CODE FOR THE SEISMIC DESIGN OF CONCRETE STRUCTURES

Compiled by

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P R E F A C E

The West African Building Code - Part III: "Loads" which deals with dead and imposed loads to be used in design was published in 1960, for use throughout Nigeria, Ghana, Sierra Leone and Gambia.

The Building and Road Research Institute published a Draft Ghana Building Code in 1977. Part 3 of the Draft Code covered Structural Loads and Procedures which had a subsection on Effects of Earthquakes.

The feedback from practising engineers has indicated a need for a Code dealing specifically with the Seismic Design of Reinforced Concrete Structures. The present publication is intended to satisfy this need.

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1.1 SCOPE AND FIELD OF APPLICATION

1.1.1 This Code sets down minimum design requirements to be met when dealing with seismic situations, i.e. situations in which the earthquake action is considered as a critical action in conjunction with other dead loads or live loads. It applies to:

(i) Reinforced and Prestressed concrete buildings for ordinary uses, having structural resisting systems belonging to one of three types defined below:

(a) Frame System: A system in which both vertical loads and lateral forces are resisted by space frames.

(b) Wall System: A system in which both vertical loads and lateral forces are resisted by vertical structural walls either single or coupled.

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

1.1.2 Other structural systems not included in 1.1.1 (e.g. inverted pendulum structures, flat slab systems etc.) can be designed subject to documented proof that they satisfy all the requirements of this Code with, at least, the same amount of reliability.

1.1.3 Buildings with special characteristics e.g. elements of lifeline systems or buildings involving high induced risk (e.g. chemical or nuclear facilities) are outside the scope of this Code.

1.2 GENERAL REQUIREMENTS

1.2.1 For the planning, design, and construction of structures in seismic regions, in addition to the general rules for non seismic regions, the following requirements apply:

(i) Structural Safety: It is required that the entire structure and all of 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, retain with adequate reliability, their integrity and a residual capacity after the seismic action has ceased.

(2) Serviceability: It is required that the building as a whole, including structural and non-structural elements, be protected with adequate reliability against the occurrence of damage and limitations of use as a consequence of the seismic action.

TABLE 1.2.1. SUMMARY OF REQUIREMENTS FOR SAFETY AND SERVICEABILITY

A. DESIGN CRITERIA: I

Limit-states verification	
Safety	**Serviceability**
- Stability	- Ultimate resistance of critical regions
- Control of collapse mehanism	- Ductility
- Ultimate resistance of critical regions	- Limit deformations
- Ductility	

B. DESIGN CRITERIA: II

Other Measures	
Global ductility	**Quality assurance**
- Quality of materials	- Correspondence between structural model adopted for analysis and the actual structure, considering all the elements, either structural or not, which could alter the intended behaviour. - Workmanship in the detailing especially in those areas indicated as critical by the designer (extremities of columns and beams, base zones of walls, lintels, etc.)

1.3 DESIGN CRITERIA

1.3.1 Reliability Differentiation. Structures shall be classified under the following reliability levels:

(1) Class I: buildings that are required to remain functional and to suffer reduced damages after a strong seismic attack (e.g. essential rescue facilities such as hospitals, fire and police stations, electricity stations, etc., buildings with likely large number of occupants such as schools audience or spectacle halls, etc.)

(2) Class II: buildings not included in 1.3.1(1).

(3) The different reliability levels proper to each Class shall be obtained by amplifying the design action with a factor I, called 'importance factor'.

1.3.2 Ductility Levels. Structural systems covered by the Code may be designed to possess different "ductility" levels according to the following classification:

(1) Ductility Level I (DLI) - is that proper to structures proportioned in accordance to B.S. 8110 (1985) with additional requirements on detailing contained in 1.5.

Ductility level I is associated with relatively large design lateral forces so that little inelastic response should occur even for large earthquakes. This makes ductility level I suitable for low rise buildings.

(2) Ductility Level II (DLII) - for this level 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.

(3) Ductility Level III (DLIII) - special procedures for the evaluation of 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. DLIII structures should be preferred whenever large uncertainties exist (e.g. local amplification effects of difficult evaluation etc.).

(4) 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 'behaviour factor' K (Section 1.4.1.3).

1.4 METHODS OF ASSESSMENT

1.4.1 BASIC DATA

1.4.1.1 Material Characteristics

1.4.1.1 (1) Concrete

Normal concrete grades shall satisfy the following requirements.

TABLE 1.4.1.1

Ductility Level	Minimum Grades
DL I	C20
DL II	C20
DL III	C 25

1.4.1.1 (2) Steel

(a) DLI and DLII Structures

The reinforcing steel is defined by its characteristic strength.

(b) DLIII Structures

The following additional requirements shall be satisfied

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

(ii) Steel grades with characteristic strengths higher than S400 (400 N/mm^2) shall not be used, unless it is demonstrated that the use of higher grades in special section arrangements does not affect unfavourably the ductility.

(iii) The actual yield stress shall not exceed its nominal values by more than 15%.

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

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

1.4.1.2 Material Safety Factor γ_m

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

Concrete γ_c = 1.5

Steel γ_s = 1.15

1.4.1.3 Structure Behaviour Factors

(1) The values of the bahaviour factor K, defining the intensity of the design action (Section 1.6.4.4) as a function of the structural type and of the selected ductility level, are given in Table 1.4.1.3.

TABLE 1.4.1.3. DESIGN BEHAVIOUR FACTORS (K)

Structural System	Ductility level I	Ductility level II	Ductility level III
Frame	2	3.5	5
Wall and Dual	2	3	4

(2 The values of K in Table 1.4.1.3 for wall and dual structures apply if, at least, 50% of the lateral force in both directions is resisted by coupled walls.

(3 If condition in 1.4.1.3(2) is not satisfied, the K values for wall and dual structures shall be reduced by a factor of 0.7.

(4 Ductility level I is permitted only for Class II structures

in areas of moderate seismicity.

(5) Class I structures to be built in high seismicity areas shall be preferably designed for ductility level III. If appropriate, K values relative to DL II could be used in this case.

1.4.1.4 Design Load Combination

The fundamental combination of load effects to be used for all limit-states veritification (Sect. 1.4.4.6), is

$$Sd = S(G + P + E + \sum \psi_i \cdot Q_{ik}) \quad \ldots\ldots \quad (1.4.1.4.1)$$

where G = all the permanent loads at their nominal value

P = the long-term prestressing force

E = the design seismic action as defined in Sect. 1.6.4.4.

Q_{ik} = fractile values of extreme distributions of all live loads whose duration of application is long enough for the probability of their joint occurrence with earthquake action to be considered.

ψ_i = factors required to change the fractile values Q_{ik} to the average values of Q_i in their instantaneous distribution (see Table 1.4.1.4).

S = site coefficient.

TABLE 1.4.1.4. COMBINATION FACTOR ψ_i FOR LIVE LOADS

Live loads from persons and equipment	0.3
Live loads from persons at places with likelihood of large number of occupants (halls)	0.5
Long term storage (warehouses, libraries)	0.9
Live loads on staircases and corridors	1.0

1. .2 STRUCTURAL ANALYSIS

1.· .2.1 Building Configuration

Allowable methods of structural analysis shall be different fo: buildings which according to the definition in 1.4.2.1(1) and 1.· .2.1(3), are classified as "regular" or "irregular".

(1. Regular buildings can be designed according to the simplified method of analysis (indicated as equivalent static analysis) described in 1.4.2.4 provided their height does not exceed 80m, and fundamental period is shorter than 2 secs.

(2: If conditions in 1.4.2.1(1) are not satisfied or if the building is of irregular type, the dynamic method in 1.4.2.5 shall be applied.

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

(a) PLAN CONFIGURATION

i) 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 per cent of the building external dimension.

ii) 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 'resistance radius', defined as the square root of the ratio of the storey torsional and translational stiffnesses.

(b VERTICAL CONFIGURATION

i The stiffness and mass properties are approximately uniform along the building height.

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

iii In the case of a gradual setback along its height, the setback at any floor is not greater than 10% of the plan dimension in the direction of the setback. This clause need not be complied with if the setback occurs within the lower 15% of the total height of the building.

γ_i = distribution factor, depending on the height of the floor, measured from the building base

W_i = total gravity load at floor i.

b) <u>Fundamental period:</u> The fundamental period of the building, which is required for the evaluation of Cd shall be calculated using the elastic properties of the structure by means of ordinary methods of mechanics, taking into account all the elements which can contribute to the building stiffness. For frame structures, an approximate expression of the fundamental period, based on analytical and experimental result is given by

$$T = n/12 \qquad \ldots\ldots \qquad (1.4.2.4.2)$$

where n is the number of storeys from the foundation level.

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

$$T = 1.8 \left(\frac{m.h}{E.1} \right)^{1/2} \qquad \ldots\ldots\ldots \qquad (1.4.2.4.3)$$

where m is the building mass per unit length

h is the height of the building from the foundation level

E.I is the flexural stiffness of the "equivalent" cantilever.

Of more general applicability is an expression derived from Rayleigh procedure:

$$T = 2\pi \left(\frac{1}{g} \sum Wi \cdot \delta_i^2 / \sum Fi\,\delta_i \right)^{1/2} \qquad \ldots\ldots \qquad (1.4.2.4.4)$$

where Fi (i = 1,...,N) is a set of forces linearly distributed along the height of the building

δi (i = 1,...,N) is the corresponding set of storey displacements.

In cases where the period T is not calculated from methods of mechanics Cd shall be taken as:

$$Cd = I.A.S.\alpha.$$

c) The distribution factor γi is given by the following expression

$$\gamma i = hi \frac{\sum W_i}{\sum W_i h_i}$$

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

(3) Torsional Effects

a) At each floor of the building the lateral design force shall be assumed to be displaced from its nominal location at the distances e1 and e2 as illustrated in figure 4.1, whichever is most unfavourable for every member to be checked.

b) The expression for e1 and e2 are

$$e1 = 0.5d + 0.05a$$

$$e2 = 0.05a$$

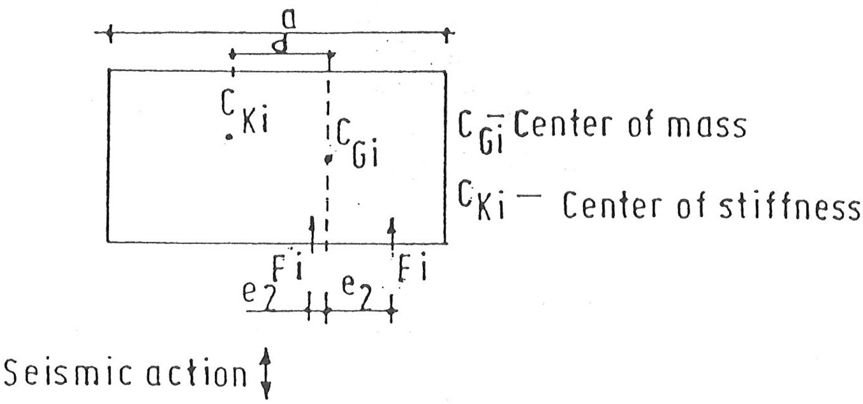

a
d
C Ki
C Gi
C Gi - Center of mass
C Ki - Center of stiffness
Fi
Fi
e2
e2
Seismic action

FIG 4.1

c) 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.

d) Symmetrical cases _ When complete symmetry of stiffness and mass about one axis parallel to the direction of the seismic excitation exists, torsion effects can be accounted for by means of the following simplified procedure

i) the lateral design force shall be applied at the floor centre of gravity, to be distributed to the various resisting elements as above;

ii) 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 seismic action.

(4) Second Order Effects

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

$$\Theta = \frac{W \cdot \Delta el \cdot K}{V \cdot h} \quad 0.10 \quad .... \quad (1.4.2.4.4)$$

where Θ = deformability index

V = seismic design shear force acting across the storey considered

Δel = elastic interstorey drift due to design actions

K = behaviour factor

h = floor height

W = total gravity load above the considered storey.

b) The deformability index Θ shall not in any case exceed the value 0.20

c) For $0.10 \leq \Theta \leq 0.20$ second order effects shall be accounted for by means of one of the statical methods indicated in BS 8110 (1985).

1.4.2.5 Modal Analysis Procedure

(1) Modelling

a) 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.

b) The condition stated in 1.4.2.5(1)(a) shall be assumed to occur when 1.4.2.1(3)(a)(ii) is satisfied.

c) When 1.4.2.1(3)(a)(ii) is not satisfied the model shall account for the non-planar motion of the structure.

(2) Modes

a) 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 secs, whichever is greater.

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

c) The mode considered shall be those with the greatest participation coefficients for the direction under consideration.

(3) Combination of Modal Responses

a) The response quantities (force, displacements, etc.) separately obtained for each mode under the effect of the design response spectrum given in Section 1.6.4.4 shall be combined to obtain their corresponding design values by taking the square root of the sum of the squares of modal values.

(4) Torsional Effects

a) 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 ±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.

b) When the building is analysed by means of planar models (Clause 1.4.2.5(1)) torsional effects can be accounted for by increasing the action effects due to the translational oscillations of the building by the factor ξ 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.

(5) Second-order Effects

Clause 1.4.2.4(4) applies.

1.4.3 DESIGN ACTIONS

(1) Structural elements shall be dimensioned and verified (see Section 1.4.4) for the design actions as defined in this Section.

(2) Design actions shall be derived from the actions obtained from the structural analysis in Section 1.4.2, appropriately modified as a function also of the selected design ductility level.

(3) DL I structures shall be dimensioned directly on the basis of the results of structural analysis, with a possible redistribution of action effects in accordance with BS.8110 (1985).

1.4.3.1 Ductility Level II: DL II

(1) Elements subject to bending ($N_d \leq 0.1\ A_g.f_{cd}$)

a) Bending moments: The design bending moments shall be those obtained from the linear analysis of the structure for the load combination given by equation 1.4.1.4.1. Redistribution according to BS 8110 (1985) is permitted.

b) Shear forces

(i) 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.

(ii) The end moments shall correspond to the design flexural strengths of the end sections based on actual reinforcement provided.

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

(iv) The algebraic ratio between the maximum and minimum values of shear shall be denoted by ζ. The value of ζ should not be taken less than minus one (Fig. 4.2).

(2) <u>Elements subject to bending and axial force (Nd).1 Ag.fcd)</u>

a) <u>Axial forces and bending moments</u>

(i) 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 1.4.1.4.1 eventually redistributed according to BS 8110 (1985).

(ii) For regular structures, three storeys and higher, to which equivalent static analysis has been applied, the column moment due to the lateral forces alone shall be multiplied by the dynamic magnification factor w as given by the following expressions:

Planar frames:

$$w = 0.6\ T_1 + 0.85 \quad (1.3 \leqslant w \leqslant 1.8$$

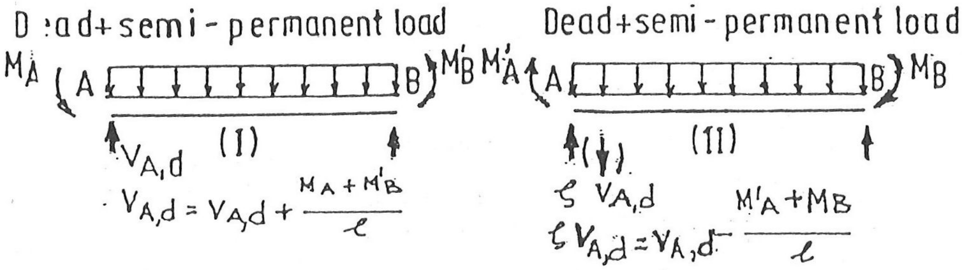
Spatial frames:

$$w = 0.5\ T_1 + 1.10 \quad (1.5 \leqslant w \leqslant 1.9$$

where T_1 is the fundamental period of the structure.

(iii) The values of the dynamic factor w given in 1.4.3.2(2)(ii) are applicable to storeys within the upper two-thirds of the building height. Below this level a linear variation of w should be assumed: the value at first floor level should be taken as 1.3 and 1.5 for planar and spatial frames respectively (Fig. 4.3).

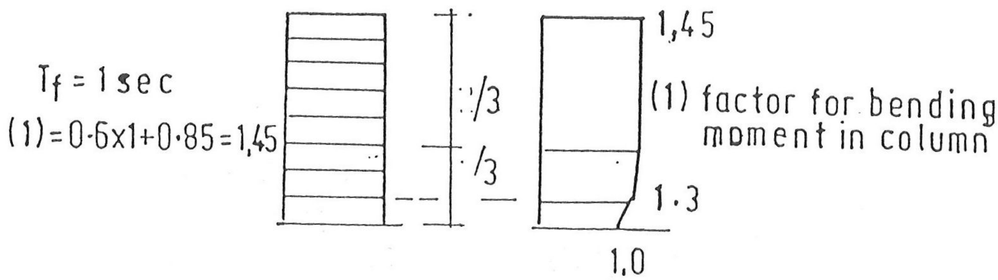
(iv) Column moments in addition shall satisfy the condition on the relative strength between columns and beams framing into a joint as specified in Clause 1.4.4.1(3).



$V_{A,d} = V_{A,d} + \frac{M_A + M'_B}{\ell}$

$\varsigma V_{A,d} = V_{A,d} - \frac{M'_A + M_B}{\ell}$

FIG 4·2



T_f = 1sec
(1) = 0·6x1+0·85 = 1,45
/3
/3
1,45
(1) factor for bending moment in column
1·3
1,0

FIG 4·3

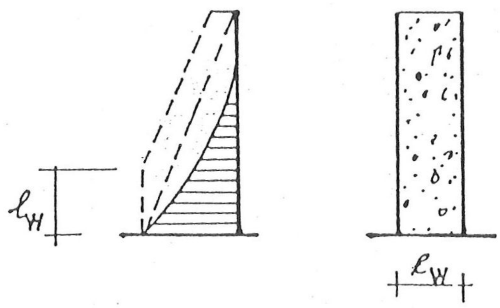


FIG 4·4 BENDING MOMENT DESIGN ENVELOPE

b) Shear forces.

(i) In evaluating the design shear forces from the condition of static equilibrium, the design end moments shall be the most adverse ones (i.e. those producing the maximum shear force), obtained from analysis of the structure under Code load combination (formula 1.4.1.4.1.) modified if appropriate by the dynamic magnification factor.

(3) Beam-Column Joints

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

(4) Structural Walls

a) The design actions shall be those obtained from a linear analysis of the building under the Code load combination (formula 1.4.1.4.1) modified as appropriate in accordance with Clauses 1.4.3.1(b) to 1.4.3.1(4)(e).

b) Redistribution

(i) 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%.

(ii) 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.

c) Bending Moment Design Envelope

(i) 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.4.4).

d) Earthquake Induced Axial Load in Coupled Walls

(i) 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.

(ii) The shear strength of the beams calculated in 1.4.3.2(4)(d) shall be further amplified by a factor of 1.25.

e) Dynamic Magnification Factors

(i) Where the equivalent static analysis is adopted, the shear forces in the walls shall be magnified by the dynamic amplification factor w as given by the expression below for buildings up to 5 storeys high:

$$w = 0.1\,N + 0.9$$

where N is the number of storeys.

(ii) For walls taller than five storeys, w, shall be linearly increased up to the value of w = 1.8 for N = 15.

1.4.3.2 Ductility Level III: DL III

(1) Elements Subject to Bending ($Nd \leq 0.1.Ag.fcd$)

a) Bending moments: The design bending moments shall be those obtained from the linear analysis of the structure for the load combination given by formula 1.4.1.4.1. Redistribution according to BS 8110 (1985) is permitted.

b) Shear forces:

(i) The design shear forces shall be determined from the conditions of equilibrium of the element subjected to the relevant transverse loads, if any, and to a rational adverse combination of the end moments, as specified in 1.4.3.3(1)(ii).

(ii) The end moments shall correspond to the design flexural strengths of the end sections based on actual reinforcement provided, multiplied by the factor.

$$\gamma_n = 1.25$$

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

(iv) 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 (Fig.4.5).

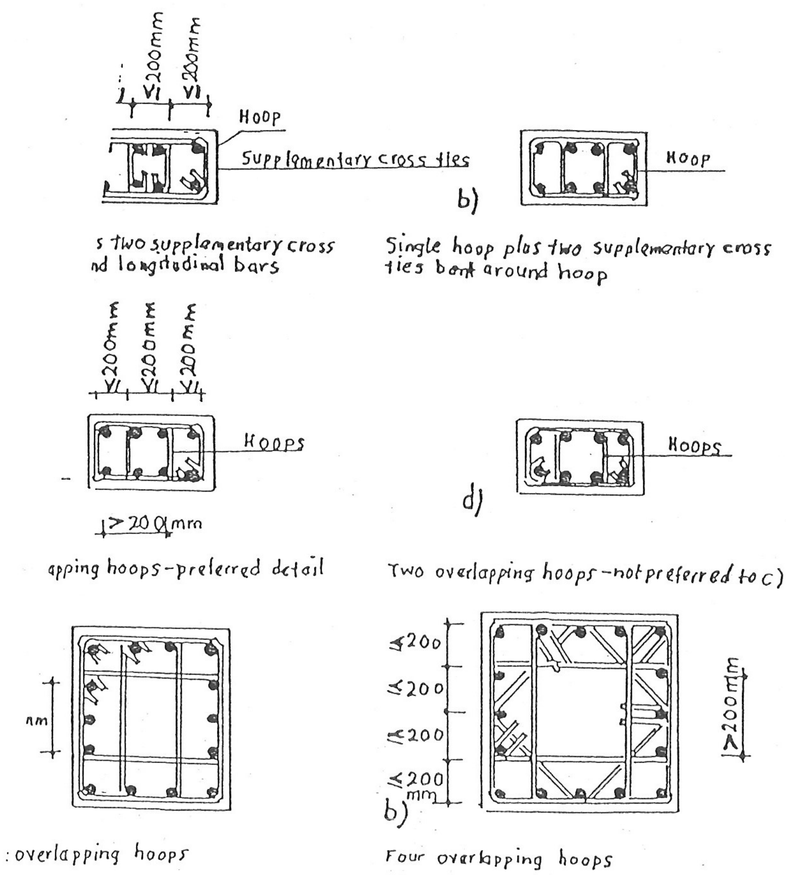


FIG 5-8 TYPICAL DETAILS USING OVERLAPPING HOOPS

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

a) Axial forces and bending moments

(i) 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 formula 1.4.1.4.1, eventually redistributed according to BS 8110 (1985).

(ii) For regular structures, three storeys and 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 factor, w, as given by the following expressions:

Planar frames:

$$w = 0.6T1 + 0.85 \qquad (1.3 \leq w \leq 1.8)$$

Spatial frames:

$$w = 0.5T1 + 1.10 \qquad (1.5 \leq w \leq 1.9)$$

where T1 is the fundamental period of the structures.

(iii) The values of the dynamic factor, w, as given in 1.4.3.3(2)(a)(ii) are applicable to storeys within the upper two-thirds of the building height. Below this level a linear variation of, w, should be assumed; the value at first floor level should be taken as 1.3 and 1.5 for planar and spatial frames respectively (Fig. 4.6).

(iv) Column moments shall satisfy the condition on the relative strength between columns and beams framing into a joint (see 1.4.4.1(3)).

b) Shear forces

(i) In evaluating the design shear forces from the conditions of static equilibrium the design end moments shall be the most adverse ones (i.e. those producing the maximum shear forces) as obtained from the analysis of the structure under Code load combination (formula 1.4.1.4.1).

(ii) The end moments as calculated above shall be further amplified, if appropriate, by the dynamic magnification factors, and by the γ_n factor:

$$\gamma_n = 1.10$$

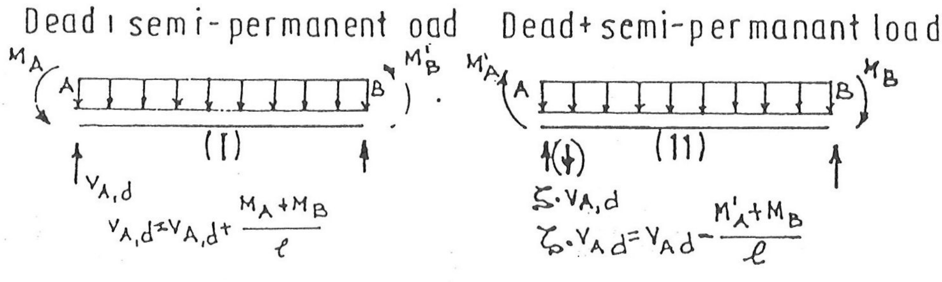
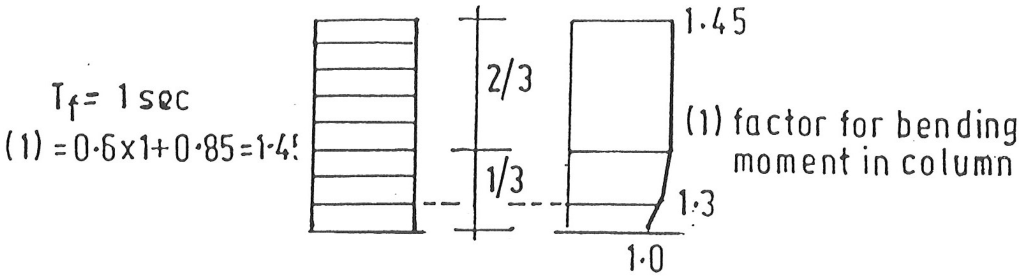

Dead ı semi-permanent oad
Dead+ semi-permanant load
M_A
M'_B
A
B
(I)
$V_{A,d}$
$V_{A,d} = V_{A,d} + \frac{M_A + M_B}{\ell}$
M'_A
M_B
(II)
$\zeta \cdot V_{A,d}$
$\zeta_0 \cdot V_{A}d = V_{A}d - \frac{M'_A + M_B}{\ell}$

FIG 4.5



T_f = 1 sec
(1) = 0·6x1+0·85=1·45
2/3
1/3
1.45
(1) factor for bending moment in column
1·3
1·0

FIG 4·6

(3) <u>Beam-Column Joints</u>

a) The design actions shall be those induced in the joint when the design ultimate moments of the beam or beam : multiplied by a factor γ_n equal to 1.25 are developed, exc·pt in cases when hinges are permitted to form in the columns (see Clause 1.4.4.1(3)). The axial force in the column sh: ll be the minimum corresponding to the design seismic acti ns.

b) Action effects, namely, bending moment and shea: of columns as well as horizontal, Vjh, and vertical, Vjv, shear forces through the joint core shall be evaluated from a rational analysis taking into account the effect of all fc rces acting on the equilibrium.

(i) The horizontal shear force Vjh across a typical interior joint with conventionally reinforc ed concrete members without prestressing may be calculat ed from

$$V_{jh} = \gamma_n \, (A_{s1} + A_{s2}) \, f_{yd} - V_{col}$$

where $\gamma_n = 1.25$

Similar expression may be obtained for exter al joints. A conservative value is given by

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

when $\frac{h_c}{l_1} \geqslant 0.08$ and $\frac{l_2}{l_1} \geqslant 0.7$

(ii) The value of the column shear with bear ultimate moments M1, M2 may be estimated using

$$V_{col} = 2\left(\frac{l_1}{l_{1n}} . M_1 + \frac{l_2}{l_{2n}} . M_2\right) / (l_c + l'_c)$$

with l_1, l_2 = centre to centre span of adja ent beams

l_{1n}, l_{2n} = clear spans of adjacent beams

l_c, l'_c = centre to centre upper and lo er column heights (Fig. 4 7).

(iii) The vertical shear force may be approx mated as follows:

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

c) When two non conplanar frames have common joints, veritication of these joints may be considered in each direction separately.

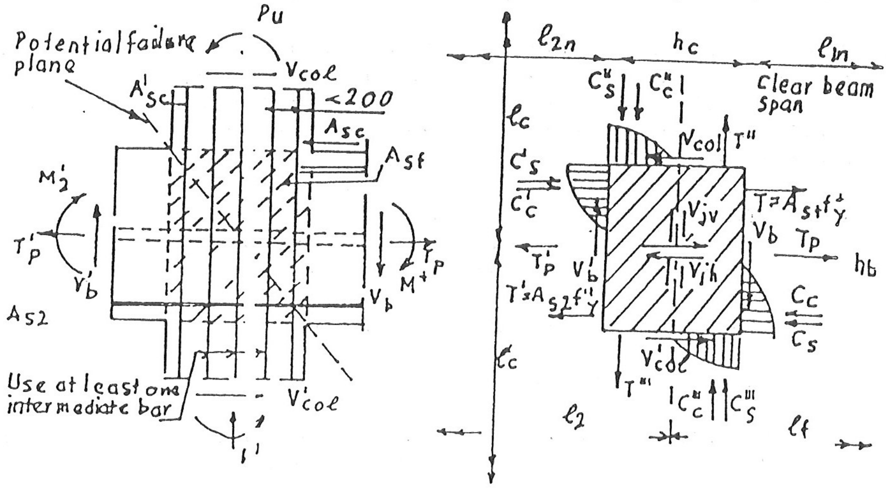

Pu
Potential failure plane
Vcol
A'sc
<200
Asc
Ast
M'2
T'p
V'b
Tp
M+
Vb
As2
Use at least one intermediate bar
V'col
ℓ2n
hc
ℓ1n
Clear beam span
C''s
C''c
ℓc
Vcol
T''
C's
C'c
T=Ast f'y
Vjv
Vb
Tp
hb
T'p
V'b
Vjh
T'=As2 f''y
Cc
Cs
V'col
T'''
C''c
C''s
ℓ2
ℓ1

FIG 4·7

(4) Structural Walls

a) The design actions shall be those obtained from a linear analysis of the building under the Code load combination (formula 1.4.1.4.1) modified as appropriate in accordance with Clauses 1.4.3.3(4)(b)-(f).

b) Redistribution

(i) 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%.

(ii) 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.

c) Bending Moment Design Envelope

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.4.8).

d) Earthquake induced Axial Load in Coupled Walls

(i) 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 which should be calculated from the 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.

e) Dynamic Magnification Factor

Where the equivalent static analysis has been adopted, the shear forces in the walls shall be increased by the dynamic amplification factors w as given by the expression below for buildings up to 5 storeys high:

$$w = 0.1N + 0.9$$

where N is the number of storeys.

For walls taller than 5 storeys, w shall be linearly increased up to the value of $w = 1.8$ for $N = 15$.

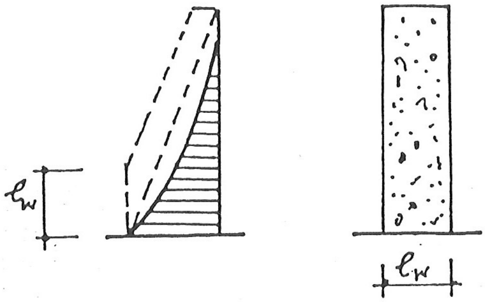


FIG 4·8 BENDING MOMENT DESIGN ENVELOPE

f) <u>Shear Forces</u>

(i) The design shear forces in walls shall be compatible with the actual flexural strength that can possibly be developed at the base of the walls.

(ii) The design shear forces shall be obtained by multiplying the shear forces due to the Code loading by the following factor,

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

where M_d is the design moment obtained from the analysis, and M 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.

(iii) The factor γ_n need not be taken greater than 4.

the axial force) shall not be less than the sum of th absolute values of the design ultimate moments of the beam framing into that joint (Fig. 4.9).

b) DL III Structure

(i) For columns of DL III structures, the design bendin moments shall account for the possible increase i strength of the beams framing into the joint.

(ii) Unless otherwise justified, the global strengtl increase can be assumed as:

$$\gamma n = 1.15$$

and is applicable to all storeys, including the columr bases in the first storey.

(iii) Development of plastic hinges in columns and of columns hinge mechanism (i.e. exemption from the prescription on beam-column ratio) is permitted in the following cases:-

for frames having four or 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 multi-storey building.

(4) Resistance to Shear

a) Contribution of Concrete

The magnitude of the term Vcd, expressing design resistance contributed by concrete shall be taken as follows:

(i) When $Nd \leq 0.1$ Ag.fcd, Vcd shall be assumed to be zero in all regions where stirrup-ties are required in accordance with Clause 1.5.1.3 (with the exception of case C).

(ii) When $Nd > 0.1$.Ag.fcd, Vcd shall be computed by the expression

$$Vcd = 2 \,.\, \tau_{Rd} .\, bw.d.\beta_1 \,, \quad \ldots\ldots\ldots 1.4.4.1(4)$$

where the values of τ_{Rd} are given in BS110 (1985) as functions of concrete grades and β_1 is given by

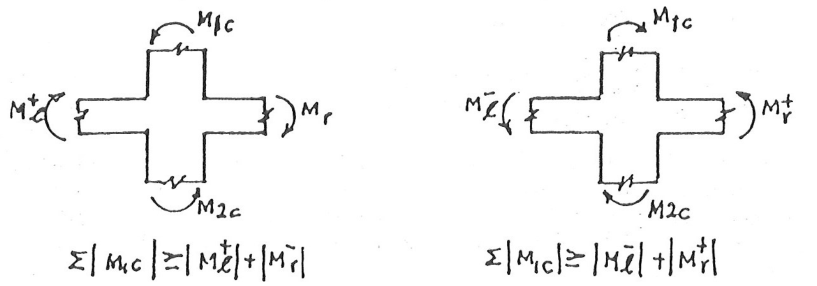


FIG 4·9 BEAM - COLUMN STRENGTH RATIO

$$\beta_1 = 1 + \frac{Mo}{Md} \leq 2$$

where Mo and Md are decompression moment and design moment respectively. Mo shall be computed by the expression

$$Mo = N.h/2$$

b) Transverse Reinforcement

(i) $Nd \leq 0.1.Ag.fcd$. Two cases shall be considered, depending on the value of the ratio

(I) $\zeta \geq 0$. The resistance to shear provided by the reinforcement: Vwd shall be assessed on the basis of the truss model. Hence

$$Vwd = 0.9.h.\frac{Asw.fyk}{s \cdot \gamma s}$$

where h is the effective depth, Asw the individual cross-sectional area of a mat of transverse reinforcement and s the spacing of the mats of transverse reinforcement measured parallel to the axis of the beam.

(II) $\zeta < 0$. For Vsd not exceeding the limit value V_{Rd1}, where

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

same requirement as in (I) above applies.

For Vsd exceeding the limit value V_{Rd2} where:

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

the entire shear shall be resisted by diagonal reinforcement across the web, that is, steel bars inclined in two directions shall equilibrate with their compression and tension components the shear forces of opposite sign Vsd and ζ.Vsd 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) $N_d \geq 0.1.A_g.f_{cd}$. The resistance to shear shall be checked as for case (i)(I).

1.4.4.2 Beam-Column Joints (DL III ONLY)

(1) Horizontal Joint Shear

a) Nominal Horizontal Shear Stress

(i) The nominal horizontal shear stress in the joint, as shown by the following expression

$$\tau_{jh} = \frac{V_{jh}}{b_j h_c} \quad \ldots\ldots\ldots\ldots \quad 1.4.4.2(1)$$

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

b) The effective joint width bj shall be taken as

(i) when $b_c > b_w$

either $b_j = b_c$

or $b_j = b_w + 0.5.h_c$ whichever is smaller (Fig. 4.10)

(ii) when $b_c < b_w$

either $b_j = b_w$

or $b_j = b_c + 0.5.h_c$ whichever is smaller

c) Mechanisms of Joint Core Shear Resistance

Two mechanisms for transmission of horizontal shear force Vjh through the joint core are in general possible:

(i) A diagonal concrete strut across the compressed joint corners carrying a shear force Vch;

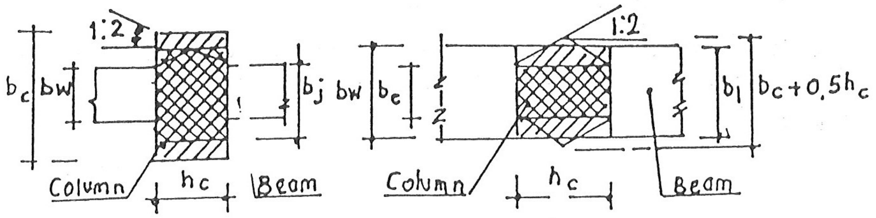
(ii) A truss mechanism consisting of horizontal stirrups and diagnonal concrete struts carrying a shear force Vsh where

$$V_{sh} + V_{ch} = V_{jh} \quad \ldots\ldots\ldots \quad 1.4.4.2(2).1$$

(Fig. 4.11)

d) Shear Force carried by Concrete

The value of shear force carried by the concrete strut Vch shall be assumed zero except for the following:



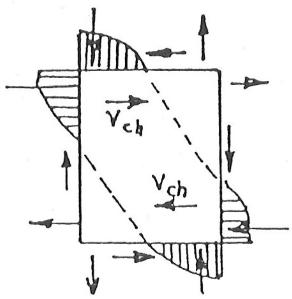
$b_j = b_c$
or $b_j = b_w + 0.5h_c$

(a) Whichever is smalle

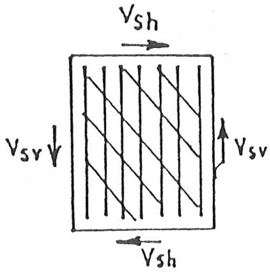
$b_j = b_w$
or $b_j = b_c + 0.5h_c$

(b) whichever is smaller

FIG 4.10 EFFECTIVE JOINT WIDTH AND EFFECTIVE JOINT AREAS



(a) concrete strut



(b) steel - concrete truss

FIG 4·11

(i) When minimum average compression stress σ_{cm} on the gross concrete area of the column above the joint including prestress where applicable, exceeds 0.1.fck

$$Vch = (2.\tau_{Rd} . \sqrt{\sigma_{cm} - 0.1.f_{ck}}).bj.hc \quad ... \quad (1.4.4.2.3.1)$$

(ii) When beams are prestressed through the joint:

$$Vch = 0.7\ Pcs \quad .... \quad (1.4.4.2.3.2)$$

where Pcs is the permanent force in the prestressing steel that is located within the central third of the beam depth.

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

$$Vch = \frac{As^{1}}{As} . \frac{Vjh}{2} \left(1 + \frac{Nd}{0.4.Ag.fck}\right) \quad ... \quad (1.4.4.2.3.3)$$

where the ratio As'/As of the compression to the tension longitudinal beam reinforcement shall not be taken larger than 1.0. When the axial column load results in tensile stresses over the gross concrete area exceeding 0.2fck the entire joint shear shall be resisted by reinforcement.

For axial tension smaller than this limit, the value of Vch may be linearly interpolated between zero and the values given by equation (1.4.4.2.3.3) with Nd taken as zero.

(iv) When parts Aa1 and Aa2 of the tensile reinforcement As1 and As2 of the adjacent beams are bent vertically and achored in the tensile face of the column

$$Vch = Aa.\frac{fyk}{\gamma s} \quad ...... \quad (1.4.4.2.3.4)$$

where Aa is the smaller of Aa1 and Aa2.

The values of Vch obtained from equations (1.4.4.3.1/2/4) may be added where applicable.

(Fig. 4.12)

e Horizontal Shear Reinforcement

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

$$Vsh = Vjh - Vch$$

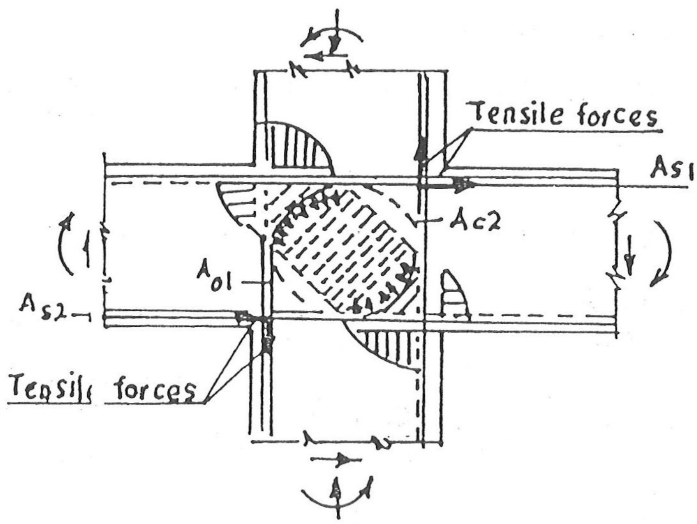


FIG 4-12

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

$$A_{jh} = \frac{V_{sh}}{f_{yk}/\gamma_s} \quad \ldots\ldots\ldots \quad (1.4.4.2.4.1)$$

(ii) Horizontal sets of stirrup ties shall be distributed as uniformly as practicable between layers of the top and bottom beam reinforcement.

(2) Vertical Joint Shear

a) Vertical Joint Reinforcement

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

$$V_{sv} = V_{jv} - V_{cv}$$

where the value Vcv shall be determined from:

$$V_{cv} = \frac{A_{sc}'}{A_{sc}} \cdot V_{jv} \left(0.6 + \frac{N_d}{A_g . f_{ck}}\right) \ldots\ldots (1.4.4.2.2.1)$$

where Asc' and Asc are the areas of longitudinal compression and tension reinforcement in columns respectively, with the following exceptions:

(i) Where axial load results in tensile stresses over the column section, the value of Vcv shall be linearly interpolated between the value given by equation (1.4.4.2.2.1) with Nd taken as zero when the axial tension over the gross concrete area is 0.2 fck;

(ii) Where plastic hinges are expected to form in the column above or below a joint, as part of the primary seismic energy dissipating mechanism, Vcv shall be assumed to be zero for any value of the axial load on the column.

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

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

c) 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 vertical bars, placed in the column and adequately anchored to transmit the required tensile forces within the joint.

d) 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.

(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 centre lines of the two members, the effective joint width bj shall not be taken larger than:

$$0.5\ (bw + bc + 0.5\ hc) - e$$

1.4.4.3 Structural Walls

(1) General

The design strengths of walls and coupling beams shall be evaluated as for linear elements (Section 1.4.4.1) except as modified by provisions in Section 1.4.4.3

(2) Resistance to Shear

a) Maximum Allowable Shear Stress

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

$$\tau_d = V_d / A_g$$

where Vd is the design force computed in accordance with Clauses 1.4.3.1(4) or 1.4.3.2(4) shall not exceed the following limit:

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

b) Contribution of Concrete to Shear Strength

(i) In the potential plastic zone, as defined in Clause 1.5.5.3(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.fcd. In which case the shear contributed by concrete shall be computed by:

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

with the values of τ_{Rd} given in BS8110(1985) and β_1 given in Clause 1.4.4.1.4(1).

(ii) Outside the potential hinge zone, and when the average compressive stress is less than 0.1.fcd, the shear stress contributed by concrete shall be taken as

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

while in case the average stress is greater than 0.1.fcd.

$$\tau_{cd} = 2.5.\tau_{Rd}.\beta_1$$

c) Web Reinforcement

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

$$\rho_v = \frac{A_h}{b.S_v} = \frac{\tau_d - \tau_{cd}}{f_{yd}}$$

while the vertical reinforcement ratio shall be:

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

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

(c) Coupling Beams

a) Symmetrical flexural reinforcement ($\rho = \rho'$) shall be adopted in case of usual arrangement.

b) Design for flexure and shear shall be carried out as for ordinary beams unless the following limits are exceeded

$$\tau_d \geqslant 6.\tau_{Rd}$$

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

(ρ = longitudinal reinforcement ratio, top or bottom) in which case all flexural and shear actions shall be resisted by diagnonal reinforcement in both directions.

1.4.4.4 Diaphragms and Stair Slabs

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

(2) When it is shown that the forces to be transmitted do not introduce yielding in a diaphragm provisions indicated in BS8110 (1985) apply.

(3) If yielding in a diaphragm cannot be avoided provisions for ductile structural walls and in particular Clause 1.5.5.3(1) for confining reinforcement in boundary zones shall be applied.

(4) 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.

(5) Stair slabs (inclined) shall be appropriately designed so that relative inter-storey displacements are compatible with axial and flexural rigidity of stair slabs.

(Fig. 4.13)

1.4.4.5 Prestressed Concrete Members

a) This Section covers the design of prestressed or partially prestressed concrete members of frame structure.

b) All the provisions relative to non-prestressed elements, as given in the relevant chapters of the document, apply, except as otherwise indicated in this Section.

(1) General

a) Unless special proof of adequacy is given, tendons in frame members shall be grouted, exceptions being allowed in the following cases:

(i) prestressed concrete floor or roof systems not contributing to the bending strength of the frame;

(ii) partially prestressed beams where the ordinary reinforcement provides at least 80% of the required strength, and tendons passing through the joint core are located in the middle third of the beam depth, at the face of the column.

b) Anchorages of post-tensioned tendons shall not be placed within beam column joint, cores, and shall be as far as practicable from potential hinge regions.

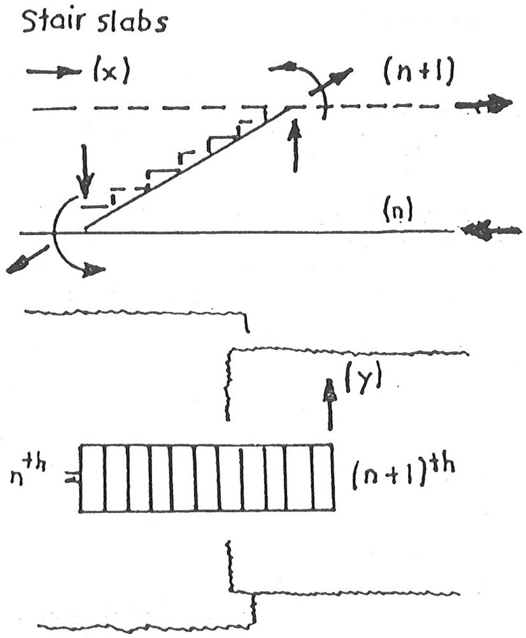


FIG 4·13

(2) Flexural Members

a) The total prestressed and non-prestressed steel reinforcement in potential hinge regions (see Clause 1.5.1.3) shall be such as to satisfy the following condition

$$x \leqslant 0.2h$$

where x is the depth of equivalent rectangular concrete compressive stress block at the flexural strength of the member and h is the overall depth of the member.

(i) The limit can be raised to: $x \leqslant 0.3h$ if special transverse confinement is provided in accordance with Clause 1.5.2.3(3) (critical regions of DLIII structures).

b) The flexural strength of a section shall exceeed its cracking strength by at least 1.25 times. The cracking moment shall be evaluated allowing for a possible reduction of prestressing force.

(3) Columns

Prestressed columns shall be provided with special transverse confinement in critical region in accordance with Clause 1.5.2.3(3).

(4) Beam-Column Joints

A convenient fraction of the tendons passing through joint cores shall be placed near both top and bottom of the beam framing into the joint, except in the case of Clause 1.4.4.5(1)(a)(ii).

1.4.4.6 Verifications

(1) Collapse Verification

a) For the purpose of the present Code, a structure shall be deemed to satisfy the safety requirement against collapse if the following conditions are met:

(i) the strength and stability verifications are satisfied;

(ii) the elements are dimensioned and detailed in accordance with the rules given in Sections 1.4 and 1.5, relative to the appropriate structural type and intended ductility level.

(2) Strength Verification

The following condition must be satisfied for every element:

$$S_d \leq R_d \quad \ldots\ldots\ldots \quad (1.4.4.6.2.1)$$

where

Sd is the design load effect on the element considered evaluated according to Section 1.4.3.

Rd is the design strength of the same element, evaluated according to Section 1.4.4.

(3) Stability Verification

The stability verification shall be considered satisfied if:

(i) the deformability index Θ (formula 1.4.2.4.3.1) is less than 0.1

(ii) for $0.1 \leq \Theta \leq 0.2$, the second order effects are calculated by means of one of the statical methods of BS 8110 (1985) and added to the design forces.

(iii) the stability verification cannot be satisfied if

$$\Theta > 0.20$$

(4) Serviceability Verification

a) The elastic drift, el resulting from the application of the horizontal forces specified in 1.4.2.4 or from the dynamic procedure as in 1.4.2.5, shall at any storey satisfy the condition:

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

where h is the clear height of the floor.

b) For Class II buildings, the indicated limits may be increased 50% if it can be demonstrated that the finishes adopted are not brittle-type and can accommodate without significant damage to those limits.

c) When the limits in (a) and (b) are ececeded, separation of the non-structural elements is required, of an amount adequate for permitting an interstorey drift equal at least to:

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

to take place without restraint.

d) In no case shall interstorey drift el exceed the limit:

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

(5) Max: mum Expected Displacements

The maximum expected displacements of the building shall be obtained by multiplying the di acements produced by the system of horizontal forces specified in 1.4.2.4 or those obtained from dynamic analysis as in 1.4.2.5 by the appropriate values of the behaviour factor K.

1.5 DETAILING, EXECUTION, USE

Wher no explicit distinction is made, the provisions in this Section apply to both DL II and DL III structures. Provisions applicable to DL I structures are always explicitly stated.

1.5.1 Elements Subject to Bending ($Nd \leq 0.1.Ag.fcd$)

1.5.1.1 Geometrical Constraints

DL II and DL III Structures

Unless special proofs for exemption are given, the following dimensional limitatins shall be satisfied:

(i) The width shall not be less than 200mm or more than the width of the supporting column, plus lengths, on each side of this member not exceeding one fourth of the depth of the column cross section.

(ii) The ratio b/h shall not be less than 0.25.

(iii) The ratio l/h shall not be less than 4 (This requirement does not apply to coupling beams in wall structures, Clause 1.4.4.3(3). (Fig. 5.1).

(iv) The eccentricity of any beam relative to the columns into which it frames as measured by the distance between geometrical centre lines of the two members, shall not exceed 1/4.bc (Fig. 5.2).

1.5.1.2 Longitudinal Reinforcement

(1) DL I and DL III Structures

a) At any section of the member the tensile reinforcement ratio for top or the bottom reinforcement shall not be less than:

$$\rho_{min} = \frac{1.4}{fyk} \quad \text{(fyk in MPa) .....(1.5.1.2.1)}$$

nor greater than:

$$\rho_{max} = \frac{7}{f_{yk}} \qquad \ldots\ldots\ldots \quad (1.5.1.2.2)$$

with ρ min and ρ max referred to the gross concrete area Ag.

b) At least two 12mm bars shall be provided both top and bottom throughout the length of the members.

c) Within any potential plastic hinge region, the compression reinforcement ratio ρ' shall not be less than one half of the tension reinforcement ratio at the same section.

d) At least one quarter of the larger of the top reinforcement required at either end of the member shall be continued throughout its length.

e) In T and L beams built integrally with slabs, the effective reinforcement to be considered near column faces in addition to all longitudinal bars placed within the web width of beam, shall be as follows:

(i) At interior columns when a transverse beam of similar dimensions frames into the column, all reinforcement within that part of the slab which extends a distance 4 times the slab thickness from each side of the column (Fig. 5.3a).

(ii) At interior columns where no transverse beam exists, all reinforcement within that part of the slab which extends a distance of 2.5 times the thickness of the slab from each side of the column (Fig.5.3b).

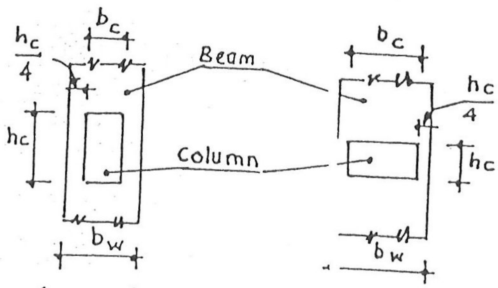
(iii) At exterior columns where a transverse beam of similar dimensions frames into the column and where the beam reinforcement is to be anchored all reinforcement within that part of the slab which extends a distance of twice the slab thickness from each side of the column (Fig. 5.3c).

(iv) At exterior columns where no transverse beam exists, all reinforcment within the width of the column (Fig. 5.3d).

(v) In all cases, at least 75% of the reinforcement in each face providing the required flexural capacity, must pass through or be anchored in the column core.

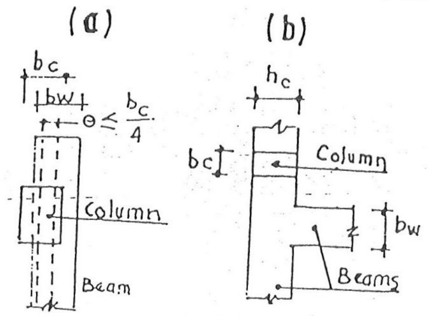
(2) <u>DL I Structures</u>

Clause 1.5.1.2(1)(a) only needs to be satisfied.



b_w maximum $\leq b_c + h_c/2 ; 2b_c$

FIG 5·1



(a) Max. eccentricity of beams
(b) Example of bad structural layout.
For this and analogous cases, if they can not be avoided special detailing shall be provided to ensure continuity with ductile behavior

FIG 5·2

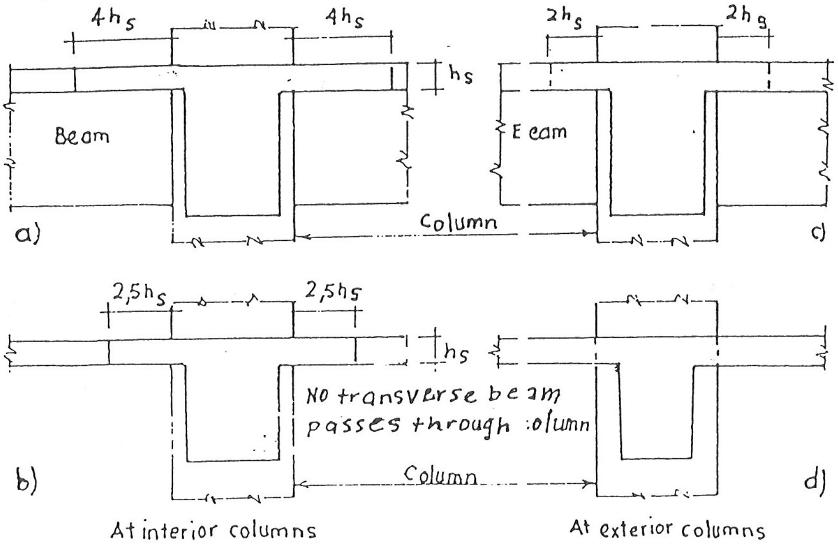


FIG 5·3 AREAS OF EFFECTIVE REINFORCEMET

1.5.1.3 Minimum Transverse Reinforcement

(1) Transverse reinforcement as specified in this section shall be provided unless larger amount is required to resist shear (Section 1.4.4.1(4). Portions of the beams to be considered as 'critical' regions are:

(a) Twice the member depth, measured from the face of the supporting column, or beam, towards midspan at both ends of the beam.

(b) Twice the member depth on both sides of a section where yielding may occur.

(c) Wherever compression reinforcement is required.

(2) DL II Structures

In the critical regions as defined in 1.5.1.3(1), stirrup-ties of not less than 6mm diameter shall be provided, with maximum spacings not exceeding the smaller of:

(a) h/4
(b) eight times the minimum diameter of the longitudinal bars.
(c) 24 times the diameter of the hoop bars
(d) 200m

The first hoop shall be located not more distant than 50mm from the face of the supporting member.

(3) DL III Structures

In the critical regions as defined in 1.5.1.3(1) stirrup-ties of not less than 6mm diameter shall be provided, with maximum spacings not exceeding the smaller of:

(a) h/4
(b) six times the diameter of the longitudinal bars
(c) 150mm

The minimum area of one leg of the transverse reinforcement shall be:

$$A_{s,min} = \frac{\sum A_b . f_{yk}}{16 f_{ykt}} . \frac{S}{100} \quad \ldots\ldots\ldots \quad (1.5.1.3.1)$$

$\sum A_b$ = Sum of the areas of longitudinal bars at the section considered to be restrained by the transverse leg

f_{yk} = yield strength of longitudinal bars

fykt = yield strength of stirrups

S = spacing of stirrups in mm

The first hoop shall be located not more than 50mm from the face of the supporting member (Fig.5.4).

1.5.2 Elements Subject to Bending and Axial Force ($Nd > 0.1.Ag.fcd$)

Observations of damages produced by earthquakes frequently show that corner columns are more vulnerable than interior ones, due to unanticipated torsional effects. It is therefore recommended that corner columns be subjected to particular care on detailing (e.g. by adopting DL III requirements) or even be made somewhat stronger than required by analysis.

1.5.2.1 Geometrical Constraints

(1) DL II Structures

(a) The minimum cross-section dimension shall not be less than 250mm.

(b) The ratio 1/b shall not exceed 25.

(2) DL III Structures

(a) The minimum cross-section dimension shall not be less than 300mm.

(b) The ratio 1/b shall not exceed the value of 16 for columns having moments of opposite sign at the two extremeties; 10 for cantilever columns.

1.5.2.2 Longitudinal Reinforcement

a) The reinforcement ratio shall not be less than 0.01 nor larger than 0.06 including the region of lap splices.

b) For S400 steel, the reinforcement ratio outside the splices shall not be greater than 0.045.

c) The bars shall not be spaced further apart between centres than 200mm.

DL I Structures: The provisions above must be satisfied also by DL I structures.

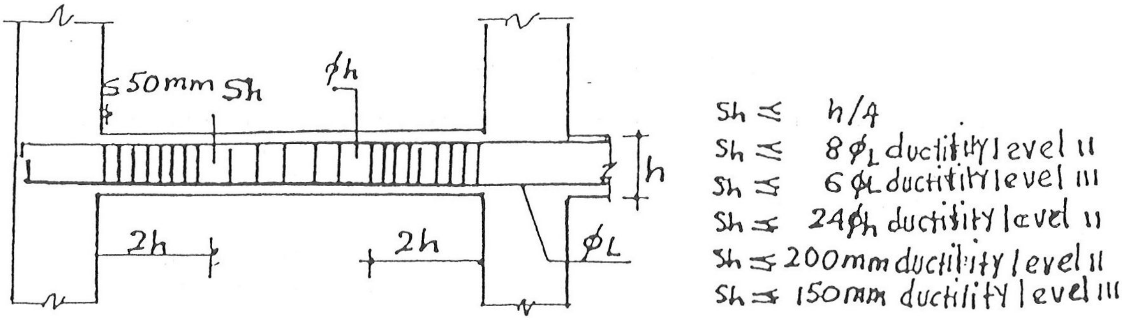


FIG 5-4 REGIONS AND SPACING OF TRANSVERSE REINFORCEMENT

1.5.2.3 Transverse Reinforcement

a) A basic amount of reinforcement shall be provided all over the height of the column, while special reinforcement shall be placed in the column critical regions defined in the following Clause 1.5.2.3(1).

b) The amount of reinforcement required by 1.5.2.3 shall be provided unless a larger amount is required to resist shear according to Clause 1.4.4.1(4).

1.5.2.3(1) Column Critical Regions

a) For usual cases, critical regions are considered to be the regions at each end of a column above and below connections over a length from the faces of the connection of not less than the larger of:

(i) the longer column cross-section dimension in the case of a rectangular cross-section, or the diameter of the section in case of a circular column

(ii) one-sixth of the clear height of the column

(iii) 450mm

b) When a masonry infill wall is in contact with one or both of the two opposite sides of a column, over the whole height or part of it, the entire column height shall be considered as a critical region (Fig. 5.5a).

c) In case of columns with part of their height restrained due to a connection with a wall, the free part of the column shall be considered as a critical region (Fig. 5.5b).

1.5.2.3(2) DL II Structures

Critical regions

a) Special transverse reinforcement having a minimum diameter of 8mm in the form of spiral or hoop reinforcement shall be provided.

b) Cross ties to restrain longitudinal bars not directly held by hoops shall be used in accordance with BS 8110 (1985).

c) The maximum spacing between spirals or hoops shall not exceed the smaller of:

(i) eight times the minimum diameter of the longitudinal bars

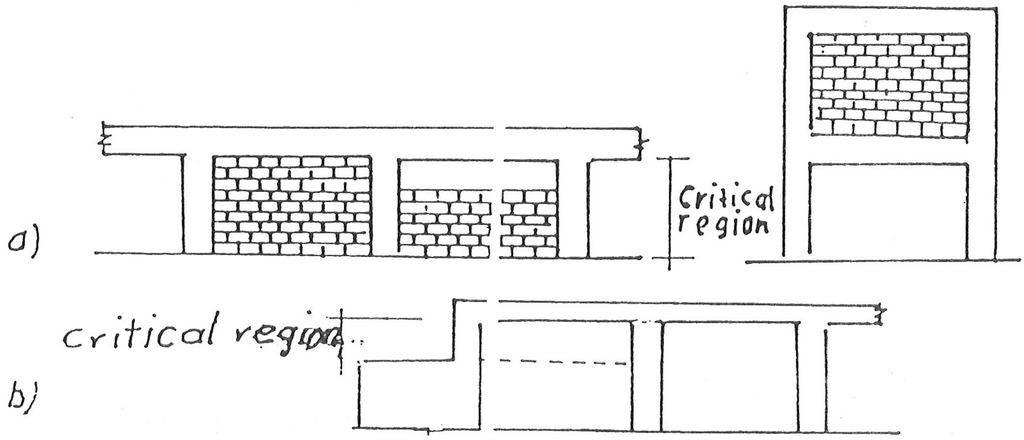


FIG 5.5

(ii) one half the least cross-sectional dimension of the section

(iii) 200mm.

d) The transverse reinforcement in the amount specified above shall be continued throughout the length of the beam-column joint.

Non-critical regions

The minimum transverse reinforcement in non-critical regions shall be in accordance with BS 8110 (1985). (see Table 5.1) (Fig.5.6).

TABLE 5.1. TRANSVERSE REINFORCEMENT DUCTILITY LEVEL II

	Critical Region lc = max (h, 1/6, 450mm)
Spacing	Critical Region Sh = min (8∅L, 1/2b, 200mm)
	Elsewhere sn = min (12∅L, b, 300mm)

1.5.2.3(3) DL III Structures

Critical regions

TABLE 5.2. TRANSVERSE REINFORCEMENT DUCTILITY LEVEL III

	Critical Region lc = max (h, 1/6, 450mm)
Spacing	Critical Region sh = min (6∅L, 1/4b, 150mm)
	Elsewhere sn = min (8∅L, 1/2b, 200mm)

(Fig. 5.7).

a) The volumetric ratio of transverse reinforcement (spiral or hoops) shall not be less than the greater of

$$\rho s = \lambda 1 . \frac{fck}{fyk} \quad \ldots\ldots \quad (1.5.2.3.3.1)$$

or $$\rho s = \lambda 2 \left(\frac{Ag}{Ac} - 1\right) \frac{fck}{fyk} \quad \ldots\ldots. \quad (1.5.2.3.3.2)$$

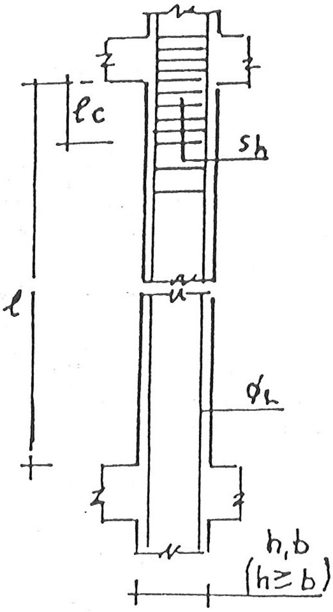


FIG 5-7 SPECIAL TRANSVERSE REINFORCEMENT: CRITICAL REGION AND SPACING

where A_g = gross sectional area

A_c = confined area of concrete

and the values of λ_1 and λ_2 are given in Table 5.3 as function of the reduced axial force ratio $N_d/A_c.f_{ck}$

TABLE 5.3. VALUES OF λ_1 AND λ_2

Nd/Ac.fck	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

b) The volumetric ratio is the ratio of volume of spiral or hoop reinforcement to total volume of concrete core (out-to-out of bars) within spacing sh.

c) The volumetric ratio ρ_s for rectangular sections is defined as

$$\rho_s = A_{sh}/s_h.h'$$

where Ash is the total area of hoop bars and supplementary cross ties in each of the principal directions of the cross section, Sh is the spacing and the h' is the distance between centres of outer bars.

d) The minimum diameter of spiral or hoops shall be 8mm.

e) The maximum spacing between spirals or hoops shall not exceed the smaller of:

(i) six times the minimum diameter of longitudinal bars;

(ii) one fourth of the smallest lateral dimension of the section

(iii) 150mm

f) Each longitudinal reinforcement bar or bundle of bars shall be laterally supported by the corner of a hoop having an included angle of not more than 135 or by a supplementary cross tie, except that the following bars are exempted from this requirement:

(i) bars or bundles of bars which lie between two bars supported by the same hoop where the distance between the laterally supported bars or bundles of bars does not exceed 200mm between centres.

(ii) inner layers of reinforcing bars within the concrete core centred more than 75mm from the inner face of hoops.

g) The yield force of the hoop bar or supplementary tie shall be at least one-sixteenth of the yield force of the bar or bars it is to restrain including the contribution from the bar or bars exempted under 1.5.2.3.(3)(f)(i).

h) Each end of a supplementary tie shall engage either a longitudinal bar or the peripheral hoop besides a longitudinal bar with a bent of at least 135° and an extension beyond the bent of at least 10 tie bar diameters. Supplementary ties and legs of hoops shall not be spaced transversely more than either 200mm or one-quarter of the column section dimension perpendicular to the direction of the transverse steel (Fig.5.8).

Non-critical regions

The requirements relative to the critical regions of DL II columns apply.

1.5.3 Beam - Column Joints

1.5.3.1 Confinement

(1) DL I and DL II Structures

The horizontal transverse confinement reinforcement in beam-column joints shall not be less than that required for the columns.

(2) DL III Structures

a) The horizontal transverse confinement reinforcement in beam-column joints shall not be less than that required for the columns with the exception of joints connecting beams at all four column faces that are designed according to Clause 1.4.4.2(1)(d)(ii) or (iii) in which case the transverse joint reinforcement may be reduced to one half of that required for the columns, but in no case shall the stirrup tie spacing in the joint core exceed ten times the diameter of the column bar, or 200mm, whichever is less.

b) When the width of the column is larger than the effective joint width specified in Clause 1.4.4.2(1)(a) and (b), all flexural reinforcement in the column that is required to interact with the narrow beam, shall be placed within the effective joint area, bj.hc. Additional longitudinal column reinforcement shall be placed outside this effective area.

1.5.4 Structural Walls

1.5.4.1 Geometrical Constraints

(i) Wall thickness shall not be less than 150mm.

(ii) Openings in the walls not regularly arranged to form coupled walls shall be preferably avoided, unless their influence on the behaviour, of the wall under seismic action is either insignificant or accounted for by rational analysis.

(1) DL III Structures

In addition to the requirements in 1.5.4.1(i) and (ii), DL III shall also safisfy the following:

a) the height (hw) to width (lw) ratio shall not be less than 2;

b) the local thickness of a wall shall not be less than hn/10 (hn is storey height) wherever the maximum compressive strain exceeds the value $\varepsilon cu/3$), unless:

(i) the distance of the critical fibre (i.e. where $\varepsilon c = \varepsilon cu/3$) from the wall edge is smaller than 2b or 0.2 lw,

(ii) the distance of the critical fibre from a transverse wall, or from a flange the width of which is at least hn/5, is smaller than 3b (Figs. 5.9 - 5.11).

1.5.4.2 Longitudinal Reinforcement

The vertical reinforcement ratio in any part of the section shall not be less than 0.25%, not greater than 4%.

(a) At least two orthogonal grids of reinforcement shall be used, one hear each side of the wall.

(b) The diameter of the bars used in any part of a wall shall not exceed: b/10.

(c) The maximum spacing between bars shall be 300mm except where the section is required to be confined, in which case the spacing shall not exceed 200mm.

(d) Curtailing: Vertical flexural reinforcement shall be curtailed in accordance with the bending moment envelope, allowing for the development lengths of the curtailed bars.

(e) Splicing: Splicing of the vertical reinforcement in potential areas of yielding (see 1.5.4.3(1) shall be

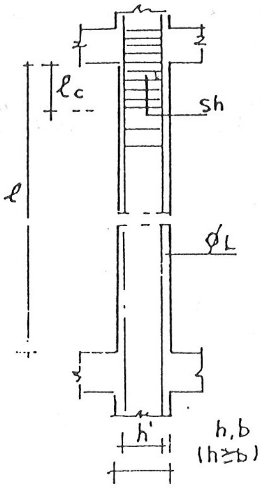


FIG 5-6 SPECIAL TRANSVERSE REINFORCEMENT
CRITICAL REGION AND SPACING

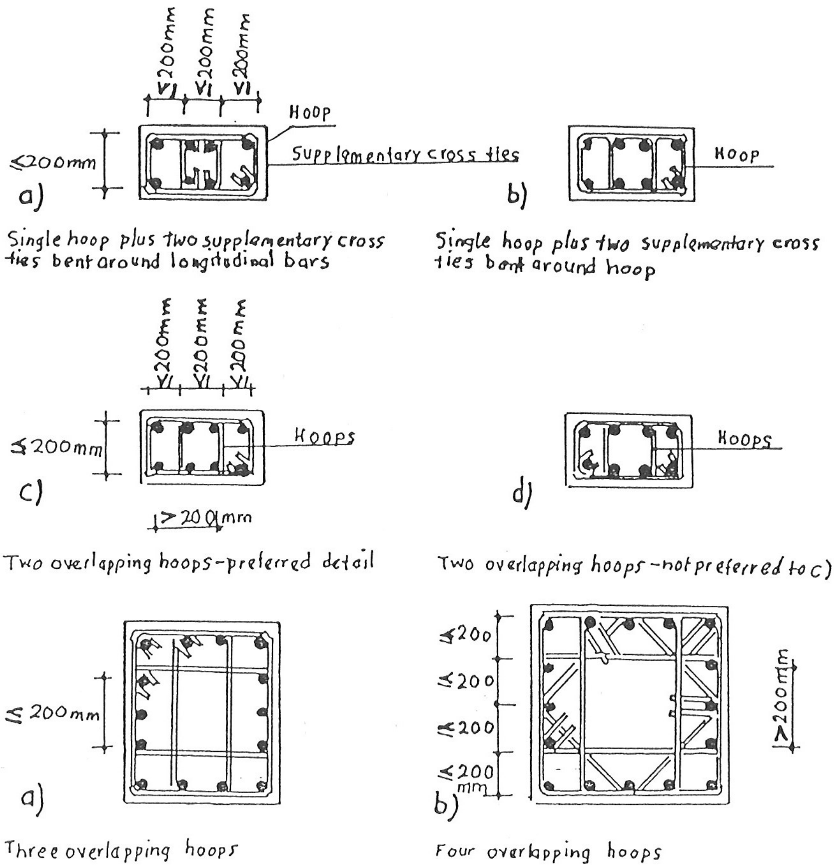


FIG 5-8 TYPICAL DETAILS USING OVERLAPPING HOOPS

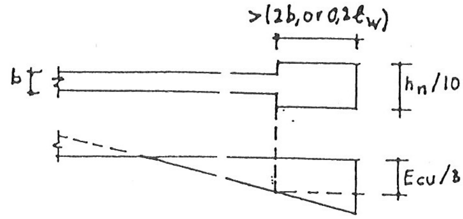


FIG 5·9 MINIMUM WIDTH REQUIREMENTS

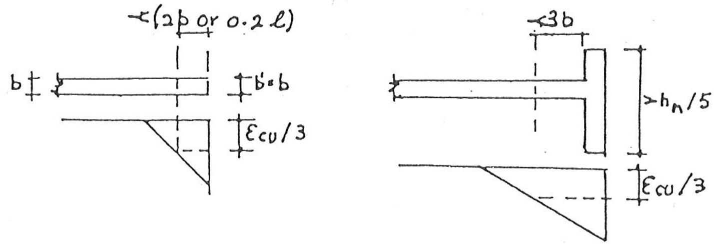


FIG 5·10 EXEMPTION FROM C1.5.5.1b) ON MINIMUM WALL WIDTH

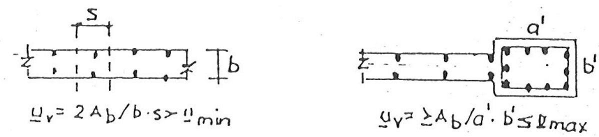


FIG 5 11 DEFINITION OF VERTICAL REINFORCEMENT RATIO avoided whenever possible. In no case shall more than one-third of such reinforcement be spliced in those areas. Special care shall be taken for splicing of the main (flexural) vertical bars. The splices should be staggered in the longitudinal direction at least twice the spliced length.

(f) Construction joints: The ratio of vertical reinforcement crossing a construction joint shall be such as to provide for the transfer of the entire shear capacity of concrete and is given by the expression:

$$\rho v = (1.3\ fctm - 0.7\ \frac{Nd}{Ag})/fyk > 0.0025 \quad \ldots \quad (1.5.4.2.1)$$

where: ρv = Ast/b.lw with Ast = total vertical wall reinforcement, including that in boundary elements provided to resist flexure.

Ag is the gross area of the effective wall section including boundary elements.

Nd is the minimum compression force in the wall. Tension shall be taken as negative.

1.5.4.3 Transverse Reinforcement

The requirements for minimum reinforcement ratio, maximum diameter and maximum spacing, shall be as for longitudinal reinforcement (Clause 1.5.4.2).

(1) Zones with special transverse reinforcement

a) The zones of walls requiring special reinforcement as specified in (b) below are defined as follows:

(i) in the vertical direction, they shall extend from the base over the probable plastic hinge length, which for the purpose is assumed to be the greater of; the length (lw), or 1/6 of the height (hw) of the wall,

(ii) in the plan section, whenever the computed concrete strain exceeds the value: $\varepsilon cu/3$. The strain profile over the section shall correspond to development of its flexural strength, under the maximum design axial compression force occurring for a load combination including the seismic action.

b) The amount of special transverse reinforcement to be provided is a function of the computed depth of the neutral axis: x in the base section of the wall, and of the selected ductility level as follows:

DL II Structures

The critical neutral axis depth, computed for the most adverse, design bending moment Md, is given by:

$$\bar{x} = 0.20 \; \frac{(\overset{+}{Mu,d})}{Md} \cdot lw$$

when:

(i) $x \leq \bar{x}$ Transverse reinforcement shall satisfy the minimum requirements set forth in Clause 1.5.4.3. Cross ties to restrain longitudinal bars shall be used in accordance with BS 8110 (1985).

(ii) $x > \bar{x}$ Transverse reinforcement shall satisfy the requirements of Clause 1.5.2.3(2) (ductility level II columns in critical regions).

DL III Structures

The critical neutral axis depth, computed for the most adverse bending moment Md, is given by:

$$\bar{x} = 0.10 \; \frac{(\overset{+}{Mu,d})}{Md} \cdot lw$$

when:

(i) $x \leq \bar{x}$ Transverse reinforcement shall satisfy the requirements of Clause 1.5.2.3(3) (DL III columns in non-critical regions).

(ii) $x > \bar{x}$ Transverse reinforcement volumetric ratio shall not be less than the greater of:

$$\rho s = \lambda 1 \; \frac{fck}{fyk} \quad \ldots\ldots \quad (1.5.4.3.1.1)$$

$$\text{or } \rho s = \lambda 2 \; \left(\frac{Ag}{Ac} - 1\right) \frac{fck}{fyk} \quad \ldots\ldots \quad (1.5.4.3.1.2)$$

where the values of $\lambda 1$ and $\lambda 2$ are given in the following Table 5.1 as functions of neutral axis depth ratio.

TABLE 5.4. VALUES OF λ_1 AND λ_2 IN EQUATIONS 1.5.4.3.1.1 AND 1.5.4.3.1.2

x/l_w	0.10	0.20	0.30	0.40	0.50	0.60	0.70
λ_1	0.07	0.08	0.09	0.105	0.115	0.125	0.135
λ_2	0.18	0.205	0.23	0.26	0.285	0.31	0.34

The volumetric ratio is defined as:

$$\rho_s = \frac{A_{sh}}{s_h \cdot h'}$$

with h' = dimension of wall concrete core measured perpendicular to the direction of hoop bars to outside of peripheral hoops.

Ash = total steel area of hoop bars and supplementary cross ties in direction under consideration, within spacing Sh. (Figs. 5.12 and 5.13).

1.5.4.4 Coupling Beams

a) The diagonal reinforcement in each direction shall be enclosed by rectangular stirrups, hoops or spirals in accordance with Clause 1.5.2.3(3), however, their spacing or pitch shall not exceed 100mm.

b) Minimum thickness for diagonally reinforced beams shall be 200mm. The anchorage length of diagonal reinforcement in the adjacent walls shall be increased by 50% of the lengths prescribed in BS 8110 (1985) (Fig. 5.14).

1.5.5 Anchorage and Splicing of Reinforcement

1.5.5.1 General

In addition to the rules of BS 8110 (1985), the following requirements shall be satisfied in order to ensure reliable behaviour during cyclic loading reversals caused by seismic action:

a) All reinforcement bars should be considered to be in non-good bond conditions except when anchorage is made in regions confined by means of special transverse reinforcement, where good bond condition can be assumed.

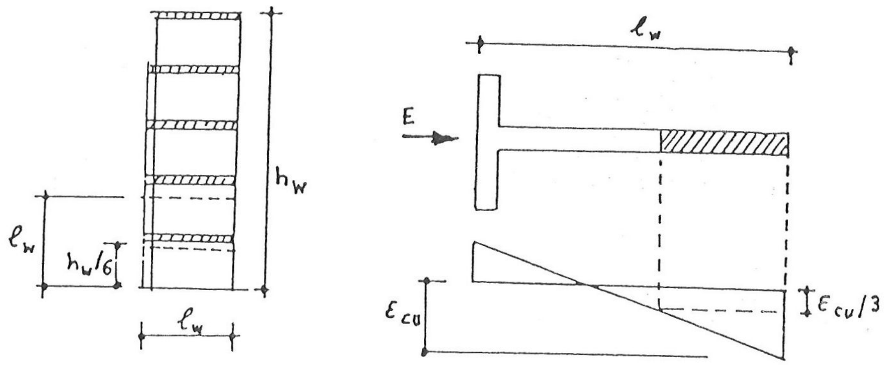


FIG 5·12

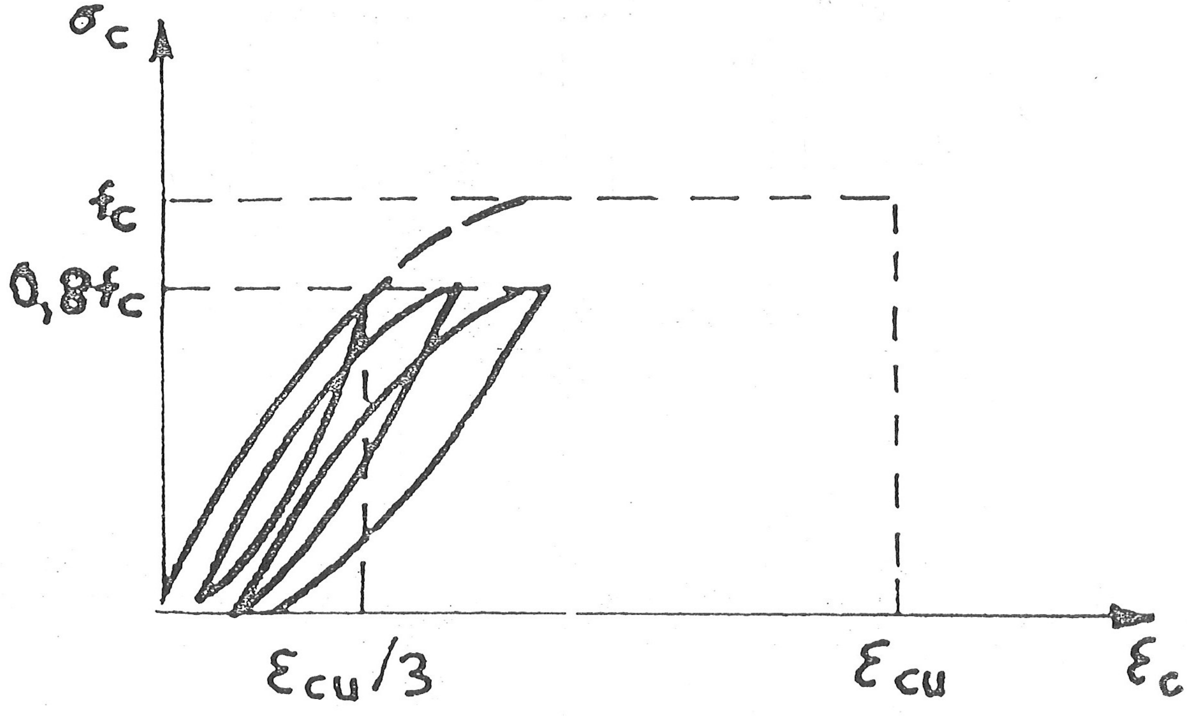


FIG 5·13

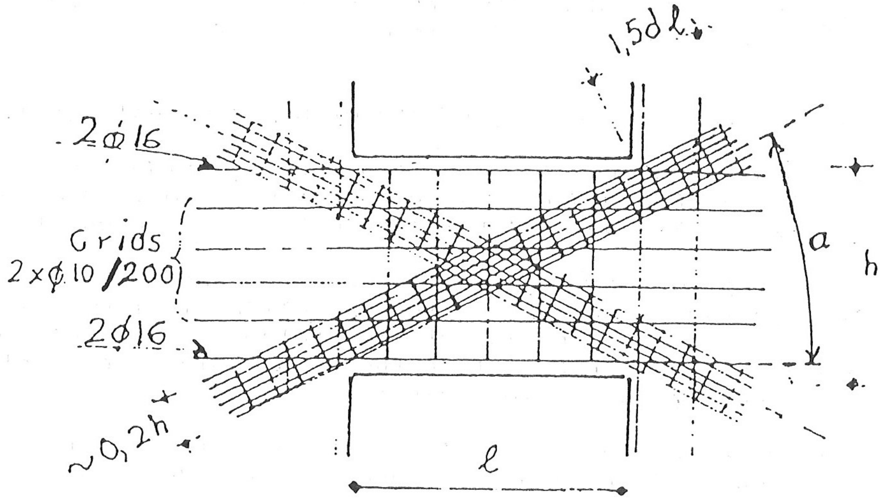


FIG 5.14

b) All bars should be able to develop their maximum strength (γ_{Rd}).fyk when a plastic hinge is formed.

1.5.5.2 Flexural Members: Anchorage of Longitudinal Reinforcement

a) Flexural members framing into opposite sides of a column shall have top and bottom reinforcement provided at ends of members continuous through the column where possible.

b) When top or bottom reinforcement cannot be continuous through the column due to the variations in flexural members cross section, and in exterior columns, the reinforcement shall be anchored within the beam column connection in accordance with the following:

(i) Reinforcement shall be extended to the far face of the confined region and anchored to develop its yield strength.

(ii) Every bar shall terminate with a standard 90-deg hook or equivalent anchorage device, as near as practically possible to the far face of the column core. Top bars should be bent down and bottom bars bent up.

(iii) Development length of beam reinforcement shall be computed beginning at a distance of 10Ø from the near face of column.

c) For DL III Structures, when beams frame into opposite sides of a column, the maximum diameter of the longitudinal beam bars which are continuous through the column should not exceed the following fractions of the column depth (parallel to the bar) in Table 5.5.

TABLE 5.5

Steel Grade	Fraction of hc
S220 smooth	1/35
S220 deformed	1/20
S400 deformed	1/30

1.5.5.3 Column : Anchorage of Longitudinal Reinforcement

a) The maximum diameter of longitudinal column bars which are continuous through a joint shall not exceed the following fractions of maximum depth of the beams framing into the column.

TABLE 5.5

Steel Grade	Fraction of hb
S220 smooth	1/25
S220 deformed	1/15
S400 deformed	1/25

When hinges are permitted to form in columns the valves indicated in Clause 1.5.5.2 shall be applied.

b) The anchorage of a column bar into an intersection beam shall be made by a horizontal 90-deg standard hook or equivalent device, as near the far face of the beam as practically possible. The direction of the horizontal leg of the standard hook must always be towards the core of the project.

c) When columns terminate at joints at the top of frames or at joints between columns and foundation members, the anchorage of the longitudinal column bars into the joint region shall be assumed to begin at a distance equal to one half of the depth of the beam or 100, whichever is less, from the face at which the column bar enters the beam (Fig. 5.15).

1.5.5.4 <u>Splices of Longitudinal Reinforcement</u>

a) Splices are not permitted within beam-column joints or within potential plastic hinge regions.

b) If it can be shown that plastic hinges cannot develop, splices are permitted in the end sections of columns, provided that transverse reinforcement, spaced vertically no further than 6 bar diameters, is present.

c) Stirrup-ties shall be provided over the length of all lap splices of reinforcement in beams and columns. The maximum spacing of the stirrup-ties shall not exceed 10 times the diameter of the bar being spliced.

For DL III structures the maximum spacing shall also not exceed 150mm.

d) Welded splices or approved mechanical corrections conforming with BS 8110 (1985) may be used, provided that not more than alternate bars in each layer of longitudinal reinforcement are spliced at a section, and the distance between splices of adjacent bars is 600mm or more measured along the longitudinal axis of the frame component.

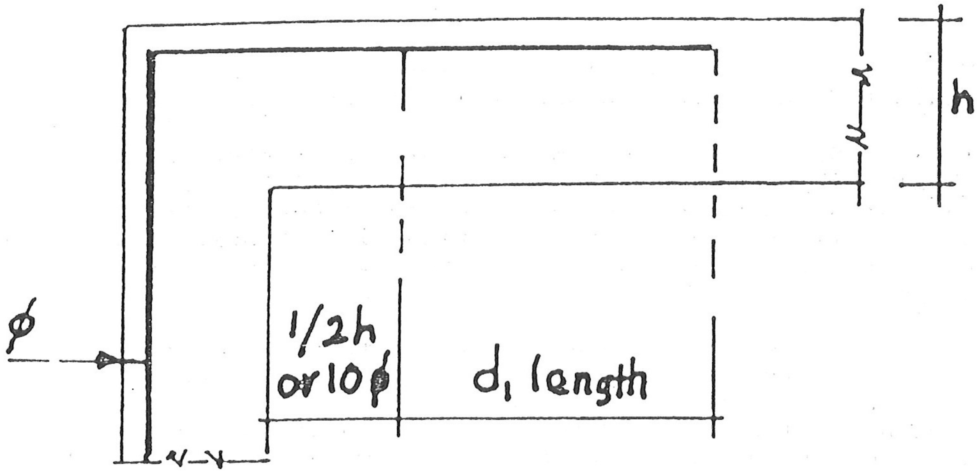


FIG 5·15

1.5.5.5 Anchorage and Splicing of Transverse Reinforcement

a) Transverse hoop reinforcement shall be anchored by at least a 135° bent around a longitudinal bar with a minimum extension at the free end of 10 bar diameters. Alternatively, the ends of the hoops can be spliced by welds capable of developing the full strength of the bar.

b) Transverse reinforcement shall not be lap-spliced in cover concrete within beam-column joints or within potential plastic hinge regions. Deformed bars shall be used for lap splices.

c) When the anchorage for a spiral terminates with a 135 bend around a longitudinal bar, the extension beyond the bend shall be at least 10 spiral bar diameters.

1.6 SEISMIC ACTION

1.6.1 Regional Seismicity

The seismicity activity of a region can be conveniently described by means of seismic risk maps.

The seismic risk maps give the spatial distribution of the values of one or more parameters defining intensity of the seismic event, for given average return periods or, equivalently, for given annual probabilities of exceedance.

These maps are constructed on the basis of historical records where available, and/or geological and seismotectonic data.

The intensity of a sesmic event can be expressed either by means of a phenomenological scale, such as the MM or MSK scales or by a (set of) parameter(s) representative of the ground motion.

For the purpose of structural design, the most suitable parameter (notwithstanding certain limitations) is the maximum ground acceleration: Amax, complemented in general by Vmax and Smax, maximum ground velocity and displacement respectively.

1.6.2 Seismic Zones

For the application of this Code, seismic risk map (Fig. 6.1) has been used to discretize the area of Ghana into a number of zones. Within each zone the normalized ground acceleration is assigned a constant value as shown in Table 6.1.

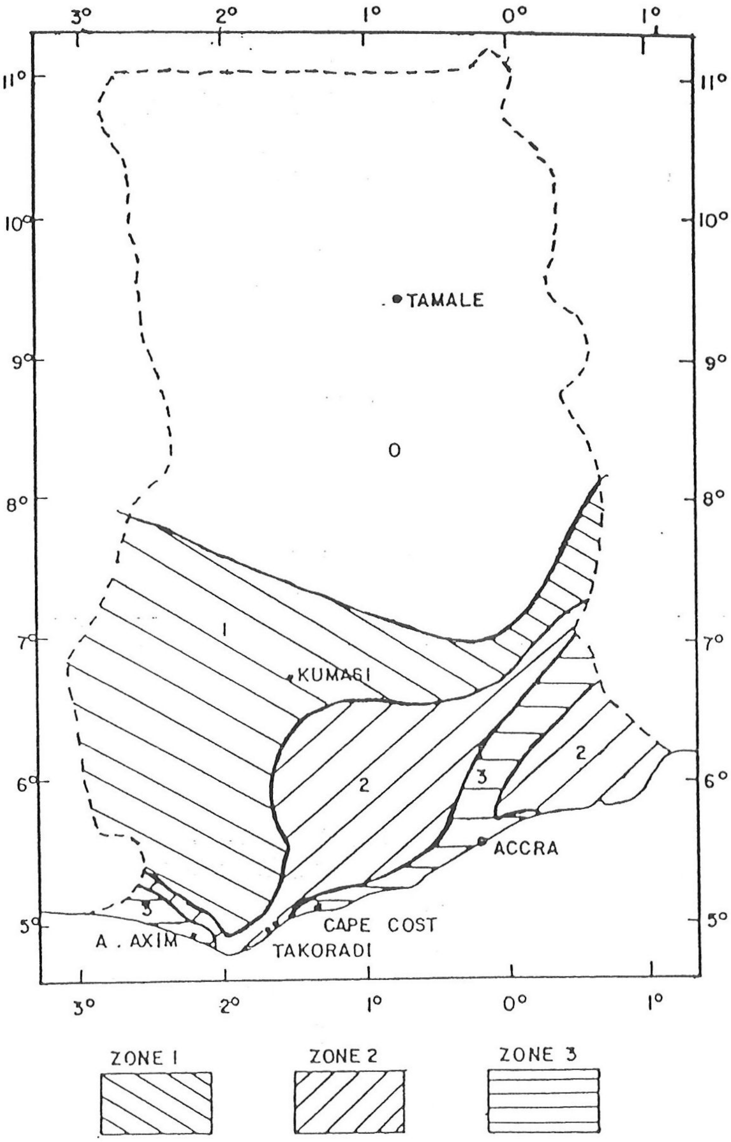


FIG 6.1 SEISMIC RISK MAP OF GHANA

TABLE 6.1. DEFINITION OF SEISMIC ZONE

Seismic Zone	Assigned Horizontal Design Ground Acceleration: A g (unit of gravity)
0	0
1	0.15
2	0.25
3	0.35

1.6.3. Characteristics of Seismic Actions

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

(1) For the purpose of this Code, the ground motion shall be adequately described by means of:

(a) the peak ground acceleration Amax, treated as a random variable of known distribution;

(b) one or more response spectra for horizontal motion, having a form appropriate for the area and firm soil conditions, normalized to Amax = 1, and probabilistically characterized;

(c) one or more response spectra for vertical motion, scaled to 2/3 of the corresponding horizontal motion response spectra.

(2) For particular zones, for instance 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.

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

1.6.4 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 provisions of this Code and with those of BS 8110 (1985) warrants the

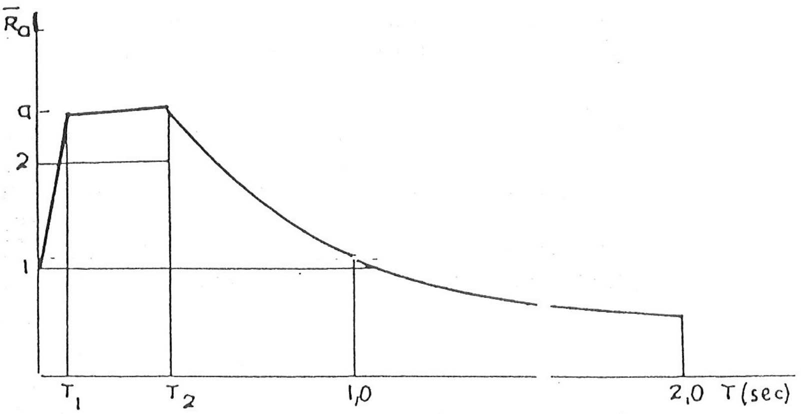


FIG 6-2 NORMALIZED ELASTIC RESPONSE SPECTRUM

satisfaction of the general requirements set forth in Section 1.2 with the established level of reliability.

1.6.4.1 Normalized Elastic Response Spectrum

For the purpose of this Code, the shape of the 'standard' (rocky or firm soil condition) elastic response spectrum normalized to a unit peak ground acceleration shall be idealized as shown in Fig. 6.2.

The spectrum is intended for a damping ratio of 5 per cent.

Average values of T1 and T2 (Fig. 6.2) obtained from several accelerograms (recorded at distances of less than 80 km from the epicentres of moderate to large earthquakes) are:

$T1 = 0.12$ secs $\qquad T2 = 0.4$ secs

For values of Amax in excess of say 0.25g an amplification factor $\alpha = 2.5$ can be assumed to give a probability of non-exceedance lying between 70% and 80%.

For an approximately constant spectral density (band-limited white-noise) as expected for a non-distant, moderate-to-large intensity motion travelling on rock, the value of β consistent with $\alpha = 2.5$ would be comprised between 1.0 and 0.9.

1.6.4.2 Site Effects

When more detailed knowledge on the effects of local soil conditions and on the characteristics of ground motions arriving at the site from possibly different sources is not available, the procedure in Clauses 1.6.4.2(1) and (2), 1.6.4.3 shall be applied.

(1) Soil profile types

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

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

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

(iii) SOIL PROFILE S3: soft-to-medium stiff clays and sands, characterized 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 type or where the profile does not fit any of the three types, soil profile S2 shall be used.

(2) Site Coefficient

The site coefficient S is used to modify the standard elastic response spectrum to account for the site condition. Its values are given in Table 6.2.

TABLE 6.2. SITE COEFFICIENT

Site Coefficient	Soil Profile Type		
	S1	S2	S3
S	1.0	1.2	1.5

1.6.4.3 Site-Dependent Normalized Elastic Response Spectra

The site-dependent normalized elastic spectra for the three soil profiles are shown in Fig. 6.3, their ordinates being defined as the smallest from the following expressions:

- $\bar{R}as = 1 + (\alpha - 1).\ T/T1$

- $\bar{R}as \begin{cases} = \alpha & \text{for soil types S1, S2, S3,} \\ = 0.8\alpha & \text{for soil S3 if A, as defined in Clause 1.6.4.4 is greater than 0.3g} \end{cases}$

- $\bar{R}as = S.\ \alpha\ .(T2/T)^{\beta}$

In case of lack of specific site-related information, T1, T2, α and β can be assigned values as proposed in Clause 1.6.4.1. However, for soil type S3 the value T1 = 0.25 secs. can be adopted.

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.

1.6.4.4 Design Response Spectrum

The ordinates of design response spectrum are given by the smallest of the following expressions:

- $Ra(T) = I.A.\alpha.\frac{1}{K}$ for soil type S1, S2, and S3 or
- $Ra(T) = 0.8.I.A.\alpha.\frac{1}{K}$ for soil type S3 if $A \geqslant 0.3g$
- $Ra(T) = I.A.S.\alpha.(T2/T)^{\beta}.\frac{1}{K}$

where:

- I is the importance factor defined in section 1.3.2 (see Table 6.3)
- A is the peak ground acceleration to be adopted for the seismic zone of interest (Table 6.1)
- S is the site coefficient as given in Table 6.2
- K is the behaviour factor as given in Table 1.4.1.3.

In the case of lack of specific site-related information, α, β and T2 are assigned the following values:

$\alpha = 2.5$ $\beta = 2/3$ $T2 = 0.4$ sec.

TABLE 6.3. IMPORTANCE FACTOR

Class	Factor I
I	1.4
II	1.0

Representative design spectra for the three soil types, A = 0.35g, I = 1 and K = 1 are shown in Fig. 6.4.

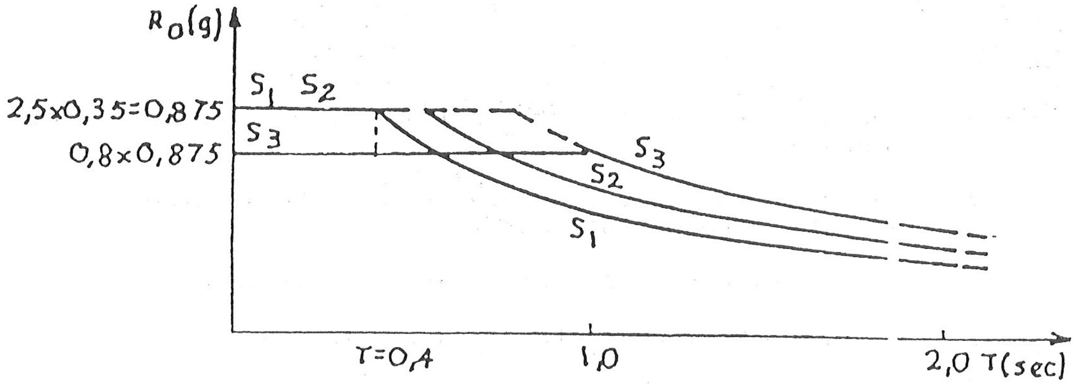


FIG 6-4 DESIGN SPECTRA FOR **A** = 0·35g I = 1 / ND K = 1

DEFINITIONS AND NOTATIONS

<u>Cross-tie:</u> A continuous bar with a minimum diameter of 6mm, having a 135° hook with a ten-diameter extension at one end, and a 90° hook with a six diameter extension at the other end. The hooks shall engage hoop bars and be secured to longitudinal bars.

<u>Hoop:</u> A closed tie or continuously wound tie with a minimum diameter of 6mm the ends of which have 135° hooks with ten-diameter extensions that encloses the longitudinal reinforcement.

<u>Boundary elements:</u> Portions along the edges of walls and diaphragms strengthened by longitudinal and transverse reinforcement. Boundary elements do not necessarily require an increase of thickness of the wall or diaphragms may also have to be provided with boundary elements.

A	=	peak ground acceleration
Ac	=	confined area measured to outside peripheral transverse reinforcement.
Ag	=	gross sectional area of concrete
Cd	=	design seismic coefficient (1.4.2.4(1)).
Cg	=	centre of mass (1.4.2.1.(1))
Ck	=	centre of stiffness (1.4.2.1(1))
E	=	design seismic action (symbolic) (1.4.1.4(1))
I	=	importance factor (1.3.2)
Mu,d	=	ultimate moment of a concrete section, evaluated with factored values of concrete and steel strengths
$^{+}$Mu,d	=	ultimate moment of a concrete section, evaluated with characteristic values of concrete and steel strengths.
Nd	=	design axial force under the most unfavourable load combination including the seismic action
K	=	behaviour factor (1.4.1.3)
S	=	site coefficient (1.6.4.2(2))
Si	=	soil type index (1.6.4.2(1))
Vcd	=	shear force carried by concrete in beam or column sections.
a	=	plan dimension of the building in the direction orthogonal to that of seismic action (1.4.2.4(2)).

b_w = web width of a concrete section

h, b = height and width of beams, major and minor sides in columns

h', b' = distance between reinforcement bars located at the ends of sides h and b, respectively, measured to outside the peripheral bars.

d = distance from centre of stiffness and centre of gravity of the generic floor (1.4.2.4(2))

S_h = spacing of transverse reinforcement in beams, columns and walls.

f_{cd} = design concrete strength

h = height of a floor

l_w = horizontal wall length

h_w = total height of a wall

h_n = vertical distance between floors in walls

α = spectral amplification factor (1.6.·.1)

β = parameter of the elastic response spectrum (1.6.4.1)

γ_i = distribution factor (1.4.2.4.(1))

γ_n = over-capacity factor

Δ_{el} = elastic interstorey drift under the seismic actions (1.4.2.4(3)

ζ_o = ratio between maximum and minimum shear force at a beam end (1.4.3.1(1), (1.4.3.2(1).

Θ = deformability index (1.4.2.4(3))

ξ = amplification factor for torsional effects (1.4.2.4(2).

w = dynamic magnification factor (1.4.3.1(2), (1.4.3.2(2), (1.4.3.2(d)(iv)

τ_{Rd} = shear design stress of concrete

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