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SYMBOLS

LATIN UPPER CASE

A	Seismic ground acceleration (Chapter 2 and Annex A), random response variable (Chapter 3), cross-sectional area (Annex C)
exA	Probable extreme value, positive or negative, of variable A (Chapter 3)
$B_{,A}$	Value of variable B simultaneous with the limit value of variable A (Chapter 3)
D_r	Coefficient for the equivalent static eccentricity (Annex F)
E	Design seismic action (Chapter 4)
F	Design combination of seismic loads (Chapter 3)
F_d	Design axial force of a connecting beam (Chapter 5)
F_i	Seismic load of storey i (Chapter 3)
G	Permanent actions (Chapter 4)
G_k	Action-effects by permanent actions with their characteristic value (Chapter 4)
H	Height of building (Chapter 3), depth of wall below the free surface (Chapter 5)
H_p	Horizontal seismic force of appendage (Chapter 4)
I	Moment of inertia of a section (Chapter 4)
K_i	Stiffness of storey i (Chapter 3)
L	Width of storey perpendicular to the direction of the seismic action (Chapter 3), length of a building on the direction of analysis (Chapter 3)
M	Bending moment (Chapter 4), total oscillating mass of structure (Chapter 3)
$M_{CD,c}$	Capacity design moment at the end of column (Chapter 4)
M_d	Design bending moment (Chapter 4 and Annex C)
M_E	Moment due to seismic loading (Chapter 5)
M_{EW}	Maximum seismic moment at the wall base (Annex B)
M_{pc}	Bending capacity (Annex C)
M_{pd}	Ultimate design resistance in bending (Annex C)
M_R	Design moment resistance to bending (Chapters 4 and 5)
M_{RC}	Design resistance of column (Annex B)
M_{Rd}	Design moment capacity / resistance (Chapter 4, Annex C)
M_S	Maximum moment due to earthquake combinations (Annex C)
M_v	Moment of the sum of non seismic actions of the seismic combination (Chapter 5)
N	Axial force (Chapter 4), number of levels (floors) (Chapter 3)
N_{cr}	Ideal critical Euler load (Annex C)
N_m	Average of the vertical loads (Chapter 5)

$N_{o\lambda}$ Total axial force of vertical elements of the storey (Chapter 4)
N_{pd} Design ultimate resistance in tension (Annex C)
N_S Maximum axial force due to seismic combinations (Annex C)
P_∞ Action effects from prestressing after time dependant losses (Chapter 4)
$Q_{k,i}$ Action-effects from the characteristic value of live load i (Chapter 4)
R_d Design resistance (Chapter 4)
$\Phi_d(T)$ Value of design spectral acceleration for horizontal component (Chapter 2)
$\Phi_{d,v}(T)$ Value of design spectral acceleration for the vertical component (Chapter 3)
$\Phi_e(T)$ Spectral acceleration of elastic spectrum (Annex A)
R_f Coefficient for equivalent static eccentricity (Annex F)
R_{fy} Yield strength (Annex C)
S Design combination of internal forces A, B, … of a section (Chapter 3)
S_d Design action effect from seismic combinations (Chapter 4)
S_E Seismic action (Chapter 5)
S_{fd} Design action at foundations of superstructure elements (Chapter 5)
S_v Action by the sum of the non seismic loadings (Chapter 5)
T Fundamental period of the building (Chapter 3)
T_1, T_2 Characteristic spectral periods (Chapter 2 and Annex A)
T_Π Period of appendage (Chapter 3)
V_{CD} Capacity design shear force (Annex B)
V_{EW} Maximum shear by the seismic action at the wall base (Annex B)
V_H Additional seismic force at the top of building (Chapter 3)
V_M Shear force corresponding to the ultimate flexural resistance of beam ends (Annex C)
V_0 Total seismic load (base shear force) (Chapter 3)
$V_{o\lambda}$ Total shear force of vertical elements of storey (Chapter 4)
V_{0b} Shear of a beam considered as simply supported (Annex B)
V_{pc} Shear capacity of coupling beam (Annex C)
V_{pd} Ultimate design resistance in shear (Annex C)
W_p Weight of appendage (Chapter 4)

LATIN LOWER CASE

c Arbitrary lever arm of seismic forces F_i (Chapter 3)
d Column dimension parallel to the masonry infill (Chapter 4)
e Eccentricity (Chapter 3)
e_{oi} Static eccentricity of storey i (Chapter 3)

$e_{\tau i}$ Accidental eccentricity of storey i (Chapter 3)

e_{fi} Equivalent static eccentricity of storey i relative to the flexible side (Chapter 3 and Annex F)

e_{ri} Equivalent static eccentricity of storey i relative to the rigid side (Chapter 3 and Annex F)

f_y Yield stress of steel (Chapter 4 and Annex C)

h Storey height (Chapter 4)

i Angle of ground surface to the horizontal (Annex D)

k Permeability (Chapter 5)

l Beam span (Annex C)

l_c Column length (Annex B), coupling beam length (Annex C)

m_i Concentrated mass at level i (Chapter 3)

n Damping correction factor (Chapter 2, Annex B)

p Quotient of the areas of columns and walls (Chapter 3)

p(z) Hydrodynamic variation of the water pressure (Chapter 5)

q Seismic behaviour factor (Chapters 1, 2, 3 and 4)

q_p Reduction factor of appendage (Chapter 4)

q_w Retaining wall behaviour factor (Chapter 5)

r_i Polar radius of inertia of diaphragm relative to the centre of mass M_i (Chapter 4)

r Ratio of a natural period to the preceding natural period, $r = T_j / T_i$ (Chapter 3)

z Appendage support level (Chapter 3), depth of point under examination (Chapter 5)

z_i Distance of level i from the structure base (Chapter 3)

GREEK UPPER CASE

Α,Β,Γ,Δ, Χ Soil classes in terms of seismic risk (Chapter 2)

Δ Design relative lateral displacement of mass centres (interstorey drift) of floor slabs (Chapter 2)

$\Delta_{\varepsilon\lambda}$ Interstorey drift by elastic analysis (Chapter 4)

Σ Symbol of sum

GREEK LOWER CASE

α Soil acceleration normalized to gravity acceleration (Chapter 2), orientation of the principal axis of a building (Chapter 3)

α_k Horizontal active acceleration at the base/top of embankment due to earthquake

(Chapter 5)

α_{CD} Capacity magnification factor of joint (Chapters 4 and 5)

α_h Horizontal seismic coefficient (Chapter 5 and Annex D)

α_v Vertical seismic coefficient (Chapter 5)

β Angle of wall inner face to the vertical, amplification factor for appendage acceleration (Chapter 3)

β_0 Amplification factor of spectrum (Chapter 2 and Annex A)

γ Soil unit weight (Chapter 5 and Annex D)

γ' Soil buoyant unit weight (Chapter 5)

γ_I Importance factor of structure (Chapter 2 and Annex A)

γ_m Material safety factor (Chapter 4)

γ_p Importance factor of appendage (Chapter 4)

γ_{Rd} Factor which transforms the design resistance of beams in its probable maximum value (Chapter 4)

γ_w Unit weight of water (Chapter 5)

δ Angle of friction between wall and ground (Chapter 5 and Annex D)

ε Seismic factor of appendage (Chapters 3 and 4)

ζ Critical damping ratio (Chapter 2)

η Damping modification factor (Chapter 2 and Annex A)

η_v Ratio of shear force taken by walls at the base over the total base shear force (Chapter 4)

θ Foundation factor (Chapter 2), sensitivity factor for lateral deformation (Chapter 4)

λ Slenderness (Annex B)

$\bar{\lambda}$ Normalized slenderness of steel diagonals (Annex C)

ρ Ratio of the wall area in one direction to the total wall and column area

ρ_x , ρ_y Radii of torsional stiffness in the principal directions x and y respectively (Chapter 3)

$\rho_{mx,i}$, $\rho_{my,i}$ Radii of torsional stiffness relative to the centre of mass Mi of diaphragm (i) in the principal directions x and y respectively (Chapter 3)

φ Angle of soil shearing resistance (Chapter 5 and Annex D)

φ_i Translational component of a mode shape at the centre of mass of level i on the direction of the horizontal seismic action (Chapter 3)

Ø Diameter of reinforcing bar

ψ_2 Live load combination factor (Chapter 4)

CHAPTER 1

SCOPE, REQUIREMENTS AND DESIGN CRITERIA

1.1 INTRODUCTION

1.1.1. Scope and field of application

[1] This Code applies to the design of structures (buildings and civil engineering works) against earthquake. The Code does not cover structures for which partial or full earthquake isolation is applied. Additional provisions concerning specific materials are included in the relevant Codes.

[2] The criteria and design rules included in the Code may be applied generally, while application rules are applicable mainly to buildings. For other specific types of structures or for structures for which partial or full earthquake isolation is applied, supplementary provisions are required.

[3] Projects involving high risks for the general population, such as nuclear reactors or dams are not covered by the present Code.

[4] The seismic design procedure proposed in this Code forms a set of rules of maximum acceptable simplification, which, when applied, is considered to satisfy the fundamental requirements for structural integrity. Beside what is referred to in this Code, application of more accurate methods for the analysis and design of structures may be accepted, following the consent and approval of the responsible Public Authority, if satisfaction of these fundamental requirements is directly shown. The above alternative methods of analysis must be based on well founded and recognized scientific principles and, simultaneously, they must achieve the same level of safety as the one aimed for by the present Code.

1.1.2. Content of the Code

[1] This Code includes obligatory provisions which define:

- the minimum seismic design actions and the corresponding load combinations
- the requirements for the structural behaviour under the aforementioned load combinations, as well as, the criteria for checking safety
- the calculation methods of the stresses and strains of the structures,
- the specific construction details for the structural elements and materials.

[2] The responsible Public Authority, simultaneously and according to the clauses of this Code, publishes Comments which refer to subjects of more specific meaning, remarks that help in understanding the text or assure correlation of the paragraphs, or finally methods of restricted validity which may be applied under certain prerequisites.

1.1.3. Correlation with other Codes - Prerequisites

[1] This Code applies in parallel with the Design Codes for structures from specific materials (concrete, masonry, steel, timber, etc) which also include respective special criteria, as well as more detailed practical rules of designing against seismic action.

[2] Reliability of this Code's provisions is largely affected by the exact compliance to the provisions against non seismic actions included in the specific Codes for each material.

[3] For structures designed on the basis of the present Code, no modification of the structural or non structural elements, or change of use of the structures is allowed without prior study of any consequences induced by the aforementioned changes.

1.2. FUNDAMENTAL REQUIREMENTS FOR SEISMIC BEHAVIOUR

[1] The design, construction and use of a structure are considered to sufficiently withstand the seismic hazard - that is, they allow limited and repairable damages on the structural elements under the design earthquake while minimising damages under earthquakes of smaller intensity and greater probability of occurrence - when, under the design seismic actions (see Chapter 2) with acceptably small probability to be exceeded during

the life time of the structure, the following requirements for seismic behaviour are met.

1.2.1. No collapse requirement

[1] The probability of collapse of the entire structure (or parts of it) must be sufficiently small, as defined in the specific criteria included in this Code, as well as in the other applicable Codes, and must be combined with retaining the integrity and sufficient strength reserve following the end of the seismic event.

1.2.2. Damage limitation requirement

[1] Damages to structural elements under the design earthquake must be limited and repairable, while damages after earthquakes of smaller intensity and greater probability of occurrence must be minimized.

1.2.3 Minimum serviceability level requirement

[1] A minimum serviceability level of the structure must be ensured - depending on its use and importance - after the structure has suffered an earthquake with the design earthquake characteristics.

1.3. GENERAL DESIGN CRITERIA

[1] The design seismic actions for the analysis of all structures are divided into:

- global actions acting on the whole structure
- local actions which act on certain structural or non structural elements only or on certain facilities (appendages).

[2] Sufficient quality control shall be performed during all stages of construction and service of the structure, that is check of the design and supervision during construction and service of the building.

[3] The requirements of cl. 1.2 are considered to be satisfied when all the following criteria are satisfied simultaneously, in accordance with the relevant requirements.

1.3.1. General criteria to avoid collapse

The requirement of cl. 1.2.1 is considered to be met, under the effect of the design earthquake (see Chapter. 2), when :

[1] Transfer to the ground of the actions of every element of the superstructure, which is supported on the ground, is reliably ensured without inducing large permanent deformations .

[2] The required strength of all structural elements of the structure is ensured taking into account 2nd order effects, when applicable.

[3] The plastic response mechanism of the structure under the design earthquake is checked using the following specific criteria:

- Capacity design which aims to assure formation of a reliable elastoplastic mechanism with regard to the number and location of plastic hinges, and, at the same time, to avoid brittle failure of the elements, as well as concentration of the plastic hinges in a few structural members only (e.g. soft storey).
- Ensuring a satisfactory relation between the available and the required local ductility at the regions of plastic hinges.

This Code proposes, as a maximum acceptable simplification, a design procedure which assures a satisfactory degree of local ductility so that this criterion can be considered as indirectly fulfilled without being necessary to directly calculate the required and available local ductility.

[4] A minimum ductility level is ensured at every critical region for which even a slight probability for a plastic hinge formation exists. Such regions are, for example, the base and top of all frame columns regardless of performing or not the respective capacity design checks.

[5] The behaviour of the structure is, to a sufficient degree, consistent with the models used (for the analysis and design), that is minimization of the uncertainties related to such calculations is sought after.

[6] Protection measures must also be taken for the building designed, as well as for any adjacent buildings, if such exist, against adverse consequences of collisions during earthquakes.

1.3.2 General criteria for damage limitation

The requirement of cl. 1.2.2 is considered fulfilled if, further to the criteria in cl. 1.3.1, the following three additional criteria are satisfied:

[1] The relative movements of the storeys must be smaller than certain values which are considered to correspond to an acceptable degree of damage of the non-structural elements and specifically of the masonry.

[2] Sufficient strength of the support elements of all kinds of installations and appendages of the structure must be ensured, which corresponds to an acceptable degree of their damage depending on the function and importance of the structure and appendages.

1.3.3 General criteria for minimum serviceability level

[1] In general the Code does not provide specific criteria for meeting the particular requirement of cl. 1.3.2 Such criteria may be required in cases of special structures (hospitals, fire department buildings, etc.).

[2] In the absence of specific criteria, the criteria of cl. 1.3.1 and 1.3.2, which aim at satisfying the requirements to avoid collapse and damage limitation, may also be considered to cover indirectly the minimum serviceability level requirement.

CHAPTER 2

DESIGN SEISMIC ACTIONS

2.1 GENERAL

[1] Design seismic actions may be defined as the earthquake induced oscillatory motion of the ground for which the design of the structures must be performed. Hence, this motion shall be described as ground seismic excitation or ground seismic vibrations.

[2] The intensity of design seismic excitation is conventionally determined by a single parameter, namely the design acceleration A , depending on the seismic risk zone of the project (see cl. 2.3.3).

[3] The ground acceleration A may vary further within the same zone (values $\gamma_1 \cdot A$) depending on the importance category of the "normal risk" structures (see para. 2.3.4).

2.2 MODELING OF SEISMIC EXCITATIONS

2.2.1 Direction and level of application

[1] The design seismic excitation is defined at the free surface of the ground.

[2] The seismic motion of a random point of the ground is determined by the two horizontal components, orthogonal to each other (at random orientation) and the vertical component. Those three components are considered as statistically independent.

[3] Within the ground plan of common buildings, all the points of the ground are considered to undergo the same motion. This motion is considered as constant in the ground surface as well as the foundation level or levels. More specifically, in cases of buildings with different levels of foundation, the design seismic excitation is assumed uniform for all levels.

2.2.2 Determination of seismic excitation

[1] The design seismic excitation is determined by the use of response spectra (in terms of acceleration) of a simple oscillator.

[2] The two horizontal components of the ground seismic excitation are characterized by the same "elastic acceleration spectrum" Φ_e, given in Annex A.

[3] The vertical component's spectrum is deduced from the horizontal components' spectrum by multiplying the latter's ordinates by 0.70 .

[4] For the "equivalent" linear analysis of structures in their post-elastic behaviour region, the "design spectra" R_d of cl. 2.3, determined by modification of the elastic spectra, shall be used.

[5] In special cases of verification of the seismic response by use of time history accelerograms, the accelerograms are defined in Annex A.

2.3 DESIGN SPECTRA

2.3.1 Horizontal components

[1] The design spectra of the earthquake's horizontal components are determined by the following relations (Figure 2.1):

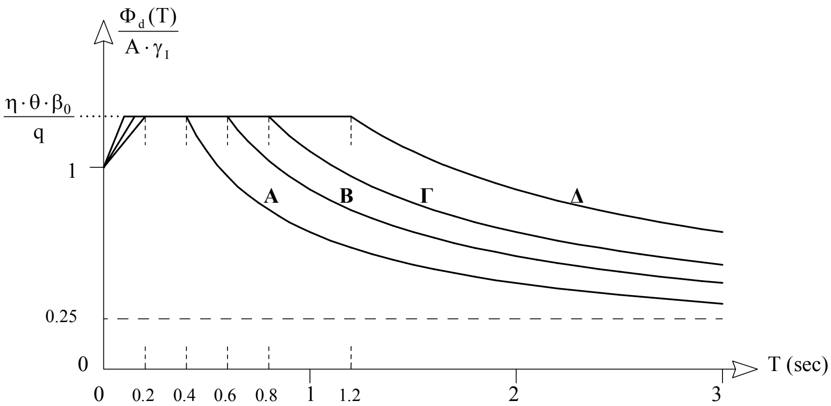


Fig. 2.1: Design Spectrum: $\frac{\Phi_d(T)}{A \cdot \gamma_I}$ [Drawing for $\frac{\eta \cdot \theta \cdot \beta_0}{q}$ =2.5/2.0]

Period Range	Formula	
$0 \le T < T_1$	$\Phi_d(T) = \gamma_I \cdot A\,[1 + \frac{T}{T_1}\,(\frac{\eta.\theta}{q}\,\beta_0 - 1)]$	(2.1.a)
$T_1 \le T \le T_2$	$\Phi_d(T) = \gamma_I \cdot A\,\frac{\eta.\theta.\beta_0}{q}$	(2.1.b)
$T_2 < T$	$\Phi_d(T) = \gamma_I \cdot A\,\frac{\eta.\theta.\,\beta_0}{q}\,(T_2/T)^{2/3}$	(2.1.c)

where:

$A = \alpha \cdot g$, is the maximum horizontal ground seismic acceleration (cl. 2.3.3),

g acceleration of gravity

γ_I is the importance factor of the structure (cl. 2.3.4)

q is the behaviour factor of the structure (cl. 2.3.5)
η is the damping modification factor (par. [2])
θ is the influence factor of the foundation (cl. 2.2.2.6)
T_1 and T_2 are characteristic periods of the spectrum
β_0 is the spectral amplification factor taken equal to 2.5
A,B,C,D category of ground (cl. 2.3.6)

[2] The damping modification factor is calculated using the relation:

$$\eta = \sqrt{\frac{7}{2+\zeta}} \geq 0.7 \qquad (2.2)$$

where the values of critical damping ζ(%), are given in table 2.8 for all types of structures. In special cases of systems which are proven to possess high damping (e.g. radiation damping in the subsoil), the lower bound of coefficient η may be reduced down to the value 0.5, following the prior consent of the Project Owner and special permit by the pertinent Authority. For issuing this permit, a detailed special study is required which fully justifies the reason of the increased damping (e.g. soil – dynamic study in the case of radiation damping) and also includes a quantitative assessment of its contribution to the total damping of the system.

[3] If the natural period T is not calculated, then $\Phi_d(T)$ shall be taken from formula (2.1.b).

[4] It is a requirement for all cases that:

$$\frac{\Phi_d(T)}{A\gamma_i} \geq 0.25 \qquad (2.3)$$

2.3.2 Vertical Component

[1] The spectrum of the vertical component is determined by relations (2.1) with the following modifications:

- instead of the horizontal ground acceleration A, the corresponding vertical component A_v is used and this is taken equal to 0.70 A
- instead of the behaviour factor q, the factor $q_v = 0.5\ q \geq 1.0$ is used
- the value of the foundation factor θ is always taken equal to 1.0.

2.3.3 Ground seismic acceleration

[1] In order for this Code to be applied, the country (Greece) is divided into four Seismic Risk Zones I, II, III, and IV; their limits are defined on the Seismic Risk Map of Greece (Fig. 2.2).

[2] Table 2.1 gives a list of the inhabited areas of Greece along with the Seismic Risk Zone they belong to.

[3] To every Seismic Risk Zone there is a corresponding value of ground seismic acceleration A according to Table 2.2.

[4] The values of the ground seismic accelerations of Table 2.2 are estimated to have a 10% probability of exceedance in 50 years according to the seismological data.

2.3.4 Importance Factor of the Structure

[1] Structures are divided into four categories of importance depending on the hazard for human life and the socioeconomic consequences that may be caused by their possible destruction or interruption of operation.

[2] To every category of importance there is a corresponding value of the importance factor γ_I according to Table 2.3.

2.3.5 Behaviour Factor q

[1] This factor introduces a reduction of the seismic loads caused by the post-elastic behaviour of the actual system compared with those derived by analysis of an infinitely elastic system.

[2] The maximum values of q are given in Table 2.6 according to the material of the structural system and the type of structure. These values are valid upon condition that the system starts to yield at the design earthquake (formation of the first plastic hinge) and then, upon further increase of loading, the formation of a reliable yielding mechanism through creation of a sufficient number of plastic hinges is possible (ductile

behaviour).

[3] In vases where an elastic behaviour is required, q is taken as equal to 1.

2.3.6 Soil Classification

[1] With respect to seismic risk, soils are divided into five classes A, B, Γ, Δ and X as described in Table 2.5.

[2] Construction of permanent works on soil class X can only be executed following detailed research and studies, and provided that adequate measures for improving the soil properties are taken and the existing specific problems are dealt with (see Chapter 5).

[3] A formation of thickness less than 5 m can be considered belonging to the previous soil class with the exception of class X.

2.3.7 Foundation Factor

[1] The foundation factor θ depends, in general, on the depth and stiffness of the foundations.

[2] In soil classes A or B the factor θ takes the value 1.00. In soil classes Γ or Δ the factor θ is permitted to take the values given in Table 2.7 if at least one of the conditions referred to in Table 2.7 is fulfilled and, provided that the resulting design spectral acceleration is not less than what would result for soil class B.

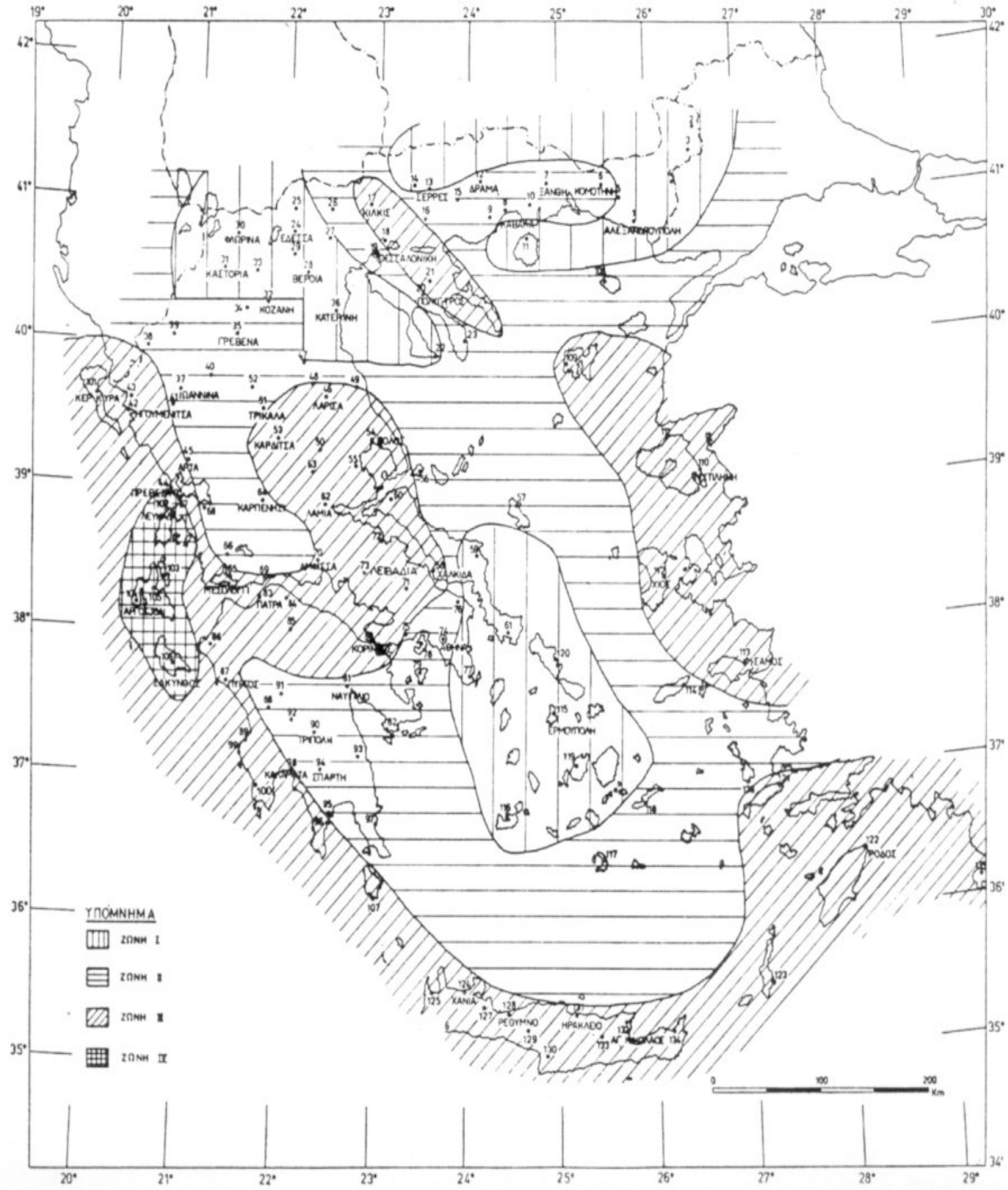


Fig. 2.2: Map of Seismic Risk Zones in Greece

Table 2.1: Inhabited areas (cities, villages, islands) of Greece shown on the Map of Seismic Risk Zones. The first column gives the name of the area, the second the No on the Map and the third the Seismic Risk Zone.

NAME OF AREA	No	ZONE
AEGINA	79	II
AEGION	84	III
AGIA	49	II
AGIOS KYRIKOS	114	II
AGIOS NIKOLAOS	132	III
AGRINIO	66	II
ALEXANDROUPOLIS	1	I
ALMYROS	55	III
AMARION	129	III
AMFILOCHIA	68	II
AMFISSA	70	III
AMORGOS	118	II
ANDRAVIDA	86	III
ANDRITSAINA	88	II
ANDROS	120	I
AREOPOLIS	96	II
ARGOSTOLI	104	IV
ARIDAIA	25	I
ARNAIA	21	III
ARTA	45	II
ATALANTI	72	III
ATHENS	74	II
CHALKIDA	58	III
CHANIA	124	III
CHIOS	112	III
CHRYSOUPOLIS	10	II
CORINTH	80	III
CYME	59	I
DELVINAKIO	38	II
DIDYMOTICHO	3	I
DIMITSANA	91	II
DOMOKOS	63	III
DRAMA	12	II
EDESSA	24	I
ELASSON	47	II
ELEFTHEROUPOLIS	9	II
ERMOUPOLIS	115	I
FARSALA	50	III
FILIATRA	99	III
FILIATTAI	43	III
FLORINA	30	I
GAIUS		III
GIANNITSA	27	I
GOYMENITSA	26	II
GREVENA	35	II
GYTHIO	95	II
KARDITSA	53	III
KARPATHOS	123	III
KARPENISI	64	II
KARYSTOS	61	I
KASTELION	133	III
KASTELORIZO	135	III
KASTORIA	31	I
KATERINI	36	I
KAVALA	8	II
KERKYRA	101	III
KILKIS	17	III
KISSAMOS	125	III
KOMOTINI	6	II
KONITSA	39	II
KOS	121	III
KOZANI	32	II
KYPARISSIA	89	III
KYTHIRA	107	III
LAMIA	62	III
LANGADAS	18	III
LARISSA	46	III
LAYRIO	77	I
LEHAINA		III
LEONIDION	93	II
LEVADIA	73	III
LEYKADA	102	IV
MEGALOPOLIS	92	II
MEGARA	75	II
MESOLOGGI	65	II
METSOVO	40	II
MILOS	116	I
MOIRAI	130	III
MONEMVASIA	97	II
MYRINA	109	III
MYTHIMNA	111	III
MYTILINI	110	III
NAOYSA	29	I
NAUPAKTOS	69	III
NAUPLIO	81	II
NIGRITA	16	II
N. DODONI	41	II
ORESTIADA	2	I
PALIOYRI	22	I
PAROS	119	I
PATRA	83	III
POLYGYROS	20	II

NAME OF AREA	No	ZONE
IGOYMENITSA	42	III
IOANNINA	37	II
IRAKLEIO	131	III
ISTIAIA	60	III
ITHACA	103	IV
KALAMATA	98	III
KALAMBAKA	52	II
KALAMOS	76	II
KALAVRITA	85	III
KALYMNOS	136	II
KANTANOS	126	III
SAPPAI	5	I
SARTI	23	II
SERRES	13	I
SIATISTA	34	I
SIDIROKASTRO	14	I
SITIA	134	III
SKIATHOS	56	II
SKYROS	57	II
SOUFLI	4	I
SPARTA	94	II
THASSOS	11	I
THEBES	71	III

NAME OF AREA	No	ZONE
PORTOCHELI	82	II
PREVEZA	44	III
PTOLEMAÏS	33	I
PYLOS	100	III
PYRGOS	87	III
RETHYMNO	128	III
RODOS	122	III
SALAMINA	78	II
SAMI	105	IV
SAMOS	113	III
SAMOTHRAKI	108	II
THESSALONIKI	19	II
THIRA	117	II
TRIKALA	51	II
TYLOS		III
TYRNAVOS	48	II
VAMOS	127	III
VERIA	28	I
VOLOS	54	III
VONITSA	67	III
XANTHI	7	II
ZAKYNTHOS	116	IV
NEA ZICHNI	15	II

Table 2.2: Ground Seismic acceleration: $A = \alpha \cdot g$ (g: acceleration of gravity).

Seismic Risk Zone	I	II	III	IV
α	0.12	0.16	0.24	0.36

Table 2.3: Importance Factors

Importance Category		γ_1
Σ1	Buildings of small importance for public safety e.g. agricultural buildings, sheds, stables etc.	0.85
Σ2	Ordinary residential and office buildings, industrial buildings, hotels, etc.	1.00
Σ3	School buildings, public assembly buildings, airport terminals and generally buildings where a large number of people gather during the greater part of the day. Buildings that house installation of great economic value (e.g. buildings that house computer centres, special industries, etc).	1.15
Σ4	Buildings whose operation both during an earthquake and after the event is of vital importance, such as telecommunication buildings, power stations, hospitals, fire stations, public services buildings. Buildings that house works of unique artistic value (e.g. museums)	1.30

Table 2.4: Values of characteristic Periods T_1,T_2 (sec)

Soil Class	A	B	Γ	Δ
T_1	0.10	0.15	0.20	0.20
T_2	0.40	0.60	0.80	1.20

Table 2.5: Soil Classes

CLASS	DESCRIPTION
A	Rock or semi-rock formations extending in wide area and large depth provided that they are not strongly weathered. Layers of dense granular material with little percentage of silt-clay mixtures having thickness less than 70 m. Layers of stiff over consolidated clay with thickness less than 70 m.
B	Strongly weathered rocks or soils which can be considered as granular materials in terms of their mechanical properties. Layers of granular material of medium density with thickness larger than 5 m or of high density with thickness over 70 m. Layers of stiff over consolidated clay with thickness over 70 m.
Γ	Layers of granular material of low relative density with thickness over 5 m or of medium density with thickness over 70 m. Silt-clay soils of low strength with thickness over 5 m.
Δ	Soft clays of high plasticity index (I_p > 60) with total thickness over 12 m.
X	Loose fine-grained silt-sand soils under the water table which may liquefy(unless a specific study proves that such a hazard can be excluded or their mechanical characteristics will be improved). Soils which are close to apparent tectonic faults. Steep slopes covered with loose debris. Loose granular soils or soft silty-clayey soils which have been proved hazardous in terms of dynamic compaction or loss of strength. Recent loose backfills. Organic soils. Soils of class Γ with excessively steep inclination

Table 2.6: Maximum values of Behaviour Factor q.

MATERIAL	STRUCTURAL SYSTEM	q
1. REINFORCED CONCRETE	a. Frames or dual system	3.50
	b. Structures consisting of walls acting as cantilevers	3.00
	c. Systems where at least 50% of the total mass is found in the upper 1/3 of the height of the system.	2.00
2. STEEL	a. Moment resisting frames	4.00
	b. Eccentric braced frames *	4.00
	c. Concentric braced frames	
	• diagonal bracings	3.00
	• bracings of types V ή L	1.50
	• bracings of type K (where permitted*)	1.00
	* See Annex C.	
3. MASONRY	a. With horizontal bond beams	1.50
	b. With horizontal and vertical bond elements	2.00
	c. Reinforced masonry (vertically and horizontally)	2.50
4. TIMBER	a. Cantilevers	1.00
	b. Beams – Arches– Glued Panels	1.50
	c. Frames with bolted joints	2.00
	d. Nailed Panels	3.00

Table 2.7: Foundation Factor θ.

Conditions		
1a.	The building has one basement	0.90
1b.	The building has mat foundation	
1c.	The foundation of the building is with piles bearing pile cap beams	
2a.	The building has at least two basements	0.80
2b.	The building has at least one basement and mat foundation	
2c.	Foundation of the building is with piles connected with a common pilecap (not necessarily of a uniform thickness)	
NOTE: A floor is considered as a basement when it has perimeter walls so that the connected slabs are practically immovable.		

Table 2.8: Values of critical damping percentage ζ.

Type of structure		ζ %
Steel:	welded joints	2
	bolted joints	4
Concrete:	unreinforced	3
	reinforced	5
	prestressed	4
Masonry:	reinforced	6
	confined	5
Timber:	glued joints	4
	bolted joints	4
	nailed joints	5

CHAPTER 3

SEISMIC RESPONSE OF STRUCTURES

3.1 GENERAL PRINCIPLES AND ASSUMPIONS

3.1.1 Basis of Analysis

[1] In the context of the present Code, buildings are considered mainly as structures with either an linear-elastic seismic response or, more commonly, with material non-linearities or limited geometric non-linearities (2^{nd} order effects).

[2] In all cases, the seismic response is the result of an “equivalent” linear analysis using the appropriate design spectrum and the corresponding behaviour factor q.

[3] For the estimation of the actual (post-elastic) displacements of the system, the displacements arising from linear analysis using the design seismic action shall be multiplied by the relevant behaviour factor q.

[4] The two horizontal and orthogonal to each other components of the earthquake may have a random orientation relative to the structure.

[5] In general, the influence of the vertical component of the earthquake may be neglected, except in cases of prestressed concrete structures and beams bearing columns in zones of seismic hazard III and IV. In these cases, it is allowed to model and analyse the above structural elements, according to cl. 3.6, independent of the remaining structure. For buildings with bearing masonry, the influence of the vertical seismic component must, in general, be examined.

3.1.2 Methods of Analysis

[1] The present Code specifies the following two methods for linear analysis of the seismic response:

a. (Dynamic) Response Spectrum Method

b. Simplified Spectrum Method (Equivalent Static Method)

The field and way of application of these two methods are defined in cl. 3.4 and 3.5 respectively.

[2] In quite special cases, it is allowed in addition to the above methods, to apply other well founded analysis methods such as linear or non linear analysis with time history accelerograms, etc. These methods shall apply as supplementary checks only and on the safe side.

[3] In general, a 3D model shall be used for the application of any method of analysis for buildings. Use of a 2D model is permitted following a relevant justification of its reliability.

3.2 MODELLING

3.2.1 Degrees of freedom

[1] The number and type of degrees of freedom are chosen, in all cases, so that they can express, in sufficient approximation, all the important deformations and inertia forces of the structure.

[2] For buildings subjected to horizontal seismic actions and under the condition that the diaphragm action of the slabs is ensured, it is sufficient to assume three degrees of freedom per floor (two translations and one rotation).

[3] For buildings where the above diaphragm action is not ensured, the introduction of a sufficient number of additional degrees of freedom is required, with proper discretization, in order to represent the deformation of the plates within their plane.

[3] The support of the structures on the ground is generally considered fixed. Introduction of additional degrees of freedom on the support points is allowed (elastic support).

3.2.2 Modelling of masses

[1] Distinction of the distributed mass of the structure into idealized concentrated masses is performed using the following rules:

- Every point of mass concentration is provided with the mass and moment of inertia of the rigid portion it corresponds to, taking into account the number and type of the degrees of freedom this has.
- Distribution of the concentrated masses over the area of the structure is done so as to maintain the centres of gravity and moments of inertia of the distributed masses.
- It is allowed to justifiably omit the mass moment of inertia and to delete from the model the respective dynamic degrees of freedom.

[2] For buildings subjected to horizontal seismic actions and under the condition that the diaphragm action of the slabs is ensured, it is sufficient to concentrate the mass of every floor and the respective mass moment of inertia about the vertical axis on the centre of gravity of the floor.

[3] The mass values are derived from the vertical loads $G_k+\psi_2Q_k$, where G_k and Q_k are the representative values of the permanent and live loads respectively and ψ_2 is a reduction factor given in Table 4.1.

3.2.3 Modelling of stiffness of load-bearing elements

[1] In the structural model, all load-bearing elements having a considerable contribution in the system stiffness shall be taken into account. In the context of "equivalent linear analysis" adopted by the present Code the stiffness of elements must reflect with sufficient approximation the deformation under the maximum stresses due to the design seismic action. For elements which develop plastic hinges, the secant stiffness in the design yielding point shall be used.

[2] When the material of the structural system is reinforced concrete, the stiffness of the elements shall be calculated assuming stage II condition. Unless a more accurate

estimate of the stiffness at stage II is performed, the flexural stiffness at stage II is allowed to be taken as follows : for columns: equal to that of stage I (uncracked section) without considering the contribution of the reinforcement (stiffness of geometrical section), for structural walls: equal to 2/3 of the above value, and for horizontal elements: equal to 1/2; the torsional stiffness of all elements (when not ignored) shall be taken equal to 1/10 of the corresponding value of stage I.

[3] The scope of the linear methods of analysis, which are acceptable by this Code, provides for :

- Use of a linear model for the behaviour of the structure introducing an adequate "behaviour factor" q.
- Replacement all al types of damping (except hysteretic) with an equivalent viscous-linear damping, represented as a percentage ζ(%) of the critical viscous damping.
- Adoption of construction measures to minimize any special phenomena of non linearity (see cl. 4.1.2.2, 4.1.7 and 5.2.4).

[4] In modelling the foundation soil, it is allowed, in general, to ignore its inertia and damping characteristics and to consider only the elastic characteristics (spring constants).

3.3 DESIGN ECCENTRICITIES

3.3.1 Accidental eccentricity

[1] In order to compensate for the torsional stresses of a building due to factors that cannot practically be modelled, the mass m_i or the seismic force F_i on every floor shall be taken displaced - on each direction successively - from the centre of gravity, perpendicular to the seismic action at a distance equal to the accidental eccentricity $e_{\tau i}$ of the floor i.

[2] The accidental eccentricity $e_{\tau i}$ shall be taken equal to 0.05L where L is the floor dimension perpendicular to the direction of the seismic action.

3.3.2 Application of dynamic spectrum method

[1] When this method is applied, the mass m_i of each floor shall be displaced from the theoretic centre of mass M_i in each direction successively, in accordance with the previous paragraph. Therefore four different systems arise for analysis with the said method.

[2] Because of inherent uncertainty about the value of the accidental eccentricity, it is allowed to evaluate its results without displacement of masses, by using an additional static loading consisting of torsional pairs of the same sign equal to $\pm\, e_{\tau i} F_i$ on every floor. The seismic load of the floor F_i, - if not calculated more accurately - may be taken from relation (3.15) for each direction of the analysis. The results of this loading are added algebraically to the results of the dynamic spectrum method in the direction under consideration.

3.3.3 Application of simplified spectrum method

[1] When this method is applied, for each direction of the building and for each diaphragm, the seismic forces F_i are applied on each side of the centre of mass M_i with the following design eccentricities relative to the (actual or fictitious) elastic axis of the building (Fig. 3.1):

$$\max e_i = e_{fi} + e_{\tau i} \tag{3.1.a}$$

$$\min e_i = e_{ri} - e_{\tau i} \tag{3.1.b}$$

where: $e_{\tau i}$ is the accidental eccentricity and e_{fi} , e_{ri} the equivalent static eccentricities.

[2] The actual or fictitious elastic axis of the building is defined as the vertical axis corresponding to the pole of rotation P_0 of the building's diaphragm (i_0) nearest to level z_0 = 0.8 H, under the action of torsional loading of all diaphragms with torsional moments M_{zi} = + c F_i of the same sign, where h is the height of the building and c an arbitrary lever arm of forces F_i (e.g. c = 1).

[3] In general, the orientation of the principal directions x, y of the building relative to the arbitrary system of reference P_0XY is determined by the angle α from the relation:

$$\tan 2\alpha = \frac{2 \cdot u_{XY}}{u_{XX} - u_{YY}} \tag{3.2}$$

where u_{XX} , u_{YY} and $u_{XY} = u_{YX}$ are the displacements of point P_0 induced by the following loadings of the building with seismic forces F_i :

- Loading along X direction: u_{XX} , u_{YX}
- Loading along Y direction: u_{XY} , u_{YY}

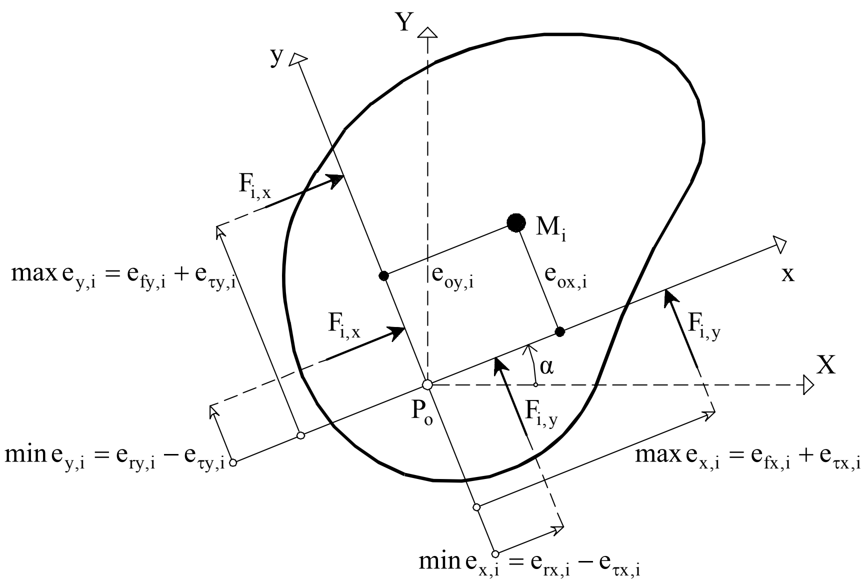


Fig.3.1 Design Eccentricities

[4] In the special case of buildings with parallel arrangement of the main inertia axes of all vertical stiffness elements, the main directions x , y of the building are taken as parallel to those axes.

[5] In buildings without torsional sensitivity, if not calculated more accurately, the equivalent structural eccentricities are given by the following approximations:

$$e_{fi} = 1.50.e_{oi}\ ,\ e_{ri} = 0.50.e_{oi} \qquad (3.3.a,b)$$

where e_{oi} is the structural eccentricity of storey i orthogonal to the assumed direction of forces F_i (i.e. $e_{ox,I}$ or $e_{oy,I}$).

[6] Buildings with torsional sensitivity require either a more accurate calculation of e_{fi} , e_{ri} using the structural eccentricity e_{oi} and the radius of torsional stiffness ρ (see Annex G), or application of the dynamic spectrum method.

[7] A building is considered sensitive to torsion, when at least along one main direction (x or y) the angle of torsional stiffness $\rho_{m,i}$ relative to the center of mass M_i of each diaphragm is smaller or equal to the radius of inertia r_i of the diaphragm ($\rho_{m,i} \le r_i$). The the radii of torsional stiffness $\rho_{mx,i}$ and $\rho_{my,i}$ are given by the relations:

$$\rho_{mx,i} = \sqrt{\rho_x^2 + e_{ox,i}^2} \qquad (3.4.a)$$

$$\rho_{my,i} = \sqrt{\rho_y^2 + e_{oy,i}^2} \qquad (3.4.b)$$

where:

$e_{ox,i}$ and $e_{oy,i}$ are static eccentricities along the directions of the main axes x and y

ρ_x and ρ_y the corresponding radii of torsional stiffness relative to the elastic axis, calculated by the relations:

$$\rho_x = \sqrt{\frac{c \cdot u_y}{\theta_z}}\ ,\ \rho_y = \sqrt{\frac{c \cdot u_x}{\theta_z}} \qquad (3.5.a,b)$$

where:

u_x , u_y displacements of point P_0 for loading of the building with seismic forces F_i along the main directions x and y respectively, and

θ_z rotation angle in diaphragm (i_0) for the torsional loading with the same sign torsional moments $M_{zi} = +c.F_i$.

3.4 RESPONSE SPECTRUM METHOD

3.4.1 General

[1] The response spectrum method is applied without restrictions to all types of structures covered by the present Code.

[2] By this method, the probable extreme values of a random response variable may be calculated using the SRSS (square root of the sum of squares) of the modal values for the said variable.

[3] When applying this method it is sufficient to consider a single orientation of the two horizontal (and orthogonal to each other) components of the earthquake. For q = 1 the elastic spectrum $\Phi_e(T)$ is used (with the use of an appropriate value of the foundation coefficient θ), while for q > 1 the design spectrum $\Phi_d(T)$ is used.

[4] In the common case of structures composed of the same material, it is permitted to use a constant percentage of critical damping ζ for all oscillation modes of the system.

3.4.2 Number of significant modes

[1] For each component of seismic excitation it is obligatory to take into account a sufficient number of modes until the sum of the effective modal masses ΣM_i reaches 90% of the total oscillating mass M of the system.

[2] In cases of special structures (e.g. with large variations of stiffness) where the above mentioned limit cannot be achieved up to the mode with natural period T = 0.03 sec, then the contribution of the remaining modes is taken into account as an approximation, by multiplying the final values of the displacement and internal forces by the factor $M/\Sigma M_i$.

[3] Modes with natural period $T \geq 0.20$ sec are always taken into account.

3.4.3 Superposition of modal responses

[1] Two modes i and j (I < j) with natural periods T_i and T_j ($T_i \geq T_j$) are considered independent when:

$$\frac{1}{r} = \frac{T_i}{T_j} \geq 1 + 0.1\zeta \tag{3.6}$$

where ζ (as %) is the percentage of critical damping of the modes.

[2] For each component of seismic excitation, the probable extreme values ex A of any response variable A are given by the relation:

$$exA = \pm\sqrt{\sum_i \sum_j \left(\varepsilon_{ij} \cdot A_i \cdot A_j\right)} \tag{3.7}$$

where Ai (i = 1,2,…) are the modal values of the variable A, and:

$$\varepsilon_{ij} = \frac{8 \cdot \zeta^2 \cdot (1+r) \cdot r^{\frac{3}{2}}}{10^4 \cdot \left(1-r^2\right)^2 + 4 \cdot \zeta^2 \cdot r \cdot (1+r)^2} \tag{3.8}$$

the correlation coefficient of the two modes i and j ($\varepsilon_{ij} = 1$, $\varepsilon_{ij} = \varepsilon_{ji}$). For independent modes ε_{ij} is taken equal to 0 and in case all modes are independent:

$$exA = \pm\sqrt{\sum_i A_i^2} \tag{3.9}$$

[3] In general, it is not permitted to derive (statistically) the probable extreme value of a response variable using the probable extreme values of two or more variables.

3.4.4 Spatial superposition

[1] For simultaneous action of the three seismic components, the probable extreme value ex A of a response variable A is given by the relation:

$$\text{ex A} = \pm\sqrt{(\text{exA},_{x})^2 + (\text{exA},_{y})^2 + (\text{exA},_{z})^2} \qquad (3.10)$$

where ex $A,_x$, ex $A,_y$ and ex $A,_z$ are the probable extreme values of the said variable for independent seismic action along the directions x, y and z respectively (relations 3.7 or 3.9).

[2] The probable value $B,_A$, simultaneous to ex A, of another response variable is given by the relation:

$$B,_A = \frac{P_{AB}}{\text{exA}} \qquad (3.11.a)$$

where:

$$P_{BA} = P_{AB} = \sum_i \sum_j \varepsilon_{ij} \cdot \left(A_i,_x \cdot B_j,_x + A_i,_y \cdot B_j,_y + A_i,_z \cdot B_j,_z\right) \qquad (3.11.b)$$

is the correlation factor of variables A,B, and

$(A_i,_x, B_j,_x), (A_i,_y, B_j,_y), (A_i,_z, B_j,_z)$, i, j = 1,2,…N

are the modal values of variables A and B for independent seismic action along directions x, y and z , respectively.

[3] For dimensioning of reinforced concrete elements which are acted upon by more than one internal force, it is sufficient to consider consecutively the extreme value of each force and the probable simultaneous (to the said limit value) values of the other forces.

[4] As an alternative to the above mentioned methodology, dimensioning using the least

favourable of the following combinations of internal forces is permitted:

$$S = \pm S_x \pm \lambda S_y \pm \mu S_z$$

$$S = \pm \lambda S_x \pm S_y \pm \mu S_z$$

$$S = \pm \lambda S_x \pm \mu S_y \pm S_z$$

where $\lambda = \mu = 0.30$. In these symbolic relations S_x , S_y and S_z represent vectors of the extreme values of the internal forces A, B…. of the cross section under consideration for independent seismic excitation along directions x, y and z, respectively. In the usual case where the vertical seismic component is ignored (see cl. 3.1.1.[5]) the third combination is omitted and μ is taken as equal to 0 in the first two combinations. Conservative dimensioning using extreme values of all internal forces of the cross section, while taking into account all possible combinations of their signs, is also permitted.

3.5 SIMPLIFIED SPECTRUM METHOD

3.5.1 General – Field of Application

[1] The simplified spectrum method is derived from the dynamic spectrum method by considering, as an approximation, only the fundamental mode of oscillation for each direction considered (single mode method). This simplification allows the direct calculation of the seismic response, by the use of “equivalent” seismic forces, applied as static loads upon the structure, in accordance with cl. 3.3.3.

[2] When applying this method, the two horizontal seismic components are taken parallel to the principal directions of the building and the design spectrum Φd(T) is always used.

[3] The method is applicable in the following cases:

a. Regular buildings up to 10 floors.

b. Non-regular buildings up to 5 floors, provided that diaphragm operation of the slabs is ensured. Non-regular buildings of importance Σ_4 higher than 2 floors are excluded, regardless of the seismic zone they are in; the same stands for non-regular buildings of importance Σ_3 higher than 2 floors in seismic zones III and IV.

[4] A building shall be defined as regular when it satisfies the following conditions:

a. The floors act as non-deformable diaphragms within their plane. This function, in the absence of a more accurate verification, is not considered as ensured for long rectangular buildings (or parts of buildings) with a ratio of their sides greater than 4, as well as for buildings with gaps exceeding 30% of the ground plan of the floor.

b. The increase or reduction $\Delta K_i = K_{i+1} - K_i$ of the stiffness K_i of a floor at any horizontal direction does not exceed the values $0.35K_i$ and $0.50K_i$ respectively. The stiffness of a floor on one direction shall be calculated as the sum of the stiffnesses $E{\cdot}I/h$ of the floor vertical elements.

c. The variation $\Delta m_i = m_{i+1} - m_i$ of the mass m_i of a floor does not exceed the value $0.35m_i$ when there is an increase and $0.50m_i$ when there is a reduction. From this criterion, the upper floor and any staircase end may be excluded.

3.5.2 Equivalent Seismic Loads

[1] The overall magnitude of the lateral seismic load V_0 (base shear force) is computed by the formula:

$$V_0 = M.R_d(T) \qquad (3.12)$$

where :

M is the total oscillating mass of the structure

$R_d(T)$ is the value of the design spectrum acceleration as calculated by formula (2.1)

T is the fundamental uncoupled period of transitional oscillation along the main direction of the building and may be calculated by any recognized approximate method of Mechanics. For a rectangular plan view, the following formula may be applied for the calculation of the fundamental period :

$$T = 0.09 \cdot \frac{H}{\sqrt{L}} \cdot \sqrt{\frac{H}{H + \rho \cdot L}} \tag{3.13}$$

where:

H the height of the building

L the length of the building along the considered direction of analysis

ρ the ratio of the areas of wall sections along a seismic action direction to the total area of the walls and columns.

[2] Distribution of the lateral seismic forces over the height of the structure is performed by the relation:

$$F_i = (V_0 - V_H) \cdot \frac{m_i \cdot \varphi_i}{\sum_j m_j \cdot \varphi_j} \qquad i, j = 1, 2, ...N \tag{3.14}$$

where:

m_i is the concentrated mass on level i

φ_i is the component at level i of the fundamental mode of translational oscillation along the main direction under consideration of the building which may be calculated approximately by any recognized method of Mechanics.

V_H = $0.07\ T\ V_0\ (\leq 0.25\ V_0)$ is an additional force applied at the top of the building when $T \geq 1.0$ sec , and

N is the number of floors.

[3] For regular buildings the distribution of the lateral seismic forces over the building's height is allowed, in general, to be done according to the following formula:

$$F_i = (V_0 - V_H) \cdot \frac{m_i \cdot z_i}{\sum_j m_j \cdot z_j} \qquad i, j = 1, 2, ...N \tag{3.15}$$

where z_i is the distance of level i from the base.

[4] Application of distribution of the lateral seismic loads according to formula (3.15) is also permitted in the following cases:

a. Non-regular buildings of importance Σ_1, Σ_2 and Σ_3 up to 2 floors in any seismicity zone
b. Non-regular buildings of importance Σ_1 and Σ_2 up to 3 floors in seismicity zones I, II and III
c. Non-regular buildings of importance Σ_1 and Σ_2 up to 4 floors in seismicity zones I and II.

3.5.3 Spatial superposition

[1] For simultaneous structural action of the horizontal seismic loads F_i along the principal directions x, y of the building in accordance with cl. 3.3.3, as well as that of the vertical seismic loads in accordance with cl. 3.6, the probable extreme values ex A of any response variable A are calculated by the relation:

$$\text{ex } A = \pm\sqrt{A,_x{}^2 + A,_y{}^2 + A,_z{}^2} \tag{3.16}$$

where $A,_x$, $A,_y$, $A,_z$ are are the values of the said variable (with their sign) for independent static loading of the building along the directions under consideration, x, y and z, respectively.

[2] The probable value $B,_A$, simultaneous to ex A, of another response variable B is calculated by using the relation:

$$B,_A = \frac{A,_x}{exA} \cdot B,_x + \frac{A,_y}{exA} \cdot B,_y + \frac{A,_z}{exA} \cdot B,_z \tag{3.17}$$

where $B,_x$, $B,_y$, $B,_z$ are the values of variable B (with their sign) for independent static loading of the building along the directions under consideration, x, y and z, respectively.

[3] For the dimensioning of reinforced concrete elements, the methodology of cl. 3.4.4.[3] is applied.

[4] Alternatively, instead of the previous methodology, dimensioning by using the least favourable of the following combinations of static loadings is permitted:

$F = \pm F_x \pm \lambda \cdot F_y \pm \mu \cdot F_z$

$F = \pm \lambda \cdot F_x \pm F_y \pm \mu \cdot F_z$

$F = \pm \lambda \cdot F_x \pm \mu \cdot F_y \pm F_z$

Where $\lambda = \mu = 0.30$. In these symbolic relations, F_x , F_y and F_z represent the vectors of seismic loads along directions x, y and z and F represents the "resultant" seismic loading. In the usual case of ignoring the third vertical component of the earthquake (see cl. 3.1.1.[5]), the third combination is omitted and μ is taken equal to 0 in the first two.

3.6 VERTICAL SEISMIC ACTION

[1] Checking of isolated structural elements against vertical seismic action can be performed by the following simplified procedure :

a. The vertical seismic action is applied at the support points of the element

b. The fundamental period of the element is calculated by the Rayleigh formula:

$$T = 2\pi \cdot \sqrt{\frac{\sum_i m_i \cdot y_i^2}{\sum_i m_i \cdot y_i}} \qquad (3.18)$$

where y_i (I = 1,2,...,n) are the displacements of the concentrated masses m_i due to the vertical loads $m_i \cdot I$.

c. The vertical seismic loads are given by the formula:

$$F_i = M \cdot \Phi_{d,v}(T) \cdot \frac{m_i \cdot y_i}{\sum_j m_j \cdot y_j} \qquad (i, j = 1,2,...n) \qquad (3.19)$$

where M is the oscillating mass of the structure, $\Phi_{d,v}(T)$ is the value of design spectral acceleration and n is the number of concentrated masses m_i.

[2] The seismic loads F_i are applied statically upon the structure and the resulting internal forces of the structure as well as that of its supporting elements is added to the internal forces due to the horizontal components of the earthquake, in the absence of a more accurate form of superposition.

[3] The procedure of the above paragraph may be applied independently of the analysis method for the horizontal seismic excitation.

3.7 APPENDAGES OF BUILDINGS

[1] Appendages of buildings are structures or parts of structures which do not constitute actual members of the structural system as for example parapets, chimneys, etc. The seismic response of an appendage is affected by the seismic response of the building because the motion of the support point on the building is different from the motion of the ground.

[2] Unless a more accurate calculation is performed, the horizontal seismic force for the analysis of appendages and their support elements is computed by relation (4.17) where the seismic factor ε is given by the formula :

$$\varepsilon = \alpha \cdot \beta \cdot (1+z/H) \qquad (3.20)$$

where

$\alpha = A/g$,

$$\beta = \frac{2}{1+\left(1-T_{\pi}/T\right)^2} \geq 1 \qquad (3.21)$$

T_{π} = period of appendage considered as fully fixed in the supporting substructure
T = the fundamental period of the building
z = support level of the appendage
H = height of the building

[3] In case of installations of great importance or hazardous ones, it is recommended to perform more accurate analysis using response spectrum of the supporting base and realistic models of the installations.

CHAPTER 4

DESIGN CRITERIA AND APPLICATION RULES

4.1 AVOIDANCE OF COLLAPSE

4.1.1 Criteria

[1] Formation of an elastoplastic mechanism with reliably safe post-elastic behaviour is generally acceptable, in the response of a structure to the design earthquake. Such a behaviour is deemed to be ensured by the following criteria:

- Provision for a minimum level of strength (resistance) of all load-bearing elements (including the foundations) which corresponds to the design seismic actions of chapter 2, increased, wherever necessary, by 2nd order effects.
- Provision for overall ductility, i.e for sufficient capability of energy absorbtion by post-elastic deformation.
- Minimization of factors causing uncertainties in the evaluation of the seismic response.

The relevant application rules are given in the following paragraphs.

4.1.2 Design actions

4.1.2.1 Earthquake Load Combination

[1] The design earthquake defined in chapter 2 constitutes an accidental action whose action-effects are combined with those of the other actions as follows:

$$S_d = G_k + P_{00} \pm E + \Sigma\psi_{2,i}Q_{ki} \qquad (4.1)$$

[2] In this formula the following symbols for the action-effects are used:

G_k due to permanent loads with their characteristic value
P_∞ due to prestressing after the time dependent losses
E due to the design earthquake
Q_{ki} due to the characteristic value of live load i
ψ_2 is the value of the combination factor for long term ("quasi permanent") variable actions

[3] Imposed deformations, such as those caused by temperature variation and gradient, concrete shrinkage and settlement of supports, do not need to be included in the earthquake combination. Moreover, earthquake is not combined with other accidental actions (e.g. collision of vehicles or ships).

[4] Until defined in a special Code, the values of the combination factor shall be taken from the following table 4.1.

Table 4.1 : Load Combination Factors ψ_2

No		**Live Loads**	ψ_2
1	1.1	Dwelling units, offices, shops, hotels, hospitals	0.3
	1.2	Public assembly areas often used (schools, theatres, grandstands, etc)	0.5
	1.3	Parking areas	0.6
	1.4	Areas for long-term storage (libraries, records rooms, warehouses, tanks, silos, water towers, etc)	0.8
	1.5	Non-accessible roofs	0.0
2		Wind	0.0
3		Snow (for non-accessible roofs only)	0.3

4.1.2.2 2nd Order Effects

[1] Unless a more rigorous analysis is performed, the variation of the stress caused by deformations of the whole structure under the earthquake combination of relation (4.1) (P-Δ effects) may be omitted, when the sensitivity factor θ against lateral deformation,

as defined in relation (4.2), does not exceed 0.10.

$$\theta = \frac{N_{o\lambda}\Delta}{V_{o\lambda}h} \qquad (4.2)$$

where:

$N_{o\lambda}$, $V_{o\lambda}$ are respectively the total axial and shear force of the vertical elements of the storey for the combination (4.1)

h is the storey height

Δ is the relative design displacement of the storey slabs. The value of Δ shall be computed by the formula

$$\Delta = q \cdot \Delta_{\varepsilon\lambda} \qquad (4.3)$$

where:

q is the behaviour factor used in the analysis

$\Delta_{\varepsilon\lambda}$ is the relative displacement of the mass centre of the storey slabs, measured in the plane of the most onerous boundary frame, as derived for the combination (4.1) by an elastic analysis, using either the equivalent static method or the dynamic response spectrum method.

[2] The limitation of θ shall be checked independently on two orthogonal axes X and Y.

[3] In case $0.10 < \theta \leq 0.20$, the 2nd order effect, due to lateral deformation of the storeys, may be taken into account approximately as an increase of the respective seismic action by the factor $1/(1-\theta)$.

[4] The value of θ shall not exceed 0.20 in any case.

[5] It is noted that exception from 2nd order effects verification due to lateral deformation, as determined in paragraph [1], as well as the corresponding effects, as determined in paragraphs [3] and [4], cover all 2nd order effects due to lateral deformation (sway) of the storeys. Therefore, the further verification of vertical load-carrying elements under seismic combination effects may be performed under the assumption of a non-sway structure.

4.1.3 Resistance verifications

[1] At the critical sections of all elements of a structure, the basic safety inequality must be satisfied

$$S_d \leq R_d \qquad (4.4)$$

where:

S_d is the design action-effect as derived by combination (4.1)

R_d is the design resistance according to the Codes for the relevant material using the values of partial material factors (γ_m) which are valid for the basic combinations of the usual actions.

[2] When the action-effect has more than one component with considerable interaction in the strength (e.g. bending with axial force or bi-axial bending with axial force), the safety inequality shall be satisfied for a maximum and a minimum value of each component taking into account the interaction of the corresponding values of the other components.

4.1.4 Requirement for energy dissipation capability (ductility) of the overall structure - General rules for capacity design

[1] In order to ensure the capability of the structure to dissipate energy during its response under the design seismic action, without global or local collapse, the post-elastic response must be of ductile type and be distributed in the largest possible number of load bearing elements and in regions of limited length (=plastic hinges). This requires avoidance of all probable brittle failure modes which may precede reaching this state.

[2] For members under flexure, the post-elastic response must be limited to the formation of plastic hinges at the ends of the elements. For steel vertical bracing elements, post elastic response may be foreseen on tensile diagonals or on shear or flexural hinges of limited length (eccentric braced frames).

[3] "Probable" or intended regions of plastic hinges are those for which formation of hinges has a great probability, or is intended to occur. “Potential” regions of plastic hinges are

those which, though having small probability of hinge formation, must however have increased ductility because of their highly critical location for the stability of the structure. Such regions are all the ends of columns even when the probable regions of plastic hinges are in beams.

[4] Such a reliable elastoplastic response mechanism of the structure at the peaks of the earthquake action is ensured by the capacity design, that is by a procedure providing an adequate hierarchy of resistances of the structural elements. Specifically, the general methodology of the capacity design is the following:

- In all potential and probable regions of plastic hinges, sufficient local ductility is ensured (curvature ductility for moment resisting frames) and the respective strength verification (bending with axial force for frames) is performed with the section forces derived by the most adverse seismic combination (relation 4.1).

- The section forces for the capacity design are computed; namely, the forces derived by the equilibrium conditions of an element or group of elements when at the potential regions of plastic hinges, the probable maximum value of ductile strength (overstrength) is developed. For all structural elements which include or are close to plastic hinges, the verification for avoidance of brittle failure modes is performed using these capacity design effects. The same effects are also used for verification of ductile failure modes (e.g. flexure) in regions where the formation of plastic hinges must be avoided.

- For multi-storey buildings, measures are taken in order to avoid formation of "soft storey mechanism", that is of concentration of the plastic hinges on one storey only.

- In the capacity design verifications defined below (see 4.1.4.1.[2], 5.2.2 and Annex B), the design moment resistance M_R of a plastic hinge section, on the basis of which the overstrength is determined, shall be taken equal to the maximum value that corresponds to simultaneous action of the axial force which is caused by the seismic combination used in the respective capacity design check. This resistance is always calculated using the final (actual) dimensions and the final total reinforcement of the section.

[5] In reinforced or prestressed concrete structures and steel or bearing wall buildings the verifications for ensuring a reliable elastoplastic mechanism are not required when a behaviour factor q not exceeding the minimum of 1.5 or q/2, and in any case not smaller than 1.0 is used, where q are the values given in Table 2.6.

Therefore in such structures the capacity verifications of cl. 4.1.4.1 are not required. Similarly, the requirements of cl. 4.1.5 and 4.1.6 as well as the corresponding application rules of Annexes B and C (with the exception of the requirements of cl. C.5.2[2]) may also be waived. In checking the foundations in accordance with cl. 5.2.2, the value of factor α_{CD} shall be taken as equal to 1.

In steel buildings where the seismic load resisting system includes class 4 cross sections to Eurocode 3, a behaviour factor q = 1 shall used.

4.1.4.1 Avoidance of "soft storey mechanism"

[1] For buildings consisting of moment resisting frames, formation of soft storey mechanism must be avoided. If a more detailed analysis is not performed, this can be achieved by avoiding development of plastic hinges in the columns and by foreseeing the probable regions of plastic hinges in the beams. For this purpose, with the exception of the cases mentioned in cl. 4.1.4.2, the columns shall be verified against bending with axial force, using the capacity design moments (M_{CD}) instead of the moments deriving from the load combination (4.1).

[2] The capacity design moment $M_{CD,c}$ at the end of a column acting within a plane frame may be derived from the maximum moment of the column M_{Ec}, at the same location and direction, as determined by the analysis under the seismic action, using the following relation:

$$M_{CD,c} = \alpha_{CD} M_{Ec} \qquad (4.5)$$

where factor α_{CD} (factor of capacity magnification of the joint), common to both the columns above and below the joint, is:

$$\alpha_{CD} = \gamma_{Rd}\, \Sigma M_{Rb} / |\, \Sigma M_{Eb}\,| \qquad (4.6)$$

where:

ΣM_{Rb} is the sum of the final resistance moments of the beams at the frame joint, in the direction they are activated by the seismic action that causes moment M_{Ec}

ΣM_{Eb} is the sum of moments of the same beams as determined by the analysis under the same seismic action that causes moment M_{Ec}

γ_{Rd}=1.40 is a factor which transforms the design resistance of beams to its probable maximum value.

[3] The sign of the acting moments must be consistent with a common direction of application on the joints. Verification of the columns is permitted to be performed at the interfaces with the upper and lower faces of the beam, with a respective reduction of the capacity moments, based on the shear forces that will be derived.

[4] For every joint of a plane frame, two values of factor α_{CD} are calculated in general, which correspond to the resistances of the beams as these are activated by two seismic actions of opposite direction.

[5] For joints where the moment of the vertical element above $M_{Ec,1}$ is larger than the sum of the moments exerted by the girders ($|M_{Ec,1}| > |\Sigma M_{Eb}|$) the capacity design moment shall be computed by the formula:

$$M_{CD,c} = 1.40\, M_{Ec} \geq M_{SC} \qquad (4.7)$$

where M_{SC} is the moment obtained from the seismic combination (4.1).

[6] If the column belongs to a frame in the other direction also, the verification shall be performed against bi-axial bending using the capacity moment in the first direction while in the other direction the moment derived by combination (4.1) corresponding to a seismic action in the first direction shall be applied. In this case, capacity check on the direction of the other frame must be performed in a similar way.

4.1.4.2 Exceptions from the rule to avoid plastic hinges in columns

[1] The following cases are excepted from the mandatory application of the rule to avoid formation of plastic hinges on columns:

a. Buildings of any structural system

[1] The vertical elements of the top storey, as well as of any staircase ends above it. Also, the vertical elements of one-storey buildings, as well as of two-storey regular buildings for which addition of another storey is not foreseen.

[2] The regions of support of vertical elements on foundation elements (footings or basement walls). In these regions it is not feasible to avoid the probability of plastic hinge formation. Checking of the column sections in these regions is performed with a moment 1.35 $M_{Ec} \geq M_{Sc}$ in order to approach the resistance level of the other critical regions of the column and to reduce, accordingly, the required ductility.

[3] Rectangular walls, which participate in a frame system with their weak sectional moment of inertia, do not need to be checked in capacity about the weak axis as long as the frame function is ensured by the other vertical elements.

[4] In intermediate columns of plane frames, the factor α_{CD} does not need to be taken greater than the value of the behaviour factor q which was used to determine the seismic action (i.e. $\alpha_{CD} \leq q$).

b. Buildings with dual system of adequate arrangement

[1] In buildings with load-bearing system consisting of frames and sufficient number of adequately reinforced concrete walls properly arranged, it is not mandatory to apply the rule of avoiding plastic hinge formation in the columns (for definitions concerning walls see B.1.4).

[2] The walls are considered sufficient in a certain direction if in this direction the ratio n_v= shear taken by walls at the base over the total base shear force, satisfies the following

condition

$$n_v \geq 0.40 \quad (4.8)$$

For this check the walls and columns may be taken as fully fixed at the base.

[3] The arrangement of walls must be such as to eliminate the possibility of soft storey formation through torsional deformation of the building. This is considered to be achieved when one the following conditions is satisfied:

a) If, for every floor, except the uppermost one, and along at least one direction, two parallel walls are available on both sides of the centre of mass, the distance between the two exceeding 1/3 of the corresponding plan dimension of the building's structural system and paragraph [2] is satisfied in both directions

b) If the building is not torsionally sensitive in accordance with the criterion of cl. 3.2.3.[7].

c) If the first two significant modes are mainly translational. This is considered to take place when the distance between the pole of rotation of the diaphragms in the said modes and the centre of mass, is greater than the inertia radius of the diaphragm. In general it is sufficient to perform this check in the first floor only and to floors over any vertical discontinuity of the walls, except for the uppermost floor.

[4] In buildings where one of the conditions (a), (b), (c) of paragraph [3] is satisfied, the frames parallel to the direction having adequate walls according to the condition (4.8) are excepted from application of the rule of cl. 4.1.4.1.

4.1.5 Special Requirements for Reinforced Concrete Buildings

[1] Sufficient overstrength of the structural elements, which are to remain in the elastic range, must be provided for, and avoidance of brittle modes of failure must be ensured.

[2] In regions of plastic hinges, measures must be taken to ensure sufficient local ductility.

[3] The aforementioned requirements are considered to be met when the special rules of application given in Annex B are adhered to.

4.1.6 Special Requirements for Steel Buildings

[1] Sufficient overstrength of the structural elements, which are to remain in the elastic range, must be provided for so as to ensure that yield is limited in the regions of plastic hinges. The overstrength factor shall be taken at least equal to the ratio of the upper and lower limits of the yield stress and not less than 1.20.

[2] The plastic hinge regions must have sufficient strength to withstand the actions deriving from the seismic combinations. It must also be ensured that yield will occur in the foreseen ductile mode (tension of the whole section, yield of the flanges in bending, yield of the web in shear).

[3] The configuration of the sections in plastic hinge regions must ensure sufficient local ductility.

[4] Until a specific Code for Steel Structures is issued, the aforementioned requirements are considered to be met when the special rules of application given in Annex C are adhered to.

4.1.7 Minimization of Uncertainties for the Seismic Behaviour

4.1.7.1 Configuration of the Structural System

[1] In the phase of configuring the structural system, minimization of the uncertainties about the seismic behaviour must be targeted. As a general rule, the system's configuration must aim at the maximum feasible degree of simplicity and regularity, as well as - at the same time - at structural redundancy of the system, so that alternative force paths are ensured. Adverse interaction of the load-bearing system and the infills must also be avoided.

Specifically, the accomplishment of the following targets must be aimed at:

a. Configuration of the system in plan

[1] Arrangement of the vertical elements (columns or/and walls) in such a way as to minimize the rotational deformation of the building. This is achieved by the symmetrical arrangement of the stiffer vertical elements near the perimeter or, if this is not possible, by the arrangement of walls parallel to and near to three at least (non-collinear) sides of the perimeter.

[2] Ensuring a substantial frame action for most of the columns in conjunction with girders (beams) of sufficient stiffness. Wherever this is not possible (e.g. flat slabs or ribbed slabs), it is necessary to provide for sufficient structural walls in both directions (according to cl. 4.1.4.2.b).

[3] Appropriate configuration of the slabs in plan on every floor so as to ensure substantial diaphragmatic function (rigid plate action) with regard to both deformation and strength. To this purpose, elongated plan shapes with a ratio of maximum to minimum dimension larger than 4.0 must be avoided; so must shapes consisting of elongated elements (of shape L, Π, etc). Wherever this is not possible, the effect of the plate deformation on the distribution of horizontal forces must be taken into account with sufficient approximation. Large recesses, which create weak areas in the diaphragm, must also be avoided. The diaphragm strength at such areas must be checked and sufficient reinforcement must be provided for, even by using simplified but conservative assumptions. For the same reason, differences in elevation of slabs within the same floor must be avoided. Finally, the integrity of wall to slab connections on every floor must be ensured in the direction of the wall in the areas of staircases, lift pits, ductwork openings, skylights, etc.

In case of partial connection of a wall with a slab, it must be checked that the entire transferred force is carried by reinforcement. This verification shall be performed with the design value of the force as determined by the capacity design of the wall (Annex B, B1.3) or by using a behaviour factor q = 1.0.

[4] In order to minimize uncertainties in the post-elastic interaction of the load-bearing system with an infill having substantial stiffness, it is recommended that a dual system

is selected with frames and structural walls according to cl. 4.1.4.2.b. This option becomes obligatory in buildings when the infill is discontinued at a floor, either because it was so designed or because it is possible to be so in the future, (e.g. Pilotis or shops without masonry infills at the ground floor).

b. Configuration in elevation

[1] Achieving of continuous and regular distribution of the stiffness of the vertical elements (frames or walls), as well as of the masses and the masonry infills. In areas of abrupt change (discontinuity) in stiffness of the vertical elements (e.g. at the interruption of important walls at a certain floor or due to the introduction of the peripheral walls of the basement below the ground floor), the necessary redistribution of the shear force on the vertical elements through the diaphragmatic action of the corresponding slab must be ensured. In case of doubt, the adequacy of the diaphragmatic function of the slab must be checked even with approximate methods.

[2] Foundations of the vertical elements to be homogeneous and at the same level as far as possible.

c. Detailing

[1] For concrete elements cast in situ, certain minimum dimensions for the main bearing elements which ensure reliable quality of construction must be kept.

[2] Avoiding of eccentric connections between horizontal and vertical elements at frame joints.

[3] Embedment - in the longitudinal direction - of pipes for drainage, water, sewage, etc, or electrical ductwork within vertical reinforced concrete elements is not allowed. Also, transverse crossing of pipes with vertical elements at regions of potential or possible plastic hinges is not allowed.

[4] It must be avoided to have discontinuity in elevation of the masonry infills between columns in such a way that the shear action of the infills creates an intermediate lateral

support to the column.

[5] In case of non-monolithic support of a structure upon another structure (e.g. sliding support, Gerber beams etc.) a sufficient support length must be provided in order to avoid loss of support.

4.1.7.2 Contact with Adjacent Buildings

[1] Protection measures must be taken for both the building under design and the existing building, against adverse consequences from impact during the seismic response.

[2] Consequences may be particularly adverse when there is a possibility for slabs or other elements of one building to ram the columns of the other. In this case, the protective measure must consist of the provision of a full separation seismic joint.

[3] In absence of an accurate analysis, a seismic joint must be provided, which may have a width equal to the square root of the sum the squares of the maximum seismic displacements ($\Delta = q\Delta_{\varepsilon\lambda}$) of the two buildings at the location of the critical columns, taking into account also the effect of rotation about a vertical axis. If a more accurate assessment of the displacements of the existing building is not possible, then these may be taken equal to the corresponding displacements of the building under design.

[4] For buildings which are in contact with each other but there is no possibility for any columns to be rammed, the width of the respective joint, in the absence of a more accurate analysis, may be determined on the basis of the total number of storeys in contact above the ground as follows:

- 4 cm up to and including 3 storeys in contact
- 8 cm from 4 to 8 storeys in contact
- 10 cm for more than 8 storeys in contact

For underground floors a seismic joint is not obligatory.

4.2 DAMAGE LIMITATION

4.2.1 Load-bearing system

[1] The values of the behaviour factor in chapter 2 are considered to ensure limited and repairable damages on the elements of the load-bearing system under the design earthquake, while they minimize the damages under earthquakes of lower intensity and greater probability of occurrence.

4.2.2 Infill

[1] In buildings with masonry infill, it should be checked that the angular distortion on all perimeter walls, taking also into account the relative rotation of the successive slabs about a vertical axis, does not exceed the value 0.005. When the infill is less sensitive to shear deformation (panels of steel frame, glass partitions, etc), the angular distortion should not exceed the value 0.007.

[2] The check shall be performed with the displacement values derived by the elastic seismic analysis according to chapter 3, multiplied by the ratio q/2.50 which should not be taken less than 1.00. These values correspond to an earthquake of lower intensity and greater frequency of occurrence than the design earthquake.

4.2.3 Appendages

[1] The appendages, as well as their supporting elements and anchorages, shall be checked against failure under the effect of the vertical loads and a horizontal seismic force:

$$H_p = \varepsilon W_p \gamma_p / q_p \qquad (4.17)$$

where:

W_p is the appendage weight

ε is the seismic factor defined in cl. 3.6.[2]

γ_p is the importance factor of the appendage

q_p is a reduction factor which expresses the appendage's capability to sustain considerable post-elastic deformations without failure.

[2] Generally, the importance factor γ_p shall be taken equal to the importance factor of the building but in the following cases of high hazard appendages it shall not be taken less

than 1.50 :

- Anchorages of installations and equipment for lifeline systems
- Tanks and containers which contain considerable quantity of high toxic or explosive substances that constitute a hazard to the public safety.

[3] The following maximum values of the factor q_p shall be used for the corresponding categories of appendages:

$q_p = 1.0$

- Cantilevered parapets and decorative elements
- Signs and plates
- Chimneys, masts and elevated tanks acting as free cantilevers in more than 1/2 of their total height
- The high hazard appendages mentioned in the previous paragraph.

$q_p = 2.5$

- External and internal walls. Fence walls of height more than 2.0 m.
- Chimneys, masts and elevated tanks which have props or anchorages by guys such as to act as free cantilevers in a height not exceeding 1/2 of their total height
- Tanks and their contents
- Anchorages of permanent racks or lofts supported on the ground
- Anchorages of false ceilings and lighting fixtures of considerable weight
- Electromechanical equipment and relevant ductwork, piping and air ducts weighing more than 2 kN.

[4] The following appendages are excepted from the obligation to be checked: appendages in buildings of importance Σ1 and Σ2 in seismicity zone I, and appendages of category q_p=2.5 in buildings of importance Σ2 in seismicity zone II.

CHAPTER 5

FOUNDATIONS, RETAINING STRUCTURES, EARTH STRUCTURES

5.1 ADEQUACY OF THE FOUNDATION SUBSOIL

5.1.1 General requirements

[1] The subsoil, the topography and the general geology of the area of civil engineering structures should ensure, with sufficient probability, that there will be no risk of soil rupture, slope instability or extended liquefaction during an earthquake vibration compatible with the intensity and spectral characteristics of the design earthquake provided in this Code.

5.1.2 Proximity to Active Seismotectonic Faults

[1] In general, it is not allowed to build structures of importance Σ2, Σ3 and Σ4 in close proximity to seismotectonic faults which are characterised as active.

[2] The characterization of faults as seismically active shall be based on historical and seismotectonic data and the probable size of any seismic rupture shall be taken into account. Recognition and description of seismotectonic faults constitutes, in general, a subject of special study that concerns a greater construction area and not specific buildings. Such a study is necessary for housing development in an area and is subject to check and approval by the state. Studies for recognition of seismically active faults are generally not required in already developed housing areas, unless there are strong indications to the contrary, based on official geological-tectonic maps.

[3] In cases where there are specific reasons for construction in the immediate vicinity of seismotectonic faults which are considered seismically active, construction is allowed only following a special seismic – geological – geotechnical - structural study. This study shall investigate the effects of the proximity of the fault and will take measures to effectively mitigate them. The design seismic action in the immediate vicinity of such

faults shall be taken increased by at least 25% relative to that determined in chapter 2.

5.1.3 Slope Stability

[1] It is obligatory to check the general stability against sliding of the slope on which a structure will be founded, as well as slopes located uphill or downhill relative to the structure when their failure may affect the structure. The stability analysis may be performed according to the provisions of cl. 5.4. The check shall be based on a suitable geotechnical survey, and on a geological survey if deemed necessary by the former.

5.1.4 Liquefaction Hazard

[1] The hazard of extensive liquefaction of saturated loose soil materials must be checked using standard methods of Soil Dynamics, also taking into account a potential amplification of the soil movement due to local soil conditions. In any case, it must be noted that the soil accelerations determined in Chapter 2 are “effective” values (not maximum) and therefore they should not undergo any further reduction.

[2] In case where the above mentioned check indicates that soil resistance to liquefaction may be inadequate, it is obligatory to apply measures to ensure the integrity of structures or soil structures to be born on such soil.

[3] In similar soils, even if they are considered to have sufficient resistance against liquefaction, the need for reduction of the design value of the effective angle of friction due to excessive pore pressure build-up during the cyclic design seismic action must be investigated (see annex F.5).

5.1.5 Shear settlement of the soil due to cyclic loading

[1] Loose non-saturated sandy formations may be subject to a dynamic volume reduction (settlement), resulting in permanent settlement and deformations. Similar effects may occur in very soft and sensitive clays due to the gradual reduction of their shear resistance during cyclic loading of long duration. The possibility of occurrence of such

phenomena must be investigated by means of well established geotechnical methods by studies compiled on the basis of the results of *in situ* or laboratory tests. Soils of this type are characterised as seismically sensitive and their existence must be pointed out in the geotechnical design.

5.2 FOUNDATIONS

5.2.1 Criteria and Application Rules

[1] Under the design earthquake, the foundation system must reliably ensure the transfer of the actions from every founded elements of the superstructure to the ground without causing large permanent deformations.

[2] The design of the system should minimize the uncertainties of the seismic response. Therefore, energy dissipation shall not be provided for through extensive plastic deformations of the foundation soil but shall be limited to development of plastic hinges in selected positions of the superstructure. Relevant application rules are given in the following paragraphs.

5.2.2 Design Actions

[1] The design actions S_{Fd} at a foundation element shall generally be computed based on the overstrength of the ductile element of the superstructure supported on the foundation element, as follows:

$$S_{Fd} = S_v + \alpha_{CD} S_E \quad (5.1)$$

where:

S_v is the section force from all the non-seismic actions of the seismic combination, and

S_E is the same section force from the seismic action corresponding to the seismic moment (M_E) used to determine the capacity factor α_{CD}, in accordance with relation (5.2).

[2] For foundations of single columns or walls, the capacity magnification factor α_{CD} shall be calculated, independently for each one of the two horizontal earthquake components, by the formula:

$$\alpha_{CD} = 1.20\, M_R / M_E - M_V / M_E \le q \quad (5.2)$$

where:

M_R and M_E are respectively the design resistance and the seismic bending moment at the closest location of a potential or possible plastic hinge in the superstructure element which is founded on foundation element under examination (see 4.1.4.[3] and [4]) and

M_v the total moment due to non-seismic loadings of the combination

[3] In foundations of braced frames of steel structures, where the ductile element is the tension diagonal, the value of α_{CD} shall be calculated according to C.5.3.[1].

[4] Whenever the foundation element carries more than one superstructure element (strip footings, raft foundations, etc), it is allowed to apply relation (5.1) with a uniform value of α_{CD} either equal to 1.35 or calculated on the basis of that superstructure element which has the maximum ductile seismic action effect.

[5] When the spatial superposition defined in cl. 3.4.4.[2] and 3.5.3.[4] is applied, for the design actions of foundation elements, it is permitted to use the value $\alpha_{cd} = 1.0$, for the components multiplied by the factor $\lambda = \mu = 0.3$.

[6] Whenever the foundation element under consideration also bears elements independent of the superstructure (e.g. independent retaining wall), the design actions of relation (5.1) shall be increased by the design seismic actions of these independent elements, taken with a seismic action direction and sense identical to that of the superstructure.

5.2.3 Soil Resistance

5.2.3.1 Basic Requirement

[1} The design seismic action of cl. 5.2.2 must be transferred to the ground without exceeding the limit states of bearing capacity of the soil-foundation system. In addition to the limit states referred to in cl. 5.2.3.2 or 5.2.3.3, the following shall also be considered:

- General stability of the overall structure (i.e. of the structure and the affected part of the ground)
  This must be checked in cases of foundation in soils with steep inclination or near slopes (natural or artificial). Checking must be performed in accordance with the provisions of cl. 5.4.

- Large permanent deformations

Application rules for avoidance of large deformations are given in cl. 5.2.3.2 and 5.2.3.3, depending on the type of foundation.

[2] For the verification of soil resistance in accordance with cl. 5.2.3.2 or 5.2.3.3 and Annex F, suitably estimated design values of the ground parameters c_d and φ_d shall be used. Those values shall not, as a rule, exceed the design values under an equivalent static loading.

5.2.3.2 Shallow foundations

[1] The ultimate limit states defined in sub clauses a, b and c, below, shall be investigated:

a. Bearing resistance failure (ultimate load)

[1] This criterion requires satisfaction of the following relation:

$$N_{Fd} \leq R_{Nd} \qquad (5.3)$$

where:

N_{Fd} is the design axial force (normal to the bearing area) as determined by relation (5.1),and

R_{Nd} is the design bearing capacity (ultimate load) of the foundation under a load normal to the bearing area, the determination of which must take into account the coexisting moments and the components of the load parallel to the seating area, as determined by actions S_{Fd} of relation (5.1).

[2] The bearing capacity R_{Nd} may be calculated pseudostatically with ground parameters taking into account the cyclic character of the ground deformations. In saturated soils, because of the speed of transmission of the seismic action, loading shall, in general, be considered under undrained conditions.

[3] In annex F, an indicative analytic method for the bearing capacity for rectangular footings is given. The soil design parameters for the application of this method shall not exceed those corresponding to static actions.

[4] When the load eccentricity in one direction exceeds 1/3 of the corresponding dimension of the foundation, satisfaction of criterion (5.3) becomes extremely sensitive to variations of the actions as well as of the dimensions of the foundation and of the ground parameters, since the effective area, according to annex F, is reduced below 1/3 of the footing's area (indeed, if there is a corresponding eccentricity in the other direction, it reaches 1/9 of the surface of the footing). Therefore, eccentricities exceeding 1/3 of the corresponding dimension of the foundation are only allowed when all of the following conditions are satisfied:

- The uncertainties of all actions have been minimised, including satisfaction of the provisions of cl. 5.2.4.1

- Severe limits for tolerances concerning the dimensions and the location of the foundation have been ensured.

- The design of the structure provides for ductile post elastic response (use of q > 1.0) and the factor of capacity magnification α_{CD} of relation (5.2) corresponding to the particular foundation element is smaller than q.

- The foundation soil is not seismically sensitive, according to cl. 5.1.5. In the case of seismically sensitive soils, the eccentricities may not exceed ¼ of the corresponding dimension of the foundations, so that large permanent deformations are avoided.

b. Failure by sliding

[1] This criterion requires satisfaction of the following relation:

$$V_{Sd} \leq R_{Sd} + R_{Pd} \qquad (5.4)$$

where:

V_{Sd} is the shear force parallel to the bearing area derived from the seismic action of relation (5.1), increased by potentially existing active pressures acting on vertical forces of the foundation, and by potentially existing seismic actions of independent elements, as described in cl. 5.2.2.[6].

R_{Sd} is the sliding resistance of the soil-foundation interface as determined below, and

R_{Pd} are the mobilised resisting (passive) earth pressures acting at vertical faces of the foundation. For reasons of limitation of the permanent deformations, this resistance shall not exceed 40% of the minimum full passive pressure under seismic conditions. For R_{Pd} to be taken into account it must be ensured during construction that the vertical faces of the foundation are in direct contact either with undisturbed soil or with an adequately dense backfill and that it is not possible to subsequently remove the resisting soil.

[2] The sliding resistance R_{Sd} may be calculated as follows:

1) In granular soils:

$$R_{Sd} = N'_{Fd} \cdot \tan(\delta_d) \tag{5.5}$$

where:

N'_{Fd} is the effective axial force normal to the bearing area and corresponding to the design action of relation (5.1) and

δ_d is the design value of the friction angle in the soil-foundation interface, taken equal to:

- The angle of design shear resistance φ_d , in cases of concrete foundation cast directly on the ground,
- $(2/3)\varphi_d$, in cases of precast concrete foundation with a smooth bearing surface, and
- the angle of friction of membrane, in cases where an impermeable membrane is inserted between foundation and ground.

2) In cohesive soils:

$$R_{Sd} = A' \cdot s_u \leq 0.4 \cdot N_{Fd} \tag{5.6}$$

where

A' is the effective bearing area, in accordance with Annex G for rectangular footings or defined in a similar way for footings of a different shape

s_u is the design value of the undrained shear strength of the soil layers under the foundation

N_{Fd} is the normal force on the soil – foundation interface

c. Failure of structural elements of the foundation

[1] The structural elements of the foundation shall be verified at ultimate limit state under the design action effects of relation (5.1) and the corresponding soil reactions. The latter may be calculated from equilibrium conditions, either by use of an elastic subgrade reaction coefficient (Winkler type) which is consistent with the form and size of the element under consideration, or by assumption of a linear distribution of the soil reactions.

5.2.3.3 Deep foundations (Piles, Diaphragms, Shafts)

[1] This clause refers mainly to piles. In cases of diaphragms or shafts, the same general principles may be applied, on condition that the differences due to the particular properties of such systems shall be taken into account through satisfactory approximation.

a. Analysis

[1] In the absence of a more rigorous analysis, an equivalent elastic model may be used, continuous or discrete, in which the following elements are represented with sufficient accuracy:

- The lateral stiffness of the soil
- The stiffness of the pile (in bending and in the longitudinal direction)
- The stiffness of the pile-cap beams and of the superstructure

[2] The lateral resistance of shallow layers sensitive to liquefaction or strength loss (see cl. 5.1.5) must be suitably reduced or eventually be neglected altogether.

[3] It is not advisable to transfer horizontal seismic forces to the ground by means of axial forces of inclined piles. If inclined piles are used, verification of their flexural stress is also required.

[4] The longitudinal and lateral stiffness of the piles shall be taken from the secant stiffness in the “elastic” region of their load-displacement curve, i.e. before initiation of slip relative to the ground. Values corresponding to static loading may be used.

[5] Under seismic action, the internal forces induced in piles or other deep foundation elements are generally due to the following causes:

- the supporting action, that is the transfer of forces from the superstructure to the ground and vice versa, and
- the "kinematic" action, due to the deformation of the surrounding ground caused by the transmission of the seismic waves.

[6] The piles and pile-caps are always checked for the supporting action. The kinematic stress shall be taken into consideration, even by means of simplified methodology, when all of the following conditions are present:

- Soil class Γ containing layers with distinctly different properties, as mentioned in clause b3.[3] below.
- Seismicity zone III or IV
- Structure of importance Σ3 or Σ4.

b. Ultimate limit states

[1] It shall be verified that the ultimate limit states determined in sub clauses b1, b2 and b3, below, are not exceeded.

b1 Axial force failure (compressive or tensile)

[1] This criterion requires satisfaction of the following condition:

$$N_{Pd} \leq R_{Nd} \tag{5.7}$$

where:

N_{Pd} Is the axial force of the most severely loaded pile as derived from the analysis under the effect of the action of relation (5.1), and

R_{Nd} is the bearing capacity (ultimate load) of the pile as determined under static conditions, in accordance with valid codes, established methods of analysis and/or test loadings. If there are reasons to reduce the soil strength due to seismic loading, R_{Nd} shall be reduced accordingly.

[2] In case where the full development of the axial load produces significant permanent settlement of the pile, the ultimate load must be reduced to values which correspond to acceptable permanent deformations. If there are no special reasons concerning the sensitivity of the superstructure's structural system, a permanent deformation of up to 40 mm may be considered acceptable.

b2 Transverse Strength Failure

[1] This criterion requires satisfaction of the following relation:

$$V_{T,d} \leq R_{Td} \tag{5.8}$$

where :

$V_{T,d}$ is the maximum shear force of the pile as derived form the analysis under the effect of the action of relation (5.1), and

R_{Td} is the soil resistance to transverse loading combined with head moment corresponding to $V_{T,d}$, in accordance with valid codes, established methods of analysis and/or test loadings. With the exception of the seismically sensitive soils of cl. 5.1.5, it is generally permitted to consider the transverse soil resistance under static conditions.

b3 Failure of structural elements of the foundation

[1] The structural elements of the foundation shall be verified at ultimate limit state in accordance with the results of the analysis as described in cl. 5.2.3.3.a.

[2] In pile foundations it must, in general, be ensured (by verification using the capacity

actions of relation (5.1) that the piles remain within the elastic range. When this is not possible, confinement of the concrete core at the potential and possible regions of plastic hinges must be effected, as well as capacity shear check of the piles by applying actions similar to (5.1) and (5.2).

[3] Potential region for a plastic hinge is considered a region of length 2d below the pile cap. If the pile transverses the interface of successive soil layers which have much different shear moduli (ratio of shear moduli > 5), then regions of $\pm 2d$ about the possible limits of this interface shall be deemed to be regions of possible plastic hinges. In these regions, confinement and bending strength equal to that of the pile top shall be provided. This rule does not apply for the region of the foundation layer for end-bearing piles provided that conditions of full fixity of the piles are not developed there.

[4] In case where the analysis takes into account the kinematic moments (see 5.2.3.3.a(6)), and if the bending moment derived at the location of soil discontinuity is less than 30% of the pile-top moment, the corresponding region does not need to be considered as a possible plastic hinge.

5.2.4 Minimization of uncertainties

5.2.4.1 General

[1] The foundation system must be homogeneous and must ensure, as far as possible, a uniform distribution of the seismic actions to the ground. The arrangement of shallow foundations of vertical elements of the same building at different horizontal levels with significant differences in elevation shall be avoided. Where this is not possible, construction measures shall be taken to ensure common horizontal movement of the foundations located at different levels. Such measures are not necessary in foundations on stiff rocky soil.

5.2.4.2 Connecting beams

[1] Single footings and pile caps shall be connected to each other with connecting beams

in two horizontal directions.

[2] The connecting beams may be neglected in the structural analysis. In any case, they shall be verified at least for the action of an axial force

$$F_d = \zeta \alpha N_m \quad (5.9)$$

where:

α is the normalized seismic acceleration of the ground (=A/g)

N_m is the average of the vertical loads of the connected elements, and

ζ= 0.40 for soil class A

0.50 for soil class B

0.60 for soil class Γ or Δ

[3] Provision of connecting beams is not obligatory in the following cases:

- For soil class A and seismicity zones I and II, provided that all foundations are made on the same horizontal level.
- Between footings of columns in sheds with span larger than 12.00 m in the direction of the span.

[4] In case of eccentric footings, connecting beams must always be provided along the direction of the eccentricity. In the verification of these beams, account shall be taken of the bending moments due to eccentricity of the vertical loads, which loads shall include the most onerous contribution of the seismic action according to relation (5.1).

5.2.4.3 Foundations of structural walls of the superstructure

[1] In buildings without basements, it is in certain cases difficult, to fulfil the requirements of cl. 5.2.3.1 and 5.2.3.2 using isolated foundation for each structural wall of the superstructure, because of the high bending moment at the base of the wall. In such cases it is advisable to provide foundations common with adjacent vertical elements using strip foundations or connecting beams with sufficient stiffness.

[2] In buildings with basements which have perimeter walls, the maximum moments (and

the probable plastic hinges) of the walls occur, in general, at the floor of the ground level. The corresponding seismic shear forces are transferred through shear action of the plate diaphragms to the perimeter walls and from there on to the ground. The perimeter walls of the basements must be constructed and reinforced so as to ensure the aforementioned transfer of forces. Due care should be taken for the integrity of the shear connection between the ground floor slab and the perimeter walls at areas with openings.

5.3 RETAINING STRUCTURES

[1] The retaining structures shall be designed so as to fulfil their purpose during and after the design earthquake without occurrence of substantial damages either to themselves or to the retained structures. For transferring the forces to the ground, the relevant provisions of cls. 5.2.3.2 or 5.2.3.3 must be adhered to, depending on the type of foundation. The permanent displacements must be compatible with the functional and aesthetic requirements of the structure.

[2] The application rules referred to below are, in general, sufficiently conservative for the usual cases of retaining walls. For special cases of high walls (height greater than 10m) founded on soft soil layers of great depth (>30m), the possibility of amplification of the effective seismic ground acceleration must be investigated.

[3] In he absence of a more accurate assessment, the earth pressure due to the design earthquake can be evaluated by the following methods:

a. Walls having the possibility to slide or deform

[1] This category includes walls which either are free to slide/rotate at their base or are deformable with an anticipated displacement at the top equal to at least 0.10% of their height. For walls in this category, the increased pressure during an earthquake may be calculated by the Mononobe-Okabe method, that is by assuming a plane failure surface corresponding to a horizontal force equal to α_h.W and an additional vertical force equal to -α_v.W on the critical wedge with a weight of W. Alternatively, methods based on the

general deformation theory (with elastic or elastoplastic behaviour of the soil), in accordance with paragraph [7] below, may be used.

[2] The horizontal "seismic coefficient" is taken from the relation:

$$\alpha_h = \alpha/q_w \tag{5.10}$$

where:
α is the normalized seismic acceleration of the ground and

q_w is the behaviour factor which takes the following values:

Type of Wall	Factor q_w
Wall free to slide 300α (in mm)	2.00
Wall free to slide 200α (in mm)	1.50
Anchored wall or flexible wall founded on piles or rock	1.20
Rigid wall founded on piles or rock	1.00
Retaining wall supported by struts	0.70

[3] The vertical seismic coefficient α_v is taken equal to 0.30a. This value incorporates the influence of the coefficients of spatial superposition λ = μ = 0.30 of cl. 3.4.4.[4] and 3.5.3.[4].

[4] The seismic coefficients α_h and α_v shall also be applied both to the weight of the wall and to the weight of the backfill supported directly by its footing (L-shape walls).

[5] The friction angle between wall and soil on the soil-wall interface shall not be taken larger that $(2/3)\varphi_d$, where φ_d is the angle of shear resistance of the soil.

[6] Annex D provides formulae to estimate the increased soil pressure during an earthquake, according to the Mononobe-Okabe method of limit equilibrium.

[7] Instead of the above limit equilibrium method, methods based on the general deformation theory (elastic or elastoplastic) may be used, with analytical or numerical ? modelling of the soil. Analysis with these methods must be compatible with the actual kinematic restrictions of the wall and must correspond in a satisfactory way to the design parameters of the soil and the retained material.

b. Rigid Walls

[1] This category includes walls which are practically undeformable and are a rigidly supported. Such walls are, for example, the perimeter walls of building basements connected with the floor slabs, or walls of caissons, underground tanks, etc.

[2] The static at-rest pressures acting on such walls are increased during an earthquake by a linear diagram of additional horizontal pressures with maximum value at the ground surface equal to $1.5\alpha\gamma H$ and a minimum value equal to $0.5\alpha\gamma H$ at the bottom level of the wall at depth H (γ = unit weight of the soil; the depth H need not be taken larger than 10.0 m). It is generally sufficient to check, with these increased pressures, only the integrity of the elements directly affected, i.e. of the walls and counterforts (if such exist).

c. Saturated soils - Hydrodynamic pressure

[1] In most soils, in the region which is below the ground water level, independent movement of the water from the soil is not possible during an earthquake. In this case, the seismic action may be taken on the sum of masses of soil and water. Hence, for walls of subclause (a) the pressure rise due to earthquake may be calculated form the difference K_{AE} - K_A of the pressure coefficients K_{AE} and K_A as derived from the Mononobe-Okabe Method (see Annex B), with seismic action $\{\alpha_h, \alpha_v\}$ and without seismic action, respectively. For the part of the backfill that is under the water level, this difference is applied upon the total mass of soil and water, i.e. the unit weight γ is taken as the weight of the saturated soil γ_s.

[2] In very permeable soils (permeability $k > 0.50\ 10^{-3}$ m/sec) the seismic actions on the masses of soil and water shall be calculated independently and superposition of the results shall be performed. In this case, the earth pressures calculated as above using

the buoyant unit weight of the soil (but without increase of the seismic coefficients) shall be increased by the hydrodynamic variation of the water pressure.

$$p(z) = \pm(7/8)\, \alpha \gamma_w \sqrt{Hz} \quad (5.11)$$

where:

- H is the depth of the wall below the free surface
- z is the depth of the point under examination
- γ_w is the unit weight of the water.

[3] When the not backfilled well face is also covered by water, the hydrodynamic variation of the water pressure p(z) acting on this face shall be assumed to act in the same sense as that of the backfilled face (negative pressure).

d. Anchorages

[1] Anchorages must ensure stability of the critical sliding wedge under seismic conditions. In the absence of a more accurate assessment, the distance between the wall and the anchoring bulb centre shall be taken equal to the distance required under static conditions multiplied by the factor 1+1.50 α.

[2] In soils with liquefaction potential, a safety factor at least 2.00 against liquefaction of the soil surrounding the anchor must be ensured.

5.4 SLOPES - EMBANKMENTS

5.4.1 Slopes

[1] Stability of natural or artificial slopes during a seismic action shall be verified using the following additional effective accelerations acting on the soil mass.

Horizontal: $\alpha_h = \alpha_\pi$ (5.12)

Vertical: $\alpha_V = \pm\, 0.50\, \alpha_\pi$ (5.13) where α_π is the design seismic acceleration of the slope, taken equal to 0.5a for natural slopes or equal to $(\alpha_B + \alpha_K)/2$ for the slopes of embankments according to 5.4.2.

[2] In soil type Γ, seismicity zones III or IV and when the structure under design is of importance Σ3 or Σ4 or when the general stability of the area is affected, the estimation of the shear strength parameters must be based on suitable in situ and/or laboratory tests under cyclic loading. For clay soils the residual strength (after considerable deformation) shall be used.

5.4.2 Embankments

[1] Stability of embankments up to 15.0m height shall be checked using additional horizontal active accelerations on their mass which vary from $\alpha_B = 0.5\alpha$ at the base up to $\alpha_K = \alpha_B\,\beta(T)$ at the top of the embankment,
where,
α is the normalised seismic ground acceleration, and
β(T) is the spectral magnification that corresponds to the fundamental period of the structure.
In the absence of a more accurate analysis, $T = 2.5\,(H/V_s)$ may be used,
where,
Vs is the average velocity of shear wave in the embankment.

[2] The design of embankments of height larger than 15m, embankments bearing important structures and dams, is not covered by the present Code. These cases require a special geotechnical and seismic design. In the absence of a detailed and complete seismological study, the seismic action at the level of the natural soil may be taken in accordance with Chapter 2, using a suitable value for the importance factor γ_I and values q = 1.0, η = 1.0 and θ = 1.0.

5.4.3 Stability Check

[1] Stability shall be checked using the most unfavourable sliding surface and ensuring a safety factor γ at least equal to 1.0.

ANNEX A

GROUND SEISMIC MOTIONS

A.1 ELASTIC ACCELERATION SPECTRUM

[1] The horizontal components of the ground seismic motion are determined by an elastic acceleration spectrum $R_e(T)$:

$0 < T < T_1$ $\quad R_e(T) = A.\gamma_I.\,[1 + (n.\beta_o - 1).\,(T/T_1)]$

$T_1 < T < T_2$ $\quad R_e(T) = A.\gamma_I.\,n.\beta_o$

$T < T_2$ $\quad R_e(T) = A.\gamma_I.\,n.\beta_o.\,(T_2/T)$

where:

$R_e(T)$	spectral acceleration
T	period in sec
T_1, T_2	characteristic periods of the spectrum in sec per Table 2.4
A	ground seismic acceleration per Table 2.2
γ_I	importance factor per Table 2.3
$\beta_o = 2.50$	is the spectral amplification factor
n	correction factor for percentage of critical damping different from 5%

[2] The elastic spectrum of the vertical component of te earthquake is derived from the above, by multiplying its ordinates by 0.70.

[3] In case of uncertainty with regard to the soil, the most unfavourable spectrum shall be used.

A.2 ACCELEROGRAMS

[1] Use of real and/or synthetic accelerograms is allowed; hence, in this Code, these will

be called “design accelerograms”, provided that they fulfil the provision of cl. A.2.1.

A.2.1 Real Accelerograms

[1] Use of real design accelerograms is allowed provided that:

a) At least five (5) different accelerograms are used. For horizontal motions, horizontal components are selected. The design accelerograms for horizontal motion are allowed to be used for the vertical motion as well, under the conditions of cl. A.1.[2]. If different design accelerograms are used for the vertical motion, then vertical components must be selected.

b) They are selected in such a way as to represent, as far as possible, the seismotectonic, geological, soil-dynamic, and in general, the local conditions of the area of the structure.

c) They are digitised at intervals of 0.02 sec maximum.

d) They have a duration that is consistent with the seismotectonic, geological, soil-dynamic, and in general, the local conditions of the area of the structure.

e) The average spectrum, i.e. the average of the spectra of the design accelerograms, is equivalent to the spectrum of cl. A.1 for 5% damping. The two spectra are considered equivalent if the ordinates of the average spectrum satisfy the following conditions:

- They are higher or equal to the respective ordinates of the spectrum of cl. A.1 for periods up to 0.2 sec.
- For periods over 0.2. sec, 10% of the values may be up to 5% lower.

f) The ordinates of the spectrum of the design accelerograms and of the average spectrum are computed as a minimum for periods resulting from:

- 18 equal steps between 0.01 and 1 sec
- 10 equal steps between 1 and 2 sec
- 8 equal steps between 2 and 4 sec

[2] The design accelerograms for horizontal motions may be used for both components.

A.2.2 Synthetic Accelerograms

[1] Use of synthetic design accelerograms is allowed provided that their spectrum encloses the spectrum of cl. A1.

ANNEX B

SPECIAL APPLICATION RULES FOR REINFORCED CONCRETE STRUCTURAL ELEMENTS

B.1 AVOIDANCE OF BRITTLE FAILURE MODES - SHEAR FAILURE

[1] Unless a more accurate calculation is performed, application of the general capacity rule of cl. 4.1.4.[4] shall be performed with the following particular rules:

B.1.1 Columns

[1] Design shear force in the direction of each frame, to which the column belongs:

$$V_{CD,c} = 1.40\,(M_{R,c1} + M_{R,c2}) / \ell_c \leq qV_{E,c} \qquad \text{(B.1)}$$

where:

$M_{R,c1}$, $M_{R,c2}$ are the design resistances in bending with axial force, at the ends of the column, as activated by the seismic action. The maximum of the values resulting from two opposite directions of the seismic action shall be used (see also 4.1.4.[4]),

$V_{E,c}$ is the seismic shear of the column, and

ℓ_c is the length of the column.

B.1.2 Beams

[1] Design Shear Force

$$V_{CD,b} = V_{0,b} + \Delta V_{CD,b} \qquad \text{(B.2.a)}$$

where

$$\Delta V_{CD,b} = 1.2\,(M_{R,b1} + M_{R,b2}) / \ell_b \leq q\,V_{E,b} / 1.20 \qquad \text{(B.2.b)}$$

and

$V_{0,b}$ is the shear force of the beam under the non-seismic loads of combination (4.1),

$M_{R,b1}$, $M_{R,b2}$ are the moment resistances at the ends of the beam in the sense in which they are activated by the seismic action,

$V_{E,b}$ is the seismic shear of the beam, and

ℓ_b is the beam length.

B.1.3 Columns and beams at whose ends the formation of plastic hinges is not possible

[1] In beams and columns whose large dimensions do not allow formation of plastic hinges at their ends, it is permitted to apply, instead of rules B.1.1 or B.1.2, the capacity rule of 4.1.4.[4] on the basis of the overstrengths of the potential regions of plastic hinges at the joints on either side.

[2] For this purpose, the capacity magnification factor α_{CD} shall be calculated at the joints on either side according to relation (4.6) or (4.7). In joints where the sum of the beams' resistances exceeds the sum of the columns' resistances ($\Sigma M_{R,b} > \Sigma M_{R,c}$), $\Sigma M_{R,c}$ shall be used in relation (4.6) instead of $\Sigma M_{R,b}$ (see cl. 4.1.4.[4]).

[3] The design shear force of an element e (column or beam) need not be taken larger than the value

$$V_{CD,e} = V_{0,e} + \Delta V_{CD,e} \qquad \text{(B.3.a)}$$

where:

$$\Delta V_{CD,e} = (\alpha_{CD,1} M_{E,e1} + \alpha_{CD,2} M_{E,e2}) / \ell_e$$

(B.3.b)

and

$V_{0,e}$ is the shear of the element under the non-seismic loads of combination (4.1),

$\alpha_{CD,1}$, $\alpha_{CD,2}$ are the capacity magnification factors at the joints of the element ends according to paragraph [2],

$M_{E,e1}$, M_E are the seismic moments at the ends of the element, and

l_e is the length of the element.

[4] All the rules mentioned above refer to isolated elements inside of which no formation of plastic hinges is possible. When whole areas of the load-bearing system are outside the plastic mechanism, cl. B.2.[4] is applicable.

B.1.4 Walls

[1] Walls are vertical elements that have, in general, an elongated cross section (length to width ratio l / b > 4) and possess a high stiffness compared to the horizontal elements (beams) with which they are connected in frame action. Under horizontal loading, walls act mainly as bending cantilevers with full or partial fixity in the base, where the main bending action is concentrated. For the post elastic seismic response, walls are designed with capacity principles so as to have a single critical area, at the location of maximum moment. Because of the elongated cross section of the walls, confinement of the critical area may be limited to the ends of their cross section.

[2] The design shear force of a plastic hinge region which can possibly be formed at the location of the maximum moment, that is at the wall base in general, shall be calculated using the flexural overstrength of the plastic hinge as follows:

$$V_{CD,w0} = \alpha_{CD} V_{E,w0} \quad \text{(B.4.a)}$$

with

$$\alpha_{CD} = \gamma_{Rd} M_{R,w0} / M_{E,w0} \leq q \quad \text{(B.4.b)}$$

where:

γ_{Rd} is the overstrength factor to be taken equal to 1.30 for steels commonly used today,

$M_{E,w0}$ and $V_{E,w0}$ are respectively the maximum moment and shear resulting from the seismic action at the plastic hinge section (base), and

$M_{R,w0}$ is the design resistance in bending with axial force, of the same section calculated according to 4.1.4.[4].

[3] For the other floors, the design shear shall be taken equal to the maximum shear

derived by the seismic analysis multiplied by the factor α_{CD} of relation (B.4.b) but not less than 1/3 of the design shear at the plastic hinge, that is

$$V_{CD,w} = \alpha_{CD} V_{E,w} \geq V_{CD,w0} / 3 \qquad \text{(B.5)}$$

[4] In order to limit the post-elastic response of the wall at the intended plastic hinge region, the design moments at any location shall be taken equal to the seismic moments multiplied by the factor α_{CD} of relation (B.4.b). These moments shall be taken neither less than 1/3 of the design resistance $M_{R,w0}$ of the plastic hinge section, nor larger than $M_{R,w0}$, that is

$$M_{CD,w} = \alpha_{CD} M_{E,w} \qquad \text{(B.6.a)}$$

and

$$M_{R,w0} /3 \leq M_{CD,w} \leq M_{R,w0} \qquad \text{(B.6.b)}$$

It is noted that for the envelope of tensile forces which will result from the above envelope of bending moments and from the axial forces of the seismic combination, the shifting rule, due to the coexistence of shear forces, shall be applied, as defined by the Code for Design and Construction of Concrete Structures. The longitudinal reinforcement of the wall shall be kept constant in the region of the plastic hinge, while at the adjacent region no larger reinforcement shall be provided.

[5] All above rules are valid for walls having constant cross-section all over the height of the building – such a configuration must be generally aimed at. In case the wall cross-section is reduced, the minimum values

$$V_{CD,w0}/3 \text{ and } M_{CD,w0}/3$$

mentioned in paragraph. [3] and [4] may be multiplied by the ratio $(J_w/J_{w0})^{1/3}$, where J_w and J_{w0} are respectively the inertia moments of the wall sections at the location under consideration and at the plastic hinge, respectively.

B.2 ENSURING SUFFICIENT LOCAL DUCTILITY AT PLASTIC HINGE REGIONS

[1] At plastic hinges of reinforced concrete elements, ductile behaviour of the compression zone must be ensured. Special measures are required when the required curvature at the plastic hinge cannot be achieved with concrete strain less than the ultimate value ε_{cu} = 0.35%.

When the compression zone is deep, the measures may be limited up to the depth where the strain has a value of $0.50\varepsilon_{cu}$. Such measures are defined in the "Code for Design and Construction of Concrete Structures" and are:

- for columns, confinement of the concrete by transverse reinforcement, and
- for beams, limitation of the tensile reinforcement ratio.

[2] In frame joints adjacent to plastic hinges, adequate anchorage inside the joint core must be ensured for the rebars which are intended to yield, as defined in the Code for Concrete Structures.

[3] At regions of potential plastic hinges, it is recommended that lap splicing of longitudinal bars is avoided. This is to be definitely avoided at the wall bases.

[4] At regions of the structure where formation of plastic hinges during the seismic response is not possible, it is not required to ensure an increased local ductility and the verification against brittle modes of failure is performed using capacity action-effects derived by the seismic action, multiplied by the factor α_{CD} corresponding to the nearest potential plastic hinge. Such regions are for example the columns and beams of basements within which the seismic actions are actually carried by the perimeter walls, where it is ensured that all elements remains within the elastic range. In such regions, the capacity verifications against shear failure, as well as the requirements for increased ductility, may in general be limited to the vertical elements and the roof beams of the first basement.

ANNEX C

SPECIAL APPLICATION RULES FOR STEEL STRUCTURAL ELEMENTS

C.1 COMPRESSION ELEMENTS

[1] In regions of potential and possible plastic hinges of steel sections, local buckling of the webs and flanges must be avoided by using an upper limit for the ratio of width to thickness (b/t). This limitation depends on the behaviour factor selected (q) and sections are divided into categories A, B and Γ as shown in Table C.1.

C.2 TENSION ELEMENTS

[1] In tension elements, the ratio of the cross-sectional net area at locations of bolt holes to the gross area must not be less than the value

$$A_{net} / A = 1.262\, f_y / f_u$$

where f_y is the yield stress and f_u is the ultimate tensile strength of the steel used. This may require reinforcement of the holes' regions with additional welded plates.

C.3 CONNECTIONS

[1] Connections in plastic hinge regions must have sufficient overstrength so that yielding is limited to the ductile members. In the relevant verifications, the probable upper bound of the yield stress of the probable ductile member (i.e. of the weaker member) shall be considered.

[2] Connections in plastic hinge regions which are achieved by full penetration welds are deemed to satisfy the above criterion of overstrength.

[3] Connections by fillet welds or bolted connections must satisfy the following relation:

$$R_d \geq 1.20 R_{fy} \qquad \text{(C.1)}$$

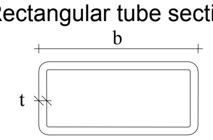
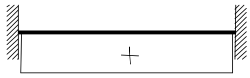
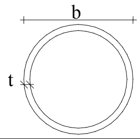
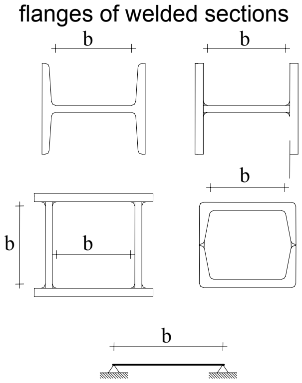
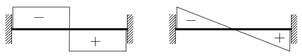
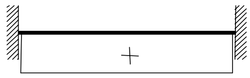
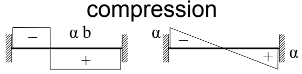
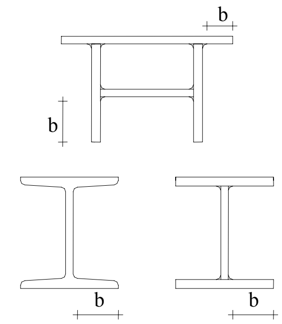
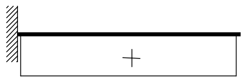
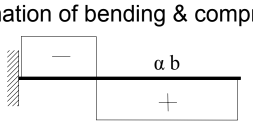
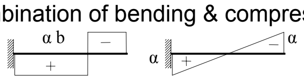
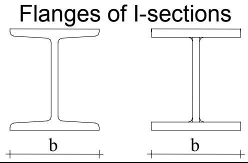
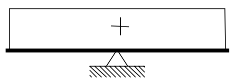
where:

R_d = the ultimate capacity of the connection

R_{fy} = the yield strength of the ductile member

[4] In bolted connections, the critical failure must be bearing failure of the projected area of bolt on the hole perimeter and not shear failure of the bolts.

TABLE 1: Limits of b/t ratios for compressive section areas for various section categories

Section	**Stress distribution** (positive compression)	**Section Category**		
		A	**B**	**Γ**
		$q \geq 4$	$4 \geq q \geq 2$	$2 > q$
Rectangular tube section 	Compression 	33ε	38ε	42ε
Circular tube section 	Compression Bending Compression + Bending	$50\,\varepsilon^2$	$70\,\varepsilon^2$	$90\,\varepsilon^2$
I-section Webs, webs & flanges of welded sections 	 Plastic distribution / Elastic distribution	66ε	78ε	90ε
	Compression 	33ε	39ε	41ε
	Combination of bending & compression  Plastic distribution / Elastic distribution	$\frac{33}{\alpha}\varepsilon$	$\frac{39}{\alpha}\varepsilon$	$\frac{41}{\alpha}\varepsilon$
Projecting flanges of box type sections or flanges of I-sections 	Compression 	9ε	10ε	12ε
	Combination of bending & compression 	$\frac{9}{\alpha}\varepsilon$	$\frac{10}{\alpha}\varepsilon$	$\frac{12}{\alpha}\varepsilon$
	Combination of bending & compression 	$\frac{9}{\alpha\sqrt{\alpha}}\varepsilon$	$\frac{10}{\alpha\sqrt{\alpha}}\varepsilon$	$\frac{12}{\alpha\sqrt{\alpha}}\varepsilon$
Flanges of I-sections 	Compression 	20ε	22ε	26ε

Note to Table 1:

In general $\varepsilon = \sqrt{235 / f_y}$

α in the denominator is a non dimensional number < 1 (or equal) which represents the ratio of the length of the compressed part (+) to the total length of the element.

f_y	235	275	355
ε	1.00	0.92	0.81

C.4 MOMENT RESISTING FRAMES

C.4.1 Avoidance of soft storey mechanism

The provisions of clauses 4.1.4.1 and 4.1.4.2 are applicable.

C.4.2 Beams

[1] Verifications against lateral buckling or torsional buckling of the beams shall be performed assuming that on one end a flexural plastic hinge has formed.

[2] In order to ensure the minimum required strength and sufficient ductility in rotation at the plastic hinge regions, the following conditions must be satisfied:

$$M_s / M_{pd} \leq 1.00 \quad (2.1)$$

$$N_s / N_{pd} \leq 0.15 \quad (2.2)$$

$$(V_0+V_M) / V_{Pd} \leq 0.50 \quad (2.3)$$

where:

M_s is the maximum moment resulting from the earthquake combinations

N_s is the corresponding axial force

N_{pd}, M_{pd}, V_{Pd}	are the ultimate design resistance in axial force, moment and shear of the section at the region of the plastic hinge
V_0	is the beam shear at the region of a plastic hinge when the beam is considered as simply supported
$V_M = (M_{RA}+M_{RB}) / l$	is the shear corresponding to the ultimate flexural resistance of the beam calculated with the upper bound of the yield stress
l	is the span of the beam.

[3] The connections of the beam with the columns must satisfy the requirements of cl. C.3 considering the ultimate resistance in bending M_{pd} of the plastic hinge section and a shear force equal to V_0+V_M as defined above.

C.4.3 Columns

[1] Columns are checked in bending with axial force according to cl. 4.1.4.1.

[2] The most adverse shear force of the column from the earthquake combinations must satisfy the relation:

$$V/V_{pd} \leq 0.50 \qquad (C.3.1)$$

[3] In beam-column joints, the shear force of a web bay enclosed on all four sides by flanges of the connected elements or by their extensions, shall satisfy the condition:

$$V/V_{pd} \leq 1.0 \qquad (C.3.2)$$

[4] Connections of column extensions shall be designed with strength higher than that of the connected members.

C.5 CONCENTRIC BRACED FRAMES

C.5.1 Action and ductile elements

[1] In braced frames without eccentricity the horizontal forces are mainly resisted by

elements subjected to axial forces. Ductile elements in such braced frames are primarily the tension diagonals.

[2] Braced frames capable of resisting seismic forces belong to the following 2 types:

- **Diagonal bracings**. In this type, horizontal forces having alternating sense are usually resisted by only those diagonals which are in tension while the contribution of the compression diagonals is ignored (they are not verified in compression). The diagonals of the opposite direction may be in the same bay (X type bracing) or in a different bay. In the latter case, the magnitude Acosφ (where A is the cross-sectional area and φ the angle of inclination of the diagonal with respect to the horizontal plane) must not differ by more than 10% between 2 opposite diagonals of the same floor.

- **Bracings of V or Λ type**. In this type the contribution of the compression diagonal to resist the horizontal forces is necessary. The diagonals may have a V- or Λ-shape and their common point is located within the span without interrupting its structural continuity.

[3] Bracings of K type, with the diagonals intersecting at an intermediate point of the column height, require the contribution of the column in the yield mechanism and cause extremely adverse 2nd order effects, with the consequence not to offer possibility for ductile behaviour (q=1.0). Their use is permitted only in seismicity zones I and for structures of Σ1 importance.

C.5.2 Diagonals

[1] The diagonals shall satisfy the condition:

$$N_s/N_{pd} \leq 1.00 \qquad (C.4)$$

where:

N_s is the maximum tension force resulting from the seismic combinations, and

N_{pd} is the design ultimate resistance in tension

The conditions of cls. C.2 and C.3 shall also be satisfied.

[2] The normalized slenderness $\bar{\lambda}$ of the diagonals must be limited according to the relation:

$$\bar{\lambda} = \sqrt{Af_y / N_{cr}} \leq 1.50 \qquad \text{(C.5)}$$

where:

A is the area of the section

f_y is the yield stress, and

$N_{cr} = \pi^2 EI / l^2$ is the ideal critical Euler load of the diagonal.

NOTE: The aforementioned relation $\bar{\lambda} \leq 1.5$ is equivalent to a slenderness $\lambda \leq 140$ for steel grade S235, $\lambda \leq 129$ for steel S275 and $\lambda \leq 114$ for steel S355. This rule must also be applied in the case of X type diagonal bracings where the seismic shear force is considered to be fully resisted by the tensile diagonals. The above relation (C.5) shall be applied even in cases where cl. 4.1.4.[5] is used, the latter not requiring satisfaction of the application rules of the present Annex C.

C.5.3 Columns and Beams

[1] The columns and beams of every floor shall be verified against in buckling under the effect of the seismic combination (4.1) and the seismic action-effects shall be multiplied by the capacity magnification factor :

$$\alpha_{CD} = (1.2\, N_{Pdi} - N_{vdi}) / N_{Edi} \leq q$$

where:

N_{Pdi} is the design resistance of the tensile diagonal of the floor

N_{vdi} is the tensile force of the same diagonal under the non seismic actions of the seismic combination ($N_{vdi} = 0$ as a rule), and

N_{Edi} is the tensile force of the diagonal under only the seismic action of combination (4.1).

[2] The horizontal beams of bracing elements of shapes V or Λ must be designed in such a way as to be able to resist the vertical loads without taking into account the

intermediate support by the diagonals.

C.6 Eccentric braced frames

C.6.1 Action and ductile elements

[1] The basic characteristic of these braced frames is that the connection of at least the one end of every diagonal to the horizontal beam is effected with an eccentricity to the respective node (node between beam and column or between beam and other diagonal). The part of the beam comprising the eccentric coupling is called "coupling beam" and is subjected to large shear and flexural stress induced by horizontal loads. Consequently, it is much easier for the ductility requirements to be concentrated in this part.

[2] The yield mechanism of the coupling beam depends on the ratio of its length lc to the length

$$l_o = 2\, M_{pc} / V_{pc}$$

where M_{pc} and V_{pc} are the capacities in bending and shear of the section of the coupling beam.

When $l_c / l_o \leq 0.80$, yielding due to shear is mainly developed (shear plastic hinge). When $l_c / l_o \geq 1.30$, yielding is mainly flexural (two flexural plastic hinges).

In the intermediate range, yielding is composite. In all cases it is possible to have large ductility.

[3] The coupling beams must be designed and configured so that they provide sufficient ductility. The other elements (columns, diagonals and the remaining length of the beam) must be checked by the capacity design so that yield is limited to the coupling beams.

C.6.2 Coupling beams

[1] The cross-sections of the coupling beams must be of category A according to Table C.1. Provision of strengthening plates or of holes is not permitted in the webs.

[2] The ends of the coupling beams must be reinforced by stiffness on both sides along

the height of the web. The thickness of these plates must be at least equal to $0.75t_w$ or 10 mm.

[3] When $I_c / I_o \leq 1.4$, it is required to arrange intermediate stiffeners also. These intermediate stiffeners must extend over the entire height of the web so as to secure the web and flanges against buckling; in beams with heights up to 600 mm, stiffness on one face only may be used. The maximum distance between successive stiffeners shall be taken equal to

$56t_w$-d/5 for $I_c / I_o \geq 1.15$

or

$38t_w$-d/5 for $I_c / I_o \leq 0.80$.

For values of I_c / I_o between the aforementioned limits, linear interpolation shall be performed.

[4] The resistances of coupling beams in axial load, bending moment and shear force are given by the following relations:

$$N_{pc} = 2b_f t_f f_y + h_w t_w f_y \qquad (C.6.1)$$

$$M_{pc} = b_f t_f (h_w + t_f) f_y + 0.25 t_w h_w f_y \qquad (C.6.2)$$

$$V_{pc} = h_w t_w f_y / \sqrt{3} \qquad (C.6.3)$$

where:

b_f and t_f	are respectively the width and the thickness of the flanges
h_w and t_w	are respectively the height and the thickness of the webs, and
f_y	is the yield stress.

[5] Coupling beams with flexural plastic hinge mechanism are designed as beams of frames (see cl. 4.2).

[6] Coupling beams with shear plastic hinge mechanism must satisfy the following conditions:

$$N_{Sc}/N_{pc} \leq 0.10 \qquad (C.7.1)$$

$M_{Sc}/M_{pc} \leq 0.70$ (C.7.2)

$V_{Sc}/V_{pc} \leq 1.00$ (C.7.3)

where: N_{Sc} , M_{Sc}, V_{Sc} are the axial force, moment and shear as derived by the seismic combinations for max V_{Sc}.

C.6.3 Columns and Diagonals

[1] These shall be verified against bending and buckling with the actions defined in cl. 5.3 using a capacity factor:

$\alpha_{CD} = 1.20 \min (V_{pdi} / V_{Sdi}, M_{pdi}/M_{Sdi})$ (C.8)

where:

V_{Sdi}, M_{Sdi} are respectively the shear and the moment derived by the seismic combination on the plastic hinge (coupling beam) of the same storey, and

V_{pdi}, M_{pdi} are the corresponding ultimate resistances of the coupling beam section.

C.7 Diaphragms - Horizontal bracing elements

[1] The diaphragms or the horizontal bracing elements must ensure transfer of the seismic forces to the vertical elements (vertical bracings or/and frames) with sufficient overstrength so that, both development of plastic hinges at the intended locations only is ensured, and moreover, sufficient capability for redistribution of forces is available; the latter is necessary, as the vertical elements do not enter simultaneously in the post-elastic range.

[2] As a rule, the above requirement is covered if the members of the horizontal bracings are checked with the actions derived by the seismic combinations multiplied by a magnification factor α=1.50.

ANNEX D

ACTIVE EARTH PRESSURES ON WALLS DURING EARTHQUAKES
MONONOBE-OKABE METHOD

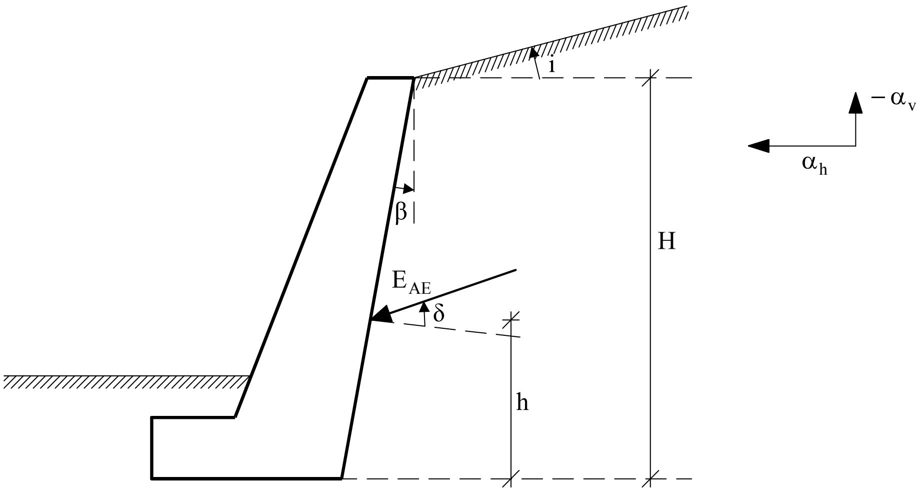


Fig. D.1: Retaining Wall

[1] The total active thrust E_{AE} due to gravity and seismic action is:

$E_{AE} = 0.5 \cdot \gamma \cdot H^2 \cdot (1-a_v) \cdot K_{AE}$

where:

$$K_{AE} = \frac{\cos^2(\varphi-\theta-\beta)}{\cos\theta \cdot \cos^2\beta \cdot \cos(\delta+\beta+\theta)\left[1+\sqrt{\dfrac{\sin(\varphi+\delta)\cdot\sin(\varphi-\theta-i)}{\cos(\delta+\beta+\theta)\cdot\cos(i-\beta)}}\right]^2},$$

$$\theta = \arctan\left(\frac{\alpha_h}{1-\alpha_v}\right)$$

γ	is the soil unit weight,
α_h and α_v	are respectively the horizontal and the vertical seismic coefficient,
H	is the height of the wall,
β	is the angle of the wall inner face to the vertical,
i	is the angle of the ground surface to the horizontal (β and i with the sign defined in Figure D.1),
φ	is the angle of shear resistance (internal friction) of the soil,
δ	is the angle of wall to fill friction.

[2] When the value of $\sin(\varphi - \theta - i)$, in relation (D.2), becomes negative, this value to be assumed equal to 0.

[3] In the absence of a more precise estimation, the application height h of the resultant of the thrusts may be taken from the relation:

$$h/H = 0.40 \tag{D.4}$$

[4] The respective relation for the passive earth thrust which is mobilised when the wall moves toward the soil is the following:

$$K_{PE} = \frac{\cos^2(\varphi - \theta + \beta)}{\cos\theta \cdot \cos^2\beta \cdot \cos(\delta - \beta + \theta)\left[1 - \sqrt{\dfrac{\sin(\varphi + \delta)\cdot \sin(\varphi - \theta + i)}{\cos(\delta - \beta + \theta)\cdot \cos(i - \beta)}}\right]^2} \tag{D.5}$$

[5] In L – shaped walls (with b ≥ H/3), in the absence of a more rigorous analysis, the active thrust shall be taken as acting on the vertical plane AB passing through the rear edge of the foundation, assuming $\delta = i$. In this case, in addition to the gravity forces, inertia forces will also be taken into account (with accelerations $a_h - a_v$) acting on the body of the wall as well as upon the soil prism which lies over the foundation on the backfill side.

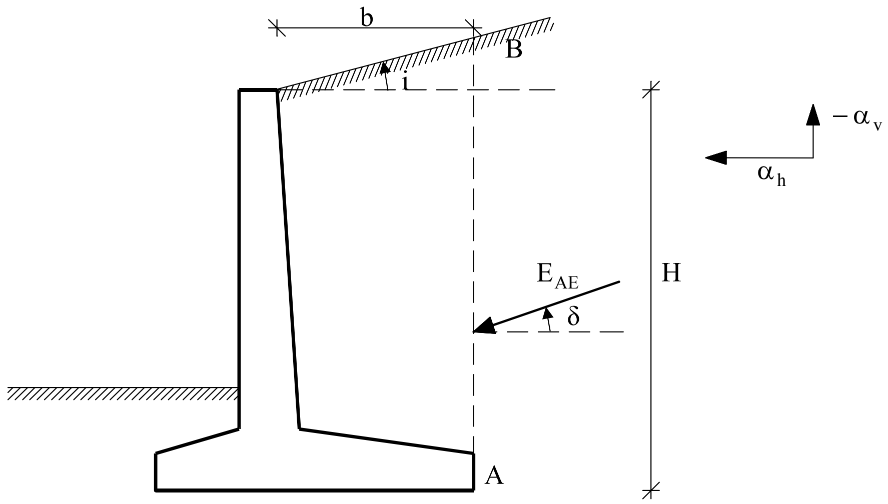


Fig. D.2: L-shaped Retaining Wall

ANNEX E

SPECIFIC RULES FOR EXTENSIONS TO EXISTING BUILDINGS

[1] In cases of extensions which are not structurally independent from the existing building (e.g. extensions in height), the seismic resistant design and, in general, the design of the extension itself and of the required strengthening, if any, of the existing building shall be performed in accordance with all provisions of the present Code (EAK) with the exceptions mentioned in the following paragraphs.

The verification of the existing building, when performed according to the provisions of the present Code, may be limited to satisfy the criteria for avoiding collapse and specifically to satisfy the requirements of cls. 4.1.2 and 4.1.3 of Chapter 4 concerning the Actions for the Analysis and the Resistance Verifications.

[2] All existing buildings are classified in the following categories based on how their load-bearing system has been constructed:

Category A: Buildings without an approved design for seismic resistance or when, during construction, the approved design for seismic resistance has not been applied.

Category B: Buildings with an approved design for seismic resistance performed according to the Royal Decree Code for Seismic Resistance (R.D.) of 19/26.2.1959 (FEK 36/A) "About the Code for Seismic Resistance of Buildings".

Category C: Buildings with an approved design for seismic resistance performed according to the R.D. Code for Seismic Resistance of 19/26.2.1959 (FEK 36/A) as amended by 1) Decision ΕΔ2α/01/44/ΦΝ275/4.4.84 (FEK 239/B) "Amendment and Supplement of the R.D. of 19/26.2.1959" and 2) Decision ΕΔ2γ/01/94/ΦΝ275/30.9.85 (FEK 587/B) "Substitution of article 12 of R.D. of 19/26.2.1959".

Category D: Buildings with an approved design for seismic resistance performed according to the Code of Decision Δ17α/08/32/ΦΝ275/30.9.92 (FEK 613/B) "New Greek Code for Seismic Resistant Structures".

[3] Exemptions from the requirements to verify a building according to the present EAK.

Depending on the importance of the building, the magnitude of the extension and the category of the existing building according to the aforementioned cl. E.2, the following exemptions are permitted.

a) In cases of extensions to buildings of importance Σ1 and Σ2 per table 2.3 of the present EAK and of category A, B, C of the above cl. E.2, the verification of the existing building may be performed in accordance with the R.D. Code of 19/26.2.1959 (FEK 36/A) as amended by 1) Decision ΕΔ2α/01/44/ΦΝ275/4.4.84 (FEK 239/B) "Amendment and Supplement of the R.D. of 19/26.2.1959" and 2) Decision ΕΔ2γ/01/94/ΦΝ275/30.9.85 (FEK 587/B) "Substitution of article 12 of the R.D. of 19/26.2.1959". If the amended article 12 has already been applied to the existing building, the verification of the existing building is performed with the present EAK.

b) In cases of extensions to existing buildings of category C and D per above cl. E.2 and regardless of their importance, exemption from the verification of seismic resistance is permitted when the total seismic load (base shear), considering the extension and any foreseen future storeys, does not exceed 1.10 of the respective seismic load of the existing building. Assessment of the seismic loads is performed according to the provisions of the present EAK and this exemption is valid only once in the life time of the building.

c) In cases of extensions to existing buildings of category C and D per above paragraph [2] and regardless of their importance, exemption from the verification of seismic resistance is permitted if such extensions are foreseen in their approved designs.

ANNEX F

EQUIVALENT STATIC ECCENTRICITIES

[1] The equivalent static eccentricities of seismic forces $F_{i,x}$ and $F_{i,y}$ in a normal height building are calculated using the relations:

$$e_f = \frac{\rho^2}{r} \cdot R_f \geq e_0$$

$$e_r = \frac{\rho^2}{r} \cdot \frac{1 - D_r}{I_r - \varepsilon_0} R_f \leq \frac{1}{2} e_0$$

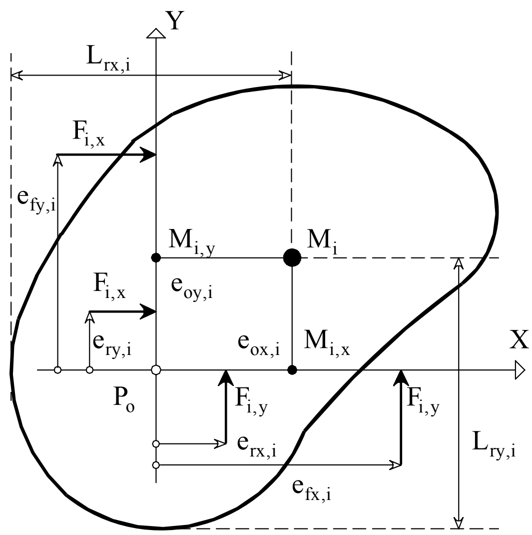


Fig. F.1: Equivalent static eccentricities

where:

$$R_f = \frac{\sin 2\theta}{2} \cdot \left(\frac{1}{A_1^{2n}} + \frac{1}{A_2^{2n}} - 2\varepsilon_{12} \cdot \frac{1}{A_1^n \cdot A_2^n} \right)^{1/2}$$

$$D_r = \frac{\sin 2\theta}{2} \cdot \left(\frac{\delta_{r1}^2}{A_1^{2n}} + \frac{\delta_{r2}^2}{A_2^{2n}} + 2\varepsilon_{12} \cdot \frac{\delta_{r1} \cdot \delta_{r2}}{A_1^n \cdot A_2^n} \right)^{1/2}$$

and:

$$\tan 2\theta = \frac{2\varepsilon_0}{\varepsilon_0^2 + \mu^2 - 1} \longrightarrow \text{angle } \theta, \ \ \varepsilon_0 = \frac{e_0}{r}, \ \mu = \frac{\rho}{r},$$

$A_1 = 1 - e_0 \cdot \tan\theta$, $A_2 = 1 + \varepsilon_0 \cdot \cot\theta$, r = inertia radius

$\delta_{r1} = \cot\theta - l_r$ $\delta_{r2} = \tan\theta + l_r$ $l_r = \dfrac{L_r}{r}$

$$r_{12} = \sqrt{A_2 / A_1} , \longrightarrow \varepsilon_{12} = \frac{8\zeta^2 \cdot (1 + r_{12}) \cdot r_{12}^{3/2}}{10^4 \cdot (1 - r_{12}^2)^2 + 4\zeta^2 \cdot r_{12} \cdot (1 + r_{12})^2}$$

For the determination of angle θ, first the acute angle α_0 is calculated (positive or negative) from the relation $\tan\alpha_0 = 2 \cdot \varepsilon_0 / (\varepsilon_0^2 + \mu^2 - 1)$ and then θ is taken as equal to $\alpha_0 / 2$ for $\alpha_0 > 0$ or $u = 90^0 - |\alpha_0 / 2|$ for $\alpha_0 < 0$. The eccentricity ε_0 is always taken with a positive sign and the positive values of e_f , e_r are measured from P_0 towards directions $\overrightarrow{P_0M_{i,x}}$ or $\overrightarrow{P_0M_{i,y}}$ of the projections of the centre of mass M_i on the main axes x or y.

More specifically, the relations are applied separately for each principal direction x or y of the building and for each diaphragm (i) (see Figure F.1) the following are introduced:

- The static eccentricity e_{oi} along the principal direction x or y under consideration (i.e. $e_{ox,i}$ or $e_{oy,I}$).
- The radius of gyration r_i of diaphragm (i).
- The torsional stiffness radius ρ of the building along the principal direction x or y (i.e. ρ_x or ρ_y .
- The ratios $\varepsilon_{oi} = e_{oi}/r_i$, $\mu_i = \rho/r_i$, $l_{ri} = L_{ri}/r_i$ along the principal direction x or y under consideration (i.e. $\varepsilon_{ox,i} = e_{ox,i}/r_i$, $\mu_{x,i} = rx/r_i$, $l_{rx,i} = L_{rx,i}/r_i$, etc.).

- The parameter n = 1 for $T \leq T_2$ and n = 2/3 for $T > T_2$, where T is the fundamental uncoupled natural frequency along the principal direction x or y under consideration (i.e. T_x or T_y).
- The percentage of critical damping ζ (in %).

The eccentricity e_r may also take negative values in systems sensitive to torsion. The limitations $e_f \geq e_o$ and $e_r \leq \frac{1}{2} e_o$, aim at reducing the post elastic displacements of the flexible side and the ductility requirements of the stiff side of the building.

ANNEX G

INDICATIVE ANALYTIC METHOD FOR THE CALCULATION OF THE ULTIMATE LOAD FOR A RECTANGULAR FOOTING

G.1 General

[1] For the analysis of the vertical ultimate load (bearing resistance) R_{Nd} of an horizontal and rectangular bearing area, the following approximate relations may be used. These relations have been derived through a combination of theory (of plasticity) and test results and are valid for homogeneous soil. The following parameters generally influence the bearing resistance and their influence shall be taken into account:

- the angle of friction φ' and cohesion c' or undrained shear strength S_u (design values),
- the eccentricity e = M /N and the shear force V. Where N, M and V are respectively the normal force, bending moment and shear force transferred to the soil though the bearing area (M and V are generally applied in both directions),
- the shape, depth and slope of the foundation,
- the pressure of underground water and, in case of flow, the hydraulic gradients, and
- the variability of strength from point to point and more specifically the layering of the soil.

[2] The following parameters are involved in the analysis :

δ	the angle of cohesion/friction at the base of the foundation (design value in accordance with cl. 5.2.3.2.b.[2]).
q	the total load pressure at the level of the base of the foundation,
q'	the effective load pressure at the level of the base of the foundation,
γ	the total unit weight of the soil,
γ'	the buoyant (effective) unit weight of the soil under the foundation

	level $\gamma' = \gamma - \gamma_w$. This is reduced to $\gamma' = \gamma - \gamma_w (1+j)$ in cases of water flow with an upwards hydraulic gradient equal to j,
$B' = B - 2e_B$	the effective width of he foundation, where e_B the eccentricity along the direction of width B,
$L' = L - 2e_L$	the effective length of the foundation, where e_L is the eccentricity along the direction f the length $L \geq B$,
$A' = B' L'$	the effective area of the footing, defined as the base of the foundation or, in case of eccentric loading, the reduced foundation surface with centre at the point where the resultant of the loads is applied, and
κ, ι	values of dimensionless shape coefficients of the foundation and of the slope of the load, respectively. Subscripts c, q, and γ indicate actions due to cohesion, load and weight of the soil. These coefficients are only valid when the shear parameters are independent of the direction.

G.2 LOADING OF CLAY-RICH SOILS UNDER UNDRAINED CONDITIONS

[1] The ultimate axial load R_{Nd} (bearing resistance), under the simultaneous presence of V and M is calculated from the relation:

$$R_{Nd} / A = (2 + \pi)\cdot S_u \cdot k_c \cdot \iota_c + q \qquad \text{(G.1)}$$

with the following values of dimensionless coefficients for:

- the shape of the foundation:

$$\kappa_c = 1 + 0.2\cdot(B'/L') \qquad \text{(G.2)}$$

- the slope of the load, caused by the shear force V:

$$\iota_c = 0.5\left(1 + \sqrt{1 - V / A' \cdot S_u}\right) \qquad \text{(G.3)}$$

- for simultaneous action of shear forces in both directions linear interpolation is applied, as determined at the end of the following clause, in terms of ι_c, taken from the relation (G.3), in each of the two directions.

G.3 LOADING NOT INDUCING DEVELOPMENT OF WATER PORE PRESSURES IN THE SOIL

[1] The ultimate axial load R_{Nd} (loading capacity), under the simultaneous presence of V and M is calculated from the relation:

$$R_{Nd} / A' = c' \cdot N_c \cdot \kappa_c \cdot \iota_c + q' \cdot N_q \cdot \kappa_q \cdot \iota_q + 0.5 \cdot \gamma' \cdot B' \cdot N\gamma \cdot \kappa_\gamma \cdot \iota_\gamma \qquad (G.4)$$

with the following values of dimensionless coefficients for:

- the **soil strength** for homogeneous soil:

$$N_q = e^{\pi \tan\varphi'} \tan^2(45+\varphi'/2) \qquad (G.5.a)$$

$$N_c = (N_q - 1) / \tan\varphi' \qquad (G.5.b)$$

$$N_\gamma = 2 \cdot (N_q - 1) \cdot \tan\varphi' \qquad (G.5.c)$$

on condition that $\delta \geq \varphi' / 2$ (rough seating).

- the **shape** of the foundation:

$$\kappa_q = 1 + (B' / L')\tan\varphi' \qquad (G.6.c)$$

$$\kappa_\gamma = 0.3\,(B' / L') \qquad (G.6.c)$$

$$\kappa_c = 1 + (B' / L')\,(N_q / N_c) \qquad (G.6.c)$$

- the **slope of the load**, resulting from the shear force V_L, parallel to L:

$$\iota_q = 1 - V_L / (N + A' \cdot c' \cdot \cot\varphi') \qquad (G.7.a)$$

$$\iota_\gamma = \iota_q \qquad (G.7.b)$$

$$\iota_c = (\iota_q N_q - 1) / (N_q - 1) \quad \text{(G.7.c)}$$

- the slope of the load, resulting from the shear force V_B, parallel to B:

$$\iota_q = [1 - V_B / (N + A' \cdot c' \cdot \cot\varphi')]^3 \quad \text{(G.8.a)}$$

$$\iota_\gamma = [1 - V_B / (N + A' \cdot c' \cdot \cot\varphi')]^3 \quad \text{(G.8.b)}$$

$$\iota_c = (\iota_q N_q - 1) / (N_q - 1) \quad \text{(G.8.c)}$$

- for simultaneous action of shear forces V_L parallel to L, and V_B parallel to B, the values of ι shall be calculated through linear interpolation between the values i_B and i_L , as determined from the relations (G.8) and (G.7), as follows:

$$\iota = \iota_B (1 - \theta / 90) + i_L (\theta / 90) \quad \text{(G.9)}$$

$$\tan\theta = V_B / V_L \quad \text{(G.10)}$$

G.4 RESTRICTIONS

[1] The above relations do not take into account the effect of the following factors:

- non-uniform soil layers down to the depth influenced b the foundation
- inclination of the surface of the soil or of the foundation itself (inclined base) or proximity of the end of the foundation to a slope
- development of inertia forces within the soil itself (due to seismic acceleration) at the moment of the assumed failure
- shear strength of the soil which lies above the base of the foundation (significant only for considerable depth of cover)

[2] The existence of such factors must be taken into account either indirectly though suitable parameter or coefficient values or by complementary or more rigorous analysis.

[3] The influence of water pore pressures is not taken into account in the relations of G.3. Therefore the method may be directly applied either to unsaturated soils in general or to saturated soils whose structure and/or drainage conditions allow the omission of the influence of water pore pressures. A simplified approach for estimating this effect on saturated granular soils is given in cl. G.5 below:

G.5 SIMPLIFIED APPROACH FOR ESTIMATING DEVELOPMENT OF PORE PRESSURES

[1] In saturated and relatively loose sand/silt soil formations undergoing large deformations during the design earthquake, significant pore pressures Δu may develop and build up during the successive cycles of seismic deformation.

[2] The maximum developing pore pressure Δu increases with the amplitude of the shear deformation and with the difficulty of dissipation of the pressure due to lack of permeability of the soil. The effect of the pore pressure may be taken into account in calculations by reducing the value of the angle of friction φ' to the "effective" value φ_E in accordance with the relation:

$$\tan\varphi_E = (1 - \Delta u / \sigma'_0)\tan\varphi' \qquad \text{(G.11)}$$

where the ratio $\Delta u / \sigma'_0$ = pore pressure / active vertical stress, may be considered as an average value along the length of the final area of failure resulting under design seismic loading with soil parameters c' and φ_E.

[3] In the absence of a more accurate analysis, the following indicative values of φ_E may be used :

- $\varphi_E = 0.60\,\varphi'$ in seismic risk zones I and II
- $\varphi_E = 0.40\,\varphi'$ in seismic risk zones III and IV.

G.6 ESTIMATION OF BEARING CAPACITY FROM PREVIOUS EXPERIENCE

[1] In buildings of importance Σ1 and in small buildings of importance Σ2 (with an above ground floor volume not exceeding 4000 m^3), it is allowed to estimate the bearing capacity of the soil on the basis of existing experience of nearby structures founded upon identical soil formations. Such structures should not show any indications of significant settlement and should have behaved appropriately during past seismic events of considerable magnitude.

[2} When the experience is based on a value σ_E of the allowed stress under the usual service loadings (without increase), the bearing capacity R_{Fd} of the foundation may be estimated as follows:

$$R_{Fd} / A' = 2\ i\ \sigma_E \qquad \text{(G.12)}$$

The reduction factors i, due to the existence of a total shear force V (resultant of the shear forces in the two directions), may be taken from the relation:

$$i = (1 - V/N)^{1.4} \qquad \text{(G.13)}$$

and the active area of the foundation A' shall be calculated from the eccentricities in accordance with the definitions of cl. G.1.