S U P P L E M E N T No. 3

TO

THE SOVEREIGN BASE AREAS GAZETTE

No. 1084 of 18th November, 1996

S U B S I D I A R Y   L E G I S L A T I O N

C O N T E N T S :

The following SUBSIDIARY LEGISLATION is published in this Supplement which forms part of this Gazette : –

No. 67

THE STREETS AND BUILDINGS REGULATION (CONSOLIDATION) ORDINANCE, 1984

(Ordinances 7 of 1984, 2 of 1987, 13 of 1987, 18 of 1988 and 10 of 1996).

REGULATIONS MADE BY THE ADMINISTRATOR UNDER SUBSECTION (1) OF SECTION 22.

In exercise of the powers vested in the Administrator under subsection (1) of Section 22 of the Streets and Buildings Regulation (Consolidation) Ordinance, 1984, the Administrator hereby makes the following Regulations:-

P.I.Nos.29 of 1984 and 33 of 1985.

1. These Regulations may be cited as the Streets and Buildings (Consolidation) (Amendment) Regulations, 1996 and shall be read as one with the Streets and Buildings (Consolidation) Regulations, 1984 as amended by the Streets and Buildings (Consolidation) Regulations, 1985 (hereinafter referred to as "the principal Regulations").

2. Regulation 2 of the principal Regulations is hereby amended by adding at the end thereof the following definition:

"Seismic Code" means the Seismic Code appearing in Appendix I to these Regulations.

3. The principal Regulations are hereby amended by adding the following new Part and new Regulation immediately after regulation 37:

"PART XIIA - SAFETY OF BUILDINGS

37A. In relation to any building requiring the use of reinforced concrete an anti-seismic study shall be submitted to the appropriate authority in accordance with the Seismic Code:

Provided that no anti-seismic study shall be required under this regulation in relation to the following:-

(a) Buildings with a cubic content of less than 200 cubic metres;

(b) Auxiliary buildings;

(c) Farm and livestock installations; and

(d) Additions and alterations, whether horizontal or vertical, to existing buildings, except public buildings".

4. The principal Regulations are hereby amended by inserting at the end thereof the following new Appendix to be numbered Appendix I and by re-numbering the existing Appendix as Appendix II:-

APPENDIX I

(Regulation 37A)

Seismic Code for Reinforced Concrete Structures

Table of Contents:

1. SCOPE AND FIELD OF APPLICATION

This code sets down minimum design requirements to be met when dealing with seismic situations, that is, situations in which the earthquake action is considered as a critical action in conjunction with other permanent or variable actions.

The aim of seismic design is:

- To save human lives
- To ensure the continuation of vital services
- To minimise property loss

The code applies to reinforced concrete buildings for ordinary uses, having structural resisting systems belonging to one of the three types as defined below:

<u>Frame System</u>: A system in which both vertical loads and lateral forces are resisted by space frames.

<u>Wall System</u>: A system in which both vertical loads and lateral forces are resisted by vertical structural walls, either simple or coupled. A coupled wall is composed by two or more simple walls, connected in a regular pattern by adequately reinforced ductile beams ("coupling beams")

<u>Dual System</u>: A system in which support for the vertical loads is essentially provided by a space frame. Resistance to lateral action is contributed in part by the frame system and in part by structural walls, isolated or coupled.

Other structural systems, not included in the above classification (eg. inverted pendulum structures, flat slab systems, etc.) can be designed subject to a documented proof that they satisfy all the requirements of this code.

Buildings having special characteristics or buildings involving high induced risk (eg. chemical or nuclear facilities) are outside the scope of this code.

2 REQUIREMENTS

2.1 <u>Structural Safety</u>

The entire structure and all its elements, including the main structural system designed to resist the total seismic action, as well as any secondary connected system not possessing a seismic resistance of its own, must retain their integrity and a residual capacity after the seismic action has ceased.

2.2 <u>Serviceability</u>

It is required that the building as a whole, including structural and non structural elements, be protected with adequate reliability against the occurrence of damages and limitations of use as a consequence of the seismic action.

3 DESIGN CRITERIA

3.1 Definitions

Design criteria comprise the set of operations to be performed in order to satisfy the general requirements set forth in ch. 2. These operations include:

- Consideration of the relevant limit-states of structural behaviour, and checking of these limit-states by means of analytical procedures based on appropriate models and values of actions and resistances.

- Detailing of structural elements according to the provisions contained in this code.

- Adopting quality assurance procedures both in the design and construction processes.

3.1.1 Stability

Stability verifications include rigid body equilibrium (sliding and overturning) and foundation stability. Foundation systems have to be dimensioned so that the soil strains will remain essentially elastic, that is without appreciable residual deformations.

3.1.2 Collapse Mechanism

The provisions of this code have been developed on the assumption that structures should resist earthquake actions by means of a stable, non linear energy dissipating response mechanism. This aim will be achieved by following the dimensioning rules of the various elements in ch. 4.

3.1.3 Strength and Ductility

Critical regions (i.e. where most of the energy dissipation is expected) must be provided by an appropriate balance of these properties, both of them contributing to safety and serviceability. Specific analytical provisions, which take into account the influence of cumulative damage and degrading of mechanical properties are given in chs 4 & 5.

3.1.4 Limiting of Deformations

The amplitude of the structure's deformation under the design forces shall be limited in accordance with cl. 4.5.4.

3.1.5 Global Ductility

Use of appropriate materials (cl. 4.1) as well as of experimentally validated detailing arrangements (ch. 5) contribute to ductile behaviour at least as significantly as the available analytical procedures.

3.1.6 Quality Assurance

Attention should be paid to the correspondence between the structural model adopted for the analysis and the actual structure, considering all the elements, either structural or not, which could alter the predicted behaviour

Checks of good workmanship in the detailing should be carried out, especially in those areas indicated as critical by the designer (extremities of columns and beams, bases of walls, lintels, etc.)

3.2 Reliability Differentiation

Target reliabilities shall be established on the basis of the consequences of failure, in which monetary and non monetary losses are included, depending principally on their use and the importance of their functions.

Structures are divided into five different reliability levels. According to the required protection, the structures will be classified as follows:

Class I Buildings whose destruction could have disastrous consequences - like nuclear power stations, toxic flammable chemicals storage buildings, dams - or buildings over 15 floors or very important buildings.

Class II Buildings or places of mass meetings - cinemas, theatres, big reception halls etc. - or buildings of great importance to the community - schools, hospitals, airports, fire stations etc. - or industrial buildings with exceptionally expensive equipment.

Class III Houses, flats, restaurants, hotels, industrial establishments and other permanent buildings that do not belong to classes I and II.

Class IV Auxiliary buildings and agricultural establishments.

Class V Temporary constructions whose possible destruction does not endanger human lives.

The different reliability levels apropriate for each class shall be obtained by multiplying the design action with a factor I, called "importance factor", given in Table 3.2.

Class	I	Comments
I	-	not covered by the code
II	1.5	
III	1.0	
IV	0.5	
V	-	no need for seismic analysis

Table 3.2

In addition or as an alternative to the use of the factor I, checking of specific limit-states relevant to damage or loss of function can be required for certain types of buildings.

3.3 Ductility Levels

Structural systems covered by this code can be designed for different "ductility levels", according to the following classification.

Ductility level I (DL I) - This level includes structures designed to the ordinary concrete code with the few additional requirements on detailing contained in ch. 5.

Ductility level II (DL II) - For this level specific seismic provisions are to be adopted, enabling the structure to enter the inelastic range of response under repeated reversed loading while avoiding premature brittle-type failures.

Ductility level III (DL III) - For this level special procedures for the evaluation of the design actions and for the proportioning and detailing of the elements are to be adopted to ensure the development of selected stable mechanisms associated with large energy - dissipation capacities.

The greater the ductility level conferred to a structure, the lower is the seismic action to be considered for the design, as quantified by the value of the "behaviour factor" K. (cl. 4.1.3)

4 METHODS OF ASSESSMENT

4.1 Basic Data

4.1.1 Material Characteristics

4.1.1.1 Concrete

Normal aggregate concrete shall satisfy the following minimum requirements:

Ductility Level	DL I	DL II	DL III
Minimum Grades	C16/20	C20/25	C25/30

Table 4.1.1

4.1.1.2 Steel

DL I and DL II Structures

Steel properties shall be in accordance with the requirements contained in the Code for Reinforced Concrete

DL III Structures

The following additional requirements shall be satisfied:

a) It must be proven that the steel used possesses adequate ductility under repeated reversed deformations.

b) Steel grades higher than S400 shall not be used unless it is demonstrated that the use of higher grades in special sectional arrangements does not affect unfavourably the ductility.

c) The actual yield stress shall not exceed its nominal value by more than 15%.

d) The ratio of the mean value of the ultimate strength to the actual yield stress shall not be less than 1.25 for S220 and 1.15 for S400.

e) Only high bond steel shall be used for flexural reinforcement, unless adequate provisions are taken to ensure bond and anchorage.

4.1.2 Material Safety Factors

Design values of strength for concrete and steel shall be obtained from their respective values by using the factors:

Concrete: $\gamma_c = 1.5$ Steel: $\gamma_s = 1.15$

4.1.3 Structure Behaviour Factors

The values of the behaviour factor K, defining the intensity of the design action (cl. 6.4.4) as a function of the structural type and of the selected ductility level, are given in Table 4.1.3.

	DUCTILITY LEVEL		
STRUCTURAL SYSTEM	DL I	DL II	DL III
Frame	2	3.5	5
Wall & Dual	2	3	4

Table 4.1.3
Structure Behaviour Factors

The values of K in Table 4.1.3 for wall & dual structures apply if at least 50% of the lateral force in both directions is resisted by coupled walls. If this condition is not satisfied, the K values shall be reduced by a factor of 0.7.

Ductility level I is permitted only for Class III, IV, and V structures.

Class II structures to be built in high seismicity areas shall be preferably designed for DL III. If appropriate, for greater safety, K values for DL II could also be used in this case.

4.1.4 Design Load Combination

The fundamental combination of load effects to be used for all the limit-states verifications is:

$$S_d = S(G+E+\psi Q) \qquad (4.1.4)$$

where:

G: includes all the permanent loads at their nominal value.

E: is the design seismic action as defined in cl. 6.4.4.

Q: are all the variable loads at their nominal values, whose duration of application is long enough for the probability of their joint occurrence with earthquake action being not negligible.

Ψ: factor determining the percentage of variable loads to be considered in seismic analysis. Values for ψ are given below.

Type of Construction	ψ
Roofs and Ceilings	0
Houses, Apartments, Offices, Hotels etc.	0.25
Public halls, Hospitals, Schools, Mass meeting areas etc.	0.50
Storage rooms, Refrigerators, Factories	0.75
Tanks, Water towers etc.	1.00

Table 4.1.4

Fraction of Variable Loads to be Considered in Seismic Analysis

The estimate of the seismic action will be based on all loads of gravity appearing in equation 4.1.4.

4.2 Structural Analysis

4.2.1 Building Configuration

Allowable methods of structural analysis shall be different for buildings which according to the definition given in this chapter are classified as "regular" or "irregular".

Regular buildings can be designed according to the simplified methods of analysis (indicated as equivalent static analysis), described in cl. 4.2.4 provided that their height does not exceed 50m and the fundamental period is shorter than 2 seconds.

If these conditions are not satisfied or if the building is of the irregular type, the dynamic method in cl. 4.2.5 shall be applied.

In irregular buildings the application of the equivalent static method in the following cases is allowed as an exception:

a) In buildings of Class III and IV up to three storeys.

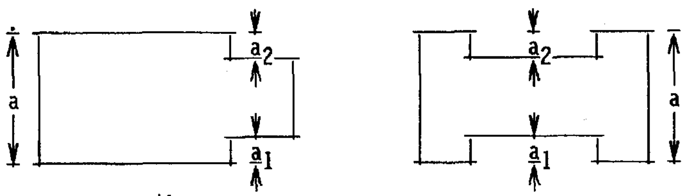
b) In Class III buildings, including those of more than three storeys, provided factor K from Table 4.1.3 is decreased by 25%

In irregular buildings of Class II analysis by the dynamic method should be applied.

A building shall be classified as regular when the following conditions, in both plan and vertical configuration are satisfied.

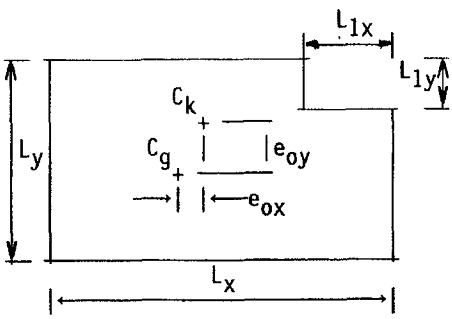
4.2.1.1 Plan Configuration

The building has an approximately symmetrical plan configuration with respect to at least two orthogonal directions along which the earthquake resisting elements are oriented. When re-entrant corners are present, they do not exceed 25% of the building's external dimensions.



$a_1 + a_2 < a/4$

At any storey the distance (measured in the direction orthogonal to that of the seismic action) between the centre of mass and that of stiffness does not exceed 15% of the "resilience radius", defined as the square root of the ratio of the storey torsional and translational stiffness.



$$\frac{L_{1x}}{L_x} \leq 0.25 \text{ or } \frac{L_{1y}}{L_y} \leq 0.25$$

$$\frac{e_{oy}}{r_x} \leq 0.15 \qquad r_x^2 = \frac{K_t}{K_x}$$

$$\frac{e_{ox}}{r_y} \leq 0.15 \qquad r_y^2 = \frac{K_t}{K_y}$$

$$K_x = \Sigma K_{xi} \qquad K_y = \Sigma K_{yi} \qquad K_t = \Sigma (K_{xi} y_i^2 + K_{yi} x_i^2 + T_i)$$

where:

K_{xi}: stiffness of element in direction x
K_{yi}: stiffness of element in direction y
x_i : distance of element in direction x from the elastic centre
y_i : distance of element in direction y from the elastic centre
T_i : torsional stiffness of element around its axis

4.2.1.2 Vertical Configuration

The stiffness and mass distributions are approximately uniform along the building height. Change of stiffness of 50% in two consecutive floors is considered an irregular arrangement.

In frame structures, the ratio between actual shear capacity (sum of the shear forces contributed by all vertical elements at their design strengths) and the design shear does not differ more than 20% for any two storeys of the building.

In the case of gradual setback along its height, the setback at any floor is not greater than 15% of the plan dimension in the direction of the setback. This clause need not be complied with if the setback occurs within the lower 20% of the total height or within the first 6m from the ground or in the last storey.

4.2.1.3 Importance of Regularity

Regularity (generally meaning a strong, symmetrical plan shape and uniform mass and stiffness distribution) is by itself a very desirable property, because it naturally leads to an effective, economical and more predictable seismic behaviour.

Also, because there are many uncertainties, the actual behaviour of irregular buildings is difficult to predict even with the use of complicated mathematical models. Thus, non-symmetrical arrangements lead to major torsional effects resulting in concentrations of ductility demand in some elements that can therefore change into critical points for the global stability of the building.

Equally dangerous ductility demand concentrations can occur due to discontinuities in stiffness and/or strength of the structural system along the height, as produced by a softer intermediate storey or by a sudden variation of the building plan dimensions.

4.2.2 Application of Seismic Action

Horizontal Action

The seismic actions shall be applied to the building in the directions producing in each element the most unfavourable effect.

In buildings having one axis of structural symmetry the seismic action can be assumed as acting separately along this axis and its orthogonal direction.

Vertical Action

The vertical component of the seismic action shall be considered in the design of non-vertical cantilevers and in cases of large concentrated loads.

4.2.3 Analytical Model

The determination of the seismic effects on the structure shall be based on an idealised mathematical model which is adequate for representing the actual behaviour; the model shall also account for all the non-structural elements that can influence the response of the main resisting system.

The interaction between structural and non-structural elements is generally considered undesirable and should, wherever possible, be reduced or avoided.

For the purpose of the present code, the determination of the load effects due to design forces may be based on a linear elastic model of the structural system.

4.2.4 Equivalent Static Analysis

The equivalent static analysis can be adopted for buildings classified as regular according to 4.2.1 provided their height does not exceed 50m and the fundamental period is not greater than 2 seconds. These limits are given because in buildings with greater fundamental period (generally higher buildings) the effect of higher modes of vibration increases.

4.2.4.1 Horizontal Design Forces

The design lateral force to be applied at each floor level, in the direction being analysed, shall be given by:

$$F_i = C_d \cdot \gamma_i \cdot W_i \qquad (4.2.4.1.1)$$

where:

C_d=Design seismic coefficient, equal in value to the design response spectrum as given in cl. 6.4.4.

γ_i=Distribution factor depending on the height of floor i, measured from the building base.

W_i=Total gravity load at floor i.

The fundamental period of the building, which is required for the evaluation of C_d, shall be calculated using the elastic properties of the structure by ordinary methods of mechanics, taking into account all the elements which contribute to the building stiffness.

For frame structures an approximate expression of the fundamental period, based on analytical and experimental results is:

$$T = N/12$$

where N is the number of storeys.

In various cases, a sufficiently accurate estimate of the period can be obtained with reference to an "equivalent" uniform cantilever, whose period is given by the expression:

$$T = 1.8h\left(\frac{mh^2}{EI}\right)^{\frac{1}{2}}$$

where:

-m is the building mass per unit height
-h is the height of the the building from the foundation level
-EI is the flexural stiffness of the "equivalent" cantilever

In case the period is not calculated, C_d shall be taken as:

$$C_d = I.\ Amax.\alpha.\frac{1}{K}$$

where:

-I = Importance factor cl. 3.2.
-Amax= Maximum ground acceleration cl. 6.2.
-α = Spectrum factor cl. 6.4.1.
-K = Factor of behaviour cl. 4.1.3.

The distribution factor γ_i is given by the following expression:

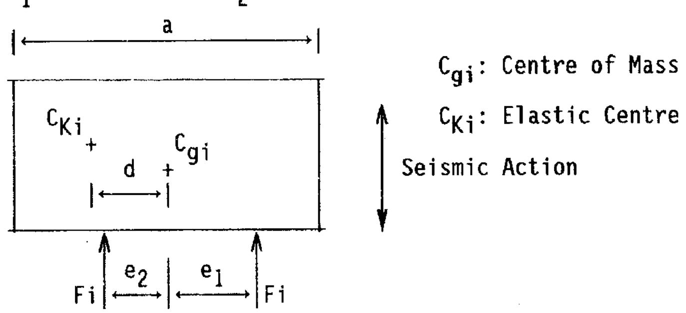
$$\gamma_i = h_i \frac{\Sigma W_i}{\Sigma W_i h_i}$$

where h_i is the height of floor i from the foundation level.

4.2.4.2 Torsional Effects

At each floor of the building the lateral design force shall be assumed to be displaced from its nominal location at the distances e_1 and e_2 as illustrated in the figure below, whichever is most unfavourable for every member to be checked. The expressions for e_1 and e_2 are:

e_1=0.5d+0.05a e_2=0.05a



The total shear force and torsional moment at the generic floor shall be distributed to the various resisting elements below that floor with due consideration of their relative stiffness as well as of the stiffness of the diaphragm.

Symmetrical Cases

Where complete symmetry of stiffness and mass about one axis parallel to the direction of seismic action exists, torsional effects can be accounted for by means of the following simplified procedure:

- The lateral design force shall be applied at the floor centre of gravity to be distributed to various elements as above.

- The actions in each of the elements shall be further multiplied by a factor ξ defined as:

$$\xi = 1+0.6\,\frac{x}{a}$$

  where x is the distance of the element from the floor centre of gravity, measured perpendicularly to the direction of the seismic action

4.2.4.3 Second-Order Effects

Second order effects on storey shears and moments need not to be considered when the following condition is satisfied at every floor.

$$\theta = \frac{W.\Delta el.K}{V.h} \leq 0.10$$

where:

θ= Deformability Index ie. the ratio of the second order moment to the moment due to the shear force on the storey.
V= Seismic design shear force acting across the storey considered.
Δel=Elastic interstorey drift due to the design actions.
K= Behaviour factor. (cl.4.1.3)
h= Floor height.
W= Total gravity load above the considered storey.

It is to be recalled that under the action of a crit:~al earthquake the structures are expected to undergo large inelastic dynamic displacements and a state of collapse can be reached if the deformations are excessively increased due to the effects of the 2nd order terms.

Since the prevailing contribution to these terms is given by the inelastic part of the deformation, the best remedy against collapse due to instability is to strengthen the structure so as to reduce the amount of the inelastic demand for displacement.

The deformability index θ shall not in any case exceed 0.15.

For $0.10 \leq \theta \leq 0.15$ second-order effects shall be accounted for by means of one of the statical methods indicated in ch. 14 of the code for concrete. In these limits it has been considered that an elastic static approach, although conceptually inappropriate, could all the same serve the purpose of providing extra strength.

4.2.5 Modal Analysis Procedure

4.2.5.1 Modelling

If the building can vibrate in two orthogonal directions without significant coupling, it can be analysed by means of two separate planar models, one for each orthogonal direction. This condition is assumed to occur when ch. 4.2.1.1, paragraph 2, is satisfied. When the above requirement is not complied with the model shall account for the non-planar motion of the structure.

For the purpose of determining the global inertia forces acting at each floor the building can be modelled as a system of masses lumped at floor levels, each mass having two translational and one rotational degrees-of-freedom in the case of spatial model.

The number of lumped masses should be consistent with the desired number of vibration modes to be used. In general, the number of degrees of freedom should be at least double the number of vibration modes that can be determined with accuracy.

Once the global forces are obtained, they shall be distributed to the various vertical resisting elements (frames, walls, etc.) with due consideration of the relative stiffness of the vertical components and the diaphragm.

4.2.5.2 Modes

In the case of planar models, the analysis shall include for each of the two orthogonal axes at least the lowest three modes of vibration or all modes of vibration with periods greater than 0.4 seconds, whichever is greater

For non-planar models the analysis shall include for each direction of application of the seismic action at least four modes, two of them predominantly translational and two of them predominantly rotational, or all modes of vibration with periods greater than 0.4 seconds whichever is greater.

The modes considered shall be those with the greatest participation coefficients for the direction under consideration.

4.2.5.3 Combination of Modal Responses

The response quantities (force, displacements etc.) separately obtained for each mode shall be combined to obtain their corresponding design values by taking the square root of the sum of the squares of modal values.

4.2.5.4 Torsional Effects

At each floor of the building the mass contributing to inertia forces shall be assumed to be displaced from its nominal location by the amount $\pm 0.05a$, whichever is more unfavourable for the element to be checked, a being the dimension of the building in the direction orthogonal to that of the considered seismic action.

When the building is analysed by means of planar models (cl. 4.2.5.1) torsional effects can be accounted for by increasing the action effects due to the translational oscillations of the building by the factor defined as:

$$\xi = 1 + 0.6 \frac{x}{a}$$

where x is the distance of the planar element considered from the floor centre of gravity, measured perpendicularly to the direction of the seismic action.

4.2.5.5 Second-Order Effects

Clause 4.2.4.3 applies.

4.3 Design Actions

Structural elements shall be dimensioned and verified (see cl. 4.4) for the design actions as defined in the present chapter.

Design actions derive from the actions obtained from the structural analysis in cl. 4.2 above, appropriately modified for the selected design ductility level.

DL I structures shall be dimensioned directly on the basis of the results of structural analysis, with a possible redistribution of action effects as permitted in the code for concrete.

The difference between DL II and DL III design procedures lies essentially in the use, for the latter, of partial factors γ_n. These factors are meant to ensure, as implied in the definition of DL III (cl. 3.3), the development of stable mechanisms associated with large energy-dissipation capacities.

Factors γ_n are used to amplify action effects, namely bending moments, when these latter become, through the laws of static equilibrium, actions by which other actions are to be calculated.

These derived action effects include:

- Shear forces in beams
- Shear forces in columns
- Shear forces in walls
- Column bending moment at sections adjacent to beam-column joints
- Shear forces and bond stresses in joint cores.

4.3.1 Ductility levels II and III

4.3.1.1 Elements Subject to Bending ($N_d \leq 0.1 A_g . f_{cd}$)

Bending Moments

The design bending moments shall be those obtained from the linear analysis of the structure for the load combination given by equation 4.1.4. Redistribution according to the code for concrete cl. 8.3 is permitted.

Columns not carrying significant vertical loads behave similarly to beams. Thus, for axial loads below the commonly accepted limit of $N_d = 0.1A_g.f_{cd}$, the provisions for beams apply to columns also.

N_d is the element axial force in the most unfavourable load combination including the seismic action.

Shear Forces

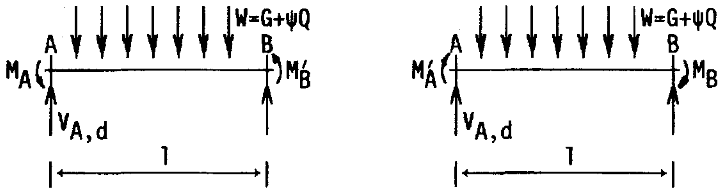
The design shear forces shall be determined from the condition of static equilibrium of the element subjected to the relevant transverse load, if any, and to a rational combination of the end moments. The latter shall correspond to the design flexural strength of the end sections based on the actual reinforcement provided. For DL III structures these values will be multiplied by a factor γ_n=1.25.

At each end section, two values of shear force shall be calculated, ie. the maximum and minimum value, corresponding to positive and negative moment yielding at hinges.

The algebraic ratio between the maximum and minimum values of shear force at a section shall be denoted by ζ.

For the purposes to follow, the value of ζ should not be taken smaller than minus one.

With the notations and signs shown in the following figure, the maximum and minimum shear forces at A will be:



$$V_{A,d} = V_A + \frac{M_A + M'_B}{l} \qquad \zeta \cdot V_{A,d} = V_A - \frac{M'_A + M_B}{l}$$

with the end moments at their design ultimate values.

For DL III structures these values will be multiplied by a factor γ_n = 1.25.

4.3.1.2 Elements subject to Bending and Axial Force

Axial forces and bending moments

The axial forces and bending moments to be used for column design shall be determined by considering all the possible unfavourable combinations as obtained from a linear analysis of the structure for the load combination given by equation 4.1.4 eventually redistributed according to the code for concrete, cl. 8.3.

The bending moments thus obtained shall be further modified as required by the following clause.

For regular structures, three storeys or higher, to which the equivalent static analysis has been applied, the column moment due to the lateral forces alone shall be multiplied by the dynamic magnification factors ω as given by the following expressions.

Planar Frames: $\omega = 0.6T + 0.85$ $\quad (1.3 \leq \omega \leq 1.8)$
Spatial Frames: $\omega = 0.5T + 1.10$ $\quad (1.5 \leq \omega \leq 1.9)$

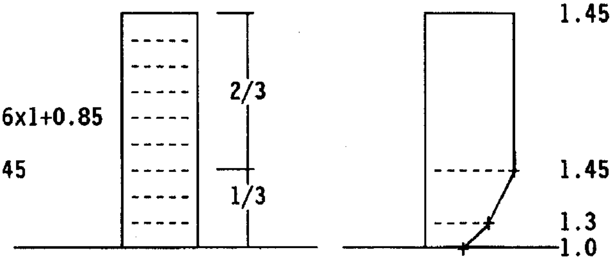
where T is the fundamental period of structure.

The values of the dynamic factor ω as given by formulae above are applicable to storeys within the upper two thirds of the building height. Below this level a linear variation of ω should be assumed. The value at first floor level should be taken as 1.3 and 1.5 as appropriate to planar and spatial frames.

$T = 1$

$\omega = 0.6x1+0.85$

$= 1.45$



In addition to the above, column moments shall satisfy the condition on the relative strength between columns and beams framing into a joint, see cl. 4.4.1.3.

Shear Forces

In evaluating the design shear forces from the condition of static equilibrium, the design end moments shall be the most adverse ones (ie. those producing the maximum shear force) obtained from the analysis of the structure under code load combination (equation 4.1.4) modified, if appropriate, by the dynamic magnification factors. For DL III structures they will be multiplied by a factor γ_n=1.10 as well.

4.3.1.3 Beam-Column Joints

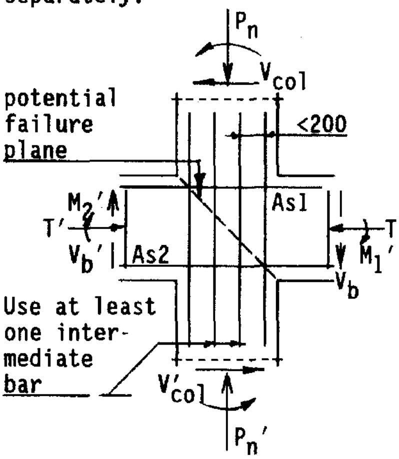
No explicit evaluation of the internal forces in the joint is required for DL II structures.

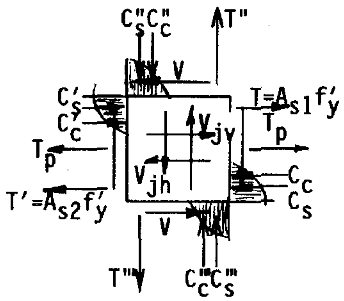
DL III Structures

The design bending moments shall be obtained by multiplying the design flexural strength of the end sections by a factor γ_n = 1.25, except where plastic hinges in columns are allowed (cl. 4.4.1.3). The axial force on the beam shall be the minimum corresponding to the seismic design forces.

Results of actions, - column moment and shear force, horizontal and vertical shear forces in the core of the joint - shall be calculated by a rational analysis taking into account the effect of all the forces acting on the joint's equilibrium.

When two frames which are not on the same plane have common joints, verification of these joints can be done in each direction separately.





The simplified formula

$$V_{jh} = \gamma_n\,(A_{s1} + A_{s2})\,f_{yd}\left(1 - \frac{2h_b}{l_c + l_c'}\right) \qquad (4.2.1.3)$$

where l_c, l_c' are the heights above and below the column, gives a conservative value of the horizontal shear force on the joint when

$$\frac{h_c}{l_1} \geq 0.08, \qquad \frac{l_2}{l_1} \geq 0.7$$

where l_1, l_2 are the spans of the beams. The vertical shear force is given approximately by the formula

$$V_{jv} = V_{jh}\,\frac{b_b}{h_c}$$

4.3.1.4 Structural Walls

The design actions shall be obtained from a linear analysis of the building under the code load combination (equation 4.1.4) modified as appropriate in accordance with cl. 4.3.1.4 1/2/3/4/5 below.

4.3.1.4.1 Redistribution

The distribution of the total force to the various walls as obtained from the elastic analysis, may be subsequently modified, provided the global equilibrium is maintained and the maximum value of the action in any wall is not reduced by more than 30%.

In a coupled wall, the elastic shear forces in the coupling beams can also be modified, with a maximum reduction of 20% provided that corresponding increases in the shear capacities of beams at other floors are made.

4.3.1.4.2 <u>Bending Moment Design Envelope</u>

The design moments along the height of the wall shall be those given by a linear envelope of the calculated moment diagram, vertically displaced by a distance equal to the horizontal length of the wall.

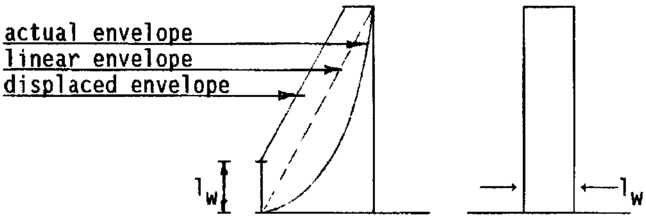


fig. 4.3.1.2 Bending moment design envelope

4.3.1.4.3 <u>Earthquake Induced Axial Load in Coupled Walls</u>

The design axial load in the walls due to the lateral action shall be computed using the shear strengths of the coupling beams above the section considered, calculated by using characteristic values of concrete and steel strength.

The shear strength of the beams thus calculated shall be further amplified by a factor of 1.25 to account for possible unfavourable increase of the beam strength with respect to the design values.

4.3.1.4.4 <u>Dynamic Magnification Factors</u>

In case the equivalent static analysis is adopted, the shear forces in the walls shall be magnified by the dynamic amplification factor ω as given by the expression below for buildings up to 5 storeys high:

$$\omega = 0.1N+0.9$$

where N is the number of storeys.

For walls taller than five storeys, ω shall be linearly increased up to the value of ω=1.8, for N=15

4.3.1.4.5 <u>Shear Forces</u> (DL III Structures Only)

a) The design shear forces in walls shall be compatible with the actual flexural strength that can be possibly developed at the wall base. This shall be obtained by multiplying the shear forces due to code loading by the following γ_n factor:

$$\gamma_n = M^{+}_{u,d}/M_d$$

Where M_d is the design moment obtained from the analysis, and $M^+_{u,d}$ is the flexural strength of the section on the basis of actual reinforcement provided, calculated by using the characteristic values of concrete and steel strengths.

In evaluating the flexural strength of the base section the appropriate axial load shall also be considered.

b) The factor γ_n need not to be taken greater than 4.

4.4 Dimensioning and Verification

4.4.1 Linear Elements

4.4.1.1 General

The design strengths of the structural elements in bending, bending with axial force, shear and torsion shall be evaluated in accordance with the code for concrete except as modified by provisions in the present cl. 4.4.1.

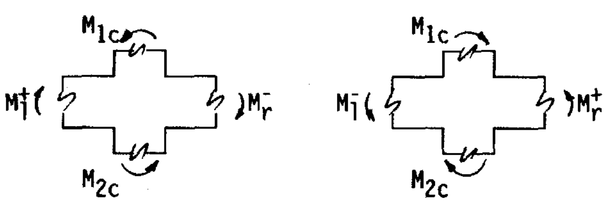
4.4.1.2 Limiting Axial Load

For ductility reasons the design axial compression load under the most severe load combination including the seismic action shall not exceed the following limit:

$$0.5A_g \cdot f_{ck}$$

4.4.1.3 Beam-Column Strength Ratio

Except for cases where hinge formation in columns is permitted (see below) at any beam-column joint the sum of the absolute values of the design ultimate moments of the columns (under the most unfavourable value of the axial force) shall not be less than the sum of the absolute values of the design ultimate moments of the beams framing into that joint.



$\Sigma|M_{ic}| > |M_l^+| + |M_r^-|$ $\quad$ $\Sigma|M_{ic}| > |M_l^-| + |M_r^+|$

4.4.1.3 Beam-column Strength Ratio

Development of plastic hinges in columns should be avoided because sources of energy dissipation should be located in beams rather than in columns. Attention is called to the potentially dangerous, yet rather commonly adopted, 'soft first storey' concept. The behaviour of this type of building is difficult to control, because of its marked sensitivity to both the structural and ground motion characteristics.

For the above reasons this solution, which is not included in the structural types covered by this code, can be designed if design forces are increased by 50% and special detailing provisions for soft first storey columns satisfying DL III demands are adopted.

DL III Structures

For columns of DL III structures, the design bending moments shall account for the possible increase in strength of the beams connecting at the joint. Unless otherwise justified, the global strength increase can be assumed as $\gamma_n = 1.5$ and is applicable to all storeys, including the column bases in the ground floor.

Development of plastic hinges in columns and of columns hinge mechanisms (ie. exception from the prescription on beam-column strength ratio) is permitted in the following cases:

- For frames having four of more columns, hinging is permitted to occur in one column for every three others remaining elastic.
- Column hinge mechanisms are permitted in single and two-storey buildings and in the top storey of a multi-storey building.

4.4.1.4 Resistance to shear

4.4.1.4.1 Contribution of Concrete

The magnitude of the term: V_{cd} expressing the design resistance contributed by concrete (Code for Concrete cl. 11.2.2, equation 11.8) shall be taken as follows:

a) When $N_d \leq 0.1 A_g . f_{cd}$ shall be assumed to be zero where stirrups are required in accordance with cl. 4.5.1.3 (except case c).

b) When $N_d > 0.1 A_g . f_{cd}$ shall be computed by the expression:

$$V_{cd} = 2\tau_{Rd} . b_w . d . \beta_1 \qquad (4.4.1.4.1.1)$$

where the values of τ_{Rd} and β_1 are given in the code for concrete as functions of the concrete grades (Table 11.1 equation 11.3).

4.4.1.4.2 Transverse Reinforcement

I. $N_d < 0.1 A_g . f_{cd}$. - Two cases shall be considered, depending on the value of the ratio ζ.

a) $\zeta > 0$: The resistance to shear provided by the reinforcement V_{wd} shall be assessed on the basis of the truss model, in accordance with the procedure given in the code for concrete cl. 11.2.

b) $\zeta < 0$: Here reversal of shear takes place.

- The same requirements as in a) exist when V_{sd} does not exceed the limit value V_{Rd1} where:

$$V_{Rd1} = 3\ (2+\zeta)\ \tau_{Rd}\cdot b_w\cdot d$$

- For V_{sd} exceeding the limit value V_{Rd2} where:

$$V_{Rd2} = 6\ (2+\zeta)\ \tau_{Rd}\cdot b_w\cdot d$$

the entire shear shall be resisted by diagonal reinforcement across the web. Steel bars inclined in two directions shall balance with their compression and tension components the shear forces of opposite sign V_{sd} and $\zeta.V_{sd}$ occurring at the section.

- For $V_{Rd1} < V_{sd} < V_{Rd2}$

one half of the maximum shear force shall be carried by doubly diagonal bars, the other half by transverse reinforcement.

II. $N_d > 0.1.A_g.f_{cd}$. - The resistance to shear shall be checked as for the case I. a).

4.4.2 Beam-Column Joints (DL III Structures Only)

4.4.2.1 Horizontal Joint Shear

4.4.2.1.1 Nominal Horizontal Shear Stress

The nominal horizontal shear stress in the joint as given by the following expression

$$\tau_{jh} = \frac{V_{jh}}{b_j h_j} \qquad (4.4.2.1.1.1)$$

shall not exceed the value $20\tau_{Rd}$.

The effective joint width b_j shall be taken as:

a) When $b_c > b_w$ either $b_j = b_c$
or $b_j = b_w + 0.5h$ whichever is smaller.

b) When $b_c < b_w$ either $b_j = b_w$
or $b_j = b_c + 0.5h_c$ whichever is smaller.

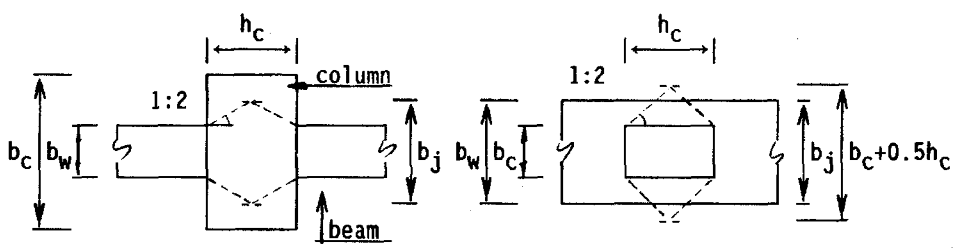


fig.4.4.2.1.1 Effective Joint Width

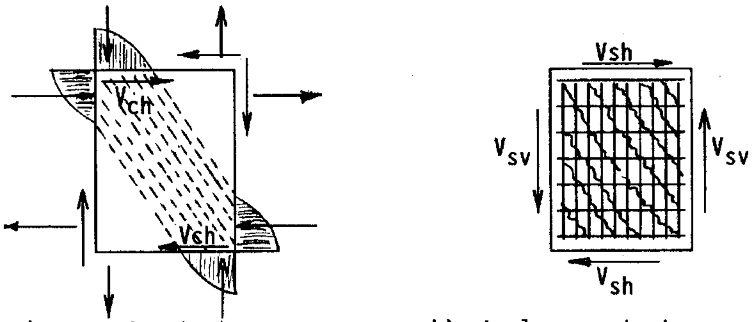
4.4.2.1.2 Mechanisms of Joint Core Shear Resistance

Two mechanisms for transmission of the horizontal shear force V_{jh} through the joint core are in general possible:

a) A diagonal concrete strut across the compressed joint corners carrying a shear force V_{ch}.

b) A truss mechanism consisting of horizontal stirrups and diagonal concrete struts carrying a shear force V_{sh} where:

$$V_{sh} + V_{ch} = V_{jh} \qquad (4.4.2.1.2.1)$$



a) concrete strut b) steel-concrete truss

fig. 4.4.2.1.2 Mechanisms of shear resistance

4.4.2.1.3 Shear Force Carried by Concrete

The value of shear force carried by the concrete strut, V_{ch}, shall be assumed zero except for the following:

a) When the minimum average compression stress on the gross concrete area of the column above the joint, exceeds $0.1f_{ck}$:

$$V_{ch} = 2\tau_{Rd}\sqrt{\sigma_{cm}-0.1f_{ck}}\,.b_j.h_c \qquad (4.4.2.1.3.1)$$

b) When the design precludes the formation of any beam plastic hinge at the joint, or when all beams of the joint are detailed so that the critical section of the plastic hinge is located at a distance from the column face not less that h_b or for external joints where the flexural steel is anchored outside the column core in a beam stub

$$V_{ch} = \frac{A'_s}{A_s}\frac{V_{jh}}{2}\left(1 + \frac{N_d}{0.4A_g.f_{ck}}\right) \qquad (4.4.2.1.3.3)$$

where the ratio A'_s/A_s of the compression to the tension longitudinal beam reinforcement shall be taken larger than 1.0.

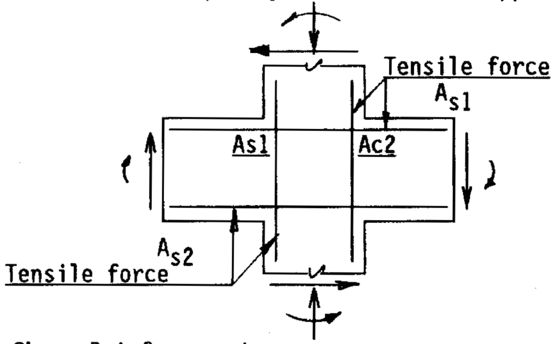
When the axial column load results in tensile stresses over the gross concrete area exceeding $0.2f_{ck}$ the entire joint shear shall be resisted by reinforcement.

For axial tension smaller than this limit the value of V_{ch} may be linearly interpolated between zero and the values given by equation 4.4.2.1.3.3 with N_d taken as zero.

c) When parts A_{a1} and A_{a2} of the tensile reinforcement A_{s1} and A_{s2} of the adjacent beams are bent vertically and anchored in the tensile face of the column:

$$V_{ch} = A_a \frac{f_{yk}}{\gamma_s} \tag{4.4.2.1.3.4}$$

where A_a is the smaller of A_{a1} and A_{a2}. The values obtained from equations 4.4.2.1.1/4 may be added where applicable.



4.4.2.1.4 Horizontal Shear Reinforcement

The horizontal shear reinforcement shall be capable of carrying the design joint shear force:

$$V_{sh} = V_{jh} - V_{ch}$$

across a corner-to-corner potential failure plane. The effective total area of horizontal reinforcement that crosses the critical diagonal plane and is situated within the effective joint width b_j, shall not be less than

$$A_{jh} = \frac{V_{sh}}{f_{yk}/\gamma_s} \tag{4.4.2.1.4.1}$$

Horizontal sets of stirrups shall be placed as uniformly as practicable between the top and bottom beam reinforcement.

4.4.2.2 Vertical Joint Shear

4.4.2.2.1 Vertical Joint Reinforcement

The vertical joint shear reinforcement shall be able to resist a vertical shear force

$$V_{jv} - V_{cv}$$

where the value V_{cv} shall be determined from:

$$V_{cv} = \frac{A'_{sc}}{A_{sc}} \; V_{jv} \; (0.6 + \frac{N_d}{A_g \cdot f_{ck}}) \qquad (4.4.2.2.1.1)$$

where A'_{sc} and A_{sc} are the areas of longitudinal compression and tension reinforcement in columns, with the following exceptions:

a) Where axial load results in tensile stresses over the column section, the value given by eq. 4.4.2.2.1.1 with N_d taken as zero; and zero when the axial tension over the gross concrete area is $0.2f_{ck}$.

b) Where plastic hinges are expected to form in the column above or below a joint, as part of the primary seismic energy dissipating mechanism, V_{cv} shall be assumed to be zero for any value of the axial load or the column.

The required area of vertical joint shear reinforcement within the effective joint width j shall be determined from:

$$A_{jv} = \frac{V_{sv}}{f_{yk}/\gamma_s}$$

The vertical joint shear reinforcement shall consist of intermediate column bars, placed in the plane of bending between corner bars, or of vertical stirrup ties or special bars, placed in the column and adequately anchored to transmit the required tensile forces within the joint.

The spacing of vertical joint reinforcement in each plane of any beam framing into a joint shall not exceed 200mm, and in no case shall there be less than one intermediate bar in each side of the column in that plane.

4.4.2.3 Eccentric Beam-Column Joints

All design provisions of this section apply, except that in case of an eccentricity of a beam relative to the column into which it frames, as measured by the distance between the geometric centrelines of the two members, the effective joint width shall not be taken larger than

$$0.5(b_w + b_c + 0.5h_c) - e$$

4.4.3 Structural Walls

The purpose of the provisions in this chapter, as well as those in cl. 5.5 relative to detailing, is to provide walls with adequate ductility and energy-dissipation capacity through flexural yielding in clearly defined hinge zones.

In addition, the likelihood of failure due to shear or inadequate anchorage, or even the occurrence of significant shear inelastic deformations (which progressively impair the energy-dissipation capacity of the structure), is reduced to a minimum.

4.4.3.1 General

The design strengths of walls and coupling beams shall be evaluated as for linear elements (cl. 4.4.1), except as modified by the provision in the present cl. 4.4.3.

4.4.3.2 Resistance to Shear

4.4.3.2.1 Maximum Allowable Shear Stress

The maximum nominal shear design shear stress in a wall section, evaluated by means of the expression:

$$\tau_d = V_d/A_g$$

where V_d is the design shear force computed in accordance with cl. 4.3.1.4 shall not exceed the following limit:

$$\tau_d \leq 10\tau_{Rd}$$

4.4.3.2.2 Contribution of Concrete to Shear Strength

In the potential plastic hinge zone, as defined in cl. 5.5.3.1 the contribution of concrete to shear resistance is assumed to be zero, unless the minimum design axial load produces an average compression stress over the gross concrete area of the wall equal at least to $0.1.f_{cd}$. In this case the shear stress contributed by concrete shall be computed by:

$$\tau_{cd} = 2\tau_{Rd}\cdot\beta_1$$

with the values of τ_{Rd} and β_1 given in the code for concrete cl. 11.1.2 Table 11.1 and cl. 11.1.2.2 equation 11.3.

Outside the potential hinge zone, and when the average compressive stress is less than $0.1f_{cd}$ the shear stress contributed by concrete shall be taken as:

$$\tau_{cd} = 2.0\tau_{Rd}$$

while in cases where the average stress is grater than $0.1f_{cd}$:

$$\tau_{cd} = 2.5_{Rd}\cdot\beta_1$$

4.4.3.2.3 Web Reinforcement

Horizontal bars, fully anchored at the extremities of the wall section, shall be provided in the amount:

$$\rho_h = \frac{A_h}{b.s_v} = \frac{\tau_d - \tau_{cd}}{f_{yd}}$$

while the vertical reinforcement ratio shall be:

$$\rho_v = \frac{A_v}{b.s_h} = \frac{\tau_d - \tau_{cd} - N_d/A_g}{f_{yd}}$$

The vertical shear reinforcement can be assumed to fully contribute to the required flexural strength.

4.4.3.3 Coupling Beams

Coupling beams are ductile, energy-dissipating elements connecting in a regular pattern two or more walls. Symmetrical flexural reinforcement ($\rho=\rho'$) shall be adopted in case of the usual arrangement. Design for flexure and shear shall be carried out as for ordinary beams unless the following limits are exceeded:

$$\tau_d \geq 6\tau_{Rd}$$

$$\rho = \frac{1}{4}\frac{1}{h}\ \frac{\sqrt{f_{ck}}}{f_{yk}} \qquad (f_{ck},\ f_{yk} \text{ in MPa})$$

(ρ = longitudinal reinforcement ratio, top or bottom)

in which case all flexural and shear actions shall be resisted by diagonal reinforcement in both directions.

4.4.4 Diaphragms and Stair Slabs

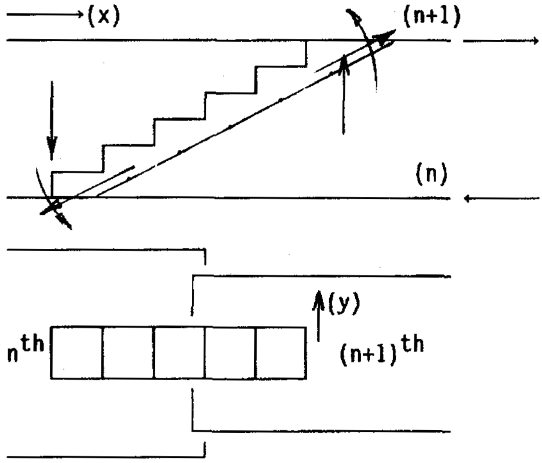
Floor systems connecting vertical seismic elements (frames, walls, cores) shall be checked for the forces to be transmitted to the seismic elements to enable them to develop their maximum capacity.

When it is shown that the forces to be transmitted do not produce yielding in a diaphragm, provisions indicated in the code for concrete apply. If yielding in a diaphragm cannot be avoided provisions for ductile structural walls and in particular, cl. 5.5.3.1 for confining reinforcement in boundary zones, shall be applied.

Openings in diaphragms shall be so arranged that unintentional failures across weak lines do not reduce the strength of the diaphragm. Boundary elements around openings shall be provided whenever needed, with a rational assessment of their required strength.

Adequate connection must be provided between the diaphragm and the vertical seismic resistant elements. This aspect is particularly important when staircases and elevator shafts act as seismic elements since this usually involves interruption of the diaphragm.

Stair slabs (inclined) shall be appropriately designed so that relative interstorey displacements are compatible with axial and flexural rigidity of stair slabs. Axial forces, bending moments and shear forces may be developed due to relative displacements along the x-axis. Bending and shear may also be developed due to relative displacements along the y-axis.



4.5 Verifications

4.5.1 Collapse Verification

For the purpose of the present code a structure shall be deemed to satisfy the safety requirements against collapse if the following conditions are met:

- The strength and stability verifications are satisfied.
- The elements are dimensioned and detailed in accordance with the rules given in chs. 4 and 5 relative to the appropriate structural type and intended ductility level.

4.5.2 Strength Verification

The following condition must be satisfied for every element:

$$S_d \leq R_d \tag{4.5.2.1}$$

where: S_d is the design load effect on the element evaluated according to cl. 4.3.

R_d is the design strength of the element evaluated according to cl. 4.4.

4.5.3 Stability Verification

The stability verification shall be considered satisfied if:

- The deformability index θ is less than 0.1.

- For $0.1 \leq \theta \leq 0.15$ the 2nd order effects are calculated by means of one of the statical methods in ch. 14 of the Code for Concrete and added to the design forces.

- The stability verification cannot be satisfied if $\theta > 0.15$.

4.5.4 Serviceability Verification

The elastic interstorey drift, Δel, resulting from the application of the horizontal forces specified in cl.4.2.4 or from the dynamic procedure as in cl.4.2.5, shall at any storey satisfy the condition:

$$\Delta_{el} \leq \frac{0.010}{K}.h$$

where h is the clear height of the floor.

For class III or IV buildings, the indicated limits may be increased of 50% if it can be demonstrated that the finishes adopted are not brittle-type and can accommodate without significant damage those limits.

When the limits above are exceeded separation of the non-structural elements is required, of an amount adequate for permitting an interstorey drift equal at least to

$$\bar{\Delta} = 0.35\Delta_{el}.K$$

to take place without restraint.

In no case shall the interstorey drift, Δ_{el} exceed the limit:

$$\Delta_{max} = \frac{0.025}{K}.h$$

4.5.5 Maximum Expected Displacements

The maximum expected displacements of the building shall be obtained by multiplying the displacements produced by the system of horizontal forces specified in cl. 4.2.4 or those obtained from the dynamic analysis as in cl. 4.2.5 by the appropriate values of the behaviour factor K.

5. DETAILING, CONSTRUCTION AND USE OF STRUCTURE

When there is no distinction, the requirements of this chapter apply in common for structures of DL II and DL III. Requirements for structures of DL I are always referred to explicitly.

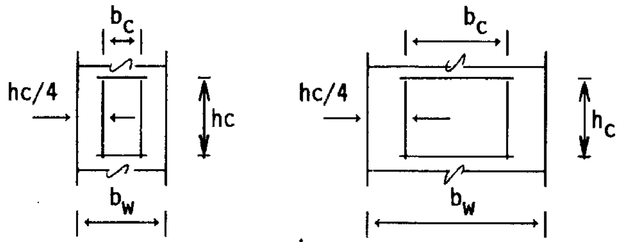
5.1 Elements Subject to Bending (N_d<0.1 $A_g.f_{cd}$)

5.1.1 Geometrical constraints

DL II and DL III Structures

The following restrictions must be satisfied, unless it is specifically proved that they are not required.

a) In order to ensure that there is effective transfer of moment from the beam to the column, the width of the beam shall not be less than 200mm or greater than the width of the column plus 1/4 of the depth of the column on each side. In no case shall the width of the beam exceed two times the width of the column.



$$b_{wmax} \leq b_c + \frac{h_c}{2} \leq 2b_c$$

b) In order that there is no risk of lateral buckling under non-linear response, the ratio of the width to the depth of the beam shall not be less than 0.25.

c) The behaviour of members in portal frames whose l/h ratio is less than 4, is significantly different from that of slender members. For this reason, the ratio l/h shall not be less than 4. (This requirement is not valid for coupling beams in structures with walls, cl. 4.4.3.3).

d) The eccentricity of a beam with respect to the column on which it is connected, as measured by the distance between the geometrical axes of the two elements, shall not be greater than 1/4 of the width of the column.

5.1.2 <u>Longitudinal Reinforcement</u>

<u>DL II and DL III Structures</u>

a) In every section, in order to ensure that the ultimate moment is greater than the moment of the cracked section, the compression reinforcement, top or bottom shall not be less than

$$\rho_{min} = 1.4/f_{yk} \quad (f_{yk} \text{ in MPa}) \qquad (5.1.2.1)$$

and to ensure adequate ductility it shall not be more than

$$\rho_{max} = 7.0/f_{yk} \quad (f_{yk} \text{ in MPa}) \qquad (5.1.2.2)$$

where ρ_{min} and ρ_{max} refer to the total concrete area A_g.

b) At least two bars of 12mm diameter shall be placed both at the top and bottom, along the whole length of the element.

c) In order to ensure that there is adequate ductility and strength for stress reversal, the compression reinforcement shall not be less than half the tension reinforcement, in areas where plastic joints might develop. ($\rho' > 0.5\rho$)

d) At least one quarter of the top reinforcement at the end of the element shall extend to the whole of its length.

e) In flanged beams with a T or L shape which are monolithic with the slab, the reinforcement that is taken into consideration near the columns, in addition to the reinforcement within the links, shall be as follows:

I. For interior columns, if there is a transverse beam which is monolithic with the column, all the reinforcement in the slab within a distance of 4 times the slab thickness on each side of the column.

II. For interior columns, in the absence of a transverse beam, all the reinforcement within a distance of 2.5 times the slab thickness on each side of the column.

III. For exterior columns, if there is a transverse beam of similar dimensions and which is monolithic with the column on which the beam reinforcement is anchored, all the reinforcement of the slab within a distance of 2 times the slab thickness on each side of the column.

IV. For exterior columns, in the absence of a transverse beam, all the of reinforcement within the width of the column.

In any event, at least 75% of the required reinforcement on each side shall pass through the column or be anchored in the core of the column.

<u>DL I Structures</u>

Only cl. 5.1.2 a. is required.

5.1.3 <u>Minimum Transverse Reinforcement.</u>

Transverse reinforcement shall be placed in accordance with the requirements of this part, unless more reinforcement is required in order to resist the shear force (cl. 4.4.1.4). The aim of the transverse reinforcement is:

- to confine the concrete in order to increase the ultimate deformation and the bonding strength of the reinforcement.

- to tie the reinforcement and prevent its buckling.

- to provide shear resistance.

Regions of beams which are considered as "Critical" regions are:

a) Twice the depth of the beam, starting from the column face, towards the middle of the span, on both sides of the beam.

b) Twice the depth of the beam on both sides of a possible plastic hinge.

c) Where compression reinforcement is required.

DL II Structures

In critical regions, as defined above, the links shall be at least 8mm diameter with spacing not exceeding the minimum of the following:

a) h/4
b) $8\Phi l$ (Φl is the diameter of the longitudinal reinforcement)
c) $24\Phi h$ (Φh is the diameter of the link)
d) 200mm

The first link shall be placed not more than 50mm from the side of the column. At least one in every two separate longitudinal bars in the beam shall be restrained by a 90° bend of a link.

DL III Structures

In the critical regions, as defined above, the links shall be at least 8mm diameter with spacing not exceeding the minimum of the following:

a) h/4
b) $6\Phi l$
c) 150mm

The minimum area of one leg of the link shall be:

$$A_{s,\ min} = \frac{\Sigma A_b . f_{yk}}{16 f_{ykt}} \; \frac{s}{100} \qquad 5.1.3.1$$

so as to prevent the deflection of the longitudinal bars in the event of large reverse plastic deformations.

ΣA_b = The sum of the area of the longitudinal bars restrained by the leg of the link.
f_{yk} = The ultimate strength of the longitudinal bars
f_{ykt} = The ultimate strength of the links
s = The link spacing

The first link shall be placed not more than 50mm from the side of the column.

At least one in every two separate longitudinal bars in the beam must be restrained by a 90° bend of a link.

Elements Subject to Bending and Axial Force ($N_d > 0.1 A_g . f_{cd}$)

The aim of this clause is to provide the columns with adequate ductility which might prove necessary if there is a deviation from the expected static behaviour of the structure.

Observations on damage caused by earthquakes frequently indicate that corner columns are more vulnerable than interior ones, due to unforeseen torsional phenomena. It is therefore suggested that particular care in detailing shall be given to corner columns or even to make them stronger than what is required from the analysis.

5.2.1 Geometrical Constraints

Structures of DL II

The minimum section dimension shall not be less than 250mm.

The l/b ratio shall not be greater than 25.

Structures of DL III

The minimum section dimension shall not be less than 300mm.

The l/b ratio shall not be less than:

- 16 for columns with moments of opposite sign at the two ends.
- 10 for cantilever columns.

5.2.2 Longitudinal Reinforcement

The reinforcement ratio shall not be less than 1% or greater than 6% even at sections where there are reinforcement connections $(1.0\%<p<6.0\%)$.

For steel S400 the reinforcement outside the connections shall not be greater than 4%.

When the dimensions are determined by architectural requirements, the minimum reinforcement can be reduced. When the calculated required reinforcement is less than 0.5% then the reinforcement that may be used is twice the required reinforcement. In no event shall the reinforcement be less than 0.5%.

The spacing between bars shall not be more than 250mm for structures of DL II or 200mm for structures of DL III.

DL I Structures

The above requirements shall also be satisfied for DL I structures.

5.2.3 Transverse Reinforcement

Nominal reinforcement shall be placed throughout the height of the column, whilst special reinforcement shall be placed in critical regions, as defined in paragraph 5.2.3.1.

The reinforcement ratio which shall be used shall be as defined in this article, unless more reinforcement is required for shear resistance in accordance with cl. 4.4.1.4.

5.2.3.1 Critical Regions in columns

a) For ordinary cases, critical regions are considered to be the two ends of the column, above and below the joints and over a length from the face of the joint which shall not be less than the maximum of the following:

- the longest dimension of the section
- 1/6 of the clear height of the column
- 450mm

b) Where there is an infill wall in contact with one or both sides of a column, in part of the height of the column, then the whole length of the column shall be considered as critical.

c) In the case where part of the column is tied to a concrete wall, the untied part of the column shall be considered as critical region.

Critical regions of columns require links more closely spaced and well anchored than the rest of the column to ensure better confinement of the concrete (and hence adequate ductility), lateral support of the longitudinal bars and shear strength.

5.2.3.2 DL II Structures

Critical Regions

Special transverse reinforcement shall be placed having 8mm minimum diameter in link or spiral form.

Additional links which tie the bars which are not directly tied to the links, shall be placed in accordance with the concrete code.

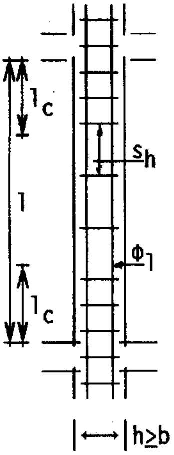
The spacing between the spirals or links shall not exceed the minimum of the following:

a) 8 times the diameter of the smaller bar of the main reinforcement.
b) 1/2 the minimum section dimension
c) 200mm.

The above transverse reinforcement shall extend throughout the height of the beam joint.

Non-Critical Regions

The minimum transverse reinforcement in the non critical regions shall be in accordance with the concrete code.



DUCTILITY LEVEL II

CRITICAL REGION	l_c= max(h, l/6, 450mm)
SPACING	CRITICAL REGION: s=min($8\Phi_l$, b/2, 200mm)
	ELSEWHERE: s=min($12\Phi_l$, b, 300mm)

Special transverse reinforcement: critical regions and spacing

5.2.3.3 DL III Structures

Critical regions

The volumetric ratio of the transverse reinforcement (spiral or links) shall not be less than the maximum of the following:

$$\rho_s = \lambda_1 \frac{f_{ck}}{f_{yk}} \qquad (5.2.3.3.1)$$

or

$$\rho_s = \lambda_2 \left(\frac{A_g}{A_c} - 1\right) \frac{f_{ck}}{f_{yk}} \qquad (5.2.3.3.2)$$

where A_g = total cross sectional area
A_c = area of confined concrete

and the values of λ1 and λ2 are given by the following table as functions of the axial loading.

$N_d/A_g \cdot f_{ck}$	0.10	0.20	0.30	0.40	0.50
λ_1	0.05	0.06	0.07	0.08	0.09
λ_2	0.18	0.22	0.26	0.30	0.34

Values of λ_1 and λ_2 in equations 5.2.3.3.1 and 5.2.3.3.2

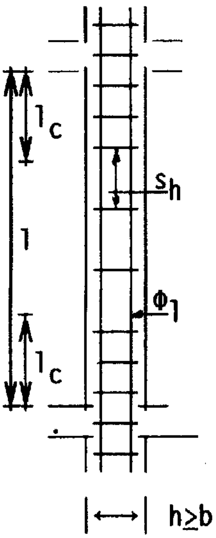
The volumetric ratio is the ratio of the volume of the spiral reinforcement or the links over the total area of the core of the concrete (measured from the exterior side of the bars) within the distance between the spirals or the links s_h. The volumetric ratio for rectangular sections is defined as:

$$\rho_s = A_{sh}/s_h \cdot h' \qquad (5.2.3.3.3)$$

where A_{sh} is the total area of the links in each of the main directions of the section, s_h is the spacing between the links and h' is the distance between the centres of the exterior bars.

The minimum diameter of the links or spiral shall be 8mm. The maximum distance between the spirals or the links shall not exceed the minimum of the following:

a) 6 times the minimum diameter of the longitudinal reinforcement.
b) 1/4 of the minimum section dimension
c) 150mm.



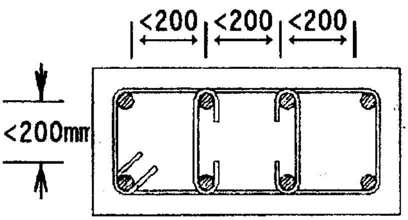
DUCTILITY LEVEL III		
CRITICAL REGION	l_c = max(h, l/6, 450mm)	
SPACING	CRITICAL REGION:	s=min($6\Phi_l$, b/4, 150mm)
	ELSEWHERE:	s=min($8\Phi_l$, b/2, 200mm)

Special transverse reinforcement:
critical regions and spacing

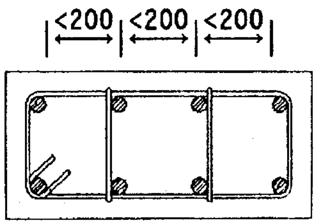
3. Every longitudinal bar or bundle of bars shall be restrained by a bend of a link of at least 135° or by additional links, except:

a) bars or bundles of bars between two bars which are restrained by the same link and their spacing is not greater than 200mm.

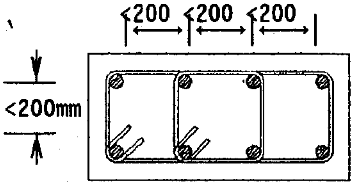
b) interior layers of bars in the core of the concrete whose center is 75mm or more from the interior side of the links.



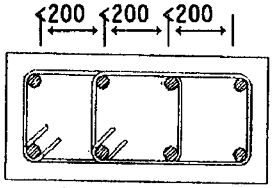
a) Complementary ties around main reinforcement



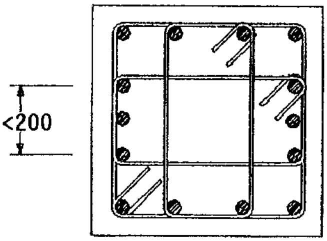
b) Complementary ties around the link



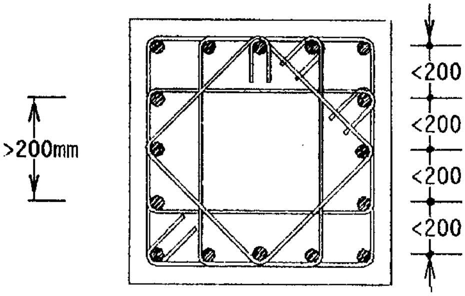
c) Two links
-preferred detail



d) Two links
-detail c preferred



e) Three links



f) Four links

4. The tensile strength of the leg of a link or additional link shall be at least 1/6 the tensile strength of the bar or bars which it restrains, including bars that are exempted by cl. 3.a.

5. The end of every additional link shall be tied to a longitudinal bar, or the peripheral link at a point next to a bar, with a bend of at least 135° and the straight part shall extend beyond the bend at least 10 times the diameter of the additional link. Additional links and legs of links shall not have in between spacing of more than 200mm or 1/4 of the dimension of the column perpendicular to the direction of the transverse reinforcement.

<u>Non-critical regions</u>

The requirements for critical regions of DL II columns shall apply

5.3 <u>Beam-Column Joints</u>

5.3.1 <u>Confinement</u>

<u>DL I and DL II Structures</u>

The horizontal transverse reinforcement in the joints shall not be less than what is required in the columns.

<u>DL III Structures</u>

The horizontal transverse reinforcement in the beam-column joints shall not be less than what is required in the columns apart from the case where there are beams on the four sides of the column designed in accordance with cl. 4.4.2.1.3/b or c, in which case the transverse reinforcement of the joint can be reduced to one half of what is required in the columns, but the spacing between the links shall in no case be greater than 10 times the diameter of the bars of the column or 200mm, whichever is less.

When the width of the column is greater than the width of the joint, as defined in cl. 4.4.2.1.1, all the bending reinforcement in the column which is required to be tied to the narrow beam shall pass within the width of the joint $b_j h_c$. Additional column longitudinal reinforcement shall be placed outside the width of the joint.

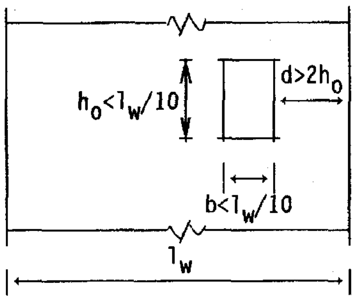
5.4 Structural Walls

5.4.1 Geometrical Constraints

The thickness of structural walls shall not be less than 150mm.

Openings in the walls which are not in a regular arrangement so as to form coupled walls shall be avoided, except if the influence on the seismic performance of the wall is negligible or specific analysis of the performance of the wall locally is made.

Openings that can be considered as negligible are those whose largest dimension does not exceed 1/10 the width of the wall and their distance from the edge of the wall or from another opening is not less than twice their height. Special reinforcement shall be placed around every opening in order to compensate for the strength of the part which is removed.

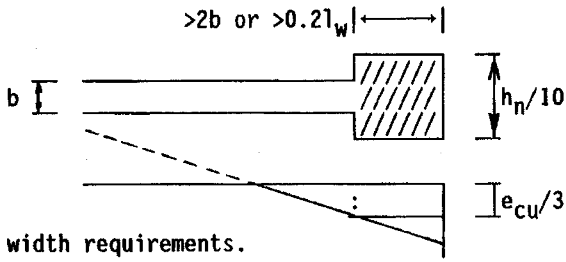


Dimensions of wall openings that can be considered negligible.

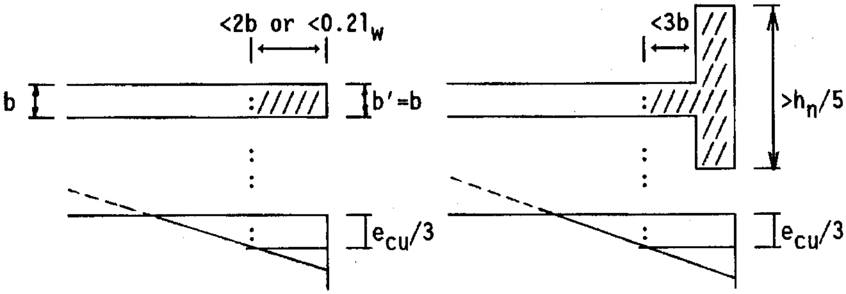
<u>DL III Structures</u>: In addition to the above requirements, the following shall be implemented for structures of DL III:

a) The ratio of the total height (h_w) to the length (l_w) of the wall shall not be less than 2.

b) The local thickness of a wall shall not be less than $h_n/10$, (where h_n the height of the floor) in cases where the largest compressive deformation exceeds the value $\varepsilon_{cu}/3$. The value of ε_{cu} is given in the Concrete Code, cl. 7.1.2.



Minimum width requirements.



Exemptions from the requirements of clause 5.4.1 b.

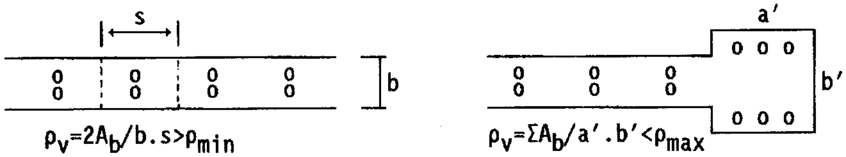
The following cases are exempted:

1) When the distance from the critical fibre, i.e. when $\varepsilon_c = \varepsilon_{cu}/3$, from the edge of the wall is less than 2b or $0.2l_w$, and

2) when the distance between the critical fibre and the vertical wall at which the wall ends, or from an end wall with width not less than $h_n/5$, is less than 3b.

5.4.2 Vertical reinforcement

The total vertical reinforcement shall not be less than 0.25% of the cross sectional area, or greater than 4%.



Definition of vertical reinforcement ratio

At least two layers of reinforcement shall be placed, one on each side of the wall. The reinforcement bar diameter in any part of the wall shall not exceed b/10.

The spacing between the reinforcement bars shall not exceed 300mm. When the section is required to be confined, the bar spacing shall not be greater than 200mm.

Curtailement of Bars

The vertical reinforcement shall be curtailed according to the bending moment diagram, adding the anchorage lengths of the curtailed bars.

Splicing of Bars

Vertical bar connection shall be avoided in regions where the formation of plastic hinges is expected. In no case shall connections be made in more than 1/3 of the reinforcement in these regions. Particular attention shall be placed in the connection of the main (bending) vertical reinforcement. The connections shall be staggered and the in-between spacing in the vertical direction, shall be at least twice the overlap length.

Construction Joints

The ratio of the vertical reinforcement passing through a construction joint, shall be such that it can withstand all the shear resistance of the concrete and given by the expression:

$$\rho_v = (1.3f_{ctm} - 0.7\frac{N_d}{A_g})/f_{yk} > 0.0025 \qquad (5.4.2.1)$$

where

$\rho_v = A_{st}/b_{lw}$ with A_{st} the total vertical reinforcement including the reinforcement of the end elements which resist bending.

A_g is the total wall cross sectional area, including the end elements.

N_d is the minimum bending force in the wall. If the wall is to be subjected to tension, then N_d shall be taken as negative.

5.4.3 Horizontal Reinforcement

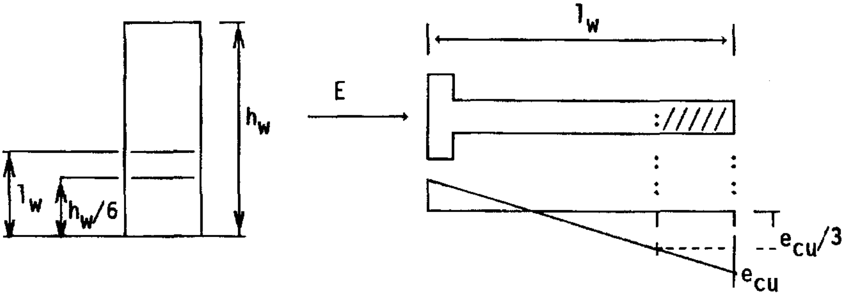
The requirements for minimum reinforcement, largest bar diameter and maximum bar spacing, shall be the same as those for the vertical reinforcement cl. 5.4.2.

5.4.3.1 Regions with Special Horizontal Reinforcement

a) Regions of walls in which special horizontal reinforcement is required, as defined in paragraph b, are defined as follows:

In the vertical direction they shall extend from the base above the point of the likely plastic hinge, which in this case shall be considered the largest of the length l_w or 1/6 of the height hw of the wall.

In plan, in cases where the calculated concrete deformation exceeds the value of $\varepsilon_{cu}/3$.



b) The amount of special transverse reinforcement required is a function of the calculated depth of the neutral axis, at the base of the wall and the ductility level, as follows:

<u>DL II Structures</u>: The depth of the neutral axis is calculated for the worst moment M_d and is given from the formula:

$$\bar{x} = 0.20 \left(\frac{M_{u,d}}{M_d}\right) l_w \qquad 5.4.3.1.1$$

when:

$x \leq \bar{x}$: The horizontal reinforcement shall satisfy the requirements of cl. 5.4.3. Links that tie the vertical reinforcement shall be used in accordance with the concrete code.

$x > \bar{x}$: The horizontal reinforcement shall satisfy the requirements of cl. 5.2.3.2 (columns of DLII in critical regions).

<u>DL III Structures:</u> The depth of the neutral axis is calculated for the worst moment M_d and is given by the equation:

$$\bar{x} = 0.10 \left(\frac{M_{u,d}}{M_d}\right) L_w \qquad (5.4.3.1.2)$$

when:

$x \leq \bar{x}$: The horizontal reinforcement must satisfy the requirements of cl. 5.2.3.3 (columns of DL III in non critical regions).

$x > \bar{x}$: The horizontal reinforcement must satisfy the requirements of cl. 5.2.3.3 (columns of DL III in critical regions) with the values of λ1 and λ2 as given in the following table as a function of the depth of the neutral axis.

x/lw	0.10	0.20	0.30	0.40	0.50	0.60	0.70
λ1	0.07	0.08	0.09	0.105	0.115	0.125	0.135
λ2	0.18	0.205	0.23	0.26	0.285	0.31	0.34

Table 5.4.3.1. - Values of λ1 and λ2 which shall be used in equations 5.2.3.3.1 and 5.2.3.3.2

5.4.4 Coupling Beams

The diagonal reinforcement in each direction shall be tied with closed links in accordance with the requirements of cl. 5.2.3.3 and the spacing between the links shall not exceed 100mm.

The minimum width of the diagonally reinforced beams shall not be less than 200mm. The anchorage length of the diagonal reinforcement in the adjacent walls shall be increased by 50% above the normal length.

5.5 Anchorage and Connection of Reinforcing Bars

5.5.1 General

In addition to the rules of the concrete code, chapter 17, the following requirements must be satisfied so that reliable performance is ensured under cyclic loading conditions caused by the seismic actions.

All the reinforcing bars shall be in a position to develop their maximum strength $\gamma_n . f_{ck}$ when a plastic hinge is formed.

All the bars shall be considered to be in a non-effective anchorage bond condition, except when the anchorage is done in areas confined with transverse reinforcement where effective anchorage bond can be achieved.

When the equation (concrete code, cl. 17.4.1.2, equation 17.5) is applied in a region where plastic hinges are likely to be formed, the ratio $A_{s,cal}/A_{s,eff}$ shall be taken equal to 1.

5.5.2 Beams: Anchorage of Longitudinal Reinforcement

Beams connected to opposite sides of a column shall have both top and bottom reinforcement continuous through the column, where this is possible. When the reinforcement cannot be continuous through the column, due to a change in the beam cross section or in exterior columns, the reinforcement shall be anchored in the joint in accordance with the following:

a) The reinforcement shall extend to the opposite face of the confined area and shall be anchored satisfactorily so that it can develop its ultimate strength.

b) Every bar shall end in a 90° angle or equivalent anchorage method as close to the opposite face of the column as possible.

c) The anchorage length shall be calculated starting from a distance of 10Φ from the face of the column.

DL III Structures: When the beams are connected on opposite sides of the column, the maximum diameter of the longitudinal bars which continue through the column, shall not exceed 1/30 the depth of the column (parallel to the bar).

5.5.3 Columns: Anchorage of Longitudinal Reinforcement

The maximum diameter of the longitudinal bars of columns passing through a joint shall not exceed 1/25 of the depth of the beam. When plastic hinges are allowed in columns that value can be 1/30.

The anchorage of the bars of the column in the beam shall be made with a 90° hook or in a similar way, as close to the top face of the beam as possible. The direction of the horizontal part of the hook shall be towards the core of the joint.

When columns end at the top of the frame or at a joint with a foundation, the anchorage of the column bars shall be considered that it starts at a distance of 1/2 the depth of the beam or 10Φ, whichever is less, from the entrance of the bar into the joint.

5.5.4 Splicing of Longitudinal Reinforcement

Splices are not allowed within beam-column joints or in regions where plastic hinges are likely to form.

If it can be shown that a plastic hinge cannot be formed, then splices at the edge of the columns are allowed, if links are placed with spacing not greater than 6 diameters.

Links shall be placed along the length of the splice in beams and columns. The link spacing shall not exceed 10 diameters of the bar which is spliced.

DLIII Structures: In addition, the spacing shall not exceed 150mm.

Welding or mechanical connections in accordance with the concrete code, cl. 17.4, can be used, if alternating bars are spliced in each layer of longitudinal reinforcement and the spacing between splices of adjacent bars is 600mm or more, along the length of the frame member.

5.5.5 Anchorage and Connection of Transverse Reinforcement

The transverse reinforcement (links) shall be anchored at an angle of at least 135° around a longitudinal bar with minimum extension of the free end of 10 diameters. Alternatively, the ends of the links can be welded in such a way that they can develop the complete strength of the bar.

Transverse reinforcement shall not be connected by overlaps in concrete cover, in joints or in regions in which plastic region might develop. Connection by overlapping is allowed only for deformed (high bond) bars.

When the anchorage of spiral reinforcement ends at an angle of 135° around a longitudinal bar, the extension beyond the bend shall be at least 10 diameters of the spiral bar.

5.6 Foundations

5.6.1 Foundation Level

The foundations of a structure must be at the same horizontal level. When this is not practical, for example in particularly adverse topographical or geotechnical conditions or building arrangements, special measures shall be taken to ensure uniform performance of the structure during an earthquake.

5.6.2 Connection of Footings

In cases of foundations with isolated footings or strip footings in one direction the foundations shall be connected between them with tie-bars in two directions in order to avoid horizontal displacements.

The tie-bars shall connect the footings and not the columns, that is, they shall be at the level of the footings or they shall touch the top surface of the footings.

The tie-bars shall be at least 250mm wide and 500mm high. In single and two-story structures the height is allowed to be 400mm.

Every tie bar shall be in a position to withstand a tensile or compressive axial force at least equal to 1/10 of the largest vertical load of the footings it connects.

The minimum longitudinal reinforcement shall be 0.4% top and 0.4% bottom. The transverse reinforcement shall be links of 8 mm minimum diameter placed every 200mm. The anchorage requirements for beam reinforcement shall also apply for tie-bars.

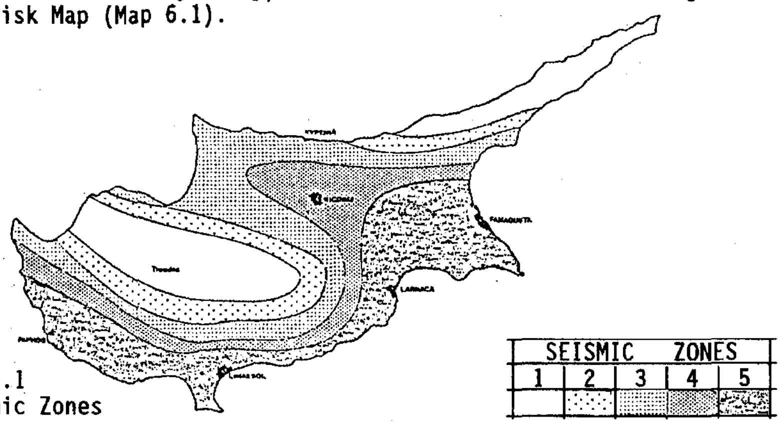
5.6.3 Increase in Allowable Stress

During the checks of the foundations, an increase of the allowable soil stress of 30% is allowed, when the seismic actions are taken in combination with the vertical loads. Reduction in the foundation dimensions derived from calculations for vertical loads only is not allowed under any circumstances.

6 SEISMIC ACTION

6.1 Regional Seismicity

The seismic activity in Cyprus is described in the following Seismic Risk Map (Map 6.1).



Map 6.1
Seismic Zones

For the purpose of structural design the most suitable parameter is the maximum value of ground acceleration Amax.

6.2 Seismic Zones

For code application, Cyprus is divided into 5 zones based on the expected seismic stresses. For every zone, calculation values for the maximum ground acceleration Amax, are given in table 6.2 as a fraction of g.

Zone	Amax
1,2,3	0.075
4	0.100
5	0.150

Table 6.2: Seismic Zones

6.3 Characteristics of Seismic Actions

Seismic actions result from the vibrations of the soil transmitted to the structures during the earthquakes.

For the purpose of this code the ground motion shall be adequately described by means of:

- the peak ground acceleration Amax, treated as a random variable of known distribution.

- the response spectrum for horizontal motion for firm soil conditions, normalised to Amax=1.

- the response spectrum for vertical motion, scaled to 2/3 of the corresponding horizontal motion response spectrum.

In sites where geological evidence indicates the possibility of "near field" type of shocks (for which the response spectrum concept is inadequate), or where there is extensive and deep soil layering (for which selective amplification can occur) the expected characteristics of ground motion shall be determined by special studies.

More simple geotechnical and morphological site conditions shall be accounted for by suitable modification of the basic spectrum relative to the area.

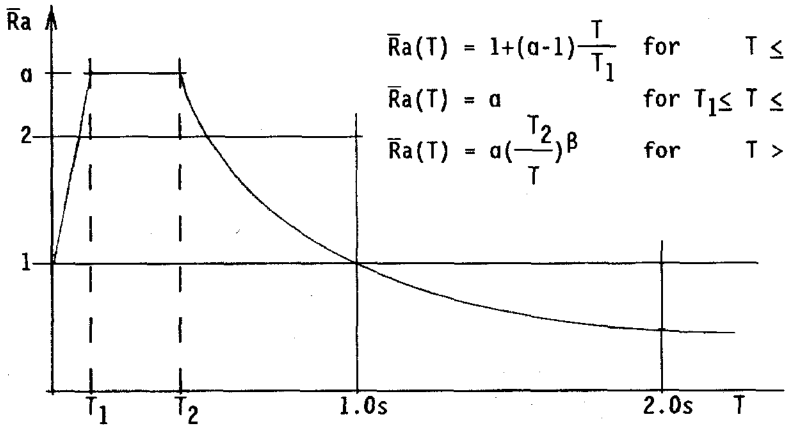
6.4 Design Seismic Action

The design seismic action is, by definition, the action that, when used in conjunction with other permanent and variable loads to design structures in accordance with the present provisions and with those of the Concrete Structures Code, satisfies the general requirements set forth in ch. 2 with the established level of reliability.

6.4.1 Normalised Elastic Response Spectrum

For the purpose of these recommendations, the shape of the "standard" (rocky or firm soil conditions) elastic response spectrum normalised to a unit peak ground acceleration shall be idealised as shown in fig. 6.4.1.

The spectrum is intended for a damping ratio of 5%. Spectral amplification shapes different from the proposed one can be adopted, conditioned to site-specific historical and/or geophysical evidence.



$$\bar{R}a(T) = 1+(\alpha-1)\frac{T}{T_1} \quad \text{for} \quad T \leq T_1$$

$$\bar{R}a(T) = \alpha \quad \text{for} \quad T_1 \leq T \leq T_2$$

$$\bar{R}a(T) = \alpha\left(\frac{T_2}{T}\right)^{\beta} \quad \text{for} \quad T > T_2$$

fig. 6.4.1 Normalised elastic response spectrum.

For a probability of non-exceedance 70-80%: a = 2.5, β = 1.0
When no more specific information is given: T_1 = 0.1s, T_2 = 0.4s

6.4.2 Site Effects

When more detailed knowledge on the effects of local soil conditions on the characteristics of ground motion arriving at the site from possibly different sources is not available, the procedure in cl. 6.4.2.1/2/3 shall be applied.

6.4.2.1 Soil Profile Types

The effects of site conditions on building response shall be established based on the soil profile types defined as follows:

SOIL PROFILE S1: Rock of any characteristic, either shale-like or crystalline (such material may be characterised by a shear wave velocity greater than 800 m/sec.); or stiff soil conditions where the soil depth is less than 60m and the soil types overlying rock are stable deposits of sands, gravels or stiffer clays.

SOIL PROFILE S2: Deep cohesionless or stiff clay, including sites where the soil depth exceeds 60m and the soil types overlying rock are stable deposits of sands, gravels or stiff clays.

SOIL PROFILE S3: Soft-to-medium stiff clays and sands, characterised by 10m or more of soft-to-medium stiff clay with or without intervening layers of sand and other cohesionless soils.

In locations where soil properties are not known in sufficient detail to determine the soil profile, or where the profile does not fit any of the three types, soil profile S2 shall be used.

Special in-situ and laboratory studies shall be carried out to investigate the possibilities and the conditions for the erection of buildings in cases where there is evidence of:

a) Dynamic instability by liquefaction of sand or other soils
b) Excessive settlement
c) Landslides or falling rocks and
d) Faulting

If not absolutely necessary, buildings on these kinds of soils should not be built.

6.4.2.2 Site Coefficient

The site coefficient S is used to modify the standard elastic response spectrum to account for the site conditions. Its values are given in Table 6.4.2.2.

Soil Profile Type	S1	S2	S3
Site Coefficient S	1.00	1.25	1.50

Table 6.4.2.2

6.4.3 Site-dependent Normalised Elastic Response Spectra

The site-dependent normalised elastic spectra for the 3 soil profiles are shown in fig. 6.4.3, their ordinates being defined as the smallest from the following expressions:

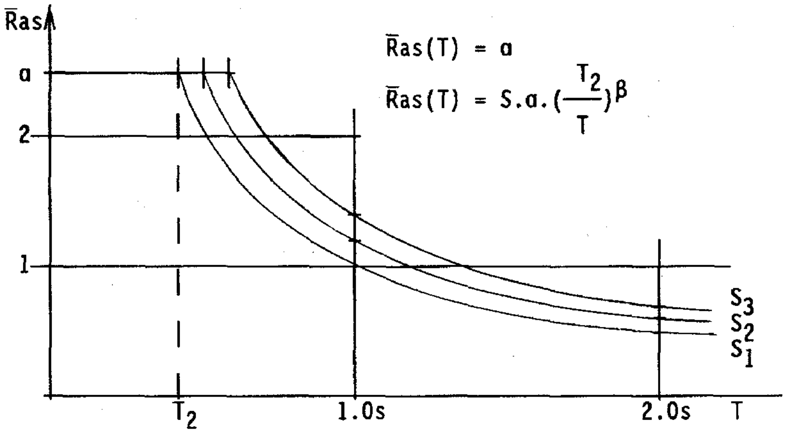


Fig. 6.4.3 Site-dependent Normalised Elastic Response Spectrum

In lack of specific site-related information T2, a and β can be assigned values as proposed in cl. 6.4.1.

Spectra for vertical motions may be determined with sufficient accuracy by multiplying the ordinates of the spectra for horizontal motions by a factor of 2/3.

6.4.4 Design Response Spectrum

The ordinates of design response spectrum are given by multiplying the ordinates of the site-dependent normalised response spectrum with the factors:

$$Ra = \bar{R}as \cdot I \cdot \frac{1}{K} \cdot Amax$$

where:

I = the importance factor defined in cl. 3.2.
K = the behaviour factor as given in Table 4.1.3.
Amax= the peak ground acceleration as given in Table 6.2.

SYMBOLS

CAPITAL LATIN

Amax	Peak ground acceleration (6.2)
A_c	Confines area measured to outside peripheral transverse reinforcement
A_g	Gross sectional area of concrete
A_s	Tension bar reinforcement
A'_s	Compression bar reinforcement
A_{s1}	Cross-sectional area of top joint reinforcement (4.3.1.3)
A_{s2}	Cross-sectional area of bottom joint reinforcement (4.3.1.3)
C_d	Design seismic coefficient (4.2.4.1)
C_g	Centre of mass (4.2.1.1)
C_k	Centre of stiffness (4.2.1.1)
E	Seismic Action (4.1.4.1)
F_{id}	Design lateral force on floor i (4.2.4.1)
G	Permanent loads (4.1.4)
I	Structure importance factor (3.2)
K	Structure behaviour factor (4.1.3)
M_d	Design moment (4.3.1.4.5)
$M_{u,d}$	Ultimate moment of a concrete section evaluated with factored values of concrete and steel strengths (4.3.1.4.5)
$M^+_{u,d}$	Ultimate moment of a concrete section evaluated with characteristic values of concrete and steel strengths
N_d	Design axial force under the most unfavourable load combination including the seismic action
Q	Variable loads (4.1.4)
R_d	Design strength (4.5.2)
S	Site coefficient (6.4.2.2)
S_d	Fundamental combination of load effects (4.1.4)
S_i	Site type index (6.4.2.1)
T	Fundamental period of building (4.2.4.1)
V_{cd}	Shear force carried by concrete in beam or column or sections (4.4.1.4, 4.3.1.3)
V_d	Design shear force
V_{jh}	Horizontal shear joint (4.3.1.3)
V_{jv}	Vertical shear joint (4.3.1.4)
V_{Rd}	Design shear force (4.4.1.4)
V_{Sd}	Design shear of action (4.4.1.4)
W_i	Total gravity load on floor i (4.2.4.1)

SMALL LATIN

a	plan dimension of the building in a direction orthogonal to that of the seismic action
b_w	web width of concrete section
h,b	height and width of beams, major and minor sides in columns
h′,b′	distance between bars located at the ends of sides h and b respectively
d	effective depth of cross-section
e	eccentricity
s_h	spacing of transverse reinforcement in beams, columns and walls
f_{ck}	characteristic concrete cylinder strength
f_{ctm}	mean tensile concrete strength
f_{yd}	design steel strength
f_{yk}	characteristic steel strength
h	height of floor
l_w	horizontal wall height
h_w	total height of wall
h_n	vertical distance between floors in walls

GREEK

α	spectral amplification factor
β	parameter of the elastic response spectrum
β	shear multiplication factor
γ_c	partial safety factor for concrete
γ_s	partial safety factor for steel
γ_i	load distribution factor
γ_n	additional partial safety factor for DL III structures
Δ	inter-storey drift under service load
Δ_{el}	elastic inter-storey drift under design seismic actions
Δ_{max}	maximum inter-storey drift under seismic actions
ζ	ratio between maximum and minimum shear force at the end of a beam
θ	deformability index
ξ	amplification factor for torsional effects
ρ	tensile reinforcement ratio
ρ'	compressive reinforcement ratio
ρ_s	volumetric ratio of transverse reinforcement
σ_{cm}	minimum mean compression stress on the column above the joint
τ_{cd}	shear stress contributed by the concrete
τ_d	maximum nominal design shear stress
τ_{jh}	nominal horizontal shear stress in joint
τ_{Rd}	design shear stress
ω	dynamic magnification factor

Other symbols are defined in the appropriate chapters.

Dated this 14th day of November, 1996.

By the Administrator's Command,
P.A. ROTHERAM,
Chief Officer,
Sovereign Base Areas.

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