e.g. Schools, assembly halls, cultural institutions etc.	**1.2**
IV	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. Hospitals, fire stations, power plants, etc.	**1.4**

Representation of Seismic Action in TEC2007: The spectral acceleration coefficient, $A(T)$, to be considered for determining seismic loads is given by Eqn. 2.10. The elastic spectral acceleration $S_{ae}(T)$, which is defined as the ordinate of 5% damped elastic design acceleration spectrum is equal to spectral acceleration coefficient times the acceleration of gravity, g.

$$A\ (T) = A_0\ I\ S(T) \tag{2.10}$$

$$S_{ae}\ (T) = A(T)\ g \tag{2.11}$$

Effective Ground Acceleration Coefficient: The effective ground acceleration coefficient, A_o, given in Eqn. 2.10 is stated in Table 2.5.

Table 2.5. Effective Ground Acceleration

Seismic Zone	***Ao***
1.00	0.40
2.00	0.30
3.00	0.20
4.00	0.10

Building Importance Factor: The building importance factor, I , given in Eqn. 2.10 is stated in Table 2.6.

Table 2.6. Building Importance Factor

Purpose of Occupancy or Type of Building	***Importance Factor (I)***
1. Buildings to be utilized after the earthquake and buildings containing hazardous materials **a)** Buildings required to be utilized 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, country and municipality administration buildings, first aid and emergency planning stations) **b)** Buildings containing or storing toxic, explosive and flammable materials, etc.	1.5
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	1.4
3. Intensively but short-term occupied buildings Spor facilities, cinema, theatre and concert halls, etc	1.2
4. Other buildings Buildings other than above-defined buildings. (Residential and office buildings, hotels, building-like industrial structures, etc.)	1.0

Spectrum Coefficient: The spectrum coefficient, *S(T)*, given in Eqn. 2.11 and seen in Figure 2.3 shall be defined by following expressions. The spectrum coefficient depends on local site conditions and the building natural period, T.

$$S\,(T) = 1 + 1.5\,\frac{T}{T_A} \qquad (0 \leq T \leq T_A) \qquad (2.12)$$

$$S\,(T) = 2.5 \qquad (T_A \leq T \leq T_B) \qquad (2.13)$$

$$S\,(T) = 2.5\,\left(\frac{T_B}{T}\right)^{0.8} \qquad (T_B\ \leq T) \qquad (2.14)$$

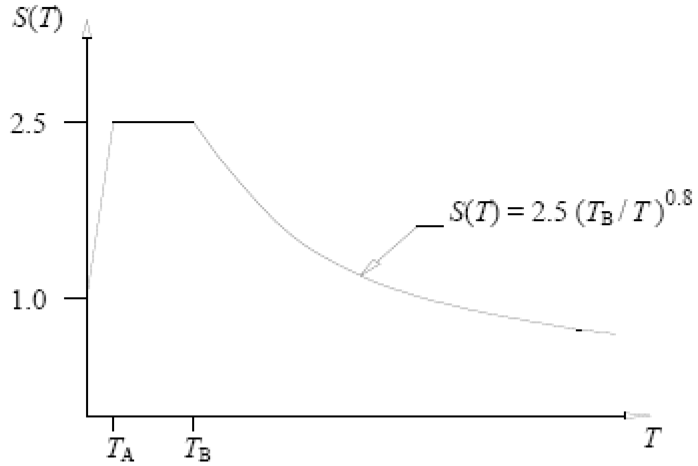


Figure 2.3. Design Acceleration Spectrum

Spectrum characteristic periods, T_A and T_B are specified in Table 2.7 depending on local site classes.

Table 2.7. Spectrum Characteristic Periods

Local Site Class	**Ta (second)**	**Tb (second)**
Z1	0.10	0.30
Z2	0.15	0.40
Z3	0.15	0.60
Z4	0.20	0.90

Special Design Acceleration Spectra: Elastic acceleration spectrum may be determined through special investigations by considering local seismic and site conditions. However spectral acceleration coefficient corresponding to acceleration spectrum ordinates shall not be less than those determined by Eqn. 2.10 based on relevant characteristic periods given in Table 2.7.

Reduction of Elastic Seismic Loads: Elastic seismic loads determined in terms of spectral acceleration coefficient shall be divided to below-defined Seismic Load Reduction Factor to account for the specific nonlinear behaviour of the structural system during earthquake. Seismic Load Reduction Factor, $R_a(T)$, shall be determined by Eqn. 2.15 and Eqn. 2.16 in terms of Structural Behaviour Factor, R, defined in the following chapters for various structural systems, and the natural vibration period T. [2]

$$R_a(T) = 1.5 + (R-1.5)\, T/T_A \qquad (0 \leq T \leq T_A) \qquad (2.15)$$

$$R_a(T) = R \qquad (T_A < T) \qquad (2.16)$$

2.3.2. Definition of Load Combinations According to EC8 and TEC2007

Combination of the Seismic Action with Other Actions According to EC8: The design value E_d of the effects of actions in the seismic design situation shall be determined in accordance with the following combination; [3]

$$\sum G_{kj} + \gamma_1 A_{Ed} + \sum \varphi_{2i} Q_{ki} \qquad (2.17)$$

Where;

G_{kj} : Characteristic values of dead loads j

γ_1 : Importance Factor

A_{Ed} : Design value for return period of specific earthquake motion

φ_{2i} : Combination coefficient of live loads

Q_{ki} : Characteristic value of live loads

The inertial effects of the design seismic action shall be evaluated by taking into account the presence of the masses associated with all gravity loads appearing in the following combination of actions;

$$\sum G_{kj} + \sum \Psi_{E,i} Q_{ki} \qquad (2.18)$$

Where;

$\Psi_{E,i}$: The combination coefficient for variable action *I*

The values of variable action are stated in the calculation of base shear due to the seismic action in the further chapters.

The combination coefficient $\Psi_{E,i}$ take into account the likelihood of the loads $Q_{k,i}$ not being present over the entire structure during the earthquake. These coefficients may also account for a reduced participation of masses in the motion of structure due to the non-rigid connection between them. [4]

Combination of the Seismic Action with Other Actions According to TEC2007: The design value E_d of the effects of actions in the seismic design situation shall be determined in accordance with the following combination; [2]

$$G + Q \pm E_x \pm 0.3E_y \qquad (2.19)$$

$$G + Q \pm E_y \pm 0.3E_x \qquad (2.20)$$

or in the case of unfavorable results,

$$0.9G \pm E_x \pm 0.3E_y \qquad (2.21)$$

$$0.9G \pm E_y \pm 0.3E_x \qquad (2.22)$$

The seismic weight of the structure shall be determined by Eqn. 2.23.

$$W = \sum G_{i,N} + \sum n\, Q_{i,N} \qquad (2.23)$$

Live Load Participation Factor, n , is given in Table 2.8. In industrial buildings, n=1 shall be taken for fixed equipment weights while crane payloads shall not be taken into account in the calculation of storey weights. In the calculation of roof weights for seismic loads, 30% of snow loads shall be considered.

Table 2.8. Live Load Participation Factors

Purpose of Occupancy of Building	n
Depot, warehouse, etc.	0.80
School, dormitory, sport facility, cinema, concert hall, car park, restaurant, shop, etc.	0.60
Residence, office, hotel, hospital, etc.	0.30

3. GENERAL RULES FOR BUILDING STRUCTURAL SYSTEM

3.1. Criteria for Structural Regularity According to EC8

The building structures are categorized into being regular or non-regular for the purpose of seismic design. This distinction has implications for the following aspects of the seismic design.

- The structure model, which can be either a simplified planar model or a spatial model.
- The method of analysis, which can be either a simplified response spectrum analysis (lateral force procedure) or a modal one.
- The value of the behaviour factor q, which shall be decreased for buildings non-regular in elevation.

Separate consideration is given in Table 3.1 to the regularity characteristics of the building in plan and in elevation in terms of the implications of structural regularity on analysis and design.

Table 3.1. Structural Regularity

Regularity		Allowed Simplification		Behaviour factor
Plan	Elevation	Model	Linear - elastic Analysis	(for linear analysis)
Yes	Yes	Planar	Lateral force	Reference value
Yes	No	Planar	Modal	Decreased value
No	Yes	Spatial	Lateral force	Reference value
No	No	Spatial	Modal	Decreased value

3.1.1. Criteria for Regularity in Plan

The building structure shall be approximately symmetrical in plan with respect to two orthogonal axes in terms of the lateral stiffness and mass distribution. The plan configuration should be compact. For instance, each floor should be delimited by a polygonal convex line. If re-entrant corners or edge recesses exist in plan, regularity in plan may still be considered as being satisfied that these setbacks don not affect the floor in plan stiffness and the area between the outline of the floor and a convex polygonal line enveloping the floor does not exceed 5% of the floor area. The L, C, H, I, and x plan shapes should be carefully examined with respect to the rigid diaphragm condition. The in-plan stiffness of the floors should be sufficient in comparison with the lateral stiffness of the vertical structural elements. The slenderness $\lambda = L_{max} / L_{min}$ of the building shall not be higher than 4, where L_{max} and L_{min} are respectively the larger and smaller dimensions of the buildings in plan. The structural eccentricity in the direction of analysis considered shall not be greater than 30% of the torsional radius which is defined as the square root of the ratio of the global torsional stiffness to the lateral stiffness.

3.1.2. Criteria for Regularity in Elevation

All lateral load resisting systems, such as cores, structural walls, or frames shall run without interruption from their foundations to the top of the building. Both the lateral stiffness and the mass of the individual stories shall remain constant or reduce gradually, without abrupt changes, from the base to the top of a particular building. In framed buildings, the ratio of the actual storey resistance to the resistance required by the analysis should not vary disproportionately between adjacent stories. For the structures with setbacks in elevation, the following conditions should be considered.

- The setbacks at any floor shall not be greater than 20% of the previous plan dimension in the direction of the setback. (See Figure 3.1.a and 3.1.b)

- For a single setback within the lower 15% of the total height of the main structural system, the setback shall not be greater than 50% of the previous plan dimension. In this case the structure should be designed to resist at least 75% of the horizontal shear forces that would developed in that zone in a similar building without the base enlargement. (See Figure 3.1.c)

- If the setbacks do not preserve symmetry, in each face of the sum of the setbacks at all stories shall be not greater than 30% of the plan dimension at the ground floor above the foundation or above the top of a rigid basement, and the individual setbacks shall be not greater than 10% of the previous plan dimension. (See Figure 3.1.d) [1]

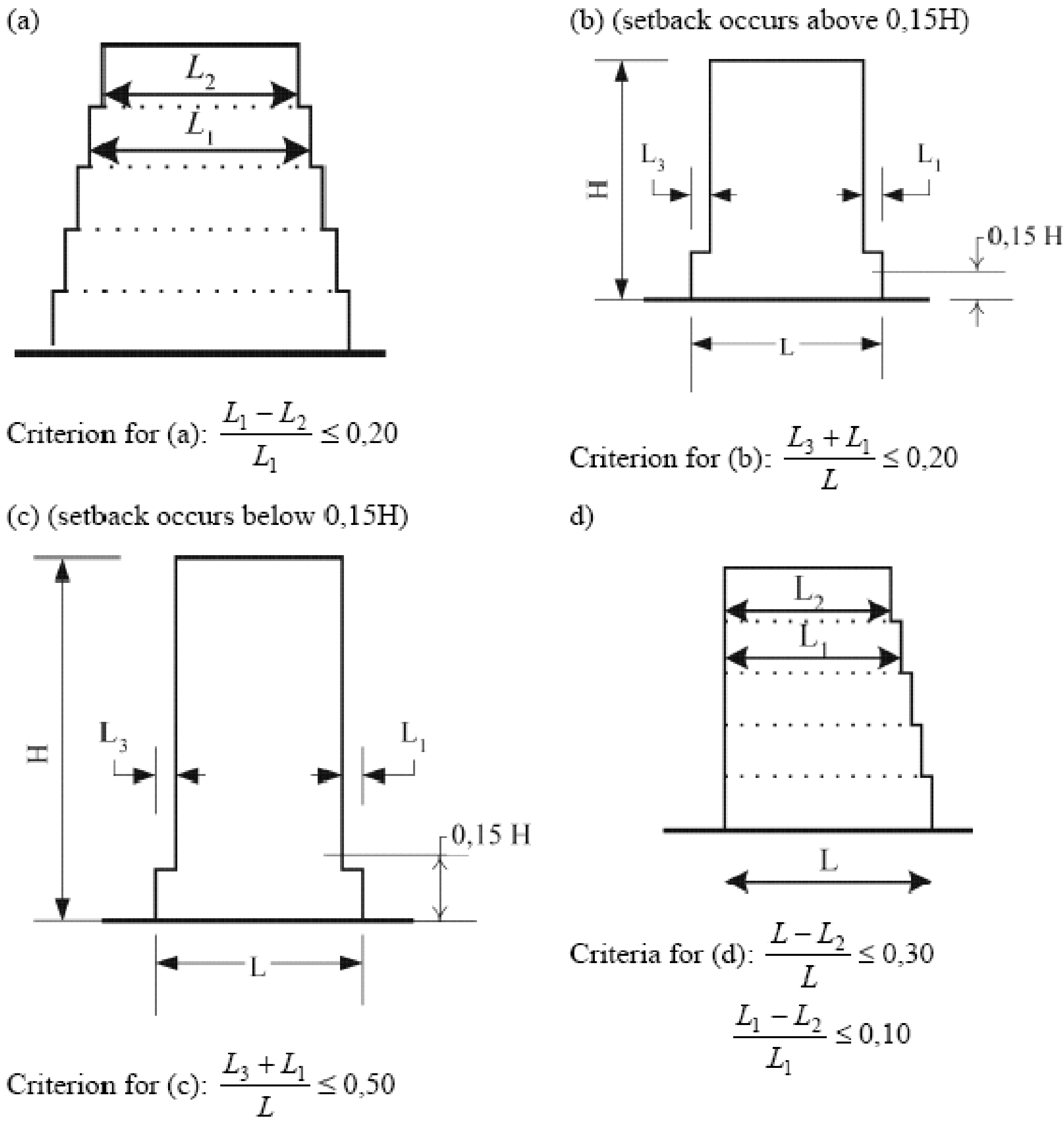


Figure 3.1. Criteria for regularity of buildings in elevation

3.2. Criteria for Structural Regularity According to TEC2007

Regarding the definition of irregular buildings whose design and construction should be avoided because of their unfavorable seismic behaviour, types of irregularities in plan and in elevation are given below.

3.2.1. Criteria for Regularity in Plan

A1- Torsional Irregularity: The case where *Torsional Irregularity Factor* η_{bi} which is defined for any of the two orthogonal earthquake directions as the ratio of the maximum storey drift at any storey to the average storey drift at the same storey in the same direction, is greater than 1.2.

$$\eta_{bi} = (\Delta_i)_{max} \,/\, (\Delta_i)_{ort} > 1.2 \qquad (3.1)$$

Storey drifts shall be calculated by considering the effects of ± 5% additional eccentricities.

A2- Floor Discontinuities: In any floor;

- The case where the total area of the openings including those of stairs and elevator shafts exceeds 1/3 of the gross floor area.
- The case where local floor openings make it difficult the safe transfer of seismic loads to vertical structural elements.
- The cases of abrupt reductions in the in-plane stiffness and strength of floors.

A3- Projections in Plan: The cases where projections beyond the re-entrant corners in both of the two principals directions in plan exceed the total plan dimensions of the building in the respective directions by more than 20%.

3.2.2. Criteria for Regularity in Elevation

B1- Interstorey Strength Irregularity (Weak Storey): In reinforced concrete buildings, the case where in each of the orthogonal earthquake directions, *Strength Irregularity Factor* η_{ci}, which is defined as the ratio of the effective shear area of any storey to the effective shear area of any storey to the effective shear area of the storey immediately above, is less than 0.80.

$$\eta_{ci} = (\Sigma A_e)_i \,/\, (\Sigma A_e)_{i+1} < 0.80 \qquad (3.2)$$

Definition of effective shear area in any storey:

$$\Sigma A_e = \Sigma A_w + \Sigma A_g + 0.15\, \Sigma A_k \qquad (3.3)$$

B2- Interstorey Stiffness Irregularity (Soft Storey): The case where in each of the two orthogonal earthquake directions, *Stiffness Irregularity Factor* η_{ki}, which is defined as the ratio of the average storey drift at any storey to the average storey drift at the storey immediately above or below, is greater than 2.0.

$$\eta_{ki} = (\Delta_i/h_i)_{ort} \,/\, (\Delta_{i+1}/h_{i+1})_{ort} > 2.0 \qquad (3.4)$$

$$\eta_{ki} = (\Delta_i/h_i)_{ort} \,/\, (\Delta_{i-1}/h_{i-1})_{ort} > 2.0 \qquad (3.5)$$

Storey drifts shall be calculated by considering the effects of ± 5% additional eccentricities.

B3- Discontinuities of Vertical Structural Elements: The cases where vertical structural elements (columns or structural walls) are removed at some stories and supported by beams or gusseted columns underneath, or the structural walls of upper stories are supported by columns or beams underneath. [2]

3.2.3. Conditions for Irregular Buildings

In buildings with irregularity types A2 and A3, it shall be verified by calculation in the first and second seismic zones that the floor systems are capable of safe transfer of seismic loads between vertical structural elements.

In buildings with irregularity type B1, if total infill wall area at i'th storey is greater than that of the storey immediately above, then infill walls shall not be taken into account in the determination of η_{ci}. In the range of $0.60 \leq (\eta_{ci}) < 0.80$, structural behaviour factor shall be multiplied by 1.25 $(\eta_{ci})_{min}$ which shall be applicable to the entire building in both earthquake directions. In no case, $\eta_{ci} < 0.60$ shall be permitted. Otherwise strength and stiffness of the weak storey shall be increased and the seismic analysis shall be repeated.

Conditions related to buildings with irregularities of type B3 are given below :

- In all seismic zones, columns at any storey of the building shall in no case be permitted to rest on the cantilever beams or top of or at the tip of gussets provided in the columns underneath.
- In the case where a column rests on a beam which is supported at both ends, all internal force components induced by the combined vertical loads and seismic loads in the earthquake direction considered shall be increased by 50% at all sections of the beam and at all sections of the other beams and columns adjoining to the beam.
- In no case the walls shall be permitted to rest on columns underneath.
- Structural walls shall in no case be permitted in their own plane to rest on the beam span at any storey of the building. [2]

3.3. Selection of Analysis Method

3.3.1. Structural Modeling & Analysis According to EC8

The model of the building shall adequately represent the distribution of stiffness and mass in it so that all significant deformation shapes and inertia forces are properly accounted for the consideration of seismic action. The structure may be considered to consist of vertical and lateral load resisting systems connected by horizontal diaphragms. When the floor diaphragms of the building may be taken as being rigid in their planes, the masses and the moments of inertia of each floor may be lumped at the center of gravity.

The structural analysis may be performed using two planar models for each main direction in the buildings conforming to the criteria for regularity in plan.

Accidental Torsional Effects: In order to account for uncertainties in the location of masses and in the spatial variation of seismic action, the calculated centre of mass at each floor shall be considered as being displaced from its nominal location in each direction by an accidental eccentricity.

$$e_{ai} = \pm 0.05 L_i \tag{3.6}$$

where

e_{ai} : The accidental eccentricity of storey mass *i* from its nominal location, applied in the same direction at all floors.

L_i : The floor dimension perpendicular to the direction of seismic action.

Methods of Analysis: The seismic effects and the effects of the other actions included in the seismic design may be determined on the basis of the linear elastic behaviour of the structure. One of the following reference method for determining the seismic effects should be selected depending on the structural characteristics of the building.

- Lateral force method of analysis

- Modal response spectrum analysis

As an alternative to a linear method, a non-linear method may also be used such as non-linear static (pushover) analysis, non-linear time history (dynamic) analysis.
Depending on the importance class of the building, linear elastic analysis may be performed using two planar models, one for each main horizontal direction, even if the criteria for regularity in plan are not satisfied, provided that all of the following special regularity conditions are met. [1]

- The building shall have well-distributed and relatively rigid cladding and partitions.
- The building height shall not exceed 10 m.
- The in-plane stiffness of the floors shall be large enough in comparison with the lateral stiffness of the vertical structural elements, so that a rigid diaphragm behaviour may be assumed.
- The centers of lateral stiffness and mass shall be each approximately on a vertical line and in the two horizontal directions of analysis satisfy the conditions:

$$r_x^2 > l_s^2 + e_{ox}^2 \tag{3.7}$$

$$r_y^2 > l_s^2 + e_{oy}^2 \tag{3.8}$$

where
l_s is the radius of gyration.
r_x and r_y are the torsional radius.
e_{ox} and e_{oy} are the natural eccentricities.

Lateral Force Method of Analysis: This type of analysis may be applied to buildings whose response is not significantly affected by contributions from modes of vibration higher than the fundamental mode in each principal direction. And also, the structure should satisfy the criteria for regulation in plan and elevation and the following condition.

- The fundamental periods of vibration T_1 in the two main directions which are smaller than the following values;

$$T_1 \le \begin{cases} 4T_C \\ 2.0\ s \end{cases} \qquad (3.9)$$

Tc: Upper limit of period of the constant spectral acceleration branch.

Base Shear Force: The seismic base shear force F_b for each horizontal direction in which the building analyzed shall be determined using the Eqn. 3.10.

$$F_b = S_d(T_1)\ m\ \lambda \qquad (3.10)$$

$S_d(T_1)$: Fundamental period of vibration of the building for lateral motion in the considered earthquake direction.

m : The total mass of the building above the foundation or above the top of a rigid basement computed as $\Sigma G_{k,j} + \Sigma \Psi_{E,i} * Q_{k,i}$ (where $\Psi_{E,i}$ is the combination coefficient for variable action)

λ : The correction factor, the value of which is equal to the $\lambda = 0.85$ if $T_1 \le 2T_C$ and the building has more than two stories or the $\lambda = 1.0$ otherwise.

The combination coefficients $\Psi_{E,i}$ for the calculation of the effects of the seismic actions shall be computed from the Eqn. 3.11.

$$\Psi_{E,i} = \varphi\ \Psi_{2i} \qquad (3.11)$$

Ψ_{2i} : The combination coefficients for variable actions

The values of φ are given in Table 3.2.

Table 3.2 Values of φ for calculating $\Psi_{E,i}$

Type of variable action	Storey	φ
Categories A-C*	Roof Storeys with correlated occupancies Independently occupied storeys	1.0 0.8 0.5
Categories D-F* and Archives		1.0

Rayleigh method may be used for the determination of the fundamental period of vibration period T_1 of the building. For the buildings with heights of up to 40 m, the value of T_1 may be approximated by the Eqn. 3.12.

$$T_1 = C_t H^{3/4} \tag{3.12}$$

C_t : 0.085 for the moment resistant space steel frames.

0.075 for the moment resistant space concrete frames and eccentrically braced steel frames.

0.050 for all other structures.

H : The height of the building from the foundation or from the top of a rigid basement.

Alternatively, the estimation of T_1 may be made by using the Eqn. 3.13.

$$T_1 = 2\sqrt{d} \tag{3.13}$$

d : The lateral elastic displacement of the top of the building due to the gravity loads applied in the horizontal direction.

Distribution of the Horizontal Seismic Forces: The seismic action effects shall be determined by applying horizontal forces F_i to all stories. The horizontal force acting on storey i ;

$$F_i = F_b \frac{s_i\, m_i}{\sum s_j\, m_j} \tag{3.14}$$

F_b : The seismic base shear in accordance with Eqn. 3.10.
s_i, s_j : The displacements of masses m_i, m_j in the fundamental mode shape.
m_i, m_j : The storey masses computed

When the fundamental mode shape is approximated by horizontal displacements increasing linearly along the height, the horizontal forces F_i should be computed by using the heights of the masses instead of the displacements of masses.

$$F_i = F_b \frac{z_i\, m_i}{\sum z_j\, m_j} \tag{3.15}$$

z_i, z_j : The heights of the masses m_i, m_j above the level of application of the seismic action.

Torsional Effects: If the lateral stiffness and masses are symmetrically distributed in plan and unless the accidental eccentricity is taken into account by a more exact method, the accidental torsion effects may be accounted for by multiplying the action effects in the individual load resisting elements by a factor δ.

$$\delta = 1 + 0.6 \frac{x}{L_e} \tag{3.16}$$

x : The distance of the element under consideration from the centre of mass of the building in plan measured perpendicularly to the direction of the seismic action considered.
L_e : The distance between the two outermost lateral load resisting elements measured perpendicularly to the direction of the seismic action considered.

If the analysis is performed using two planar models, torsional effects may be determined by doubling the accidental eccentricity e_{ai} of Eqn. 3.6 and applying Eqn. 3.16 with factor 1.2 instead of 0.6.

Modal Response Spectrum Analysis: This type analysis shall be applied to buildings which do not satisfy the criteria for regulation in plan and elevation. The response of all modes of vibration contributing significantly to the global response shall be taken into account. The sum of the effective modal masses for the modes taken into account amounts in dynamic analysis of structure shall be more than 90% of the total mass of the structure. Beside to this, all modes with effective modal masses which are greater than 5% of the total mass shall be taken into account. When using a spatial model, the above conditions should be verified for each relevant direction. If torsional modes make a significant contribution for the modes with effective modal masses in buildings, the minimum number of "k" of modes to be taken into account in a spatial analysis should satisfy both the following conditions.

$$k \geq 3\sqrt{n} \quad \text{and} \quad T_k \leq 0.20 \text{ sec}$$

k : The number of modes taken into account

n : The number of storeys above the foundation or the top of the a rigid basement

T_k : The period of vibration of mode k

Combination of Modal Responses:

The response in two vibration modes i and j including both translational and torsional modes may be taken as independent of each other, if their periods T_i and T_j satisfy the following conditions.

$$T_j \leq 0.9T_i \quad \text{and} \quad T_j \leq T_i$$

Whenever all relevant modal responses may be regarded as independent of each other, the maximum value E_E of a seismic action effect may be taken as the following Eqn. 3.17.

$$E_E = \sqrt{\sum E_{Ei}^{\,2}} \qquad (3.17)$$

E_E : The seismic action effect under consideration (Force, displacement, etc.)

E_{Ei} : The value of seismic action effect due to the vibration mode i

If the modes are not independent of each other, more accurate procedure for the combination of the modal maxima, such as “Complete Quadratic Combination” (CQC) shall be adopted.

Torsional Effects: Whenever a spatial model is used for the analysis, the accidental torsional effects may be determined as the envelope of the effects resulting from the application of static loadings, consisting of sets of torsional moments M_{ai} about the vertical axis of each storey i. M_{ai} shall be defined as the following Eqn. 3.18.

$$M_{ai} = e_{ai} \cdot F_i \qquad (3.18)$$

M_{ai} : The torsional moment applied at storey i about its vertical axis

e_{ai} : The accidental eccentricity of storey mass i

F_i : The horizontal force acting on storey I

The effects of loadings should be taken into account with positive and negative signs.

Combination of the Effects of the Components of the Seismic Action: The horizontal components of the seismic action shall be taken as acting simultaneously. The structural response to each component shall be evaluated separately, using the combination rules for modal responses as previously explained in modal response spectrum analysis. Another method is that the square root of the sum of the squared values of the action effect due to each horizontal component. Beside to these, the action effects due to the combination of the horizontal components of the seismic action may be computed using both of the two following combination as an alternative method.

$$E_{Edx} + 0.30E_{Edy}$$

$$0.30\ E_{Edx} + E_{Edy}$$

E_{Edx} : The action effects due to the application of the seismic action along the chosen horizontal axis x of the structure

E_{Edy} : The action effects due to the application of the seismic action along the chosen horizontal axis y of the structure

If the structural system or the regularity classification of the building in elevation is different in different horizontal directions, the value of the behaviour factor q may also be different. For the buildings satisfying the regularity in plan and in which walls or independent bracing systems in the two main horizontal directions are the only primary seismic elements, the seismic action may be assumed to act separately and without combination along the two main orthogonal horizontal axes of the structure.

Vertical Components of the Seismic Action: The vertical component of the seismic action should be taken into account in the cases listed below.

- For horizontal or nearly horizontal structural members spanning 20 m or more
- For horizontal or nearly horizontal cantilever components longer than 5 m
- For horizontal or nearly horizontal pre-stressed components
- For the beams supporting columns
- In base isolated structures

The effects of the vertical component shall be taken into account only for the elements under consideration stated above and their directly associated supporting elements or substructures. If the horizontal components of the seismic action are also relevant for these elements, the following combinations may be used for the computation of the seismic action effects.

$$E_{Edx} + 0.30E_{Edy} + 0.30E_{Edz}$$
$$0.30E_{Edx} + E_{Edy} + 0.30E_{Edz}$$
$$0.30E_{Edx} + 0.30E_{Edy} + E_{Edz}$$

E_{Edx} : The action effects due to the application of the seismic action along the chosen horizontal axis x of the structure

E_{Edy} : The action effects due to the application of the seismic action along the chosen horizontal axis y of the structure

E_{Edz} : The action effects due to the application of the vertical component of the design seismic action

3.3.2. Structural Modeling & Analysis According to TEC2007

Methods to be used for the seismic analysis of buildings and building-like structures are Equivalent Seismic Load Method, Mode Combination Method and other non-linear methods. Buildings for which Equivalent Seismic Load Method is applicable are listed in Table 3.3. [2]

Table 3.3. Application Limits of Equivalent Seismic Load Method

Seismic Zone	***Type of Building***	***Total Height Limit***
1, 2	Buildings with torsional irregularity coefficient satisfying the condition $\eta_{bi} \leq 2.0$ at every storey	$H_N \leq 25$ m
1, 2	Buildings with torsional irregularity coefficient satisfying the condition $\eta_{bi} \leq 2.0$ at every storey and at the same time without type **B2** irregularity	$H_N \leq 40$ m
3, 4	All buildings	$H_N \leq 40$ m

Equivalent Seismic Load Method: Total base shear calculated by means of Equivalent Seismic Load Method, V_t, acting on the entire building in the earthquake direction considered shall be determined by Eqn. 3.19.

$$V_t = WA(T_1) / R_a(T_1) \geq 0.10A_0IW \qquad (3.19)$$

The fist natural vibration period of the building, T_1, shall not be greater than the value calculated by Eqn. 3.20.

$$T_1 = 2\pi\left[\sum_{i=1}^{N}(m_i d_n^2) / \sum_{i=1}^{N}(F_{fi} d_{fi})\right]^{1/2} \qquad (3.20)$$

Regardless of value calculated by Eqn. 3.20, natural period shall not be taken longer than 0.1 in buildings with N > 13 excluding basements.

Total weight of the building, W, to be used in Eqn. 3.19 as the seismic weight shall be determined by Eqn. 3.21.

$$W = \sum_{i=1}^{N} w_i \tag{3.21}$$

Storey weights w_i of Eqn. 3.21 shall be calculated by Eqn. 3.22.

$$w_i = g_i + n\, q_i \tag{3.22}$$

Determination of Design Seismic Loads Acting at Storey Levels: Total equivalent seismic load determined by Eqn. 3.19 is expressed by Eqn. 3.23 as the sum of equivalent seismic loads acting at storey levels. (Figure 3.2(a))

$$V_t = \Delta F_n + \sum_{i=1}^{N} F_i \tag{3.23}$$

The value of additional equivalent seismic load, ΔF_n, acting at the N'th storey (roof) of the building shall be determined by Eqn. 3.24.

$$\Delta F_n = 0.0075\, N\, V_t \tag{3.24}$$

Excluding ΔF_n , the remaining part of the total equivalent seismic load shall be distributed to stories of the building (including N'th storey) in accordance with Eqn. 3.25.

$$F_i = (V_t - \Delta F_N)\, \frac{w_i H_i}{\sum_{i=1}^{N} w_j H_j} \tag{3.25}$$

In buildings with reinforced concrete peripheral walls at their basement being very rigid relative to upper stories and basement floors behaving as rigid diaphragms in horizontal planes, equivalent seismic loads acting on the basement stories and on the upper stories shall be calculated independently as in the following. These loads shall be applied together to the combined structural system. In determining the total equivalent seismic load and equivalent storey seismic loads, appropriate R factors shall be selected without considering the rigid peripheral basement walls and seismic weights of the upper stories only shall be taken into account. In this case, foundation top level considered in the relevant definitions and expressions shall be replaced by the ground floor level. Fictitious loads used for the calculation of the first natural vibration period shall also be based on seismic weights of the upper stories only (Figure 3.2(b)). In calculating equivalent seismic loads acting on rigid basement stories, seismic weights of basements only shall be taken into account and calculation shall be independent of upper stories. For such parts of the building, spectrum coefficient shall be taken as S(T) = 1 without calculating the natural vibration period. In determining equivalent seismic loads acting on each basement storey, spectral acceleration shall be multiplied directly with the respective weight of the storey and resulting elastic loads shall be reduced by dividing them to $R_a(T) = 1.5$ (Figure 3.2(c)). In-plane strength of ground floor system, which is surrounded by very stiff basement walls and located in the transition zone between upper stories, shall be checked according to the internal forces obtained from the analysis. [2]

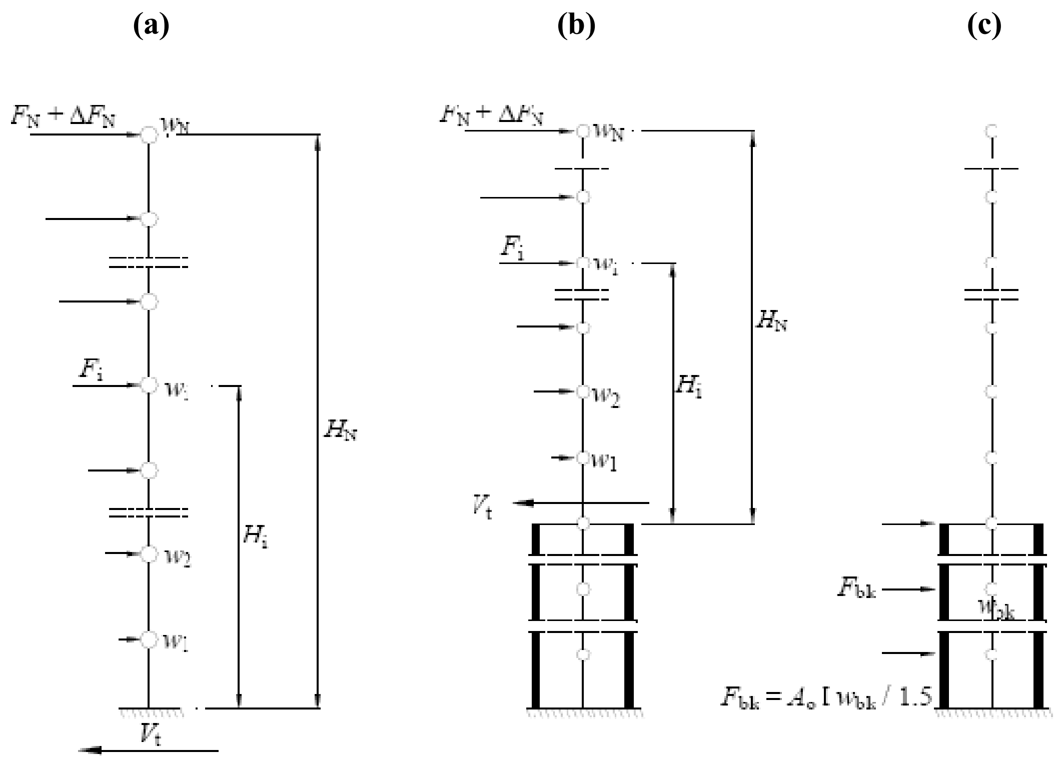


Figure 3.2. Design Seismic Loads Acting at Storey Levels

Displacement Components to be Considered and Application Points of Seismic Loads: In buildings where floors behave as rigid horizontal diaphragms, two lateral displacement components and the rotation around the vertical axis shall be taken into account at each floor as independent static displacement components. At each floor, equivalent seismic loads shall be applied to the floor mass center as well as to the points defined by shifting ±5% of the floor length in the perpendicular direction to the earthquake direction considered in order to account for the additional eccentricity effects (Figure 3.3).

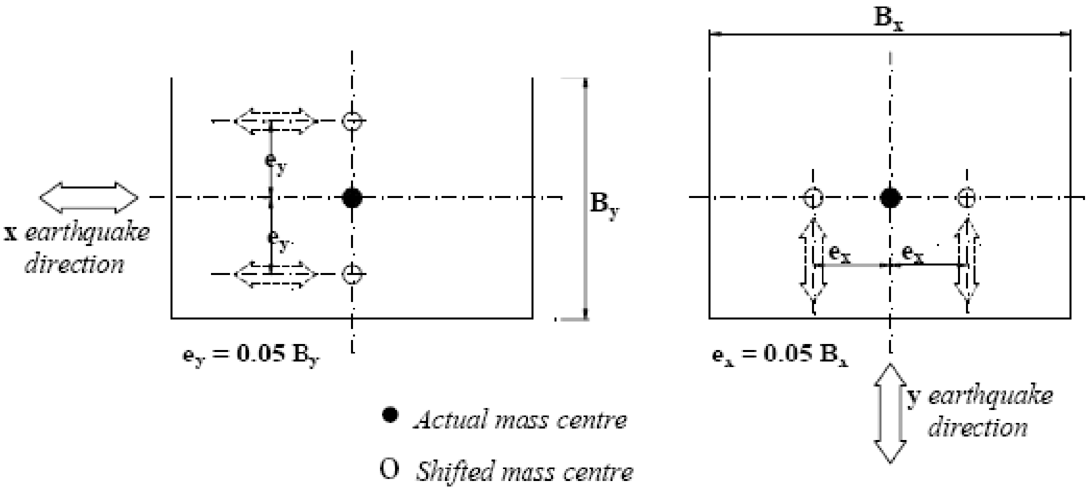


Figure 3.3. Accidental Eccentricity

In buildings where type A2 irregularity exists and floors do not behave as rigid horizontal diaphragms, sufficient number of independent static displacement components shall be considered to account for the in-plane deformation of floors. In order to consider additional eccentricity effects, each of the seismic loads acting on the individual masses distributed over each floor shall be shifted by ±5% of the floor length in perpendicular direction to the earthquake direction considered (Figure 3.4). [2]

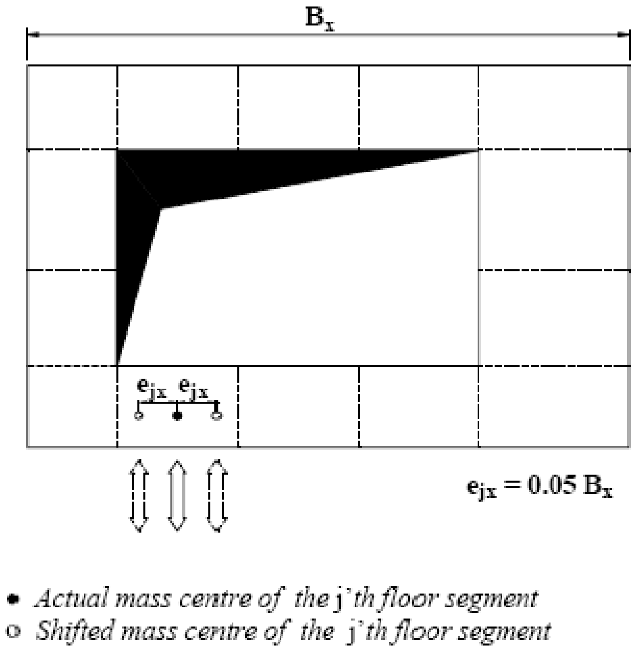


Figure 3.4. Accidental Eccentricity with A2 irregularity

In the case where type A1 irregularity exists at any i'th storey such that the condition $1.2 \leq \eta_{bi} \leq 2.0$ is satisfied, ±5% additional eccentricity applied to this floor shall be amplified by multiplying with coefficient D_i given by Eqn. 3.26 for both earthquake directions.

$$D_i = \left(\frac{\eta_{bi}}{1,2}\right)^2 \qquad (3.26)$$

Internal Forces in Element Principal Axes: Under the combined effects of independently acting x and y direction earthquakes to the structural system, internal forces in element principal axes a and b shall be obtained by Eqn. 3.27 and Eqn. 3.28 such that the most unfavorable results yield (Figure 3.5).

$$B_a = \pm B_{ax} \pm 0.30\, B_{ay} \text{ or } B_a = \pm 0.30\, B_{ax} \pm B_{ay} \qquad (3.27)$$

$$B_b = \pm B_{bx} \pm 0.30\, B_{by} \text{ or } B_b = \pm 0.30\, B_{bx} \pm B_{by} \qquad (3.28)$$

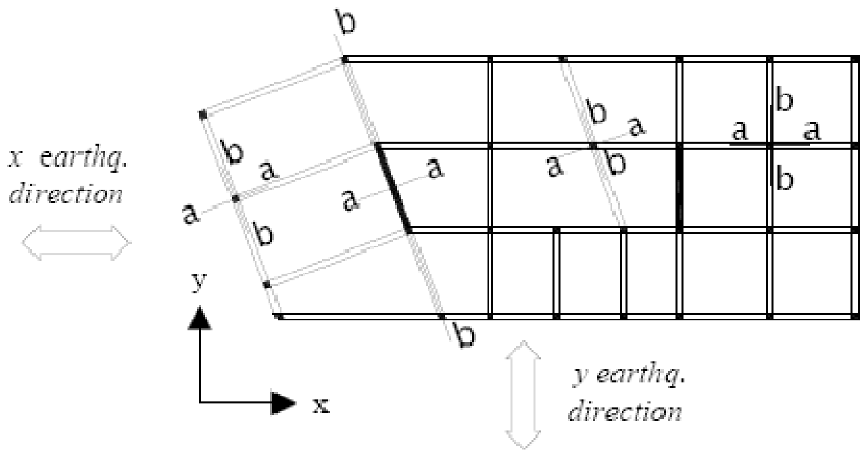


Figure 3.5. Combined Effects of Independently Acting x and y Direction Earthquakes

Mode Combination Method: In this method, maximum internal forces and displacements are determined by the statistical combination of maximum contributions obtained from each of the sufficient number of natural vibration modes considered.

Acceleration Spectrum: Reduced acceleration spectrum ordinate to be taken into account in any n'th vibration mode shall be determined by Eqn. 3.29

$$S_{aR}(T_n) = S_{ae}(T_n) / R_a(T_n) \qquad (3.29)$$

In the case where elastic design acceleration spectrum is determined through special investigations, relevant spectrum ordinate shall be considered in the above Eq. in lieu of $S_{ae}(T_n)$.

Dynamic Degrees of Freedom to be Considered: In buildings where floors behave as rigid horizontal diaphragms, two horizontal degrees of freedom in perpendicular directions and a rotational degree of freedom with respect to the vertical axis passing through mass center shall be considered at each storey. At each floor, modal seismic loads shall be determined for those degrees of freedom and shall be applied to the floor mass center as well as to the points defined by shifting it ±5% of the floor length in the perpendicular direction to the earthquake direction considered in order to account for the additional eccentricity effects (Figure 3.3). In buildings where type A2 irregularity exists and floor do not behave as rigid horizontal diaphragms, sufficient number of dynamic degrees of freedom shall be considered to account for the in-plane deformation of floors. In order to consider additional eccentricity effects, each of the modal seismic loads acting on the individual masses distributed over each floor shall be shifted by ±5% of the floor length in perpendicular direction to the earthquake direction considered (Figure 3.4). In such buildings, internal force and displacement quantities due to the additional eccentricity effects alone may also be calculated in accordance with Equivalent Seismic Load Method. Such quantities shall be directly added those combined in accordance with modal combination without taking into account additional eccentricity effects.

Sufficient Number of Vibration Modes to be Considered: Sufficient number of vibration modes, Y, to be taken into account in the analysis shall be determined to the criterion that the sum of effective participating masses calculated for each mode in each of the given x and y perpendicular lateral earthquake directions shall in no case be less than 90% of the total building mass.

$$\sum_{n=1}^{Y} M_{xn} = \sum_{n=1}^{Y} \frac{L_{xn}^2}{M_n} \geq 0.90 \sum_{i=1}^{N} m_i \qquad (3.30)$$

$$\sum_{n=1}^{Y} M_{yn} = \sum_{n=1}^{Y} \frac{L_{yn}^2}{M_n} \geq 0.90 \sum_{i=1}^{N} m_i \qquad (3.31)$$

The expressions of L_{xn}, L_{yn} and modal mass M_n shown in Eqn. 3.30 and Eqn. 3.31 are given below for buildings with rigid floor diaphragms.

$$L_{xn} = \sum_{i=1}^{N} m_i \varphi_{xin} \qquad (3.32)$$

$$L_{yn} = \sum_{i=1}^{N} m_i \varphi_{yin} \qquad (3.33)$$

$$M_n = \sum_{i=1}^{N} (m_i \varphi_{xin}^2 + m_i \varphi_{yin}^2 + m_{\theta i} \varphi_{\theta in}^2) \qquad (3.34)$$

In buildings with reinforced concrete peripheral walls at their basement being very rigid relative to upper stories and basement floor behaving as rigid diaphragms in horizontal planes, it may be sufficed with the consideration of vibration modes which are effective in the upper stories only. In this case, in the analysis performed by the Mode

Combination Method which corresponds to the analysis by Equivalent Seismic Load method, the coefficient R shall be selected without considering the rigid peripheral basement walls whereas the upper storey masses only shall be taken into account.

Combination of Modal Masses: Rules to be applied for the statistical combination of non-simultaneous maximum contributions of response quantities calculated for each vibration mode, such as the base shear, storey shear, internal force components, displacements and storey drifts, are specified in the following provided that they are applied independently for each response quantity:

In the case where natural periods of any two vibration mode with $T_m < T_n$ always satisfy the condition $T_m / T_n < 0.80$, Square Root of Sum of Squares (SRSS) Rule may be applied for the combination of maximum modal contributions.

In the case where the above given condition is not satisfied, Complete Quadratic Combination (CQC) Rule shall be applied for the combination of maximum modal contributions. In the calculation of cross correlation coefficients to be used in the application of the rule, modal damping factors shall be taken as 5% for all modes.

In the case where the ratio of the base shear in the given earthquake direction, V_{tB}, which is obtained through modal combination, to the base shear V_t, obtained by Equivalent Seismic Load Method is less than the below given value of β ($V_{tB} < \beta V_t$), all internal force and displacement quantities determined by Mode Combination Method shall be amplified in accordance with Eqn. 3.35. [2]

$$B_D = \frac{\beta V_t}{V_{tB}} B_B \tag{3.35}$$

B_B : Any response quantity obtained by modal combination in the Mode Superposition Method

B_D : Amplified value of B_B

If at least one of the irregularities of type A1, B2 or B3 exists in a building $\beta = 0.90$, whereas none of them exists $\beta = 0.80$ shall be used in the Eqn. 3.35.

Internal Forces in Element Principal Axes: Under the combined effects of independently acting x and y direction earthquakes to the structural system, the directional combination rule given in the Equivalent Seismic Load Method shall be additionally applied to the internal forces obtained in element principal axes by modal combination.

3.3.3. Serviceability Conditions

3.3.3.1 Limitation of Displacements. If linear analysis performed the displacements induced by the design seismic action shall be calculated on the basis of the elastic deformations of the structural system by means of the following simplified expression.

$$d_s = q_d \, d_e \qquad (3.36)$$

d_s : The displacement of a point of the structural system induced by the design seismic action.

q_d : The displacement behaviour factor, assumed equal to the q unless otherwise specified.

d_e : The displacement of the same point of the structural system, as determined by a linear analysis based on the design response spectrum.

The value of d_s does not need to be larger than the value derived from the elastic spectrum. When determining the displacements d_e, the torsional effects of the seismic action shall be taken into account.

The following limits of interstorey drifts shall be observed as stated below:

- For buildings having non-structural elements of brittle materials attached to the structure.

$$d_r \nu \leq 0{,}005h$$

- For buildings having ductile non-structural elements.

$$d_r\nu \leq 0{,}0075h$$

- For buildings having non-structural elements fixed in a way so as not interfere with structural deformations, or without non-structural elements.

$$d_r\nu \leq 0{,}010\ h$$

d_r : The design interstorey drifts, evaluated as the difference of the average lateral displacements d_s at the top and bottom of the storey under consideration.
h : The storey height
ν : The reduction factor which takes into account the lower period of the seismic action associated with the damage limitation requirement.

The value of the reduction factor ν may also depend on the importance class of the building. Implicit in its use is the assumption that the elastic response spectrum of the seismic action under which the "demage limitation requirement" should be met. [1]

Calculation and Limitation of Effective Storey Drifts According to TEC2007: The reduced storey drifts Δ_i, of any column or structural wall shall be determined by the following Eq. as the difference of displacements between the two consecutive stories.

$$\Delta_i = d_i - d_{i-1} \qquad (3.37)$$

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

d_i and d_{i-1} represent lateral displacements obtained from the analysis at the ends of any column or structural wall at stories i and i-1 under reduced seismic loads. Effective storey drift, δ_i, of columns or structural walls at the i'th storey of a building shall be obtained for each earthquake direction by Eqn. 3.38.

$$\delta_i = R\ \Delta_i \qquad (3.38)$$

The maximum value of effective storey drifts, $(\delta_i)_{max}$, obtained for each earthquake direction at columns or structural walls of a given i’th storey of a building shall be satisfy the condition given by Eqn. 3.39.

$$\frac{\delta_{imax}}{h_i} \leq 0.02 \qquad (3.39)$$

The limits may be exceeded by 50% in single storey frames where seismic loads are fully resisted by steel frames with joints capable of transferring cyclic moments. In the case where the condition given by Eqn. 3.39 is not satisfied at any storey of the building, the seismic analysis shall be repeated with increased stiffness of the structural system. However, even if the condition is satisfied, serviceability of nonstructural brittle elements (e.g. facade elements) under effective storey drifts shall be verified by calculation. [2]

3.3.3.2 Second Order Effects. Second order effects (P-Δ effects) according t EC8 need not be taken into account if the following condition is fulfilled in all storeys.

$$\theta = \frac{P_{tot} d_r}{V_{tot} h} \leq 0.10 \qquad (3.40)$$

θ : The Interstorey drift sensitivity coefficient

P_{tot} : The total gravity load at and above the storey considered in the seismic design situation.

d_r : The design Interstorey drift, evaluated as the difference of the average lateral displacements d_s at the top and bottom of the storey under consideration.

V_{tot} : The total seismic storey shear

h : The Interstorey height

If $0.1 < \theta \leq 0.2$, the second order effects may approximately be taken into account by multiplying the relevant seismic action effects by a factor equal to $1/(1-\theta)$. The value of the coefficient θ shall not exceed 0.3. [1]

Second Order Effects According to TEC 2007: Unless a more refined analysis considering the nonlinear behaviour of structural system is performed, second-order effects may be taken into account.

In the case where Second Order Effect indicator, θ_i, satisfies the condition given by Eqn. 3.41 for the earthquake direction considered at each storey, second order effects shall be evaluated in accordance with the currently enforced specifications of reinforced concrete or structural steel design.

$$\theta_i = \frac{\Delta_{i\,ort} \sum_{j=i}^{N} w_j}{V_i h_i} \leq 0.12 \quad (3.41)$$

$(\Delta_i)_{ort}$ shall be determined as the average value of reduced storey drifts calculated for i'th storey columns and structural walls. In the case where the given condition is not satisfied, seismic analysis shall be repeated with sufficiently increased stiffness of the structural system. [2]

3.3.3.3 Seismic Joints. Buildings shall be protected from earthquake induced pounding from adjacent structures or between structurally independent units of the same building. For buildings, or structurally independent units that do not belong to the same property, the distance from the property line to the potential points of impacts is not less than the maximum horizontal displacement of the building at the corresponding level. And also, for buildings, or structurally independent units, belonging to the same property, the distance between them is not less than the square root of the sum of the squares (SRSS) of the maximum horizontal displacements of the two buildings or units at the corresponding level. If the floor elevations of the building or independent units under design are the same as those of the adjacent building or unit, the above referred minimum distance may be reduced by a factor of 0.7. [1]

Seismic Joint Conditions According to TEC2007: Excluding the effects of differential settlements and rotations of foundations and the effects of temperature change, sizes of gaps to be retained in the seismic joints between building blocks or between the old and newly constructed buildings shall be determined in accordance with the following conditions:

Unless a larger value is obtained, sizes of gaps shall not be less than the square root of sum of average storey displacements multiplied by the coefficient α specified below. Storey displacements to be considered are the average values of reduced displacements d_i calculated within a storey at the column or structural wall joints. In the cases where the seismic analysis is not performed for the existing old buildings, the storey displacements shall not be assumed to be less than those obtained for the new building at the same stories.

- α = R/4 shall be taken if all floor levels of adjacent buildings or building blocks are the same.
- α = R/2 shall be taken if any of the floor levels of adjacent buildings or building blocks are not the same.

Minimum size of gaps shall be 30 mm up to 6 m height. From thereon a minimum 10 mm shall be added for every 3 m height increment. Seismic joint shall be arranged to allow the independent movement of building blocks in all earthquake directions. [2]

4. SPECIFIC RULES AND GUIDELINES FOR STEEL BUILDINGS

4.1. Rules According to EC8

4.1.1. Design Concept

Earthquake resistant steel buildings shall be designed in accordance with one of the following concept.

- Low-dissipative structural behaviour
- Dissipative structural behavior

The action effects may be calculated on the basis of an elastic global analysis without taking into account a significant non-linear behaviour for low-dissipative structures. When using the design spectrum, the upper limit of the reference value of the behaviour factor q may be taken between 1.5 and 2. The capability of parts of the structure to resist earthquake actions through inelastic behaviour is taken into account for dissipative structures. Structures designed in accordance with dissipative structural behaviour shall belong to structural classes medium or high (DCM or DCH). These classes correspond to increased ability of the structure to dissipate energy in plastic mechanisms. Depending on the ductility class, specific requirements such as class of steel sections, rotational capacity of connections shall be met.

4.1.2. Material Conditions and Material Safety Factors

Structural steel shall conform to standards referred to in EC3. The distribution of material properties, such as yield strength and toughness, in the structure shall be such that dissipative zones form where they are intended to in the design. Dissipative zones are expected to yield before other zones leave elastic range during the earthquake. In bolted connections of primary seismic members of a building, high strength bolts of bolt grade 8.8 or 10.9 should be used. In the capacity design checks of dissipative structures, the possibility that the actual yield strength of steel is higher than the nominal yield strength should be taken into account by a material overstrength factor γ_{ov}.

4.1.3. Classification of Steel Structural Systems

Steel buildings shall be assigned to one of the following types according to the behaviour of their primary resisting structure under seismic actions. [1]

- Moment resisting frames: The horizontal forces are mainly resisted by members acting in an essentially flexural manner. The dissipative zones should be mainly located in plastic hinges in the beams or the beam column joints so that energy is dissipated by means of cyclic bending. The dissipative zones may also be located at the base of the columns, at the top of the columns in the upper storey of multi-storey buildings, or at the top and bottom of columns in single storey buildings in which N_{Ed} in columns conform to the inequality, $N_{Ed} / N_{pl,Rd} < 0.3$ (Figure 4.1).

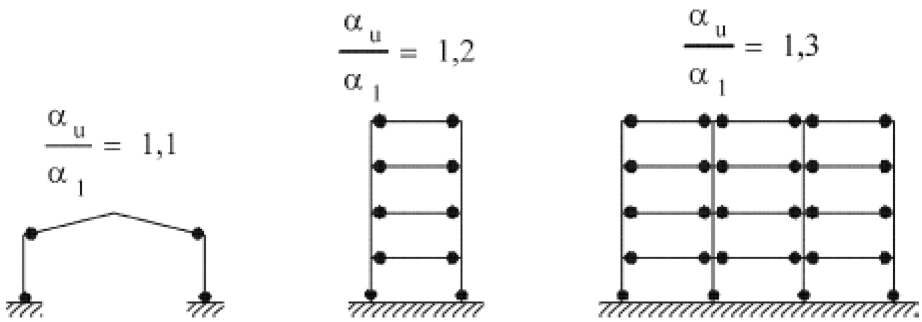
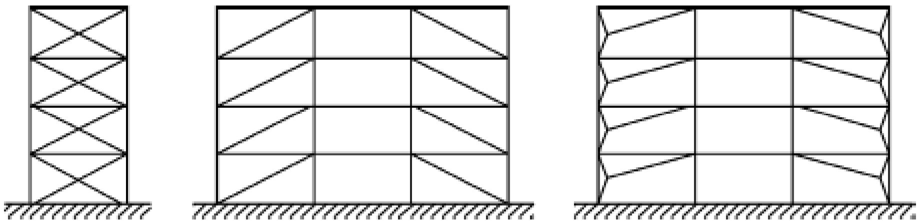
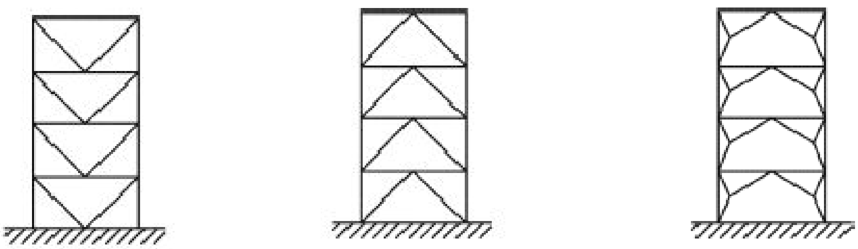


Figure 4.1. Moment Resisting Frames with Default Values of α_u/α_1

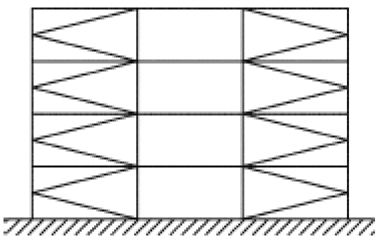
- Frames with concentric bracings: The horizontal forces are mainly resisted by members subjected to axial forces (Figure 4.2(a)). The dissipative zones should be mainly located in the tensile diagonals. The bracing may be active tension diagonal bracing, in which the horizontal forces can be resisted by the tension diagonals only, neglecting the compression diagonals. The bracing may be in V shape, in which the horizontal forces can be resisted by taking into account both tension and compression diagonals (Figure 4.2(b)). The intersection point of these diagonals lies on a horizontal member which shall be continuous. K shape bracing, in which the intersection of the diagonals lies on a column, may not be allowed (Figure 4.2(c)).



(a)



(b)



(c)

Figure 4.2. Frames with Concentric Bracings

- Frames with eccentric bracings: The horizontal forces are mainly resisted by axially loaded members, but where the eccentricity of the layout is such that energy can be dissipated in seismic links by means of either cyclic bending or cyclic shear. Bracing configurations should be used that ensure that all links will be active (Figure 4.3).

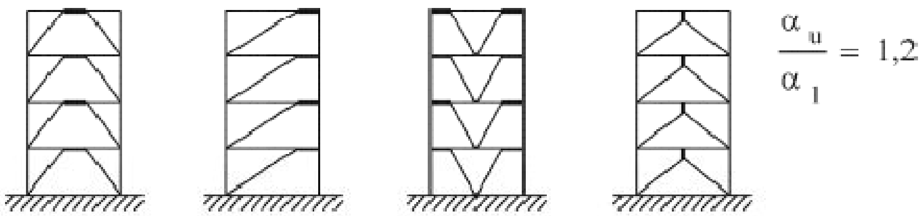


Figure 4.3 Frames with Eccentric Bracings with Default Values of α_u/α_1

- Inverted pendulum structures: Dissipative zones are located at the bases of columns (Figure 4.4).

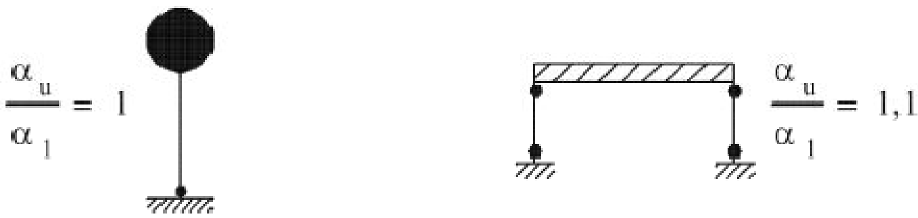


Figure 4.4. Inverted Pendulum with Default Values of α_u/α_1

- Structures with concrete cores or concrete walls: The horizontal forces are mainly resisted by these cores or walls (Figure 4.5).

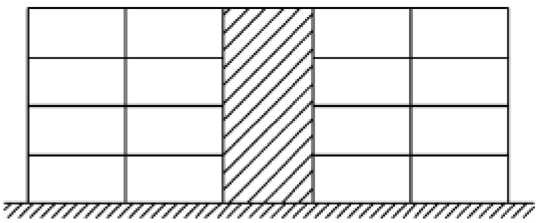


Figure 4.5. Structures with Concrete Cores or Concrete Walls

- Moment resisting frames combined with concentric bracings (Figure 4.6).

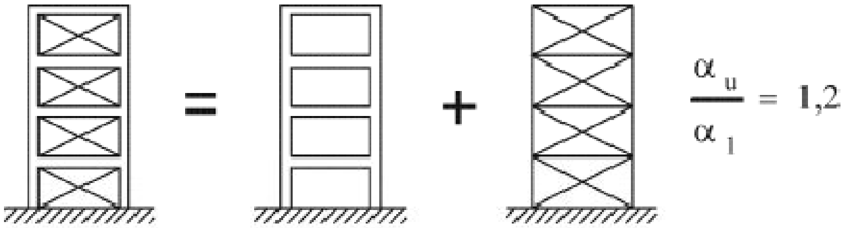


Figure 4.6. Moment Resisting Frames Combined with Concentric Bracings with Default Values of α_u/α_1

- Moment resisting frames combined with infills (Figure 4.7).

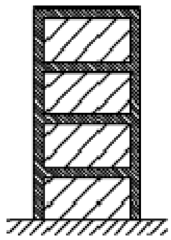


Figure 4.7. Moment Resisting Frames Combined with Infills

Behaviour Factor: The behaviour factor q, accounts for the energy dissipation capacity of the structure. For regular structural systems, the behaviour factor q should be taken with upper limits to the reference values which are given in Table 4.1. [1]

Table 4.1. Upper Limit of reference values of behaviour factors for systems regular in elevation

<table>
<tr><th rowspan="2">STRUCTURAL TYPE</th><th colspan="2">Ductility Class</th></tr>
<tr><th>DCM</th><th>DCH</th></tr>
<tr><td>a) Moment resisting frames</td><td>4</td><td>$5\alpha_u/\alpha_1$</td></tr>
<tr><td>b) Frame with concentric bracings
Diagonal bracings
V-bracings</td><td>
4
2</td><td>
4
2.5</td></tr>
<tr><td>c) Frame with eccentric bracings</td><td>4</td><td>$5\alpha_u/\alpha_1$</td></tr>
<tr><td>d) Inverted pendulum</td><td>2</td><td>$2\alpha_u/\alpha_1$</td></tr>
<tr><td>e) Structures with concrete cores or concrete walls</td><td colspan="2">See section 5</td></tr>
<tr><td>f) Moment resisting frame with concentric bracing</td><td>4</td><td>$4\alpha_u/\alpha_1$</td></tr>
<tr><td>g) Moment resisting frames with infills
Unconnected concrete or masonry infills, in contact with the frame</td><td>2</td><td>2</td></tr>
<tr><td>Connected reinforced concrete infills</td><td colspan="2">See section 7</td></tr>
<tr><td>Infills isolated from moment frame (see moment frames)</td><td>4</td><td>$5\alpha_u/\alpha_1$</td></tr>
</table>

If the building is non –regular in elevation, the upper limit values of q listed in Table 4.1 should be reduced by 20%. For the buildings that are regular in plan, if the calculations to evaluate α_u/α_1, are no performed, the approximate default values of the ratio α_u/α_1 may be used.

α_1 : The value by which the horizontal seismic design action is multiplied in order to first reach the plastic hinges in a number of sections sufficient for the development of overall structural instability, while all other design actions remain constant.

α_u : The value by which the horizontal seismic design action is multiplied, in order to form plastic hinges in a number of sections sufficient for the development of overall structural instability, while all other design actions remain constant. The factor α_u may be obtained from a nonlinear static (pushover) global analysis.

For the buildings which are not regular in plan, the approximate value of α_u/α_1 that may be used when calculations are not performed for its evaluation are equal to the average of 1.0 and the given α_u/α_1 values in the above figures. The higher values of α_u/α_1 can be allowed if they are confirmed by calculation of α_u/α_1 with a nonlinear static global analysis. The maximum value of α_u/α_1 that may be used in a design is equal to 1.6, even if the nonlinear analysis indicates higher potential values.

4.1.4. Design Criteria & Detailing Rules for Dissipative Structural Behaviour Common to All Structural Types

The earthquake resistant parts of structures shall be designed in accordance with the concept of dissipative structural behaviour and the detailing rules given in following chapters should be satisfied.

Structures with dissipative zones shall be designed such that yielding or local buckling or other phenomena due to hysteric behaviour do not affect the overall stability of the structure. Dissipative zones shall have adequate ductility and resistance. The resistance shall be verified in accordance with EC3. Dissipative zones may be located in the structural members or in the connections. If dissipative zones are located in the structural members, the non-dissipative parts and the connections of the dissipative parts to the rest of the structure shall have sufficient overstrength to allow the development of cycling yielding in the dissipative parts. When dissipative zones are located in the connections, the connected members shall have sufficient overstrength to allow the development of cycling yielding in the connections.

Dissipative Elements in Compression or Bending: Sufficient local ductility of members which dissipate energy in compression or bending shall be ensured by restricting the width – thickness ratio b/t according to the cross-sectional classes specified in EC3, chapter 1-1. Depending on the ductility classes and the behavior factor q used in the design, the requirements regarding the cross sectional classes of the steel elements which dissipate energy are indicated in Table 4.2.

Table 4.2. Reference Behaviour Factor Depending on Ductility Class

Ductility class	Reference value of behaviour 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

Connections in Dissipative Zones: The design of connections shall be such as to limit localization of plastic strains, high residual stresses and prevent fabrication defects. Non dissipative connections of dissipative members made by means of full penetration butt welds may be deemed to satisfy the overstrength criterion. For fillet weld or bolted non dissipative connections, the following expression should be satisfied.

$$R_d \geq 1.1\gamma_{ov}R_{fy} \qquad (4.1)$$

R_d : The resistance of the connection

R_{fy} : The plastic resistance of the connected dissipative member based on the design yield stress of the material

γ_{ov} : The overstrength factor

Categories B and C bolted joints in shear in accordance with EC3, chapter 1-8 and category E of bolted joints in tension in accordance with EC3, 1-8 should be used. Shear joints with fitted bolts are also allowed. Friction surfaces should belong to class A or B as defined ENV 1090-1. For bolted shear connections, the design shear resistance of the bolts should be higher than 1.2 times the design bearing resistance. The adequacy of design should be supported by experimental evidence whereby strength and ductility of members and their connections under cycling loading should be supported by experimental evidence, in order to conform to the specific requirements.

4.1.5. Design and Detailing Rules for Moment resisting Frames

Design Criteria: Moment resisting frames shall be designed so that plastic hinges form in the beams or in the connections of the beams to the columns, but not in the columns. This requirement is waived at the base of the frame, at the top level of multi storey buildings and for single storey buildings.

Beams: Beams should be verified as having sufficient resistant against lateral and lateral torsional buckling in accordance with EC3, assuming the formation of a plastic hinge at one end of the beam. The beam end that should be considered is the most stressed end in the seismic design situation. For plastic hinges in the beams it should be verified that full plastic moment of resistance and rotation capacity are no decreased by compression and shear forces. To this end, for sections belonging to cross sectional classes 1 and 2, the following inequalities should be verified at the location where the formation of hinges is expected.

$$\frac{M_{Ed}}{M_{pl,Rd}} \leq 1.0 \qquad (4.2)$$

$$\frac{N_{Ed}}{N_{pl,Rd}} \leq 0.15 \qquad (4.3)$$

$$\frac{V_{Ed}}{V_{pl,Rd}} \leq 0.5 \qquad (4.4)$$

Where

$$V_{Ed} = V_{Ed,G} + V_{Ed,M} \qquad (4.5)$$

N_{Ed} : The design axial force

M_{Ed} : The design bending moment

V_{Ed} : The design shear

$N_{pl,Rd,A}$, $M_{pl,Rd}$, $V_{pl,Rd}$: Design resistance in accordance with EC3

$V_{Ed,G}$: The design value of the shear force due to the non seismic actions

$V_{Ed,M}$: The design value of the shear force due to the application of plastic moments $M_{pl,Rd,A}$ and $M_{pl,Rd,B}$ with opposite signs at the end sections A and B of the beam.

$$V_{Ed,M} = (M_{pl,Rd,A} + M_{pl,Rd,B}) / L \tag{4.6}$$

For sections belonging to cross sectional class 3, expressions stated above should be checked replacing $N_{pl,Rd,A}$, $M_{pl,Rd}$, $V_{pl,Rd}$ with $N_{el,Rd,A}$, $M_{el,Rd}$, $V_{el,Rd.}$

Columns: The columns shall be verified in compression considering the most unfavorable combination of the axial force and bending moments. In the checks, N_{Ed}, M_{Ed}, V_{Ed} should be computed as:

$$N_{Ed} = N_{Ed,G} + 1.1\gamma_{ov}\Omega N_{Ed,E} \tag{4.7}$$

$$M_{Ed} = M_{Ed,G} + 1.1\gamma_{ov}\Omega M_{Ed,E} \tag{4.8}$$

$$V_{Ed} = V_{Ed,G} + 1.1\gamma_{ov}\Omega V_{Ed,E} \tag{4.9}$$

$N_{Ed,G}$, $M_{Ed,G}$, $V_{Ed,G}$ are respectively compression force, bending moment and shear force in the column due to the non-seismic actions included in the combination of actions for the seismic design situation.

N_{Ed}, M_{Ed}, V_{Ed} are respectively compression force, bending moment and shear force in the column due to the design seismic action.

γ_{ov} is the overstrength factor.

Ω is the minimum value of $\Omega_i = M_{pl,Rd,i} / M_{Ed,i}$ of all beams in which dissipative zones are located. $M_{Ed,i}$ is the design value of the bending moment in beam *i* in the seismic design situation and $M_{pl,Rd,i}$ is the corresponding plastic moment.

The column shear force V_{Ed} resulting from the structural analysis should satisfy the following equation.

$$\frac{V_{Ed}}{V_{pl,Rd}} \leq 0.5 \tag{4.10}$$

The transfer of the forces from the beams to the columns should conform to the design rules given in EC3.

The shear resistance of framed web panels of beam-column connections should satisfy the following equation (Figure 4.8).

$$\frac{V_{wp,Ed}}{V_{wp,Rd}} \leq 1.0 \qquad (4.11)$$

$V_{wp,Ed}$: The design shear force in the web panel due to the action effects, taking into accounts the plastic resistance of the adjacent zones in bemas or connections.
$V_{wp,Rd}$: The shear resistance of the web panel in accordance with EC3, chapter 1-8.

It is not required to take into account the effect of the stresses of the axial force and bending moment on the plastic resistance in shear.

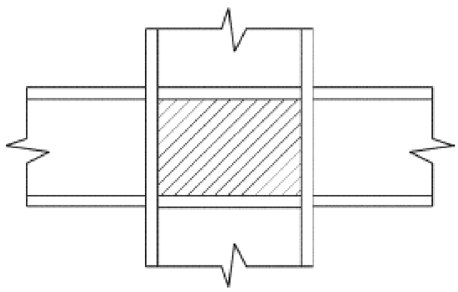


Figure 4.8. Web Panel Framed by Flanges and Stiffener

The shear buckling resistance of the web panels should also be checked to ensure that it conforms to the following expression.

$$V_{wp,Ed} < V_{wb,Rd} \qquad (4.12)$$

$V_{wb,Rd}$: The shear buckling resistance of the web panel.

Beam to Column Connection: If the structure is designed to dissipate energy in the beams, the connections of the beams to the columns should be designed for the required degree of overstrength taking into account the moment of resistance $M_{pl,Rd}$ and the shear force ($V_{Ed,G}$ + $V_{Ed,M}$). Dissipative semi rigid and partial strength connections are permitted, provided that all of the following requirements are verified.

- The connections have a rotation capacity consistent with the global deformations.
- Members framing into the connections are demonstrated to be stable at the ultimate limit state.
- The effect of connection deformation on the global drift is taken into account using singular nonlinear static (pushover) global analysis or nonlinear time history analysis.

The connection design should be such that the rotation capacity of the plastic hinge region θ_p is not less than 35 mrad for structures of ductility class DCH and 25 mrad for structures of ductility class DCM with $q > 2$. The rotation θ_p is defined as the following expression.

$$\theta_p = \delta / 0.5L \qquad (4.13)$$

δ : The beam deflection at midspan

L : The beam span

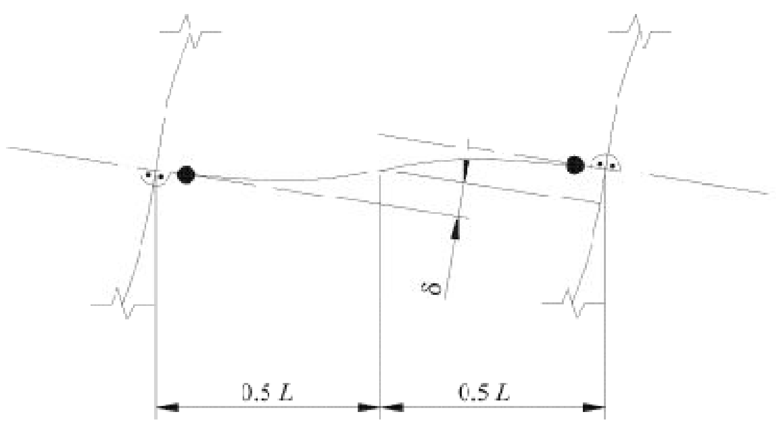


Figure 4.9. Beam Deflection for the Calculation of θ_p

The rotation capacity of the plastic hinge region θ_p should be ensured under cyclic loading without degradation of strength and stiffness greater than 20%. This requirement is valid independently of the intended location of the dissipative zones.

In experiments made to asses θ_p the column web panel shear resistance should conform to Eqn. 4.13 and the column web panel shear deformation should not contribute for more than 30% of the plastic rotation capability θ_p. The column elastic deformation should not be included in the evaluation of θ_p. When partial strength connections are used, the column capacity design should be derived from the plastic capacity of the connections.

4.1.6. Design and Detailing Rules for Frames with Concentric Bracing

Design Criteria: Concentric braced frames shall be designed so that yielding of the diagonals in tension will take place before failure of the connections and before yielding or buckling of the beams or columns. The diagonal elements of bracing shall be placed in such a way that the structure exhibits similar load deflection characteristics at each storey in opposite senses of the came braced direction under load reversals. To this end, the following rule should be met at every storey.

$$\frac{|A^{+}-A^{-}|}{A^{+}+A^{-}} \leq 0.05 \qquad (4.14)$$

A^+ and A^- are the areas of the horizontal projections of the cross sections of the tension diagonals, when the horizontal seismic actions have appositive or negative direction respectively (Figure 4.10).

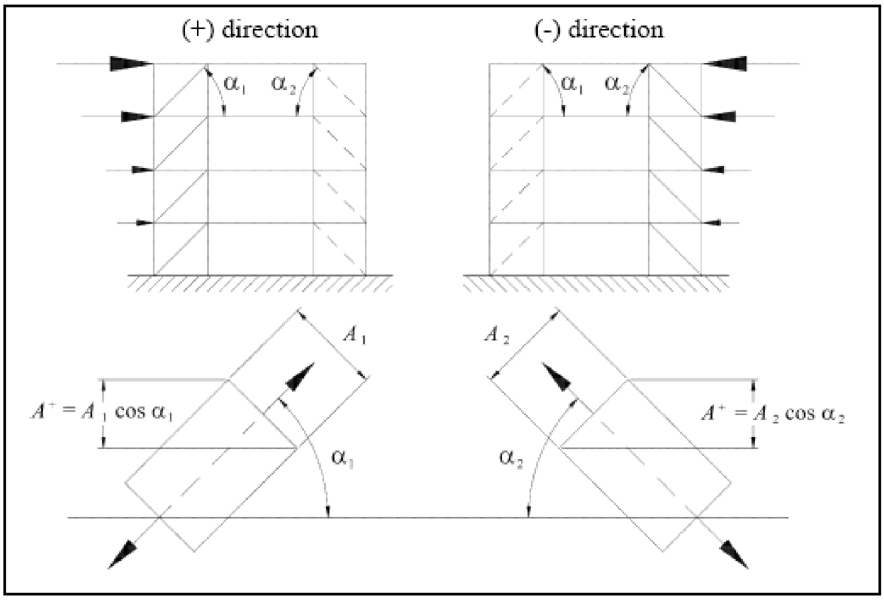


Figure 4.10. Concentric Braced Frames

Upper gravity load conditions, only beams and columns shall be considered to resist such loads, without taking into account the bracing members. The diagonals shall be taken into account as follows in an elastic analysis of the structure for the seismic action.

- In frames with diagonal bracings, only the tension diagonals shall be taken into account
- In frames with V shape bracing, both tension and compression diagonals shall be taken into account.

Taking into account of both tension and compression diagonals in the analysis of any type of concentric bracing is allowed provided that all of the following conditions are satisfied.

- A nonlinear static (pushover) global analysis or nonlinear time history analysis is used.
- Both pre-buckling and post-buckling situations are taken into account in the modeling of the behavior of diagonals.
- Background information justifying the model used to represent the behavior of diagonals is provided.

Diagonal Members: In frames with X diagonal bracings, the non-dimensional slenderness λ as defined in EC3, chapter 1-1, should be limited to $1.3 < \lambda \leq 2.0$. In frames with diagonal bracings in which the diagonals are not positioned as X diagonal bracing, the non-dimensional slenderness λ should be less than or equal to 2.0. In frames with V bracings, the non-dimensional slenderness λ should be less than or equal to 2.0. In structures up to two storeys, no limitation to λ. The yield resistance $N_{pl,Rd}$ of the gross cross section of the diagonals should be such that $N_{pl,Rd} \geq N_{Ed}$. In frames with V bracings, the compression diagonals should be designed for the compression resistance in accordance with EC3. The connections of the diagonals to any member should satisfy the design rules of dissipative structures. In order to satisfy a homogeneous dissipative behavior of the diagonals, it should be checked that the maximum overstrength $\mathbf{\Omega}_i$ does not differ from the minimum value $\mathbf{\Omega}$ by more than 25%. Dissipative semi-rigid and partial strength connections are permitted, provided that all of the following conditions are satisfied.

- The connections have an elongation capacity consistent with global deformations.
- The effect of connections deformation on global drift is taken into account using nonlinear static (pushover) global analysis or non-linear time history analysis.

Beams and Columns: Beams and columns with axial forces should meet the following minimum resistance requirement.

$$N_{pl,Rd}(M_{Ed}) \geq N_{Ed,G} + 1.1\gamma_{ov}\mathbf{\Omega} N_{Ed,E} \tag{4.15}$$

$N_{pl,Rd}(M_{Ed})$ is the design buckling resistance of the beam or the column in accordance with EC3, taking into account the interaction of the buckling resistance with the bending moment M_{Ed}, defined as its design value in the seismic design situation.

$N_{Ed,G}$ is the axial force in the beam or in the column due to the non-seismic actions included in the combination of actions for the seismic design situation.

$N_{Ed,E}$ is the axial force in the beam or in the column due to the design seismic action.

γ_{ov} is the overstrength factor.

$\mathbf{\Omega}$ is the minimum value of $\mathbf{\Omega}_i = N_{pl,Rd,i} / N_{Ed,i}$ over all the diagonals of the braced frame system.

$N_{pl,Rd,i}$ is the design resistance of diagonal *i*.

$N_{Ed,i}$ is the design value of the axial force in the same diagonal *i* in the seismic design situation.

In frames with V bracings, the beams should be designed to resist:

- All non-seismic actions without considering the intermediate support given by the diagonals.
- The unbalanced vertical seismic action effect applied to the beam by the braces after buckling of the compression diagonal. This action effect is calculated using $N_{pl,Rd}$ for the brace in tension and $\gamma_{pb}N_{pl,Rd}$ for the brace in compression.

γ_{pb} is equal to 0.3.

In frames with diagonal bracings in which the tension and compression diagonals are not intersecting, the design should take into account the tensile and compression force which develop in the columns adjacent to the diagonals in compression and correspond to compression forces in these diagonals equal to their design buckling resistance.

4.1.7. Design and Detailing Rules for Frames with Eccentric Bracing

Design Criteria: Frames with eccentric bracing shall be designed so that specific elements or parts of elements called seismic links are able to dissipate energy by the formation of plastic bending and plastic shear mechanism. The structural system shall be designed so that a homogeneous dissipative behavior of the whole set of seismic links is realized. Seismic links may be horizontal or vertical components (Figure 4.3).

Seismic Links: The web of a link should be of single thickness without doubler plate reinforcement and without a hole or penetration. Seismic links are classified into 3 categories according to the type of plastic mechanism developed.

- Short links, which dissipate energy by yielding essentially in shear.
- Long links, which dissipate energy by yielding essentially in bending.
- Intermediate links, in which the plastic mechanism involves bending and shear.

For I sections, the following parameters are used to define the design resistance and limits of categories.

$$M_{p,link} = f_y\, b\, t_f\, (d-t_f) \tag{4.16}$$

$$V_{p,link} = (f_y\, 3^{1/2})\, t_w\, (d-t_f) \tag{4.17}$$

b : The width of flange of I sections
d : The height of th I sections
t_f : The thickness of flange of I sections
t_w : The thickness of web of I sections

If $N_{Ed} / N_{pl,Rd} \leq 0.15$, the design resistance of the link should satisfy both of the following relationships at both ends of the link:

$$V_{Ed} \leq V_{p,link} \tag{4.18}$$

$$M_{Ed} \leq M_{p,link} \tag{4.19}$$

N_{Ed}, M_{Ed}, V_{Ed} are the design action effects, respectively the design axial force, design bending moment and design shear, at both ends of the link.

If $N_{Ed} / N_{pl,Rd} > 0.15$, expressions stated above should be satisfied with the following reduced values $V_{p,link,r}$ and $M_{p,link,r}$ used instead $V_{p,link}$ and $M_{p,link}$.

$$V_{p,link,r} = V_{p,link}\, [1-(N_{Ed} / N_{pl,Rd})^2]^{0.5} \tag{4.20}$$

$$M_{p,link,r} = M_{p,link}\, [1-(N_{Ed} / N_{pl,Rd})] \tag{4.21}$$

If $N_{Ed} / N_{pl,Rd} > 0.15$, the link length e should not exceed:

$$e \leq 1.6\ M_{p,link} / V_{p,link} \text{ when } R < 0.3 \quad \text{or}$$

$$e \leq (1.15\text{-}0.5R)\ 1.6\ M_{p,link} / V_{p,link} \text{ when } R \geq 0.3$$

where
$R = N_{Ed}\ t_w\ (d\text{-}2t_f) / (V_{Ed}\ A)$ in which A is the gross area of the link.

To achieve a global dissipate behavior of the structure, it should be checked that the individual values of the ratios Ω_i do not exceed the minimum value Ω by more than 25% of this minimum value. In designs where equal moments would form simultaneously at both ends of the link, links may be classified according to the length e (Figure 4.11(a)). For I sections, the categories are stated below.

- Short links $e < e_s = 1.6\ M_{p,link} / V_{p,link}$
- Long links $e > e_L = 3.0\ M_{p,link} / V_{p,link}$
- Intermediate links $e_s < e < e_L$

In designs where only one plastic hinge would form at one end of the link, the value of the length e defines the categories of the links (Figure 4.11(b)). For I sections, the categories are stated below.

- Short links $e < e_s = 0.8(1+\alpha)\ M_{p,link} / V_{p,link}$
- Long links $e > e_L = 1.5(1+\alpha)\ M_{p,link} / V_{p,link}$
- Intermediate links $e_s < e < e_L$

α : The ratio of the smaller bending moments $M_{Ed,A}$ at one end of the link in the seismic design situation, to the greater bending moments $M_{Ed,B}$ at the end where the plastic hinge would form, both moments being taken as absolute values.

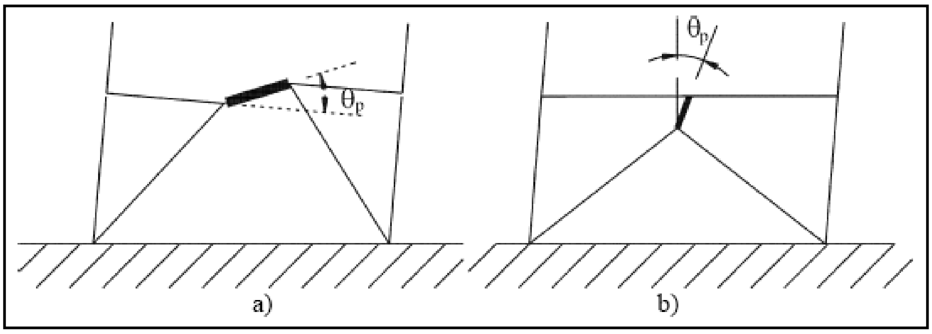


Figure 4.11. Equal and Unequal Moments at Link Ends

The link rotation angle θ_p between the link and the element outside of the link should be consistent with global deformations. It should not exceed the following values:

- Short links $\theta_p \leq \theta_{pR} = 0.08$ radians
- Long links $\theta_p \leq \theta_{pR} = 0.02$ radians
- Intermediate links $\theta_p \leq \theta_{pR}$ = the value determined by linear interpolation between the above values.

Full-depth web stiffeners should be provided on both sides of the link web at the diagonal brace ends of the link. These stiffeners should not have a combined width of not less than $(b_f - 2t_w)$ and a thickness not less than $0.75t_w$ nor 10mm, whichever is larger. Links should be provided with intermediate web stiffeners as follows:

- Short links should be provided with intermediate web stiffeners spaced at intervals not exceeding $(30t_w - d/5)$ for a link rotation angle θ_p of 0.08 radians or $(52t_w - d/5)$ for link rotation angles θ_p of 0.02 radians or less. Linear interpolation should be used for values of θ_p between 0.08 and 0.02 radians.
- Long links should be provided with one intermediate web stiffeners placed at a distance of 1.5 times b from each of the link where a plastic hinge would form.
- Intermediate links should be provided with intermediate web stiffeners meeting the requirements stated above.
- Intermediate web stiffeners are not required in links of length e greater than $5M_p/V_p$.

- Intermediate web stiffeners should be full depth. For links that are less than 600 mm in depth d, stiffeners are required on only one side of the link web. The thickness of one-sided stiffeners should not be less than (b/2) – t_w. For links that are 600 mm in depth or greater, similar intermediate stiffeners should be provided on both sides of the web.

Fillet welds connecting a link stiffener to the link web should have design strength adequate to resist a force of $\gamma_{ov} f_y A_{st}$, where A_{st} is the area of the stiffener. The design strength of fillet welds fastening the stiffener to the flanges should be adequate to resist a force of $\gamma_{ov} f_y A_{st} / 4$. Lateral supports should be provided at both the top and bottom link flanges at the ends of the link. End lateral supports of links should have a design axial resistance sufficient to provide lateral support for forces of 6% of the expected nominal axial strength of the link flange computed as $f_y b t_f$. In beams where a seismic link is present, the shear buckling resistance of the web panels outside of the link should be checked to conform to EC3, chapter 1-5.

Member Not Containing Seismic Links: The members not containing seismic links like the columns and diagonal members, if horizontal links in beams are used, and also the beam members, if vertical links are used, should be verified in compression considering the most unfavorable combination of the axial force and bending moments.

$$N_{Rd}(M_{Ed}, V_{Ed}) \geq N_{Ed,G} + 1.1\gamma_{ov}\Omega N_{Ed,E} \qquad (4.22)$$

$N_{Rd}(M_{Ed}, V_{Ed})$ is the axial design resistance of the column or diagonal member in accordance with EC3, taking into account the interaction with the bending moment M_{Ed} and the shear V_{Ed} taken at their design value in the seismic situation.

$N_{Ed,G}$ is the compression force in the column or diagonal member due to the nonseismic actions included in the combination of actions for their seismic design situation.

$N_{Ed,E}$ is the compression force in the column or diagonal member due to the design seismic action.

γ_{ov} is the overstrength factor.

Ω is a multiplicative factor which is the minimum of the followings values:

- The minimum value of $\Omega_i = 1.5\ V_{p,link,i} / V_{Ed,i}$ among all short links.
- The minimum value of $\Omega_i = 1.5\ M_{p,link,i} / M_{Ed,i}$ among all intermediate and long links.

$V_{Ed,i}$, $M_{Ed,i}$ are the design values of shear force and of the bending moment in link I in the seismic design situation.

$V_{p,link,i}$, $M_{p,link,i}$ are the shear and bending plastic design resistance of link i.

Connection of the Seismic Links: If the structure is designed to dissipate energy in the seismic links, the connection of the links or of the element containing the links should be designed for action effects E_d computed as follows:

$$E_d \geq E_{d,G} + 1.1\gamma_{ov}\Omega_i E_{d,E} \qquad (4.23)$$

$E_{d,G}$: The action effect in the connection due to the non-seismic actions included in the combination of actions for the seismic design situation.

$E_{d,E}$: The action effect in the connection due to the design seismic action.

γ_{ov} : The overstrength factor.

Ω_i : The overstrength factor for the link.

In the case of semi-rigid and partial strength connections, the energy dissipation may be assumed to originate from the connections only. This is allowable, provided that all of the following conditions are satisfied.

- The connections have rotation capacity sufficient for the corresponding deformation demands.
- Members framing into the connections are demonstrated to be stable at the ultimate limit state.
- The effects of connection deformations on global drift are taken into account.

When partial strength connections are used for the seismic links, the capacity design of the other elements in the structure should be derived from the plastic capacity of the links connections.

4.1.8. Design Rules for Inverted Pendulum Structures:

In inverted pendulum structures, the columns should be verified in compression considering the most unfavorable combination of the axial force and bending moments. In the checks, N_{Ed}, M_{Ed}, V_{Ed} should be computed as in the columns of moment resisting frames. The non-dimensional slenderness of the columns should be limited to $\lambda \leq 1.5$. The interstorey drift sensitivity coefficient θ should be limited to $\theta \leq 0.20$.

4.1.9. Design Rules for Moment Resisting Frames Combined with Concentric Bracings:

Dual structures with both moment resisting frames and braced frames acting in the same direction should be designed using a single q factor. The horizontal forces should be distributed between the different frames according to their elastic stiffness. The moment resisting frames and the braced should conform to conditions in moment resisting frames and braced frames. [1]

4.2. Rules According to TEC2007

4.2.1. Design Concept

Lateral load carrying systems of structural steel buildings covered in this chapter may be comprised of steel frames only, of steel braced frames only or of combination of frames with steel braced frames or reinforced concrete structural walls.

4.2.2. Material Conditions and Allowable Stresses

All weldable structural steel sections defined in TS 648 and other internationally accepted standards may be used in accordance with TEC2007. Rolling profiles with flange thickness of minimum 40 mm, sheet iron with thickness of 50 mm and built-up sections produced by this sheet iron should have minimum Charpy-V-Notch strength of 27 Nm at 218C^o.

In bolted connections of primary seismic members of a building, high strength bolts of bolt grade ISO 8.8 or 10.9 should be used. These bolts should be prestressed about applicable prestress force for the moment resisting connections and half of the prestress loads for the other type of connections. In bolted connections of non-seismic members of a building, bolt grade of ISO 4.6 or 5.6 shall be used.

In welded connections, appropriate electrode should be used in accordance with the material of the steel and welding method, also the yielding strength of electrode should be greater than the connected steel materials. In the welded column-beam connection of the moment resisting frames, full penetrated but welds or filled weld should be used. The minimum Charpy-V-Notch strength of the electrodes used in these welds should be 27 Nm at -298C^o. In connections of primary seismic members, welded and bolted connection cannot be used together at the same connection point.

In section design to be performed with Allowable Stress Method under the effect of seismic and other actions, maximum 33% increase is permitted for allowable stresses in TS-648. However, increase in allowable stresses cannot exceed 15% for joints and splices. Beside to these, joints and splices should be checked in accordance with capacity of frame sections and increased seismic actions.

In the calculation of capacity of steel frame section and connections, increased yielding strength, $D_a\sigma_a$, should be used instead of nominal yielding strength, σ_a. D_a coefficients used in the calculation of increased yield strength are stated in Table 4.3 in accordance with classes of steel section and type of frame element. [2]

Table 4.3. D_a Increase Factors

Steel Section Class and Type of Frame Element	D_a
Rolling sections produced by Fe 37 steel	1.2
Rolling Sections produced by other steel material	1.1
Steel sheets produced by all steel materials	1.1

Increased Seismic Actions: In the design of steel frame sections and connections, increased seismic actions shall be used. The load combination gives the increased seismic actions is stated below.

$$1.0\ G + 1.0\ Q \pm \Omega_o E$$

or

$$0.9\ G \pm \Omega_o E$$

Magnification factor applied for the seismic actions is given in Table 4.4 in accordance with the type of the structural system.

Table 4.4 Magnification Factors

Structural System	Ω_0
Systems of High Ductility Level	2.5
Systems of Nominal Ductility Level	2.0
Concentric Bracings (High or Nominal Ductility Level)	2.0
Eccentric Bracings (High or Nominal Ductility Level)	2.5

Capacity of Internal Forces and Limit Stress Values: To be used in appropriate circumstances, capacity of internal forces of structural elements and limit stress values of connection elements are defined below.

Capacity of internal forces of structural elements:

Capacity of Bending Moment : $M_p = W_p \sigma_a$
Capacity of Shear Force : $V_p = 0.60 \sigma_a A_k$
Capacity of Axial Compression : $N_{bp} = 1.7 \sigma_{bem} A$
Capacity of Axial Tension : $N_{cp} = \sigma_a A_n$

Limit stress values of connection elements:

Full penetrated weld : σ_a
Partial penetrated weld and Filled Weld : $1.7 \sigma_{em}$
Bolted Connections : $1.7 \sigma_{em}$

σ_{em} is the allowable stresses for the related connection element (nominal stress, shear stress, and cru. stress).

4.2.3. Classification of Steel Structural Systems

Structural systems of steel buildings are classified into two categories as Systems of High Ductility Level and Systems of Nominal Ductility Level in accordance with the seismic behavior of the system. Both of the systems may be consisted of moment resisting frames and/or frames with bracings.

General Conditions on Ductility Levels of Structural Systems: In structural systems denoted as being high ductility level in Table 4.5, ductility levels shall be high in both lateral earthquake directions. Systems of high ductility or mixed ductility level in one earthquake direction and of nominal ductility level in the perpendicular earthquake direction shall be deemed to be structural systems of nominal ductility level in both directions. In structural systems where ductility levels are the same in both directions or those with high ductility level in one direction and mixed ductility level in the other direction, different R factors may be used in different directions. In the first and second seismic zones, in structural steel buildings with building importance factor is I = 1.2 and I = 1.0, structural system composed of only frames of nominal ductility level may be used, provided that the condition of $H_N \leq 16$ m is met. Otherwise, the use of structural systems of high ductility level is mandatory for buildings with structural systems comprised of frames only. In all buildings with building importance factor is I = 1.5 and I = 1.4 in the first and second seismic zones, structural systems of high ductility level or structural systems of mixed ductility shall be used. Structural systems of nominal ductility level without structural walls or bracings may be permitted only in the third and fourth seismic zones if the structural steel buildings with structural systems comprised of only frames of nominal ductility level can be constructed with $H_N \leq 25$ m. [2]

Table 4.5. Structural Behaviour Factor (R)

STRUCTURAL STEEL BUILDINGS	Nominal Ductility Level	High Ductility Level
(1) Buildings in which seismic loads are fully resisted by frames	5	8
(2) Buildings in which seismic loads are fully resisted by single storey frames with columns hinged at top	---	4
(3) 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
(4) Buildings in which seismic loads are jointly resisted by braced 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

Structural systems of nominal ductility level defined above can also be constructed in all seismic zones as well as above height limits defined in the same paragraphs. However in such cases it is mandatory to have concentric or eccentric braced frames of high ductility level or nominal ductility level in structural steel buildings. Structural systems of nominal ductility level defined above paragraphs may be used in combination with concentric or eccentric bracings of high ductility level. Such structural systems are called mixed systems of ductility levels satisfies the following conditions.

- In the analysis of such mixed systems, frames and bracings shall be considered jointly, however in all cases $\alpha_s \geq 0.40$ shall be satisfied in each earthquake direction.
- When $\alpha_s \geq 2/3$ is satisfied in both earthquake directions, R factor defined in Table 4.4 for the case where seismic loads are fully resisted by concentric or eccentric bracings of high ductility level ($R = R_{YP}$) may be used for the entire structural system.
- In the range $0.40 < \alpha_s < 2/3$, the relationship of $R = R_{NC} + 1.5\ \alpha_s(R_{YP} - R_{NC})$ shall be applied in both earthquake directions. [2]

4.2.4. Frames of High Ductility Level

<u>4.2.4.1. Cross Section Requirement.</u> The conditions about flange width / thickness and web depth / thickness are stated in Table 4.6 for the beams and columns of high ductile frames. The columns should have required strength under the effects of axial force and bending moments induced by the combination of gravity loads and earthquake. Beside to this, the columns should have enough capacity under the effects of increased loading case induced by the axial compression and axial tension forces regardless of bending moments. The axial compression and tension capacity of column cross sections shall be computed in accordance with limit stress values given in the previous chapters.

Table 4.6. Cross Section Conditions

Section Definition	Slenderness Ratio	Limit Values	
		High Ductility Level	**Nominal Ductility Level**
Under Bending and Axial Compression I sections U sections	 b/2t b/t	$0.3\sqrt{E_S/\sigma_a}$	$0.5\sqrt{E_S/\sigma_a}$
I and U sections under Bending	h/t_w	$3.2\sqrt{E_S/\sigma_a}$	$5.0\sqrt{E_S/\sigma_a}$
T and L sections under Axial Compression	h/t_w	$0.3\sqrt{E_S/\sigma_a}$	$0.5\sqrt{E_S/\sigma_a}$
I and U sections under Bending and Axial Compression	h/t_w	$For\ \lvert N_d/\sigma_a A\rvert \leq 0.10$ $3.2\sqrt{E_S/\sigma_a}\ (1-1.7\left\lvert\frac{N_d}{\sigma_a}A\right\rvert)$	$For\ \lvert N_d/\sigma_a A\rvert \leq 0.10$ $5.0\sqrt{E_S/\sigma_a}\ (1-1.7\left\lvert\frac{N_d}{\sigma_a}A\right\rvert)$
		$For\ \lvert N_d/\sigma_a A\rvert > 0.10$ $1.33\sqrt{E_S/\sigma_a}\ (2.1-\left\lvert\frac{N_d}{\sigma_a}A\right\rvert)$	$For\ \lvert N_d/\sigma_a A\rvert > 0.10$ $2.08\sqrt{E_S/\sigma_a}\ (2.1-\left\lvert\frac{N_d}{\sigma_a}A\right\rvert)$
Pipe sections under Bending and Axial Compression	D/t	$0.05\frac{E_S}{\sigma_a}$	$0.08\frac{E_S}{\sigma_a}$
Box sections under Bending and Axial Compression	b/t or h/tw	$0.7\sqrt{E_S/\sigma_a}$	$1.2\sqrt{E_S/\sigma_a}$

4.2.4.2. Requirement of Having Columns Stronger Than Beams. In frame systems or in braced frame systems, sum of the bending moments capacity of columns framing into a beam-column joint in the earthquake direction considered shall be more than the 1.1D_a times of the sum of bending moment capacity of beams framing into the same joint (Figure 4.12)

$$(M_{pa} + M_{pü}) \geq 1.1D_a(M_{pi} + M_{vi} + M_{pj} + M_{vj}) \quad (4.24)$$

M_{vi}, M_{vj} : Additional bending moments occurs on the column face due to the shear force in the potential plastic hinges in the beam ends in the case of usage of weaken beam cross sections and usage of gusset plates in the beam ends.

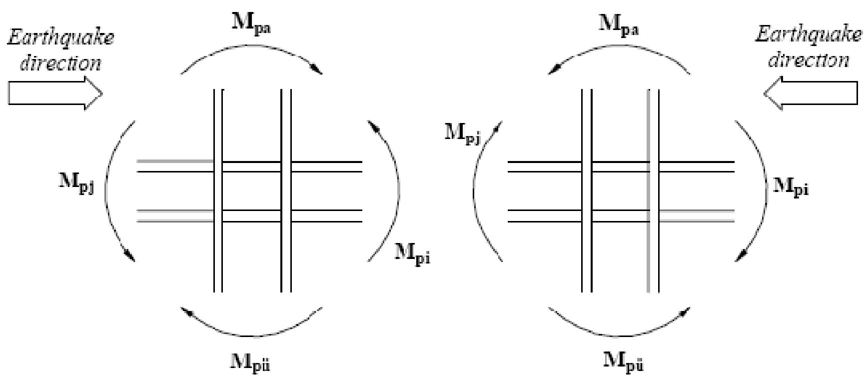


Figure 4.12. Beam-Column Joint

Eqn. 4.24 shall be applied separately for both senses of earthquake direction to yield the most unfavorable results. In calculating the column bending moment capacity, axial forces shall be considered to yield the minimum moment capacity consistent with the sense of earthquake direction. Eqn. 4.24 need no to be checked in single storey buildings and in joints of topmost storey of multi-storey buildings. [2]

<u>4.2.4.3. The Case Where Some Columns Cannot Satisfy the Requirement of Having Columns Stronger than Beams.</u> In structural systems comprised of frames only or of combination of frames and braced frames, Eqn. 4.24 may be permitted not to be satisfied in a given earthquake direction at some joints at the bottom and/or top of an i'th storey, provided that Eqn. 4.25 given below is satisfied.

$$\alpha_i = V_{is} / V_{ik} \geq 0.70 \qquad (4.25)$$

In the case where Eqn. 4.25 is satisfied, bending moments and shear force of columns satisfying Eqn. 4.24. At both bottom and top joints shall be amplified by multiplying with the ratio $1/\alpha_i$ within the range of $0.70 < \alpha_i < 1.00$. In the case where Eqn.

4.24 is not satisfied at any storey, all frames of structural systems which may be comprised of frames only or of combination of frames or braced frames shall be considered as frames of nominal ductility level, and the analysis shall be repeated by changing the Structural Behaviour Factor . However, as it is mentioned in the previous chapters, it is possible to combine frames of nominal ductility level with braced frames of high ductility level. [2]

4.2.4.4. Beam-Column Connections. The following three conditions should be ensured at the beam-column connections for moment resisting frames of high ductility level.

- The connection should have capacity provided that storey drift angle is 0.04 radian. Therefore, the connection details whose validity is proved by experimental and/or analytic methods should be applied.
- Required bending strength of the connection in the column face should not be less than the 0.80 x 1.1D_a times of the bending moment capacity of the beam connected to the column in the column face. However, the upper limit of the connection strength should be compatible with the greatest bending moment transferred to the connection joint from the columns associated to the connection joint. Also, connection strength should not be greater than bending moment calculated with the combined effect of gravity loads and earthquake forces where structural behavior effect, R, is 1.5. Bending moment capacity occurs on the column face should be calculated by sum of the beam plastic moments and additional bending moment occurs on the column face due to the shear force in the potential plastic hinges in the beam ends in the case of usage of weaken beam cross sections and usage of gusset plates in the beam ends.
- Shear force taken into account for the connection design shall be determined by the following Eqn. 4.26.

$$V_e = V_{dy} \pm 1.1D_a \frac{M_{pi}+M_{pj}}{l_n} \qquad (4.26)$$

In the design of beam-column connection, slip zone limited between the flanges of beam and column should be resized to satisfy the following conditions (Figure 4.13).

- Shear strength of the slip zone, V_{ke}, should be taken as equal to the shear force calculated as the 0.80 times of the bending moment capacity of the connected beams to the junction on the column face.

$$V_{ke} = 0.8 \sum M_p \left(\frac{1}{d_b} - \frac{1}{H_{ort}}\right) \tag{4.27}$$

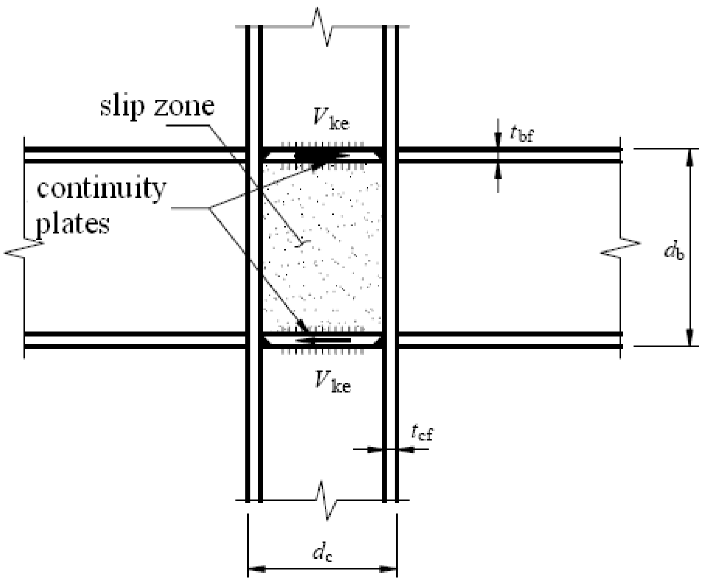


Figure 4.13. Slip Zone at the Beam-Column Joint

- Shear force capacity of the slip zone shall be determined by the following Eqn. 4.28.

$$V_p = 0.6\sigma_a d_c t_p \left[1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_p}\right] \tag{4.28}$$

For the enough shear strength of the slip zone, the following expression should be satisfied.

$$V_p \geq V_{ke} \tag{4.29}$$

Otherwise, the amount required reinforcing plate or stiffener aligned along the diagonal should be added to the slip zone.

- Minimum thickness of column web and reinforcing plates if used should satisfy the following condition.

$$t_{min} \geq u / 180 \qquad (4.30)$$

If the above condition cannot be satisfied, reinforcing plates and column web should be connected to each other by welding to work together. Also, Eqn. 4.30 should be checked according to the summation of the plate thickness.

- If the reinforcing plates are used in the slip zone, full penetration but weld or filled weld shall be used for the connection of plates to the column web (Figure 4.14). These welds should transfer the shear force met by the reinforcing plates in a safely manner.

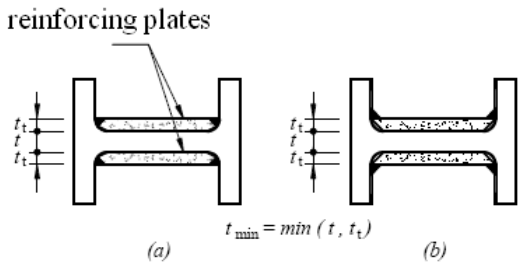


Figure 4.14. Reinforcing Plates on the Column Web

In moment resisting beam-column connections, continuity plates should be used in the column of both sides of web at level of beam flanges to transfer the tension and compression forces on the beam flanges to the column in a safely manner. The thickness of the continuity plates should not be smaller than the flange thickness of the joined beam for one sided connections and not be smaller than the larger of the flange thickness of the joined beams for the two sided connections. The full penetration but weld shall be used to connect the continuity plates to the flanges and web of the column. For the connection of the continuity plates to the web of column, filled weld shall also be used (Figure 4.14).

However, this weld should have a sufficient length and thickness to transfer the force equal to the shear capacity of the continuity plate in its own plane. If the thickness of the column flanges should satisfy both of the following conditions, the continuity plates may not be needed. [2]

$$t_{cf} \geq 0.54\sqrt{b_{bf}t_{bf}} \tag{4.31}$$

$$t_{cf} \geq \frac{b_{bf}}{6} \tag{4.32}$$

4.2.4.5. Splices of Column and Beam. Column splices practiced by the full penetration but welds or bolts shall be made away from beam-column joint by at least 1/3 the storey height. In the case of splices with filled welds or but welds not fully penetrated, this length should be minimum 1.20 m. The beam splices shall be made away from the beam-column joint at least two times of the beam height. Bending capacity of the beam and column splices should not be smaller than the connected sections and shear capacity should not be smaller than the Eqn. 4.26. Moreover, in the first and second earthquake zones, axial force capacity of the column splices shall be adequate under the axial tension and compression forces calculated according to the increased earthquake forces. In the ultimate state design of connection elements, the limit stress values given in the previous chapters should be used.

4.2.4.6. Supporting of Beam Flanges in the Lateral Direction. Top and bottom flanges of beams should be supported in the lateral direction and the length between the supported points, l_b, should satisfy the following condition.

$$l_b \leq 0.086\frac{r_y E_s}{\sigma_a} \tag{4.33}$$

Also, the points loaded by point loads, the joints where there is a sudden change in beam cross section, and the points where potential plastic hinges occur due to the nonlinear deformation of the structural system should be supported in the lateral directions. The tension and compression strength of the supports in the lateral direction should not be smaller than 2% of the axial tension strength of the beam flange.

4.2.5. Frames of Nominal Ductility Level

4.2.5.1. Cross Section Requirement. The conditions about flange width / thickness and web depth / thickness are stated in Table 4.6 for the beams and columns of nominal ductile frames. However, in most two storey buildings, limits in Table 4.6 can be exceeded if the local buckling checks are fulfilled. The conditions given in chapter 4.2.4.1 for the columns of steel frames of high ductility level are valid for the columns of steel frames of nominal ductility level. Requirements of having stronger columns than beams are not to be observed for the frames of nominal ductility level.

4.2.5.2. Beam-Column Connections. In beam-column connections transferring moment of frames of nominal ductility level, stress checks should be performed under the internal forces occur common effects of gravity loads and earthquake loads. Moreover, ultimate capacity of connection should provide the smallest one of the internal forces defined below.

- Bending moment capacity of beam connected to column and required shear strength calculated as in the frames of high ductility level.
- Bending moment and shear force on the column face occur due to the increased loading cases.

The limit stress values given in the previous chapters should be used in the calculation of the ultimate strength of connection. In the design of beam-column connection, slip zone limited between the flanges of beam and column should be resized to satisfy the following conditions (Figure 4.13).

- The smallest shear force occurs due to the increased earthquake loading and calculated according to the Eqn. 4.27 shall be used in the calculation of required shear strength, V_{ke}, of slip zone.
- Shear force capacity of the slip zone shall be determined as in the frames of high ductility level.

- The conditions given for the design of slip zone and continuity plates in the frames of high ductility level are also valid for the frames of nominal ductility level. [2]

4.2.5.3. Splices of Column and Beam. The conditions given for the spices of column and beam in the frames of high ductility level are also valid for the frames of nominal ductility level.

4.2.6. Concentric and Eccentric Steel Bracings

Steel bracings provide lateral resistant are consisted of moment resisting frames and concentric and eccentric bracings. The horizontal forces are mainly resisted by members subjected to axial forces instead of members have bending strength. Steel bracings are divided into two groups according to the bracing arrangement.

- Concentric Steel Bracings (Figure 4.15)
- Eccentric Steel Bracings (Figure 4.16)

Concentric steel bracings are designed as a system of high ductility level or a system of nominal ductility level. However, eccentric steel bracings should be designed as a system of high ductility level.

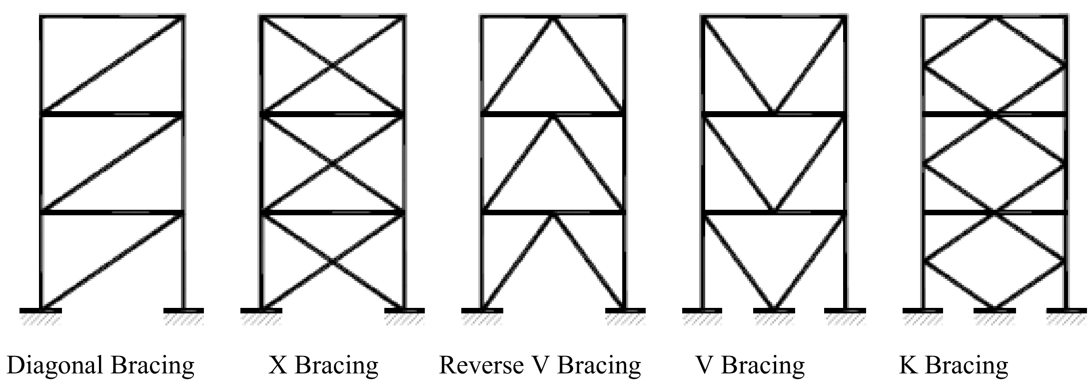


Figure 4.15. Concentric Bracings

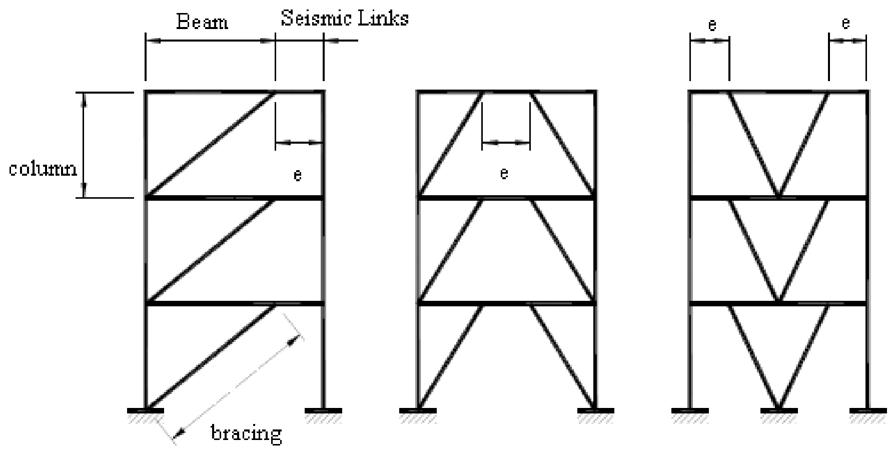


Figure 4.16. Eccentric Bracings

4.2.7. Steel Concentrically Braced Frames of High Ductility Level

Steel bracings of high ductility level should be design without any crucial loss of strength in the system even if some of the compression members buckle. Requirements to be applied to such elements are as specified below.

4.2.7.1. Cross Section Requirement. The conditions about flange width / thickness and web depth / thickness are stated in Table 4.6 for the beams, columns, and bracings of high ductile frames. Slenderness ratio of roof and vertical plane bracing systems designed to resist compressive forces too shall not be more than $4.0\sqrt{E_s/\sigma_a}$.

4.2.7.2. Distribution of Horizontal Forces. The vertical concentric bracings on the one axes of the building to resist compressive forces should meet at least 30% and at most 70% of the horizontal earthquake forces.

4.2.7.3. Connections of Bracings. In connections of bracings, stress checks should be performed under the internal forces occur common effects of gravity loads and earthquake loads. Moreover, ultimate capacity of connection should provide the smallest one of the internal forces defined below.

- The axial tension or compression force capacity of bracing

- The largest axial force transferring to the bracings in accordance with the capacity of other members connected to the joint.
- The axial force capacity of bracing under the increased earthquake forces.

The limit stress values given in the previous chapters should be used in the calculation of the ultimate strength of connection. In the design of beam-column connection, plates of connection joints which connect the bracings to column and/or beams should satisfy the following conditions.

- Bending moment capacity of connection joint plate in its own plane should not be less than the capacity of bracing connected joint.
- The distance from the tip point of bracing to the column or beam should be less than two times of the thickness of joint plate to prevent the out of plane buckling of the joint plate. Otherwise, additional stiffeners should be used to prevent the out of plane buckling of the joint plate.

4.2.7.4. Additional Conditions for Special Bracing Arrangements. Reverse V shape and V shape bracing systems should satisfy the following conditions.

- The beams which bracings are connected should be continuous.
- Bracings should be designed under the internal forces occur common effects of gravity loads and earthquake loads. But, the beams which bracings are connected should provide resistant to gravity loads safely without considering the bracings.
  K shape bracings shall not be allowed for the systems of high ductility level.

4.2.8. Steel Concentrically Braced Frames of Nominal Ductility Level

Requirements to be applied to steel bracings of nominal ductility level are as specified below.

<u>4.2.8.1. Cross Section Requirement.</u> The conditions about flange width / thickness and web depth / thickness are stated in Table 4.6 for the beams, columns, and bracings of nominal ductile frames. However, in most two storey buildings, limits in Table 4.6 can be exceeded if the local buckling checks are fulfilled. Slenderness ratio of roof and vertical plane bracing systems designed to resist compressive forces too shall not be more than $4.0\sqrt{E_s/\sigma_a}$. Regarding brace cross-sections made up multiple elements to resist compressive forces, rules given in TS-648 for intermediate coupling elements are applicable. In the case where braces are designed to resist tension only, slenderness ratio of braces shall not exceed 250. However, in most two storey buildings, the slenderness limit can be exceeded if the tension force calculated in structural analysis is multiplied by Ω_0 given in Table 4.4.

<u>4.2.8.2. Connections of Bracings.</u> In connections of bracings, stress checks should be performed under the internal forces occur common effects of gravity loads and earthquake loads. Moreover, ultimate capacity of connection should provide the smallest one of the internal forces defined below.

- The axial tension or compression force capacity of bracing
- The largest axial force transferring to the bracings in accordance with the capacity of other members connected to the joint.
- The axial force capacity of bracing under the increased earthquake forces.

The limit stress values given in the previous chapters should be used in the calculation of the ultimate strength of connection. In the design of beam-column connection, plates of connection joints which connect the bracings to column and/or beams should satisfy the following conditions.

- Bending moment capacity of connection joint plate in its own plane should not be less than the capacity of bracing connected joint.
- The distance from the tip point of bracing to the column or beam should be less than two times of the thickness of joint plate to prevent the out of plane buckling of the joint plate. Otherwise, additional stiffeners should be used to prevent the out of plane buckling of the joint plate.

4.2.8.3. Additional Conditions for Special Bracing Arrangements. Reverse V shape and V shape bracing systems should satisfy the following conditions.

- The beams which bracings are connected should be continuous.
- Bracings should be designed under the internal forces occur common effects of gravity loads and earthquake loads. But, the beams which bracings are connected should provide resistant to gravity loads safely without considering the bracings.

4.2.9. Steel Eccentrically Braced Frames of High Ductility Level

Eccentric bracing of high ductility level shall be designed so that seismic links have capability of nonlinear deformation under the earthquake forces. The structural system shall be designed so that columns, bracings, beams without seismic links are in the elastic region of stress-strain diagram while the plastic hinges occur in the seismic links. Rules for the design of eccentrically braced frames of high ductility level are stated below.

4.2.9.1. Cross Section Requirement. The conditions about flange width / thickness and web depth / thickness are stated in Table 4.6 for the beams, columns, and eccentric bracings of high ductile frames. Slenderness ratio of bracing shall not be more than $4.0\sqrt{E_s/\sigma_a}$.

4.2.9.2. Seismic Links. In the eccentric bracings of high ductility level, each bracing shall have at least one end seismic links. The length of seismic link should satisfy the following condition.

$$1.0\, M_p/V_p \leq e \leq 5.0\, M_p/V_p \qquad (4.34)$$

M_p : Capacity of Bending Moment
V_p : Capacity of Shear Force

Design shear force of the seismic link should satisfy the following conditions.

$$V_d \leq V_p \qquad (4.35)$$

$$V_d \leq 2\, M_p/e \tag{4.36}$$

If the design axial force of seismic link, M_p and V_p shall be replaced by M_{pn} and V_{pn} respectively.

$$M_{pn} = 1.18 M_p \left[1 - \frac{N_d}{\sigma_a A}\right] \tag{4.37}$$

$$V_{pn} = V_p \sqrt{1 - (N_d/\sigma_a A)^2} \tag{4.38}$$

The web of seismic links should be one piece and on the web of the seismic link, reinforcing plates shall not be available. Opening on the web of the seismic link shall not be permitted.

4.2.9.3. Supporting of Seismic Link in the Lateral Direction. Top and bottom flanges of seismic links should be supported in the lateral direction at two ends of link and be supported at one end of link if the seismic links are located at column edge. The tension and compression strength of the supports in the lateral direction should not be smaller than 6% of the axial tension strength of the beam flange. Also, the rest of beam should be supported in the lateral directions in a range of $0.45 b_{bf}\sqrt{E_s/\sigma_a}$. The tension and compression strength of these supports in the lateral direction should not be smaller than 1% of the axial tension strength of the beam flange.

4.2.9.4. Rotation Angle of Seismic Link. The link rotation angle γ_p between the link and the element outside of the link should be consistent with storey drift, θ_p (Figure 4.17). It should not exceed the following values:

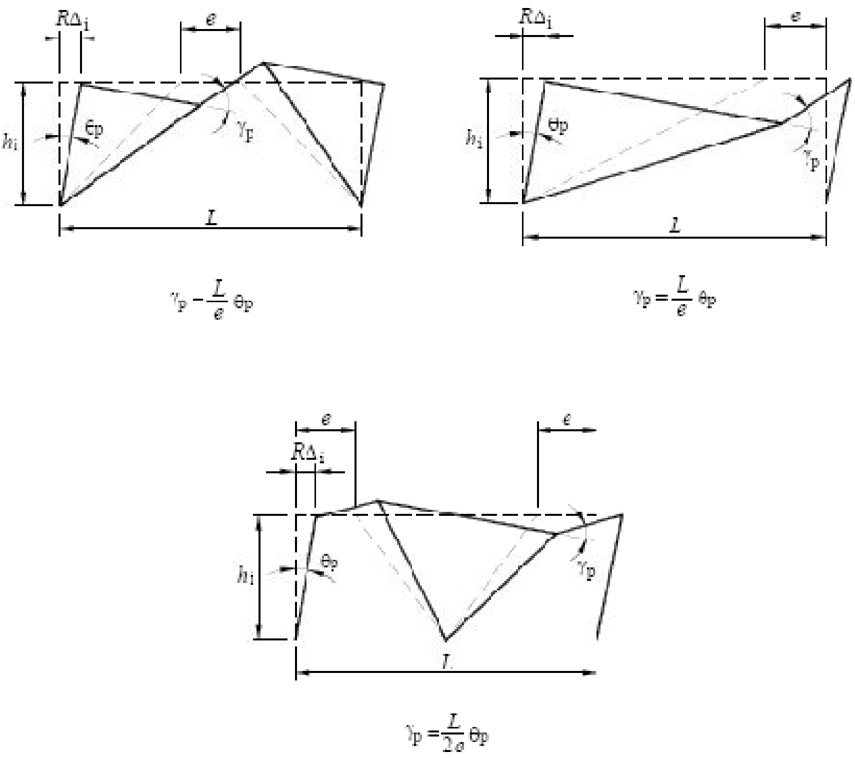


Figure 4.17. Rotation Angle of Seismic Link

- $\gamma_p \leq 0.10$ radian if $e \leq 1.6M_p/V_p$
- $\gamma_p \leq 0.03$ radian if $e \leq 2.6M_p/V_p$
- $1.6M_p/V_p \leq e \leq 2.6M_p/V_p$, the value determined by linear interpolation between the above values.

4.2.9.5. Stiffener Plates. Full-depth web stiffeners should be provided on both sides of the link web at the diagonal brace ends of the link. These stiffeners should not have width of not less than $(b_f - t_w)/2$ and a thickness not less than $0.75t_w$ nor 10mm, whichever is larger (Figure 4.18). Fillet welds connecting a link stiffener to the link web should have design strength adequate to resist a force of f_yA_{st}, where A_{st} is the area of the stiffener.

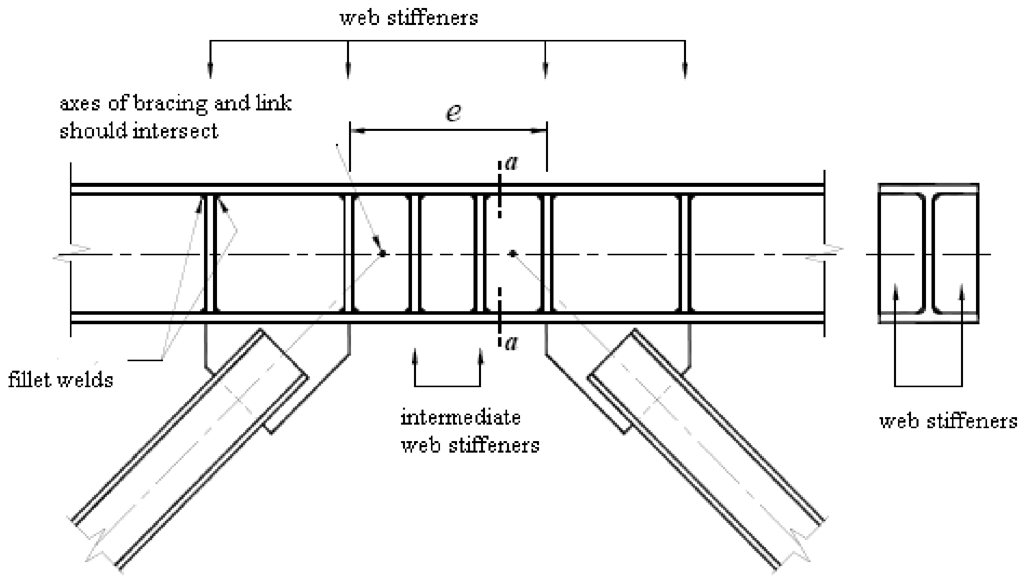


Figure 4.18. Seismic Link and Stiffeners

Links should be provided with intermediate web stiffeners as follows:

- Seismic links with length of less than $1.6M_p/V_p$ should be provided with intermediate web stiffeners spaced at intervals not exceeding $(30t_w - d_b/5)$ for a link rotation angle θ_p of 0.10 radians or $(52t_w - d_b/5)$ for link rotation angles θ_p of 0.03 radians or less. Linear interpolation should be used for values of θ_p between 0.10 and 0.03 radians.
- Seismic links with length of between $2.6M_p/V_p$ and $5.0M_p/V_p$ should be provided with one intermediate web stiffeners placed at a distance of 1.5 times b_{bf} from each of the link where a plastic hinge would form.
- Seismic links with length of between $1.6M_p/V_p$ and $2.6M_p/V_p$ should be provided with intermediate web stiffeners meeting the requirements stated above.

4.2.9.6. Beams, Columns and Bracings. The load causes plastic hinges on the seismic links shall be determined by multiplying the internal forces calculated according to the structural analysis results with Design Magnification Factor, M_p/M_d, V_p/V_d, calculated according to the selection of seismic link cross section. Bracings shall be designed according to $1.25D_a$ times of the internal forces cause plastic hinges on the seismic links. The rest of the beam shall be designed according to 1.10D_a times of the internal forces cause plastic hinges on the seismic links. Columns shall be designed under the internal forces occur common effects of gravity loads and earthquake loads. Ultimate capacity of column should not exceed the following values:

- 1.10D_a times of the internal forces cause plastic hinges on the seismic links.
- Internal forces occur due to the increased earthquake forces.

Capacity of beams, columns, and bracings shall be determined according to the limit stress values given chapter 4.2.2.

4.2.9.7. Connection of Seismic Link and Column. The length of the seismic link connected to the column shall satisfy the following condition.

$$e \leq 1.6M_p / V_p \qquad (4.39)$$

The bending and shear strength of the connection on the column face should not be less than the bending moment capacity of seismic link, M_p, and shear force capacity of seismic link, V_p. Full penetration but welds shall be used to connect flanges of seismic links to the column (Figure 4.19). [2]

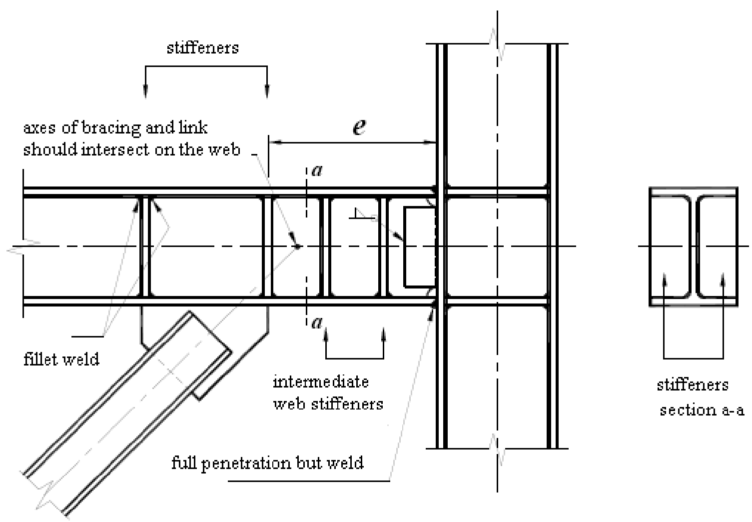


Figure 4.19. Seismic Link and Stiffeners

5. DESIGN OF STEEL STRUCTURES ACCORDING TO EC3

5.1. Scope of EC3

The resistance of cross sections and members specified in EC3 for the ultimate limit states are based on tests in which the material exhibits sufficient ductility to apply simplified design models.

5.2. Material Properties

The nominal values of material properties should be adopted as characteristic values in design calculations. The nominal values of the yield strength f_y and the ultimate strength f_u for structural steel shall be obtained by using the given Table 5.1. [5]

Table 5.1. Nominal Values of the Yield Strength and the Ultimate Strength for Hot Rolled Structural Steel

Standard And steel grade	Nominal thickness of the element t [mm]			
	$t \leq 40$ mm		40 mm $< t \leq 80$ mm	
	fy [N/mm^2]	fu [N/mm^2]	fy [N/mm^2]	fu [N/mm^2]
EN 10025-2				
S235	235	360	215	360
S 275	275	430	255	410
S 355	355	510	335	470
S 450	440	550	410	550
EN 10025-3				
S 275 N/NL	275	390	255	370
S 355 N/NL	355	490	335	470
S 420 N/NL	420	520	390	520
S 460 N/NL	460	540	430	540
EN 10025-4				
S 275 M/ML	275	370	255	360
S 355 M/ML	355	470	335	450
S 420 M/ML	420	520	390	500
S 460 M/ML	460	540	430	530
EN 10025-5				
S 235 W	235	360	215	340
S 355 W	355	510	335	490
EN 10025-6				
S 460 Q/QL/QL1	460	570	440	550

The material coefficients to be adopted in calculations for the structural steels should be taken as follows.

Modulus of Elasticity $E = 21\ 000\ 000$ t/m^2

Shear Modulus $G = E / 2(1+\upsilon) = 8\ 100\ 000$ t/m^2

Poisson Ratio $\upsilon = 0.3$

Coefficient of Linear Thermal Expansion $\alpha = 12*10^{-6}$ per^{o}C (for $T \leq 100^o$C)

Bolts of grades lower than 4.6 or higher than 10.9 shall not be used unless test results prove their acceptability in a particular application. The nominal values of the yield strength f_{yb} and the ultimate tensile strength f_{ub} for bolts are given in Table 5.2.

Table 5.2. Nominal Values of the Yield Strength and the Ultimate Strength for Bolts

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

5.3. Serviceability Limit State

The limiting values for vertical deflections given below are illustrated by reference to the simply supported beam shown in Figure 5.1.

$$\delta_{max} = \delta_1 + \delta_2 + \delta_0 \tag{5.1}$$

δ_{max} : The sagging in the final stage relative to the straight line joining the supports.
δ_0 : The pre-camber (hogging) of the beam in the unloaded stage (stage 0).
δ_1 : The variation of deflection of the beam due to the permanent loads immediately after loading (stage 1).
δ_2 : The variation of deflection of the beam due to the variable loading plus any time dependent deformations due to the permanent load (stage 2).

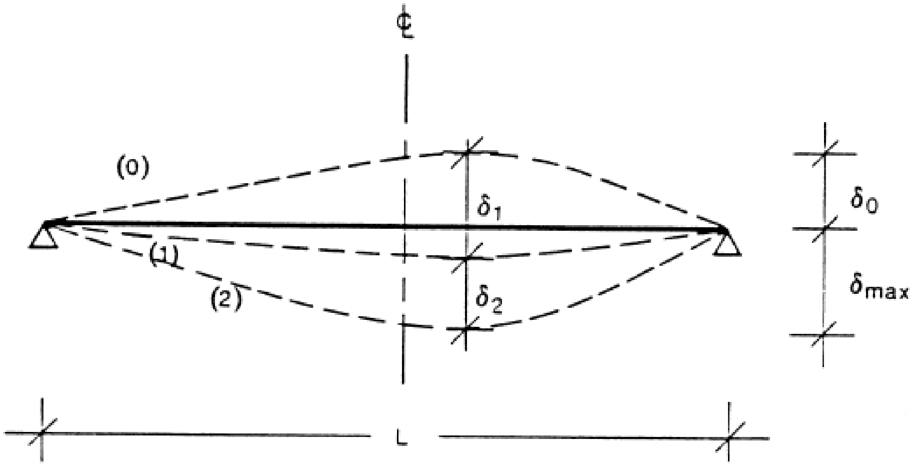


Figure 5.1. Vertical Deflections

For buildings, the recommended limits for vertical deflections are given in Table 5.3 in which L is the span of the beam. For cantilever beams the length L to be considered is twice the projecting length of the cantilever. [5]

Table 5.3. Recommended Limits for Vertical Deflections

Conditions	Limits	
	δ_{max}	δ_2
Roofs generally	L/200	L/250
Roofs frequently carrying personnel other than for maintenance	L/250	L/300
Floors generally	L/250	L/300
Floors and roofs supporting plaster or other brittle finish or non-flexible Partitions	L/250	L/350
Floors supporting columns (unless the deflection has been included in the global analysis for the ultimate limit state)	L/400	L/500
Where δ_{max} can impair the appearance of the building	L/250	-

For buildings the recommended limits for horizontal deflections at the tops of the columns are:

- Portal frames without gantry cranes ……….. h/150

- Other single storey buildings ……………….. h/300
- In a multistory building;
  - In each storey ……………………….. h/300
  - On the structure as a whole ………… $h_0/500$

Where h is the height of the column or the storey and h_0 is the overall height of the structure.

5.4. Ultimate Limit State

The structural member and connections should be designed in accordance with the Ultimate Limit State method. The values of the design actions should be considered with the design strength of the members and connections.

$$S_d \leq R_d \tag{5.2}$$

S_d : Design values of actions
R_d : Design strength of the members

Partial safety factors γ_m for the calculation of design strengths shall be taken as follows. [5]

- Resistance of Class 1, 2, 3 cross sections ……… $\gamma_{m0} = 1.1$
- Resistance of Class 4 cross sections ……….……. $\gamma_{M1} = 1.1$
- Resistance of members to buckling ……….……. $\gamma_{M1} = 1.1$
- Resistance of net section at bolt holes …………... $\gamma_{M2} = 1.25$

Frames shall be checked for ;

- Resistance of cross section
- Resistance of members
- Resistance of connections
- Frame stability
- Static Equilibrium

5.5. Classification of Cross Sections

When plastic global analysis is used, the members shall be capable of forming plastic hinges with sufficient rotation capacity to enable the required redistribution of bending moments to develop. However, any class of cross sections may be used for the members, provided that the design of members takes into account the possible limits on the resistance of cross sections due to local buckling when elastic global analysis is used.

Four classes of cross sections are defined as follows.

- Class 1: The cross sections which can form a plastic hinges with the rotation capacity required for plastic analysis.
- Class 2: The cross sections which can develop their plastic moment resistance, but have limited rotation capacity.
- Class 3: The cross sections in which the calculated stress in the extreme compression fiber of the steel member can reach its yield strength, but local buckling is liable to prevent development of the plastic moment resistance.
- Class 4: The cross sections in which it is necessary to make explicit allowances for the effects of local buckling when determining their moment resistance or compression resistance.

Effective widths may be used in Class 4 cross sections to make the necessary allowances for reductions in resistance due to the effects of local buckling. The classification of a cross section depends on the proportions of each of its compression elements (Table 5.4 and Table 5.5). One of the major factors in determining the limiting width – thickness ratio is ε. This parameter is used to reflect the influence of yield stress on the section classification.

$$\varepsilon = \sqrt{\frac{235}{f_y}} \qquad (5.3)$$

Table 5.4. Limiting Width-Thickness Ratio for Classification of Cross Sections (Compression and Bending)

Section	Element	Ratio Checked	Class 1	Class 2	Class 3
I-SHAPE	Web	d/tw	If α>0.5 , $\frac{396\varepsilon}{(13\alpha-1)}$, $else\ if\ \alpha \geq 0.5$, $\frac{36\ \varepsilon}{\alpha}$.	If α>0.5 , $\frac{456\varepsilon}{(13\alpha-1)}$, $else\ if\ \alpha \geq 0.5$, $\frac{36\ \varepsilon}{\alpha}$.	$If\ \psi > -1$, $\frac{42\varepsilon}{0.67+0.33\psi}$, $else\ if\ \psi \geq -1$, $\frac{62\ \varepsilon\ (1-\psi)}{\sqrt{-\psi}}$.
	Flange	c/tf (rolled)	10ε	11ε	15ε
		c/tf (welded)	9ε	10ε	14ε
BOX	Web	d/tw	Same as I-Shape	Same as I-Shape	Same as I-Shape
	Flange	$(b-3t_f)/t_f$ (rolled)	42 ε	42 ε	42 ε
		b/t_f (welded)	42 ε	42 ε	42 ε
CHANNEL	Web	d/tw	Same as I-Shape	Same as I-Shape	Same as I-Shape
	Flange	b/t_f	10 ε	11 ε	15 ε
T-SHAPE	Web	d/tw	33 ε	38 ε	42 ε
	Flange	$b/2t_f$ (rolled)	10 ε	11 ε	15 ε
		$b/2t_f$ (welded)	9 ε	10 ε	14 ε
DOUBLE ANGLE	-	h/t (b+h)/[2max(t,b)]	Not applicable	Not applicable	15 ε 11.5 ε
ANGLE	-	h/t (b+h)/[2max(t,b)]	Not applicable	Not applicable	15 ε 11.5 ε
PIPE	-	d/t	$50\varepsilon^2$	$70\varepsilon^2$	$90\varepsilon^2$
ROUND BAR	-	None	Assumed Class 1		
ROUND BAR	-	None	Assumed Class 2		

Table 5.5. Limiting Width-Thickness Ratio for Classification of Cross Sections (Bending)

Section	Element	Ratio Checked	Class 1	Class 2	Class 3
I-SHAPE	web	d/tw	72ε	83ε	124ε
	flange	c/tf (rolled)	10ε	11ε	15ε
		c/tf (welded)	9ε	10ε	14ε
BOX	web	d/tw	72ε	83ε	124ε
	flange	$(b-3t_f)/t_f$ (rolled)	33 ε	38 ε	42 ε
		b/t_f (welded)	33 ε	38 ε	42 ε
CHANNEL	web	d/tw (Major axis)	72ε	83ε	124ε
		d/tw (Minor axis)	33ε	38 ε	42 ε
	flange	b/t_f	10 ε	11 ε	15 ε
T-SHAPE	web	d/tw	33 ε	38 ε	42 ε
	flange	$b/2t_f$ (rolled)	10 ε	11 ε	15 ε
		$b/2t_f$ (welded)	9 ε	10 ε	14 ε
DOUBLE ANGLE	-	h/t (b+h)/[2max(t, b)]	Not applicable	Not applicable	15 ε 11.5 ε
ANGLE	-	h/t (b+h)/[2max(t, b)]	Not applicable	Not applicable	15 ε 11.5 ε
PIPE	-	d/t	$50\varepsilon^2$	$70\varepsilon^2$	$90\varepsilon^2$
ROUND BAR	-	None	Assumed Class 1		
RECTANGLE	-	None	Assumed Class 2		
GENERAL	-	None	Assumed Class 3		

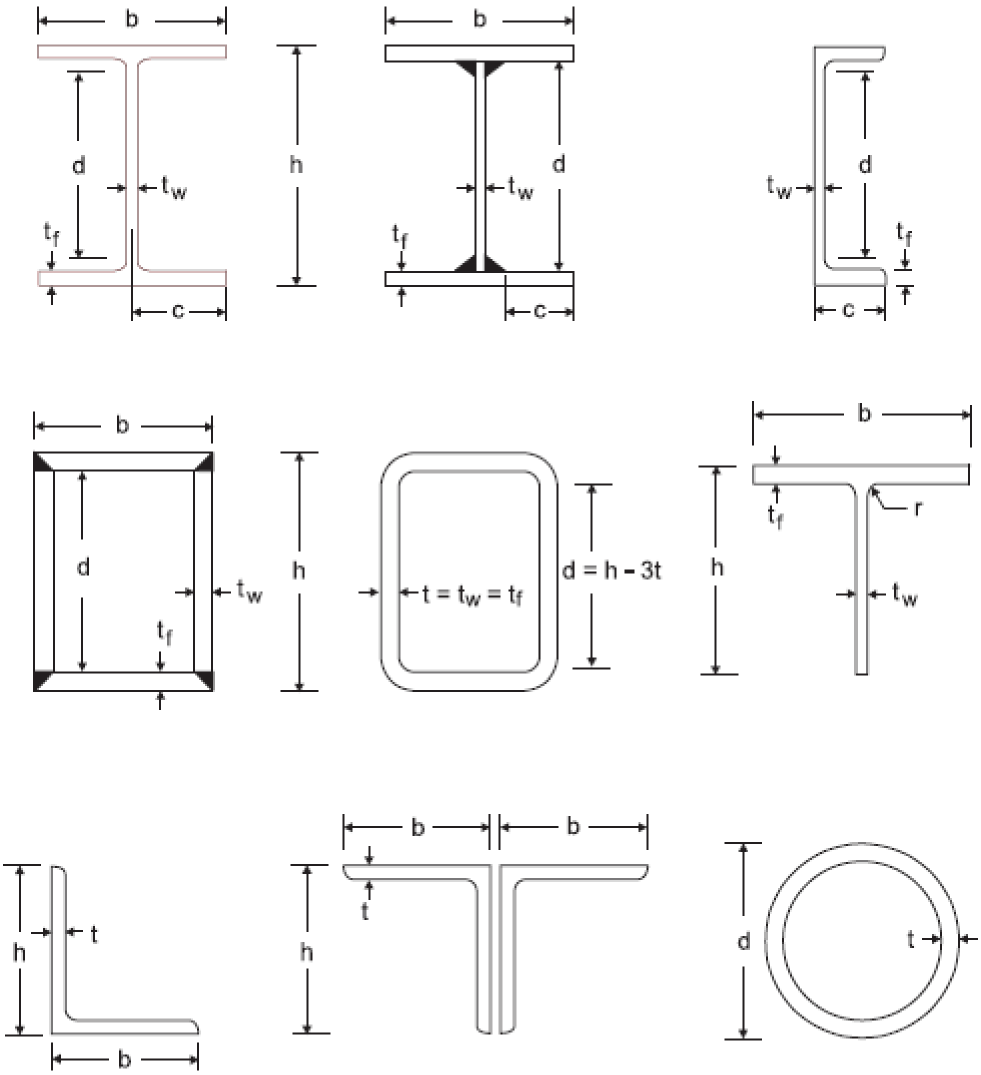


Figure 5.2. Definitions of Geometric Properties

In classifying I, box, Channel, Double-Channel, and T sections, two other factors are defined as follows.

$$\alpha = \begin{cases} \frac{1}{2} - \frac{1}{2}\frac{N_{c,Sd}}{h t_w f_f} \; for\ I, Channel, and\ T\ sections \\ \frac{1}{2} - \frac{1}{2}\frac{N_{c,Sd}}{2h t_w f_f} \; for\ Box\ and\ Double - Channel \end{cases} \tag{5.4}$$

$$\Psi = -\left(1 + 2\frac{N_{c,Sd}}{A f_y}\right) \tag{5.5}$$

$$0 < \alpha \leq 1.0 \tag{5.6}$$

$$-3.0 < \psi \leq 1.0 \quad (5.7)$$

In the above expression $N_{c,Sd}$ is taken as positive for tension and negative for compression. α equals 0.0 for full tension, 0.5 for pure bending and 1.0 for full compression. ψ equals -3.0 for full tension, -1.0 for pure bending and 1.0 for full compression.

Compression elements include every element of a cross section which is either totally or partially in compression due to axial force or bending moment, under the load combination considered. The various compression elements in a cross section (such as web or a flange) can be in different classes.

5.6. Analysis of Cross Sections

5.6.1. Connections

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

The partial safety factor γ_M shall be taken as follows.

- Resistance of bolted connections : $\gamma_{Mb} = 1.25$
- Resistance of welded connections : $\gamma_{Mw} = 1.25$

5.6.1.1 Bolted Connections. The design of a bolted connection loaded in shear and tension shall conform one of the following categories (Table 5.6.). [5]

Table 5.6. Categories of Bolted Connections

Shear Connections		
Category	**Criteria**	**Remarks**
A Bearing Type	$F_{v,Sd} \leq F_{v,Rd}$ $F_{v,Sd} \leq F_{b,Rd}$	No preloading required All grades from 4.6 to 10.9
B Slip Resistance at serviceability	$F_{v,Sd.ser} \leq F_{s,Rd.ser}$ $F_{v,Sd} \leq F_{v,Rd}$ $F_{v,Sd} \leq F_{b,Rd}$	Preloaded high strength bolts No slip at serviceability limit state
C Slip Resistance at ultimate	$F_{v,Sd} \leq F_{s,Rd}$ $F_{v,Sd} \leq F_{b,Rd}$	Preloaded high strength bolts No slip at ultimate limit state
Tension Capacity		
Category	**Criteria**	**Remarks**
D Non-preloaded	$F_{t,Sd} \leq F_{t,Rd}$	No preloading required All grades from 4.6 to 10.9
E Preloaded	$F_{t,Sd} \leq F_{t,Rd}$	Preloaded high strength bolts

At the ultimate limit state the design shear force $F_{v,Sd}$ on a bolt shall not excedd the lesser of;

- The design shear resistance $F_{y,Rd}$
- The design bearing resistance $F_{b,Rd}$

The design tensile force $F_{t,Sd}$ inclusive of any force due to prying action shall not exceed the design tension resistance $B_{t,Rd}$ of the bolt-plate assembly. The design tension resistance of the bolt-plate assembly $B_{t,Rd}$ shall be taken as the smaller of the design tension resistance $F_{t,Rd}$ and the design punching shear resistance of the bolt head and the nut, $B_{p,Rd}$ obtained from;

$$B_{p,Rd} = 0.6\ \pi\ d_m\ t_p\ f_u\ /\ \gamma_{Mb} \qquad (5.8)$$

t_p : The thickness of the plate under the bolt head or the nut.
d_m : The mean of the across points and across flats dimensions of the bolt head or the nut whichever is smaller.

Shear resistance per shear plane: If the shear plane passes through the threaded portion of the bolt;

- For the strength grades 4.6, 5.6, and 8.8:

$$F_{v,Rd} = \frac{0.6 f_{ub} A_s}{\gamma_{Mb}} \quad (5.9)$$

- For strength grades 4.8, 5.8, and 10.9:

$$F_{v,Rd} = \frac{0.5 f_{ub} A_s}{\gamma_{Mb}} \quad (5.10)$$

If the shear plane passes through the unthreaded portion of the bolt;

$$F_{v,Rd} = \frac{0.6 f_{ub} A}{\gamma_{Mb}} \quad (5.11)$$

Bearing resistance:

$$F_{b,Rd} = \frac{2.5 \alpha f_u d t}{\gamma_{Mb}} \quad (5.12)$$

Where α is the smallest of;

$$\frac{e_1}{3d_0} \; ; \; \frac{p_1}{3d_0} - \frac{1}{4} \; ; \; \frac{f_{ub}}{f_u} \; or \; 1.0$$

Tension resistance:

$$F_{t,Rd} = \frac{0.9 f_{ub} A_s}{\gamma_{Mb}} \quad (5.13)$$

Bolts subject to shear force and tensile force shall in addition satisfy the following equation.

$$\frac{F_{v,Sd}}{F_{v,Rd}} + \frac{F_{t,Sd}}{1.4F_{t,Rd}} \leq 1.0 \qquad (5.14)$$

5.6.1.2 Welded Connections. Welds shall generally be classified as fillet welds, slot welds, butt welds which may be either full penetration or partial penetration, plug welds (Table 5.7).

Resistance of Welds: The resistance of a fillet weld may be assumed to be adequate if, at every point in its length, the resultant of all the forces per unit length transmitted by the weld does not exceed its design resistance $F_{w,Rd}$. Independent of the orientation of the weld, the design resistance per unit length $F_{w,Rd}$ shall be determined from Eqn. 5.15.

$$F_{w,Rd} = f_{vw,d}\, a \qquad (5.15)$$

where $f_{vw,d}$ is the design shear strength of the weld.

The design shear strength $f_{vw,d}$ of the weld shall be determined from Eqn. 5.16.

$$f_{vw,d} = \frac{f_u\sqrt{3}}{\beta_w \gamma_{Mw}} \qquad (5.16)$$

where f_u is the nominal ultimate tensile strength of the weaker part joined and β_w is the appropriate correlation factor. The value of the correlation factor β_w should be taken as given in Table 5.8. For intermediate values of f_u, the value of β_w may be determined by linear interpolation.

Table 5.7. Common Types of Welded Joints

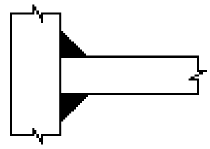
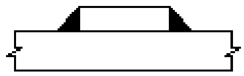
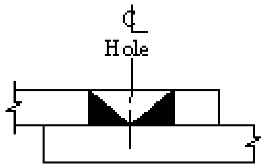
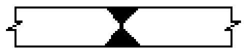
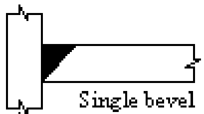
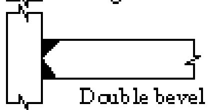
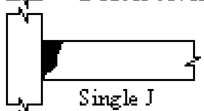
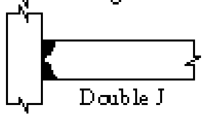
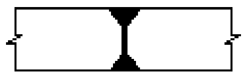
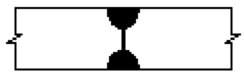
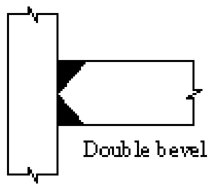
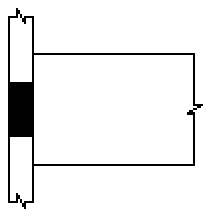
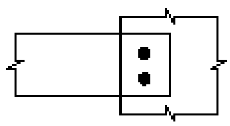
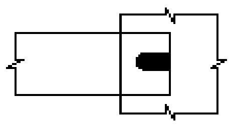
Type of weld	Type of joint		
	Butt joint	Tee-butt joint	Lap joint
Fillet weld			
Slot weld			
Full penetration butt weld[a]	 Single V  Double V  Single U  Double U	 Single bevel  Double bevel  Single J  Double J	
Partial penetration butt weld[a]	 Double V  Double U	 Double bevel	
Plug weld			 
Flare groove welds			
[a] Butt welds can sometimes be formed without cutting any weld preparation in either component			

Table 5.8. Correlation Factors

Steel Grade	Ultimate Tensile Strength f_u	Correlation Factor β_w
Fe 360	360 N/mm^2	0.8
Fe 430	430 N/mm^2	0.85
Fe 510	510 N/mm^2	0.9

5.6.2. Tension Members

For members in axial tension, the design value of the tensile force N_{sd} at each cross-section shall satisfy the following Eqn. 5.17.

$$N_{sd} \leq N_{t,Rd} \qquad (5.17)$$

where $N_{t,Rd}$ is the design tension resistance of the cross-section, taken as the smaller of the following conditions.

- The design plastic resistance of the gross cross-section

$$N_{pl,Rd} = Af_y/\gamma_{M0} \qquad (5.18)$$

- The design ultimate resistance of the net cross-section at holes for fasteners

$$N_{u,Rd} = 0.9A_{net}f_u/\gamma_{M2} \qquad (5.19)$$

where $\gamma_{M0} = 1.10$

$\gamma_{M2} = 1.25$

5.6.3. Compression Members

5.6.3.1 Buckling Resistance. The design buckling resistance of a compression member shall be taken as given in Eqn. 5.20.

$$N_{b,Ed} = \chi \beta_A A\, f_y / \gamma_{M1} \tag{5.20}$$

β_A = 1 for Class 1, 2 or 3 cross-sections
$\beta_A = A_{eff}/A$ for Class 4 cross-sections

χ is the reduction factor for the relevant buckling mode.

For hot rolled steel members with the types of the cross section commonly used for compression members, the relevant buckling mode is generally flexural buckling. In some cases the torsional or flexural torsional modes may govern.

For constant axial compression in members of constant cross section, the value of χ for the appropriate non-dimensional slenderness λ^*, may be determined from Eqn. 5.21.

$$\chi = \frac{1}{\emptyset + [\emptyset^2 - \lambda^{*2}]^{0.5}} \text{ but } \chi \leq 1 \tag{5.21}$$

where $\Phi = 0.5[1+\alpha(\lambda^* - 0.2) + \lambda^{*2}]$ (5.22)

α is an imperfection factor

$\lambda^* = [\beta_A A f_y / N_{cr}]^{0.5} = (\lambda / \lambda_1)\,[\beta_A]^{0.5}$ (5.23)

λ is the slenderness for the relevant buckling mode

$\lambda_1 = \pi[E/f_y]^{0.5} = 93.9\varepsilon$ (5.24)

$\varepsilon = [235/f_y]^{0.5}$ (f_y in N/mm^2) (5.25)

N_{cr} is the elastic critical force for the relevant buckling mode.

The imperfection factor α corresponding to the appropriate buckling curve shall be obtained from Table 5.9.

Table 5.9. Imperfection Factors

Bucking curve	a	b	c	d
Imperfection factor α	0,21	0,34	0,49	0,76

The values of the reduction factor χ for the appropriate non-dimensional slenderness may be obtained from Table 5.10.

5.6.3.2 Flexural Buckling. For flexural buckling, the appropriate buckling curve shall be determined from Table 5.11. The slenderness λ shall be taken as given in Eqn. 5.26.

$$\lambda = l \, / \, \mathrm{i} \tag{5.26}$$

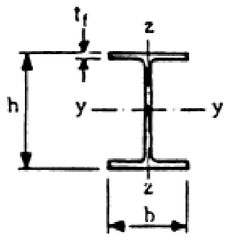
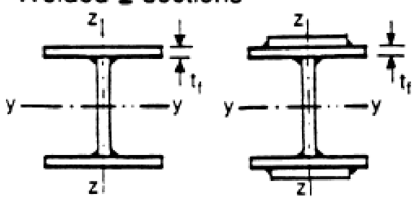
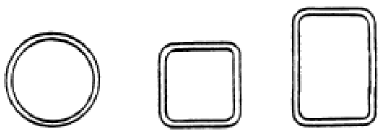
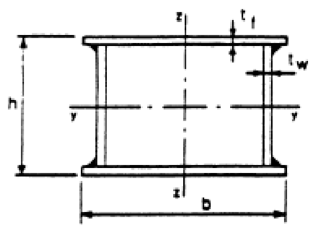
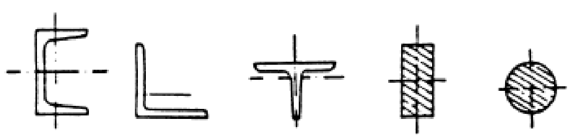
i : The radius of gyration about the relevant axis, determined using the properties of the gross cross section.

The buckling length l of a compression member with both ends effectively held in position laterally may conservatively be taken as equal to its system length L.

Table 5.10. Reduction Factors χ

$\bar{\lambda}$	Buckling curve			
	a	b	c	d
0,2	1,0000	1,0000	1,0000	1,0000
0,3	0,9775	0,9641	0,9491	0,9235
0,4	0,9528	0,9261	0,8973	0,8504
0,5	0,9243	0,8842	0,8430	0,7793
0,6	0,8900	0,8371	0,7854	0,7100
0,7	0,8477	0,7837	0,7247	0,6431
0,8	0,7957	0,7245	0,6622	0,5797
0,9	0,7339	0,6612	0,5998	0,5208
1,0	0,6656	0,5970	0,5399	0,4671
1,1	0,5960	0,5352	0,4842	0,4189
1,2	0,5300	0,4781	0,4338	0,3762
1,3	0,4703	0,4269	0,3888	0,3385
1,4	0,4179	0,3817	0,3492	0,3055
1,5	0,3724	0,3422	0,3145	0,2766
1,6	0,3332	0,3079	0,2842	0,2512
1,7	0,2994	0,2781	0,2577	0,2289
1,8	0,2702	0,2521	0,2345	0,2093
1,9	0,2449	0,2294	0,2141	0,1920
2,0	0,2229	0,2095	0,1962	0,1766
2,1	0,2036	0,1920	0,1803	0,1630
2,2	0,1867	0,1765	0,1662	0,1508
2,3	0,1717	0,1628	0,1537	0,1399
2,4	0,1585	0,1506	0,1425	0,1302
2,5	0,1467	0,1397	0,1325	0,1214
2,6	0,1362	0,1299	0,1234	0,1134
2,7	0,1267	0,1211	0,1153	0,1062
2,8	0,1182	0,1132	0,1079	0,0997
2,9	0,1105	0,1060	0,1012	0,0937
3,0	0,1036	0,0994	0,0951	0,0882

Table 5.11. Selection of a Buckling Curve for a Cross Section

Cross section	Limits	Buckling about axis	Buckling curve
Rolled I-sections 	$h/b > 1{,}2$: $t_f \le 40$ mm	$y-y$ $z-z$	a b
	$40 \text{ mm} < t_f \le 100$ mm	$y-y$ $z-z$	b c
	$h/b \le 1{,}2$: $t_f \le 100$ mm	$y-y$ $z-z$	a b
	$t_f > 100$ mm	$y-y$ $z-z$	d d
Welded I-sections 	$t_f \le 40$ mm	$y-y$ $z-z$	b c
	$t_f > 40$ mm	$y-y$ $z-z$	c d
Hollow sections 	hot rolled	any	a
	cold formed — using f_{yb}[a]	any	b
	cold formed — using f_{ya}[a]	any	c
Welded box sections 	generally (except as below)	any	b
	thick welds and $b/t_f < 30$ $h/t_w < 30$	$y-y$ $z-z$	c c
U-, L-, T- and solid sections 		any	c

5.6.4. Flexural Members

In the absence of shear force, the design value of the bending moment M_{Sd} at each cross-section shall satisfy the following Eqn. 5.27.

$$M_{Sd} \leq M_{c,Rd} \qquad (5.27)$$

where $M_{c,Rd}$ is the design moment resistance of the cross section, taken as the smallest of the following conditions.

- The design plastic resistance moment of the gross section

$$M_{pl,Rd} = W_{pl} f_y / \gamma_{M0} \qquad (5.28)$$

- The design local buckling resistance moment of the gross section

$$M_{o,Rd} = W_{eff} f_y / \gamma_{M1} \qquad (5.29)$$

where W_{eff} is the effective section modulus.

- The design ultimate resistance moment of the net section at bolt holes $M_{u,Rd}$.

For a Class 3 cross-section the design moment resistance of the gross section shall be taken as the design elastic resistance moment given by:

$$M_{el,Rd} = W_{el} f_y / \gamma_{M0} \qquad (5.30)$$

5.6.5. Shear Resistance of Structural Members

The design value of the shear force V_{Sd} at each cross-section shall satisfy the following Eqn. 5.31.

$$V_{Sd} \leq V_{pl,Rd} \quad (5.31)$$

where $V_{pl,Rd}$ is the design plastic shear resistance given by:

$$V_{pl,Rd} = A_v(f_y/\sqrt{3})/\gamma_{M0} \quad (5.32)$$

where A_v is the shear area.

The shear area A_v may be taken as follows:

- $A_v = A - 2bt_f + (t_w + 2r)t_f$ for rolled I and H sections load parallel to web
- $A_v = A - 2bt_f + (t_w + r)t_f$ for rolled channel sections load parallel to web
- $A_v = \sum(dt_w)$ for welded I, H, and box sections load parallel to web
- $A_v = A - \sum(dt_w)$ for welded I, H, channel and box sections load parallel to flanges
- $A_v = Ah / (b + h)$ for rolled rectangular hollow sections of uniform thickness load parallel to depth
- $A_v = Ab / (b + h)$ for rolled rectangular hollow sections of uniform thickness load parallel to breadth
- $A_v = 2A / \pi$ for circular hollow sections and tubes of uniform thickness
- $A_v = A$ for plates and solid bars

A : The cross-section area
b : The overall breadth
d : The depth of the web
h : The overall depth
r : The root radius
t_f : The flange thickness
t_w : The web thickness

5.6.6. Bending and Shear

The theoretical plastic resistance moment of a cross-section is reduced by the presence of shear. For small values of shear force this reduction is so small that is counter-balanced by strain hardening and may be neglected. However, when the shear force exceeds half the plastic shear resistance, allowance shall be made for its effect on the plastic resistance moment. Provided that the design value of the shear force V_{Sd} does not exceed 50% of the design plastic shear resistance $V_{pl,Rd}$ no reduction need be made in the resistance moments. When V_{Sd} exceeds 50% of $V_{pl,Rd}$ the design resistance moment of the cross-section should be reduced to $M_{V,Rd}$ the reduced design plastic resistance moment allowing for the shear force, obtained as follows:

- For cross-sections with equal flanges, bending about the major axis

$$M_{V,Rd} = \left[W_{pl} - \frac{\rho A_v^2}{4t_w}\right] f_y/\gamma_{M0} \qquad \text{but } M_{V,Rd} \leq M_{c,Rd} \tag{5.33}$$

$$\text{where } \rho = \left(2V_{Sd}/V_{pl,Rd} - 1\right)^2 \tag{5.34}$$

- For other cases, $M_{V,Rd}$ should be taken as the design plastic resistance moment of the cross-section, calculated using a reduced strength (1-p)f_y for the shear area, but not more than $M_{c,Rd}$.

5.6.7. Bending and Axial Force

<u>5.6.7.1 Class 1 and 2 Cross-Sections.</u> For class 1 and 2 cross-sections, the criterion to be satisfied in the absence of shear force is:

$$M_{Sd} \leq M_{N,Rd} \tag{5.35}$$

where $M_{N,Rd}$ is the reduced design plastic resistance allowing for the axial force.

For a plate without bolt holes, the reduced plastic resistance moment is given by:

$$M_{N,Rd} = M_{pl,Rd}\left[1 - \left(N_{Sd}/N_{pl,Rd}\right)^2\right] \qquad (5.36)$$

and the criterion becomes:

$$\frac{M_{Sd}}{M_{pl,Rd}} + \left[\frac{N_{Sd}}{N_{pl,Rd}}\right]^2 \leq 1 \qquad (5.37)$$

In flanged sections, the reduction of the theoretical plastic resistance moment by the presence of small axial force is counter-balanced by strain hardening and may be neglected. However, for bending about the y-y axis, allowance shall be made for the effect of the axial force on the plastic resistance moment when the axial force exceeds half the plastic tension resistance of the web, or a quarter of the plastic tension resistance of the cross-section, whichever is smaller. Similarly, for bending about z-z axis, allowance shall be made for the effect of the axial force when it exceeds the plastic tension resistance of the web.

For cross section without bolt holes, the following approximations may be used for standard rolled I or H sections.

$$M_{Ny,Rd} = M_{pl,Rd}(1-n)/(1-0.5a) \quad \text{but } M_{Ny,Rd} \leq M_{pl,y,Rd} \qquad (5.38)$$

$$\text{for } n \leq a \quad M_{Nz,Rd} \leq M_{pl,z,Rd} \qquad (5.39)$$

$$\text{for } n > a \quad M_{Nz,Rd} = M_{pl,x,Rd}\left[1 - \left[\frac{n-a}{1-a}\right]^2\right] \qquad (5.40)$$

$$\text{where } n = N_{Sd}/N_{pl,Rd} \qquad (5.41)$$

$$\text{and} \quad a = (A-2bt_f)/A \quad \text{but } a \leq 0.5 \qquad (5.42)$$

For bi-axial bending the following approximate criterion may be used:

$$\left[\frac{M_{y,Sd}}{M_{Ny,Rd}}\right]^{\alpha} + \left[\frac{M_{z,Sd}}{M_{Nz,Rd}}\right]^{\beta} \leq 1 \qquad (5.43)$$

In which α and β are constants, which may conservatively be taken as unity, otherwise as follows.

- I and H sections, $\alpha=2$ and $\beta=5n$ but $\beta \geq 1$
- Circular tubes, $\alpha=2$ and $\beta=2$ but $\beta \geq 1$
- Rectangular hollow sections, $\alpha = \beta = \frac{1.66}{1-1.13n^2}$ but $\alpha = \beta \leq 6$
- Solid rectangles and plates, $\alpha = \beta = 1.73 + 1.8n^3$ where $n = N_{Sd}/N_{pl,Rd}$

As a further conservative approximation, the following criterion may be used:

$$\frac{N_{Sd}}{N_{pl,Rd}} + \frac{M_{y,Sd}}{M_{pl,y,Rd}} + \frac{M_{z,Sd}}{M_{pl,z,Rd}} \leq 1 \qquad (5.44)$$

5.6.7.2 Class 3 Cross-Sections. In the absence of shear force, Class 3 cross-sections will be satisfactory if the maximum longitudinal stress $\sigma_{x,Ed}$ satisfy the criterion:

$$\sigma_{x,Ed} \leq f_{yd} \quad \text{where } f_{yd} = f_y/\gamma_{M0} \qquad (5.45)$$

For cross-sections without holes for fasteners, the above criterion becomes:

$$\frac{N_{Sd}}{Af_{yd}} + \frac{M_{y,Sd}}{W_{el,y}f_{yd}} + \frac{M_{z,Sd}}{W_{el,z}f_{yd}} \leq 1 \qquad (5.46)$$

5.6.7.3 Class 4 Cross-Sections. In the absence of shear force, Class 4 cross-sections will be satisfactory if the maximum longitudinal stress $\sigma_{x,Ed}$ calculated using the effective widths of the compression elements satisfies the criterion:

$$\sigma_{x,Ed} \leq f_{yd} \quad \text{where } f_{yd} = f_y/\gamma_{M1} \qquad (5.47)$$

For cross-sections without holes for fasteners, the above criterion becomes:

$$\frac{N_{Sd}}{A_{eff} f_{yd}} + \frac{M_{y,Sd} + N_{Sd}\, e_{Ny}}{W_{eff,y} f_{yd}} + \frac{M_{z,Sd} + N_{Sd}\, e_{Nz}}{W_{eff,z} f_{yd}} \leq 1 \tag{5.48}$$

where

A_{eff} : The effective area of the cross-section when subject to uniform compression.

W_{eff} : The effective section modulus of the cross-section when subject only to moment about the relevant axis.

e_N : The shift of the relevant centroidal axis when the cross-section is subject to uniform compression.

5.6.8. Bending, Shear and Axial Force

When the shear force exceeds half the plastic shear resistance, allowance shall be made for the effect of both shear force and the axial force on the reduced plastic resistance moment. Provided that the design value of the shear force V_{Sd} does not exceed 50% of the design plastic shear resistance $V_{pl,Rd}$, no reduction need to be made in combinations of moment and axial force that meet the criteria in chapter 5.7. When V_{Sd} exceeds 50% of $V_{pl,Rd}$ the design resistance of the cross section to combinations of moment and axial force should be calculated using a reduced yield strength $(1-\rho)f_y$ for the shear area, where $\rho = (2V_{Sd}/V_{pl,Rd} - 1)^2$.

6. DESIGN OF STEEL STRUCTURES ACCORDING TO TSE 648

6.1. Scope of TSE 648

Steel structures shall be designed and constructed in accordance with TSE 648. The general structural steel classes covered in TSE 648 are Fe33, Fe34, Fe37, Fe42, Fe46, Fe50, Fe52, Fe60, and Fe70 according to the smallest tension strength.

6.2. Material Properties

The tension strength, yielding strength, modulus of elasticity, shear modulus and coefficient of thermal expansion for the structural steel are stated in Table 6.1. [6]

Table 6.1. Material Properties of Steel

Steel Class	**Tension Strength** kgf/cm^2	**Yielding Limit** kgf/cm^2	**Modulus of Elasticity** kgf/cm^2	**Shear Modulus** kgf/cm^2	**Coefficient of Thermal Expansion** α_t
Fe 37	3700	2400	2100000	810000	0.000012
Fe 42	4200	2600			
Fe 52	5200	3600			

6.3. Load Patterns and Loading Conditions

Loads should be compatible with TSE 498 for the design and stability checks of steel structures. Loads acting on the structure shall be divided into two groups.

- Principal Loads: Dead loads, live loads, snow loads.
- Additional Loads: Wind loads, earthquake loads, break loads of cranes, temperature loads.

The following loading conditions shall be taken into account in the design of steel structures.

- EY Loading: Sum of the principal loads.
- EIY Loading: Sum of the principal loads and additional loads.

6.3.1. Cross Sections Considering in Design of Steel Structural Elements

Cross sections taken into account in the stress checks of structural steel members are given in Table 6.2. [6]

Table 6.2. Cross Sections Considering in Design of Steel Structural Members

Stress Type	**Stress**	**Cross Section Taken into Consideration**
Axial Force	Compression	F
	Tension	$F_n = F - \Delta F$
Shear Force	Shear	F_{web}
Bending Force	Tension or Compression	$W_n = I_n/e = (I - \Delta I)/e$
Shear Stress on web $\tau = Q/F_{web}$		

where

F = Cross sectional area of the member under compression force

F_n = Useful cross sectional area of the member under tension force

ΔF = Amount of weakened cross sectional area

Useful net area of member cross section shall be taken as its gross area less appropriate deductions for all holes and other openings. When the fastener holes are staggered, the total area to be deducted for fastener holes shall be the sum of the cross sectional areas of all holes in any diagonal or zig-zag line extending progressively across the member or part of the member, less $s^2t/4g$ for each gauge space in the chain of holes.

s : The staggered pitch, the spacing of the centers of two consecutive holes in the chain measured parallel to the member axis.
g : The spacing of the centers of the same two holes measured perpendicular to the member axis.
t : The thickness.

The net width calculated by taking account the holes should be less than the 85% of the total width of the member.

6.4. Analysis of Cross Sections

The member stresses and support reactions should be calculated separately for each load combinations. The member stresses calculated according to the most unfavorable load combination should be compared with the working stresses.

Frames shall be checked for stress, stability, over-turning and deformations.

- Stress Checks: The stress checks should be performed according to the working stress for the loading conditions EY and EIY.
- Stability Checks: The stability checks are consisted of buckling, crumpling and local buckling of the structural member.
- Over-turning Checks: The safety factor for over-turning of the structural part should be at least 2. The safety factor for lifting of supports should be 1.3 at continuous beams and 2 at the whole f the structure.
- Deformation Checks: Steel structures and components shall be so proportioned that deflections are within the limits according to the intended use and the nature of the materials to be supported. The limiting value of deformations for the beams with span length greater than 5m should be less than L/300 and the limiting value of deformations for the cantilever beams should be less than L/250. [6]

6.4.1. Tension Members

For members in axial tension, the working stress should be less than 0.6 times of the yielding strength.

$$\sigma_{cem} \leq 0.6\sigma_a \qquad (6.1)$$

Also, this value should be less than the 0.5 times of the tension strength of member.

$$\sigma_{cem} \leq 0.5\sigma_d \qquad (6.2)$$

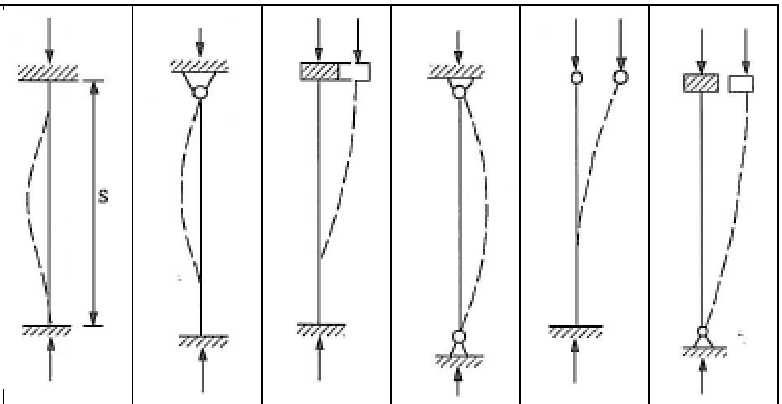
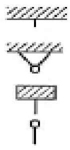
According to the loading conditions, the members are sometimes in axial tension and sometimes in axial compression should be designed as compression member and the slenderness ratio should be less than 250.

$$\lambda \leq 250 \qquad (6.3)$$

6.4.2. Compression Members

For members in axial tension, the in plane buckling length of the member, S_k, is equal to the real length (system length) of the member. However, the out of plane buckling length of the compression member is equal to the real length if the out of plane translation of nodal points in the compression region of the member should be prevented. The variable buckling lengths of compression members are stated in Table 6.3.

Table 6.3. Variable Buckling Lengths

Note: Hidden Lines shows the buckling shape of the compression member						
Theoretical Buckling Length	0.5s	0.7s	1.0s	1.0s	2.0s	2.0s
Recommended Buckling Length	0.65s	0.8s	1.2s	1.0s	2.1s	3.0s
Information		Rotation and Translation Restrained				
		Free Rotation and Translation Restrained				
		Rotation Restrained and Free Translation				
		Free Rotation and Translation				

The design of compression member shall be perform by “ω” method according to the TSE 648. For the compression members, the following condition should be satisfied.

$$\sigma = P/F \leq \sigma_{bem} \qquad (6.4)$$

The buckling allowable stresses, σ_{dem}, are variable in accordance with slenderness ratio of compression member, λ. In practice, since working with variable allowable is not preferred, the “ω” numbers will be introduced.

$$\omega = \sigma / \sigma_{bem} \qquad (6.5)$$

by convince,

$$\sigma = \frac{\omega P}{F} \leq \sigma_{cem} \qquad (6.6)$$

S : The maximum compression force in member (kgf)

F : The cross-section area (cm^2)

σ_{cem} : Tension allowable stress according to the loading condition and material class (kgf/cm^2)

ω : The buckling coefficient according to slenderness ratio of compression member, λ (Table 6.4)

$$\lambda = \lambda_x = S_{kx} / i_x \quad \text{or} \tag{6.7}$$

$$\lambda = \lambda_y = S_{ky} / i_y \tag{6.8}$$

For the compression members whose slenderness ratio, λ, is smaller than 20, the buckling check is not required ($\omega = 1$).

Table 6.4. The Buckling Coefficients

λ	0	1	2	3	4	5	6	7	8	9
20	1.02	1,03	1.03	1.04	1,05	1,06	1,06	1,07	1,08	1,09
30	1.09	1.10	1.11	1.11	1.12	1.13	1.14	1.15	1.15	1.16
40	1.17	1.18	1.20	1.20	1.20	1.21	1.22	1.23	1.24	1.25
50	1.26	1.27	1.28	1.29	1.30	1.31	1.32	1.33	1.34	1.35
60	1.36	1.37	1.38	1.39	1.40	1.41	1.43	1.44	1.45	1.46
70	1.47	1.49	1.50	1.51	1.53	1.54	1.56	1.57	1.58	1.59
80	1.67	1.62	1.64	1.65	1.67	1.69	1.70	1.72	1.74	1.75
90	1.77	1.79	1.81	1.82	1.84	1.86	1.88	1.90	1.92	1.94
100	1.96	1.99	2.01	2.03	2.05	2.06	2.10	2.13	2.15	2.18
110	2.20	2.23	2.26	2.29	2.32	2.35	2.38	2.41	2.84	2.48
120	2.51	2.55	2.59	2.63	2.66	2.74	2.75	2.79	2.45	2.88
130	2.83	2.96	3.03	3.07	3.12	3.17	3.21	3.25	3.31	3.36
140	3.91	3.45	3.50	3.55	3.60	3.65	3.70	3.75	3.80	3.96
150	3.40	3.96	4.01	4.07	4.12	4.17	4.23	4.28	4.34	4.39
160	4.45	4.50	4.58	4.61	4.67	4.73	4.79	4.84	4.90	4.96
170	5.02	5.08	5.14	5.20	5.26	5.32	5.38	5.44	5.50	5.57
180	5.63	5.89	5.75	5.82	5.88	5.94	6.01	6.07	6.14	6.20
190	6.27	6.34	6.40	6.47	6.54	6.60	6.67	6.74	6.81	6.88
200	6.95	7.02	7.09	7.15	7.23	7.30	7.37	7.44	7.51	7.69
210	7.60	7.73	7.81	7.88	7.95	8.03	8.10	8.18	8.25	8.33
220	8.41	8.48	8.58	8.64	8.72	8.79	8.87	8.95	9.03	9.11
230	9.19	9.27	9.35	9.43	9.51	9.57	9.67	9.76	9.84	9.92
240	10.00	10.09	10.17	10.26	10.34	10.43	10.51	10.60	10.68	10.77
250	10.96									

λ	0	1	2	3	4	5	6	7	8	9
10	1.00	1.00	1.00	1.00	1.00	1.01	1.02	1.03	1.03	1.04
20	1.05	1.06	1.07	1.08	1.09	1.10	1.11	1.11	1.12	1.13
30	1.14	1.15	1.16	1.17	1.18	1.19	1.20	1.22	1:23	1.24
40	1.25	1.26	1.27	1.28	1.30	1.31	1.32	1.33	1.35	1.36
50	1.37	1.38	1.40	1..41	1.43	1.44	1.45	1.47	1.49	1.50
60	1.52	1.54	1.55	1.57	1.59	1.60	1.62	1.64	1.66	1.68
70	1.70	1.72	1.74	1.76	1.78	1.80	1.83	1.85	1.87	1.90
80	1.92	1.95	1.97	2.00	2.03	2.06	2.08	2.11	2.15	2.18
90	2.21	2.24	2.28	2.32	2.35	2.39	2.43	2.47	2.52	2.56
100	2.61	2.65	2.70	2.75	2.81	2.86	2.92	2.96	3.04	3.10
110	3.15	3.21	3.27	3.33	3.39	3.45	3.51	3.57	3.63	3.69
120	3.75	3.81	8.88	3.94	4.01	4.07	4.14	4.20	4.27	4.34
130	4.40	4.47	4.54	4.61	4.68	4.75	4.82	4.89	4.96	5.03
140	5.11	5.18	5.25	5.33	5.40	5.48	5.55	5.63	5.71	5.78
150	5.86	5.94	6.02	6.10	6.18	6.26	6.34	6.42	6.50	6.59
160	6.57	6.75	6.84	6.92	7.01	7.09	7.18	7.27	7.35	7.44
170	7.53	7.62	7.71	7.80	7.89	7.98	8.07	8.16	8.25	8.35
180	8.44	8.54	8.63	8.73	8.82	8.92	9.01	9.11	9.21	9.31
190	9.41	9.50	9.60	9.70	9.81	9.91	10.01	10.11	10.21	10.32
200	10.42	10.53	10.63	10.74	10.84	10.95	11.06	11.15	11.27	11.36
210	11.49	11.60	11.71	11.82	11.93	12.04	12.16	12.27	12.36	12.50
220	12.61	12.73	12.84	12.96	13.07	13.19	13.31	13.43	13.34	13.50
230	13.78	13.90	14.02	14.14	14.27	14.39	14.51	14.63	14.76	14.88
240	15.01	15.13	15.26	15.38	15.51	15.04	15.77	15.90	16.02	16.15
250	16.25									

The buckling coefficient tables are only applicable for Fe37 and Fe52 type steel classes. Therefore, the allowable compression stresses should be computed for the other type of steel classes. If the slenderness ratio of compression member is smaller than λ_p ($\lambda \leq \lambda_p$), the allowable compression stress is:

$$\sigma_{bem} = \frac{\left[1 - \frac{1}{s}\left(\frac{\lambda}{\lambda_p}\right)^2\right]\sigma_a}{n} \tag{6.9}$$

If the slenderness ratio is greater than λ_p ($\lambda > \lambda_p$), the allowable compression stress is:

$$\sigma_{bem} = \frac{1}{n}\frac{\pi^2 E}{\lambda^2} \cong \frac{8290000}{\lambda^2} \tag{6.10}$$

$$\lambda < 20 \qquad n = 1.67 \tag{6.11}$$

$$\lambda \leq \lambda_p \qquad n = 1.5 + 1.2\left(\frac{\lambda}{\lambda_p}\right) - 0.2\left(\frac{\lambda}{\lambda_p}\right)^3 \tag{6.12}$$

$$\lambda > \lambda_p \qquad n = 2.5 \tag{6.13}$$

E : Modulus of Elasticity (kgf/cm^2)

σ_a : Yielding strength of steel (kgf/cm^2)

λ_p : Plastic slenderness ratio (critical slenderness)

$$\lambda_p = \sqrt{\frac{2\pi^2 E}{\sigma_a}} = \frac{6438.4}{\sqrt{\sigma_a}} \tag{6.14}$$

σ_{bem} : Allowable compression stress (kgf/cm^2)

6.4.3. Bending and Axial Compression Force

Member in axial compression and flexure should satisfy the following conditions without considering the buckling.

$$\frac{\sigma_{eb}}{\sigma_{bem}} + \frac{C_{mx}\sigma_{bx}}{\left(1-\frac{\sigma_{eb}}{\sigma_{ex}}\right)\sigma_{Bx}} + \frac{C_{my}\sigma_{by}}{\left(1-\frac{\sigma_{eb}}{\sigma_{ey}}\right)} \leq 1.0 \qquad (6.15)$$

$$\frac{\sigma_{eb}}{0.6\sigma_{ba}} + \frac{\sigma_{bx}}{\sigma_{Bx}} + \frac{\sigma_{by}}{\sigma_{By}} \leq 1.0 \qquad (6.16)$$

When $\frac{\sigma_{eb}}{\sigma_{ba}} \leq 0.15$, instead of above equations, the following equation shall be used.

$$\frac{\sigma_{eb}}{\sigma_{ba}} + \frac{\sigma_{bx}}{\sigma_{Bx}} + \frac{\sigma_{by}}{\sigma_{By}} \leq 1.0 \qquad (6.17)$$

where

σ_{bem} : Allowable stress under only compression

σ_B : Allowable stress under only flexure

$\sigma_e = \frac{\pi^2 e}{(KS_b/I_b)^2}\frac{1}{2.5} = \frac{8290000}{(KS_b/I_b)^2}$

S_b : Unsupported length between the supports

I_b : Moments of inertia with respect to the axis perpendicular bending plane

K : Coefficient to calculate the buckling length with respect to the bending axis

σ_{eb} : Stress under the compression forces only

σ_b : The compression stress under bending moments only

C_m : The coefficient in accordance with end moments and moment of span

- $C_m = 0.85$ for frames of lateral displacements are allowed
- $C_m = 0.6 - 0.4M_1/M_2 \geq 0.4$ for frames of lateral displacements of joints are prevented and if there is no loading on the bending plane of the member.

M_1/M_2 is the ratio of smallest bending moment to the biggest one at the ends of member. The ratio is positive for the two directional bending and negative for one directional bending (Figure 6.1).

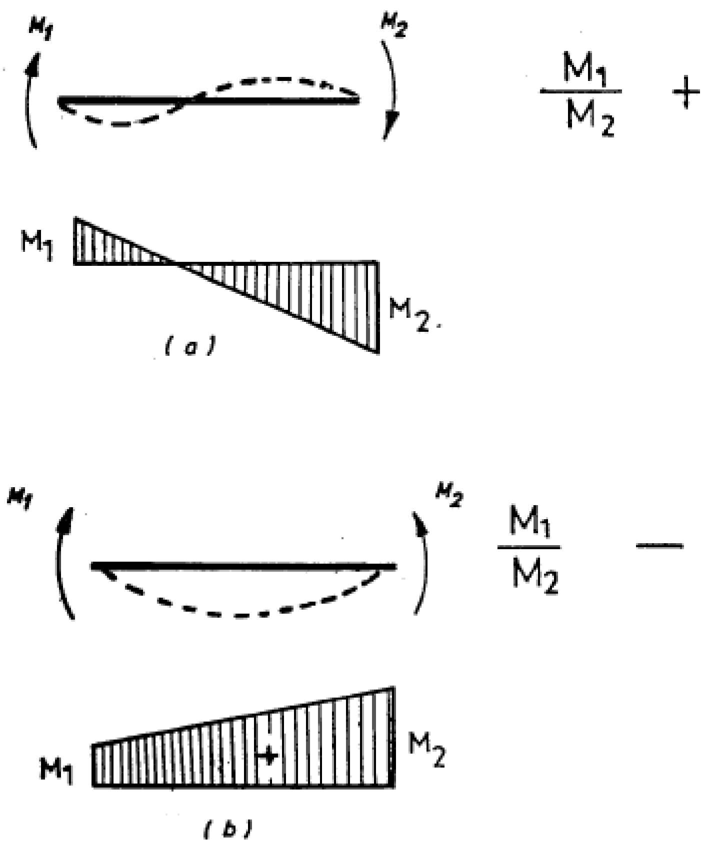


Figure 6.1. Ratio of M_1/M_2

For the frames of lateral displacements are prevented and if there is loading on the bending plane of the member, the coefficient of C_m should be calculated by more accurate method (Figure 6.2).

$$C_m = 1 + \psi \frac{\sigma_{eb}}{\sigma_{e'}} \tag{6.18}$$

$$\psi = \frac{\pi^2 \delta_0 EI}{M_0 S^2} \tag{6.19}$$

δ_0 : The maximum displacements due to the bending force

M_0 : The maximum moment of span

s : The length of the member

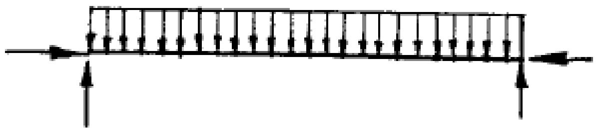
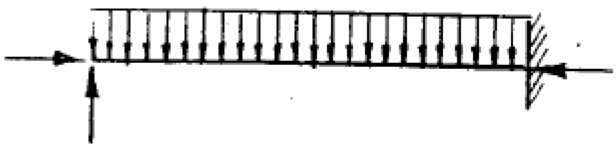
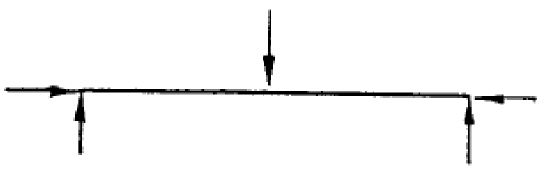
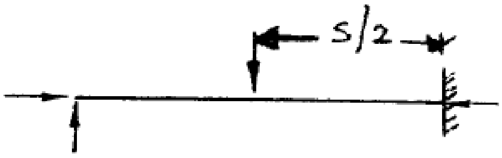
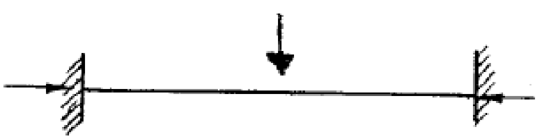
Condition	**Ψ**	**C_m**
	0	1.0
	-0.3	$1\text{-}0.3\sigma_{eb}/\sigma_{e'}$
	-0.4	$1\text{-}0.4\sigma_{eb}/\sigma_{e'}$
	-0.2	$1\text{-}0.2\sigma_{eb}/\sigma_{e'}$
	-0.4	$1\text{-}0.4\sigma_{eb}/\sigma_{e'}$
	-0.6	$1\text{-}0.6\sigma_{eb}/\sigma_{e'}$

Figure 6.2. Ψ Numbers

6.4.4. Stability Checks

6.4.4.1 Lateral Buckling. The steel beams are used in such a manner that their compression flanges are restrained against lateral buckling. When the compression flanges of a beam is long enough and slender enough, it may possibly buckle unless lateral support is provided. Lateral support of the compression flange may be provided with connecting beams or with special members inserted for that purpose.

6.4.4.2 Allowable Stress of Lateral Buckling. The allowable compression stress shall be computed by the following equation for the symmetrical cross sections, loading through the web direction and U sections loading through the major principal axis. However, this value cannot be greater than 0.60 σ_a.

$$\frac{s}{i_y} \leq \sqrt{\frac{30000000C_b}{\sigma_a}} \quad (6.20)$$

$$\sigma_B = \left[\frac{2}{3} - \frac{\sigma_a(s/i_y)^2}{9000000C_0}\right]\sigma_a \leq 0.6\sigma_a \quad (6.21)$$

$$\frac{s}{i_y} \geq \sqrt{\frac{30000000C_b}{\sigma_a}} \quad (6.22)$$

$$\sigma_B = \frac{1000000C_b}{(s/i_y)^2} \quad (6.23)$$

If the compression flange is plate and rectangular cross-section and the cross-section of compression flange is not smaller than the cross-section of tension flange;

$$\sigma_B = \frac{840000C_b}{s.d/F_b} \quad (6.24)$$

s : The unsupported length of the compression flange against rotation and lateral displacement

i_y : The moment of inertia of compression flange and one third of the compression region of the web with respect to the web axis.

F_b : The cross-section area of compression flange

d : Outside to outside distance between flanges

σ_b : Allowable compression stress considering lateral buckling

σ_a : Yielding stress of compression flange

C_b : The coefficient

$$C_b = 1.75 + 1.05\left(\frac{M_1}{M_2}\right) + 0.3\left(\frac{M_1}{M_2}\right) \leq 2.3 \qquad (6.25)$$

M_1 : The smallest moment at the lateral support

M_2 : The biggest moment at the lateral support

If the moment between the lateral supports is greater than the end moments, C_b is equal to 1.0.

7. STUDY CASE FOR COMPARISON OF CODES

7.1. Introduction

In this chapter, an existing building is remodeled by means of SAP2000 structural analysis program and the steel sections such as columns, frame beams, secondary beams used in the building are redesigned according to the EC3 which had been designed according to the TEC 2007 and TS 648. Earthquake loads acting on the structure are calculated according to the TEC 2007 although the design of steel sections is compatible with the EC3. Using EC8 in the calculation of earthquake loads will cause confusion in the selection of steel sections since the calculated base shear will show difference between TEC 2007 and EC 8. Therefore, the difference between EC3 and TS 648 shall be determined clearly without changing the earthquake code.

The occupancy purpose of the structure is depot with dimensions of 30 m * 17 m. The total height of the structure is 39.40 m. The site class of the structure is Z2 and it is located in the first seismic zone. The plans and sections of the structure are given in appendix and 3D model of the structure is given in Figure 7.1.

The seismic loads are fully resisted by eccentrically braced frames (systems of nominal ductility level). By corresponding the axis system seen in the appendix 1, vertical supports are formed by HEM 600, HEM 500, HEB 450, HEB 400, HEA400 steel sections and bracings which assist to the system in order to transfer of seismic loads safely to the frames are designed as 10 inch, 8 inch, 6 inch, 4 inch pipes. Horizontal supports are formed by HEA 340, IPE 500, IPE 400 frame beams which connect vertical supports to each other and castellated secondary beams made up of IPE 300 with height of 450 mm are designed as slab system. Beside to these, 10 cm thick concrete slabs are designed as rigid diaphragm.

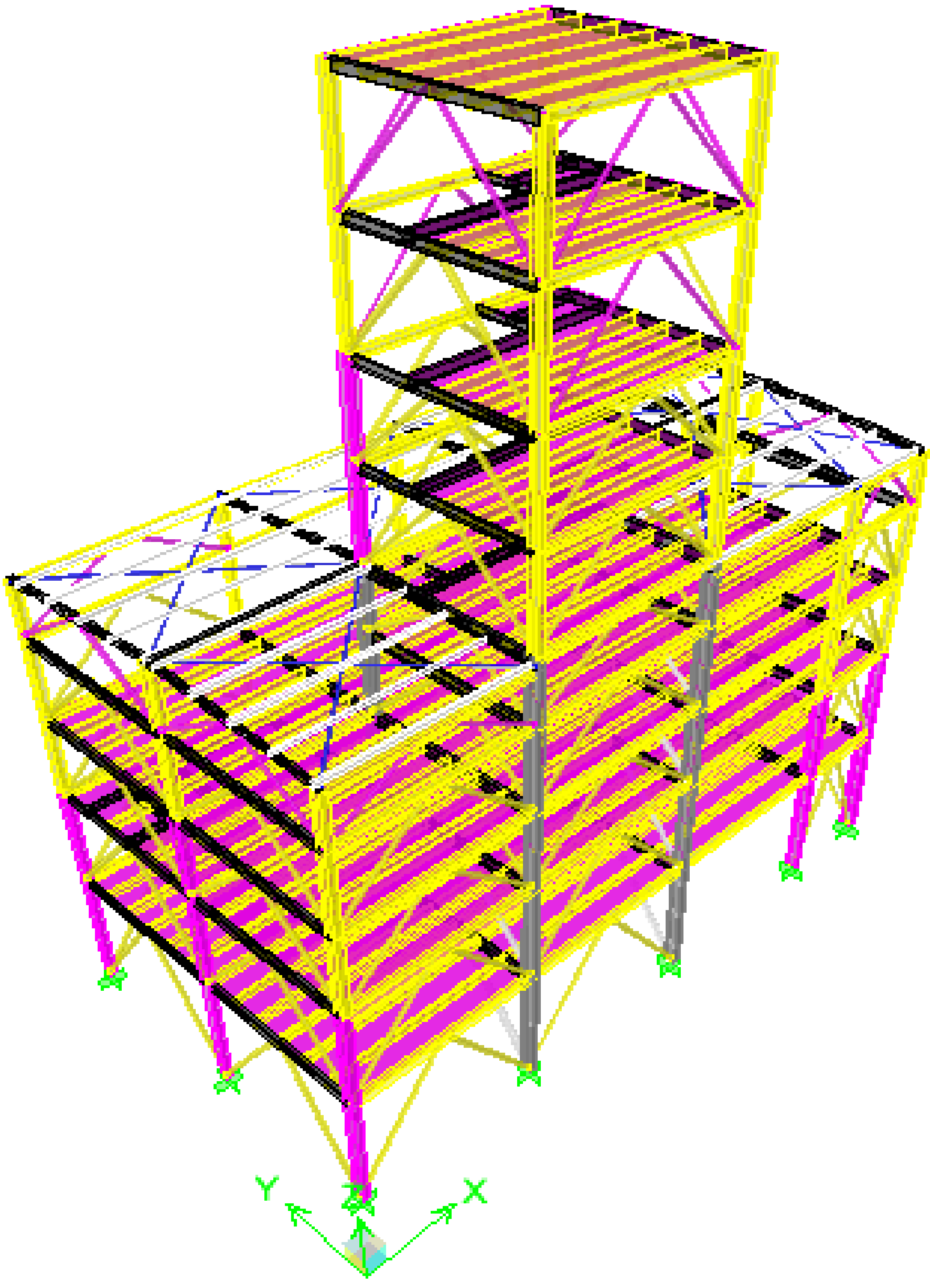


Figure 7.1. 3D Model of Structure

7.2. Loads Acting On The Structure

Structure is to be designed for the most critical values of both sustained and instantaneous loads. Calculations will include the loads the loads listed below.

- Self weight loads
- Topping loads
- Cover and Wall
- Live-Equipment
- Wind load (WL)
- Snow load (SL)
- Earthquake Load (EQ)

Self weight loads are the loads which consist of self weight of structures such as slab, beam, and column. The self weights of all structural elements are calculated internally by the SAP2000 software. The unit weight of concrete material is γ =2.5 tons/m^3 and the unit weight of steel material is γ =7.85 tons/m^3.

Cover and wall loads will be calculated according to properties of layers of cover and type of related wall sections.

Cover …………………….. 0.03 * 2.2 = 0.066 tons/m^2
Cement Finish …….……... 0.02 * 2.0 = 0.040 tons/m^2
Suspended Ceiling ……..... 0.02 * 2.0 = 0.030 tons/m^2

$$\sum g = 0.136 \text{ tons/m}^2$$

Live loads are formed by temporary loads that are not a part of any structure; which are such as humans and vehicles. General live loads that will be applied to all concrete and steel structure used as depot are 0.75 tons/m^2. In the office part of the structure, the live loads are 0.50 tons/m^2. [7]

Wind loads will be considered according to TS 498 and will not be less then values below. In addition Eurocode 1 standard could be used at calculations.

W = 0.11 ton/m²

Surface coefficient: Pressure = 0.80

Suction = 0.40

Snow loads will be considered according to TS 648 and will not be less then values below. In addition Eurocode 1 standard could be used at calculations.

S = 0.10 ton/m²

7.2.1. Earthquake Loads

TEC 2007 is taken into consideration in the calculation of earthquake forces.

Earthquake Zone ………………………………………………. I
Building Importance Factor (I) …………………………….... 1.0
Live Load Participation Factor (n) ……………………………. 0.8
Effective Ground Acceleration Coefficient (A_o) …………...…… 0.4
Structural Behaviour Factor (R) ………………………………. 5
Local Site Class (Z_n) ……………………………………………… Z2
Spectrum Characteristic Periods (T_A) …………………………… 0.15 sec
Spectrum Characteristic Periods (T_B) …………………………… 0.40 sec
Natural Period of Structure (T_x) ………………………………. 0.668 sec
Natural Period of Structure (T_y) ………………………………. 0.598 sec

The seismic weight of structure and natural period of structure are calculated internally by the ETABS software.

If $T > T_b$,

$S(T) = 2.5*(T_b/T)^{0.8}$

$S(T_x) = 2.5*(0.4/0.668)^{0.8} = 1.66$

$S(T_y) = 2.5*(0.4/0.598)^{0.8} = 1.81$

Seismic acceleration is calculated as;

$A(T_1) = A_o*I*S(T)$

$A(T_x) = 0.4*1*1.66 = 0.663$

$A(T_y) = 0.4*1*1.81 = 0.725$

Total equivalent seismic load is calculated as;

$V_T = W*A(T_1) / R_a(T_1) \geq 0.1\ A_o*I*W$

Table 7.1. Earthquake Forces Acting Each Storey

	Level (m)	H_i (m)	m_i	w_i (tons)	w_i*H_i	Fx_i	$Fx_i + \Delta Fn$	Fy_i	$Fy_i + \Delta Fn$
Storey 9	39.50	39.50	4,39	43,11	1702,96	14,81	30,25	16,19	33,06
Storey 8	35.00	35.00	4,81	47,23	1653,16	14,38	14,38	15,71	15,71
Storey 7	30.25	30.25	4,82	47,25	1429,25	12,43	12,43	13,58	13,58
Storey 6	26.20	26.20	4,81	47,19	1236,38	10,75	10,75	11,75	11,75
Storey 5	22.00	22.00	4,96	48,67	1070,70	9,31	9,31	10,18	10,18
Storey 4	19.00	19.00	18,23	178,79	3397,00	29,55	29,55	32,29	32,29
Storey 3	15.00	15.00	45,62	447,48	6712,25	58,38	58,38	63,80	63,80
Storey 2	11.00	11.00	43,49	426,63	4692,91	40,82	40,82	44,60	44,60
Storey 1	6.00	6.00	44,62	437,73	2626,40	22,84	22,84	24,96	24,96
Foundation	0.00								
			$\sum =$	1724.09	24521.01		228.72 tons		249.93 tons

$V_{Tx} = 1724.09*0.663 / 5 = 228.72$ tons $> 0.1*0.4*1*1724.09 = 68.96$ tons

$V_{Ty} = 1724.09*0.725 / 5 = 249.93$ tons $> 0.1*0.4*1*1724.09 = 68.96$ tons

Additional equivalent seismic load acting at the top storey of the structure shall be determined as;

$\Delta F_N = 0.0075*N*V_T$

$\Delta F_{Nx} = 0.0075*9*228.72 = 15.44$ tons

$\Delta F_{Ny} = 0.0075*9*249.93 = 16.87$ tons

7.3. Irregularity Checks

7.3.1. Irregularities In Plan

A1 – Torsional Irregularity: The torsional irregularity ratios for each storey given in Table 7.2.

Table 7.2. Torsional Irregularity

Storey	Loading	Maximum Displacements Δ max (cm)	Δ ort (cm)	η_{bi} Δ max / Δ ort	Limit
Storey 9	E	0,484	0,483	1,00	1.20
Storey 8	E	0,592	0,588	1,01	1.20
Storey 7	E	0,456	0,452	1,01	1.20
Storey 6	E	0,565	0,521	1,08	1.20
Storey 5	E	0,18	0,146	**1,23**	1.20
Storey 4	E	0,367	0,271	**1,35**	1.20
Storey 3	E	0,318	0,314	1,01	1.20
Storey 2	E	0,487	0,418	1,17	1.20
Storey 1	E	0,631	0,621	1,02	1.20
Storey 9	F	0,625	0,621	1,01	1.20
Storey 8	F	0,68	0,675	1,01	1.20
Storey 7	F	0,582	0,577	1,01	1.20
Storey 6	F	0,671	0,661	1,02	1.20
Storey 5	F	0,324	0,169	**1,92**	1.20
Storey 4	F	0,428	0,286	**1,50**	1.20
Storey 3	F	0,28	0,271	1,03	1.20
Storey 2	F	0,385	0,379	1,02	1.20
Storey 1	F	0,448	0,429	1,04	1.20

7.3.2. Irregularities In Elevation

B2 – Interstorey Stiffness Irregularity: The interstorey stiffness irregularity ratios for each storey given in Table 7.3.

Table 7.3. Interstorey Stiffness Irregularity

Storey	H_{storey} (m)	Loading	Δ ort (cm)	Δ ort / H	ήki	ήki	Limit
Storey 9	450	E	0,483	0,0011	0,87	1,00	2.00
Storey 8	475	E	0,588	0,0012	1,11	1,15	2.00
Storey 7	405	E	0,452	0,0011	0,90	0,90	2.00
Storey 6	420	E	0,521	0,0012	**2,55**	1,11	2.00
Storey 5	300	E	0,146	0,0005	0,72	0,39	2.00
Storey 4	400	E	0,271	0,0007	1,00	1,39	2.00
Storey 3	400	E	0,314	0,0008	0,94	1,16	2.00
Storey 2	500	E	0,418	0,0008	0,81	1,06	2.00
Storey 1	600	E	0,621	0,0010	1,00	1,24	2.00
Storey 9	450	F	0,621	0,0014	0,97	1,00	2.00
Storey 8	475	F	0,675	0,0014	1,00	1,03	2.00
Storey 7	405	F	0,577	0,0014	0,91	1,00	2.00
Storey 6	420	F	0,661	0,0016	**2,79**	1,10	2.00
Storey 5	300	F	0,169	0,0006	0,79	0,36	2.00
Storey 4	400	F	0,286	0,0007	1,06	1,27	2.00
Storey 3	400	F	0,271	0,0007	0,89	0,95	2.00
Storey 2	500	F	0,379	0,0008	1,06	1,12	2.00
Storey 1	600	F	0,429	0,0007	1,00	0,94	2.00

Since the height of the structure is greater than 25 m and there is interstorey stiffness irregularity ($ή_{ki}$ > 2.0), equivalent seismic load method cannot be applicable and mode superposition analysis should be applied for the seismic analysis of the structure.

Effective Storey Drift Limit: The torsional irregularity ratios for each storey given in Table 7.4.

Table 7.4. Effective Storey Drift

STOREY	H_{storey} (m)	Loading	Δ max (cm)	Δ max / H	Limit
Storey 9	450	E	0,484	0,0011	0.004
Storey 8	475	E	0,592	0,0012	0.004
Storey 7	405	E	0,456	0,0011	0.004
Storey 6	420	E	0,565	0,0013	0.004
Storey 5	300	E	0,180	0,0006	0.004
Storey 4	400	E	0,367	0,0009	0.004
Storey 3	400	E	0,318	0,0008	0.004
Storey 2	500	E	0,487	0,0010	0.004
Storey 1	600	E	0,631	0,0011	0.004
Storey 9	450	F	0,625	0,0014	0.004
Storey 8	475	F	0,680	0,0014	0.004
Storey 7	405	F	0,582	0,0014	0.004
Storey 6	420	F	0,671	0,0016	0.004
Storey 5	300	F	0,324	0,0011	0.004
Storey 4	400	F	0,428	0,0011	0.004
Storey 3	400	F	0,280	0,0007	0.004
Storey 2	500	F	0,385	0,0008	0.004
Storey 1	600	F	0,448	0,0007	0.004

The maximum value of effective storey drifts, obtained for each earthquake direction at columns of each storey satisfy the limit value, 0.02 / R, equal to 0.004.

There are no local slab abrupt reductions in the plane stiffness and strength of floors and seismic loads are safely transferred to vertical structural elements. Therefore, floor discontinuities irregularity (A2) does not exist.

Since the re-entrant corners in both of the two principal directions in plan do not exist, there is A3 type irregularity in the structure.

There is no case where vertical structural elements are not removed at some stories and supported by beams or gusseted columns underneath. Therefore, irregularity of discontinuity of vertical structural elements does not exist.

7.4. Used Structural Materials

Structural steel	: S275
Yield Strength	: σ = 27 500 t/m^2
Weight per Unit Volume	: 7.85 ton/m^3
Modulus of Elasticity	: E_c = 21 000 000 t/m^2
Poisson Ratio	: υ = 0.3
Coeff. of Thermal Expansion	: α =12 * 10^{-6} ^{o}C
Material Type	: Isotropic

7.5. Design of Structural Members

7.5.1. Design of Columns

Design of columns subject to the bending and axial compression will be examined according to the TS 648 and EuroCode 3.

<u>7.5.1.1 Section Design of Columns According to TS 648.</u> The maximum internal forces are shown at column, CS.09, at axes B/3. The bending moment at major axis is 22.50 tons.m and the axial compression force is 412 tons. The cross section properties of the used steel section, HEM 600, are given below.

Table 7.5. Column Section According to TSE 648

Cross Section	**h (mm)**	**b (mm)**	**t_w (mm)**	**t_f (mm)**	**A (cm^2)**	**d (mm)**	**G (kg/m)**	**r (mm)**
HEM 600	620	305	21	40	363.7	486	285	27
	I_y (cm^4)	**W_y (cm^3)**	**$W_{pl,y}$ (cm^3)**	**i_y (cm)**	**I_z (cm^4)**	**W_z (cm^3)**	**$W_{pl,z}$ (cm^3)**	**i_z (cm)**
	237400	7660	8607	25.55	18980	1244	1920	7.22

Cross section checks should be done according to TEC 2007;

$$\left|{}^{N_d}/_{\sigma_a F}\right| = \frac{395.12}{2.7 * 363.7} = 0.40 > 0.10$$

$$\frac{h}{t_w} \leq 2.08\sqrt{E_s/\sigma_a}\left(2.1 - \left|\frac{N_d}{\sigma_a F}\right|\right) \quad \text{for I sections under bending and axial compression}$$

$$\frac{h}{t_w} = \frac{620}{21} = 29.52 < 2.08\sqrt{21000000/27000}\,(2.1 - 0.4) = 98.61$$

$$\frac{b}{2t} \leq 0.5\sqrt{E_s/\sigma_a} \quad \text{for I sections under bending and axial compression}$$

$$\frac{b}{2t} = \frac{305}{80} = 3.81 < 0.5\sqrt{21000000/27000} = 13.94$$

Member in axial compression and flexure should satisfy the following conditions.

$$\frac{\sigma_{eb}}{\sigma_{bem}} + \frac{C_{mx}\sigma_{bx}}{\left(1 - \frac{\sigma_{eb}}{\sigma_{ex}}\right)\sigma_{Bx}} + \frac{C_{my}\sigma_{by}}{\left(1 - \frac{\sigma_{eb}}{\sigma_{ey}}\right)\sigma_{By}} \leq 1.0$$

$$\sigma_{ex} = \frac{\pi^2 E}{(Ks_b/i_b)^2} * \frac{1}{2.5} = \frac{8290000}{(1.5 * \frac{600}{25.55})^2} * \frac{1}{2.5} = 6.681 \text{ tons/cm}^2$$

$$\sigma_{Bx} = 0.6\sigma_a = 0.6 * 2.700 = 1.62 \text{ tons/cm}^2$$

C_m is equal to 0.85 for frames of lateral displacements are allowed. The minor moment in the direction of simple beams shall be ignored.

$$\sigma_{eb} = \frac{N}{F} = \frac{395.12}{363.70} = 1.086 \text{ tons/cm}^2$$

$$\sigma_{bx} = \frac{M}{W} = \frac{2250}{7660} = 0.294 \text{ tons/cm}^2$$

$$\sigma_{bem} = \frac{\left[1 - \frac{1}{2}\left(\frac{\lambda}{\lambda_p}\right)^2\right]\sigma_a}{n}$$

Since $20 < \lambda < \lambda_p$;

$$n = 1.5 + 1.2\left(\frac{\lambda}{\lambda_p}\right) - 0.2\left(\frac{\lambda}{\lambda_p}\right)^3$$

$$\lambda_p = \sqrt{\frac{2\pi^2 E}{\sigma_a}} = 123.84$$

$$\lambda = \frac{600}{25.55} = 23.48$$

$$n = 1.5 + 1.2\left(\frac{23.48}{123.48}\right) - 0.2\left(\frac{23.48}{123.48}\right)^3 = 1.73$$

$$\sigma_{bem} = \frac{\left[1 - \frac{1}{2}\left(\frac{23.48}{123.48}\right)^2\right] 2.700}{1.73} = 1.533 \text{ tons/cm}^2$$

$$\frac{1.086}{1.533} + \frac{0.85 * 0.294}{\left(1 - \frac{1.086}{6.681}\right) * 1.62} = 0.892 \leq 1.0$$

Since the demand and capacity ratio is smaller than 1.0, the HEM 600 section is suitable.

According to the increased earthquake definition in TEC 2007, the section should have adequate strength capacity under increased loading conditions. The magnification factor, Ω_o, is equal to 2. The load combinations used in the capacity checks of the section are given below.

G + Q + 2E
G + Q + 2F
0.9G + 2E
0.9G + 2F

The column is subjected to axial force, N = 495.275 tons, under the increased earthquake forces.

$$N \leq N_{bp}$$

$$N_{bp} = 1.7\sigma_{bem}F$$

$$\sigma_{bem} = 1.533 \text{ tons/cm}^2$$

$$N = 495.275 \text{ tons} < N_{bp} = 1.7 * 1.533 * 363.7 = 947.84 \text{ tons}$$

7.5.1.2 Section Design of Columns According to Eurocode 3. The maximum internal forces are shown at column, CS.09, at axes B/3. The bending moment at major axis is 8.40 tons.m and the axial compression force is 407 tons. The cross section properties of the used steel section, HEM 450, are given below.

Table 7.6. Column Section According to EC3

Cross Section	**h (mm)**	**b (mm)**	**t_w (mm)**	**t_f (mm)**	**A (cm^2)**	**d (mm)**	**G (kg/m)**	**r (mm)**
HEM 450	478	307	21	40	335.4	344	263	27
	I_y (cm^4)	**W_y (cm^3)**	**$W_{pl,y}$ (cm^3)**	**i_y (cm)**	**I_z (cm^4)**	**W_z (cm^3)**	**$W_{pl,z}$ (cm^3)**	**i_z (cm)**
	131500	5501	6210	19.8	19340	1260	1929	7.59

Class of the cross section should be determined;

$$\varepsilon = \sqrt{235/f_y} = \sqrt{235/270} = 0.933$$

$$\frac{c}{t_f} = \frac{153.5}{40} = 3.84 < 10\varepsilon = 9.33$$

$$\alpha = \frac{1}{2} * \left(\frac{0.3 * A}{d * t_w} + 1\right) = \frac{1}{2} * \left(\frac{0.3 * 335.4}{34.4 * 2.1} + 1\right) = 1.20 > 0.5$$

$$\frac{d}{t_w} = \frac{344}{21} = 16.38 < \frac{396\varepsilon}{13\alpha - 1} = \frac{396 * 0.933}{(13 * 1.20) - 1} = 25.31$$

Class 1 steel section.

The bending strength check should be done;

$$M_{el,d} = \frac{W_{el,d} * \sigma_a}{\gamma_{M0}} = \frac{5501 * 27}{1.1} = 1350 \text{ kN.m}$$

$$M_{s,d} = 84 \text{ kN.m} < M_{el,d} = 1350 \text{ kN.m}$$

Member in axial compression and flexure should satisfy the following conditions.

$$\frac{N_{s,d}}{\chi_{min} A\sigma_a/\gamma_{M1}} + \frac{k_y M_{y,sd}}{W_{y,pl}\sigma_a/\gamma_{M1}} + \frac{k_z M_{z,sd}}{W_{z,pl}\sigma_a/\gamma_{M1}} \leq 1.0$$

Lateral torsional buckling check should be done;

$$\lambda = \frac{600}{19.8} = 30.30$$

$$\lambda_1 = \pi * \sqrt{\frac{E}{\sigma_A}} = 87.61$$

$$\beta_A = 1.0$$

$$\lambda^- = \frac{\lambda}{\lambda_1} * \sqrt{\beta_A} = \frac{30.30}{87.61} * \sqrt{1.0} = 0.346$$

$$\frac{h}{b} = \frac{478}{307} = 1.56 > 1.2 \quad \textit{and} \quad t_f = 40 \text{ mm} \leq 40 \text{ mm}$$

Therefore, buckling curve is "a" type.

$$\alpha = 0.21$$

$$\emptyset = \frac{1}{2} * [1 + \alpha * (\lambda^- - 0.2) + \lambda^{-2}] = \frac{1}{2} * [1 + 0.21 * (0.346 - 0.2) + 0.346^2] = 0.552$$

$$\chi = \frac{1}{\emptyset + \sqrt{\emptyset^2 - \lambda^{-2}}} \leq 1$$

$$\chi = \frac{1}{0.552 + \sqrt{0.552^2 - 0.346^2}} = 1.018$$

$$k_y = 1 - \frac{\mu_y N_{sd}}{\chi_y A \sigma_a} \leq 1.5$$

$$\mu_y = \overline{\lambda}\left(2\beta_{My} - 4\right) + \left[\frac{W_{pl,y} - W_{el,y}}{W_{el,y}}\right] \leq 0.90 \qquad \beta_{My} = 2.167$$

$$\mu_y = 0.346(2 * 2.167 - 4) + \left[\frac{6210 - 5501}{5501}\right] = 0.25$$

$$k_y = 1 - \frac{0.25 * 4070}{1 * 335.4 * 27} = 0.888$$

$$\frac{4070}{1.0*335.4*27/1.1}+\frac{0.748*84}{6210*27/1.1}=0.536$$

Since the demand and capacity ratio is smaller than 1.0, the HEM 450 section is suitable.

7.5.2. Design of Beams

Design of frame beams and secondary beams subject to the bending and shear will be examined according to the TS 648 and Eurocode 3.

7.5.2.1 Section Design of Frame Beams According to TSE 648. The maximum internal forces are shown at beam, KK306 at third storey. The bending moment at major axis is 31.08 tons.m and the shear force is 22.57 tons. The cross section properties of the used steel section, IPE 500, are given below.

Table 7.7. Beam Section According to TSE 648

Cross Section	**h (mm)**	**b (mm)**	**t_w (mm)**	**t_f (mm)**	**A (cm^2)**	**d (mm)**	**G (kg/m)**	**r (mm)**
IPE 500	500	200	10.2	16	115.5	426	90.7	21
	I_y (cm^4)	**W_y (cm^3)**	**$W_{pl,y}$ (cm^3)**	**i_y (cm)**	**I_z (cm^4)**	**W_z (cm^3)**	**$W_{pl,z}$ (cm^3)**	**i_z (cm)**
	48200	1928	2194	20.43	2142	214.2	336	4.31

Cross section checks should be done according to TEC 2007;

$$\frac{h}{t_w}\leq 5.0\sqrt{E_s/\sigma_a}\quad \text{for I sections under bending}$$

$$\frac{h}{t_w}=\frac{500}{10.2}=49.02<5.0\sqrt{21000000/27000}=139.44$$

$$\frac{b}{2t}\leq 0.5\sqrt{E_s/\sigma_a}\quad \text{for I sections under bending}$$

$$\frac{b}{2t} = \frac{200}{32} = 6.25 < 0.5\sqrt{21000000/27000} = 13.94$$

Stress checks should be done,

$$\sigma_{max} = \frac{M_{max}}{W_x} = \frac{3108}{1928} = 1.612 \text{ tons/cm}^2 < \sigma_{em} = 1.62 \text{ tons/cm}^2$$

$$\tau_{max} = \frac{Q_{max}}{F_g} = \frac{22.57}{42.6 * 1.02} = 0.519 \text{ tons/m}^2 < \sigma_{em} = 0.935 \text{ tons/cm}^2$$

$$\sigma_{v,m} = \sqrt{1.612^2 + 3 * 0.519^2} = 1.846 \text{ tons/cm}^2$$

$$\sigma_{v,m} < 0.75\sigma_a = 2.025 \text{ tons/m}^2$$

Maximum deflection should be checked;

$$\delta_{max} = 0.873 \text{ cm} < {}^{L}/_{300} = {}^{750}/_{300} = 2.5 \text{ cm}$$

Local buckling check should be done due to shear force;

$$c_b = 1.75 + 1.05 * \left(\frac{-28.66}{31.08}\right) + 0.30 * \left(\frac{-28.66}{31.08}\right) = 1.04 > 0.67$$
$$c_b = 0.67$$

The frame beam is restrained at 5 points by secondary beams and the length between the restrained points of beam;

s = 1.250 m

The radius of gyration of compression flange is calculated as given below.

$$i_y = \sqrt{\frac{1/_{12} * t_b * {b_b}^3}{t_b * b_b + 1/_6 * t_g * (d - 2t_b)}}$$

$$i_y = \sqrt{\frac{1/_{12} * 1.6 * 20^3}{1.6 * 20 + 1/_6 * 1.02 * 42.6}} = 5.21 \text{ cm}$$

$$\frac{s}{i_y} = \frac{125}{5.21} = 24 < \sqrt{\frac{30 * 10^6 * 0.67}{2700}} = 86.28$$

Therefore;

$$\sigma_B = \left[\frac{2}{3} - \frac{2700 * 24^2}{90 * 10^6 * 0.67}\right] * 2700 = 1.739 \text{ tons/cm}^2 > 0.6\sigma_a = 1.620 \text{ tons/cm}^2$$

$$\frac{M_{max}}{W_x} = 1.612 \text{ tons/cm}^2 < \ \sigma_B = 1.739 \text{ tons/cm}^2$$

There is no local buckling.

According to the increased earthquake definition in TEC 2007, the section should have adequate strength capacity under increased loading conditions. The magnification factor, Ω_o, is equal to 2. The load combinations used in the capacity checks of the section are given below.

G + Q + 2E
G + Q + 2F
0.9G + 2E
0.9G + 2F

The column is subjected to bending moment, M_x = 36.33 tons.m, under the increased earthquake forces.

$$M_x \leq M_p$$

$$M_p = W_p \sigma_a$$

$$M_x = 3633 \text{ tons.cm} < M_p = 2194 * 2.7 = 5924 \text{ tons.cm}$$

<u>7.5.2.2 Section Design of Frame Beams According to Eurocode 3.</u> The maximum internal forces are shown at beam, KK306 at third storey. The bending moment at major axis is 45.23 tons.m and the shear force is 32.69 tons. The chosen steel section is IPE 500.

Class of the cross section should be determined;

$$\varepsilon = \sqrt{235/f_y} = \sqrt{235/270} = 0.933$$

$$\frac{c}{t_f} = \frac{100}{16} = 6.25 < 11\varepsilon = 10.26$$

$$\alpha = \frac{1}{2} * \left(\frac{0.3 * A}{d * t_w} + 1\right) = \frac{1}{2} * \left(\frac{0.3 * 115.5}{42.6 * 1.02} + 1\right) = 0.899 > 0.5$$

$$\frac{d}{t_w} = \frac{426}{10.2} = 41.76 > \frac{456\varepsilon}{13\alpha - 1} = \frac{456 * 0.933}{(13 * 0.899) - 1} = 39.81$$

Class 3 steel section.

The shear strength check should be done;

$$V_{sd} \leq V_{pl,Rd} = \frac{A_v * f_y}{\sqrt{3} * \gamma_{M0}}$$

$$A_v = 1.04 * h * t_w = 1.04 * 50 * 1.02 = 53.04 \text{ cm}^2$$

$$V_{pl,Rd} = \frac{53.04 * 27}{\sqrt{3} * 1.1} = 752\text{kN} > V_{sd} = 327\text{kN}$$

$$0.5V_{pl,Rd} = 0.5 * 752 = 376\text{kN} > V_{sd} = 327\text{kN}$$

Therefore, there is no need to decrease the bending strength of the section due to the shear force.

The bending strength check should be done;

$$M_{sd} \leq M_{c,Rd}$$

$$M_{c,Rd} = \frac{W_{el} * f_y}{\gamma_{M0}} = \frac{1928 * 27}{1.1} = 473 \text{ kN.m} > M_{sd} = 452 \text{ kN.m}$$

Shear buckling resistance should be checked;

$$\frac{d}{t_w} = \frac{426}{10.2} = 41.76 < 69\varepsilon = 64.38$$

Therefore there is no need to check for resistance to shear buckling.

<u>7.5.2.3 Section Design of Secondary Beams According to TSE 648.</u> The maximum bending moment for secondary beams is 11.68 tons.m and the maximum shear force is 5.84 tons. The cross section properties of the used steel section, IPE 300 castellated beam with height of 450 mm, are given below.

Table 7.8. Secondary Beam Section According to TSE 648

Cross Section	**h (mm)**	**b (mm)**	**t_w (mm)**	**t_f (mm)**	**A (cm^2)**	**d (mm)**	**G (kg/m)**	**r (mm)**
IPE 300 Castel h=450	400	180	8.6	13.5	84.46	331	66.3	21
	I_y (cm^4)	**W_y (cm^3)**	**$W_{pl,y}$ (cm^3)**	**i_y (cm)**	**I_z (cm^4)**	**W_z (cm^3)**	**$W_{pl,z}$ (cm^3)**	**i_z (cm)**
	23130	1156	1307	16.55	1318	146.4	229	3.95

Cross section checks should be done according to TEC 2007;

$$\frac{h}{t_w} \leq 5.0\sqrt{E_s/\sigma_a} \quad \text{for I sections under bending}$$

$$\frac{h}{t_w} = \frac{400}{8.6} = 46.51 < 5.0\sqrt{21000000/27000} = 139.44$$

$\frac{b}{2t} \leq 0.5\sqrt{E_s/\sigma_a}$ for I sections under bending

$$\frac{b}{2t} = \frac{180}{27} = 6.67 < 0.5\sqrt{21000000/27000} = 13.94$$

Stress checks should be done,

$$\sigma_{max} = \frac{M_{max}}{W_x} = \frac{1168}{799} = 1.462 \text{ tons/cm}^2 < \sigma_{em} = 1.62 \text{ tons/cm}^2$$

$$\tau_{max} = \frac{Q_{max}}{F_g} = \frac{5.84}{9.13} = 0.640 \text{ tons/m}^2 < \sigma_{em} = 0.935 \text{ tons/cm}^2$$

Maximum deflection should be checked;

$$\delta_{max} = 2.0 \text{ cm} < {}^{L}/_{300} = {}^{800}/_{300} = 2.67 \text{ cm}$$

Local buckling check should be done due to shear force;

$$c_b = 1.75 + 1.05 + 0.30 = 1.0$$

The unrestrained length of compression flange;

s = 8.0 m

The radius of gyration of compression flange is calculated as given below.

$$i_y = \sqrt{\frac{{}^{1}/_{12} * t_b * {b_b}^3}{t_b * b_b + {}^{1}/_{6} * t_g * (d - 2t_b)}}$$

$$i_y = \sqrt{\frac{{}^{1}/_{12} * 1.07 * 15^3}{1.07 * 15 + {}^{1}/_{6} * 0.71 * 37.29}} = 3.83 \text{ cm}$$

$$\frac{s}{i_y} = \frac{800}{3.83} = 209 > \sqrt{\frac{30 * 10^6 * 1}{2700}} = 105$$

Therefore;

$$\sigma_B = \left[\frac{10 * 10^6 * 1}{209^2}\right] = 0.229 \text{ tons/cm}^2 < 0.6\sigma_a = 1.620 \text{ tons/cm}^2$$

The restrained length of the compression length should be decreased. The secondary beam is restrained at 2 points and the length between the restrained points of beam;

$$s = 800/3 = 266.67 \text{ cm}$$

$$\frac{s}{i_y} = \frac{266.67}{3.83} = 69.7 < \sqrt{\frac{30 * 10^6 * 1}{2700}} = 105$$

Therefore;

$$\sigma_B = \left[\frac{2}{3} - \frac{2700 * 69.7^2}{90 * 10^6 * 1}\right] * 2700 = 2.306 \text{ tons/cm}^2 > 0.6\sigma_a = 1.620 \text{ tons/cm}^2$$

$$\frac{M_{max}}{W_x} = 1.462 \text{ tons/cm}^2 < \sigma_B = 2.306 \text{ tons/cm}^2$$

There is no local buckling.

7.5.2.4 Section Design of Secondary Beams According to Eurocode 3. The maximum bending moment for secondary beams is 16.90 tons.m and the maximum shear force is 8.45 tons. The chosen steel section is IPE 300 castellated beam with height of 450 mm.

Class of the cross section should be determined;

$$\varepsilon = \sqrt{235/f_y} = \sqrt{235/270} = 0.933$$

$$\frac{c}{t_f} = \frac{75}{10.7} = 7.01 < 11\varepsilon = 10.26$$

$$\alpha = \frac{1}{2} * \left(\frac{0.3 * A}{d * t_w} + 1\right) = \frac{1}{2} * \left(\frac{0.3 * 41.23}{45 * 0.71} + 1\right) = 0.694 > 0.5$$

$$\frac{d}{t_w} = \frac{450}{7.1} = 63.38 > \frac{456\varepsilon}{13\alpha - 1} = \frac{456 * 0.933}{(13 * 0.694) - 1} = 53.04$$

Class 3 steel section.

The shear strength check should be done;

$$V_{sd} \leq V_{pl,Rd} = \frac{A_v * f_y}{\sqrt{3} * \gamma_{M0}}$$

$$A_v = 1.04 * h * t_w = 1.04 * 45 * 0.71 = 33.23 \text{ cm}^2$$

$$V_{pl,Rd} = \frac{33.23 * 27}{\sqrt{3} * 1.1} = 471\text{kN} > V_{sd} = 84.5\text{kN}$$

$$0.5V_{pl,Rd} = 0.5 * 471 = 236\text{kN} > V_{sd} = 84.5\text{kN}$$

Therefore, there is no need to decrease the bending strength of the section due to the shear force.

The bending strength check should be done;

$$M_{sd} \leq M_{c,Rd}$$

$$M_{c,Rd} = \frac{W_{el} * f_y}{\gamma_{M0}} = \frac{799 * 27}{1.1} = 196 \text{ kN.m} > M_{sd} = 169 \text{ kN.m}$$

Shear buckling resistance should be checked;

$$\frac{d}{t_w} = \frac{450}{7.1} = 63.38 < 69\varepsilon = 64.38$$

Therefore there is no need to check for resistance to shear buckling.

7.5.3. Design of Braces

7.5.3.1 Section Design of Braces According to TSE 648. The maximum axial compression force for braces is 65.34 tons. The cross section properties of the used steel section, 10" pipe, are given below.

Table 7.9. Bracing Section According to TSE 648

Cross Section	**d (mm)**	**t (mm)**	**A (cm^2)**	**I (cm^4)**	**W (cm^3)**	**i (cm)**
10" PIPE	273.2	8	66.65	5865	429.35	9.38

Cross section checks should be done according to TEC 2007;

$$\frac{D}{t} \leq 0.08\frac{E_s}{\sigma_a} \quad \text{for pipe sections under axial compression}$$

$$\frac{D}{t} = \frac{273.2}{8} = 34.15 < 0.08\frac{21000000}{27000} = 62.22$$

Stress checks should be done,

$$s_k = K * s = 1 * 707.5 = 707.5 \text{ cm}$$

K=1 (both ends of brace rotate and translation is restrained)

$$\lambda = \frac{s_k}{i} = \frac{707.5}{9.381} = 75.42 \quad \rightarrow \quad \omega = 1.54$$

$$\lambda \leq 4.0\sqrt{E_s/\sigma_a} = 4.0 * \sqrt{21000000/27000} = 111.5$$

$$\sigma = \frac{\omega P}{F} = \frac{1.54 * 65.34}{66.65} = 1.510\ \text{tons/cm}^2 < 0.6\sigma_a = 1.620\ \text{tons/cm}^2$$

7.5.3.2 Section Design of Braces According to Eurocode 3. The maximum axial compression force for braces is 55.55 tons. The cross section properties of the used steel section, 8” pipe, are given below.

Table 7.10. Bracing Section According to EC 3

Cross Section	**d (mm)**	**t (mm)**	**A (cm^2)**	**I (cm^4)**	**W (cm^3)**	**i (cm)**
8" PIPE	219.1	6.3	42.12	2386	218	7.53

Axial compression strength of the cross section should be checked;

$$N_{sd} \leq N_{c,Rd}$$

$$N_{c,Rd} = {}^{A\sigma_a}/_{\gamma_{M0}}$$

$$N_{c,Rd} = {}^{42.12 * 27}/_{1.1} = 1034\ \text{kN}$$

Buckling strength shall be computed;

$$N_{b,Rd} = {}^{\chi\beta_A A\sigma_a}/_{\gamma_{M1}}$$

$$\beta_A = 1.0$$

$$\lambda = 75.42$$

$$\lambda_1 = \pi * \sqrt{\frac{E}{\sigma_A}} = 87.61$$

$$\lambda^- = \frac{\lambda}{\lambda_1} * \sqrt{\beta_A} = \frac{75.42}{87.61} * \sqrt{1.0} = 0.861$$

Buckling curve is “a” type.

$\alpha = 0.21$

$$\lambda_1 = \pi * \sqrt{\frac{E}{\sigma_A}} = 87.61$$

$$\emptyset = \frac{1}{2} * [1 + \alpha * (\lambda^{-} - 0.2) + \lambda^{-2}] = \frac{1}{2} * [1 + 0.21 * (0.861 - 0.2) + 0.861^2] = 0.94$$

$$\chi = \frac{1}{\emptyset + \sqrt{\emptyset^2 - \lambda^{-2}}} \leq 1$$

$$\chi = \frac{1}{0.94 + \sqrt{0.94^2 - 0.861^2}} = 0.759$$

$$N_{b,Rd} = {}^{0.759 * 1 * 42.12 * 27}\!/_{1.1} = 785\ \mathrm{kN}$$

$$N_{sd} = 556\ \mathrm{kN} < N_{b,Rd} = 785\ \mathrm{kN}$$

8. CONCLUSIONS AND RECOMMENDATIONS

In this study, the differences in the design of steel structures according to the building codes used in Europe and Turkey were presented. Eurocodes are widely used throughout the world instead of local buildings codes to improve the standardization in civil engineering. In the European Union entry process, the examination of Eurocodes is very essential for the adoption of the construction sector in Turkey to the construction sector in Europe.

The general principle of earthquake resistant design according to TEC 2007 is to prevent structural and non-structural elements of buildings from any damage in low intensity earthquakes; to limit the damage in structural and non-structural elements to repairable levels in medium intensity earthquakes, and to prevent the overall or partial collapse of buildings in high intensity earthquakes in order to avoid the loss of life.

Structures according to EC 8 are to be designed and constructed to withstand the design seismic action without local or global collapse, thus retaining its structural integrity and a residual load bearing capacity after the seismic events. The structures are be designed and constructed to withstand a seismic action having larger probability of occurrence than the design seismic action, without the occurrence of damage and the associated limitations of use.

In both earthquake codes, seismic action is represented with elastic design acceleration spectrum. And also, elastic acceleration spectrum may be determined by considering local seismic and site conditions. In both earthquake codes, the elastic response spectrum reduced by dividing to the behaviour factor due to the capacity of the structure to dissipate energy through mainly ductile behaviour. Beside these, the building importance factor is introduced for determination of spectral acceleration coefficient and elastic response spectra will differ according to the earthquake zone of a country. The building structures are categorized into being regular and non-regular to determine the method of earthquake analysis, equivalent seismic load method and mode combination method.

Steel structures are designed according to the ultimate limit state theory in EC 3 and according to the class of steel section; elastic or plastic analysis method shall be selected. However, only elastic analysis method is permitted in TSE 648. The elastic section modulus or the plastic section modulus is used to determine the strength of the steel member according to the class of steel section. Classification of section is an important concept in EC 3 due to selection of design method. Steel sections are divided into four classes by taking into account the d/t and c/t_w ratios. The plastic analysis is permitted for the class 1 and 2 cross sections, and elastic analysis is permitted for the class 3 and 4 cross sections. There is no such a classification in TSE 648 since the steel sections are designed according to the elastic analysis.

The partial safety factors for design actions and the partial safety factors for resistances are introduced in EC 3. According to the type of the action, the safety factor will take different values. The partial safety factors for resistances shall be used to determine the design strength of steel sections. However, allowable stress values shall be used in TSE 648.

EC 3 is more comprehensive and complicated than TSE 648 in terms of design principle. However, TSE 648 should be reviewed and republished by taking account the today’s innovations. The design methods introduced in EC 3 are more detailed, descriptive and more systematic which do not give initiation to designer. Under certain loading conditions, the more economical steel sections can be chosen according to the design results in EC 3 since the design of sections is performed by limit state theory. The design sections according to the EC 3 and TSE 648 are given in Table 8.1.

Table 8.1. Design Sections

	TSE 648	**EC 3**
Member	**Section**	**Section**
Columns	HEM 600	HEM 450
Frame Beams	IPE 500	IPE 500
Secondary Beams	IPE 300 Castel h=450	IPE 300 Castel h=450
Bracings	10" Pipe	8" Pipe

For the case study reported in the thesis, the sections given in Table 8.1. indicate that the plastic analysis cannot be used due to the slender cross sections according to the EC 3. For example, HEA cross sections are used for columns to determine the difference between EC 3 and TSE 648. HEA cross sections are compact section with high values of width-thickness ratios. However, IPE sections are used for frame beams and due to the slenderness of IPE sections, the elastic analysis should be used instead of plastic analysis. Therefore, there is no difference between codes for the design of frame beams.

In this respect, the civil engineer of the future should be fully equipped with the proper understanding of the limit state theory concept and be prepared for the limit state design. TEC and TS 648 should be gradually removed and Eurocodes should be translated into Turkish and put into action due to the deficiencies in the regional building codes.

APPENDIX: DRAWINGS

The drawings of the structure stated in case study are attached to the back page of the thesis.

REFERENCES

1. Eurocode 8, *Design of Steel Structures for Earthquake Resistance,* Part 1, CEN, Brussels, 1998.

2. Turkish Earthquake Code 2007, *Specification of Buildings to be Built in Disaster Areas,* The Ministry of Public Works and Settlement, Ankara, 2007.

3. Eurocode 1, *Actions on Structures*, CEN, Brussels, 1991.

4. Celep, Z., *Deprem Mühendisliğine Giriş ve Depreme Dayanıklı Yapı Tasarımı*, Beta Dağıtım, Istanbul, 2004

5. Eurocode 3, *Design of Steel Structures,* Part 1-1, CEN, Brussels, 1993.

6. TSE 648, *Çelik Yapıların Hesap ve Yapım Kurallaru,* Türk Standartları Enstitiüsü, Ankara, 1980.

7. TSE 498, *Design Loads for Buildings*, Turkish Standards Institution, Ankara, 1987.

Odabaşı, Y., *Ahşap ve Çelik Yapı Elemanları*, Beta Yayınları, İstanbul, 1997.

Deren, H., *Çelik Yapılar*, Çağlayan Kitapevi, İstanbul, 2003.