GOVERNMENT OF DUBAI

DUBAI MUNICIPALITY

SEISMIC DESIGN CODE FOR DUBAI

Dubai Municipality

2013

A

SEISMIC ANALYSIS AND DESIGN REQUIREMENTS A
FOR BUILDINGS A

A

A

A

A

A

A
A
A
A
A
A
A
A
A
A
A
A
A
A
A
A
A
A
A
A
A
A
A
A
A
A

A

CHAPTER 1
GENERAL REQUIREMENTS

1.1. SCOPE, NOTATIONS, REFERENCE STANDARDS

1.1.1. Scope

1.1.1.1 – This standard covers the seismic analysis and design requirements of reinforced concrete and steel building structures to be constructed within boundaries of Emirate of Dubai.

1.1.1.2 – This standard is applicable to low- to medium rise buildings as well as to tall buildings, as defined in **1.3.1**.

a) All parts of this standard excluding **Chapters 6** and **7** are applicable to low- to medium rise buildings.

b Special seismic analysis and design requirements applicable to tall buildings are given in **Chapters 6** and **7**. Parts of sections **1.2** and **1.3** of **Chapter 1** as well as parts of **Chapter 2** that are referred to in **Chapter 6** are also applicable to tall buildings.

1.1.1.3 – Civil engineering structures other than buildings are outside the scope of this code.

1.1.1.4 – Base-isolated buildings as well as buildings equipped with active or passive control systems and devices are outside the scope of this code.

1.1.2. Notations

A = Gross area of seismic link
A_c = Total effective area of structural walls in the first storey for empirical calculation of predominant period in the eartquake direction [m^2]
A_e = Maximum acceleration acting on nonstructural element or component
A_j = Effective area of the j'th structural walls in the first storey for empirical calculation of predominant period in the eartquake direction [m^2]
A_{pl} = Horizontal area of the plate
A_{st} = Area of one leg of the transverse reinforcement; area of stiffener
B_e = Amplification factor for nonstructural element or component
b = Width of the flange
b_b = Width of composite beam or bearing width of the concrete of the slab on the column
b_c = Cross sectional dimension of column
b_e = Partial effective width of flange on each side of the steel web
b_{eff} = Effective flange width of beam in tension at the face of a supporting column; total effective width of concrete flange
b_i = Distance between consecutive bars engaged by a corner of a tie or a cross-tie in a column
b_o = Width of a confined core in a column or in the boundary element of a wall (to centerline of hoops)
b_w = Width of the web of a beam
b_{wo} = Web thickness of wall
C_t = Empirical factor for the calculation of predominant period in the earthquake

direction **A**
D_i A = Torsion amplification factor at i'th storey A
D_o A = Diameter of confined core in a circular column A
d A = Effective depth of section A
d_{bL} A = Longitudinal bar diameter A
d_{bw} A = Diameter of hoop A
d_{fi} A = Fictitious displacements at i'th storey used in Rayleigh quotient A
d_{ji} A = Reduced storey displacement of the j'th vertical element at i'th storey A
E_a A = Modulus of Elasticity of steel A
E_{cm} A = Mean value of Modulus of Elasticity of concrete in accordance with EN 1992-1- A
1:2004 A
E_d A *D*esign value of an action effect A
E_{di} A = Design value of the action effect on the zone or element *i* in the seismic design A
situation A
E_E A = Action effect due to seismic load A
E_{Fd} A *D*esign value of an action effect on the foundation A
E_G A = Action effect due to dead load A
$E_{F,E}$ A = Action effect from the analysis of the design seismic action A
$E_{F,G}$ A = Action effect due to the non-seismic actions included in the combination of A
actions for the seismic design situation A
$E_{Q\,A}$ = Action effect due to live load A
e A = Length of seismic link A
F_{fi} A = Fictitious forces at i'th storey used in Rayleigh quotient A
F_i A = Equivalent seismic load acting at i'th storey A
F_{xin} A = Modal seismic load in the n'th mode acting at i'th storey in x direction A
F_{yin} A = Modal seismic load in the n'th mode acting at i'th storey in y direction A
$F_{\theta in}$ A Modal seismic torque in the n'th mode acting at i'th storey around the vertical axis A
passing through mass centre A
$_{cd}$ A = Design value of concrete compressive strength A
$_{ce}$ A = Exopected value of concrete compressive strength A
$_{ck}$ A Characteristic value of concrete compressive strength A
$_{ctm}$ A = Mean value of concrete tensile strength A
$_{y}$ A = Nominal value of steel yield strength A
$_{yd}$ A = Design value of steel yield strength A
$_{ye}$ A = Expected value of steel yield strength A
$_{ydf}$ A = Design yield strength of steel in the flange A
$_{yd,v}$ A = Design value of yield strength of the vertical web reinforcement A
$_{ydw}$ A = Design strength of web reinforcement A
$_{yk}$ A = Characteristic value of steel yield strength A
$_{yld}$ A = Design value of yield strength of longitudinal reinforcement A
$_{ywd}$ A = Design value of yield strength of transverse reinforcement A
$_{e}$ A = Equivalent seismic load acting at the mass centre of nonstructural element A
G_i A = Total dead load at i'th storey of building A
g A = Acceleration of gravity 9.81 m/s^2 A
H_i A = Total height of building measured from the top foundation level A
(In buildings with rigid peripheral basement walls, total height of building A
measured from the top of the ground floor level) [m] A
H_N A = Total height of building measured from the top foundation level A
(In buildings with rigid peripheral basement walls, total height of building A
measured from the top of the ground floor level) [m] A

H_w A = Total wall height measured from top foundation level or ground floor level A
h A = Cross sectional depth A
h_b A = Depth of composite beam A
h_c A = Cross sectional depth of a column in a given direction A
h_f A = Flange depth A
h_i A = Height of i'th storey of building [m] A
h_o A = Depth of confined core in a column (to centerline of hoops) A
h_w A = Depth of beam A
I A = Building Importance Factor A
I_a A = Second moment of area of the steel section part of a composite section, with A respect to the centroid of the composite section A
I_c A = Second moment of area of the concrete part of a composite section, with respect to A the centroid of the composite section I_{eq} equivalent second moment of area of the A composite section A
I_e A = Element nonstructural Importance Factor A
I_s A = Second moment of area of the rebars in a composite section, with respect to the A centroid of the composite section A
k_e A = Effective stiffness coefficient of the nonstructural element or component. A
k_r A = Rib shape efficiency factor of profiled steel sheeting A
k_t A = Reduction factor of design shear resistance of connectors in accordance with EN A 1994-1-1:2004 A
L A = Beam span A
l_c A = Column height A
l_{cl} A = Clear length of a beam or a column A
l_{cr} A = Length of critical region A
l_w A = Length of wall cross-section A
l_{wj} A = Plan length of j'th structural wall or a piece of coupled wall at the first story A
M_{Ed} A = Design bending moment obtained from analysis for the seismic design situation A
$M_{Ed,E}$ A= Bending moment due to design seismic action A
$M_{Ed,G}$ A= Bending moment due to non-seismic actions in seismic design situation A
$M_{Ed,W}$ = Design bending moment obtained from analysis at the base of the wall for the A seismic design situation A
M_i A = i'th storey mass of building $M_i \quad W_i / g$ A
$M_{i,d}$ A = End moment of a beam or column for calculating capacity design shear A
M_N A = Nominal plastic moment of RC section A
M_n A = Modal mass of the n'th natural vibration mode A
$M_{pl,Rd}$ = Design value of plastic moment resistance **A**
$M_{pl,Rd,A}$ = Design value of plastic moment resistance at end A of a member **A**
$M_{pl,Rd,B}$ = Design value of plastic moment resistance at end B of a member A
$M_{pl,Rd,c}$ = Design value of plastic moment resistance of column, taken as lower bound and A computed taking into account the concrete component of the section and only the A steel components of the section classified as ductile **A**
$M_{Rb,i}$ A= Design moment resistance of a beam at end i A
$M_{Rc,i}$ A= Design moment resistance of a column at end i **A**
M_{Rd} A = Design bending moment resistance A
$M_{Rd,W}$ = Design bending moment resistance at the base of the wall **A**
M_{xn} A = Effective participating mass of the n'th natural vibration mode of building in A the x earthquake direction considered **A**
M_{yn} A = Effective participating mass of the n'th natural vibration mode of building in A

the y earthquake direction considered **A**
M_t A = Total mass of building (M_t W_t / g A
$M_{U,Rd,b}$ = Upper bound plastic resistance of beam, computed taking into account the concrete A component of the section and all the steel components in the section, including A those not classified as ductile A
M_Y A = Bending moment corresponding to the state of first-yield in RC section A
m_e A Nonstructural element mass A
N A = Total number of stories of building from the foundation level A (In buildings with rigid peripheral basement walls, total number of stories from the A ground floor level) A
N_{Ed} A = Design axial force obtained from analysis for the seismic design situation A
$N_{Ed,E}$ A= Axial force due to design seismic action A
$N_{Ed,G}$ A= Axial force due to non-seismic actions in seismic design situation A
$N_{pl,Rd}$ A= Design value of yield resistance in tension of the gross cross-section of a member in A accordance with EN 1993-1-1:2004 A
n A = Steel-to-concrete modular ratio for short term actions A
n_1 A = Live Load Mass Reduction Factor A
n_2 A = Live Load Participation Factor A
Q_{Cx} A = Response quantity obtained by modal combination in Response Spectrum A Method for an earthquake in x direction A
Q_{Cy} A Response quantity obtained by modal combination in Response Spectrum A Method for an earthquake in y direction A
Q_D A = Design response quantity due to seismic action A
Q_i A = Total live load at i'th storey of building A
Q_{Sx} A = Scaled response quantity obtained by modal combination in Response Spectrum A Method for an earthquake in x direction A
Q_{Sy} A = Scaled response quantity obtained by modal combination in Response Spectrum A Method for an earthquake in y direction A
Q_x A Response quantity obtained in Equivalent Seismic Load Method for an earthquake A in x direction A
Q_y A = Response quantity obtained in Equivalent Seismic Load Method for an earthquake A in y direction A
q A = Behaviour Factor A
q_e A = Behaviour Factor for nonstructural element or component A
q_R T A= Seismic Load Reduction Factor A
R_d A Design resistance of an element; resistance of connection in accordance with EN A 1993-1-1:2004 A
R_{di} A Design resistance of the zone or element i f
R_{fy} A = Plastic resistance of connected dissipative member based on design yield strength A of material as defined in EN 1993-1-1:2004 A
Sf_E T) = Elastic spectral acceleration [m/s^2] A
Sf_R T) A Design (reduced) spectral acceleration [m/s^2] A
S_{SD} A = Short period (0.2 second) elastic spectral acceleration [m/s^2] A
S_{1D} A = 1.0 second elastic spectral acceleration [m/s^2] A
s A = Spacing of transverse reinforcement [mm] A
T A = Natural period of vibration [s] A
T_L A = Transition period of response spectrum to long-period range [s] A
T_o A = Response spectrum short corner period [s] A
T_S A = Response spectrum long corner period [s] A
T_1 A = Natural period of predominant mode (first mode) [s] A

A

T_n A = Natural period of n'th mode [s] A

t_f A = Flange thickness of a seismic link A

t_w A = Web thickness of a seismic link A

V_b A = Base shear in the earthquake direction considered A

V_{bx} A = Base shear in x earthquake direction A

V_{bCx} A = Base shear obtained by modal combination in x earthquake direction A

V_{by} A = Base shear in x earthquake direction A

V_{bCy} A = Base shear obtained by modal combination in y earthquake direction A

V_{Ed} A = Shear force obtained from analysis for the seismic design situation A

V_{Ed} A = Design shear force determined in accordance with capacity design rule A

$V_{Ed,E}$ A= Shear force due to design seismic action A

$V_{Ed,G}$ A= Shear force due to non-seismic actions in seismic design situation A

$V_{Ed,M}$ A= Shear force due to application of plastic moment resistances at the two A ends A

V_i A = i'th storey seismic shear in the earthquake direction considered A

V_{ic} A = Sum of seismic shear forces of all columns at the i'th storey in the earthquake A direction considered A

V_{is} A = Sum of seismic shear forces in the earthquake direction considered at the i'th storey A columns where strong column – weak beam condition is satisfied at both bottom A and top joints A

$V_{pl,Rd}$ A= Design value of shear resistance of a member in accordance with EN 1993-1-1: A 2004 **A**

$V_{wb,Rd}$ A = Shear buckling resistance of the web panel A

$V_{wp,Ed}$ A = Design shear force in web panel due to design seismic action effects A

$V_{wp,Rd}$ A = Shear resistance of the web panel in accordance with EN 1993- 1-8:2004, 6.2.4.1 A

W_i A Seismic weight of i'th storey of building A

W_t A Total seismic weight of building corresponding to total mass, M_t A

α A = Confinement effectiveness factor; ratio of the smaller bending moments $M_{Ed,A}$ at A one end of the link in the seismic design situation, to the greater bending moments A $M_{Ed,B}$ at the end where the plastic hinge develops, both moments being taken as A absolute values. A

α_G A = Coefficient used for determining the gap size of a seismic joint A

α_i A = Ratio of V_{is} / V_{ic} calculated for any i'th storey A

Δ_{ji} A = Reduced storey drift of the j'th vertical element at i'th storey A

$\Delta_{i\ avg}$ = Average reduced storey drift of the i'th storey A

δ_{ji} A = Effective storey drift of the j'th vertical element at i'th storey A

$\delta_{i\ max}$ = Maximum effective storey drift of the i'th storey A

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

ε A = Shear amplification factor of wall A

ε_a A = Total strain of steel at Ultimate Limit State A

ε_{cg} A = Upper limit (capacity of concrete compressive strain in the extreme fiber inside the A confinement reinforcement *f*

ε_{cu2} A = Ultimate compressive strain of unconfined concrete A

ε_s A = Upper limit (capacity of strain in steel reinforcement A

$\varepsilon_{sy,d}$ A = Design value of steel strain at yield A

η_{ti} A Torsional Irregularity Factor defined at i'th storey of building A

η_{ci} A = Strength Irregularity Factor defined at i'th storey of building A

η_{ki} A = Stiffness Irregularity Factor defined at i'th storey of building A

Φ_{xin} A = In buildings with floors modelled as rigid diaphragms, horizontal component A of n'th mode shape in the x direction at i'th storey of building A

Φ_{yin} = In buildings with floors modelled as rigid diaphragms, horizontal component of n’th mode shape in the y direction at i’th storey of building

$\Phi_{\theta in}$ = In buildings with floors modelled as rigid diaphragms, rotational component of n’th mode shape around the vertical axis at i’th storey of building

ϕ_y = Yield curvature corresponding to nominal plastic moment *f*

ϕ'_y = Curvature corresponding to first-yield

Γ_{xn} = Participation Factor of n’th mode for x direction earthquake

γ_{ov} = Material overstrength factor

γ_{pb} = Factor applied to design value $N_{pl,Rd}$ of yield resistance in tension of the compression brace in a V bracing

$\bar{\lambda}$ = Non-dimensional slenderness of a member as defined in EN 1993-1-1:2004

μ_φ = Curvature ductility factor

ν_d = Axial force in seismic design situation, normalised to $A_c f_{cd}$

= Value of (R_{di} / E_{di} $\leq q$/I of the element *i* of the structure which has the highest influence on the effect E_F under consideration

$_{w}$ = Mechanical ratio of vertical web reinforcement $_v = \rho_v f_{yd,v} / f_{cd}$

$_{wd}$ = Mechanical volumetric ratio of confining reinforcement

ρ = Tension reinforcement ratio

ρ' Compression reinforcement ratio

ρ_{max} = Maximum tension reinforcement ratio allowed in the critical region of a primary beam

ρ_{min} = Minimum tension reinforcement ratio to be provided along a beam

θ_i = Second Order Effect Indicator defined at i’th storey of building

θ_p = Rotation capacity of the plastic hinge region

$\sum M_{Rb}$ = Sum of design values of moment resistances of beams framing in a joint in the direction considered

$\sum M_{Rc}$ = Sum of design values of moment resistances of columns framing in a joint in the direction considered

1.1.3. Reference Standards

1.1.3.1 – The following standards are acceptable reference standards to be utilized in combination with this standard:

EN 1990: Eurocode – Basis of structural design

EN 1992-1-1: Eurocode 2 – Design of concrete structures – Part 1-1: General - Common rules for building and civil engineering structures

EN 1993-1-1: Eurocode 3 – Design of steel structures – Part 1-1: General - General rules

EN 1993-1-1: Eurocode 4 – Design of composite steel and concrete structures – Part 1-1: General rules and rules for buildings

EN 1997-1: Eurocode 7 – Geotechnical design – Part 1: General rules

EN 1998-5: Eurocode 8 – Design of structures for earthquake resistance – Part 5: Foundations, retaining structures and geotechnical aspects

1.1.3.2 – Regarding the utilization of the above-referenced Eurocodes, National Application Documents of the United Kingdom may be applied.

1.2. SEISMIC GROUND MOTION

1.2.1. Earthquake levels

The earthquake levels to be considered in this Code are defined in the following:

1.2.1.1 – (E1) Earthquake Level: This earthquake level represents relatively frequent but low-intensity earthquake ground motions with a high probability to occur during the service life of buildings within the scope of this Code. The probability of exceedance of (E1) level earthquake in 50 years is 50%, which corresponds to a return period of 72 years.

1.2.1.2 – (E2) Earthquake Level: This earthquake level represents the infrequent and higher intensity earthquake ground motions with a low probability to occur during the service life of buildings within the scope of this Code. The probability of exceedance of (E2) level earthquake in 50 years is 10%, which corresponds to a return period of 475 years.

1.2.1.3 – (E3) Earthquake Level: This earthquake level represents the highest intensity, very infrequent earthquake ground motions that the buildings within the scope of this Code may be subjected to. The probability of exceedance of (E3) level earthquake in 50 years is 2%, which corresponds to a return period of 2475 years.

1.2.2. Representation of ground motion: Elastic Response Spectrum

1.2.2.1 – Within the boundaries of Emirate of Dubai, 5% damped horizontal elastic spectral accelerations corresponding to short period (0.2 second), S_{SD}, and 1.0 second natural vibration period, S_{1D}, are given for (E1), (E2) and (E3) earthquake levels in **Table 1.1** for local soil classes defined in **Annex A**.

1.2.2.2 – Elastic response spectrum representing the horizontal component of earthquake ground motion is defined as follows (**Fig.1.1**):

$$
\begin{aligned}
S_E(T) &= 0.4\,S_{SD} + 0.6\,\frac{S_{SD}}{T_0}\,T && (T \le T_0) \\
S_E(T) &= S_{SD} && (T_0 \le T \le T_S) \\
S_E(T) &= \frac{S_{1D}}{T} && (T_S \le T \le T_L) \\
S_E(T) &= \frac{S_{1D}\,T_L}{T^2} && (T_L \le T)
\end{aligned}
\qquad \textbf{(1.1)}
$$

Spectrum corner periods T_o and T_S are defined as:

$$T_S = \frac{S_{1D}}{S_{SD}} \quad ; \quad T_0 = 0.2\,T_S \qquad \textbf{(1.2)}$$

Transition period to long-period range shall be taken for Emirate of Dubai as $T_L = 8$ s.

Table 1.1. Short period and 1.0 second elastic spectral accelerations

Soil Class	Earthquake Level					
	E1)		E2)		E3)	
	S_{SD} / g	S_{1D} / g	S_{SD} / g	S_{1D} / g	S_{SD} / g	S_{1D} / g
A	0.080	0.032	0.120	0.053	0.180	0.080
B	0.100	0.040	0.150	0.067	0.225	0.100
C	0.120	0.068	0.180	0.113	0.270	0.170
D	0.160	0.096	0.240	0.160	0.360	0.240
E	0.250	0.140	0.375	0.233	0.563	0.350
F	Site-specific geotechnical investigation and dynamic site response analysis required (see **Annex A**					

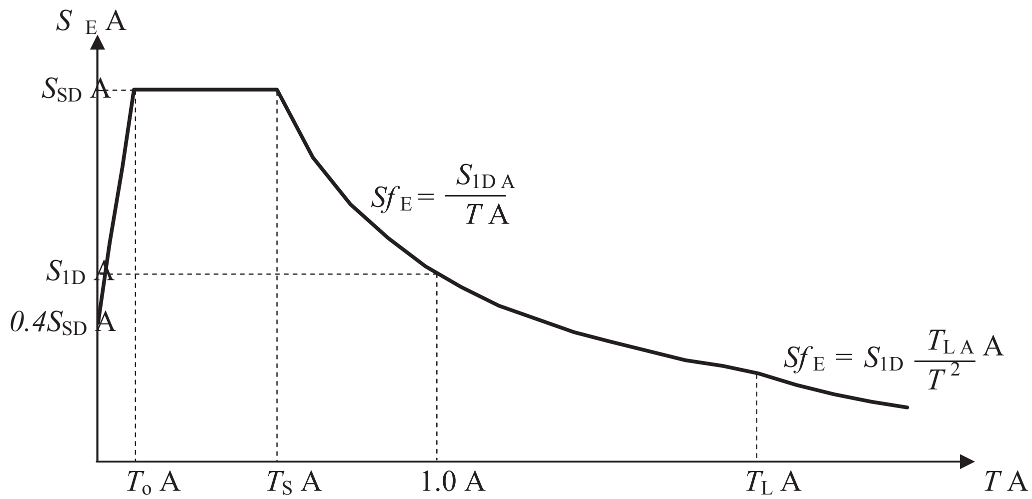

Figure 1.1. Elastic Response Spectrum

1.2.2.3 – When required, elastic acceleration spectrum may be determined through special investigations by considering local seismic and site conditions. However 5% damped acceleration spectrum ordinates shall in no case be less than those determined by **Eq.(1.1)** based on short-period and 1.0 second spectral accelerations given in **Table 1.1**.

1.2.2.4 – Elastic response spectrum representing the vertical component of earthquake ground motion may be taken as half the value of the corresponding to horizontal component.

1.2.3. Representation of ground motion in time domain

1.2.3.1 – A minimum three or seven sets of earthquake ground motions acceleration records in two perpendicular horizontal directions) with the following properties shall be selected for the analysis to be performed in the time domain. Real acceleration records may be obtained from the following data banks:

Cosmos Virtual Data Center http://db.cosmos-eq.org/

Peer Strong Motion Database http://peer.berkeley.edu/smcat/

European Strong- Motion Database http://www.isesd.cv.ic.ac.uk/ESD/frameset.htm

Japan K-NET NIED http://www.k-net.bosai.go.jp/

1.2.3.2 – In the cases where sufficient number of acceleration records cannot be found, artificial earthquake ground motions generated as compatible with the earthquake simulations or the elastic response spectrum may be used. The same acceleration record accelerogram shall not be used for both directions. The ground motion simulations shall be based on a physical model considering the fault mechanism, rupture characteristics and the geological structure of the medium between the earthquake source and recording station.

1.2.3.3 – The average of 5% damped spectral amplitudes calculated at zero period from each set of earthquake ground motion shall not be less than zero-period spectral amplitude of the elastic response spectrum $0.4\,S_{SD}$.

1.2.3.4 – The duration between the two points where acceleration amplitude first and last exceed ±0.05g shall not be shorter than 5 times the dominant natural vibration period of the building nor 15 seconds for each earthquake ground motion record.

1.2.3.5 – The resultant spectrum of an earthquake ground motion set shall be obtained through square-root-of-sum-of-squares of 5% damped spectra of the two directions. The amplitudes of earthquake ground motions shall be scaled according to a rule such that the average of amplitudes of the resultant spectra of all records between the periods $0.2T$ and $1.2T$ T Dominant natural vibration period of the building shall not be less than 1.3 times the amplitudes of the elastic response spectrum along the same period range. The scaling of both components shall be made with the same factors.

1.2.3.6 – Regarding the seismic design of tall buildings according to **Chapter 5**, if needed, parameters related to vertical component of the earthquake ground motion may be specified, subject to the approval of the *Independent Review Board* where applicable.

1.3. SEISMIC PERFORMANCE OBJECTIVES

1.3.1. Classification of buildings

For the purpose of identifying seismic performance objectives as well as analysis and design requirements, buildings shall be classified into two groups, namely low- to medium-rise buildings and tall buildings.

1.3.1.1 – Tall buildings are those of minimum 60 meter height measured from the lowest ground level, excluding basement stories completely underground and surrounded with high-stiffness peripheral walls all around.

1.3.1.2 – Buildings other than those described in **1.3.1.1** are defined as low- to medium-rise buildings.

1.3.2. Performance levels and ranges

Performance levels of low- to medium-rise and/or tall buildings, whereever applicable, are defined below with respect to estimated damage levels in earthquakes.

1.3.2.1 – *Immediate Occupancy – Minimum Damage IO – MD Per ormance Level* describes a performance condition such that no structural or nonstructural damage would occur in buildings and in their elements under the effect of an earthquake or, if any, the damage would be very limited. In this condition, the building can be occupied uninterruptedly and the problems, if any, can be fixed in a few days. *f*

1.3.2.2 – *Li e Sa ety – Controlled Damage LS – CD) Per ormance Level* describes a performance condition where limited and repairable structural and nonstructural damage is permitted in buildings and in their elements under the effect of an earthquake. In this condition, short term a few weeks or months problems related to occupancy of the building may be expected.

1.3.2.3 – *Collapse Prevention – Extensive Damage CP – ED Per ormance Level* describes a performance condition where extensive damage may occur in buildings and in their elements under the effect of an earthquake prior to the collapse of the building. In this condition, long term problems related to occupancy of the buildings may occur or the occupancy of the buildings may be terminated.

1.3.2.4 – The regions in between the above-defined performance levels are identified as *per ormance ranges* as indicated in a strength – typical deformation curve **Fig. 1.2**). The region below IO – MD) Performance Level is defined as *Immediate Occupancy / Minimum f Damage Per ormance Range*, the region in between IO – MD) Performance Level and LS – CD) Performance Level is defined as *Li e Sa ety / Controlled Damage Per ormance Range*, the region in between LS – CD) Performance Level and CP – ED) Performance Level is defined as *Collapse Prevention / Extensive Damage Per ormance Range* and the region above the (CP – ED) Performance Level is defined as *Collapse Range*.

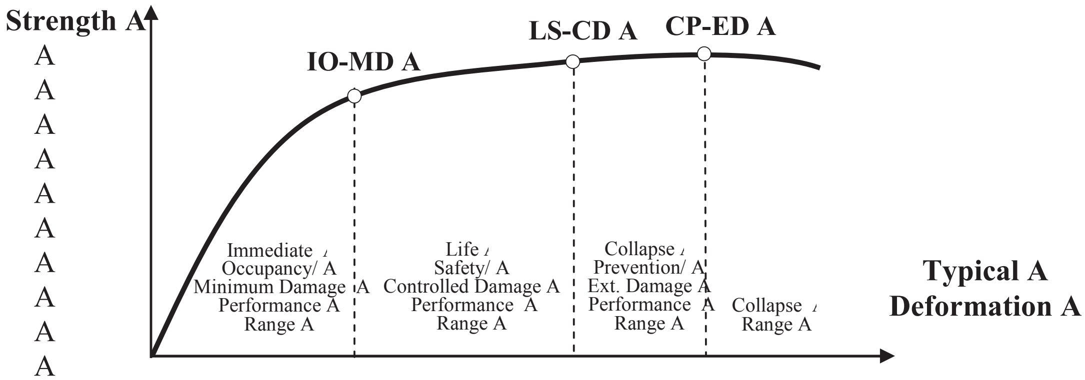

Figure 1.2. Performance levels and ranges

1.3.3. Minimum performance objective for low- to medium-rise buildings

1.3.1 – Minimum performance objective for low- to medium-rise buildings with an Importance Factor of I = 1.0 according to **Table 2.1** is identified as *Life Safety / Controlled Damage Performance Objective* under (E2) level earthquake specified in **1.2**. Without any analytical verification, it is implicitly assumed that a building designed to this performance objective would automatically satisfy *Immediate Occupancy /Minimum Damage Performance Objective* under (E1) level earthquake as well as *Collapse Prevention / Extensive Damage Performance Objective* under (E3) level earthquake.

1.3.3.2 – Minimum performance objective for low- to medium-rise buildings with an Importance Factor of I = 1.5 according to **Table 2.1** is identified as *Immediate Occupancy / Minimum Damage Performance Objective* under (E2) level earthquake specified in **1.2**. Without any analytical verification, it is implicitly assumed that a building designed to this performance objective would automatically satisfy *Life Safety / Controlled Damage Performance Objective* under (E3) earthquake level earthquake.

1.3.3.3 – Minimum performance objective for low- to medium-rise buildings with an Importance Factor between I = 1.0 and I = 1.5 according to **Table 2.1** is identified as in between *Immediate Occupancy / Minimum Damage Performance Objective* and *Life Safety / Controlled Damage Performance Objective* under (E2) level earthquake specified in **1.2**.

1.3.3.4 – Upon the requirement of the Owner or the relevant State Authority, the above-given minimum performance objectives for special low- to medium-rise buildings may be enhanced by assigning higher importance factors within the limits of **Table 2.1**.

1.3.4. Multiple minimum performance objectives for tall buildings

Minimum performance objectives identified for tall buildings are given below **Table 1.2** depending upon the earthquake levels defined in **1.2**:

1.3.4.1 – The multiple performance objectives of tall buildings in *Normal Occupancy Class* (residence, hotel, office building, etc.) are identified as *Immediate Occupancy / Minimum Damage Performance Objective* under (E1) level earthquake, *Life Safety / Controlled*

Damage Per ormance Objective under (E2) level earthquake, and Collapse Prevention / *Extensive Damage Per ormance Objective* under (E3) level earthquake.

1.3.4.2 – The performance objectives of tall buildings in *Special Occupancy Class* health, education, public administration buildings, etc.) are identified as *Immediate Occupancy / f Minimum Damage Per ormance Objective* under (E2) level earthquake, and *Li e Sa ety / f Controlled Damage Per ormance Objective* under (E3) level earthquake.

1.3.4.3 – Upon the requirement of the Owner or the relevant State uthority, higher performance objectives, such as those given in **13.4.2**, may be identified for tall buildings in *Normal Occupancy Class* (residence, hotel, office building, etc. instead of those defined in **1.3.4.1**.

Tablo 1.2. Minimum performance objectives identified for tall buildings under various earthquake levels

Building Occupancy Class	E1) Earthquake Level	E2) Earthquake Level	E3) Earthquake Level
Normal occupancy class: Residence, hotel, office building, etc.	**IO / MD**	**LS / CD**	**CP / ED**
Special occupancy class: Health, education, public admin. buildings, etc.	**—**	**IO / MD**	**LS / CD**

1.4. GENERAL GUIDELINES FOR ARRANGEMENT OF BUILDING STRUCTURAL SYSTEMS

1.4.1. Structural simplicity

1.4.1.1 – Structural simplicity is characterised by the existence of clear and direct paths for the transmission of the seismic forces.

1.4.1.2 – Modeling, analysis, dimensioning, detailing and construction of simple structures are subject to much less uncertainty and thus the prediction of their seismic behaviour is much more reliable.

1.4.2. Uniformity, symmetry and redundancy

1.4.2.1 – Uniformity in plan is characterised by an even distribution of the structural elements which allows direct transmission of the inertia forces created in the distributed masses of the building. If necessary, uniformity may be realised by subdividing the entire building by seismic joints into dynamically independent units, provided that these joints are designed against pounding of the individual units in accordance with **2.7.2**.

1.4.2.2 – Uniformity in the development of the structure along the height of the building is also essential, as it tends to eliminate the occurrence of sensitive zones where high stress or ductility demands might concentrate.

1.4.2.3 – A similarity between the distribution of masses and the distribution of resistance and stiffness eliminates large eccentricities between mass and stiffness.

1.4.2.4 – If the building configuration is symmetrical or quasi-symmetrical, a symmetrical layout of structural elements, which should be well-distributed in-plan, is appropriate for the achievement of uniformity.

1.4.2.5 – The use of evenly distributed structural elements increases redundancy and allows a more favourable redistribution of action effects and widespread energy dissipation across the entire structure.

1.4.3. Adequate resistance and stiffness

1.4.3.1 – Horizontal seismic motion is a bi-directional phenomenon and thus the building structure shall be able to resist horizontal actions in any direction. In this respect, structural elements should be arranged in an orthogonal in-plan structural pattern, ensuring similar resistance and stiffness characteristics in both main directions.

1.4.3.2 – In addition to lateral resistance and stiffness, building structures should possess adequate torsional resistance and stiffness in order to limit the development of torsional motions which tend to stress the different structural elements in a non-uniform way. In this respect, arrangements in which the main elements resisting the seismic action are distributed close to the periphery of the building present clear advantages.

1.4.4. Diaphragm action

1.4.4.1 – In buildings, floors (including the roof play a very important role in the overall seismic behaviour of the structure. They act as horizontal diaphragms that collect and transmit the inertia forces to the vertical structural systems and ensure that those systems act together in resisting the horizontal seismic action. The action of floors as diaphragms is especially relevant in cases of complex and non-uniform layouts of the vertical structural systems, or where systems with different horizontal deformability characteristics are used together (e.g. in dual or mixed systems).

1.4.4.2 – Floor systems and the roof should be provided with in-plane stiffness and resistance and with effective connection to the vertical structural systems. Particular care should be taken in cases of non-compact or very elongated in-plan shapes and in cases of large floor openings, especially if the latter are located in the vicinity of the main vertical structural elements, thus hindering such effective connection between the vertical and horizontal structure.

1.4.4.3 – Diaphragms should have sufficient in-plane stiffness for the distribution of horizontal inertia forces to the vertical structural systems in accordance with the assumptions of the analysis, particularly when there are significant changes in stiffness or offsets of vertical elements above and below the diaphragm.

1.4.4.4 – The diaphragm may be taken as being rigid, if, when it is modeled with its actual in-plane flexibility, its horizontal displacements nowhere exceed those resulting from the rigid diaphragm assumption by more than 10% of the corresponding absolute horizontal displacements under seismic loads.

1.4.5. Adequate foundation

1.4.5.1 – With regard to the seismic action, the design and construction of the foundations and of the connection to the superstructure shall ensure that the whole building is subjected to a uniform seismic excitation.

1.4.5.2 – For buildings with individual foundation elements footings or piles), the use of a foundation slab or tie-beams between these elements in both main directions is recommended.

A

1.5. REGULARITY REQUIREMENTS A

A

Regularity requirements of building structural systems are indirectly specified through the A definition of irregular buildings. A

A

1.5.1. Definition of Irregular Buildings A

A

Regarding the definition of irregular buildings, types of irregularities in plan and in elevation A are given in **Table 1.3** and relevant conditions are given in **1.5.2**. A

A

1.5.2. Conditions for Irregular Buildings A

A

Conditions related to irregularities defined in **Table 1.3** are given below: **A**

A

1.5.2.1 – Irregularity types A**1** and **B2** govern the selection of the method of seismic analysis A as specified in **2.2.2.1**. A

A

1.5.2.2 – In buildings with irregularity types A**2** and A**3**, it shall be verified by calculation that A the floor systems are capable of safe transfer of seismic loads between vertical structural A elements. A

A

1.5.2.3 – In buildings with irregularity type **B1**, in the range $0.60 \le \eta_{ci\ min} < 0.80$, *Behaviour f Factor*, given in **Chapter 3** or **Chapter 4**, as appropriate, shall be multiplied by 1.25 $\eta_{ci\ min}$ **A** which shall be applicable to the entire building in both earthquake directions. In no case, A however, $\eta_{ci} < 0.60$ shall be permitted. Otherwise strength and stiffness of the weak storey A shall be increased and the seismic analysis shall be repeated. A

A

1.5.2.4 – Conditions related to buildings with irregularities of type **B3** are given below: A

a) With the exception of paragraph **b** below, all internal force components induced by A seismic loads shall be increased by 50% for beams and columns supporting discontinuous A vertical elements. A

b Structural walls shall in no case be permitted in their own plane to rest on the beam span A or on slabs at any storey of the building. A

A

A

A

A

A

A

A

A

A

A

A

A

A

A

A

A

Table 1.3 – Irregular Buildings

– IRREGULARITIES IN PLAN	**Related Items**
<u>1 – Torsional Irregularity :</u> 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. [$\eta_{ti} = (\Delta_{i})_{max} / (\Delta_{i})_{avg} > 1.2$] *Storey dri ts shall be calculated in accordance with* **2.3**, *by considering the ef ects o ± %5 accidental eccentricities.*	**1.5.2.1**
<u>2 – Floor Discontinuities :</u> In any floor; I - The case where the total area of the openings including those of stairs and elevator shafts exceeds 1/3 of the gross floor area, II – The cases where local floor openings make it difficult the safe transfer of seismic loads to vertical structural elements, III – The cases of abrupt reductions in the in-plane stiffness and strength of floors.	**1.5.2.2**
<u>3 – Projections in Plan :</u> The cases where projections beyond the re-entrant corners in both of the two principal directions in plan exceed the total plan dimensions of the building in the respective directions by more than 20%.	**1.5.2.2**
B – IRREGULARITIES IN ELEVATION	**Related Items**
<u>B1 – Interstorey Strength Irregularity *Weak Storey*) :</u> 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 shear strength of any storey to the shear strength of the storey immediately above, is less than 0.80. [$\eta_{ci} = V_i / V_{i+1} < 0.80$] *Shear strength o a storey is the sum o design shear strengths o vertical elements according to* **Chapter 3** *or* **Chapter 4**, *as appropriate.*	**1.5.2.3**
<u>B2 – Interstorey Stiffness Irregularity *So t Storey*) :</u> The case where in each of the two orthogonal earthquake directions, *Stif ness 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 is greater than 1.5. [$\eta_{ki} = (\Delta_i/h_i)_{ort} / (\Delta_{i+1}/h_{i+1})_{ort} > 1.5$] *Storey dri ts shall be calculated in accordance with* **2.3**, *by considering the ef ects o ± %5 accidental eccentricities.*	**1.5.2.1**
<u>B3 - Discontinuity of Vertical Structural Elements :</u> The cases where columns are removed at some stories and supported by beams or columns underneath, or structural walls of upper stories are supported by columns or beams underneath.	**1.5.2.4**

1.6. PRIMARY AND SECONDARY SEISMIC MEMBERS

1.6.1. Primary members

ll structural members not designated as being secondary seismic members according to **1.6.2** are taken as being primary seismic members. They shall be taken as being part of the lateral force resisting system, and designed and detailed for earthquake resistance in accordance with the rules of **Chapters 3**,**4** and **5**.

1.6.2. Secondary members

1.6.2.1 – Certain structural members (e.g. beams and/or columns) may be designated as *secondary seismic members* or elements), not forming part of the seismic action resisting system of the building. The strength and stiffness of these elements against seismic actions shall be neglected. They do not need to conform to the requirements of **Chapters 3**,**4** and **5**. Nonetheless these members and their connections shall be designed and detailed to maintain support of gravity loading when subjected to the displacements caused by the most unfavourable seismic design condition. Allowance of second-order effects shall be made in the design of these members.

1.6.2.2 – Total contribution to lateral stiffness of all secondary seismic members shall not exceed 15% of that of all primary seismic members.

1.6.2.3 – The designation of some structural elements as secondary seismic members is not allowed to change the classification of the structure from non-regular to regular as described in **1.5**.

CHAPTER 2
SEISMIC ANALYSIS REQUIREMENTS OF BUILDINGS

2.1. PARAMETERS OF DESIGN RESPONSE SPECTRUM

2.1.1. Importance Factors

Depending on purpose of occupancy of building, *Building Importance Factors* (I) are defined as given in **Table 2.1**.

Table 2.1 – Building Importance Factors (I)

Purpose of Occupancy of Building	I
a) Buildings required to be utilised immediately after the earthquake (Hospitals, dispensaries, health wards, fire fighting buildings and facilities, PTT and other telecommunication facilities, transportation stations and terminals, power generation and distribution facilities, governorate, county and municipality administration buildings, first aid and emergency planning stations) **b)** Buildings containing or storing toxic, explosive and/or flammable materials, etc.	1.5
a) Schools, other educational buildings and facilities, dormitories and hostels, military barracks, prisons, etc. **b)** Museums	1.4
Sport facilities, cinema, theatre and concert halls, etc.	1.2
Buildings other than above-defined buildings. (Residential and office buildings, hotels, building-like industrial structures, etc.	1.0

2.1.2. Seismic Load Reduction Factors

2.1.2.1 – *Elastic seismic loads* determined in terms of *spectral accelerations* defined in **1.2** shall be divided to below-defined *Seismic Load Reduction Factor* to account for the ductile behaviour of the structural system during earthquake. Period-dependent *Seismic Load Reduction Factor*, $q_R(T)$, shall be determined by **Eqs.(2.1)** in terms of *Behaviour Factor*, q, representing the ductility capacity of the structure and the *Building Importance Factor*, I, indicating the performance objective of the building.

$$q_R(T) = 1 + \left(\frac{q}{I} - 1\right)\frac{T}{T_S} \qquad (0 \le T \le T_S)$$
$$q_R(T) = \frac{q}{I} \qquad (T_S < T) \qquad (2.1)$$

where (q/I) ratio shall not be taken less than unity.

2.1.2.2 – *Behaviour Factors* are given in **Chapter 3** for various types of reinforced concrete buildings, in **Chapter 4** for structural steel buildings and in **Chapter 5** for composite concrete-steel buildings.

.

2.1.3. Design Response Spectrum

Reduced spectral accelerations representing the *Design Response Spectrum* shall be defined by dividing the *Elastic Response Spectrum* ordinates given in **1.2.2** to the *Seismic Load Reduction Factor* given in **2.1.2**.

$$Sf_{RA}(T) = \frac{Sf_{E}(T)}{q_{RA}(T)} \qquad \textbf{(2.2)}$$

2.2. SEISMIC ANALYSIS

2.2.1. Applicable analysis methods

The analysis methods applicable for the seismic analysis of building structural systems are given in the following:

2.2.1.1 – *Equivalent Seismic Load Method* described in **2.3** is the simplified single-mode response-spectrum analysis method, which can be used for low- to medium-rise buildings with conditions given in **2.2.2**.

2.2.1.2 – *Multi-Mode Response Spectrum Analysis Method* described in **2.4** is an advanced linear dynamic analysis method, which can be used for both low- to medium-rise as well as tall buildings.

2.2.1.3 – *Linear Response History Analysis Method* described in **2.5.1** is the most advanced linear dynamic analysis method, which can be used for both low- to medium-rise as well as tall buildings.

2.2.1.4 – *Nonlinear Response History Analysis Method* described in **2.5.2** is the most advanced nonlinear dynamic analysis method, which can be used for both low- to medium-rise and tall buildings.

2.2.2. Selection of analysis method for low- to medium-rise buildings

2.2.2.1 – *Equivalent Seismic Load Method* can be used for structures with $H_N \leq 40$ m provided that type A**2** torsional irregularity factor in any story does not exceed 2 $\eta_{ti} \leq 2$ – see **Table 1.3**) type **B2** irregularity does not exists with reference to **1.5**.

2.2.2.2 – *Multi-Mode Response Spectrum Analysis Method* is the acceptable analysis method for all low- to medium-rise buildings.

2.2.3. Definition of seismic mass

Total seismic mass of the building, M_t, shall be determined by **Eq.(2.3)**:

$$M_t = \frac{W_t}{g} = \frac{1}{g}\sum_{i=1}^{N} W_i \quad ; \quad W_i = G_i + n_1 n_2 Q_i \qquad (2.3)$$

where *live load mass reduction factor* n_1 and *live load participation factor* n_2 shall be taken from **Table 2.3** and **Table 2.4**, respectively.

Table 2.3 – Live load mass reduction factor n_1

Type of occupancy	n_1
Storeys with correlated occupancies	0.80
Storeys with independent occupancies	0.30

2.2.4. Consideration of vertical component of earthquake

2.2.4.1 – Vertical component of the seismic action, as defined in **1.2.2.4**, shall be taken into account for the cases listed below:

a) Horizontal or nearly horizontal structural members spanning 20 m or more;

b Horizontal or nearly horizontal cantilever components longer than 5 m;

c) Horizontal or nearly horizontal pre-stressed components;

d Beams supporting columns.

2.2.4.2 – The analysis for determining the effects of the vertical component of the seismic action may be based on a partial model of the structure, which includes the elements on which the vertical component is considered to act e.g. those listed in **2.2.4.1**) and takes into account the stiffness of the adjacent elements.

2.2.4.3 – The effects of the vertical component need be taken into account only for the elements under consideration (e.g. those listed in **2.2.4.1** and their directly associated supporting elements or substructures.

2.3. EQUIVALENT SEISMIC LOAD METHOD

2.3.1. Displacement Components and Application Points of Seismic Loads

2.3.1.1 – Where floors act 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 determined in accordance with **2.3.3** shall be applied to the floor mass centre as well as to the points defined by shifting it +5% and −5% of the floor length in the perpendicular direction to the earthquake direction considered in order to account for the *accidental eccentricity ef ects*.

2.3.1.2 – Where floors do not act as rigid horizontal diaphragms, sufficient number of independent static displacement components shall be considered to account for the in-plane deformation of floors.

2.3.2. Base Shear

Total equivalent seismic load, i.e., the base shear, V_b, in the earthquake direction considered shall be calculated by **Eq.(2.4)**:

$$V_b = M_t\, Sf_R\, T_1) \ge 0.A1\, Mf_t\, S_{SD}\, I \qquad \textbf{2.4)}$$

where design spectral acceleration $Sf_R\ T_1)$ and elastic short period spectral acceleration S_{SD} correspond to (E2) level earthquake. Predominant natural period in the direction of earthquake, T_1, shall be calculated in accordance with **2.3.4.**

2.3.3. Storey Seismic Loads

2.3.3.1 – Total equivalent seismic load determined by **Eq.(2.4)** is expressed by **Eq. 2.5)** as the sum of seismic loads acting at storey levels.

$$V_b = \Delta F_N + \sum_{i=1}^{N} F_i \qquad \textbf{2.5)}$$

2.3.3.2 – The *additional equivalent seismic load*, ΔF_N, acting at the N'th storey roof) of the building shall be determined by **Eq.(2.6)**.

$$\Delta F_N = 0.0075\, N\, V_b \qquad \textbf{2.6)}$$

Excluding ΔF_N, remaining part of the total equivalent seismic load shall be distributed to stories of the building including N'th storey) in accordance with **Eq.(2.7)**.

$$F_i = (V_b - \Delta F_N)\,\frac{W_i\, H_i}{\sum_{k=1}^{N} W_k\, H_k} \qquad \textbf{2.7)}$$

2.3.3.3 – In the case where torsional irregularity defined in **Table 1.3** exists at any i'th storey such that the condition $1.2 < \eta_{ti} \le 2.0$ is satisfied, ±5% accidental eccentricity applied to this floor according to **2.3.1.1** shall be amplified by multiplying with coefficient D_i given by **Eq.(2.8)** for each earthquake direction.

$$D_i = \left(\frac{\eta_{ti}}{1.2}\right)^{2/\alpha} \qquad (2.8)$$

2.3.3.4 – In buildings with very stiff reinforced concrete peripheral walls at their basements, equivalent seismic loads acting on stiff basement stories and those acting on relatively flexible upper stories shall be calculated separately as given in **a)** and **b)** below. Such loads shall be combined for the analysis of the complete structural system.

a) In determining the base shear and equivalent storey seismic loads acting on relatively flexible upper stories, Clauses **2.3.2** and **2.3.3** shall be applied with seismic masses of *upper stories only* taken into account. 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 in accordance with **2.3.4.2** shall also be based on seismic masses of *upper stories only*. Appropriate behaviour factor q shall be selected from **Chapter 3** or **Chapter 4**, as appropriate, based on the structural type of the *upper stories only*.

b) In calculating equivalent seismic loads acting on the stiff basement stories, seismic masses of *basements only* shall be taken into account. Equivalent seismic loads acting on each basement storey shall be calculated with elastic spectral acceleration of $0.4S_{DS}$ to be multiplied directly with the respective storey mass, and the resulting elastic loads shall not be reduced (i.e., $q_R = 1$).

c) In the analysis of the complete structural system under the combined action of the equivalent seismic loads as defined in **a)** and **b)** above, interaction with the soil surrounding basement stories may be considered with an appropriate soil modeling.

d) In-plane strength of ground floor system, which is surrounded by very stiff basement walls and located in the transition zone with the upper stories, shall be checked for internal forces obtained from the analysis according to **c)** above.

2.3.4. Predominant period

2.3.4.1 – Predominant natural vibration period of the building in the earthquake direction, T_1, may be approximately estimated by the following expression:

$$T_1 = C_t H_N^{3/4} \qquad (2.9)$$

C_t may be taken as 0.085 for moment resistant steel frames, 0.075 for moment resistant concrete frames / eccentrically braced steel frames and 0.050 for all other structures. For structures with concrete structural walls C_t may be calculated by **Eq.(2.10)**.

$$C_t = \frac{0.075}{\sqrt{A_c}} \qquad (2.10)$$

where A_c is calculated from **Eq.(2.11)**.

$$A_c = \sum_j \left[A_j \left(0.2 + l_{wj}/H_N\right)^2\right] \qquad (2.11)$$

with the condition that $l_{wj}/H_N \le 0.9$.

2.3.4.2 – Predominant natural vibration period of the building in the earthquake direction, T_1, shall not be taken longer than the value calculated by **Eq.(2.12)**.

$$T_1 = 2\pi \left(\frac{\sum_{i=1}^{N} M_i \, d_{fi}^2}{\sum_{i=1}^{N} F_{fi} \, d_{fi}} \right)^{1/2} \qquad \textbf{(2.12)}$$

Fictitious load F_{fi} acting on the i'th storey may be obtained from **Eq.(2.7)** by substituting any value (*for example a unit value*) in place of $(V_b - \Delta F_N)$.

2.3.5. Directional Combination

2.3.5.1 – The maximum value of each response quantity due to two horizontal components of the earthquake may be estimated by the square root of the sum of the squared values of the response quantities calculated due to each horizontal component.

2.3.5.2 – As an alternative to **2.3.5.1**, the combination procedure given by **Eq.(2.13)** may be employed:

$$\begin{aligned} Q_D &= \pm\, Q_x \pm 0.30\, Q_y \\ Q_D &= \pm\, 0.30\, Q_x \pm Q_y \end{aligned} \qquad \textbf{(2.13)}$$

2.4. MULTI-MODE RESPONSE SPECTRUM ANALYSIS METHOD

In this method, maximum internal forces and displacements are determined by the statistical combination of maximum contributions obtained in sufficient number of natural vibration modes to be considered.

2.4.1. Dynamic degrees of freedom

2.4.1.1 – 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 centre shall be considered at each storey. At each floor, modal seismic loads defined for those degrees of freedom shall be applied to the floor mass centre as well as to the points defined by shifting it +5% and −5% of the floor length in the perpendicular direction to the earthquake direction considered. The latter is to account for the *accidental eccentricity effects*.

2.4.1.2 – In buildings where torsional irregularity exists and floors do not behave as rigid horizontal diaphragms, sufficient number of dynamic degrees of freedom shall be considered to model in-plane deformation of floors.

2.4.2. Modal seismic loads

2.4.2.1 – In a typical n'th vibration mode considered in the analysis, modal seismic loads acting on the i'th story level at the mass centre of the floor diaphgram is expressed by **Eqs.(2.14)**.

$$\begin{aligned} F_{xin} &= M_i \Phi_{xin} \Gamma_{xn} S_{aR}(T_n) \\ F_{yin} &= M_i \Phi_{yin} \Gamma_{xn} S_{aR}(T_n) \\ F_{\theta in} &= M_{\theta i} \Phi_{\theta in} \Gamma_{xn} S_{aR}(T_n) \end{aligned} \qquad \textbf{(2.14)}$$

where Γ_{xn} represents the participation factor of the n'th mode under an eartquake ground motion in x direction. For buildings with rigid floor diaphragms Γ_{xn} is defined as

$$\Gamma_{xn} = \frac{L_{xn}}{M_n} \qquad \textbf{(2.15)}$$

in which L_{xn} and M_n are as expressed in **2.4.3**.

2.4.2.2 – In buildings with very stiff reinforced concrete peripheral walls at their basements, unless a full modal analysis of the structural system is performed, modal seismic loads as defined in **2.4.2.1** acting on stiff basement stories and those acting on relatively flexible upper stories may be calculated separately as given in **a)** and **b)** below.

a) In calculating modal seismic loads acting on relatively flexible upper stories, *the lowest vibration modes* that are effective in the upper stories may be considered, which can be achieved by taking into account the seismic masses of the *upper stories only*. In this case, appropriate behaviour factor *q* must be selected from **Chapter 3** or **Chapter 4**, as appropriate, based on the structural type of the *upper stories only*.

b) In determining modal seismic loads acting on stiff basement stories, *the highest vibration modes* that are effective in the basements may be considered, which can be achieved by taking into account the seismic masses of the *basement stories only*. In this case, resulting elastic modal loads should not be reduced (i.e., $q_R = 1$).

c) Since vibration modes affecting the stiff basement stories and flexible upper stories are expected to be far apart, two separate response spectrum analyses may be performed based on modal seismic loads defined in **a)** and **b)** above. In each of those analyses, interaction with the soil surrounding basement stories may be considered with an appropriate soil modeling. The results of such analyses may be directly superimposed.

d) In-plane strength of ground floor system, which is surrounded by very stiff basement walls and located in the transition zone with the upper stories, shall be checked for internal forces obtained from the analysis explained in **c)** above.

2.4.3. Number of Vibration Modes

Sufficient number of vibration modes, NS, 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.

$$\sum_{n=1}^{N_S} M_{xn} = \sum_{n=1}^{N_S} \frac{L_{xn}^2}{M_n} \geq 0.90 \sum_{i=1}^{N} M_i \qquad \sum_{n=1}^{N_S} M_{yn} = \sum_{n=1}^{N_S} \frac{L_{yn}^2}{M_n} \geq 0.90 \sum_{i=1}^{N} M_i \qquad (2.16)$$

Expressions for L_{xn}, L_{yn} and modal mass M_n shown in **Eqs.(2.16)** are given below for buildings with rigid floor diaphragms:

$$L_{xn} = \sum_{i=1}^{N} M_i \Phi_{xin} \quad ; \quad L_{yn} = \sum_{i=1}^{N} M_i \Phi_{yin} \qquad M_n = \sum_{i=1}^{N} \left(M_i \Phi_{xin}^2 + M_i \Phi_{yin}^2 + M_{\theta i} \Phi_{\theta in}^2 \right) \qquad (2.17)$$

2.4.4. Modal Combination

2.4.4.1 – *Complete Quadratic Combination (CQC) Rule* shall be applied for the combination of maximum modal contributions of response quantities calculated for each vibration mode, such as the base shear, storey shear, internal force components, displacements and storey drifts. It is imperative that modal combination is applied independently for each response quantity.

2.4.4.2 – 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.

2.4.5. Scaling of Response Quantities

In the case where the base shear in the given earthquake direction, V_{bCx} or V_{bCy}, which is obtained through modal combination according to **2.4.4**, is less than 85% of the corresponding base shear, V_{bx} or V_{by}, obtained by Equivalent Seismic Load Method according to **2.3.2** ($V_{bC} < 0.85 V_b$), all internal force and displacement quantities determined by Response Spectrum Analysis Method shall be amplified in accordance with **Eq.(2.18)**.

A

$$Q_{Sx} = \frac{0.85 V_{bx}}{V_{bCx}} Q_{Cx}$$
$$Q_{Sy} = \frac{0.85 V_{by}}{V_{bCy}} Q_{Cy} \quad \textbf{(2.18)}$$

In the case where V_{bCx} or V_{bCy} is not less than 85% of the corresponding base shear V_{bx} or V_{by} , A then $Q_{Sx} = Q_{Cx}$ or $Q_{Sy} = Q_{Cy}$ shall be used in **2.4.6**. A

A

2.4.6. Directional Combination A

A

Directional combination procedures given in **2.3.5** for Equivalent Seismic Load Method are A applicable with Q_x and Q_y replaced by Q_{Sx} and Q_{Sy} , respectively. A

A
A

2.5. RESPONSE HISTORY ANALYSIS METHOD

2.5.1. Linear Response History Analysis

Linear response history analysis based on *mode-superposition procedure* may be performed in lieu of multi-mode response spectrum analysis described in **2.4**.

2.5.1.1 – The analysis shall be based on a set of earthquakes comprising three or seven earthquake records with simultaneously acting two horizontal components to be selected and scaled according to **1.2.3**.

2.5.1.2 – Sufficient number of vibration modes shall be used as described in **2.4.3**.

2.5.1.3 – In each analysis, linear response histories of design quantities obtained for each typical mode (n) shall be reduced by the corresponding Seismic Load Reduction Factor $q_{RA}(T_n)$ given by **Eqs.(2.1)** based on elastic spectrum corner period T_S.

2.5.1.4 – If three ground motions are used in the analysis, the maxima of the results shall be considered for design. If at least seven ground motions are used, the mean values of the results may be considered for design.

2.5.2. Nonlinear Response History Analysis

Nonlinear response history analysis may be performed by direct integration of nonlinear equations of motion in lieu of multi-mode response spectrum analysis described in **2.4** and linear response history analysis described in **2.5.1.** Nonlinear analysis requirements shall be the same as those given in **Chapter 5.**

2.6. SAFETY VERIFICATION

2.6.1. Strength verification

The following relation shall be satisfied for all structural elements including connections and the relevant non-structural elements:

$$E_d \leq R_d \qquad (2.19)$$

where E_d is the design value of the action effect, due to load combinations defined in **2.6.2** including, if necessary, second order effects defined in **2.6.3**, as well as due to capacity design rules, as described in **Chapters 3** and **4**. R_d is the corresponding design resistance of the element, calculated in accordance with the rules specific to the material used considering the requirements of **Chapters 3** and **4**.

2.6.2. Load combinations for seismic design

The load combinations given in **Eq.(2.20)** shall be used to define the design values of action effects. Live load participation factor n_2 is given in **Table 2.4**.

$$\begin{aligned} &E_G + n_2 E_Q \mp E_E \\ &0.9E_G \mp E_E \end{aligned} \qquad (2.20)$$

Table 2.4 – Live load participation factor n_2

Loading areas	n_2
Domestic, residential and office areas	0.3
Shopping and congregation areas	0.6
Storage areas	0.8
Traffic areas (vehicle weight ≤ 30 kN)	0.6
Traffic areas (30 kN < vehicle weight ≤ 160 kN)	0.3
Roof areas	0

2.6.3. Second-Order Effects

Unless a more refined analysis considering the nonlinear behaviour of structural system is performed, second-order effects may be taken into account in accordance with **2.6.3.1**.

2.6.3.1 – In the case where *Second-Order Effect Indicator*, θ_i, satisfies the condition given by **Eq.(2.21)** 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.

$$\theta_i = \frac{\Delta_{i,avg} \left(\sum_{k=i}^{N} W_k \right)}{V_i \, h_i} \leq 0.10 \qquad (2.21)$$

A

where $\Delta_{i\ avg}$ shall be determined in accordance with **2.7.1.1** as the average value of reduced A storey drifts, Δ_{ji} , calculated for i'th storey columns and structural walls. A

A

2.6.3.2 – In the case where $0.10 < \theta \leq 0.20$, second-order effects may approximately be taken A into account by multiplying the relevant seismic response quantity by a factor of $1/(1 - \theta$. A

A

2.6.3.3 – In the case where $\theta > 0.20$, seismic analysis shall be repeated with sufficiently A increased stiffness and strength of the structural system. A

A
A

2.7. DAMAGE LIMITATION

2.7.1. Limitation of story drifts

2.7.1.1 – The *reduced storey drift*, Δ_{ji} , of any column or structural wall shall be determined by **Eq.(2.21)** as the difference of displacements between the two consecutive stories.

$$\Delta_{ji} = d_{ji} - d_{j(i-1)} \qquad (2.22)$$

where d_{ji} and $d_{j(i-1)}$ represent lateral displacements obtained from the analysis at the j'th column or structural wall at stories i and (i – 1) under reduced seismic loads. The minimum equivalent seismic load condition defined by **Eq. 2.4)** and the scaling procedure described in **2.4.5** may not be considered in the calculation of d_{ji} and Δ_{ji}.

2.7.1.2 – When multi-mode response spectrum analysis described in **2.4** or linear response history analysis described in **2.5.1** is used, the *effective storey drift* , δ_{ji} , of the j'th column or structural wall at the i'th storey of a building shall be obtained in each direction by **Eq.(2.23)**.

$$\delta_{ji} = \frac{q}{I} \Delta_{ji} \qquad (2.23)$$

2.7.1.3 – The maximum value of effective storey drifts, $\delta_{i\,max}$, obtained in each direction for columns or structural walls of a given i'th storey of a building shall satisfy the condition given by **Eq.(2.24)**:

$$\frac{\delta_{i\,max}}{h_i} \le 0.02 \qquad (2.24)$$

This limit 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.

2.7.1.4 – The limit given by **Eq.(2.24)** may be exceeded by 20% if nonlinear analysis procedure is performed in accordance with **2.5.2**. For nonlinear analysis, the displacements determined are those obtained directly from the analysis without further modification.

2.7.1.5 – In the case where the condition given in **2.7.1.3** or **2.7.1.4**, whichever applicable, is not satisfied at any storey of the building, the seismic analysis shall be repeated with increased stiffness of the structural system.

2.7.2. Seismic Joints

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:

2.7.2.1 – Sizes of gaps to be provided shall not be less than the square root of sum of squares of average storey displacements of the adjacent buildings (or buildingblocks) multiplied by the coefficient α_G specified below. Storey displacements to be considered are the average values of reduced displacements d_{ji} calculated at the column or structural wall joints of i'th storey. In the cases where the seismic analysis is not performed for the existing old building, the storey displacements shall not be assumed to be less than those obtained for the new A building at the same stories. A

a) α_G = 0.67 *q*/I) shall be taken if all floor levels of adjacent buildings or building blocks are A the same. A

b α_G = *q*/*I*) shall be taken if any of the floor levels of adjacent buildings or building blocks A are not the same. A
A
2.7.2.2 – Seismic joints shall be arranged to allow the independent movement of building A blocks in all earthquake directions. A

A
A

2.8. ANALYSIS REQUIREMENTS FOR NONSTRUCTURAL SYSTEMS

2.8.1 – Analysis requirements for nonstructural elements in low- to medium rise buildings are given in the following paragraphs. The relevant requirements for tall buildings are given in **5.4**.

2.8.2 – Equivalent seismic loads to be applied to structural appendages such as balconies, parapets, chimneys, etc. and to all architectural elements such as façade and partition panels, etc. as well as the seismic loads to be used for the connections of mechanical and electrical equipment to the structural system elements are given by **Eq.(2.25)**.

$$f_e = 0.2\, S_{SD}\, I_e\, m_e \left(1 + 2\frac{H_i}{H_N}\right) \qquad (2.25)$$

Seismic load shall be applied horizontally to the mass centre of the element concerned in a direction to result in most unfavourable internal forces. The seismic loads to be applied to non-vertical elements shall be half the equivalent seismic load calculated by **Eq.(2.25)**.

2.8.3 – For the following non-structural elements the, the *Element Importance Factor* I_e shall not be less than 1.5:

a) Anchorage elements of machinery and equipment required for life safety systems,

b Tanks and vessels containing toxic or explosive substances considered to be hazardous to the safety of the general public.

In all other cases, the *Element Importance Factor* I_e may be assumed to be equal to unity.

2.8.4 – In the case where the sum of mechanical or electrical equipment masses, as denoted by m_e in **Eq.(2.25)**, exceeds $0.2m_i$ at any i'th storey, equipment masses and stiffness properties of their connections to the building shall be taken into account in the earthquake analysis of the building structural system.

2.8.5 – In the case where *floor acceleration spectrum* is determined by appropriate methods to define the peak acceleration at the floor where mechanical or electrical equipment is located, **Eq.(2.25)** may not be applied.

2.8.6 – Twice the seismic load calculated by **Eq.(2.25)** or determined according to **2.8.5** shall be considered for fire extinguishing systems, emergency electrical systems as well as for equipments connecting to infill walls and for their connections

CHAPTER 3
SEISMIC DESIGN REQUIREMENTS
FOR REINFORCED CONCRETE BUILDINGS

3.1. SCOPE AND DESIGN CONCEPTS

3.1.1. Scope

3.1.1.1 – This chapter applies to the seismic design of elements of reinforced concrete buildings.

3.1.1.2 – The rules given in this chapter are additional to those given in EN 1992-1-1:2004.

3.1.2. Design Concepts

3.1.2.1 – Design of earthquake resistant reinforced concrete buildings shall provide the structure with an adequate energy dissipation capacity without substantial reduction of its overall resistance against horizontal and vertical loading. Adequate resistance of all structural elements shall be provided, and non-linear deformation demands in critical regions should be compatible with the overall ductility assumed in calculations.

3.1.2.2 – Reinforced concrete buildings may alternatively be designed for low dissipation capacity and low ductility, by applying only the rules of EN 1992-1-1:2004 for the seismic design situation, and neglecting the specific provisions given in this chapter. The class of such buildings are identified as *Low Ductility Class* (DCL).

3.1.2.3 – Reinforced concrete buildings other than those to which **3.1.2.2** applies, shall be designed to provide energy dissipation capacity and an overall ductile behaviour. Overall ductile behaviour is ensured if the ductility demand involves globally a large volume of the structure spread to different elements and locations of all its storeys. To this end ductile modes of failure (e.g. flexure) should precede brittle failure modes (e.g. shear) with sufficient reliability. The class of such buildings are identified as *Normal Ductility Class* (DCN), for which reinforced concrete seismic design requirements are given in the remainder of **Chapter 3**.

3.1.2.4 – Unless a more accurate analysis of the cracked elements is performed, the elastic flexural and shear stiffness properties of reinforced concrete elements may be taken to be equal to one-half of the corresponding stiffness of the uncracked elements.

3.1.3. Structural types and Behaviour Factors

3.1.3.1 – Reinforced concrete buildings are classified with respect to structural types and their combinations as follows:

a) *Moment-resisting frame system* is defined as a structural system composed of moment-resisting frames only.

b) *Coupled structural wall system* is defined as a structural system composed of coupled structural walls only. Coupled structural walls are made from isolated structural walls connected with relatively stiff *coupling beams* such that base overturning moments of isolated walls are reduced by at least 25% under the same lateral loads.

c) *Uncoupled structural wall system* is defined as a structural system composed of uncoupled isolated) structural walls only.

d *Frame-dominant dual system* is defined as a structural system composed of moment-resisting frames, which resist more than 50% of the total calculated base shear, in combination with coupled or uncoupled walls.

e) *Wall-dominant dual system (coupled walls)* is defined as a structural system composed of coupled structural walls, which resist more than 50% of the total calculated base shear, in combination with moment-resisting frames and/or uncoupled walls.

f *Wall-dominant dual system uncoupled walls)* is defined as a structural system composed of uncoupled isolated) structural walls, which resist more than 50% of the total calculated base shear, in combination with moment-resisting frames and/or coupled walls.

g) *Inverted pendulum system* in which 50% or more of the mass is in the upper third of the height of the structure, or in which the dissipation of energy takes place mainly at the base of a single building element. One-storey frames with column tops connected along both main directions of the building and with the value of the column normalized axial load less than 0.3 are excluded.

3.1.3.2 – Reinforced concrete buildings may be classified to one type of structural system in one horizontal direction and to another in the other direction.

3.1.3.3 – Behaviour factors for all structural types of *Low Ductility Class* (DCL shall be taken as $q = 1$.

3.1.3.4 – Behaviour factors for structural types of *Normal Ductility Class* DCN) shall be taken from **Table 3.1**.

Table 3.1 – Behaviour Factors q **for reinforced concrete structural types**

Structural type	*q f*
Moment resisting frame system	3.5
Coupled structural wall system	3.5
Uncoupled structural wall system	2.0
Frame-dominant dual system	3.0
Wall-dominant dual system (coupled walls)	3.0
Wall-dominant dual system (uncoupled walls)	2.0
Inverted pendulum system	1.5

3.1.4. Design actions

3.1.4.1 – With the exception of structural walls, for which the special provisions of **3.4** apply, the design values of bending moments and axial forces shall be obtained from the analysis of the structure for the seismic design situation in accordance with **2.6**.

3.1.4.2 – The design values of shear forces of beams, columns and structural walls are determined in accordance with **3.2**, **3.3** and **3.4**, respectively.

3.1.5. Capacity Design Rules

3.1.5.1 – Brittle failure or other undesirable failure mechanisms e.g. concentration of plastic hinges in columns of a single storey of a multistorey building, shear failure of structural elements, failure of beam-column joints, yielding of foundations or of any element intended to remain elastic) shall be prevented, by deriving the design action effects of selected regions from equilibrium conditions, assuming that plastic hinges with their possible overstrengths have been formed in their adjacent areas.

3.1.5.2 – In moment resisting frame systems, including frame-dominant dual systems as defined in **3.1.3.1**, the following condition should be satisfied at all beam-column joints:

$$\sum M_{Rc} \geq 1.3 \sum M_{Rb} \qquad \textbf{3.1)}$$

3.1.5.3 – In order that **Eq.(3.1)** is applied, beams framing into the joint shall satisfy the dimensional requirements given in **3.2.1** and **3.3.1**.

3.1.5.4 – Slab reinforcement parallel to the beam and within the effective flange width shall be considered to contribute to the beam flexural capacities taken into account for the calculation of $\sum M_{Rb}$ in **Eq.(3.1)**, if it is anchored beyond the beam section at the face of the joint.

3.1.5.5 – Eq.(3.1) shall be satisfied separately for both earthquake directions and senses with the column moments always opposing the beam moments to yield the most unfavourable result. In calculating the column moment resistances, axial forces shall be taken to yield the minimum moments consistent with the sense of earthquake direction.

3.1.5.6 – If the structural system is a frame or equivalent to a frame in only one of the two main horizontal directions of the structural system, then **Eq.(3.1)** should be satisfied just within the vertical plane through that direction.

3.1.5.7 – Special situations regarding the application of **Eq.(3.1)** are given in the following:

(a) Eq.(3.1) need not to be satisfied in the case where normalized axial force is $\nu_d < 0.10$ in both columns framing into the joint.

b) Eq.(3.1) need not to be satisfied at the base of any frame.

(c) Eq.(3.1) need not to be checked in single storey buildings and in joints of topmost storey of multi-storey buildings.

3.1.5.8 – Eq.(3.1) may be permitted not to be satisfied in a given earthquake direction at a certain number of joints at the bottom and/or top of a storey, provided that **Eq.(3.2)** holds.

$$\alpha_i = \frac{V_{is}}{V_{ic}} \geq 0.75 \qquad \textbf{3.2)}$$

Columns with normalized axial force $\nu_d < 0.10$ may be taken into account in the calculation of V_{is} even if they do not satisfy **Eq.(3.1)**.

3.1.5.9 – In the case where **Eq.(3.2)** holds, bending moments and shears of columns satisfying **Eq.(3.1)** at both bottom and top joints shall be amplified by multiplying with the ratio $1/\alpha_i)$ within the range of $0.75 \leq \alpha_i < 1.00$.

3.1.6. Material requirements

3.1.6.1 – In buildings of both *Low Ductility Class* (DCL) and *Normal Ductility Class* (DCN) reinforcing steel of class B or C in EN 1992-1-1:2004, Table C.1 shall be used.

3.1.6.2 – The following material requirements shall apply for buildings of *Nominal Ductility Class* (DCN):

a) Concrete of a class lower than C 16/20 shall not be used.

b) Only ribbed bars shall be used as reinforcing steel.

c) Welded wire meshes may be used, if they meet the requirements in **b)** above and in **3.1.6.1**.

3.1.7. Local ductility requirements

3.1.7.1 – For the required overall ductility of the structure to be achieved, the potential regions for plastic hinge formation, to be defined later for each type of building element, shall possess high plastic rotational capacities.

3.1.7.2 – In order to satisfy the requirement given in **3.1.7.1**, the following conditions shall be met:

a) The curvature ductility factor μ_φ of all critical regions of elements, including column ends (depending on the potential for plastic hinge formation in columns) shall be at least equal to the following values:

$$\mu_\varphi = 1 + 2\left(\frac{q}{I} - 1\right)\frac{T_S}{T_1} \qquad (0 < T_1 \le T_S)$$

$$\mu_\varphi = 2\frac{q}{I} - 1 \qquad (T_S < T_1) \qquad \textbf{(3.3)}$$

b) Local buckling of compressed steel within potential plastic hinge regions of primary seismic elements shall be prevented. Relevant application rules are given in **3.2.3**, **3.3.3** and **3.4.3**.

3.1.7.3 – Appropriate concrete and steel qualities are adopted to ensure local ductility as follows:

a) Steel used in critical regions of seismic elements should have high uniform plastic elongation (see **3.1.6.1**);

b) Tensile strength to yield strength ratio of the steel used in critical regions of primary seismic elements should be significantly higher than unity. Reinforcing steel conforming to the requirements of **3.1.6.1** may be deemed to satisfy this requirement;

c) Concrete used in primary seismic elements should possess adequate compressive strength and a fracture strain which exceeds the strain at the maximum compressive strength by an adequate margin. Concrete conforming to the requirements of **3.1.6.2** may be deemed to satisfy these requirements.

3.1.7.4 – In critical regions of elements with longitudinal reinforcement of steel class B in EN 1992-1-1:2004, Table C.1, the curvature ductility factor μ_φ should be at least equal to 1.5 times the value given by **Eq.(3.3)**.

3.2. SEISMIC DESIGN REQUIREMENTS FOR REINFORCED CONCRETE BEAMS

3.2.1. Geometrical requirements

3.2.1.1 – The distance between the centroidal axes of a beam and the column into which it frames shall be limited to less than $b_c / 4$.

3.2.1.2 – Width b_w of a beam shall satisfy the following expression:

$$b_w \le \min\{ b_c + h_w , 2b_c \} \qquad \textbf{3.4)}$$

3.2.1.3 – The effective flange width b_{eff} may be taken as follows:

a) In beams framing into exterior columns, the effective flange width b_{eff} is taken, in the absence of a transverse beam, as being equal to the width b_c of the column, or if there is a transverse beam of similar depth, equal to this width increased by $2h_f$ on each side of the beam.

b) In beams framing into interior columns the above widths may be increased by $2h_f$ on each side of the beam.

3.2.1.4 – For a beam supporting columns discontinued below the beam, the following rules apply:

a) There shall be no eccentricity of the column axis relative to that of the beam.

b) The beam shall be supported by at least two direct supports, such as walls or columns.

3.2.2. Design shear forces of beams

3.2.2.1 – In beams the design shear forces shall be determined in accordance with the capacity design rule, on the basis of the equilibrium of the beam under: a) the transverse load acting on it in the seismic design situation and b) end moments $M_{i,d}$ (with i = 1,2 denoting the end sections of the beam), corresponding to plastic hinge formation for positive and negative directions of seismic loading:

$$V_{Ed} = V_{Ed,G} \pm \frac{M_{1,d} + M_{2,d}}{l_{cl}} \qquad \textbf{3.5)}$$

The plastic hinges should be taken to form at the ends of the beams or (if they form there first) in the vertical elements connected to the joints into which the beam ends frame.

3.2.2.2 – End moments $M_{i,d}$ may be determined as follows:

$$M_{i,d} = M_{Rb,i} \min\left(1, \frac{\sum M_{Rc}}{\sum M_{Rb}}\right) \qquad \textbf{3.6)}$$

The value of $\sum M_{Rc}$ shall be compatible with the the column axial force(s) in the seismic design situation for the considered sense of the seismic action.

3.2.2.3 – At a beam end where the beam is supported indirectly by another beam, instead of framing into a vertical member, the beam end moment $M_{i,d}$ there may be taken as being equal to the acting moment at the beam end section in the seismic design situation.

3.2.2.4 – End moments $M_{i,d}$ need not exceed those obtained from seismic analysis with $q/I = 1$.

3.2.3. Seismic detailing of beams

3.2.3.1 – The regions of a beam up to a distance $l_{cr} = h_w$ (where h_w denotes the depth of the beam) from an end cross-section where the beam frames into a beam-column joint, as well as from both sides of any other cross-section liable to yield in the seismic design situation, shall be considered as being critical regions.

3.2.3.2 – In beams supporting discontinued (cut-off) vertical elements, the regions up to a distance of $2h_w$ on each side of the supported vertical element should be considered as being critical regions.

3.2.3.3 – The following conditions shall be met at both flanges of the beam along the critical regions:

a) At the compression zone, reinforcement of not less than half of the reinforcement provided at the tension zone shall be placed, in addition to any compression reinforcement needed for the verification of the beam in the seismic design situation.

b) The reinforcement ratio of the tension zone, ρ, shall not exceed a value ρ_{max} equal to:

$$\rho_{max} = \rho' + \frac{0.0018}{\mu_{\varphi}\varepsilon_{sy,d}} \cdot \frac{f_{cd}}{f_{yd}} \qquad (3.7)$$

with the reinforcement ratios of the tension zone and compression zone, ρ and ρ′, both normalised to bd, where b is the width of the compression flange of the beam. If the tension zone includes a slab, the amount of slab reinforcement parallel to the beam within the effective flange width defined in **3.2.1.3** is included in ρ.

3.2.3.4 – Along the entire length of a beam, the reinforcement ratio of the tension zone, ρ, shall be not less than the following minimum value ρ_{min}:

$$\rho_{min} = 0.5 \frac{f_{ctm}}{f_{yk}} \qquad (3.8)$$

3.2.3.5 – Within the critical regions of beams, hoops satisfying the following conditions shall be provided:

a) The diameter d_{bw} of hoops shall be not less than 6 mm.

b) The spacing, s, of hoops (in millimetres) shall not exceed:

$$s \le \min\{h_w/4;\ 24d_{bw};\ 225;\ 8d_{bL}\} \qquad (3.9)$$

c) The first hoop shall be placed not more than 50 mm from the beam end section.

3.3. SEISMIC DESIGN REQUIREMENTS FOR REINFORCED CONCRETE COLUMNS

3.3.1. Geometrical requirements

3.3.1.1 – Shorter dimension of columns with rectangular section shall not be less than 300 mm and section area shall not be less than 90000 mm^2. Diameter of circular columns shall be at least 300 mm. Minimum column dimensions may be reduced to 250 mm and minimum area of rectangular section may be reduced to 62500 mm^2 in buildings with no more than three stories above ground.

3.3.1.2 – Normalised axial force of column, ν_d, shall satisfy the condition of $\nu_d < 0.65$.

3.3.2. Design shear forces of columns

3.3.2.1 – In columns the design values of shear forces shall be determined in accordance with the capacity design rule, on the basis of the equilibrium of the column under end moments $M_{i,d}$ (with i = 1,2 denoting the end sections of the column), corresponding to plastic hinge formation for positive and negative directions of seismic loading.

$$V_{Ed} = \frac{M_{1,d} + M_{2,d}}{l_{cl}} \qquad \text{3.10)}$$

The plastic hinges should be taken to form at the ends of the beams connected to the joints into which the column end frames, or if they form there first) at the ends of the columns.

3.3.2.2 – End moments $M_{i,d}$ may be determined as follows:

$$M_{i,d} = 1.1\, M_{Rc,i} \min\left(1, \frac{\sum M_{Rb}}{\sum M_{Rc}}\right) \qquad \text{3.11)}$$

The values of $\sum M_{Rc,i}$ and $\sum M_{Rc}$ shall be compatible with the column axial force s in the seismic design situation for the considered sense of the seismic action.

3.3.2.3 – End moments $M_{i,d}$ need not exceed those obtained from seismic analysis with $q/I = 1$.

3.3.3. Seismic detailing of columns

3.3.3.1 – The total longitudinal reinforcement ratio ρ_l shall be not less than 1% and not more than 4%. In symmetrical cross-sections symmetrical reinforcement should be provided $\rho = \rho'$.

3.3.3.2 – At least one intermediate bar shall be provided between corner bars along each column side, to ensure the integrity of the beam-column joints.

3.3.3.3 – The regions up to a distance l_{cr} from both end sections of a column shall be considered as being critical regions.

3.3.3.4 – In the absence of more precise information, the length of the critical region l_{cr} (in metres) may be computed from the following expression:

$$l_{cr} = \max\{h_c ,\ l_{cl}/6 ,\ 0.45\} \quad (3.12)$$

3.3.3.5 – If $l_c / h_c < 3$, the entire height of the column shall be considered as being a critical region and shall be reinforced accordingly.

3.3.3.6 – Confinement reinforcement for the critical regions shall not be less than given by **Eq.(3.13)**.

$$\alpha\omega_{wd} \geq 30\,\mu_{\varphi}\,\nu_d\,\varepsilon_{sy,d}\,\frac{b_c}{b_o} - 0.035 \quad (3.13)$$

where α is the confinement effectiveness factor, equal to $\alpha = \alpha_n \alpha_s$ with components α_n and α_s defined as follows:

a) For rectangular cross-sections:

$$\alpha_n = 1 - \sum_n \frac{b_i^2}{6 b_o h_o} \quad ; \quad \alpha_s = \left(1 - \frac{s}{2b_o}\right)\left(1 - \frac{s}{2h_o}\right) \quad (3.14)$$

where n is the total number of longitudinal bars laterally engaged by hoops or cross ties; and b_i is the distance between consecutive engaged bars.

b For circular cross-sections with circular hoops:

$$\alpha_n = 1 \quad ; \quad \alpha_s = \left(1 - \frac{s}{2D_o}\right)^2 \quad (3.15)$$

c) For circular cross-sections with spiral hoops:

$$\alpha_n = 1 \quad ; \quad \alpha_s = \left(1 - \frac{s}{2D_o}\right) \quad (3.16)$$

3.3.3.7 – A minimum value of $\omega_{wd} \geq 0.08$ shall be provided within the critical region at the base of columns.

3.3.3.8 – Within the critical regions of the primary seismic columns, hoops and cross-ties, of at least 6 mm in diameter, shall be provided with the following conditions:

a) The spacing, s, of the hoops (in millimetres) shall not exceed the value given by **Eq.(3.17)**.

$$s \leq \min\{b_o/2 ,\ 175 ,\ 8d_{bL}\} \quad (3.17)$$

b The distance between consecutive longitudinal bars engaged by hoops or cross-ties shall not exceed 200 mm, taking into account EN 1992-1-1:2004, 9.5.3(6).

3.3.4. Seismic detailing of beam-column joints

3.3.4.1 – The horizontal confinement reinforcement in joints of beams with columns should be not less than that specified in **3.3.3.6 – 3.3.3.8** for the critical regions of columns, with the exception of the case listed in the following paragraph.

3.3.4.2 – If beams frame into all four sides of the joint and their width is at least threequarters of the parallel cross-sectional dimension of the column, the spacing of the horizontal confinement reinforcement in the joint may be increased to twice that specified in **3.3.4.1**, but may not exceed 150 mm.

3.3.4.3 – At least one intermediate (between column corner bars) vertical bar shall be provided at each side of a joint of primary seismic beams and columns.

3.4. SEISMIC DESIGN REQUIREMENTS FOR REINFORCED CONCRETE STRUCTURAL WALLS

3.4.1. Geometrical requirements

3.4.1.1 – Structural walls are the vertical elements of the structural system where the ratio of length to thickness in plan is equal to at least 4.

3.4.1.2 – Web thickness of structural walls, b_{wo} , in metres) should satisfy the following expression:

$$b_{wo} \geq \max\{0.15\,,\; h_s/20\} \qquad \textbf{3.18)}$$

Additional requirements apply with respect to the thickness of the confined boundary elements of walls, as specified in **3.4.3.3**.

3.4.1.3 – Normalised axial force of column, ν_d , shall satisfy the condition of $\nu_d < 0.40$.

3.4.1.4 – Composite wall sections consisting of connected or intersecting rectangular segments L-, T-, U-, I- or similar sections) should be taken as integral units, consisting of a web or webs parallel or approximately parallel to the direction of the acting seismic shear force and a flange or flanges normal or approximately normal to it. For the calculation of flexural resistance, the effective flange width on each side of a web should be taken to extend from the face of the web by the minimum of a) the actual flange width; b) one-half of the distance to an adjacent web of the wall; and c 25% of the total height of the wall above the level considered.

3.4.1.5 – Discontinued structural walls shall not rely for their support on beams or slabs.

3.4.2. Design bending moments and shear forces of structural walls

3.4.2.1 – In walls with $H_w / \ell_w \leq 2.0$, design bending moments and shears determined using appropriate q factor given in **3.1.3** shall be amplified by a factor of $[3 / H_w / \ell_w)]$. However this factor shall exceed 2.

3.4.2.2 – In walls satisfying the condition $H_w / \ell_w > 2.0$, design bending moments along the critical wall height determined according to **3.4.3.1** shall be taken as a constant value being equal to the bending moment calculated at the wall base. Above the critical wall height, a linear bending moment diagram shall be applicable which is parallel to the line connecting the moments calculated at the base and at the top of the wall.

3.4.2.3 – In walls satisfying the condition $H_w / \ell_w > 2.0$, design shear forces at any cross section shall be calculated with **Eq.(3.19)**.

$$V_{Ed} = \varepsilon\, V'_{Ed} \qquad \textbf{3.19)}$$

where shear amplification factor ε is defined as

$$\varepsilon = \sqrt{2 + \left(\frac{M_{Rd,W}}{M_{Ed,W}}\right)^2} \leq \frac{q}{I} \qquad \textbf{3.20)}$$

3.4.3. Seismic detailing of structural walls

3.4.3.1 – Height of the critical region h_{cr} above the base of the wall is given by **Eq.(3.21)**:

$$h_{cr} = \max\{l_w , h_w / 6\} \qquad (3.21)$$

However, the *critical wall height* h_{cr} shall satisfy the following limitations:

$$\begin{aligned} &h_{cr} \leq 2 l_w \\ &h_{cr} \leq h_s \qquad (n \leq 6 \text{ storeys}) \\ &h_{cr} \leq 2 h_s \qquad (n \geq 7 \text{ storeys}) \end{aligned} \qquad (3.22)$$

3.4.3.2 – Boundary elements shall be appropriately defined at the extremities of the wall cross section. The length of each boundary element along the *critical wall height* shall not be less than 20% of the total plan length of the wall, nor shall it be less than two times the wall thickness. The plan length of each boundary element along the wall section above the *critical wall height* shall not be less than 10% of the total plan length of the wall, nor shall it be less than the wall thickness.

3.4.3.3 – The thickness b_w of the confined parts of the wall section (boundary elements) shall not be less than 200 mm. Moreover, if the length of the confined part does not exceed the maximum of 2 b_w and 0.2 l_w , b_w shall not be less than h_s/15. If the length of the confined part exceeds the maximum of 2 b_w and 0.2 l_w , b_w shall not be less than h_s/10.

3.4.3.4 – Mechanical volumetric ratio of the required confining reinforcement ω_{wd} in boundary elements is given by **Eq.(3.23)**:

$$\alpha\omega_{wd} \geq 30\,\mu_\varphi(\nu_d + \omega_v)\,\varepsilon_{sy,d}\frac{b_c}{b_o} - 0.035 \qquad (3.23)$$

where ω_v is the mechanical ratio of vertical web reinforcement ($\omega_v = \rho_v f_{yd,v} / f_{cd}$).

3.4.3.5 – The longitudinal reinforcement ratio in the boundary elements shall be not less than 0.5%.

3.5. REQUIREMENTS FOR ANCHORAGE AND SPLICING OF REBARS

3.5.1. General

3.5.1.1 – EN 1992-1-1:2004, Section 8 for the detailing of reinforcement applies, with the additional rules of the following sub-clauses.

3.5.1.2 – For hoops used as transverse reinforcement in beams, columns or walls, closed stirrups with 135° hooks and extensions of length $10d_{bw}$ shall be used.

3.5.1.3 – The anchorage length of beam or column bars anchored within beam-column joints shall be measured from a point on the bar at a distance $5d_{bL}$ inside the face of the joint, to take into account the yield penetration due to cyclic post-elastic deformations.

3.5.2. Anchorage of rebars

3.5.2.1 – When calculating the anchorage or lap length of column bars which contribute to the flexural strength of elements in critical regions, the ratio of the required area of reinforcement over the actual area of reinforcement shall be assumed to be unity.

3.5.2.2 – If, under the seismic design situation, the axial force in a column is tensile, the anchorage lengths shall be increased to 50% longer than those specified in EN 1992-1-1:2004.

3.5.2.3 – The part of beam longitudinal reinforcement bent in joints for anchorage shall always be placed inside the corresponding column hoops.

3.5.2.4 – To prevent bond failure the diameter of beam longitudinal bars passing through beam-column joints, d_{bL}, shall be limited in accordance with the following expressions:

a) For interior beam-column joints:

$$d_{bL} \le \frac{7.5 f_{ctm}}{f_{yd}} \cdot \frac{(1+0.8\nu_d)}{1+0.5\rho'/\rho_{max}} \qquad \textbf{(3.24)}$$

b) For exterior beam-column joints:

$$d_{bL} \le \frac{7.5 f_{ctm}}{f_{yd}} (1+0.8\nu_d) \qquad \textbf{(3.25)}$$

Eq.(3.24) and **Eq.(3.25)** are not applicable to diagonal bars crossing joints.

3.5.2.5 – If the requirement specified in **3.5.2.4** cannot be satisfied in exterior beam-column joints because the depth, h_c, of the column parallel to the bars is too shallow, the following additional measures may be taken to ensure anchorage of the longitudinal reinforcement of beams.

a) The beam or slab may be extended horizontally in the form of exterior stubs.

b) Headed bars or anchorage plates welded to the end of the bars may be used.

c) Bends with a minimum length of $10d_{bL}$ and transverse reinforcement placed tightly inside the bend of the bars may be added.

3.5.2.6 – Top or bottom bars passing through interior joints, shall terminate in the members framing into the joint at a distance not less than l_{cr} length of the member critical region from the face of the joint (see **3.2.3.1**.

3.5.3. Splicing of rebars

3.5.3.1 – There shall be no lap-splicing by welding within the critical regions of structural elements.

3.5.3.2 – There may be splicing by mechanical couplers in columns and walls, if these devices are covered by appropriate testing under conditions compatible with the selected ductility class.

3.5.3.3 – The transverse reinforcement to be provided within the lap length shall be calculated in accordance with EN 1992-1-1:2004. In addition, the following requirements shall also be met:

a) If the anchored and the continuing bar are arranged in a plane parallel to the transverse reinforcement, the sum of the areas of all spliced bars shall be used in the calculation of the transverse reinforcement.

b If the anchored and the continuing bar are arranged within a plane normal to the transverse reinforcement, the area of transverse reinforcement shall be calculated on the basis of the area of the larger lapped longitudinal bar.

c) The spacing, *s*, of the transverse reinforcement in the lap zone in millimetres shall not exceed

$$s = \min\{h/4 , 100\} \qquad \textbf{3.26)}$$

3.5.3.4 – The required area of transverse reinforcement A_{st} within the lap zone of the longitudinal reinforcement of columns spliced at the same location as defined in EN 1992-1-1:2004), or of the longitudinal reinforcement of boundary elements in walls, may be calculated from the following expression:

$$A_{st} = s \cdot \frac{d_{bl}}{50} \cdot \frac{f_{yld}}{f_{ywd}} \qquad \textbf{3.27)}$$

3.6. DESIGN AND DETAILING OF SECONDARY SEISMIC ELEMENTS

3.6.1 – Secondary seismic elements, which are defined in **1.6.2** shall be designed and detailed to maintain their capacity to support the gravity loads present in the seismic design situation, when subjected to the maximum deformations under the seismic design situation.

3.6.2 – Maximum deformations due to the seismic design situation, as mentioned in **3.6.1**, shall be calculated in accordance with **2.7**. They shall be calculated from an analysis of the structure in the seismic design situation, in which the contribution of secondary seismic elements to lateral stiffness is neglected and primary seismic elements are modeled with their cracked flexural and shear stiffness.

3.6.3 – Bending moments and shear forces of secondary seismic elements shall be calculated with maximum deformations defined in **3.6.2**, using their cracked flexural stiffnesses and, if necessary, shear stiffnesses. They shall not exceed their design flexural and shear resistances determined on the basis of EN 1992-1-1:2004.

3.7. SEISMIC DESIGN REQUIREMENTS FOR FOUNDATIONS

3.7.1. General

3.7.1.1 – The following paragraphs apply for the design of concrete foundation elements, such as footings, tie-beams, foundation beams, foundation slabs, foundation walls, pile caps and piles, as well as for connections between such elements, or between them and vertical concrete elements. The design of these elements shall follow the rules of EN 1998-5:2004, 5.4.

3.7.1.2 – Design values of the action effects E_{Fd} on the foundations shall be derived as follows:

$$E_{Fd} = E_{F,G} + \Omega E_{F,E} \qquad \textbf{3.28)}$$

3.7.1.3 – In box-type basements of dissipative structures, comprising: a a concrete slab acting as a rigid diaphragm at basement roof level; b a foundation slab or a grillage of tie-beams or foundation beams at foundation level, and c) peripheral and/or interior foundation walls, columns and beams (including those at the basement roof) are expected to remain elastic under the seismic design situation. Shear walls should be designed for plastic hinge development at the level of the basement roof slab. To this end, in walls which continue with the same cross-section above the basement roof, the critical region should be taken to extend below the basement roof level up to a depth of h_{cr} see **3.4.3.1**). Moreover, the full free height of such walls within the basement should be dimensioned in shear assuming that the wall develops its flexural overstrength 1.1 M_{Rd} at the basement roof level and zero moment at the foundation level.

3.7.2. Tie-beams and foundation beams

3.7.2.1 – Stub columns between the top of a footing or pile cap and the soffit of tie-beams or foundation slabs shall be avoided. To this end, the soffit of tie-beams or foundation slabs shall be below the top of the footing or the pile cap.

3.7.2.2 – Axial forces in tie-beams or tie-zones of foundation slabs in accordance with 5.4.1.2(6) and 7) of EN 1998-5, should be taken in the verification to act together with the action effects derived for the seismic design situation.

3.7.2.3 – Tie-beams and foundation beams should have a cross-sectional width of at least $b_{w,min}$ = 250 mm and a cross-sectional depth of at least $h_{w,min}$ = 400 mm.

3.7.2.4 – Foundation slabs arranged in accordance with EN 1998-5:2004, 5.4.1.2(2) for the horizontal connection of individual footings or pile caps, should have a thickness of at least t_{min} = 200 mm and a reinforcement ratio of at least $\rho_{s,min}$ = 0.2% at the top and bottom.

3.7.2.5 – Tie-beams and foundation beams should have along their full length a longitudinal reinforcement ratio of at least $\rho_{b,min}$ = 0.4% at both the top and the bottom.

3.7.3. Connections of vertical elements with foundation beams or walls

3.7.3.1 – The common joint) region of a foundation beam or foundation wall and a vertical element shall follow the rules of **3.3.4.1** as a beam-column joint region.

3.7.3.2 – The connection of foundation beams or foundation walls with vertical elements shall follow the rules of **3.3.4**.

3.7.3.3 – Bends or hooks at the bottom of longitudinal bars of vertical elements should be oriented so that they induce compression into the connection area.

3.7.4. Cast-in-place concrete piles and pile caps

3.7.4.1 – The top of the pile up to a distance to the underside of the pile cap of twice the pile cross-sectional dimension, *d*, as well as the regions up to a distance of 2*d* on each side of an interface between two soil layers with markedly different shear stiffness (ratio of shear moduli greater than 6 , shall be detailed as potential plastic hinge regions. To this end, they shall be provided with transverse and confinement reinforcement following the rules for column critical regions given in **3.3.3**.

3.7.4.2 – Piles required to resist tensile forces or assumed as rotationally fixed at the top should be provided with anchorage in the pile cap to enable the development of the pile design uplift resistance in the soil, or of the design tensile strength of the pile reinforcement, whichever is lower. If the part of such piles embedded in the pile cap is cast before the pile cap, dowels should be provided at the interface where the connection occurs.

CHAPTER 4
SEISMIC DESIGN REQUIREMENTS FOR STRUCTURAL STEEL BUILDINGS

4.1. SCOPE AND DESIGN CONCEPTS

4.1.1. Scope

4.1.1.1 – This Chapter applies to the seismic design of elements of structural steel buildings.

4.1.1.2 – The rules given in this Chapter are additional to those given in EN 1993-1-1:2004.

4.1.2. Design Concepts

4.1.2.1 – Design of earthquake resistant steel buildings shall provide the structure with an adequate energy dissipation capacity without substantial reduction of its overall resistance against horizontal and vertical loading. Adequate resistance of all structural elements shall be provided, and non-linear deformation demands in critical regions should be compatible with the overall ductility assumed in calculations.

4.1.2.2 – Steel buildings may alternatively be designed for low dissipation capacity and low ductility, by applying only the rules of EN 1993-1-1:2005 for the seismic design situation, and neglecting the specific provisions given in this chapter. The class of such buildings is identified as *Low Ductility Class* (DCL .

4.1.2.3 – Steel buildings other than those to which **4.1.2.2** applies, shall be designed to provide energy dissipation capacity and an overall ductile behaviour. Overall ductile behaviour is ensured if the ductility demand involves globally a large volume of the structure spread to different elements and locations of all its storeys. To this end ductile modes of failure should precede brittle failure modes with sufficient reliability. The class of such buildings is identified as *Normal Ductility Class* (DCN , for which steel seismic design requirements are given in the remainder of **Chapter 4**.

4.1.3. Structural types and Behaviour Factors

4.1.3.1 – Steel buildings are classified with respect to structural types and their combinations as follows:

a) *Moment-resisting frame system* is defined as a structural system composed of moment-resisting frames only.

b *Concentric braced rame system* is defined as a structural system composed of concentric braced frames only. *f*

c) *Eccentric braced frame system* is defined as a structural system composed of eccentric braced frames only.

d *Frame-dominant dual system* is defined as a structural system composed of moment-resisting frames, which resist more than 50% of the total calculated base shear, in combination with eccentric or concentric braced frames.

A

e) *Braced rame-dominant dual system concentric bracing* is defined as a structural system A composed of concentrically braced frames, which resist more than 50% of the total calculated A base shear, in combination with moment-resisting frames and/or eccentric braced frames. A

f *Braced frame-dominant dual system eccentric bracing* is defined as a structural system A composed of eccentrically braced frames, which resist more than 50% of the total calculated A base shear, in combination with moment-resisting frames and/or concentric braced frames. A

g) *Wall-dominant dual system coupled walls)* is defined as a structural system composed of A coupled structural walls, which resist more than 50% of the total calculated base shear, in A combination Awith Amoment-resisting Aframes Aand/or Auncoupled Awalls Aand/or Aeccentric Aor A concentric braced frames. A

h *Wall-dominant dual system uncoupled walls* is defined as a structural system composed A of uncoupled isolated) structural walls, which resist more than 50% of the total calculated A base Ashear, Ain Acombination Awith Amoment-resisting Aframes Aand/or Acoupled Awalls Aand/or A eccentric or concentric braced frames. A

i Inverted pendulum structures, which are defined in **3.1.3.1** are structures where dissipative A zones are located at the bases of columns. A

A

4.1.3.2 – Steel buildings may be classified to one type of structural system in one horizontal A direction and to another in the other. A

A

4.1.3.3 – Behaviour factors for all structural types of *Low Ductility Class* (DCL shall be A taken as $q = 1$. A

A

4.1.3.4 – Behaviour factors for structural types of *Normal Ductility Class* DCN) shall be A taken from **Table 4.1**. A

A

Table 4.1 – Behaviour Factors q **for steel structural types** **A**

Structural type A	$q\ f$
Moment resisting frame system A	5.0 A
Eccentric braced frame system A	5.0 A
Concentric braced frame system A	3.5 A
Frame-dominant dual system A	4.0 A
Braced frame-dominant dual system A (eccentric bracing A	4.0 A A
Braced frame-dominant dual system A (concentric bracing A	3.5 A A
Wall-dominant dual system (coupled walls) A	3.0 A
Wall-dominant dual system (uncoupled walls) A	2.0 A
Inverted pendulum system A	1.5 A

A

4.1.4. Material Requirements A

A

4.1.4.1 – Structural steel shall conform to standards referred to in EN 1993. A

A

4.1.4.2 – The toughness of the steels and the welds should satisfy the requirements for the seismic action at the quasi-permanent value of the service temperature (see EN 1993-1-10:2004).

4.1.4.3 – In bolted connections of primary seismic members of a building, high strength bolts of bolt grade 8.8 or 10.9 should be used.

4.1.4.4 – In the capacity design checks specified in **4.2** to **4.5**, 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 $\gamma_{ov} = 1.25$.

4.2. GENERAL DESIGN CRITERIA AND DETAILING RULES

4.2.1. Design rules for ductile elements in compression or bending

4.2.1.1 – 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 EN 1993-1-1:2004, 5.5.

4.2.1.2 – Depending on the ductility class and the behaviour 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. Required cross-sectional class

Behaviour Factor *q*	Cross-sectional class
$1.5 < q \leq 2$	Class 1,2 or 3
$2 < q \leq 4$	Class 1 or 2

4.2.2. Design rules for ductile elements in tension

For tension members or parts of members in tension, the ductility requirement of EN 1993-1-1:2004, 6.2.3(3) should be met.

4.2.3. Design rules for connections

4.2.3.1 – For fillet weld or bolted connections, **Eq.(4.1)** should be satisfied:

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

4.2.3.2 – Categories *B* and *C* of bolted joints in shear in accordance with EN 1993-1-8:2004, 3.4.1 and category *E* of bolted joints in tension in accordance with EN 1993-1-8:2004, should be used. Shear joints with fitted bolts are also allowed. Friction surfaces should belong to class A or B as defined in ENV 1090-1.

4.2.3.3 – For bolted shear connections, the design shear resistance of the bolts should be higher than 1.2 times the design bearing resistance.

4.3. DESIGN AND DETAILING RULES FOR MOMENT RESISTING FRAMES

4.3.1. Design criteria

4.3.1.1 – 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, in accordance with **4.3.1.2**.

4.3.1.2 – In moment resisting frame systems, including frame-dominant dual systems as defined in **4.1.3.1**, the following condition should be satisfied at all beam-column joints:

$$\sum M_{Rc} \ge 1.3 \sum M_{Rb} \qquad \textbf{(4.2)}$$

4.3.1.3 – Slab reinforcement parallel to the beam and within the effective flange width shall be considered to contribute to the beam flexural capacities taken into account for the calculation of $\sum M_{Rb}$ in **Eq.(4.2)**, if it is anchored beyond the beam section at the face of the joint.

4.3.1.4 – **Eq.(4.2)** shall be satisfied separately for both earthquake directions and senses with the column moments always opposing the beam moments to yield the most unfavourable result. In calculating the column moment resistances, axial forces shall be taken to yield the minimum moments consistent with the sense of earthquake direction.

4.3.1.5 – If the structural system is a frame or equivalent to a frame in only one of the two main horizontal directions of the structural system, then **Eq.(4.2)** should be satisfied just within the vertical plane through that direction.

4.3.1.6 – Special situations regarding the application of **Eq.(4.2)** are given in the following:

(a) Eq.(4.2) need not to be satisfied at the base of any frame.

b) Eq.(4.2) need not to be checked in single storey buildings and in joints of topmost storey of multi-storey buildings.

4.3.1.7 – **Eq.(4.2)** may be permitted not to be satisfied in a given earthquake direction at a certain number of joints at the bottom and/or top of a storey, provided that **Eq.(4.3)** holds.

$$\alpha_i = \frac{V_{is}}{V_{ic}} \ge 0.75 \qquad \textbf{(4.3)}$$

4.3.1.8 – In the case where **Eq.(4.3)** is satisfied, bending moments and shears of columns satisfying **Eq.(4.2)** at both bottom and top joints shall be amplified by multiplying with the ratio $(1/\alpha_i)$ within the range of $0.75 \le \alpha_i < 1.00$.

4.3.2. Beams

4.3.2.1 – Beams should be verified as having sufficient resistance against lateral and lateral torsional buckling in accordance with EN 1993, 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.

4.3.2.2 – For plastic hinges in the beams it should be verified that the full plastic moment of resistance and rotation capacity are not 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:

$$\frac{M_{Ed}}{M_{pl,Rd}} \le 1.0$$
$$\frac{N_{Ed}}{N_{pl,Rd}} \le 0.15 \qquad (4.4)$$
$$\frac{V_{Ed}}{V_{pl,Rd}} \le 0.5$$

where

$$V_{Ed} = V_{Ed,G} + V_{Ed,M} \quad ; \quad V_{Ed,M} = \frac{M_{pl,Rd,A} + M_{pl,Rd,B}}{L} \qquad (4.5)$$

For sections belonging to cross-sectional class 3, $N_{pl,Rd}$, $M_{pl,Rd}$, $V_{pl,Rd}$ must be replaced with $N_{el,Rd}$, $M_{el,Rd}$, $V_{el,Rd}$ in **Eq.(4.4)** and **Eq.(4.5)**.

4.3.2.3 – The condition in the second expression of **Eq.(4.4)** may not be verified, provided that the provisions of EN 1993-1-1:2004,6.2.9.1 are satisfied.

4.3.3. Columns

4.3.3.1 – The columns shall be verified in compression considering the most unfavourable combination of the axial force and bending moments. N_{ed} , M_{ed} , V_{ed} shall be calculated as:

$$N_{Ed} = N_{Ed,G} + 1.1\,\gamma_{ov}\,\Omega\,N_{Ed,E}$$
$$M_{Ed} = M_{Ed,G} + 1.1\,\gamma_{ov}\,\Omega\,M_{Ed,E} \qquad (4.6)$$
$$V_{Ed} = V_{Ed,G} + 1.1\,\gamma_{ov}\,\Omega\,V_{Ed,E}$$

where Ω is the minimum value of $\Omega_i = M_{pl,Rd,i} / M_{Ed,i}$ of all beams, $M_{Ed,i}$ 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.

4.3.3.2 – The resistance verification of the columns should be made in accordance with EN 1993-1-1:2004, Section 6.

4.3.3.3 – The column shear force V_{Ed} resulting from the structural analysis should satisfy the following expression :

$$\frac{V_{Ed}}{V_{pl,Rd}} \le 0.5 \qquad (4.7)$$

4.3.3.4 – The transfer of the forces from the beams to the columns should conform to the design rules given in EN 1993-1-1:2004, Section 6.

4.3.3.5 – The shear resistance of framed web panels of beam/column connections should satisfy the following expression:

$$\frac{V_{wp,Ed}}{V_{wp,Rd}} \leq 1.0 \qquad \textbf{4.8)}$$

where $V_{wp,Ed}$ is the design shear force in the web panel due to the action effects, taking into account the plastic resistance of the adjacent beams or connections; $V_{wp,Rd}$ is the shear resistance of the web panel in accordance with EN 1993- 1-8:2004, 6.2.4.1. 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.

4.3.3.6 – The shear buckling resistance of the web panels should also be checked to ensure that it conforms to EN 1993-1-5:2004, Section 5:

$$\frac{V_{wp,Ed}}{V_{wb,Rd}} \leq 1.0 \qquad \textbf{4.9)}$$

where $V_{wb,Rd}$ is the shear buckling resistance of the web panel.

4.3.4. Beam - column connections

4.3.4.1 – 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 $V_{ed,G}$ + $V_{ed,M}$ evaluated in **4.3.2.2**.

4.3.4.2 – Energy dissipating semi-rigid and/or partial strength connections are permitted, provided that all of the following requirements are verified:

a) Connections have a rotation capacity consistent with the global deformations

b Members framing into the connections are demonstrated to be stable at the ultimate limit state (ULS);

c) Effect of connection deformation on global drift is taken into account using nonlinear static (pushover) global analysis or non-linear response history analysis.

4.3.4.3 – The connection design should be such that the chord rotation capacity of the plastic hinge region θ_p is not less than 25 mrad for structures with $q > 2$.

4.3.4.4 – In experiments made to assess θ_p the column web panel shear resistance should conform to **Eq.(4.7)** and the column web panel shear deformation should not contribute for more than 30% of the plastic rotation capability θ_p.

4.3.4.5 – The column elastic deformation should not be included in the evaluation of θ_p.

4.3.4.6 – When partial strength connections are used, the column capacity design should be derived from the plastic capacity of the connections.

4.4. DESIGN AND DETAILING RULES FOR FRAMES WITH CONCENTRIC BRACINGS

4.4.1. Design criteria

4.4.1.1 – 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.

4.4.1.2 – Diagonal elements of bracings shall be placed in such a way that the structure exhibits similar load deflection characteristics at each storey in opposite senses of the same braced direction under load reversals. In this regard, the following rule should be met at every storey:

$$\frac{\left|A^{+}-A^{-}\right|}{A^{+}+A^{-}} \leq 0.05 \qquad \textbf{4.10)}$$

where A^{+} and A^{-} are the areas of the horizontal projections of the cross-sections of the tension diagonals, when the horizontal seismic actions have a positive or negative direction respectively.

4.4.2 Analysis

4.4.2.1 – Under gravity load conditions, only beams and columns shall be considered to resist such loads, without taking into account the bracing members.

4.4.2.2 – Diagonals shall be taken into account as follows in an elastic analysis of the structure for the seismic action:

a) In frames with diagonal bracings, only the tension diagonals shall be taken into account.

b In frames with V bracings, both the tension and compression diagonals shall be taken into account.

4.4.2.3 – 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) Non-linear static (pushover) global analysis or non-linear time history analysis is used,

b both pre-buckling and post-buckling situations are taken into account in the modelling of the behaviour of diagonals and,

c) background information justifying the model used to represent the behaviour of diagonals is provided.

4.4.3 Diagonal members

4.4.3.1 – In frames with X diagonal bracings, the non-dimensional slenderness $\bar{\lambda}$ as defined in EN 1993-1-1:2004 should be limited to: $1.3 < \bar{\lambda} \leq 2.0$.

4.4.3.2 – In frames with diagonal bracings in which the diagonals are not positioned as X diagonal bracings, the non-dimensional slenderness $\bar{\lambda}$ should be less than or equal to 2.0.

4.4.3.3 – In frames with V bracings, the non-dimensional slenderness $\bar{\lambda}$ should be less than or equal to 2.0.

4.4.3.4 – In structures of up to two storeys, no limitation applies to $\bar{\lambda}$.

4.4.3.5 – Yield resistance $N_{pl,Rd}$ of the gross cross-section of the diagonals should be such that $N_{pl,Rd} \geq N_{Ed}$.

4.4.3.6 – In frames with V bracings, the compression diagonals should be designed for the compression resistance in accordance with EN 1993.

4.4.3.7 – The connections of the diagonals to any member should satisfy the design rules of **4.2.3**.

4.4.3.8 – In order to satisfy a homogeneous dissipative behaviour of the diagonals, it should be checked that the maximum overstrength $_{i}$ defined in **4.4.4.1** does not differ from the minimum value by more than 25%.

4.4.3.9 – Energy dissipating semi-rigid and/or partial strength connections are permitted, provided that all of the following conditions are satisfied:

a) Connections have an elongation capacity consistent with global deformations;

b Effect of connections deformation on global drift is taken into account using nonlinear static (pushover) global analysis or non-linear time history analysis.

4.4.4 Beams and columns

4.4.4.1 – Beams and columns with axial forces should meet the following minimum resistance requirement:

$$N_{pl,Rd}(M_{Ed}) \geq N_{Ed,G} + 1.1\,\gamma_{ov}\,\Omega\,N_{Ed,E} \qquad \textbf{4.11)}$$

where $N_{pl,Rd}$ M_{Ed} is the design buckling resistance of the beam or the column in accordance with EN 1993, 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, is the minimum value of $_{i} = N_{pl,Rd,i} / N_{Ed,i}$ over all the diagonals of the braced frame system; where $N_{pl,Rd,i}$ 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.

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

a) all non-seismic actions without considering the intermediate support given by the diagonals;

b 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 γ_{pb} $N_{pl,Rd}$ for the brace in compression The factor γ_{pb} is used for the estimation of the post buckling resistance of diagonals in compression, which may be taken as 0.3).

4.4.4.3 – In frames with diagonal bracings where tension and compression diagonals are not intersecting, the design should take into account the tensile and compression forces 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.

4.5. DESIGN AND DETAILING RULES FOR FRAMES WITH ECCENTRIC BRACINGS

4.5.1. Design criteria

4.5.1.1 – Frames with eccentric bracings 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/or plastic shear mechanisms.

4.5.1.2 – Seismic links may be horizontal or vertical components.

4.5.2. Seismic links

4.5.2.1 – The web of a link should be of single thickness without doubler plate reinforcement and without a hole or penetration.

4.5.2.2 – Seismic links are classified into 3 categories according to the type of plastic mechanism developed:

a) Short links, which dissipate energy by yielding essentially in shear;

b Long links, which dissipate energy by yielding essentially in bending;

c) Intermediate links, in which the plastic mechanism involves bending and shear.

4.5.2.3 – For I sections, the following parameters are used to define the design resistances and limits of categories:

$$M_{p,link} = f_y\, b\, t_f\, (d - t_f) \qquad \textbf{4.12)}$$

$$V_{p,link} = (f_y/\sqrt{3})\, t_w\, (d - t_f) \qquad \textbf{4.13)}$$

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

$$\begin{aligned} V_{Ed} &\le V_{p,link} \\ M_{Ed} &\le M_{p,link} \end{aligned} \qquad \textbf{4.14)}$$

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

4.5.2.5 – If $N_{Ed} / N_{pl,Rd} > 0.15$, **Eqs.(4.14)** should be satisfied with the following reduced values $V_{p,link,r}$ and $M_{p,link,r}$ used instead of $V_{p,link}$ and $M_{p,link}$:

$$\begin{aligned} V_{p,link,r} &= V_{p,link}\sqrt{1 - \left(N_{Ed}/N_{pl,Rd}\right)^2} \\ M_{p,link,r} &= M_{p,link}\left[1 - \left(N_{Ed}/N_{pl,Rd}\right)\right] \end{aligned} \qquad \textbf{4.15)}$$

4.5.2.6 – If $N_{Ed} / N_{pl,Rd} \le 0.15$, link length e should not exceed:

$$e \le 1.6 \frac{M_{p,link}}{V_{p,link}} \qquad \text{(if } R < 0.3) \qquad (4.16)$$

$$e \le (1.15 - 0.5R)\, 1.6 \frac{M_{p,link}}{V_{p,link}} \qquad \text{(if } R \ge 0.3)$$

where

$$R = \frac{N_{Ed}\, t_w \,(d - 2t_f)}{V_{Ed}\, A} \qquad (4.17)$$

in which A is the gross area of the link.

4.5.2.7 – To achieve a global dissipative behaviour of the structure, it should be checked that the individual values of the ratios Ω_i defined in **4.5.2.1** do not exceed the minimum value Ω resulting from **4.5.2.1** by more than 25% of this minimum value.

4.5.2.8 – When equal moments develop simultaneously at both ends of the link, links may be classified according to the length *e*. For I sections, the categories are:

$$\text{Short links:} \quad e < e_s = 1.6 \frac{M_{p,link}}{V_{p,link}}$$

$$\text{Long links:} \quad e > e_L = 3.0 \frac{M_{p,link}}{V_{p,link}} \qquad (4.18)$$

$$\text{Intermediate links:} \quad e_s < e < e_L$$

4.5.2.9 – When only one plastic hinge develops at one end of the link, the value of the length *e* defines the categories of the links. For I sections the categories are:

$$\text{Short links:} \quad e < e_s = 0.8(1+\alpha) \frac{M_{p,link}}{V_{p,link}}$$

$$\text{Long links:} \quad e > e_L = 1.5(1+\alpha) \frac{M_{p,link}}{V_{p,link}} \qquad (4.19)$$

$$\text{Intermediate links:} \quad e_s < e < e_L$$

where α is 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 develops, both moments being taken as absolute values.

4.5.2.10 – The link rotation angle θ_p between the link and the element outside of the link as defined in **4.3.4.3** should be consistent with global deformations. It should not exceed the following values:

$$\text{Short links:} \quad \theta_p \le \theta_{pR} = 0.08 \text{ radian}$$

$$\text{Long links:} \quad \theta_p \le \theta_{pR} = 0.02 \text{ radian} \qquad (4.20)$$

$$\text{Intermediate links:} \quad \theta_p \le \theta_{pR} \text{ by interpolation}$$

4.5.2.11 – 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 have a combined width of not less than ($b_f - 2t_w$) and a thickness not less than 0.75 t_w nor 10 mm, whichever is larger.

4.5.2.12 – Links should be provided with intermediate web stiffeners as follows:

a) 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;

b Long links should be provided with one intermediate web stiffener placed at a distance of 1.5 times b from each end of the link where a plastic hinge would form;

c) Intermediate links should be provided with intermediate web stiffeners meeting the requirements of **a)** and **b)** above;

d Intermediate web stiffeners are not required in links of length e greater than 5 M_p/V_p;

e) 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 be not less than t_w or 10 mm, whichever is larger, and the width should be not 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.

4.5.2.13 – Fillet welds connecting a link stiffener to the link web should have a 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 $\gamma_{ov} f_y A_{st} / 4$.

4.5.2.14 – 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 $f_y b t_f$.

4.5.2.15 – 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 EN 1993-1-5:2004, Section 5.

4.5.3. Members 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 unfavourable combination of the axial force and bending moments:

$$N_{Rd}(M_{Ed}, N_{Ed}) \geq N_{Ed,G} + 1.1\,\gamma_{ov}\,\Omega\,N_{Ed,E} \qquad \textbf{4.21)}$$

where N_{Rd} M_{Ed}, N_{Ed}) is the axial design resistance of the column or diagonal member in accordance with EN 1993, 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 the 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 following values: the minimum value of $_{i} = 1.5\ V_{p,link,i} / V_{Ed,i}$ among all short links; the minimum value of $_{i} = 1.5(M_{p,link,i} / M_{Ed,i}$ among all intermediate and long links; where $V_{Ed,i}$, $M_{Ed,i}$ are the design values of the 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 resistances of link *i* as in **4.5.2.3**.

4.5.4. Connections of seismic links

4.5.4.1 – If the structure is designed to dissipate energy in the seismic links, the connections of the links or of the element containing the links should be designed for action effects E_d computed as follows:

$$E_d \geq E_{d,G} + 1.1\,\gamma_{ov}\,\Omega\, E_{d,E} \qquad \textbf{4.22)}$$

where $E_{d,G}$ is the action effect in the connection due to the non-seismic actions included in the combination of actions for the seismic design situation; $E_{d,E}$ is the action effect in the connection due to the design seismic action; γ_{ov} is the overstrength factor, $_{i}$ is the overstrength factor computed in accordance with **4.5.3** for the link.

4.5.4.2 – In the case of semi-rigid and/or 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:

a) the connections have rotation capacity sufficient for the corresponding deformation demands;

b members framing into the connections are demonstrated to be stable at the ULS;

c) the effect of connection deformations on global drift is taken into account.

4.5.4.3 – 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.

4.6. DESIGN RULES FOR STEEL BUILDINGS WITH CONCRETE CORES OR CONCRETE WALLS

4.6.1 – The steel elements shall be verified in accordance with this Chapter and EN 1993, while the concrete elements shall be designed in accordance with **Chapter 3**.

4.6.2 – The elements in which an interaction between steel and concrete exists shall be verified in accordance with **Chapter 5**.

A

4.7. DESIGN RULES FOR INVERTED PENDULUM STRUCTURES A

A

4.7.1 – In inverted pendulum structures defined in **4.1.3.1**), the columns should be verified in A compression considering the most unfavourable combination of the axial force and bending A moments. A

A

4.7.2 – In the checks, N_{Ed}, M_{Ed}, V_{Ed} should be computed as in **4.3.3**. A

A

4.7.3 – The non-dimensional slenderness of the columns should be limited to $\bar{\lambda} \leq 1{,}5$. A

CHAPTER 5
SEISMIC DESIGN REQUIREMENTS FOR STEEL – CONCRETE COMPOSITE BUILDINGS

5.1. SCOPE AND DESIGN CONCEPTS

5.1.1. Scope

5.1.1.1 – This Chapter applies to the seismic design of elements of composite steel-concrete buildings.

5.1.1.2 – The rules given in this Chapter are additional to those given in EN 1994-1-1:2004.

5.1.1.3 – Except where modified by the provisions of this Chapter, the provisions of Chapters **3** and **4** apply.

5.1.2. Design Concepts

5.1.2.1 – Earthquake resistant composite buildings shall be designed in accordance with one of the following design concepts (see **Table 5.1**):

a) Concept A: Low-dissipative structural behaviour.

b Concept B: Dissipative structural behaviour with composite dissipative zones;

c) Concept C: Dissipative structural behaviour with steel dissipative zones.

Table 5.1. Design concepts of composite buildings

Design concept	Structural ductility class	Behaviour factor q
: Low dissipative structural behaviour	DCL	1.0
B or C: Dissipative structural behaviour	DCN	≤ 5.0

5.1.2.2 – In concept A, the action effects may be calculated on the basis of an elastic analysis without taking into account non-linear material behaviour but considering the reduction in the moment of inertia due to the cracking of concrete in part of the beam spans, in accordance with the general structural analysis rules defined in **5.2** and to the specific rules defined in **5.5** to **5.9** related to each structural type. Behaviour factor shall be taken as q = 1.

5.1.2.3 – In concept A the resistance of the members and of the connections should be evaluated in accordance with EN 1993 and EN 1994 without any additional requirements.

5.1.2.4 – In concepts B and C, the capability of parts of the structure (dissipative zones) to resist earthquake actions through inelastic behaviour is taken into account. Behavior factor shall be taken from **Table 5.2**. When adopting concepts B or C the requirements given in **5.2** to **5.9** should be fulfilled.

5.1.2.5 – In concept C, structures are not meant to take advantage of composite behaviour in dissipative zones; the application of concept C is conditioned by a strict compliance to measures that prevent involvement of the concrete in the resistance of dissipative zones. In concept C the composite structure is designed in accordance with EN 1994-1-1:2004 under non-seismic loads and in accordance with **Chapter 4** to resist earthquake action. The measures preventing involvement of the concrete are given in **5.5.5**.

5.1.2.6 – The design rules for dissipative composite structures (concept B), aim at the development of reliable local plastic mechanisms dissipative zones) in the structure and of a reliable global plastic mechanism dissipating as much energy as possible under the design earthquake action. For each structural element or each structural type considered in this Chapter, rules allowing this general design objective to be achieved are given in **5.5** to **5.9** with reference to what are called the specific criteria. These criteria aim at the development of a global mechanical behaviour for which design provisions can be given.

5.1.2.7 – Structures designed in accordance with concept B shall belong to structural ductility class identified as *Normal Ductility Class* (DCN). This ductility class corresponds to increased ability of the structure to dissipate energy in plastic mechanisms, for which composite seismic design requirements are given in the remainder of **Chapter 5**.

5.1.3. Structural types and Behaviour Factors

5.1.3.1 – Composite steel-concrete structures shall be assigned to one of the following structural types according to the behaviour of their primary resisting structure under seismic actions:

a) *Composite moment resisting frames* are those with the same definition and limitations as in **4.1.3.1(a)**, but in which beams and columns may be either structural steel or composite steel-concrete.

b *Composite concentrically braced rames* are those with the same definition and limitations as in **4.1.3.1(b)**. Columns and beams may be either structural steel or composite steel-concrete. Braces shall be structural steel.

c) *Composite eccentrically braced rames* are those with the same definition and configurations as in **4.1.3.1(c)**. The members which do not contain the links may be either structural steel or composite steel-concrete. Other than for the slab, the links shall be structural steel. Energy dissipation shall occur only through yielding in bending or shear of these links.

d Inverted pendulum structures, have the same definition and limitations as in **4.1.3.1(i)**.

e) Composite structural systems are those which behave essentially as reinforced concrete walls. The composite systems may belong to one of the following types:

− Type 1 corresponds to a steel or composite frame working together with concrete infill panels connected to the steel structure;

− Type 2 is a reinforced concrete wall in which encased steel sections connected to the concrete structure are used as vertical edge reinforcement;

− Type 3, steel or composite beams are used to couple two or more reinforced concrete or composite walls.

f Composite steel plate shear walls are those consisting of a vertical steel plate continuous over the height of the building with reinforced concrete encasement on one or both faces of the plate and of the structural steel or composite boundary members.

A

5.1.3.2 – In all types of composite structural systems the energy dissipation takes place in the A vertical steel sections and in the vertical reinforcements of the walls. In type 3 composite A structural systems, energy dissipation may also take place in the coupling beams. A

A

5.1.3.3 – If, in composite structural systems the wall elements are not connected to the steel A structure, **Chapters 3** and **4** apply. A

A

5.1.3.4 – The behaviour factor q shall be taken from **Table 4.1** or **Table 5.2** as indicated in A the latter, provided that the rules in **5.3** to **5.9** are met. A

A

Table 5.2. Behaviour Factors q for composite structural types A

Structural type A	*q f*
Composite moment resisting frame system A	5.0 A
Composite eccentrically braced frame system A	5.0 A
Composite concentrically braced frame system A	3.5 A
Frame-dominant dual system A	4.0 A
Braced frame-dominant dual system (eccentric A bracing A	4.0 A A
Braced frame-dominant dual system (concentric A bracing A	3.5 A A
Inverted pendulum system A	1.5 A
Composite walls (Type 1 and Type 2) A	3.5 A
Composite or concrete walls coupled by steel A or composite beams (Type 3) A	A 3.5 A
Composite steel plate structural walls A	3.5 A

A

5.1.4. Material requirements A

A

5.1.4.1 – In dissipative zones, the prescribed concrete class should not be lower than C20/25. A If the concrete class is higher than C40/50, the design is not within the scope of EN 1998-1. A

A

5.1.4.2 – For Aductility Aclass ADCN Athe Areinforcing Asteel Ataken Ainto Aaccount Ain Athe Aplastic A resistance of dissipative zones shall be of class B or C in accordance with EN 1992-1-1:2004 A Table C.1. A

A

5.1.4.3 – Reinforcing steel of class B or C (EN 1992-1-1:2004, Table C.1) shall be used in A highly stressed regions of non dissipative structures. This requirement applies to both bars and A welded meshes. A

A

5.1.4.4 – Except for closed stirrups or cross ties, only ribbed bars are allowed as reinforcing A steel in regions with high stresses. A

A

5.1.4.5 – Welded meshes not conforming to the requirements of **5.1.4.2** shall not be used in A dissipative zones. If such meshes are used, ductile reinforcement duplicating the mesh should A be placed and their resistance capacity accounted for in the capacity analysis. A

A

A

5.1.4.6 – For structural steel, requirements given in **4.1.4** apply. A

5.2. STRUCTURAL ANALYSIS

5.2.1. Scope

The following rules apply to the analysis of the structure under earthquake action with *Equivalent Seismic Load Method* given in **2.3** and with the *Multi-Mode Response Spectrum Analysis Method* given in **2.4**.

5.2.2. Stiffness of sections

5.2.2.1 – The stiffness of composite sections in which the concrete is in compression shall be computed using a modular ratio *n* given in **Eq.(5.1)**.

$$n = \frac{E_a}{E_{cm}} = 7 \qquad \textbf{5.1)}$$

5.2.2.2 – For composite beams with slab in compression, the second moment of area of the section, referred to as I_1, shall be computed taking into account the effective width of slab defined in **5.4.3**.

5.2.2.3 – The stiffness of composite sections in which the concrete is in tension shall be computed assuming that the concrete is cracked and that only the steel parts of the section are active.

5.2.2.4 – For composite beams with slab in tension, the second moment of area of the section, referred to as I_2, shall be computed taking into account the effective width of slab defined in **5.4.3**

5.2.2.5 – The structure should be analysed taking into account the presence of concrete in compression in some zones and concrete in tension in other zones; the distribution of the zones is given in **5.5** to **5.9** for the various structural types.

5.3. DESIGN CRITERIA AND DETAILING RULES FOR DISSIPATIVE STRUCTURAL BEHAVIOUR COMMON TO ALL STRUCTURAL TYPES

5.3.1. Design criteria for dissipative structures

5.3.1.1 – Dissipative zones shall have adequate ductility and resistance. The resistance shall be determined in accordance with EN 1993-1-1:2004 and **Chapter 4** for concept C, and to EN 1994-1-1:2004 and **Chapter 5** for concept B (see **5.1.2.1**). Ductility is achieved by compliance to detailing rules.

5.3.1.2 – Dissipative zones may be located in the structural members or in the connections.

a) 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 cyclic yielding in the dissipative parts.

b When dissipative zones are located in the connections, the connected members shall have sufficient overstrength to allow the development of cyclic yielding in the connections.

5.3.2. Plastic resistance of dissipative zones

5.3.2.1 – Two plastic resistances of dissipative zones are used in the design of composite steel - concrete structures: a lower bound plastic resistance (index: pl,Rd) and an upper bound plastic resistance index: U,Rd).

5.3.2.2 – The lower bound plastic resistance of dissipative zones is the one taken into account in design checks concerning sections of dissipative elements; e.g. $M_{Ed} < M_{pl,Rd}$. The lower bound plastic resistance of dissipative zones is computed taking into account the concrete component of the section and only the steel components of the section which are classified as ductile.

5.3.2.3 – The upper bound plastic resistance of dissipative zones is the one used in the capacity design of elements adjacent to the dissipative zone: for instance in the capacity design verification of **4.3.1.2**, the design values of the moments of resistance of beams are the upper bound plastic resistances, $M_{U,Rd,b}$, whereas those of the columns are the lower bound ones, $M_{pl,Rd,c}$.

5.3.2.4 – The upper bound plastic resistance is computed taking into account the concrete component of the section and all the steel components present in the section, including those that are not classified as ductile.

5.3.2.5 – Action effects, which are directly related to the resistance of dissipative zones, shall be determined on the basis of the upper bound resistance of composite dissipative sections; e.g. the design shear force at the end of a dissipative composite beam shall be determined on the basis of the upper bound plastic moment of the composite section.

5.3.3. Detailing rules for composite connections in dissipative zones

5.3.3.1 – For the design of welds and bolts, **4.2.3** applies.

5.3.3.2 – In fully encased framed web panels of beam/column connections, the panel zone resistance may be computed as the sum of contributions from the concrete and steel shear panel, if all the following conditions are satisfied:

$$0.6 < h_b / h_c < 1.4 \qquad \textbf{5.2)}$$

$$V_{wp,Ed} < 0.8 V_{wp,Rd} \qquad \textbf{5.3)}$$

where h_b / h_c is the aspect ratio of the panel zone; where $V_{wp,Ed}$ is the design shear force in the web panel due to the action effects, taking into account the plastic resistance of the adjacent composite dissipative zones in beams or connections; $V_{wp,Rd}$ is the shear resistance of the composite steel - concrete web panel in accordance with EN 1994-1-1:2004.

5.3.3.3 – In partially encased stiffened web panels, an assessment similar to that in **5.3.3.2** is permitted if, in addition to the requirements of **5.3.3.4**, one of the following conditions is fulfilled:

a) Straight links of the type defined in **5.4.5.4** and complying with **5.4.5.5** and **5.4.5.6** are provided at a maximum spacing $s_l = c$ in the partially encased stiffened web panel; these links are oriented perpendicularly to the longest side of the column web panel and no other reinforcement of the web panel is required; or

b No reinforcement is present, provided that $h_b / b_b < 1{,}2$ and $h_c / b_c < 1{,}2$.

5.3.3.4 – When a dissipative steel or composite beam is framing into a reinforced concrete column, vertical column reinforcement with design axial strength at least equal to the shear strength of the coupling beam should be placed close to the stiffener or face bearing plate adjacent to the dissipative zone. It is permitted to use vertical reinforcement placed for other purposes as part of the required vertical reinforcement. The presence of face bearing plates is required; they should be full depth stiffeners of a combined width not less than $b_b - 2t$; their thickness should be not less than 0,75 t or 8 mm; b_b and t are respectively the beam flange width and the panel web thickness.

5.3.3.5 – When a dissipative steel or composite beam is framing into a fully encased composite column, the beam column connection may be designed either as a beam/steel column connection or a beam/composite column connection. In the latter case, vertical column reinforcements may be calculated either as in **5.3.3.4** or by distributing the shear strength of the beam between the column steel section and the column reinforcement. In both instances, the presence of face bearing plates as described in **5.3.3.4** is required.

5.3.3.6 – The vertical column reinforcement specified in **5.3.3.4** and **5.3.3.5** should be confined by transverse reinforcement that meets the requirements for members defined in **5.4**.

5.4. RULES FOR MEMBERS

5.4.1. General

5.4.1.1 – Composite members, which are primary seismic members, shall conform to EN 1994-1-1:2004 and to additional rules defined in this Section.

5.4.1.2 – For tension members or parts of members in tension, the ductility requirement of EN 1993-1-1:2004, 6.2.3(3) should be met.

5.4.1.3 – Sufficient local ductility of members which dissipate energy under compression and/or bending should be ensured by restricting the width-to-thickness ratios of their walls. Steel dissipative zones and the not encased steel parts of composite members should meet the requirements of **4.2.1.1** and **Table 4.2**. Dissipative zones of encased composite members should meet the requirements of **Table 5.3**. The limits given for flange outstands of partially or fully encased members may be relaxed if special details are provided as described in **5.4.4.9** and **5.4.5.4** to **5.4.5.6**.

Table 5.3. Limits of wall slenderness

Section type	**Wall slenderness**
Partially encased H or I section Fully encased H or I section Flange outstand limits c / t_f :	$14\,\varepsilon$
Filled rectangular section h / t limits:	$38\,\varepsilon$
Filled circular section d / t limits:	$85\,\varepsilon^2$

where $\varepsilon \; (f_y / 235)^{0.5}$

5.4.1.4 – More specific detailing rules for dissipative composite members are given in **5.4.2**, **5.4.4**, **5.4.5** and **5.4.6**.

5.4.1.5 – In the design of all types of composite columns, the resistance of the steel section alone or the combined resistances of the steel section and the concrete encasement or infill may be taken into account.

5.4.1.6 – The design of columns in which the member resistance is taken to be provided only by the steel section may be carried out in accordance with the provisions of **Chapter 4**. In the case of dissipative columns, the capacity design rules in **5.3.1.2** and **5.3.2.3** should be satisfied.

5.4.1.7 – For fully encased columns with composite behaviour, the minimum crosssectional dimensions *b*, *h* or *d* should be not less than 250 mm.

5.4.1.8 – The resistance, including shear resistance, of non-dissipative composite columns should be determined in accordance with the rules of EN 1994-1-1:2004.

5.4.1.9 – In columns, when the concrete encasement or infill are assumed to contribute to the axial and/or flexural resistance of the member, the design rules in **5.4.4** to **5.4.6** apply. These rules ensure full shear transfer between the concrete and the steel parts in a section and protect the dissipative zones against premature inelastic failure.

5.4.1.10 – For earthquake-resistant design, the design shear strength given in EN 1994-1-1:2004, Table 6.6, should be multiplied by a reduction factor of 0.5.

5.4.1.11 – When, for capacity design purposes, the full composite resistance of a column is employed, complete shear transfer between the steel and reinforced concrete parts should be ensured. If insufficient shear transfer is achieved through bond and friction, shear connectors should be provided to ensure full composite action.

5.4.1.12 – Wherever a composite column is subjected to predominately axial forces, sufficient shear transfer should be provided to ensure that the steel and concrete parts share the loads applied to the column at connections to beams and bracing members.

5.4.1.13 – Except at their base in some structural types, columns are generally not designed to be dissipative. However, because of uncertainties in the behaviour, confining reinforcement is required in regions called *critical regions* as specified in **5.4.4**.

5.4.1.14 – 3.5.2.1 and **3.5.3** concerning anchorage and splices in the design of reinforced concrete columns apply also to the reinforcements of composite columns.

5.4.2. Steel beams composite with slab

5.4.2.1 – The design objective of this subclause is to maintain the integrity of the concrete slab during the seismic event, while yielding takes place in the bottom part of the steel section and/or in the rebars of the slab.

5.4.2.2 – If it is not intended to take advantage of the composite character of the beam section for energy dissipation, **5.5.5** shall be applied.

5.4.2.3 – Beams intended to behave as composite elements in dissipative zones of the earthquake resistant structure may be designed for full or partial shear connection in accordance with EN 1994-1-1:2004. The minimum degree of connection η as defined in EN 1994-1-1:2004 **6.6.1.2** should be not less than 0.8 and the total resistance of the shear connectors within any hogging moment region not less than the plastic resistance of the reinforcement.

5.4.2.4 – The design resistance of connectors in dissipative zones is obtained from the design resistance provided in EN 1994-1-1:2004 multiplied by a reduction factor of 0.75.

5.4.2.5 – Full shear connection is required when non-ductile connectors are used.

5.4.2.6 – When a profiled steel sheeting with ribs transverse to the supporting beams is used, the reduction factor k_t of the design shear resistance of connectors given by EN 1994-1-1 should be further reduced by multiplying it by the rib shape efficiency factor $k_r = 0.8$ for the case of standard trapezoidal ribs.

5.4.2.7 – To achieve ductility in plastic hinges, the ratio x/d of the distance x between the top concrete compression fibre and the plastic neutral axis, to the depth d of the composite section, should conform to the following expression:

$$\frac{x}{d} < \frac{\varepsilon_{cu2}}{\varepsilon_{cu2} + \varepsilon_{a}} \qquad \textbf{5.4)}$$

where ε_{cu2} is the ultimate compressive strain of concrete see EN 1992-1-1:2004); ε_a is the total strain in steel at Ultimate Limit State.

5.4.2.8 – The rule in **5.4.2.7** is deemed to be satisfied when x/d of a section is less than the limits given in **Table 5.4**.

Table 5.4. Limit values *x/d* for ductility of beams with slabs

f_y (MPa	x/d upper limit
355	0.27
235	0.36

5.4.3. Effective width of slab

5.4.3.1 – The total effective width b_{eff} of concrete flange associated with each steel web should be taken as the sum of the partial effective widths b_{e1} and b_{e2} of the portion of the flange on each side of the centreline of the steel web. The partial effective width on each side should be taken as b_e given in **Table 5.5**, but not greater than the actual available widths b_1 and b_2 defined in **5.4.3.2**.

5.4.3.2 – The actual width b of each portion should be taken as half the distance from the web to the adjacent web, except that at a free edge the actual width is the distance from the web to the free edge.

5.4.3.3 – The partial effective width b_e of the slab to be used in the determination of the elastic and plastic properties of the composite T sections made of a steel section connected to a slab are defined in **Table 5.5**.

Table 5.5 – I. Partial effective width b_e of slab for elastic analysis

b_e	Transverse element	b_e for I (elastic)
t interior column	Present or not present	For negative M: 0.05 l *f* For positive M: 0.0375 l
t exterior column	Present	
t exterior column	Not present, or rebars not anchored	For negative M: 0 *f* For positive M: 0.025 l

Table 5.5 – II. Partial effective width b_e of slab for evaluation of plastic moment resistance

Sign of bending moment *M*	Location	Transverse element	b_e for M_{Rd} plastic
Negative *M*	Interior column	Seismic re-bars	0.1 *l*
Negative *M*	Exterior column	ll layouts with re-bars anchored to façade beam or to concrete cantilever edge strip	0.1 *l*
Negative *M*	Exterior column	ll layouts with re-bars not anchored to façade beam or to concrete cantilever edge strip	0
Positive *M*	Interior column	Seismic re-bars	0.075 *l*
Positive *M*	Exterior column	Steel transverse beam with connectors. Concrete slab up to exterior face of column of H section with strong axis or beyond (concrete edge strip). Seismic re-bars	0.075 *l*
Positive *M*	Exterior column	No steel transverse beam or steel transverse beam without connectors. Concrete slab up to exterior face of column of H section with strong axis or beyond (edge strip). Seismic re-bars	$b_b/2$ + $0.7\,h_c/2$
Positive *M*	Exterior column	ll other layouts. Seismic re-bars	$b_b/2 \leq b_{e,max}$ $b_{e,max}=0.05\,l$

5.4.4. Fully encased composite columns

5.4.4.1 – In dissipative structures, critical regions are present at both ends of all column clear lengths in moment frames and in the portion of columns adjacent to links in eccentrically braced frames. The lengths l_{cr} of these critical regions (in metres are specified by **Eq.(3.12)**, with h_c in these expressions denoting the depth of the composite section (in metres).

5.4.4.2 – To satisfy plastic rotation demands and to compensate for loss of resistance due to spalling of cover concrete, the following expression should be satisfied within the critical regions defined above:

$$\alpha\omega_{wd} \geq 30\,\mu_{\varphi}\,\nu_d\,\varepsilon_{sy,d}\,\frac{b_c}{b_o} - 0.035 \qquad \text{5.5)}$$

in which confinement effectiveness factor α is as defined in **3.3.3.6** and the normalised design axial force ν_d is defined as:

$$\nu_d = \frac{N_{Ed}}{N_{pl,Rd}} = \frac{N_{Ed}}{A_a f_{yd} + A_c f_{cd} + A_s f_{sd}} \qquad \text{5.6)}$$

5.4.4.3 – The spacing, *s,* (in millimetres) of confining hoops in critical regions should not exceed

$$s \leq \min\{b_o/2\,,\,260\,,\,9d_{bL}\} \qquad \text{5.7)}$$

f

where b_o is the minimum dimension of the concrete core (to the centreline of the hoops, in millimetres); d_{bL} is the minimum diameter of the longitudinal rebars (in millimetres).

5.4.4.4 – The diameter of the hoops shall be at least d_{bw} = 6 mm.

5.4.4.5 – In critical regions, the distance between consecutive longitudinal bars restrained by hoop bends or cross-ties should not exceed 250 mm.

5.4.4.6 – In the lower two storeys of a building, hoops in accordance with **5.4.4.3**, **5.4.4.4** and **5.4.4.5** shall be provided beyond the critical regions for an additional length equal to half the length of the critical regions.

5.4.4.7 – In dissipative composite columns, the shear resistance should be determined on the basis of the structural steel section alone.

5.4.4.8 – The relationship between the ductility class of the structure and the allowable slenderness (c/t_f) of the flange outstand in dissipative zones is given in **Table 5.3**.

5.4.4.9 – Confining hoops can delay local buckling in the dissipative zones. The limits given in **Table 5.3** for flange slenderness may be increased if the hoops are provided at a longitudinal spacing, *s,* which is less than the flange outstand: s/c < 1.0. For s/c < 0.5 the limits given in **Table 5.3** may be increased by up to 50%. For values of 0.5 < s/c < 1.0 linear interpolation may be used.

5.4.4.10 – The diameter d_{bw} of confining hoops used to prevent flange buckling shall be not less than

$$d_{bw} = \sqrt{\frac{b t_f f_{ydf}}{8 f_{ydw}}} \tag{5.8}$$

in which b and t_f are the width and thickness of the flange, respectively, and f_{ydf} and f_{ydw} are the design yield strengths of the flange and reinforcement, respectively.

5.4.5. Partially-encased members

5.4.5.1 – In dissipative zones where energy is dissipated by plastic bending of a composite section, the longitudinal spacing of the transverse reinforcement, s, should satisfy the requirements of **5.4.4.3** over a length greater or equal to l_{cr} for dissipative zones at the end of a member and $2l_{cr}$ for dissipative zones in the member.

5.4.5.2 – In dissipative members, the shear resistance should be determined on the basis of the structural steel section alone, unless special details are provided to mobilise the shear resistance of the concrete encasement.

5.4.5.3 – The allowable slenderness c/t) of the flange outstand in dissipative zones is as given in **Table 5.3**.

5.4.5.4 – Straight links welded to the inside of the flanges, as additional to the reinforcements required by EN 1994-1-1, can delay local buckling in the dissipative zones. In this case, the limits given in **Table 5.3** for flange slenderness may be increased if these bars are provided at a longitudinal spacing, s_1 , which is less than the flange outstand: s_1/c < 1.0. For s_1/c < 0.5 the limits given in **Table 5.3** may be increased by up to 50%. For values of $0.5 < s_1/c < 1.0$ linear interpolation may be used. The additional straight links should also conform to the rules in **5.4.5.5** and **5.4.5.6**.

5.4.5.5 – The diameter, d_{bw}, of the additional straight links referred to in **5.4.5.4** should be at least 6 mm. When transverse links are employed to delay local flange buckling as described in **5.4.5.4**, d_{bw} should be not less than the value given by **Eq.(5.8)**.

5.4.5.6 – The additional straight links referred to in **5.4.5.4** should be welded to the flanges at both ends and the capacity of the welds should be not less than the tensile yield strength of the straight links. A clear concrete cover of at least 20 mm, but not exceeding 40 mm, should be provided to these links.

5.4.5.7 – The design of partially-encased composite members may take into account the resistance of the steel section alone, or the composite resistance of the steel section and of concrete encasement.

5.4.5.8 – The design of partially-encased members in which only the steel section is assumed to contribute to member resistance may be carried out in accordance with the provisions of **Chapter 4**, but the capacity design provisions of **5.3.1.2** and **5.3.2.3** should be applied.

5.4.6. Filled composite columns

5.4.6.1 – The allowable slenderness d/t or h/t is as given in **Table 5.3**.

5.4.6.2 – The shear resistance of dissipative columns should be determined on the basis of the structural steel section or on the basis of the reinforced concrete section with the steel hollow section taken only as shear reinforcement.

5.4.6.3 – In non-dissipative members, the shear resistance of the column should be determined in accordance with EN 1994-1-1.

A

5.5. DESIGN AND DETAILING RULES FOR MOMENT FRAMES A

A

5.5.1. Specific criteria A

A

5.5.1.1 – 4.3.1.1 applies. A

A

5.5.1.2 – The composite beams shall be designed for ductility and so that the integrity of the A concrete is maintained. A

A

5.5.1.3 – Depending on the location of the dissipative zones, either **5.3.1.2(a)** or **5.3.1.2(b) A** applies. A

A

5.5.1.4 – The required hinge formation pattern should be achieved by observing the rules A given in **4.3.1.2**, **5.5.3**, **5.5.4** and **5.5.5**. A

A

5.5.2. Analysis A

A

5.5.2.1 – The analysis of the structure shall be performed on the basis of the section properties A defined in **5.2**. A

A

5.5.2.2 – In beams, two different flexural stiffnesses should be taken into account: EI_1 for the A part of the spans submitted to positive sagging bending uncracked section) and EI_2 for the A part of the span submitted to negative hogging bending (cracked section). A

A

5.5.2.3 – The analysis may alternatively be performed taking into account for the entire beam A an equivalent second moment of area I_{eq} constant for the entire span: A

A $$I_{eq} = 0.6\, I_1 + 0.4\, I_2$$ A **5.9) A**

5.5.2.4 – For composite columns, the flexural stiffness is given by: A

A $$EI_c = 0.9\,(EI_a + 0.5 E_{cm} I_c + EI_s)$$ A **5.10) A**

Where E and E_{cm} are the modulus of elasticity for steel and concrete respectively; I_a , I_c and I_s A denote the second moment of area of the steel section, of the concrete and of the rebars A respectively. A

A

5.5.3. Rules for beams and columns A

A

5.5.3.1 – Composite T beam design shall conform to **5.4.2**. Partially encased beams shall A conform to **5.4.5**. A

A

5.5.3.2 – Beams shall be verified for lateral and lateral torsional buckling in accordance with A EN 1994-1-1, assuming the formation of a negative plastic moment at one end of the beam. A

A

5.5.3.3 – 4.3.2.2 applies. A

A

5.5.3.4 – Composite trusses should not be used as dissipative beams. A

A

5.5.3.5 – 4.3.3.1 applies. A

A

5.5.3.6 – In columns where plastic hinges form as stated in **5.5.1.1**, the verification should assume that $M_{pl,Rd}$ is realised in these plastic hinges.

5.5.3.7 – The following expression should apply for all composite columns:

$$\frac{N_{Ed}}{N_{pl,Rd}} < 0.30 \qquad \textbf{(5.11)}$$

5.5.3.8 – The resistance verifications of the columns should be made in accordance with EN 1994-1-1:2004, 4.8.

5.5.3.9 – The column shear force V_{Ed} from the analysis) should be limited in accordance with third expression in **Eq.(4.4)**.

5.5.4. Beam to column connections

The provisions given in **4.3.4** apply.

5.5.5. Condition for disregarding the composite character of beams with slab

5.5.5.1 – The plastic resistance of a beam section composite with slab (lower or upper bound plastic resistance of dissipative zones) may be computed taking into account only the steel section (design in accordance with Concept C as defined in **5.1.2**) if the slab is totally disconnected from the steel frame in a circular zone around a column of diameter $2b_{eff}$, with b_{eff} being the larger of the effective widths of the beams connected to that column.

5.5.5.2 – For the purposes of **5.5.5.1**, *totally disconnected* means that there is no contact between slab and any vertical side of any steel element (e.g. columns, shear connectors, connecting plates, corrugated flange, steel deck nailed to flange of steel section).

5.5.5.3 – In partially encased beams, the contribution of concrete between the flanges of the steel section should be taken into account.

A

5.6. DESIGN AND DETAILING RULES FOR COMPOSITE CONCENTRICALLY A BRACED FRAMES A

A

5.6.1. Specific criteria A

A

5.6.1.1 – 4.4.1.1 applies. A

A

5.6.1.2 – Columns and beams shall be either structural steel or composite. A

A

5.6.1.3 – Braces shall be structural steel. A

A

5.6.1.4 – 4.4.1.2 applies. A

A

5.6.2. Analysis A

A

The provisions given in **4.4.2** apply. A

A

5.6.3. Diagonal members A

A

The provisions given in **4.4.3** apply. A

A

5.6.4. Beams and columns A

A

The provisions given in **4.4.4** apply. A

A
A
A
A
A
A
A
A
A
A
A
A
A
A
A
A
A
A
A
A
A
A
A
A
A

5.7. DESIGN AND DETAILING RULES FOR COMPOSITE ECCENTRICALLY BRACED FRAMES

5.7.1. Specific criteria

5.7.1.1 – Composite frames with eccentric bracings shall be designed so that the dissipative action will occur essentially through yielding in bending or shear of the links. All other members shall remain elastic and failure of connections shall be prevented.

5.7.1.2 – Columns, beams and braces shall be either structural steel or composite.

5.7.1.3 – The braces, columns and beam segments outside the link segments shall be designed to remain elastic under the maximum forces that can be generated by the fully yielded and cyclically strain-hardened beam link.

5.7.2. Analysis

5.7.2.1 – The analysis of the structure is based on the section properties defined in **5.2.2**.

5.7.2.2 – In beams, two different flexural stiffnesses are taken into account: EI_1 for the part of the spans submitted to positive sagging bending uncracked section) and EI_2 for the part of the span submitted to negative hogging) bending (cracked section).

5.7.3. Seismic links

5.7.3.1 – Links shall be made of steel sections, possibly composite with slabs. They may not be encased.

5.7.3.2 – The rules on seismic links and their stiffeners given in **4.5.2** apply. Links should be of short or intermediate length with a maximum length *e.*

a) In structures where two plastic hinges would form at link ends:

$$e = 2\frac{M_{p,link}}{V_{p,link}} \qquad \textbf{5.12)}$$

(b) In structures where one plastic hinge would form at one end of a link:

$$e < \frac{M_{p,link}}{V_{p,link}} \qquad \textbf{5.13)}$$

The definitions of $M_{p,link}$ and $V_{p,link}$ are given in **4.5.2.3**. For $M_{p,link}$, only the steel components of the link section, disregarding the concrete slab, are taken into account in the evaluation.

5.7.3.3 – When the seismic link frames into a reinforced concrete column or an encased column, face bearing plates should be provided on both sides of the link at the face of the column and in the end section of the link.

5.7.3.4 – Connections should meet the requirements of the connections of eccentrically braced steel frames as in **4.5.4**.

5.7.4. Members not containing seismic links

5.7.4.1 – The members not containing seismic links should conform to the rules in **4.5.3**, taking into account the combined resistance of steel and concrete in the case of composite elements and the relevant rules for members in **5.4** and in EN 1994-1-1:2004.

5.7.4.2 – Where a link is adjacent to a fully encased composite column, transverse reinforcement meeting the requirements of **5.4.4** should be provided above and below the link connection.

5.7.4.3 – In case of a composite brace under tension, only the cross-section of the structural steel section should be taken into account in the evaluation of the resistance of the brace.

5.8. DESIGN AND DETAILING RULES FOR STRUCTURAL SYSTEMS MADE OF REINFORCED CONCRETE STRUCTURAL WALLS COMPOSITE WITH STRUCTURAL STEEL ELEMENTS

5.8.1. Specific criteria

5.8.1.1 – The provisions in this subclause apply to composite structural systems belonging in one of the three types defined in **5.1.3.1(e)**.

5.8.1.2 – Structural system Types 1 and 2 shall be designed to behave as structural walls and dissipate energy in the vertical steel sections and in the vertical reinforcement. The infills shall be tied to the boundary elements to prevent separation.

5.8.1.3 – In structural system Type 1, the storey shear forces shall be carried by horizontal shear in the wall and in the interface between the wall and beams.

5.8.1.4 – Structural system Type 3 shall be designed to dissipate energy in the structural walls and in the coupling beams.

5.8.2. Analysis

5.8.2.1 – The analysis of the structure shall be based on the section properties defined in **Chapter 3** for concrete walls and in **5.2.2** for composite beams.

5.8.2.2 – In structural systems of Type 1 or Type 2, when vertical fully encased or partially encased structural steel sections act as boundary members of reinforced concrete infill panels, the analysis shall be made assuming that the seismic action effects in these vertical boundary elements are axial forces only.

5.8.2.3 – These axial forces should be determined assuming that the shear forces are carried by the reinforced concrete wall and that the entire gravity and overturning forces are carried by the shear wall acting compositely with the vertical boundary members.

5.8.2.4 – In structural system of Type 3, if composite coupling beams are used, **5.5.2.2** and **5.5.2.3** apply.

5.8.3. Detailing rules for composite walls

5.8.3.1 – The reinforced concrete infill panels in Type 1 and the reinforced concrete walls in Types 2 and 3 shall meet the detailing requirements of **Chapter 3**.

5.8.3.2 – Partially encased steel sections used as boundary members of reinforced concrete panels shall belong to a class of cross-section indicated in **Table 5.3**.

5.8.3.3 – Fully encased structural steel sections used as boundary members in reinforced concrete panels shall be designed in accordance with **5.4.4**.

5.8.3.4 – Partially encased structural steel sections used as boundary members of reinforced concrete panels shall be designed in accordance with **5.4.5**.

5.8.3.5 – Headed shear studs or tie reinforcement (welded to, anchored through holes in the steel members or anchored around the steel member) should be provided to transfer vertical and horizontal shear forces between the structural steel of the boundary elements and the reinforced concrete.

5.8.4. Detailing rules for coupling beams

5.8.4.1 – Coupling beams shall have an embedment length into the reinforced concrete wall sufficient to resist the most adverse combination of moment and shear generated by the bending and shear strength of the coupling beam. The embedment length l_e shall be taken to begin inside the first layer of the confining reinforcement in the wall boundary member. The embedment length l_e shall be not less than 1,5 times the height of the coupling beam.

5.8.4.2 – The vertical wall reinforcements, defined in **5.3.3.4** and **5.3.3.5** with design axial strength equal to the shear strength of the coupling beam, should be placed over the embedment length of the beam with two-thirds of the steel located over the first half of the embedment length. This wall reinforcement should extend a distance of at least one anchorage length above and below the flanges of the coupling beam. It is permitted to use vertical reinforcement placed for other purposes, such as for vertical boundary members, as part of the required vertical reinforcement. Transverse reinforcement should conform to **5.4**.

5.9. DESIGN AND DETAILING RULES FOR COMPOSITE STEEL PLATE STRUCTURAL WALLS

5.9.1. Specific criteria

5.9.1.1 – Composite steel plate shear walls shall be designed to yield through shear of the steel plate.

5.9.1.2 – The steel plate should be stiffened by one or two sided concrete encasement and attachment to the reinforced concrete encasement in order to prevent buckling of steel.

5.9.2. Analysis

The analysis of the structure should be based on the materials and section properties defined in **5.2.2** and **5.4**.

5.9.3. Detailing rules

5.9.3.1 – It shall be checked that

$$V_{Ed} < \omega V_{Rd} \qquad \textbf{5.14)}$$

with the shear resistance given by:

$$V_{Rd} = \omega A_{pl} \frac{f_{yd}}{\sqrt{3}} \qquad \textbf{5.15)}$$

where f_{yd} is the design yield strength of the plate and A_{pl} is the horizontal area of the plate.

5.9.3.2 – The connections between the plate and the boundary members (columns and beams), as well as the connections between the plate and the concrete encasement, shall be designed such that full yield strength of the plate can be developed.

5.9.3.3 – The steel plate shall be continuously connected on all edges to structural steel framing and boundary members with welds and/or bolts to develop the yield strength of the plate in shear.

5.9.3.4 – The boundary members shall be designed to meet the requirements of **5.8**.

5.9.3.5 – The concrete thickness should be not less than 200 mm when it is provided on one side and 100 mm on each side when provided on both sides.

5.9.3.6 – The minimum reinforcement ratio in both directions shall be not less than 0,25%.

5.9.3.7 – Openings in the steel plate shall be stiffened as required by analysis.

CHAPTER 6
PERFORMANCE-BASED SEISMIC DESIGN REQUIREMENTS FOR TALL BUILDINGS

6.1. ANALYSIS PROCEDURES FOR TALL BUILDINGS

6.1.1 – In the linear elastic analysis of tall buildings required for design stages described in **6.3.1** and **6.3.3**, *Multi-mode Response Spectrum Analysis* procedure described in **2.4** or *Linear f Response History Analysis* procedure described in **2.5.1** shall be employed.

6.1.2 – In the nonlinear analysis of tall buildings required for design stages described in **6.3.2** and **6.3.4**, *Direct Integration* procedure shall be employed in the time domain.

6.1.3 – In nonlinear analysis, a minimum seven earthquake ground motion sets shall be used in accordance with **1.2.3** and the acceleration records in the two perpendicular directions shall be applied simultaneously along the principal axes of the structural system. Subsequently directions of acceleration records shall be rotated by 90^o and the analysis shall be repeated. Design basis seismic demands shall be calculated as the average of results obtained from the minimum 2*7 = 14 analysis.

6.1.4 – In the linear or nonlinear analysis of tall buildings, damping ratio shall be taken $\xi = 0.05$ as a maximum. Second order $(P - \Delta)$ effects shall be taken into account.

6.1.5 – In the cases where needed, vertical component of the earthquake ground motion may be considered as well, subject to approval of the *Independent Reviewer s* .

6.2. REQUIREMENTS FOR ANALYSIS MODELING

6.2.1 – Modeling of frame elements shall be made with *frame finite elements* in linear analysis. Modeling in nonlinear analysis can be made with *plastic sections* (*plastic hinges*) in the framework of lumped plasticity approach or through *fiber elements* in the framework of distributed plasticity approach. Regarding the plastic hinge length, an appropriate empirical relationship may be selected from the literature, subject to approval of the *Independent Reviewer(s)*. In nonlinear analysis, alternative modeling approaches may be followed upon the approval of *Independent Reviewer(s)*. In linear and nonlinear models of steel frames, shear deformation in the beam-column panel zone shall be considered.

6.2.2 – In linear analysis, modeling of reinforced concrete walls and their parts shall be made with *shell finite elements*. In simple walls, frame elements may be used as an alternative. When shell elements are used, elastic modulus (E) of shell elements can be appropriately reduced in bending in order to be consistent with the *effective bending rigidities* of the frame elements with cracked sections (see **6.2.4**).

6.2.3 – In modeling reinforced concrete walls and their parts for nonlinear analysis, *fiber elements* or alternative modeling options may be used in the framework of distributed plasticity approach, subject to approval of the *Independent Reviewer(s)*. Shear stiffnesses of reinforced concrete walls shall be considered.

6.2.4 – Effective bending rigidities shall be used for reinforced concrete frame elements with cracked sections. In the preliminary design stage described in **6.3.1**, empirical relationships given in the relevant literature may be utilized. In other design and verification stages described in **6.3**, effective bending rigidity shall be obtained from the section's moment-curvature relationship as follows:

$$EI_e = \frac{M_Y}{\phi_y} = \frac{M_N}{\phi_{y}} \qquad \textbf{(6.1)}$$

where M_Y, represents the state of first-yield in the section. The corresponding curvature ϕ_y represents a state where either concrete strain attains a value of 0.002 or steel strain reaches the yield value, whichever occurs first. The nominal plastic moment M_N corresponding to effective yield curvature ϕ_y is calculated with concrete compressive strain reaching 0.004 or steel strain attaining 0.015, whichever occurs first. In calculating the moment strengths of columns, axial forces due to gravity loads only may be considered.

6.2.5 – In preliminary design stage described in **6.3.1**, design strengths, f_d, of concrete, reinforcing steel and structural steel are defined as the relevant characteristic strengths, (f_k), divided by material safety factors. In other design and verification stages in **6.3**, expected strentghs, (f_e), shall be used as design strengths without any material safety factors. The following relationships may be considered between the expected and characteristic strengths:

Concrete	$f_{ce} = 1.3 f_{ck}$	
Reinforcing steel	$f_{ye} = 1.17 f_{yk}$	
Structural steel (S 235)	$f_{ye} = 1.5 f_{yk}$	**(6.2)**
Structural steel (S 275)	$f_{ye} = 1.3 f_{yk}$	
Structural steel (S 355)	$f_{ye} = 1.1 f_{yk}$	

6.2.6 – Bi-linear backbone curves may be considered in hysteretic relationships of plastic sections plastic hinges of frame elements. Stiffness and strength degradation effects shall be considered upon the approval of *Independent Reviewer s* .

6.2.7 – At floor levels where abrupt changes in particular downward changes occur in lateral stiffness of vertical structural elements, a special care shall be paid for the arrangement of appropriate *trans er floors* with sufficient in-plane stiffness and strength.

6.2.8 – The stiffness of the foundation and the soil medium shall be considered by appropriate models to be approved by the *Independent Reviewer(s* . When needed, nonlinear behaviour of soil-foundation system may be taken into account in design stages described in **6.3.2** and **6.3.4**.

6.3. PERFORMANCE-BASED SEISMIC DESIGN STAGES OF TALL BUILDINGS

Performance-based design stages of tall buildings are described in the following.

6.3.1. Design Stage (I – A): Preliminary Design (dimensioning) with Linear Analysis for Controlled Damage/Life Safety Performance Objective under (E2) Level Earthquake

6.3.1.1 – This design stage aims at preliminary dimensioning of tall building for *Life Safety / Controlled Damage* performance objective (see **Table 6.1**).

6.3.1.2 – A linear analysis shall be performed in the framework of *Strength-Based Design* approach with reduced seismic loads according to **Chapter 2** under (E2) level earthquake for *Normal Occupancy Buildings* according to **Table 1.2**, and under (E3) level earthquake for *Special Occupancy Buildings*.

6.3.1.3 – Minimum base shear requirement given by **Eq.(2.4)** shall be applied.

6.3.1.4 – Preliminary design shall normally follow the design requirements of **Chapters 3**, **4** or **5**, however deviations from those requirements may be permitted upon the approval of *Independent Reviewer(s)*.

6.3.2. Design Stage (I – B): Design with Nonlinear Analysis for Life Safety / Controlled Damage Performance Objective under (E2) Level Earthquake

6.3.2.1 – The structural system of a tall building, which is preliminarily designed in Design Stage (I – A), shall be designed under the same level of earthquake for *Life Safety / Controlled Damage* performance objective.

6.3.2.2 – A nonlinear analysis shall be performed according to the requirements of **6.2** (see **Table 6.1**). Accidental eccentricity effects need not to be considered in this analysis.

6.3.2.3 – The seismic demands obtained according to **6.1.3** as the average of the results of minimum 2*7=14 analysis shall be compared with the following capacities:

a) Interstory drift ratio of each vertical structural element shall not exceed 0.025 at each story in each direction.

b) Upper limits of concrete compressive strain at the extreme fiber inside the confinement reinforcement and the reinforcing steel strain are given in the following for reinforced concrete sections satisfying the confinement requirements:

$$\varepsilon_{cg} = 0.0135 \quad ; \quad \varepsilon_s = 0.04 \qquad \textbf{(6.3)}$$

c) Deformation capacities of structural steel frame elements shall be taken from ASCE/SEI 41-06* for *Life Safety* performance objective.

d) Shear capacities of reinforced concrete structural elements shall be calculated from EN 1992-1-1: 2005 using expected strengths given in **6.2.5**.

___

*ASCE/SEI 41-06: Seismic Rehabilitation of Existing Buildings, American Society of Civil Engineers, 1st edition, 15/05/2007.

e) In the event where any of the requirements given in **a)** through **d** above is not satisfied, all design stages shall be repeated with a modified structural system.

6.3.3. Design Stage (II): Design Verification with Linear Analysis for Minimum Damage/ Immediate Occupancy Performance Objective under (E1) Level Earthquake

6.3.3.1 – The tall building structural system, which is preliminarily designed in Design Stage I – A) and subsequently designed in Design Stage I – B), shall be verified for *Immediate Occupancy / Minimum Damage* performance objective.

6.3.3.2 – A linear analysis shall be performed according to requirements given in **6.2** under (E1) level earthquake for *Normal Occupancy Buildings* and under (E2) level earthquake for *Special Occupancy Buildings* see **Table 6.1**). Accidental eccentricity effects need not to be considered in this analysis.

6.3.3.3 – Verification-basis internal forces shall be obtained as those calculated from linear elastic analysis i.e., q_R 1.0), irrespective of the type of the structural system. Those forces shall be shown not to exceed the strength capacities of cross sections calculated with expected material strengths given in **6.2.5**.

6.3.3.4 – Interstory drift ratio of each vertical structural element obtained according to **2.7.1** shall not exceed 0.01 at each story in each direction.

6.3.3.5 – In the event where **6.3.3.3** and/or **6.3.3.4** is not satisfied, all design stages shall be repeated with a modified structural system.

6.3.4. Design Stage (III): Design Verification with Nonlinear Analysis for Extensive Damage/ Collapse Prevention Performance Objective under (E3) Level Earthquake

6.3.4.1 – The tall building structural system, which is preliminarily designed in Design Stage I – A) and subsequently designed in Design Stage I – B , shall be verified for *Extensive Damage / Collapse Prevention* performance objective.

6.3.4.2 – A nonlinear analysis shall be performed under E3) level earthquake according to requirements given in **6.2** (see **Table 6.1** . Accidental eccentricity effects need not to be considered in this analysis.

6.3.4.3 – The seismic demands obtained according to **6.1.3** as the average of the results of minimum 2*7=14 analysis shall be compared with the following capacities:

a) Interstory drift ratio of each vertical structural element shall not exceed 0.035 at each story in each direction.

b Upper limits of concrete compressive strain at the extreme fiber inside the confinement reinforcement and the reinforcing steel strain are given in the following for reinforced concrete sections satisfying the confinement requirements:

$$\varepsilon_{cg} = 0.018 \quad ; \quad \varepsilon_s = 0.06 \qquad \textbf{6.4)}$$

A

c) Deformation capacities of structural steel frame elements shall be taken from ASCE/SEI A 41-06* for *Collapse Prevention* performance objective. A

d Shear capacities of reinforced concrete structural elements shall be calculated from EN A 1992-1-1: 2005 using expected strengths given in **6.2.5**. A

e) In the event where any of the requirements given in **a)** through **d** above is not satisfied, A all design stages shall be repeated with a modified structural system. A
A

Table 5.1. Performance-based design stages of tall buildings A

Design Stage	**Design Stage A I – A A**	**Design Stage A I – B A**	**Design Stage A II A**	**Design Stage A III A**
Design type f	Prelim. design A (dimensioning A	Design A	Verification A	Verification A
Earthquake f Level f	*Normal class f buildings f* (E2) earthquake A *Special class f buildings f* (E3) earthquake A	*Normal class f buildings f* (E2) earthquake A *Special class f buildings f* E3) earthquake A	*Normal class f buildings f* (E1) earthquake A *Special class f buildings f* (E2) earthquake A	*Normal class f buildings f* (E3) earthquake A
Per ormance f objective f	Life Safety A	Life Safety A	Immediate A Occupancy A	Collapse A Prevention A
Analysis type f	3-D Linear A Response A Spectrum A nalysis A	3-D Nonlinear A Time-history A nalysis A	3-D Linear A Response A Spectrum A nalysis A	3-D Nonlinear A Time-history A nalysis A
Behaviour f Factor f	$q \leq 5.0$ A	– A	$q = 1.0$ A	– A
Story dri t f ratio limit f	% 2 A	% 2.5 A	% 1 A	% 3.5 A
Section f stif ness in R/C f rame members f	Effective A stiffness A	Effective A stiffness A (from moment-A curvature A analysis A	Effective A stiffness A (from moment-A curvature A analysis A	Effective A stiffness A (from moment-A curvature A analysis A
Material f strengths f	Design A strength A	Expected A strength A	Expected A strength A	Expected A strength A
Acceptance f criteria f	Strength A Story drift ratio A	Strains & Story A drift ratio A	Strength A Story drift ratio A	Strains & Story A drift ratio A

A

A

A

______________________________ A

*ASCE/SEI 41-06: Seismic Rehabilitation of Existing Buildings, American Society of Civil A Engineers, 1st edition, 15/05/2007. A

6.4. DESIGN REQUIREMENTS FOR NONSTRUCTURAL ARCHITECTURAL AND MECHANICAL/ELECTRICAL ELEMENTS/COMPONENTS

6.4.1. General

6.4.1.1 – Independently responding appendages (balcony, parapet, chimney, etc) that are supported by the main structural system of the tall buildings, façade and partioning elements, architectural components, mechanical and electrical components and their connections shall be analysed for the seismic effects given in this Section.

6.4.1.2 – Component attachments shall be bolted, welded, or otherwise positively fastened without consideration of frictional resistance produced by the effects of gravity. A continuous load path of sufficient strength and stiffness between the component and the supporting structure shall be provided. Local elements of the structure including connections shall be designed and constructed for the component forces where they control the design of the elements or their connections.

6.4.1.3 – (E3) Earthquake Level (see **1.2.1**) shall be considered for the following nonstructural elements and their attachements to the structure:

a) Elements and components in buildings of *Special Occupancy Class* (**Table 1.2**),

b) Elements and components in buildings of *Normal Occupancy Class* (**Table 1.2**) that are required to remain operational immeadiately after the earthquake,

c) Elements and components related to hazardous material.

6.4.1.4 – (E2) Earthquake Level (see **1.2.1**) shall be considered for nonstructural elements and components other than those classified in **6.4.1.3**.

6.4.1.5 – If the mass of the nonstructural element or component is greater than 20% of the storey mass, the element or the component shall be considered an element of the structural system with its mass and stiffness characteristics.

6.4.2. Equivalent Seismic Loads

6.4.2.1 – The seismic design force, F_e, applied in the horizontal direction shall be centered at the component's center of gravity and distributed relative to the component's mass distribution and shall be determined as follows:

$$F_e = \frac{m_e\, A_e\, B_e}{q_e} \qquad \textbf{(6.5)}$$

where m_e represents the mass, A_e is the maximum acceleration acting on the element or component, B_e represents the amplification factor and q_e refers to behaviour factor defined for the element or component. B_e and R_e are given for architectural and mechanical/electrical components in **Table 6.2** and **Table 6.3**, respectively.

6.4.2.2 – The maximum acceleration acting on the element or component shall be defined as the maximum value to be obtained from the following:

a) Maximum value of average total accelerations obtained from nonlinear analysis at Stage I-B for *Normal Occupancy Class* buildings and at Stage III for *Special Occupancy Class* buildings may be defined as A_e .

b) In particular cases where mass and stiffness characteristics of component or its attachement is required to be considered, A_e may be calculated as a spectral acceleration corresponding to natural period, T_e , of the component from the *floor spectrum* obtained through the analysis in **(b)**. natural period, T_e , may be calculated from;

$$T_e = 2\pi \sqrt{\frac{m_e}{k_e}} \qquad \textbf{(6.6)}$$

where k_e represents the effective stiffness coefficient of the nonstructural element or component. In this case, amplification factor defined in **Eq.(6.5)** shall be taken as $B_e = 1$.

6.4.2.3 – Equivalent seismic load calculated with **Eq.(6.5)** shall not be less than the minimum load defined below:

$$\min F_e = 0.3\, m_e S_{DS} \qquad \textbf{(6.7)}$$

6.4.2.4 – Equivalent seismic load given in **Eq.(6.5)** shall be applied independently in both horizontal earthquake directions in combination with the dead load, service loads of the element or component plus a vertical seismic load equal to $\pm 0.2\, m_e S_{DS}$.

6.4.2.5 – For elements or components suspended from the structural system (with chains, cables, etc), a seismic load equal to 1.4 times the weight of the element or component shall be applied simultaneously in both horizontal and vertical directions.

A

Table 6.2. Amplification and Behaviour Factors for architectural components A

A **rchitectural element or component A** A	$B_e f$	$q_e f$
Nonstructural plain masonry internal walls and partitions A	1.0 A	1.5 A
Nonstructural other internal walls and partitions A	1.0 A	2.5 A
Cantilever elements unbraced or braced below their centres of gravity A (parapets, cantilever internal walls, laterally supported chimneys, etc A	2.5 A	2.5 A
Cantilever elements braced above the centre of gravity (cantilever A internal walls, chimneys, etc A	1.0 A	2.5 A
External walls and connections A	1.0 A	2.5 A
Wood panels A	1.0 A	1.5 A
Penthouses independent from structural system A	2.5 A	3.5 A
Suspended ceilings A	1.0 A	2.5 A
Storage cabinets and laboratory equipment A	1.0 A	2.5 A
ccess floors A	1.0 A	1.5 A
Signs and billboards A	2.5 A	2.5 A
Other rigid components A	1.0 A	2.5 A
Other flexible components A	2.5 A	2.5 A

A

Table 6.3. Amplification and Behaviour Factors for mechanical/electrical components A

A **Mechanical/electrical element or component A** **A**	$B_e f$	$R_e f$
Boilers and Furnaces A	1.0 A	2.5 A
Pressure vessels on skirts and free-standing A	2.5 A	2.5 A
Stacks A	2.5 A	2.5 A
Cantilevered chimneys A	2.5 A	2.5 A
Other A	1.0 A	2.5 A
Piping Systems A	A	A
High deformability elements and attachments A	1.0 A	3.5 A
Limited deformability elements and attachments A	1.0 A	2.5 A
Low deformability elements and attachments A	1.0 A	1.5 A
HVAC System Component A	A	A
Vibration isolated A	2.5 A	2.5 A
Non-vibration isolated A	1.0 A	2.5 A
Mounted in-line with ductwork A	1.0 A	2.5 A
Other A	1.0 A	2.5 A
Elevator Components A	1.0 A	2.5 A
Escalator Components A	1.0 A	2.5 A
General Electrical A	A	A
Distribution systems (bus ducts, conduit, cable tray A	2.5 A	4.0 A
Equipment A	1.0 A	2.5 A
Lighting Fixtures A	1.0 A	1.5 A

A

A

A

A

6.4.3. Limitation of displacements

6.4.3.1 – In cases where nonstructural elements or components are attached to two different points of the same structure experiencing different displacements, or attached to two different structural systems, the effects of relative displacements between the points of attachement shall be considered. Relative displacements shall be calculated from the results of nonlinear analysis of the structural system at Design Stage I-B for *Normal Occupancy Class* buildings (see **6.3.2**) or at Design Stage III for *Special Occupancy Class* buildings (see **6.3.4**).

6.4.3.2 – Relative displacements of nonstructural elements or components, δ_e, shall not be more than the value given in **Eq.(6.8)**.

$$\delta_e \leq (h_x - h_y)\frac{(\delta_i)_{max}}{h_i} \qquad \textbf{(6.8)}$$

where h_x and h_y represent the vertical distances of top and bottom attachement points, respectively, of the nonstructural element or component measured from the relevant floor level. $\delta_{i\,max} / h_i$ is the allowable storey drift ratio specified in **6.3.2** for *Normal Occupancy Class* buildings and in **6.3.4** for *Special Occupancy Class* buildings.

6.4.3.3 – Relative displacements of nonstructural elements or components attached to two different structural systems shall be calculated as the absolute sum of the maximum relative displacements at points of attachement and it shall not be more than the value given in **Eq.(6.9)**.

$$\delta_e \leq h_x \frac{\delta_{iA\,max}}{h_{iA}} + h_y \frac{\delta_{iB\,max}}{h_{iB}} \qquad \textbf{(6.9)}$$

where $\delta_{iA\,max} / h_{iA}$ ve $\delta_{iB\,max} / h_{iB}$ represent the allowable storey drift ratios of the first and second structural systems, respectively, specified in **6.3.2** for *Normal Occupancy Class* buildings and in **6.3.4** for *Special Occupancy Class* buildings.

6.4.4. Nonstructural façade elements and connections

Glass or curtain wall façade elements of tall buildings shall be subjected to static and dynamic tests described in the following standards:

(a) "*Recommended Static Test Method for Evaluating Curtain Wall and Storefront Systems Subjected to Seismic and Wind Induced Story Drifts*", AAMA 501.4-00, American Architectural Manufacturing Association, Schaumburg, Illinois, 2001.

(b) "*Recommended Dynamic Test Method for Determining the Seismic Drift Causing Glass Fallout from a Wall System*", AAMA 501.6-01, American Architectural Manufacturing Association, Schaumburg, Illinois, 2001.

6.5. INDEPENDENT DESIGN REVIEW

Design of all buildings according to this Code shall be peer reviewed and endorsed by independent reviewers in all design stages, starting from the structural system inception stage. The administrative structure of the independent design review process will be established by Dubai Municipality.

CHAPTER 7
STRUCTURAL HEALTH MONITORING SYSTEMS FOR TALL BUILDINGS

Health monitoring systems shall be established in all tall buildings in order to monitor the real behaviour of tall building structural systems, to improve the existing seismic and wind codes and to predict the level of seismic damage in a tall building immediately after the ocurrence of an earthquake. A typical health monitoring system shall have a minimum 8 acceleration sensors distributed in the building, as shown in **Fig.7.1**.

a) Acceleration sensors shall be syncronized and connected to a 24-bit digital recording system equipped with a GPS card. Recording system shall record the building vibrations continuously and transfer the data in real time to a prescribed centre via internet, modem or similar channels. Sufficient battery and disk capacity shall be provided against electricity and communication shortages, which will help the system operate and store data for at least a period of one week.

b Technical specification of sensors and recording systems shall be provided by Dubai Municipality.

c) Vibration records shall be transferred in real time to the *Structural Health Monitoring Centre* of Dubai Municipality. The records shall be stored at this centre as well as by the building owner.

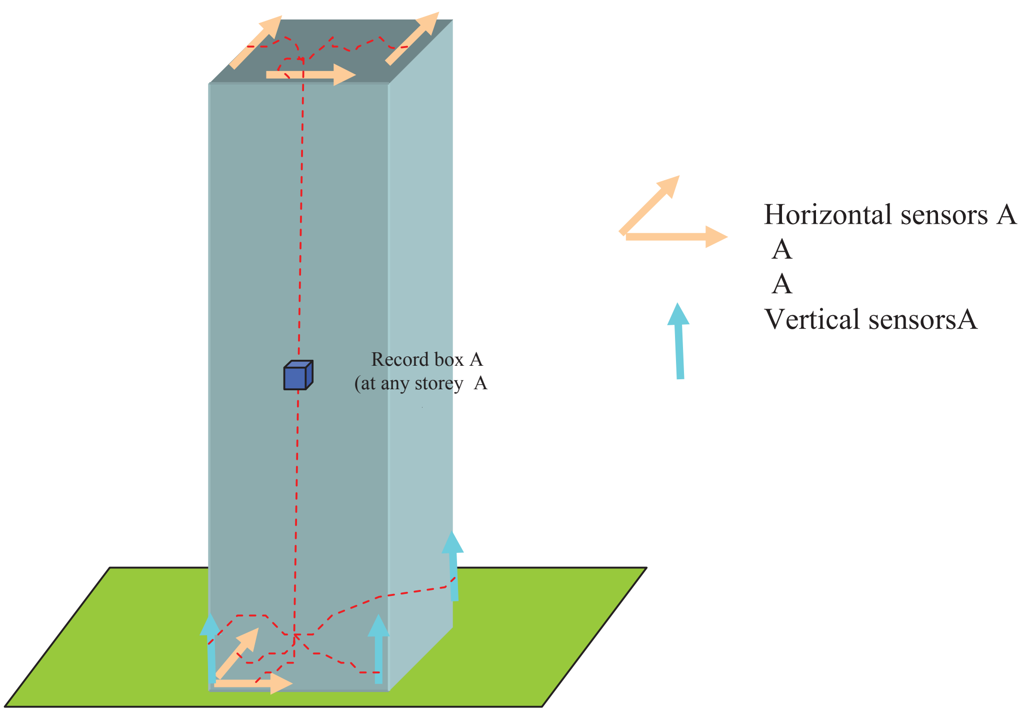

Figure 7.1

NNEX A
SOIL CLASSIFICATION FOR SPECIFICATION OF SEISMIC GROUND MOTION

.1. Soil classification procedure

.1.1 – For the purpose of specifying elastic response spectrum, the site soil shall be classified according to **Table A.1**. Where the soil properties given in **Table A.1** are not known in sufficient detail to determine the soil class, it shall be permitted to assume Soil Class D unless Dubai Municipality determines that Soil Class E or F could apply at the site or in the event that Site Class E or F is established by geotechnical data.

Table A.1. Soil classification parameters

Soil Class	$\bar{v}_s$ (m/s)	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$ (kPa)
A. Hard rock	> 1500	NA	NA
B. Rock	760 – 1500	NA	NA
C. Very dense soil and soft rock	360 – 760	> 50	100
D. Stiff soil	180 – 360	15 – 50	50 – 100
E. Soft clay soil	< 180	< 15	< 50
	or any profile with more than 3 m of soil with Plasticity index: *PI* > 20 Moisture content: $w \geq 40\%$ Undrained shear strength: $\bar{s}_u$ < 25 kPa		
F. Soils requiring site response analysis	1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils 2. Peat and/or highly organic clays with more than 3 m. 3. Very high plasticity clays with more than 7.5 m and *PI* > 75 4. Very thick, soft/medium stiff clays with more than 35 m and s_u < 50 kPa		

.1.2 – The parameters used in **Table A.1** to define the Soil Class are based on the upper 30 m of the site profile. Profiles containing distinctly different soil and rock layers shall be subdivided into those layers designated by a number that ranges from 1 to *n* at the bottom where there are a total of *n* distinct layers in the upper 30 m. The symbol *i* then refers to any one of the layers between 1 and *n*. Parameters characterizing upper 30 m is defined as follows:

(a)

$$\bar{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}} \qquad \textbf{(A.1)}$$

where v_{si} = shear wave velocity in m/s

A

d_i thickness of any layer (between 0 and 30 m). $\sum_{i=1}^{n} d_i$ is equal to 30 m. A

A

b A

$$\bar{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}} \quad \textbf{A.2)}$$

where N_i = Standard Penetration Resistance as directly measured in the field without corrections, and shall not be taken greater than 100 blows/ft. Where refusal is met for a rock layer, N_i shall be taken as 100 blows/ft. N_i and d_i in **Eq.(A.2)** are for cohesionless soil, cohesive soil and rock layers. A

A

(c) A

$$\bar{N}_{ch} = \frac{d_s}{\sum_{i=1}^{m} \frac{d_i}{N_i}} \quad \textbf{A.3)}$$

A

where N_i and d_i in **Eq.(A.3)** are for cohesionless soil layers only. A

d_s total thickness of cohesionless soil layers in the top 30 m. $\sum_{i=1}^{m} d_i = d_s$ **A**

A

d A

$$\bar{s}_u = \frac{d_c}{\sum_{i=1}^{k} \frac{d_i}{s_{ui}}} \quad \textbf{.4)}$$

A

where s_{ui} = undrained shear strength in kPa, and shall not be taken greater than 250 kPa. A

d_c total thickness of cohesive soil layers in the top 30 m. $\sum_{i=1}^{k} d_i = d_c$ A

.2. Steps for classifying Soil Classes C,D,E,F A

A

Step 1: Check for the four categories of Soil Class F see **Table A.1**) requiring site-specific evaluation. If the site corresponds to any of these categories, classify the site as Soil Class F and conduct a site-specific evaluation. A

A

Step 2: Check for the existence of a total thickness of soft clay > 3 m where a soft clay layer is defined by $s_u < 25$ kPa, $w \geq 40\%$ and $PI > 20$. If these criteria are satisfied, classify the site as Soil Class E. A

A

Step 3: Categorize the site using one of the following three methods with $\bar{v}_s$, $\bar{N}$ and $\bar{s}_u$ computed in all cases as specified in A**.1.2**: A

A

a) $\bar{v}_s$ for the top 30 m ($\bar{v}_s$ method) A

b $\bar{N}$ for the top 30 m ($\bar{N}$ method) A

c) $\bar{N}_{ch}$ for cohesionless soil layers $PI < 20$) in the top 30 m and average $\bar{s}_u$ for cohesive soil layers ($PI > 20$) in the top 30 m ($\bar{s}_u$ method) A

If s_u method is used and, $\bar{N}_{ch}$ and s_u criteria differ, the category with the softer soils shall be selected (for example, use Soil Class E instead of D).

.3. Classifying Soil Classes A,B

.3.1 – Assignment of Soil Class B shall be based on the shear wave velocity for rock. For competent rock with moderate fracturing and weathering, estimation of this shear wave velocity shall be permitted. For more highly fractured and weathered rock, the shear wave velocity shall be directly measured or the site shall be assigned to Soil Class C.

.3.2 – Assignment of Soil Class A shall be supported by either shear wave velocity measurements on site or shear wave velocity measurements on profiles of the same rock type in the same formation with an equal or greater degree of weathering and fracturing. Where hard rock conditions are known to be continuous to a depth of 30 m, surficial shear wave velocity measurements may be extrapolated to assess $\bar{v}_s$.

.3.3 – Soil Classes A and B shall not be used where there is more than 3 m of soil between the rock surface and the bottom of the spread footing or mat foundation.