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IRAQI BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE

BUILDING RESEARCH CENTRE
SCIENTIFIC RESEARCH COUNCIL

IRAQI BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE CODE 1/87

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" IN THE NAME OF ALLAH, THE COMPASSIONATE, THE MERCIFUL "

PREFACE

Iraq has witnessed, during the last two decades, a giant leap in number and volume of the implementation in the construction projects, which are an essential and integral part of the ambitious development plan of the country. Concrete structures represent a large proportion of these projects.

The absence of a national code for the design and implementation of these constructions, created a state of unhomogeneous application of various international codes on structures, which are built from unified local building materials and also subjected to unified environment. The absence of such code disturbed the process of implementation at the stages of design, checking and contracting. This consequently caused a great deal of waste at all levels and increased the cost of implementation of the projrcts. For all these reasons, the Building Research Centre (BRC), proceeded since 1978, with the project of the Iraqi Building Code Requirements for Reinforced Concrete. At the beginning a draft of the code was developed, which was considered as the first step to subject construction projects using reinforced concrete to unified rules and standards. This will at the end provide two main elements, mainly safety and the suitability of the construction to local environment. All this will surely enhance the economy of the country through the proper use of building materials. The Iraqi code project was one of the most important projects for the five year plan (1981-1985) of the centre, where a great deal of effort was put in to implement to this essential project.

A number of structural engineering researchers who belong to the staff of the centre developed the draft of the code. During the development the researchers referred to local and regional research work and investigations in this field. They also referred to the relevant international codes. The task was completed in November 1982.

A specialized code seminar was held in the centre for the period 25-27th April 1983, to discuss the proposed draft code in order to enrich this draft with the local and international experiences. This seminar was attended by Iraqi experts, making up the backbone of Iraqi experience in the field of structural engineering. In addition specialists from Arab and foreign countries were also invited to attend the seminar.

In order to implement the resolution of the seminar which adopted the draft with the view to its modification according to the discussion, the centre established a higher committee for the Iraqi building code requirements for reinforced concrete from specialists with a high level of expertism so they can work collectively to produce the final draft for the first national code for the design of reinforced concrete buildings, which satisfy the requirements of design, implementation and contracting. This code can also be referred to in the teaching process of the Iraqi engineering colleges.

The higher committee for the Iraqi code which was established in the centre consisted fo twelve members with the relevant specialization and expertism and they are employed in the various building and construction establishments in the country. The reason for this choice is the nature of work which requires a varied experience and specialization which is invariably unavailable in a single institute. The members represent the following institutions:

Ministry of Housing and Construction, Ministry of Industry and Minerals, Ministry of Higher Education and Scientific Research (represented by the Universities of the country, BaghdadUniversity, Mosul University and University of Technology), National Private Consulting Bureau, the Central Organization for Standardization and Quality Control and Iraqi Engineering Society in addition to researchers from the centre (Structural Department and Building Materials Department). The Committee started its work January 1984 and according to careful planned time schedule.

The committee adopted scientific foundations and concepts suitable with the scientific and technical development to produce the code. This was done so as to give the code sufficient flexibility to absorb any future development keeping in mind the suitability of the technical and scientific level of the code to the local environment.

I believe that issuing any code, is a dynamic process requires the updating of its various items from time to time so as to absorb all the new developed technologies and the output of the future scientific research work. On this basic the Iraqi code like any other international code requires updating as necessary every decade. The centre will perform this task.

At the time I present this code which will be issued in both Arabic and English for use by the construction sector in the country, I hope that it will participate in the support of our national economy and to enforce the idea of depending on our national experts to build our great country and the Arab homeland.

Finally, I would like to express my deepest thanks to the President of the Scientific Research Council for his continuous support. Thanks and gratitude are also due to all those who participated in accomplishing this important project, especially the members of the Iraqi code committee and the assisting staff.

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Director General of B.R.C.

FOREWORD

The committee of the Iraqi code for reinforced concrete was formed by the Building Research Centre (BRC). It comprises twelve experts in the fields of building construction, industry and higher education. The Central Organisation for Standardization and Quality Control and Iraqi Society for Engineers were also represented in the committee.

The committee commenced its work in January 1984 according to a set time schedule. The execution of work was done by dividing the committee into several specialized subcommittees. Each subcommittee was given a part of the draft code to study and revise and then prepare a proposal to be submitted for approval before it becomes as part of the first national code of practice for reinforced concrete.

The committee discussed the essence of scientific bases and knowledge in line with the scientific and technical developments in order to adopt them in the code and to give it enough flexibility to comprehend any future development without affecting its suitability for the local environmental conditions and technical levels.

On this basis the following items were adopted:

1- Adopting the draft code accomplished at the BRC as a basis, developing it according to the discussions carried out during the Iraqi code seminar in addition to the latest national and international scientific developments.

2- Confirming that the subject of the code is to be reinforced concrete for buildings as distinct from other subjects such as prestressed concrete or water retaining structures which have special additional requirements which require special publications as done internationally.

3- Adopting one set of regulations and method of analysis and permitting the use of other methods that have to be proven adequate.

4- Adopting the concept of limit states which guarantee limits of safety and serviceability of the structure during all loading stages. These limit states consist of three limit states, the ultimate limit state under the effect of factored loads and both cracking and deflection limit stats under service loads. The adoption of the concept of limit states in the design provide the flexibility in choosing the required safety factors which guarantee acceptable safety limits of all parts of the structure and thus a global safety factor concerning the adequate performance of the structure as a whole.

5- Adopting the partial safety factor given in the CEB-FIP model code for concrete and reinforcing steel and design loads. The use of different partial safety factors for materials and loads instead of one global safety factor makes possible the ability to study the effects of design loads and materials on a structure separately thus achieving balanced designs which ensure the structrue not reaching the ultimate limit state within reasonable degrees of probability in addition to ensuring enough flexibility in the code to accept future developments regarding loads and performance of materials.

6- Adopting the stress strain curve for both concrete and reinforcing steel as given in the BS 8110-1985.

7- The necessity of issuing the final form of the code in both Arabic and English languages at the same time to ensure the safe use of the code and preventing any possible misuse due to individual translation and interpretation of the various sectors in the code.

8- Adopting the results of local researches and studied carried out in the filed of reinforced concrete especially hot weather concreting and loading tests of buildings.

9- The necessity of issuing design aids which contain design tables and charts that help the designer and ensure that designs are carried out according to the Iraqi code.

The Iraqi code consists of eighteen chapters comprising specifications and methods of testing materials and construction requirements. Several appendixes were added which list various standards and these include the Iraqi standards or internationally acceptable standards in case of absence of an Iraqi standards.

The process of issuing any code is a dynamic one which requires continuous updating of its various sections in order to comply with the technological developments of future research works and studies. Therefore, the Iraqi code, like other codes, require development when necessary. Based on international experience in this field the Iraqi code will need to be reviewed during a period of 5 to 10 years which is the usual time period used in reviewing and developing the various international codes.

The Iraqi code committee believes that the use of this first national code in the design and construction should be optional for a period of two years after issuing, at the end of which the code will be enforced and considered as an official national document for the design and construcuion of reinforced concrete structures and in the field of engineering teaching in Iraq.

Code Committee

CONTENTS

CHAPTER 1- CONCRETE MATERIALS AND TESTS

1.1- Tests of materials

1.1.1- The Engineer shall have the right to order testing of any materials used in concrete construction to determine if materials are of the quality specified.

1.1.2- Tests of materials and of concrete shall be made in accordance with the Iraqi standard specifications listed in Section 1.7. In absence of any prescribed Iraqi specifications, any internationally recognized relevant standards may be accepted.*

1.1.3- A complete record of tests of materials and of concrete shall be available for inspection during the progress of work and for 2 years thereafter, and shall be preserved by the inspecting engineer for this purpose.

1.2- Cements

Cement shall conform to the Iraqi standard specification (IOS **5/84)** for portland cement.

1.3- Aggregates

1.3.1- Concrete aggregates shall conform to the Iraqi standard specification (IOS 45/80).

1.3.2- The Engineer may specify or approve on request the use of other aggregates, including types or gradings not covered by the above Iraqi standard, provided there are sufficient data by special tests and/ or actual service to produce concrete of adequate strength and durability.

1.3.3- Nominal maximum size of coarse aggregates shall not be larger than:

a) 1/5 the narrowest dimension between sides of forms, nor

b) 1/3 the depth of slabs, nor

c) 3/4 the minimum clear spacing between individual reinforcing bars or wires, or bundles of bars.

1.4- Water

Water shall be clean and free from harmful matter to concrete or steel. Where tests are required they shall be in accordance with any internationally recognized standard.*

1.5- Admixtures

1.5.1- Admixtures to be used in concrete shall be subject to prior approval of the Engineer taking into consideration durability and the effect of climate. It shall comply with any acceptable internationally recognized standard.*

1.5.2- Both the amount added and the method of use shall be to the approval of the Engineer, who shall be provided with the following data:

a) The recommended dosage and the detrimental effects of under – dosage and over – dosage.

b) Chloride content, if any, of the admixture shall be stated.

c) Whether or not the admixture leads to the entrainment of air when used at the manufacturer's recommended dosage.

d) The combined effect when more than one admixture is used concurrently.

1.6- Storage of materials

1.6.1- Cement and aggregates shall be stored in such a manner as to prevent their deterioration or intrusion of foreign matter.

1.6.2- Any material that has deteriorated or has been contaminated shall not be used for concrete.

1.7- Iraqi Standards (IOS) cited in this code

IOS 5/84 Portland Cement

IOS 45/80 Aggregates from Natural Sources for Concrete and Building Construction

IOS 50/70 Methods of Sampeling Fresh Concrete and Determination of Workability of Concrete

IOS 52/70 Test of Compressive Strength of Concrete

IOS 53/70 Preparation of Specimens and Testing Flexural Strength of Concrete in the Laboratory

IOS 54/70 Determination of Initial and Final Drying Shrinkage and Moisture Movement for Concrete

IOS 55/70 Preperation and Testing of Concrete Core Samples

* See appendix (A) for relevant internationally recognized standards

2.0- Notation

f_{cu} = characteristic compressive strength of concrete, N/mm^2

f_{cr} = average compressive strength of concrete, N/mm^2

f_{tu} = average splitting tensile strength of concrete, N/mm^2

2.1- General

2.1.1- For the design of reinforced concrete structures the value of f_{cu} shall be based on 28 days tests of 150mm cubes made and tested in accordance with IOS 52/70 specification.

2.1.2- The average splitting tensile strength f_{tu} shall be based on split tests of concrete cylinders of 150mm diameter and 300mm length or 150mm cubes as prescribed in appendix (A-3).

2.1.3- Unless otherwise specified, the test specimens in Sections 2.1.1 and 2.1.2 shall be stored in water at 24±4°C.

2.1.4- The Engineer may require strength test of cubes cured under field conditions to check the adequacy of curing and the protection of concrete in the structure. Such cubes shall be cured under field conditions and shall be molded at the same time and from the same samples as laboratory cured test cubes.

2.1.5- Design drawings submitted for approval or used for any project shall show the characteristic compressive strength f_{cu} for which each part of the structure is designed.

2.1.6- The grade of concrete appropriate for use shall be selected from Table 2.1.

TABLE 2.1 – GRADES OF CONCRETE

Grade	Characteristic Strength f_{cu}, N/mm^2	Lowest Grade for Compliance with appropriate use
C 7	7.0	Plain Concrete
C 10	10.0	
C 15	15.0	
C 20	20.0	Reinforced Concrete
C 25	25.0	
C 30	30.0	
C 35	35.0	
C 40	40.0	
C 50	50.0	

Note: To meet the requirements for durability, see the grade and mix limitations given in Tables 2.3(a) and 2.3(b)

2.2- Selection of concrete proportions

2.2.1- Concrete shall be proportioned to provide

a) Adequate workability and proper consistency to permit concrete to be worked easily into the forms and around the reinforcement under conditions of placement to be employed, without excessive segregation or bleeding.

b) Resistance to aggressive environment especially sulphate attack as required by Section 2.3.3.

c) An average compressive strength f_{cr} sufficiently high to minimize frequency of strength test, so that not more than 5% of test results fall below the characteristic compressive strength f_{cu} and in conformance with strength test requirement of Section 2.4.

2.2.2- Where a concrete production facility has a record based on at least 40 consecutive strength tests that represent similar materials and conditions to those expected, the required average compressive strength f_{cr} used as the basis of selecting concrete proportions shall exceed the required f_{cu} by 1.64 times the standard deviation of cube tests, but not less than ⅓ of the f_{cu} for concrete of characteristic strength up to and including 20 N/mm^2 or 7.5 N/mm^2 for concrete of characteristic strength exceeding 20 N/mm^2.

2.2.3- Where past records of concrete production are not available or unknown, the required average compressive strength f_{cr} shall exceed f_{cu} by ⅔ f_{cu} for concrete up to and including 20 N/mm^2, and 13 N/mm^2 for concrete exceeding 20 N/mm^2 until such records are established.

2.2.4- Concrete proportions, including water/cement ratio shall be established on the basis of field experience (Section 2.2.5) or laboratory trial batches (Section 2.2.6) with materials to be employed as required by Section 2.3.3.

2.2.5- Proportioning on the basis of field experience

Three separate batches of concrete shall be made using materials likely to be typical of the proposed supply and preferably under full scale production conditions. The workability of each of the trial batches shall be determined according to IOS 50/70. Three cubes shall be made from each batch for test at 28 days. If required, further three cubes from each batch may be made for test at an earlier age.

The trial mix proportions shall be approved if the average strength of nine cubes tested at 28 days exceeds f_{cr}.

2.2.6- Proportioning by laboratory trial batches

When laboratory trial batches are used as the basis for selecting concrete proportions, slump shall be within ± 20 mm of maxima permitted by the specifications. A curve shall be established showing the relationship between water/cement ratio and characteristic compressive strength. The curve shall be based on at least three points representing batches which produce strengths above and below required average compressive strength specified in Section 2.2.2 or 2.2.3. Maximum permissible water/cement ratio for concrete to be used in the structure shall be that shown by the curve to produce concrete strength exceeding the characteristic compressive

TABLE 2.3(a) REQUIREMENTS TO ENSURE DURABILITY UNDER SPECIFIED CONDITIONS OF EXPOSURE OF CONCRETE MADE WITH PORTLAND CEMENTS AND WITH NORMAL WEIGHT AGGREGATES

Exposure	Reinforced concrete			Plain concrete		
	free water/cement ratio not more than	cement content not less than (Kg/m³)	lowest grade of concrete	free water/cement ratio not more than	cement content not less than (Kg/m³)	lowest grade of concrete
Mild: surface protected against the weather or aggressive conditions (internal and sheltered external concrete exposed to air) except for a brief period of exposure to normal weather conditions during construction. Strip foundations and trench fill for low rise buildings where the soil conditions are non–aggressive (class 1 Table 2.3b)	0.7	250	C 20	0.8	200	C 15
Moderate: surface shelterd from severe rain and against freezing whilst wet. Burried concrete and concrete continuously under water (see also Mild above).	0.6	300	C 25	0.7	250	C 25
Severe: surface exposed to driving rain, alternate wetting and drying and to occasional freezing. Surfaces subject to heavy condensations or to contact with flowing water or aqueous solutions. Internal surfaces of structures exposed to marine conditions	0.5	350	C 30	0.6	300	C 30
Very severe: surfaces exposed to marine corrosive fumes or flowing aggressive water having PH of 4.5 or less. Surfaces subject to the effect of de-icing salts or to severe freezing conditions whilst wet.	0.4*	350	C 35	0.45	300	C 40

Note 1. The cement contents given in the table relate to 20mm normal size aggregates. In order to maintain the cement content of the mortar fraction at similar values, the minimum cement contents given above should be increased by 15% for 10mm nominal maximum size aggregates and may be decreased by 15% for 40mm nominal size aggregates but in no case may cement content be less than 250 kg/m³ for reinforced concrete.

Note 2. Different aggregates require different water contents to produce concrete of the same workability and therefore a range of free water/cement ratios is applicable to each cement content. In order to achieve satisfactory workability at the specified maximum free water/cement ratio it may be necessary to increase the cement content above the minimum specified.

* To achieve suitable workability at this water/cement ratio, the use of admixtures may be necessary.

strength f_{cu} by 10 N/mm^2.

2.2.7- Proportioning by water/cement ratio

If suitable data from a record of 40 consecutive tests (Section 2.2.2) or from laboratory trial batches are not available, permission may be granted to base concrete propotions on water/cement ratio limits shown in Table 2.3(a). These limits shall also conform to the requirement of external sulphate attack of Section 2.3.3 (b) and to the compressive strength test criteria of Section 2.4.

2.3- Requirements of hardened concrete

2.3.1- The minimum requirement for the strength and durability of concrete in the hardened state shall be decided from consideration of characteristic compressive strength of concrete but if in addition a special property of a particular surface finish is required, these minimum requirements may have to be exceeded.

2.3.2- The grade of concrete required shall depend partly on the particular use and the characteristic compressive strength needed to provide the adequate strength (see Table 2.1) and partly on the exposure conditions (Section 2.3.3).

2.3.3- Exposure conditions

a) Degree of exposure

To produce durable concrete exposed to air, moist or wet conditions or other aggressive environments, careful consideration shall be given to the quality and permeability of the concrete, particularly the specification of free water/cement ratio and the cement content of the concrete mix [see Table 2.3(a)] in addition to the required characteristic compressive strength.

b) External sulphate attack

When concrete is exposed to external sulphate attack then Table 2.3(b) should be used.

2.4- Evaluation and acceptance of concrete

2.4.1- Each strength test result shall be the average of three cubes prepared from a sample taken from randomly selected batches of concrete and tested at 28 days or the specified earlier age. The sample, whenever possible, shall be taken at the final point of discharge of the mixer or, in the case of ready mix concrete, at the final point

TABLE 2-3(b) REQUIREMENTS FOR CONCRETE EXPOSED TO SULPHATE ATTACK*

Class	Concentration of sulphates expressed as SO_3			Type of cement	Minimum cement content Kg/m^3	Maximum free water cement ratio
	in Soil		in Ground water g/ℓ			
	Total SO_3 %	SO_3 in 2:1 water: soil extract g/ℓ				
1	less than 0.2	1.0	less than 0.3	ordinary portland	280	0.55
2	0.2 to 0.5	1.0 − 1.9	0.3 − 1.2	ordinary portland	330	0.5
				sulphate-resisting	280	0.55
3	0.5 − 1.0	1.9 to 3.1	1.2 to 2.5	sulphate-resisting	330	0.5
4	1.0 to 2.0	3.1 to 5.6	2.5 to 5.0	sulphate-resisting	370	0.45
5	over 2	over 5.6	over 5	sulphate resisting plus adequate protective coatings	370	0.45

* For dense fully compacted concrete made with aggregates nominal maximum size of 20mm complying with IOS 45/80.

Notes:

1. The minimum cement content should be increased by 50 kg/m^3 when nominal max. size of aggregates is 10mm. It may be reduced by 40 kg/m^3 when the nominal max. size of aggregate is 40mm. But in no case the minimum cement content for reinforced concrete should be less than 250 kg/m^3.

2. This table applies only to concrete made with aggregates complying with the requirement of IOS 45/ 80 placed near neutral ground water of PH 6 to PH 9 containing naturally occurring suplhates but not contaminants such as ammonium salts.

3. The cement contents given in class 2 are the minima. For SO_3 contents near the upper limit of class 2, cement contents above the minima are advised.

4. When total SO_3 execeeds 0.5% then a 2:1 water:soil extract may result in lower site classification if much of the suplhate is present as low solubility calcium sulphate.

5. For severe conditions e.g thin sections, sections under hydrostatic pressure on one side only and sections partly immersed, considerations should be given to further reduction of water/cement ratio and if necessary an increase in cement content to ensure the degree of workability needed for full compaction and thus minimum permeability.

of discharge from the delivery vehicle.

2.4.2- At least one strength test shall be conducted for each grade of concrete for each day of concreting according to Table 2.4(a).

TABLE 2.4(a)- RATE OF CONCRETE SAMPLING

Rate of sampling	Rate 1	Rate 2
	$20\ m^3$ or 20 batches	$50\ m^3$ or 50 batches
	whichever is the lesser in volume	
Applicable for structures such as	All structural elements except structures described in rate 2	Raft foundation, break waters and similar structures

2.4.3- On a given project, if the total volume of concrete is such that the frequency of testing required by Section 2.4.2 would provide less than four strength tests for a given grade of concrete, tests should be made from at least four randomly selected batches or from each batch if fewer than four batches are used.

2.4.4- For laboratory cured test specimens, the quality of concrete shall be considered to be satisfactory if:

a) The average strength determined from any four consecutive strength tests for laboratory cured specimen exceeds the characteristic compressive strength f_{cu} by at least 3 N/mm^2 and;

b) no individual strength test result is below the required characteristic compressive strength f_{cu} by more than 3 N/mm^2.

2.4.5- Procedures for protecting and curing concrete shall be improved when strength of field-cured cubes at the test age designated for measuring f_{cu} is less than 85 percent of that of companion laboratory – cured cubes. When laboratory cured strengths are appreciably higher than f_{cu}, field – cured cube strength need not exceed f_{cu} by more than 3 N/mm^2 even though the 85 percent criteria is not met.

2.4.6- When cylinders or cubes of different sizes are used for concretes of grade C35 or below as test samples, f_{cu} shall be adjusted by multiplying it by the applicable correction factors given in Table 2.4 (b).

TABLE 2.4(b) CORRECTION FACTORS FOR VARIOUS SHAPES OF CONCRETE COMPRESSIVE TEST SPECIMEN

Shape of Specimen	Dimensions in mm	Correction factor
Cube	150 × 150 × 150	1.00
Cube	100 × 100 × 100	0.98
Cube	200 × 200 × 200	1.04
Cylinder	150Ø × 300 length	1.25
Cylinder	100Ø × 200 length	1.20

2.4.7- If any one strength test fails to meet the requirement of Section 2.4.4(b) then the concrete represented by that strength test only shall be considered not to comply with the strength requirements.

2.4.8- If more than one strength test fails to meet the requirement of Section 2.4.4(b) or if the average strength of any four consecutive strength test results fails to meet the requirement of Section 2.4.4(a) then all the concrete represented by group of four consecutive test results shall be deemed not to comply with the strength requirements.

2.5- Investigation of low strength test results

2.5.1- If either requirements of Section 2.4.4 are not met, steps shall be taken immediately to increase the average strength test results for subsequent concreting.

2.5.2- Action to be taken in respect of concrete in Sections 2.4.7 and 2.4.8 shall be determined by the Engineer, with due regard to the technical consequences of kind and degree of non-compliance and the ecnomic consequences of alternative remedial measures.

2.5.3- The action may range from qualified acceptance (in less severe cases if computations indicate that the load carrying capacity of the structure is not jeopardized) to rejection and removal in most severe cases.

2.5.4- If the likelihood of low strength concrete is confirmed and computations indicate that load carrying capacity may have been significantly reduced, tests of cores drilled from the area in question may be required in accordance with IOS 55/70. In such case, four cores shall be taken for each strength test more than 3 N/mm^2 below required f_{cu}.

2.5.5- If concrete in the structure will be dry under service conditions, cores shall be air dried (temperature 18 to 25°C, relative humidity less than 60 percent) for 7 days before test and shall be tested dry. If concrete in the structure will be more than superficially wet under service conditions, cores shall be immeresed in water for at least 48 hr and be tested wet.

2.5.6- Concrete in an area represented by core tests shall be considered structurally adequate if the equivalent cube strength of the average of four cores is equal to at least 80 percent of f_{cu} and if no single core is less than ⅔ of f_{cu}. To check testing accuracy, locations represented by erratic core strengths may be retested. Core cutting shall, whenever possible, avoid reinforcement, cracking zones where it is likely that bleeding and segregation may have occured.

2.5.7- If criteria of Section 2.5.6 are not met and if structural adequecy remains in doubt, the Engineer may order load tests as outlined in Chapter 18 for the questionable portion of the structure, or take other action appropriate to the circumstances, such as ultrasonic and other non – destructive testing. Such tests should be done by specialists.

3.1- Preparation of equipment and place of deposit
Preparation before concrete placement shall include the following:

a) All equipment for mixing and transporting concrete shall be clean.

b) All debris shall be removed from spaces to be occupied by concrete.

c) Forms shall be clean and properly coated.

d) Reinforcement shall be thoroughly clean of deleterious coatings.

e) All laitance and other unsound material shall be removed before additional concrete is placed against hardened concrete.

f) Masonry filler units that will be in contact with concrete shall be well drenched.

g) Water shall be removed from place of deposit before concrete is placed unless a tremie is to be used or unless otherwise permitted by the Engineer.

3.2- Mixing
3.2.1- The quantity of cement, fine aggregates and coarse aggregates shall be based on weight.

3.2.2- All concrete shall be mixed until there is a uniform distribution of materials and shall be discharged completely before mixer is recharged.

3.2.3- Job-mixed concrete shall be mixed in accordance with the following:

a) Mixing shall be done in a batch mixer of approved type.

b) Mixer shall be rotated at the speed recommended by the manufacturer.

c) Mixing shall be continued for at least 1 ½ min after all materials are in the drum, unless a shorter time is shown to be satisfactory by comparing the strength of samples mixed for different times.

3.3- Transporting, placing and compacting
3.3.1- Concrete shall be transported from the mixer to formwork as rapidly as practicable by methods which will prevent the segregation or loss of any of the ingredients, and maintain the required workability. It shall be deposited as nearly as practicable in its final position to avoid rehandling.

3.3.2- Concrete that has partially hardened or been contaminated by foreign materials shall not be deposited in the structure.

3.3.3- Retempered concrete or concrete that has been remixed after initial set shall not be used unless approved by the Engineer.

3.3.4- After concreting is started, it shall be carried on as a continuous operation until placing of a panel or section, as defined by its boundaries or preditermined joints, is completed except as permitted or prohibited by Section 4.4.

3.3.5- All concrete shall be thoroughly compacted by suitable means during placement and shall be thoroughly worked around reinforcement and embedded fixtures and into corners of forms.

3.4- Curing
3.4.1- Concrete made with normal portland cement shall be maintained in a moist condition by covering it with an absorbent material which is kept damp or ponded with water for a period of at least 7 days after placement. Other means of curing may be used on approval by the Engineer.

3.4.2- Where structural members are of considerable depth or bulk or have an unusually high proportion of cement or are precast units subjected to special or accelerated curing methods, the method of curing shall be specified in detail by the Engineer.

3.4.3- Any accepted process of accelerate curing may be adopted to accelerated strength gain and reduce time of curing.

3.4.4- Supplementary strength tests in accordance with Section 2.1.4 may be required to assure that curing is satisfactory.

3.5- Concreting in cold weather
3.5.1- Special precautions shall be taken when concreting at air temperature below 2°C during the early stages of hardening.

3.5.2- The temperature of concrete at the time of placing shall be at least 5°C.

3.5.3- The temperature of concrete shall be maintained at not less than 5°C and water curing shall not be used until concrete reaches a strength of 5 N/mm^2 as determined by tests on cubes which were cured under the same conditions as the concrete in the structure.

3.5.4- Before placing concrete, the formwork, reinforcement and any surface with which the fresh concrete will be in contact shall be free from snow, ice and frost and preferably should be at a temperature close to that of freshly placed concrete.

3.6- Concreting in hot weather
3.6.1- During hot weather, attention shall be given to ingredients, production methods, handling, placing protection, and curing to prevent excessive concrete temperatures or water evaportion that may impair required strength or serviceability of the member or structure.

3.6.2- Concrete temperature at the time of placing shall not exceed 40°C. Appropriate measures shall be taken to keep the concrete temperature within the specified temperature by cooling ingredients or any other means approved by the Engineer.

3.6.3- Concrete temperature may be calculated from the following formula:

$$T = \frac{0.22\,(T_a W_a + T_c W_c) + T_w W_w}{0.22\,(W_a + W_c) + W_w}$$

where

T = Temperature of freshly mixed concrete

T_a, T_c, T_w = temperature of aggregate, cement and mixing water respectively (°C)

W_a, W_c, W_w weight of aggregate, cement and mixing water respectively (kg).

3.6.4- After concrete placing and until the start of curing measures shall be taken to minimize evaporation, by covering concrete surfaces, fog spraying or any other means approved by the Engineer.

CHAPTER 4- FORMWORK, EMBEDDED PIPES AND JOINTS

4.1- Design of formwork

4.1.1- Forms shall result in a final structure that conforms to shapes, lines and dimensions of the members as required by the design and specifications.

4.1.2- Forms shall be substantial and sufficiently tight to prevent leakage of mortar.

4.1.3- Forms shall be properly braced or tied together to maintain position and shape.

4.1.4- Forms and their supports shall be designed so as not to damage previously placed stuctures.

4.1.5- Design of formwork shall include consideration of the following factors:

a) Rate and method of palacing concrete.

b) Construction loads, including vertical, horizontal, and impact loads.

c) Special form requirements for construction of shells, folded plates domes, architectural concrete, or similar types of elements.

4.2- Removal of forms and shores

4.2.1- Forms shall be removed in such a manner as not to impair safety and serviceability of the structure. All concrete to be exposed by form removal shall have sufficient strength not to be damaged thereby.

4.2.2- Sufficient strength shall be demonstrated by structural analysis considering proposed loads, strength of forming and shoring system, and concrete strength data. Concrete strength may be based on tests of field cured cubes or, when approved by the Engineer, on other procedures to evaluate concrete strength. Structural analysis and concrete strength test data shall be made available to the Engineer when so required.

4.2.3- No construction loads exceeding the combination of superimposed dead load plus specified live load shall be supported on any unshored portion of the structure under construction, unless analysis indicates adequate strength to support such additional loads.

4.3- Conduits and pipes embedded in concrete

4.3.1- Conduits, pipes and sleeves of any material not harmful to concrete may be embedded in concrete with approval of the Engineer, provided they are not considered to replace structurally the displaced concrete.

4.3.2- Conduits and pipes of aluminum shall not be embedded in structural concrete unless effectively coated or covered to prevent aluminmum concrete reaction or electrolytic action between alumimnum and steel.

4.3.3- Conduits and pipes, with their fittings embedded within acolumn shall not displace more than 4 percent of the area of cross section on which strength is calculated or which is required for fire protection.

4.3.4- Except when plans for conduits and pipes are approved by the Engineer, conduits and pipoes embedded within a slab, wall, or beam (other than those merely passing through) shall satisfy the following:

a) They shall not be larger, in outside dimenstion, than 1/3 the overall thickness of slab, wall, or beam in which they are embedded.

b) They shall not be spaced closer than 3 diameters or widths on center.

c) They shall not impair significantly the strength of the construction.

4.3.5- In addition to other requirements of Section 4.3, pipes that will contain liquid, gas, or vapor may be embedded in structural concrete under the following conditions:

a) Pipes and fittings shall be designed to resist effects of the material, pressure, and temperature to which they will be subjected.

b) Concrete cover for pipes and fittings shall not be less than 40 mm for concrete exposed to earth or weather, nor 20 mm for concrete not exposed to weather or in contact with ground.

c) Reinforcement with an area not less than 0.0025 times the area of concrete section shall be provided normal to the piping.

4.4- Construction joints

4.4.1- The number of construction joints shall be kept as few as possible consistent with reasonable precautions against shrinkage. Concreting shall be carried out continuously up to constrcution joints. The joints shall be at right angle to the general direction of the member.

4.4.2- Construction joints shall be so made and located as not to impair the strength of the structure. Provisions shall be made for transfer of shear and other forces through construction joints. Construction joints in floors shall be located within the middle third of spans of slabs, beams, and girders. Joints in girders shall be offset a minimum distance of two times the width of intersecting beams.

4.4.3- Beams, girders, or slabs supported by columns or walls shall not be cast or erected until concrete in the vertical support members is no longer plastic unless approved by the Engineer.

4.4.4- Beams, girders, haunches, drop panels and capitals shall be placed monolithically as part of a slab system, unless otherwise shown in design drawing or specification.

4.4.5- Surface of concrete construction joints shall be cleaned and laitance removed.

4.4.6- Immediately before new concrete is placed, all construction joints shall be wetted and standing water removed.

4.5- Movement joints

Movement joints are those specifically deigned and provided to permit relative movement of adjacent parts of a member or structure to occur without impairing the functional integrity of the member or structure. Their general function is to permit controlled movement to occur so as to prevent the build-up of harmful stresses. They may also be the connection joint between the several parts of a member or structure or they may be provided solely to permit translation or rotation or both.

4.5.1- Types of movement joint

Movement joints may be of the following types.

a) Contraction joint: A Contraction joint is a joint with a deliberate discontinuity but no initial gap between the concrete on both sides of the joint, the joint being intended to permit contraction of the concrete.

A distinction should be made between a complete contraction joint, in which both the concrete and reinforcement are interrupted, and a partial contraction joint, in which only the concrete is interrupted, the reinforcement running through.

b) Expansion joint: An expansion joint is a joint with complete discontinuity in both reinforcement and concrete and intended to accommodate either expansion or contraction of the structure.

In general, such a joint requires the provision of a sufficiently wide gap between the adjoining parts of a structure to permit the amount of expansion expected to occur. Design of the joint so as to incorporate sliding surfaces is not, however, precluded and may sometimes be advantageous.

c) Sliding joint: A Sliding joint is a joint with complete discontinuity in both reinforcement and concrete at which special provision is made to facilitate relative movement in the plane of the joint.

d) Hinged joint: A hinged joint is a joint specially designed and constructed to permit relative rotation of the members at the joint. This type of joint is usually required to prevent the occurrence of reverse moments or of undesirable restraint, for example in a three-hinged portal.

e) Settlement joint: A settlement joint is a joint permitting adjacent members or structures to settle or deflect relative to each other in cases, for example, where movements of the foundations of a building are likely due to mining subsidence. The relative movements may be large.

It may be necessary to design a joint to fulfil more than one of these items.

Joints in fire resistant walls or floors should be fire stopped to an equivalent degree of fire resistance.

4.5.2- Provision of joints

Cracking can be minimized by reducing the restraints on the free movement of the structure, and the control of cracking normally requires the subdivision of the structure into suitable lengths separated by the appropriate movement joints.

The effectiveness of movement joints in controlling crakking in a structure will also depend upon their precise location; this latter is frequently a matter of experience and may be characterized as the place where cracks would otherwise most probably develop, e.g. at abrupt changes of cross section.

The location of all movement joints should be clearly indicated on the drawings, both for the individual members and for the structure as a whole. In general, movement joints in the structure should pass through the whole structure in one plane.

4.5.3- Design of joints

A movement joint should fulfil all necessary functions. It should possess the merits of simplicity and freedom of movement, yet still retain the other appropriate characteristics necessary e.g. weatherproofness, fire resistance, resistance to corrosion, durability and sound insulation.

The design should also take into consideration the degree of control and workmanship and the tolerances likely to occur in the actual structure of the type being considered.

CHAPTER 5- STEEL REINFORCEMENT

5.0- Notation

d = distance from extreme compression fiber to centroid of tension reinforcement, mm

Ø = nominal diameter of bar or wire, mm

f_y = characteristic yield strength of steel reinforcement, N/mm^2

ℓ_d = development length, mm

5.1- Quality of reinforcing steel

5.1.1- Reinforcement shall be deformed reinforcement with physical and chemical properties and formations conforming to acceptable international standards*, except that plain reinforcement may be used for spirals or welded wire fabrics.

5.1.2- Design shall be based on characteristic strength of reinforcing steel given in Table 5.1 or a lower value if necessary to reduce deflection or control cracking.

5.1.3- Modulus of elasticity for reinforcing steel shall be taken equal to 200 KN/mm^2

TABLE 5.1- CHARCTERISTIC YIELD STRENGTH OF STEEL REINFORCEMENT

Type of reinforcement	Characteristic yield strength (f_y) N/mm^2
Plain and deformed mild steel bars	250-270
Medium tensile deformed steel bars	340-380
High yield deformed steel bars	410-550
Hard drawn steel wire and welded wire fabric	450-485
Rolled steel structural sections	235

5.2- Standard Hooks

The term "standard hook" as used in this code shall mean one of the following:

5.2.1- 180 - deg bend plus 4Ø extension, but not less than 60 mm at free end of bar.

5.2.2- 90 - deg bend plus 12Ø extension at free end of bar.

5.2.3- For stirrup and tie hooks

a) 16 mm bar and smaller, 90-deg bend plus 6Ø extension at free end of bar, or

b) 18 mm to 25 mm bar, 90-deg bend plus 12Ø extension at free end of bar, or

c) 25 mm bar and smaller, 135-deg bend plus 6Ø extension at free end of bar.

5.3- Minimum bend diameters

5.3.1- Diameter of bend measured on the inside of the bar, other than for stirrups and ties, shall not be less than the values in Table 5.3.

5.3.2- Inside diameter of bends for stirrups and ties shall not be less than 4Ø for 16 mm bar and smaller. For bars larger than 16mm, diameter of bend shall be in accordance with Table 5.3.

TABLE 5.3- MINIMUM DIAMETER OF BEND

Bar Size (mm)	Minimum diameter
6 through 25	6Ø
28,32 and 35	8Ø
42 and 56	10Ø

5.4- Bending

All reinforcement shall be bent cold, reinforcement partially embedded in hardened concrete shall not be field bent, except as shown on the design drawings or permitted by the Engineer.

5.5- Surface conditions of reinforcement

Steel reinforcement at the time concrete is placed shall be free from loose flakey rust, or other coatings that adversely affect bonding capacity.

5.6- Placing reinforcement

5.6.1- Reinforcement shall be accurately placed and adequately supported by concrete, metal, or other approved chairs, spacers, or ties and secured against displacement within tolerances permitted in Section 5.6.2.

5.6.2- Tolerance for depth **d**, and minimum concrete cover in flexural members, walls and compression members shall be as shown in Table 5.6. Except that tolerance for the clear distance to formed soffits shall be minus 6 mm and shall not exceed minus one-third the minimum concrete cover required in the drawings or in the specifications.

TABLE 5.6-TOLERENCE IN DEPTH (d) AND MINIMUM CONCRETE COVER

Depth of member mm	Tolerance on d (mm)	Tolerance on minimum Concrete cover (mm)
$d \leq 200$	± 10	- 10
$200<d\leq400$	± 15	- 15
$d > 400$	± 20	- 15

5.6.3- Tolerance for longitudinal location of bends and ends of reinforcement shall be ±50 mm except at dis-

* See appendix (A) for relevent internationally recognized standards

continuous ends of members where tolerance shall be ± 15 mm.

5.6.4- Welding of crossing bars shall not be permitted for assembly of reinforcement unless authorized by the Enginee.

5.7- Spacing limits for reinforcement

5.7.1- Clear distance between parallel reinforcement in a layer shall not be less than the largest bar diameter nor 25 mm. Clear distance must also be greater than 4/3 of the nominal maximum size aggregate used (see Section 1.3.3).

5.7.2- Where parallel reinforcement is placed in two or more layers, bars in the upper layer shall be placed directly above bars in the bottom layer with clear distance between layers not less than 25mm.

5.7.3- In spiraly reinforced or tied reinforced compression members clear distance between longitudinal bars shall not be less than 1.5Ø nor 35 mm provided the limitations given in Section 1.3.3 are observed.

5.7.4- Clear distance limitations between bars shall also apply to the clear distance between a contact lap splice and adjacent splices or bars.

5.7.5- In walls and slabs other than concrete joist construction, primary flexural reinforcement shall not be spaced farther apart than 2 times the wall or slab thickness, nor 350 mm.

5.8- Bundled bars

5.8.1- Groups of parallel reinforcing bars bundled in contact to act as a unit shall be limited to 4 in any one bundle.

5.8.2- Bundled bars shall be enclosed within stirrups or ties.

5.8.3- Bars larger than 32 mm shall not be bundled in beams.

5.8.4- Individual bars within a bundle terminated within the span of flexural members shall terminate at different points within at least 40 bar diameter stagger.

5.8.5- Where spacing limitations and minimum concrete cover are based on nominal bar diameter Ø, a unit of bundled bars shall be treated as a single bar of a diameter derived from the equivalent total area.

5.9- Concrete protection for reinforcement

5.9.1- Cast in-place concrete

The following minimum concrete cover shall be provided for reinforcement :

	Minimum cover, mm
a) Concrete cast against and permanently exposed to earth	75
b) Concrete exposed to earth or weather	
20 mm diameter bar and larger	50
below 20 mm diameter bar	40
c) Concrete not exposed to weather or in contact with earth:	
Slabs, walls, joists:	
40 mm diameter bar and larger	40
below 40 mm diameter bar	20
Beam, columns:	
Primary reinforcement, ties, strirrups and spirals	40
Shells, folded plates:	
20 mm diameter bar and larger	15
below 20 mm diameter bar	10

5.9.2- Precast concrete (manufactured under plant control conditions)

The following minimum concrete cover shall be provided for reinforcement

	Minimum cover, mm
a) Concrete exposed to earth or weather	
Wall panel:	
40 mm diameter bar or larger	40
below 40 mm diameter bar	20
Other members:	
40 mm diameter bar or larger	50
below 40 mm diameter and above 16 mm diameter bars	40
16 mm diameter bar and smaller	30
b) Concrete not exposed to weather or in contact with earth	
Slabs, walls, joists	
40 mm diameter bar and larger	30
below 40 mm diameter bar	15
Beams, columns:	
primary reinforcement diameter of bar but not less than	15
ties, stirrups, spirals	10
Shells, folded plates:	
16 mm diameter bar and smaller	10
other reinforcement diameter of bar but not less than	20

5.9.3- For bundled bars, minimum concrete cover shall be equal to the equivalent diameter of the bundle but need not be greater than 5 mm, except for concrete cast against and permanently exposed to earth, the minimum cover shall be 75 mm.

5.9.4- In corrosive environment or other severe exposure conditions, amount of concrete protection shall be suitably increased, and denseness and nonporosity of protecting concrete shall be considered, or other protection shall be provided.

5.9.5- When fire protection requires a thickness of cover greater than the minimum concrete cover specified in

Section 5.9, such greater thickness shall be used.

5.10- Shrinkage and temperature reinforcement

5.10.1-Reinforcement for shrinkage and temperature stresses normal to flexural reinforcement shall be provided in structural floor and roof slabs where the flexural reinforcement extends in one direction only.

5.10.2- Area of shrinkage and temperature reinforcement shall provided at least the following ratios of reinforcement area to gross concrete area, but not less than 0.0017:

a) Slabs where deformed bars with yield strength of 380 N/mm^2 or less are used 0.0025

b) Slabs where deformed bars or welded wire fabric (smooth or deformed) with yield strength of 410 N/mm^2 are used. 0.0022

c) Slabs where reinforcement with yield strength exceeding 410 N/mm^2 measured at a yield strain of 0.35 percent is used .... $\frac{0.0022 \times 410}{f_y}$

5.10.3- Where the concrete of a structural floor or roof slab cast and cured in summer ambient temperature exceeding 45°C, minimum steel percentages stipulated in Section 5.10.2 shall be increased at least 33 percent.

5.10.4- Shrinkage and temperature reinforcement shall not be spaced farther apart than 3 times the slab thickness, nor 350 mm.

5.10.5- At all sections where required, reinforcement for shrinkage and temperature stresses shall develop the characteristic yield strength $\mathbf{f_y}$ in tension in accordance with Chapter 12.

6.0- Notation

E_c = modulus of elasticity of concrete, N/mm²

E_s = modulus of elasticity of steel, N/mm²

f_{cu} = characteristic compressive strength of concrete, N/mm²

ℓ_n = clear span for positive moment or shear and average of adjacent clear spans for negative moment, mm

W_u = factored load per unit length of beam or unit area of slab

6.1- Design: objectives and general recommednations

6.1.1- Limit state design

The purpose of the design is the achievement of acceptable probabilities that the structure being designed will not become unfit for the use for which it is designed, i.e. that it will not reach the limit state. The characteristic strength and loads used in design take account of the variations in the strength and properties of the materials to be used and in the loads to be supported. Where the necessary data are not available, these characteristic values are based on an appraisal of experience. In addition, two partial safety factors are used, one for material strength and the other for loads and load effects.

6.1.2- Limit state requirements

6.1.2.1- General

All relevant limit states should be considered in the design so as to ensure an adequate degree of safety and serviceability. The usual approach will be to design on the basis of the most critical limit state and then check that the remaining limit states will not be reached.

6.1.2.2- Ultimate limit state

The strength of the structure should be sufficient to withstand the factored loads taking due account of the possibility of overturning or buckling.

The layout of the structure and the interaction between the structural members, should be such as to ensure a robust and stable design. The structure should be designed to support loads caused by normal function, but there should be a reasonable probability that it will not collapse catastrophically under the effect of misuse or accident. No structure can be expected to be resistant to the excessive loads or foces that could arise due to an extreme cause, but it should not be damaged to an extent disproportionate to the original cause.

6.1.2.3- Serviceability limit state

a) **Deflection**

Reinforced concrete members subjected to flexure shall be designed to have adequate stiffness to limit deflections or any deformations that may adversely affect the serviceability of the structure, the applied finishes or any non-structural construction such as partition walls.

b) **Cracking**

Cracking in reinforced concrete flexural members shall be controlled so as not to affect the appearance or durability of concrete.

6.1.2.4- Other limit states

The design must also meet other limit state requirements as appropriate, namely

a) **Vibration**; where there is likelihood of a structure being subjected to vibration from causes such as wind forces or machinery, measure should be taken to prevent discomfort, alarm or damage to the structure or its proper function.

b) **Fatigue**; when the imposed loading is predominantly cyclic in nature, effects of fatigue must be assessed and considered in design.

c) **Durability**; The recommendations in the code regarding concrete cover to reinforcement, acceptable crack widths, in association with minimum cement content and maximum water/cement ratios are intended to satisfy durability requirements of almost all structures. Where exceptionally severe environments are encountered, specialist litrature should be consulted and additional precautions may need to be considered.

d) **Fire resistance**; In designing structural members, the following must be considered with regards to fire resistance: retension of structural strength, resistance to penetration of flames and resistance to heat transmission.

6.2- Loading

6.2.1- Service load shall be in accordance with an internationally recognized standard.*

6.2.2- Consideration shall be given to effects of forces due to crane loads, vibration, impact, blast, shrinkage, temperature changes, creep and unequal settlement of supports.

* See appendix (A) for relevant internationally recognized standard

7.0- Notation

D = dead load or related internal moments and forces

E = load effects of earthquake, or related internal moments and forces

F = loads due to weight and pressures of fluids with well-defined densities and controllable maximum heights, or internal moments and forces

f_{cu} = characteristic compressive strength of concrete, N/mm²

f_y = characteristic yield strength of steel, N/mm²

H = lateral earth pressure or related internal moments and forces

L = live loads or related internal moments and forces

T = cumulative effects of temperature, creep, shrinkage and differential settlement

U = factored design load

W = wind load or related internal moments and forces

γ_m = partial safety factor on steel or concrete

7.1- General

Structures and structural members shall be designed to have design strengths at all sections at least equal to the required strengths calculated for the most severe combinations of the factored loads and forces as are stipulated in this code.

7.2- Factored loads

The factored design load **U** shall be taken as one or more of the following applicable combinations:

7.2.1- The design factored load which consists of dead load and live load only shall be at least equal to

$U = 1.4\,D + 1.7\,L$ .... (7–1)

7.2.2- If structural effects of a specified wind load **W** are to be included in design, the following combinations of **D**, **L**, and **W** shall be investigated to determine the greatest factored load.

$U = 1.1\,D + 1.3\,L + 1.3\,W$ .... (7-2)

where load combinations shall include both full value and zero value of **L** to determine the more severe condition, and

$U = 0.9\,D + 1.3\,W$ .... (7-3)

but for any combination of **D**, **L** and **W**, the factored load shall not be less than Eq. (7–1)

7.2.3- If specified earthquake load or forces **E** are included in design, load combinations of Section 7.2.2 shall apply, except that **1.1E** shall be substituted for **W**.

7.2.4- If earth pressure **H** is included in design, the design factored load shall at least equal to

$U = 1.4\,D + 1.7\,L + 1.7\,H$ .... (7–4)

except that where **D** or **L** reduce the effect of **H**, **0.9D** shall be substituted for **1.4 D** and zero value of **L** shall be used to determine the greatest factored load. For any combination of **D**, **L**, and **H**, the design factored load shall not be less than that of Section 7.2.1.

7.2.5- If loadings due to weight and pressure of fluids with well-defined densities and controllable maximum heights **F** are included in design, such loadings shall have a load factor of 1.4, and be added to all loading combinations that include live load.

7.2.6- If impact effects are taken into account in design, such effects shall be included with live load **L**.

7.2.7- Structural effects **T** of differential settlement, creep, shrinkage or temperature change when considered significant in design, shall be treated as dead load, but the design load shall not be less than

$U = 1.4\,D + 1.4\,T$ .... (7–5)

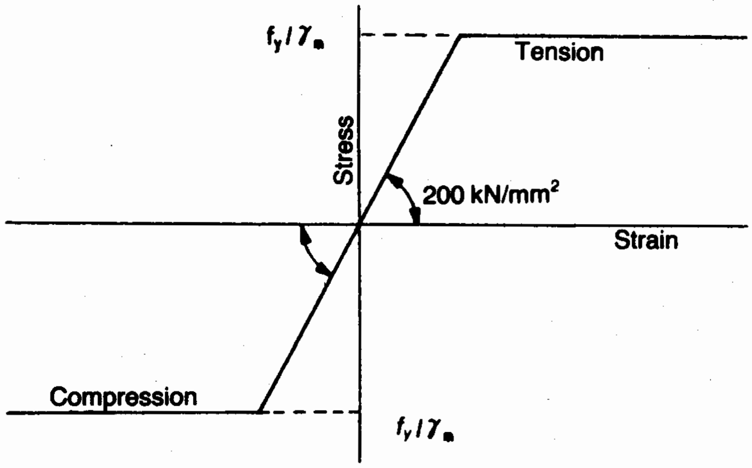
Estimations of differential settlement, creep, shrinkage, or temperature change shall be based on a realistic assessment of such effects occuring in service.

7.3- Design strength

7.3.1- Design strength provided by a member, its connections to other members and its cross sections, in terms of flexure, axial load, shear and torsion shall be based on the bilinear design stress – strain curves of reinforcing steel in tension and compression as shown in Fig. 7.3 (a) and the design compressive strength of concrete as shown in Fig 7.3(b); in addition to other assumptions and requirements of this code.

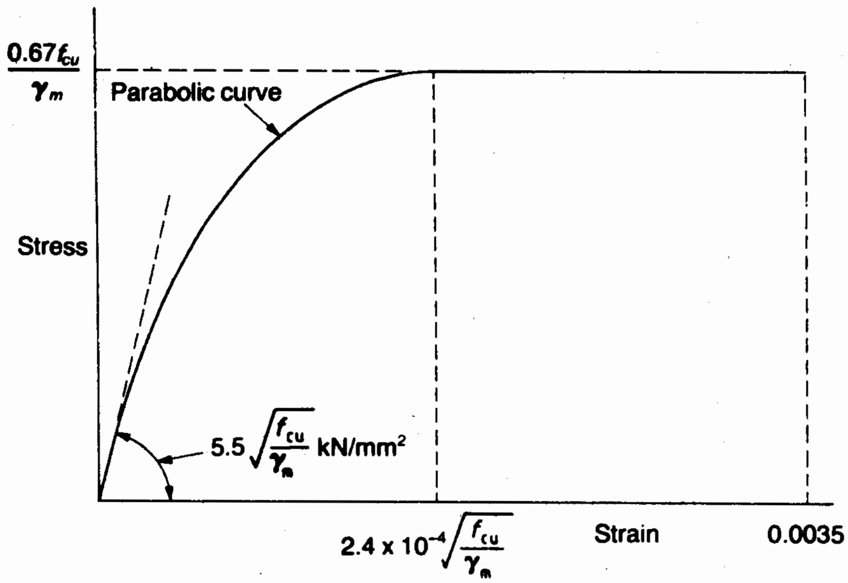
7.3.2- Design yield strength of reinforcing steel shall be based on the characteristic yield strength of steel f_y and the partial factor of safety γ_m equal to 1.15 as shown in Fig. 7.3 (a).

7.3.3- Design compressive strength of concrete shall be taken as the characteristic compressive strength of concrete divided by a partial safety factor γ_m equal to 1.5. The short term design stress – strain relationship for concrete is given in Fig. 7.3(b).



Note: f_y is in N/mm^2.

Figure 7.3(a) Short term design stress-strain curve for reinforcement



Note 1: 0.67 takes account of the relation between the cube strength and the bending strength in a flexural member. It is simply a coefficient and not a partial safety factor.

Note 2: f_{cu} is in N/mm^2.

Figure 7.3(b) Short term design stress-strain curve for normal-concrete

8.0— Notation

A = effective tension area of concrete surrounding the flexural tension reinforcement and having the same centroid as the reinforcement, divided by the number of bars or wires, mm^2. When the flexural reinforcement consists of different bar or wire sizes, the number of bars or wires shall be computed as the total area of reinforcement divided by the area of the largest bar or wire used

A_s = area of tension reinforcement mm^2

A'_s = area of compression reinforcement, mm^2

A_s (req) = area of tension reinforcement required, mm^2

A_s (prov) = area of tension reinforcement provided, mm^2

d = distance from extreme compression fiber to the centroid of tension reinforcement, mm

d_c = thickness of concrete cover measured from extreme tension fiber to the center of bar located closest thereto, mm

f_{cu} = characteristic compressive strength of concrete, N/mm^2

f_r = modulus of rupture of concrete, N/mm^2

f_y = characteristic yield strength of steel, N/mm^2

I_{cr} = moment of inertia of cracked section transformed to concrete

I_e = effective moment of inertia for computation of deflection

I_g = moment of inertia of gross concrete section about centroidal axis neglecting reinforcement

ℓ = effective span length of beam or one – way slab as defined in Section 9.1.1 or clear projection of cantilever, mm

M_a = maximum moment in member at stage deflection is being computed

M_{cr} = cracking moment

M_ℓ = actual moment at left hand support

M_o = maximum positive moment under a pin ended condition

M_r = actual moment at right hand support

W_{cr} = crack width due to flexure, mm

y_t = distance from centroidal axis of gross section neglecting reinforcement to extreme fiber in tension, mm

ρ_{req} = required percentage of tension steel

8.1- Control of deflections

8.1.1- Deflection which occurs immediately on application of the service load, shall be computed by usual methods and formulae for elastic deflection, considering effects of cracking and reinforcement on member stiffness, using the modulus of elasticity for concrete specified in Section 6.4 and the appropriate effective moment of inertia of concrete.

8.1.2- Effective moment of inertia I_e may be taken as

$$I_e = (M_{cr} / M_a)^3 (I_g - I_{cr}) + I_{cr} \quad \text{(8–1)}$$

but not greater than I_g where

$$M_{cr} = \frac{f_r\, I_g}{y_t} \quad \text{(8–2)}$$

and for normal weight concrete

$f_r = 0.55 \sqrt{f_{cu}}$, N/mm^2

8.1.3- For continuous spans, weighted average moment of inertia shall be used, which is obtained by multiplying the effective moment of inertia at mid span by $(1-\beta)$ and adding this value to the average of effective moment of interia at the supports multiplied by β where:

$$\beta = \left(\frac{M_\ell + M_r}{2\,M_o}\right)^3$$

8.1.4- In lieu of more accurate methods, additional long term deflection (including the effects of creep and shrinkage) for flexural members shall be obtained by multiplying the immediate deflection caused by the sustained load considered, by the factor

$$\left(2 - 1.2\frac{A'_s}{A_s}\right) \geq 0.6$$

8.1.5- Deflection computed in accordance with Section 8.1.1 through 8.1.4 shall not exceed limits specified in Table 8.1 (a)

8.1.6- Span to depth ratio

8.1.6.1- For a rectagular beam having a span of not more than 10 meters and not supporting nonstructural members likely to be damaged by large deflections, it shall be so deemed that the member complies with the requirements of this code for maximum permissible deflection, if the ratio of span to effective depth ℓ/d of the member does not exceed the basic ℓ/d ratio in Table 8.1 (b) multiplied by the factor:

$$\left(0.75 + \frac{0.0025}{\rho_{req}}\right)$$

where ρ_{req} is the ratio of tension steel required to carry the ultimate load, considered at the center of the span in case of beams and at the support in case of cantilevers. For larger spans deflections should be calculated to comply with the requirements of Table 8.1(b).

TABLE 8.1 (a)-MAXIMUM PERMISSIBLE COMPUTED DEFLECTIONS

Type of member	Deflection to be considered	Deflection limitation
Roof and floor construction a) not intended to support or be attached to non-structural members, or b) supporting or attached to non-structural members not likely to be damaged by deflection.	Final deflection (including the long term deflection measured below the as cast level)	$\frac{\ell}{200}$
Floors not supporting or not attached to non-structural elements likely to be damaged by large deflections.	Immediate deflection due to live load L	$\frac{\ell^{*}}{375}$
Roof or floor construction supporting or attached to non-structural elements likely to be damaged by large deflections.	That part of the total deflection occurring attachment of non-structural elements (sum of long term deflection due to all sustained loads and the immediate deflection due to any additional live loads)†	$\frac{\ell^{**}}{500}$
Roof or floor construction supporting or attached to non-structural elements not likely to be damaged by large deflections.		$\frac{\ell^{\S}}{250}$

* Limit not intended to safeguard against ponding. Ponding should be checked by suitable calculations of deflections, including added deflections due to ponder water, and considering long-term effects of all sustained loads, camber, construction tolerance and reliability of provisios for drainage.

** Limit may be exceeded if adequate measures are taken to prevent damage to supported or, attached elements.

† Long-term deflection shall be determined in accordance with Section 8.1.4 but may be reduced by amount of deflection calculated to occur before attachment of non-structural elements. This amount shall be determined on basis of accepted engineering data relating to time-deflection characteristics of members similar to those being considered.

§ But not greater than tolerance provided for non-structural elements. Limit may be exceeded if camber is provided so that total deflection minus camber does not exceed limit.

Table 8.1(b) BASIC SPAN/EFFECTIVE-DEPTH RATIO FOR PRISMATIC BEAMS OF RECTANGULAR CROSS-SECTION*

Support Condition	ℓ/d Ratio
Cantilever	7
Simply supported	18
Continuous	23

* If the charasteric yield stress f_y for steel is other than 410 N/mm² the ℓ/d ratio obtained above, shall be further multiplied by: $\left(0.3 + \frac{290}{f_y}\right)$

8.1.6.2- The **ℓ/d** ratio specified in Section 8.1.6.1 shall also apply to solid slabs except that the reinforcement at the center of the span in the width of the slab under consideration shall be considered to influence the deflection.

In case of two – way slabs the ratio shall be based on the shorter span and its amount of reinforcement in that direction.

8.2- Control of cracking

8.2.1- The calculated crack widths for members subjected to flexure, shall not exceed the limits specified in Table 8.2.

TABLE 8.2- LIMITATIONS ON CRACK WIDTHS UNDER FLEXURE

Exposure condition	Upper limit on crack width at the surface nearest to main reinforcement (mm)
Dry air or protective membranee	0.40
Humidity, moist air, soil	0.30
Seawater and seawater spray; wetting, drying aggressive soil	0.15

8.2.2- Provisions of Section 8.2.1 may not be sufficent for structures subjected to very aggressive exposure. For such structures, special investigations and precautions are required.

8.2.3- Unless values are obtained by a more comprehensive analysis, the crack widths shall be calculated by the formula

$$W_{cr} = 13 \times 10^{-6} f_s (d_c A)^{1/3} \quad (8\text{-}3)$$

where

$$f_s = 0.6 f_y \frac{A_s(req)}{A_s(prov)}$$

8.2.4- Maximum spacing of bars near the tension face of slabs shall not exceed the limit specified in Section 5.7.5.

8.2.5- Provisions for tension reinforcement to distribute cracking arising from shrinkage and temperature effect as given in Section 5.10 shall also be followed.

8.2.6- Provisions in Section 8.2.4 and 8.2.5 shall be deemed to comply with the requirements of Section 8.2.1, for normal internal and external conditions of exposure.

8.2.7- Tensile reinforcement shall be distributed in zones of maximum concrete tension including these portions of flanges of T-beams, L-beams and I-beams over a support.

8.2.8- When flanges of T- beam construction are in tension, part of the flexural tension reinforcement shall be distributed over an effective flange width or as defined in Section 9.1.2 or a width equal to 1/10th of the span whichever is smaller.

8.2.9- When the overall depth of a beam exceeds 750mm, additional longitudinal reinforcement having a total area equal to at least 10 percent of the area of flexural tension reinforcement shall be placed near the side faces of the beam and distributed over a distance of 2/3rd of the overall depth of the beam from tension face with a spacing not more than the web width nor 300mm. Such reinforcement may be included in strength calculation in shear or torsion taking into account strain compatibility.

9.0- Notation

A_s = area of tension reinforcement, mm^2
A'_s = area of compression reinforcement, mm^2
b = width of compression face of a member, mm
b_c = width of compression face midway between restraints, mm
b_e = effective width, mm
b_w = width of web in a flanged section, mm
d = distance from extreme compression fiber to centroid of tension reinforcement (effective depth), mm
d' = distance from extreme compression fiber to centroid of compression reinforcement, mm
E_c = modulus of elasticity of concrete, N/mm^2
E_s = modulus of elasticity of steel, N/mm^2
f_{cu} = characteristic compressive strength of concrete, N/mm^2
f_y = characteristic yield strength of tension steel, N/mm^2
h = overall depth or thickness of member, mm
h_f = overall thickness of flange, mm
ℓ= effective span, m
M_u = ultimate moment of resistance
z = lever arm, mm
β_{red} = ratio of the reduction in resistance moment, to the numerically largest moment given anywhere by the elastic maximum moments diagram for that particular member, covering all appropriate combinations of ultimate loads
γ_m = partial safety factor for material
ρ = ratio of tension reinforcement to effective area of cross section
ρ_{max} = maximum allowable ratio of tension reinforcement in singly reinforced rectangular sections.
Ø = bar size or diameter, mm

9.1- General Principles and Requirements

9.1.1- Effective span

9.1.1.1- The effective span of a simply supported member shall be taken as the clear span plus effective depth of member but need not exceed the distance between centers of supports.

9.1.1.2- In continuous construction, the effective span shall be taken as the distance center to center of supports.

9.1.1.3- For beams built integrally with supports, moments at faces of support may be used for design.

9.1.1.4- Solid or ribbed slabs built integrally with supports with clear span not more than 3m,may be analysed as continuous slabs on knife edge supports with effective spans equal to the clear spans of the slabs.

9.1.2- Effective width of fanged flexural members

9.1.2.1- In absence of any more accurate methods, the effective flange width of a T-beam shall not exceed:

a) The width of the web plus one seventh of the effective span for continuous beams and one fifth the effective span for simply supported or cantilevers, nor

b) The web width plus a width of slab on each side of the web not exceeding 8 times the slab thickness nor ½ the clear distance to the next web.

c) Isolated beams, in which the T-shape is used to provide a flange for additional compression area shall have a flange thickness not less than ½ the width of web and an effective flange width not more than 4 times the width of web.

9.1.2.2- For beams with a slab on one side only, the effective flange width shall not exceed:

a) **the width of the web plus one – fourteenth of the span for continuous beams and one – tenth of the span for simply supported or cantilever beams, nor**

b) **the actual width of the flange**

9.1.3- Distance between lateral support of flexural member

9.1.3.1- Spacing of lateral support for a beam shall not exceed

$$50\ b_c \text{ nor } \frac{250}{d}\ b_c^2$$

9.1.3.2- For a cantilever with lateral restraint provided only at the support, the clear distance from the end of the cantilever to the face of the support shall not exceed

$$25\ b_c \text{ nor } \frac{100}{d}\ b_c^2$$

9.1.4- Minimum reinforcement of flexural members

9.1.4.1- At any section of a flexural member, except as provided in Sections 9.1.4.2 and 9.1.4.3, where positive reinforcement is required by analysis the ratio ρ provided shall not be less than that given by

$$\rho_{min} = \frac{1.4}{f_y} \quad \text{.......} \quad (9-1)$$

In T-beams and joists where the web is in tension, the ratio ρ shall be computed for this purpose using width of web.

9.1.4.2- Alternatively, area of reinforcement provided at every section, positive or negative, shall be at least one – third greater than that required by analysis.

9.1.4.3- For structural slabs of uniform thickness, minimum area and maximum spacing of reinforcement in the direction of the span shall be as required for shrinkage and temperature according to Section 5.10.

9.1.4.4- In a flanged beam the amount of transverse reinforcement provided within the flange and across the full effective width of the fange, expressed as a percentage of the longitudinal cross sectional area of the flange shall not be less than 0.3%. The spacing of transverse reinforcement shall not exceed that required for temperature and shrinkage reinforcement according to Section 5.10.

9.1.5- Maximum area of reinforcement in flexural members

In a flexural member the area of tension reinforcement shall not exceed ρ_{max} where

$$\rho_{max} = 0.23\,\frac{f_{cu}}{f_y} + \frac{f'_s}{0.87 f_y}\,\frac{A'_s}{db}$$

9.2- Moment of resistance of flexural member

9.2.1- Design Assumptions

9.2.1.1- The strength of flexural members shall be based on assumptions given in Section 7.3 and 9.2 and on satisfaction of applicable conditions of equilibrium and compatibility.

9.2.1.2- Strains in reinforcement and concrete shall be assumed directly proportional to the distance from the neutral axis, except for deep flexural members as defined in Section 9.2.5, where non linear distribuition of strain shall be considered.

9.2.1.3- Maximum strain at extreme concrete compression fiber at failure shall be taken as 0.0035.

9.2.1.4- The stress in the concrete in compression shall be derived from the stress strain curve in figure 7.3(b) with the partial factor of safety $\gamma_m = 1.5$.

9.2.1.5- Requirements of Section 9.2.1.4 may be considered satisfied by using the simplified rectagular stress block for concrete shown in Fig. 9.2

9.2.1.6- The tensile strength of concrete shall be neglected.

9.2.1.7- The stress in reinforcement shall be derived from the stress strain curves in Fig. 7.3(a) with partial factor of safety $\gamma_m = 1.15$.

9.2.1.8- Modulus of elasticity for concrete and steel shall be taken as per Section 6.4.

9.2.1.9- Compression reinforcement in conjunction with additional tension reinforcement may be used to increase the strength of flexural members.

9.2.1.10- In case a beam is required to resist combined bending moment and a small axial thrust, the effect of ultimate axial force may be ignored if it is less than **0.1** $f_{cu} A_g$.

9.2.1.11- In case a beam is required to resist combined bending moment and axial tension, reinforcement shall be designed to carry all the tensile stresses.

9.2.2- Design formulae

In lieu of using the concrete stress – strain curve defined in Fig. 7.3(b) the following formulae may be used to calculate the ultimate moment of resistance of a solid slab or rectangular beam or a flanged beam, ribbed slab or void slab when the neutral axis lies within the flange. These formulae based on the simplified stress block of **Fig 9.2**

For values of $k \leq 0.156$ (This implies a limitation of the neutral axis to d/2)

$$M_u = (0.87\, f_y)\, A_s\, z \qquad (9\text{-}2)$$

where

$$z = d\{0.5 + \sqrt{(0.25 - k/0.9)}\} \qquad (9\text{-}3)$$

$$k = \frac{M_u}{b\, d^2 f_{cu}}$$

but shall not exceed the value of

$$M_u = 0.156\, f_{cu}\, b\, d^2 \qquad (9\text{-}4)$$

For values of $k > 0.156$ compression reinforcement will be required, and

$$A'_s = (k - 0.156) f_{cu} b d^2 / 0.87 f_y (d - d') \qquad (9\text{-}5)$$

$$A_s = 0.156 f_{cu} b d^2 / 0.87 f_y z + A'_s \qquad (9\text{-}6)$$

where

$$z = 0.78\, d \qquad (9\text{-}7)$$

provided that $d'/d \leq 0.25$ and $f_y \leq 410$ N/mm^2

If $(d'/d) \geq 0.25$ or f_y is greater than 410 N/mm^2, the compression stress may be less than $0.87\, f_y$ and should be obtained from Fig. 7.3(b).

For the case of flanged beams where the neutral axis lies below the flange and the design ultimate moment does not exceed $(\beta_f\, f_{cu}\, b\, d^2)$, **the required area of tension reinforcement may be calculated using the following equation.**

$$A_s = \frac{M_u + 0.1\, f_{cu} b_w d\, (0.45\, d - h_f)}{0.87\, f_y\, (d - 0.5\, h_f)} \qquad (9\text{-}8)$$

If the design ultimate moment exceeds $\beta_f f_{cu} b\, d^2$ then the section shall be analysed with the aid of Section 9.2.1. The values of β_f are given in Table 9.2.

Equation (9-8) is only applicable when $h_f < 0.45\, d$

The values in Table 9.2 are calculated from the following equation:

$$\beta_f = 0.45\,\frac{h_f}{d}\,\left(1 - \frac{b_w}{b}\right)\left(1 - \frac{h_f}{2d}\right) + \left(0.15\,\frac{b_w}{b}\right)$$

9.2.3- Distribution of flexural reinforcement

Flexural tension reinforcement shall be well distributed as per Section 8.2 in order to control flexural cracking in beams and slabs with due regard to spacing limitations and minimum concrete cover.

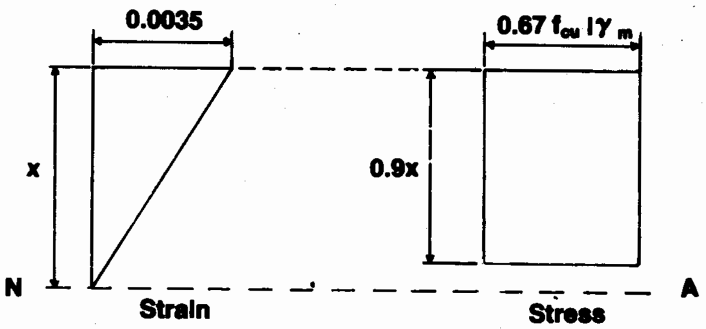


Figure 9.2 Simplified stress block for concrete at ultimate limit state

9.2.4- Lateral reinforcement for flexural members

9.2.4.1- Compression reinforcement in beams shall be enclosed by ties or stirrups satisfying the size and spacing limitations in Section 10.6.5 or by welded wire fabric of equivalent area. Such ties or stirrups shall be provided throughout the distance where compression reinforcement is required.

TABLE (9-2) VALUES OF THE FACTOR β_t

b/b_w	d/h_f					
	≤2	3	4	5	6	∞
1	0.15	0.15	0.15	0.15	0.15	0.15
2	0.15	0.14	0.12	0.12	0.11	0.08
4	0.15	0.13	0.11	0.10	0.09	0.04
6	0.15	0.13	0.11	0.09	0.08	0.03
8	0.15	0.13	0.10	0.09	0.8	0.02
∞	0.15	0.13	0.10	0.08	0.07	0.00

9.2.4.2- Lateral reinforcement for flexural framing members subject to stress reversal or to torsion shall consist of closed ties, closed stirrups, or spirals extending around the flexural reinforcement.

9.2.4.3- Closed ties or stirrups may be formed in one piece by overlapping standard stirrup or tie end hooks around a longitudinal bar, or formed in one or two pieces lap spliced or anchored in accordnce with Section 12.12.

9.2.4.4- Lateral reinforcement shall also conform with the requirements of shear and torsion as per Chapter 11.

9.2.5- Deep flexural members

9.2.5.1- Flexural members with clear span to overall depth ratios less than 2.5 for continuous spans or 2 for simple spans, shall be designed as deep flexural members taking into account non – linear distribution of strain and lateral buckling.

9.2.5.2- In lieu of a more accurate analysis, the following equations for the determination of the ultimate moment of resistance may be used.

a) for simply supported spans

$$M_u = 0.25\, h\, (3 - \frac{h}{\ell})\, A_s f_y \qquad 0.5 < \frac{h}{\ell} < 1.0$$

$$M_u = 0.5\, \ell\, A_s f_y \qquad \frac{h}{\ell} \geq 1.0$$

b) for continuous beams

$$M_u = 0.44h\, (1.8 \frac{h}{\ell})\, A_s f_y \qquad 0. < \frac{h}{\ell} < 1.0$$

$$M_u = 0.35\, \ell\, A_s\, f_y \qquad \frac{h}{\ell} \geq 1.0$$

c) for cantilever beams

$$M_u = (0.56\ell + 0.09h)\, A_s\, f_y \qquad 1.0 < \frac{h}{\ell} < 2.0$$

$$M_u = 0.75\, \ell\, A_s\, f_y \qquad \frac{h}{\ell} \geq 2.0$$

The positive reinforcement is to be uniformly distributed over a depth of **0.1 ℓ** or **0.1 h,** whichever is smaller, of the tension face.

The negative reinforcement over a support shall be uniformly spread over a zone with a depth equal to the lesser of **0.6ℓ** and **0.6 h**. This zone shall start at a level above the bottom located at **0.1ℓ** or **0.1 h** whichever is smaller.

9.2.5.3- Minimum flexural tension reinforcement shall conform to Section 9.1.4.1.

9.2.5.4- Flexural tesnion reinforcement required to resist positive bending moment in any span of a deep beam shall extend without curtailment between supports.

9.2.5.5- At least 50% of the flexural tension reinforcement required to resist negative bending moment over a support of a deep beam shall be extended over the full spans.

9.2.5.6- Design of deep flexural members for shear effects shall be in accordance with Section 11.7.

9.2.5.7- Minimum horizontal and vertical reinforcement in the side faces of deep flexural members shall be greater of the requirements of Sections 11.6.3.5 and 11.6.3.6 or Sections 14.2.2.2 and 14.2.2.3.

9.2.6- Anchorage, splices and curtailment of flexural reinforcement shall be carried out in accordance with Chapter 12.

10.0- Notation

A_c = area of core of spirally reinforced compression member measured to outside diameter of spiral, mm^2
A_g = gross area of section, mm^2
A_s = area of tension reinforcement, mm^2
A'_s = area of compression reinforcement, mm^2
A_{st} = total area of longitudinal reinforcement (bar or steel shapes), mm^2
A_t = area of structural steel shape, pipe or tubing in a composite section, mm^2
A_1 = loaded area mm^2
A_2 = maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, mm^2
b = width of compression face of a member, mm
C_m = factor relating actual moment diagram to an equivalent uniform moment diagram
d = distance from extreme compression fiber to centroid of tension reinforcement (effective depth), mm
d' = distance from extreme compression fiber to centroid of compression reinforcement, mm
e_{min} = minimum eccentricity, mm
e_o = eccentricity of column load, corresponding to equivalent uniform moment
E_c = modulus of elasticity of concrete, N/mm^2
E_s = modulus of elasticity of steel, N/mm^2
EI = Flexural stiffness of compression member, $N \cdot mm^2$
f_{cu} = characteristic compressive strength of concrete, N/mm^2
f_y = characteristic yield strength of tension steel, N/mm^2
h = overall depth or thickness of member, mm
I_g = moment of inertia of gross cross section of member about its centroidal axis, mm^4
I_{se} = moment of inertia of reinforcement about centroidal axis of cross section of member, mm^4
I_t = moment of inertia of structural steel shape, pipe or tubing about centroidal axis of cross section of composite member, mm^4
K = effective length factor for compression members
k = flexural stiffness of member = EI/ℓ
ℓ = effective span, m
ℓ_u = unsupported length of compression member, m
M_c = factored moment to be used in design of compression member
M_u = ultimate moment of resistance
M_1 = value of smaller factored end moment on compression member calculated by conventional elastic frame analysis, positive if member is bent in single curvature, negative if bent in double curvature
M_2 = value of larger factored end moment on compression member calculated by conventional elastic frame analysis, always positive
P_b = ultimate compressive axial load at balanced strain conditions
P_c = Euler's critical load
P_u = ultimate compressive axial load at given eccentricity
P_o = ultimate compressive axial load at zero eccentricity
P_{ux} = ultimate compressive axial load with zero eccentricity about Y-axis
P_{uy} = ultimate compressive axial load with zero eccentricity about X-axis
r = radius of gyration of cross section of a compression member
δ = magnification factor
δ_c = critical magnification factor
ρ = ratio of tension reinforcement
ρ_s = ratio of volume of spiral reinforcement to total volume of core (out to out of spirals) of spirally reinfoced compression member
γ_m = partial safety factor for material
α_{c_1} = ratio of the sum of column stiffnesses to the sum of the beam stiffnesses at lower end of column
α_{c_2} = ratio of the sum of column stiffnesses to the sum of the beam stiffnesses at upper end of column
$\alpha_{c\,min}$ = lesser of α_{c_1} and α_{c_2}
β_d = ratio of maximum factored dead load moment to maximum factored total moment, always positive
β_{red} = ratio of the reduction in resistance moment, to the numerically largest moment given anywhere by the elastic maximum moments diagram for that particular member covering all appropriate combinations of ultimate loads
$\varnothing$ = bar size or diameter, mm

10.1- General Principles and Requirement

10.1.1- Moments and forces in columns

10.1.1.1- Columns shall be designed to resist the axial forces from factored loads on all floors and roofs and maximum moment from factored loads on a single adjacent span of the floor or roof under consideration. Loading condition giving the maximum ratio of moment to axial load shall also be considered.

10.1.1.2- Forces and moments in the column shall be determined from appropriate elastic analysis of the structure. Such analysis shall take into account influence of axial loads and variable moment of inertia on member stiffness and fixed end moments, effect of deflections on moments and forces and the effects of duration of loads.

10.1.1.3- In lieu of the procedure prescribed in Section 10.1.1.2, evaluation of moments and forces in columns may be made in accordance with the approximate procedure presented in Section 10.1.2.

10.1.2- Approximate evalution of moments and forces

10.1.2.1- In column and beam construction not intended to resist lateral loads, the axial force in a column may be calculated on the assumption that beams and slabs transmitting force into the column are simply supported.

10.1.2.2- When a column is axially loaded or the axial force dominates as in case of columns supporting symmetrical arrangement of beams, only the factored axial force need be considered in design apart from a nominal allowance for eccentricity due to construction tolerances.

10.1.2.3- When a column is not symmetrically loaded the ultimate moments induced in it may be calculated by simple moment distribution, Kani's method of analysis etc. on the assumption that the column and beam ends remote from the junction under consideration are fixed and that the beams on either side possess half their actual stiffness.

10.1.2.4- In designing slender columns an allowance shall be made for increased moments due to slenderness effects as per Section 10.3 and Fig. 10.1(a) and 10.1(b).

10.1.2.5- For unbraced frames if the average value of $K\ell_u/h$ for all columns at a paticular level is greater than 20, bases or other members connected to the ends of such columns shall also be designed to resist the respective magnified end moment induced due to slenderness effects as indicated in Fig. 10.1(b).

10.1.3- Braced and unbraced columns

A compression member may be assumed braced in a given plane if located in a story which the bracing elements (shear walls, trusses or other types of lateral bracings) have a total stiffness resisting lateral movement of the story at least 6 times the sum of stiffnesses of all the columns within the story.

10.1.4- Unsupported length of compression members

10.1.4.1- Unsupported length ℓ_u of a compression member shall be taken as the clear distance between end restraints such as floor slabs, beams or other members.

10.1.4.2- When column capitals or hunches are present, unsupported length shall be measured to the lower extremity of cpaital or haunch in the plane considered.

10.1.5- Effective length of compression members

10.1.5.1- The effective length factor **K** for cantilever compression member shall be taken as 2.0.

10.1.5.2- For a braced column where relative lateral displacement of the ends is prevented by shear walls, bracings or similar means, the effective length factor **K** for the member shall be taken as:

a) 0.75, Where both ends are restrained against rotation, or

b) 0.85, where one end is restrained against rotation, or

c) 1.00, where both ends are unrestrained against rotation.

10.1.5.3.- In a framed structure the effective length factor shall be obtained from:

a) For a braced cloumn, the lesser of

$$K = [0.70 + 0.05\,(\alpha_{c_1} + \alpha_{c_2})] \leq 1 \quad \text{(10-1)}$$

or

$$K = [0.85 + 0.05\,\alpha_{c_{min}}] \leq 1 \quad \text{(10-2)}$$

b) For an unbraced column, the lesser of

$$K = [1.0 + 0.15\,(\alpha_{c_1} + \alpha_{c_2})] \quad \text{(10-3)}$$

or

$$K = [2.0 + 0.3\,\alpha_{c_{min}}] \quad \text{(10-4)}$$

10.1.5.4- In case the connection between a column and its base is not designed to resist other than nominal moment or when the beams framing into a column are designed as simply supported, α_c in Section 10.1.5.3 shall be taken as 10.

10.1.5.5- If a base is designed to resist the column moment, α_c in Section 10.1.5.3 shall be taken as 1.0.

10.1.5.6- In calculating α_c in Section 10.1.5.3, the stiffness of each member shall be obtained by dividing the product of moment of inertia of its concrete section and Modulus of elasticity by its ctual length, i.e. $k = EI/\ell$

10.1.5.7- For flat slab constrution, an equivalent beam shall be taken as having the width and thickness of the slab forming the column strip.

10.1.6- Radius of Gyration

The radius of gyration of a cross section of a compression member shall be calculated on the gross concrete cross section, except that it shall be permissible to assume **r** as being:

a) 0.3 times the overall dimension in the direction in which the stability is being considered, for a rectangular section and,

b) 0.25 times the diameter, for a circular section.

10.1.7- Slenderness limits for columns

10.1.7.1- For a compression member the effects of slenderness may be neglected when $K\ell_u/r$ is less than 35.

10.1.7.2- The unsupported length ℓ_u of a column should not exceed

a) 50 times the least dimension of the cross section nor **250 b^2/h** in case of braced and unbraced columns, and

b) 25 times the least dimension of the gross section nor **100 b^2/h** in case of cantilever columns, where **b** is

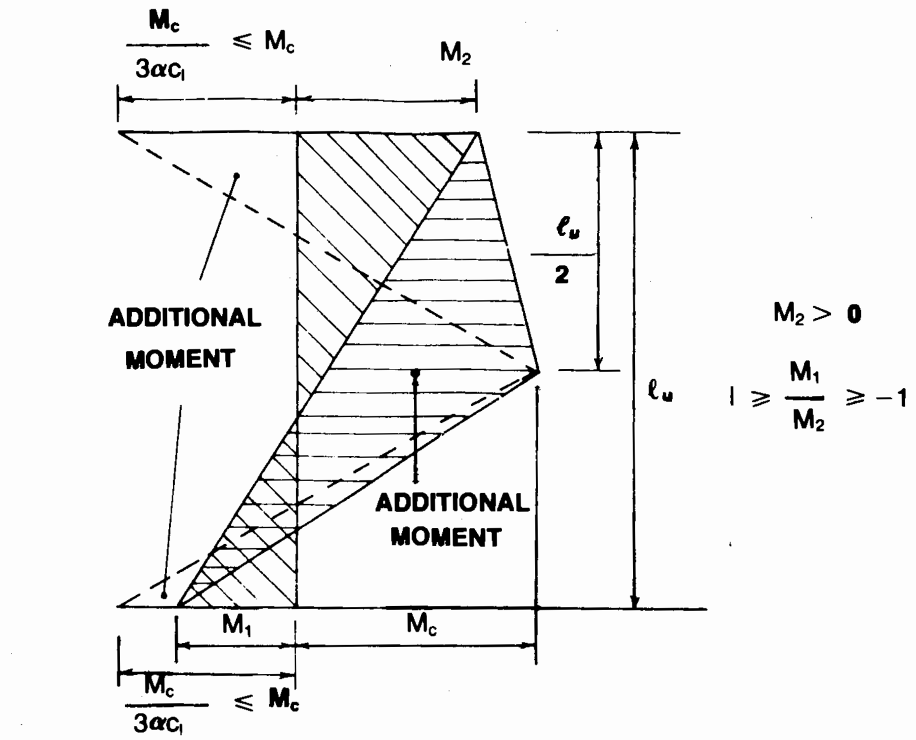


Fig. 10.1(a) BENDING MOMENT DIAGRAM FOR BRACED LONG COLUMNS.

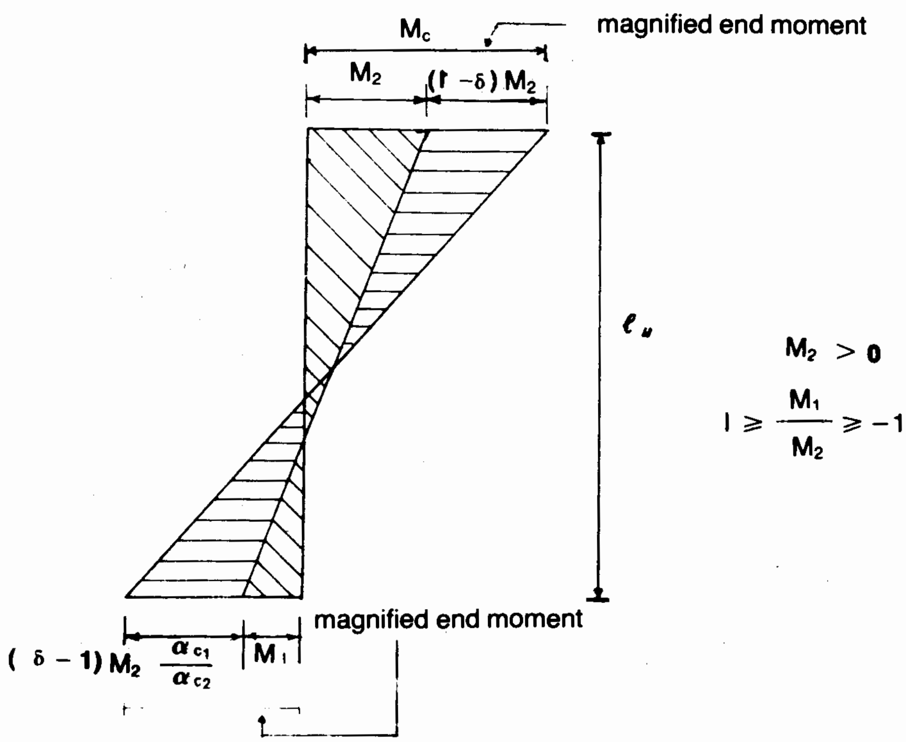


Fig. 10.1(b) BENDING MOMENT DIAGRAM FOR UNBRACED LONG COLUMNS.

$$M_c = \delta\, C_m\, M_2 \qquad (10\text{-}10)$$

Provided

$M_c \nless M_2$

where

$$\delta = \frac{1}{1 - \dfrac{P_u}{P_c}} \geq 1.0 \qquad (10\text{–}11)$$

but not greater than δ_c as defined in Section 10.3.3.4, and

$$P_c = \frac{\pi^2 EI}{(K\ell_u)^2} \qquad (10\text{–}12)$$

The variation of M_c along the column length both for braced and unbraced conditions have been indicated in Fig. 10.1(a) and 10.1(b). The dotted line in Fig.10.1(a) indicates the restraining effects of the beams at the column ends.

10.3.3.2- In lieu of more accurate calculation, **EI** in Eq. (10-12) may be taken as the greater of the following:

$$EI = \frac{E_c I_g / 6 + E_s I_{se}}{1.4\,(1 + \beta_d)} \qquad (10\text{–}13)$$

or

$$EI = \frac{E_c I_g}{3\,(1 + \beta_d)} \qquad (10\text{–}14)$$

10.3.3.3- In Eq. (10-10), for members braced against sidesway and without transverse loads between supports, C_m may be taken as

$$C_m = 0.60 + 0.40\,\frac{M_1}{M_2} \qquad (10\text{–}15)$$

but not less than 0.40

For all other cases, C_m shall be taken as 1.0.

10.3.3.4- In Eq. (10-10) the magnification factor shall be limited to the value of δ_c given by Eq. (10-16). In case it exceeds δ_c, the section shall be revised

$$\delta_c = 1 + \frac{(K\ell_u / h)^2}{1800\,(e_o / h)} \qquad (10\text{–}16)$$

10.3.3.5- Unless otherwise stated, the minimum eccentricity e_{min} shall be taken as **0.05h** mm but not less than 20 mm.

10.3.3.6- If computations show that there is no moment at both ends of a compression member or that computed end eccentricities are less than e_{min}, M_2 in Eq. (10-15) shall be based on eccentricity of e_{min} about each axis separately. Ratio M_1/M_2 in Eq. (10-15) shall be determined by either of the following:

a) when computed end eccentricities are less than e_{min}, computed end moments may be used to evaluate M_1/M_2 in Eq. 10-15.

b) If computations show that there is essentially no moment at both ends of a compression member, the ratio M_1/M_2 shall be taken equal to unity.

10.3.4- Moment magnifier δ for unbraced frames

10.3.4.1- In frames not braced against sidesway, the value of δ shall be coumputed for an entire story assuming all columns to be loaded.

10.3.4.2- In Eq. (10-11), P_u and P_c shall be replaced by the summation ΣP_u and ΣP_c for all columns in a story.

10.3.4.3- For the design of each column within a story, δ shall be taken as the larger of the values computed for the entire story according to Section10.3.4.2 or as computed for the individual column assuming column ends to be braced against sidesway.

10.3.5- Slender columns bent about both axes

Slender compression members subjected to bending about both principal axes shall be designed for axial force P_u and the magnified moment M_c about each principal axis separately.

10.4- Axially loaded members supporting slab system

Axially loaded members supporting a slab system shall be designed as provided in Chapter 10 and in accordance with the additional requirement of Chapter 13.

10.5- Transmission of column loads through floor system

When the characteristic compressive strength of concrete in a column is greater than 1.4 times that specified for a floor system, transmission of load through the floor system shall be provided by one of the following:

10.5.1- Concrete of strength specified for column shall be placed in the floor about the column for an area 4 times the column area. Column concrete shall be well integrated into floor concrete.

10.5.2- Strength of a corner or edge column through a floor system shall be based on the lower value of concrete strength.

10.5.3- For columns laterally supported on four sides by beams of approximately equal depth or by slabs, strength of the column may be based on an assumed concrete strength in the columns joint equal to 75 percent of column concrete strength plus 35 percent of floor concrete strength.

10.6- Lateral reinforcement for compression members

10.6.1- Lateral reinforcement for compression members shall conform to the provisions of Section 10.6.4 and 10.6.5 and where shear or torsion reinforcement is required, shall also conform to provisions of Chapter 11.

10.6.2- Lateral reinforcement requirements for composite compression memebers shall conform to Section 10.8.

10.6.3- Lateral reinforcement requirements of Section 10.6 may be waived where tests and structural analysis show that adequate strength and feasibility of construction can be achieved.

10.6.4- Spirals

Spiral reinforcement for compression members shall conform to Section 10.1.9.3 and to the following:

10.6.4.1- Spirals shall consist of evenly spaced continuous bar or wire of such size and so assembled to permit handling and placing without distortion from design dimensions.

10.6.4.2- For cast-in-place construction, size of spirals shall not be less than 6mm in diameter.

10.6.4.3- Clear spacing between spirals shall not exceed 75mm, nor be less than 25mm. See also Section 1.3.3.

10.6.4.4- Anchorage of spiral reinforcement shall be provided by 1½ extra turns of spiral bar or wire at each end of spiral unit.

10.6.4.5- Splices in spiral reinforcement shall be lap splices of 48Ø but not less than 300mm, or otherwise welded.

10.6.4.6- Spirals shall extend from top of footing or slab in any story to the level of lowest horizontal reinforcement in members supported above.

10.6.4.7- where beams and brackets do not frame into all sides of a column, ties shall extend above termination of spiral to bottom of slab or drop panel.

10.6.4.8- In columns with capitals, spirals shall extend to a level at which the diameter or width of capital is 2 times that of the column.

10.6.4.9- Spirals shall be held firmly and true to line by vertical spacers. For spiral bar or wire smaller than 16mm, a mimimum of 2 spacers shall be used for spirals less than 50mm in diameter, three spacers for spirals 500 to 750mm in diameter, and four spacers for spirals greater than 750mm in diameter.

10.6.4.10- For spiral bar or wire 16mm diameter or larger, a minimum of three spacers shall be used for spirals 600mm or less in diameter, and four spacers for spirals greater than 600mm in diameter.

10.6.5- Ties

Tie reinforcement for compression members shall conform to the following:

10.6.5.1- All longitudinal bars shall be enclosed by lateral ties, at least 6mm in size for longitudinal bars of up to 20mm, 8mm in size for bars exceeding 20mm and up to 25mm, 10mm in size for bars exceeding 28mm and up to 42mm and 12mm in size for bars exceeding 42mm and up to 56mm and bundled longitudinal bars. Deformed wire or welded wire fabric of equivalent area may be used.

10.6.5.2- Vertical spacing of ties shall not exceed 16 longitudinal bar diameters, 48 tie or wire diameters or least dimension of the compression member .

10.6.5.3- Ties shall be arranged such that every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie with an included angle of not more than 135 degree and no bar shall be farther apart than 150 mm clear on each side along the tie from such a laterally supported bar. Where longitudinal bars are located around the perimeter of a circle, a complete circular tie may be used.

10.6.5.4- Ties shall be located vertically not more than half a tie spacing above the top of footing or slab in any story and shall be spaced as provided herein to not more than half a tie spacing below the lowest horizantal reinforcement in members supported above.

10.6.5.5- Where beams or brackets frame into all sides of a column, ties may be terminated not more than 75mm below lowest reinforcement in shallowest of such beams and brackets.

10.7- Offset bars in columns

Offset bent longitudinal bars shall conform to the following:

10.7.1- Slope of inclined portion of an offset bar with axis of column shall not exceed 1 in 6.

10.7.2- Portions of bar above and below an offset shall be parallel to the axis of column.

10.7.3- Horizontal support at offset bends shall be provided by lateral ties, spirals or parts of the floor construction. Horizantal support provided shall be designed to resist 1½ times the horizontal component of the computed force in the inclined portion of an offset bar. Lateral ties or spirals if used, shall be placed not more than 100mm from points of bend.

10.7.4- Offset bars shall be bent before placement in the form.

10.7.5- Where a column face is offset 75mm or greater, longitudinal bars shall not be offset bent. Separate dowels, lap spliced with the longitudinal bars adjacent to the offset column faces shall be provided. Lap splices shall conform to Section 12.16.

10.8- Composite compression members

10.8.1- Composite compression members shall include all such members reinforced longitudinally with structural steel shapes, pipe or tubing with or without longitudinal bars.

10.8.2- For composite compression members with spiral or tie reinforcement around the steel core, the following shall be satisfied:

10.8.2.1- Characteristic compressive strength of concete f_{cu} shall not be less than 20 N/mm^2.

10.8.2.2- Design characteristic yield strength of structural steel shall be the specified minimum yield strength for grade of structural steel used but not to exceed 340 N/mm^2.

10.8.2.3- Area of longitudinal reinforcing bars located within ties or spirals shall not be less than 0.01 nor more than 0.08 times net area of concrete section.

10.8.2.4- Longitudinal bars located within the spirals may be used in calculating A_t and I_t.

10.8.2.5- Spiral reinforcement shall conform to Section 10.1.9.3.

10.8.2.6- Lateral ties shall extend completely around the structural steel core.

10.8.2.7- Lateral ties shall have a diameter not less than 1/50 times the greatest side dimension of the composite member but not smaller than 6mm. Welded wire fabric of equivalent area may be used.

10.8.2.8- Vertical spacing of lateral ties shall not exceed 16 times the longitudinal bar diameter, 48 tie bar diameter or 1/2 times the least side dimension of the composite member.

10.8.2.9- A longitudinal bar shall be located at every corner of a rectangular cross section, with other longitudinal bars spaced not farther apart than one half the least side dimension of the composite member.

10.8.3- Load transfer in structural steel cores of composite compression members shall be provided by the following:

10.8.3.1- Ends of structural steel cores shall be accurately finished to bear at end bearing splices, with provision for alignment of one core above the other in concentric contact.

10.8.3.2- The end bearing splices shall be considered effective to transfer not more than 50 percent of the total compressive stress in the steel core.

10.8.3.3- Base of structural steel section shall be designed to transfer the total load from the entire composite member to the footing or the base may be designed to transfer the load from the steel section only, provided ample concrete section is available for transfer of the portion of the total load carried by the reinforced concrete section to the footing by compression in concrete and by reinforcement.

10.8.3.4- Transfer of stress between column base and footing shall be in accordace with Section 15.8.

10.8.4- Strength of a composite member shall be computed for the same limited conditions applicable to ordinary reinforced concrete members.

10.8.5- Any axial load strength assigned to concrete of a composite member shall be transferred to the concrete by members or brackets in direct bearing on concrete of the composite member.

10.8.6- All axial load strength not assigned to concrete of a composite member shall be transferred by direct connection to the steel section.

10.8.7- For evaluation of slenderness effects, radius of gyration of a composite section shall not be greater than the value given by:

$$r = \sqrt{\frac{(E_c I_g/7) + E_s I_t}{(E_c A_g/7) + E_s A_t}} \qquad (10\text{-}17)$$

For computing P_c in Eq. (10-12) EI of a composite section shall not be greater than:

$$EI = \frac{(E_c I_g / 7) + E_s I_t}{1 + \beta_d} \qquad (10\text{–}18)$$

10.8.8- Structural steel encased concrete core

10.8.8.1- For a concrete core encased by structural steel, thickness of the steel encasement shall not be less than

$b\sqrt{f_y / 3E_s}$, for each face of width **b**

nor

$h\sqrt{f_y / 8E_s}$

10.8.8.2- Longitudinal bars located within the encased concrete core may be considered in computing A_t and I_t

10.9- Bearing Strength

Design bearing strength of concrete shall not exceed $0.45\, f_{cu}\, A_1$ except as follows:

10.9.1 - When the supporting surface is wider on all sides than the loaded area, design bearing strength on the loaded area may be multiplied by $\sqrt{A_2/A_1}$ but not more than 2.

10.9 2- When the supporting surface is sloped or stepped, A_2 may be taken as the area of the lower base of the largest frustrum of a right pyramid or cone contained wholly within the support and having for its upper base the loaded area, and having side slopes of 1 vertical to 2 horizontal.

CHAPTER 11- SHEAR AND TORSION

11.0- Notation

a = shear span, distance between concentrated load and face of support, mm

A_c = area of concrete section resisting shear transfer, mm^2

A_f = area of reinforcement necessary to resist moment (V_u a) in a corbel or bracket, mm^2

A_g = gross area of section, mm^2

A_h = area of shear reinforcement parallel to flexural tension reinforcement, mm^2

A_ℓ = total area of longitudinal reinforcement to resist torsion, mm^2

A_n = area of reinforcement necessary to resist horizontal tensile force N_{uc} in a corbel or bracket, mm^2

A_s = area of tension reinforcement, mm^2

A_t = area of one leg of a closed stirrup resisting torsion within a distance **s**, mm^2

A_v = area of shear reinforcement within a distance **s** or area of shear reinforcement perpendicular to flexural tension reinforcement within a distance **s** for deep flexural members, mm^2

A_{vf} = area of shear friction reinforcement, mm^2

A_{vh} = area of shear reinforcement parallel to flexural tension reinforcement within a distance s_2, mm^2

b = width of compression section for slabs and footings, mm

b_o = perimeter of critical section for slabs and footings, mm

b_w = web width, mm

c_1 = size of rectangular or equivalent rectangular column, capital or bracket, measured in the direction of the span for which moments are being determined, mm

c_2 = size of rectagular or equivalent rectangular column, capital or bracket, measured transverse to the direction of the span for which moments are being determined, mm

C_t = factor relating shear and torsional stress properties $= b_w d / \Sigma x^2 y$

d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement

f_{cu} = characteristic compressive strength of concrete, N/mm^2

f_{ct} = average splitting tensile strength of concrete, N/mm^2

f_y = characteristic tensile strength of reinforcing steel, N/mm^2

$f_{y\ell}$ = characteristic strength of longitudinal steel for torsion, N/mm^2

f_{yv} = characterisitic strength of shear reinforcement, N/mm^2

G = shear modulus, N/mm^2

h = overall thickness of member, mm

h_v = total depth of shearhead cross section, mm

h_w = total height of wall from base to top, mm

I = moment of inertia of section resisting externally applied factored loads, mm^4

J = torsional constant, mm^4

ℓ_d = development length, mm

ℓ_n = clear span measured face to face of supports, mm

ℓ_s = distance between face of support and point of zero shear, mm

ℓ_v = length of shearhead arm from the centroid of concentrated load or reaction, mm

ℓ_w = horizontal length of wall, mm

M_o = bending moment necessary to produce zero stress in concrete fiber at which tensile stresses are caused by applied loads

M_p = required plastic moment of shearhead cross section

M_u = factored moment at section

M_v = moment resistance contributed by shearhead reinforcement

N_u = factored axial load normal to cross section occuring simultaneously with V_u to be taken as positive for compression, negative for tension and to include effects of tension due to creep and shrinkage

N_{uc} = factored rensile force applied at top of bracket or corbel acting simultaneously with V_u, to be taken as positive for tension

s = spacing of shear or torsion reinforcement in direction parallel to longitudinal reinforcement, mm

s_1 = spacing of vertical reinforcement in wall, mm

s_2 = spacing of shear or torsion reinforcement in direction perpendicular to longitudinal reinforcement or spacing of horizontal reinforcement in wall, mm

T_c = torsional moment strength provided by concrete

T_s = torsional moment strength provided by torsion reinforcement

T_u = factored torsional moment at a section

v_c = shear stress provided by concrete

V_c = shear strength provided by concrete

V_{ci} = shear strength provided by concrete when diagonal cracking results from combined shear and moment

V_{cu} = shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web

V_h = shear force provided by shear reinforcement

V_s = shear strength provided by shear reinforcement

V_u = factored shear force at section

x = shorter overall dimension of a rectangular part of a cross section, mm

y = longer overall dimension of a rectangular part of a cross section, mm

Σx^2y = torsional section properties

x_1 = shorter center to center dimension of a closed rectangular stirrup, mm

y_1 = longer center to center dimension of a closed rectangular stirrup, mm

y_t = distance from centroidal axis of gross section, neglecting reinforcement to extreme fiber in tension, mm

α = angle between inclined stirrups and longitudinal axis of a member

α_t = coefficient as a function of y_1/x_1

α_v = ratio of stiffness of shearhead arm to surrounding composite slab section

β_c = ratio of long side to short side of concentrated load or reaction area

μ = coefficient of friction

γ = angle between the shear friction reinforcement and the direction of shearing action

γ_v = fraction of unbalanced moment transferred by eccentricity of shear at slab cloumn connections

ρ = ratio of tension reinforcement = $A_s/b\,d$

ρ_h = ratio of horizontal shear reinforcement area to gros concrete area of vertical section

ρ_n = ratio of vertical shear reinforcement area to gross concrete area of horizontal section

$\rho_v = (A_s + A_n)/b\,d$

$\rho_w = A_s/b_w d$

Note: Coefficients in nonhomogenious equations are based on stresses in N/mm^2 and dimensions in mm.

11.1- Shear strength

11.1.1- Design of cross section subject to factored shear force V_u shall be based on

$$V_u \leq V_s + V_c \quad \text{(11-1)}$$

Where V_c is the shear strength provided by concrete in accordance with Section 11.2

11.1.2- Maximum factored shear force at sections located less than a distance **d** from face of support may be designed for the same shear V_u as that computed at a distance **d** provided that the following conditions are satisfied:

a) support reaction, in direction of applied shear, introduced compression into the end regions of member, and

b) no concentrated load occurs between face of support and distance **d**, otherwise the critical section shall be at face of support.

11.2- Shear strength provided by concrete

11.2.1- Shear strength V_c at any cross section of a member subject to shear and flexure shall be calculated from,

$$V_c = v_c b_w d \quad \text{(11-2)}$$

where the shear stress v_c shall be taken as

$$v_c = (0.05 + 7.5\rho_w)\sqrt{f_{cu}} \quad \text{(11-3)}$$

but not greater than $0.14\sqrt{f_{cu}}$

11.2.2- In determining shear strength V_c, the effects of axial tension due to creep and shrinkage in restrained members whenever applicable shall be considered.

11.2.3- For members subject to axial compression, Eq. (11-4) shall be used to compute the shear force V_c carried by concrete

$$V_c = v_c\left(1 + 3.40\frac{N_u}{A_g f_{cu}}\right) b_w d \quad \text{(11-4)}$$

but not greater than $1.5\, v_c b_w d$

11.2.4- For members subject to axial tension exceeding $0.04\sqrt{f_{cu}}\, A_g$, shear reinforcement shall be designed to carry the total shear. In case axial tensile forces were considered in the design of longitudinal reinforcement, V_c may de computed using Eq. (11-5)

$$V_c = v_c\left(1 - \frac{N_u}{0.44\sqrt{f_{cu}}\, A_g}\right) b_w d \quad \text{(11-5)}$$

11.2.5- All sections where factored torsional moment T_u exeeds $0.025\sqrt{f_{cu}}\ \Sigma x^2y$, the shear v_c may be computed using Eq. (11-6).

$$V_c = \frac{v_c b_w d}{1 + 2.8\dfrac{T_u}{V_u}C_t} \quad \text{(11-6)}$$

11.3- Shear strength provided by shear reinforcement

11.3.1- Shear reinforcement may consist of the following types:

a) Stirrups perpendicular to the axis of the member, or making an angle of 45 deg or more with the longitudinal tension reinforcement

b) Bent – up longitudinal bars with bent angle not less than 30 deg with the longitudinal reinforcement. In this case it is not to exceed 50 percent of shear strength provided by shear reinforcement.

c) Welded wire fabric with wires perpendicular to the axis of the member.

d) Spirals.

11.3.2- Characteristic yield strength of shear reinforcement shall not exceed 410 N/mm².

11.3.3- Shear reinforcement shall extend to a distance d from extreme compression fiber and shall be anchored at both ends in order to develop the design yield strength of reinforcement.

11.3.4- Spacing limitations for shear reinforcement

11.3.4.1- Spacing of shear reinforcement placed perpendicular to the axis of the member shall not exceed the limitations given below nor 600 mm.

for	maximum spacing
$V_s < V_c$	3/4 d
$V_c \leq V_s \leq 2 V_c$	1/2 d
$2 V_c < V_s$	1/4 d

11.3.4.2- Inclined stirrups and bent longitudinal reinforcement shall be so placed that every 45 degree line shall be crossed by at least one line of shear reinforcement. Such reinforcement shall not be spaced farther apart than maximum spacing of Section 11.3.4.1 multiplied by a factor **$(1+\cot\alpha)$**, where α is the inclination of stirrups to the longitudinal axis of the member.

11.3.5- Minimum Shear Reinforcement

11.3.5.1- A minimum area of shear reinforcement shall be provided in all reinforced concrete flexural members where the factored shear force V_u exceeds 1/2 the shear strength provided by concrete V_c, except,

a) Slabs and footings.

b) Concrete joist construction

c) Beams with total depth not greater than 250 mm, 2½ times thickness of flange, or 1/2 the width of web, whichever is greater.

11.3.5.2- Minimum shear reinforcement requirements of Section 11.3.5.1 may be waived if shown by test that required flexural and shear strength can be achieved without shear reinforcement.

11.3.5.3- For reinforced concrete members, the minimum shear reinforecement when required shall be

$$A_v = \frac{0.40\, b_w\, s}{f_y} \qquad (11-7)$$

11.3.5.4- Where factored torsional moment T_u exceeds $0.025\sqrt{f_{cu}}\ \Sigma x^2 y$, minimum area of closed stirrups shall be computed by

$$A_v + 2A_t = \frac{0.4\, b_w\, s}{f_y} \qquad (11-8)$$

11.3.6- Design of shear reinforcement

11.3.6.1- Where factored shear force V_u exceeds the shear strength of concrete V_c, shear reinforcement shall be provided for the shear strength V_s, where

$$V_s = V_u - V_c \qquad (11-9)$$

Shear strength V_s shall be computed in accordance with Sections 11.3.6.1 through 11.3.6.7.

11.3.6.2- When shear reinforcement perpendicular to the axis of the member is used,

$$V_s = \frac{A_v (0.87 f_y)\, d}{s} \qquad (11-10)$$

where A_v is the area of shear reinforcement within a distance s.

11.3.6.3- When shear reinforcement consists of a series of parallel inclined bars, the shear strength V_s shall be given by,

$$V_s = \frac{A_v (0.87 f_y)(\sin\alpha + \cos\alpha)\, d}{s} \qquad (11-11)$$

11.3.6.4- Where shear reinforcement consists of a single bar or a single group of parallel bars, all bent up at the same distance from the support, A_s shall be calculated from:

$$A_s = A_v (0.87 f_y) \sin\alpha \qquad (11-12)$$

but not greater than $0.20\sqrt{f_{cu}}\, b_w d$.

11.3.6.5- Only the center three—fourth of the inclined portion of any longitudinal bent bar shall be considered effective for shear reinforcement.

11.3.6.6- When more than one type of shear reinforcement is used to reinforce the same portion of a member the shear strength V_s shall be computed as the sum of the V_s values computed for the various types.

11.3.6.7- Shear strength V_s shall not be taken greater than $4 V_c$.

11.4- Combined shear and torsion strength

11.4.1- Torsional effects shall be included with shear and flexure where the factored torsional moment T_u exceeds $0.025\sqrt{f_{cu}}\ \Sigma x^2 y$, otherwise torsional effects may be neglected.

11.4.2- For members with rectangular or flanged sections, the value of $\Sigma x^2 y$ shall be taken for the component rectangles of the section, but the over—hanging flange width used in the design shall not exceed 3 times the flange thickness.

11.4.3- A rectangular box section may be taken as a solid section provided wall thickness h is at least x/4. A box section with wall thickness between x/4 and x/10, may also be taken as a solid section except that $\Sigma x^2 y$ shall be multiplied by 4h/x. When h is less than x/10, stiffness of wall shall be considered. Fillets shall be provided at interior corners of all box sections.

11.4.4- When it is considered that torsional stiffness of members shall be taken into account in the analysis, the torsional rigidity GJ of a member may be obtained by assuming the shear modulus G equal to 0.40 times the modulus of elasticity of the concrete and torsional constant J equal to half the St. Venant value calculated for the plain concrete section.

11.4.5- In lieu of more exact analysis of an indeterminate structure, where reduction of torsional moment in a member can occur due to redistribution of internal forces, maximum factored torsional moment T_u may be reduced to $0.25\sqrt{f_{cu}}\,\Sigma x^2y/3$ and the adjoining members shall be designed for the adjusted moments and shears.

11.4.6- Torsional loading from a slab may be taken as uniformly distributed along the member.

11.4.7- Sections located less than a distance d from the face of a support may be designed for the same torsional moment T_u as that computed at a distance d.

11.4.8- Torsional moment strength provided by concrete

11.4.8.1- Torsional moment T_c for a section consisting of rectangles shall be computed by

$$T_c = \frac{0.05\sqrt{f_{cu}}\,\Sigma x^2 y}{1 + \frac{0.4}{C_t}\left(\frac{V_u}{T_u}\right)} \qquad (11-13)$$

11.4.8.2- T, L or I sections shall be divided into their component rectangles so as to maximize the value of Σx^2y.

11.4.8.3- For members subjected to axial tension in excess of $0.04\sqrt{f_{cu}}\,A_g$, torsional reinforcement shall be designed to carry the total torsional moment.

11.4.9- Torsional reinforcement requirements

11.4.9.1- Torsional reinforcement, where required, shall be provided in addition to reinforcement required to resist shear, flexure and axial forces.

11.4.9.2- Torsional reinforcement may be combined with that required for other forces, provided the area furnished is the sum of individually required areas and the most restrictive requirements for spacing and placement are met.

11.4.9.3- Torsional reinforcement shall consist of closed stirrups, closed ties or spirals combined with longitudinal bars.

11.4.9.4- Characteristic yield strength for torsional reinforcement shall not exceed 410 N/mm².

11.4.9.5- Torsional reinforcement shall extend to a distance d from extreme compression fibre and shall be anchored according to Section 12.12 to develop the design yield strength.

11.4.9.6- Torsional reinforcement shall be provided at least a distance $(b_t + d)$ beyond the point theoretically required.

11.4.10- Spacing limits for torsional reinforcement

11.4.10.1- spacing of closed stirrups shall not exceed the smaller of $(x_1 + y_1)/4$ or 300 mm.

11.4.10.2- Longitudinal torsion reinforcement shall not be less than 10 mm in diameter and shall be distributed evenly around the inside perimeter of the stirrups. The clear distance between these bars shall not exceed 300 mm and at least one bar in each corner of the links shall be provided.

11.4.10.3- Torsional reinforcement required at the level of tension or compression reinforcement may be provided by using larger bars than those required for bending alone.

11.4.11- Design of torsion reinforcement

11.4.11.1- When factored torsional moment T_u exceeds the torsional moment T_c, torsion reinforcement shall be provided for the strength T_s where

$$T_s = T_u - T_c \qquad (11-14)$$

11.4.11.2- The torsional moment T_s shall be computed by

$$T_s = \frac{\alpha_t A_t x_1 y_1 (0.87 f_{yv})}{s} \qquad (11-15)$$

where A_t is the area of one leg of a closed stirrup resisting torsion within a distance s and

$$\alpha_t = \left(2 + \frac{y_1}{x_1}\right)/3$$

but not more than 1.5.

11.4.11.3- A minimum area of closed stirrups shall be provided in accordance with Section 11.3.5.4.

11.4.11.4- Longitudinal bars A_l distributed around the perimeter of the closed stirrups shall be computed by

$$A_l = 2A_t\left(\frac{x_1 + y_1}{s}\right)\left(\frac{f_{yv}}{f_{yl}}\right) \qquad (11-16)$$

or by

$$A_l = \frac{2b_w(x_1 + y_1)}{f_{yl}} \qquad (11-17)$$

whichever is greater.

11.4.11.5- The torsional moment T_s shall not exceed $4T_c$.

tive Eq. (11-30) shall not apply.

11.8.6- Section located closer to wall base than a distance $\ell_w/2$ or one-half the height whichever is less may be designed for the same V_c as that computed at a distance $\ell_w/2$ or one-half the height.

11.8.7- When factored shear force V_u is less than $V_c/2$, reinforcement shall be provided in accordance with Section 11.8.8 or in accordance with chapter 14. When V_u exceeds $V_c/2$ wall reinforcement for resisting shear shall be provided in accordance with Section 11.8.8.

11.8.8- Shear reinforcement for walls

11.8.8.1- Where factored shear force V_u exceeds V_c, horizontal shear reinforcement shall be provided to satisfy Eq. (11-1) where shear strength V_s shall be computed by

$$V_s = \frac{A_v (0.87 f_y) d}{s_2} \qquad (11\text{-}31)$$

where A_v is the area of horizontal shear reinforcement within a distance s_2 and the distance d shall be in accordance with Section 11.8.4.

11.8.8.2- Ratio ρ_h of horizontal shear reinforcement area to gross concrete area of vertical section shall not be less than 0.0025.

11.8.8.3- Spacing of horizontal shear reinforcement s_2 shall not exceed $\ell_w/5$, 3h nor 350 mm.

11.8.8.4- Ratio ρ_v of vertical shear reinforcement area to gross concrete area of horizontal section shall not be less than:

$$\rho_v = 0.5\left(2.5 - \frac{h_w}{\ell_w}\right)\rho_h \qquad (11\text{-}32)$$

nor 0.0025, but need not be greater than the required horizontal shear reinforcement.

11.8.8.5- Spacing of vertical shear reinforcement s_1 shall not exceed $\ell_w/3$, 3h nor 350 mm.

11.9- Provisions for slabs and footings

11.9.1- The shear strength of a slab or footing in the vicinity of a concentrated force shall be governed by the more severe of two conditions:

11.9.1.1- Beam action for slab or footing with a critical section extending in a plane across the entire width and located at a distance d from the face of concentrated load or reaction area. For this condition, the slab or footing shall be designed in accordance with sections 11.1 through 11.3.

11.9.1.2- Two-way action for slab or footing with a critical section perpendicular to the plane of slab and located so that its perimeter b_o is a minimum, but need not approach closer than d/2 to the perimeter of concentrated load or reaction area. For this condition, the slab or footing shall be designed in accordance with sections 11.9.2 through 11.9.4.

11.9.2- Design of slab or footing for two-way action shall be based on V_c given by.

$$V_c = \frac{1}{8}\left(1 + \frac{2}{\beta_c}\right)\sqrt{f_{cu}}\, b_o\, d \qquad (11\text{-}33)$$

but need not be greater than $0.25\sqrt{f_{cu}}\, b_o\, d$

where β_c is the ratio of long side to short side of concentrated load or reaction area and b_o is the perimeter of critical section.

11.9.3- Shear reinforcement consisting of bars or wires may be used in solid slabs and footings of at least 200 mm thick in accordance with the following provisions:

11.9.3.1- The ultimate shear capacity V_u shall be given by

$$V_u = \frac{V_c}{2} + V_s \qquad (11\text{-}34)$$

where V_c shall be calculated as per Section 11.9.2 and required A_v and V_s shall be calculated as per Section 11.3 and anchored in accordance with Section 12.12.

11.9.3.2- The ultimate shear strength V_u shall not be taken greater than $0.40\sqrt{f_{cu}}\, b_o\, d$, where b_o is the perimeter of the critical Section defined in Section 11.9.1.2

11.9.3.3- Shear strength shall be investigated at the critical section defined in Section 11.9.1.2 and at successive sections more distant from the support.

11.9.4- Shearheads in slabs and footings

Shear reinforcement consisting of steel I or channel shapes may be used in slabs. Provisions of Sections 11.9.4.1 through 11.9.4.10 shall apply where shear due to gravity load is transferred at interior columns supports. Where moment is transfered to columns, Section 11.10.2.3 shall apply. Where shear is transfered at edge or corner columns, special designs shall be required.

11.9.4.1- Each shearhead shall consist of steel shapes fabricated by welding into four identical arms at right angles and continuous through the column section.

11.9.4.2- The depth of the steel shape shall not be more than 70 times the thickness of its web.

11.9.4.3- Every compression flange of the shearhead shall be located within 0.3d of the compression surface of the slab.

11.9.4.4- The ends of the arms may be cut tapered at anlges not less than 30 degrees with the horizontal.

11.9.4.5- The stiffness of each arm shall not be less than 0.15 times the stiffness of the surrounding composite cracked slab section of width $(c_2 + d)$ including the shearhead.

11.9.4.6- Plastic moment strength for each arm of shearhead shall be computed by

$$M_p = \frac{V_u}{8}\left[h_v + \alpha_v\left(\ell_v - \frac{c_1}{2}\right)\right] \qquad (11\text{-}35)$$

where ℓ_v is the minimum length of each shearhead arm required to comply with the requirements of Sections 11.9.4.7 and 11.9.4.8.

11.9.4.7- Critical slab section for shear shall be perpendicular to the plane of slab and shall cross each shearhead arm at 3/4 the distance $(\ell_v - \frac{c_1}{2})$ from column face to the end of shearhead arm. Critical section shall be located so that its perimeter b_o is a minimum, but need not approach closer than d/2 to the perimeter of column section.

11.9.4.8- The ultimate shear strength V_u on the critical section defined in Section 11.9.4.7 shall not be taken greater than V_c calculated in section 11.9.2. When shearhead reinforcement is provided, shear strength V_u shall not be taken greater than $0.4\sqrt{f_{cu}}\, b_o d$ on the critical section defined in Section 11.9.1.2.

11.9.4.9- The shearhead may be assumed to contribute a moment of resistance M_v to each slab column strip computed by

$$M_v = \frac{\alpha_v V_u}{8}\left(\ell_v - \frac{c_1}{2}\right) \qquad (11\text{-}36)$$

but not more than the least of

a) 30 percent of total factored moment required for each slab column strip.

b) change in column strip moment over length ℓ_v.

c) value of M_p computed by Eq. (11-35).

11.9.4.10- When unbalanced moments are considered, shearhead must have adequate anchorage to transmit M_p to column.

11.9.5- Openings in slabs

when openings transverse to the plane of the slab or discontinuous edges are located closer to the periphery of the concentrated load or reaction area than four times the slab thickness or twice the development length ℓ_d of slab reinforcement passing through the column or over the shearhead, account shall be taken of the reduction in shear caused by the openings or edges.

11.9.5.1- For slabs without shearheads, the part of the perimeter of the critical section that is enclosed by straight lines, projecting from the centroid of the load or reaction area and tangent to the boundaries of the openings shall be considered ineffective.

11.9.5.2- For slabs with shearheads, the ineffective portion of the perimeter shall be one half of that defined in Section 11.9.5.1.

11.10- Transfer of moments to columns

11.10.1- General

11.10.1.1- When unbalanced gravity forces or wind or other lateral forces cause transfer of moments at connection of framing elements to columns, shear resulting from moment transfer shall be considered in design of lateral reinforcement in columns.

11.10.1.2- Lateral reinforcement having an area A_v not less than $0.40\, b_w s/f_y$ shall be provided in that part of the column within the connections except for connections not part of a primary seismic load resisting system that are restrained on four sides by beams or slabs of approximately equal depth.

11.10.2- Special provisions for slabs

11.10.2.1- When gravity loads or wind or other lateral forces cause transfer of moment between slab and the column, a fraction of the unbalanced moment given by:

$$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3}\left(\frac{c_1 + d}{c_2 + d}\right)^{1/2}} \qquad (11\text{-}37)$$

shall be considered transferred by eccentricity of shear about centroid of a critical section perpendicular to the plane of the slab and located so that its perimeter is a minimum, but need not approach closer than d/2 to perimeter of column.

11.10.2.2- Shear stresses resulting from moment transfer by eccentricity of shear, shall be assumed to vary linearly about the centroid of the critical section defined in Section 11.9.1.2. Maximum shear stress due to factored shear forces and moments shall not exceed $\frac{1}{6}(1 + \frac{2}{\beta_c})\sqrt{f_{cu}}$, nor $0.25\sqrt{f_{cu}}$.

11.10.2.3- When shear reinforcement consisting of steel I or channel-shaped section (shearheads) is provided, the sum of shear stresses due to vertical load acting on the critical section defined by Section 11.9.4.7 and moment transferred by eccentricity of shear about centroid of the critical section defined in Section 11.9.1.2 shall not exceed $0.25\sqrt{f_{cu}}$.

CHAPTER 12- DEVELOPMENT AND SPLICES OF REINFORCEMENT

12.0- Notation

A_b = area of individual bar, mm^2

A_s = area of reinforcement, mm^2

A_v = area of shear reinforcement within a distance **s**, mm^2

a = depth of equivalent rectangular stress block as defined in Section 9.2.1.5

b_w = web width or diameter of a circular section, mm

d = distance from extreme compression fiber to centroid of tension reinforcement, mm

f_{cu} = charactersitic compressive strength of concerete, N/mm^2

f_y = characteristic yield strength of reinforcement, N/mm^2

h = overall thickness of member, mm

ℓ_a = additional embedment length at support or at point of inflection, mm

ℓ_d = development length, mm

ℓ_{db} = basic development length of a straight bar, mm

ℓ_{dh} = development length of standard hook in tension, measured from critical section to outside end of hook (straight embedment length between critical section and start of hook [point of tangency] plus radius of bend and one bar diameter), mm

ℓ_{hb} = basic development length of standard hook in tension, mm

M_u = ultimate moment capacity at section, N-mm

s = spacing of stirrups or ties, mm

s_w = spacing of wire to be developed or spliced, mm

V_u = factored shear force at section

β_b = ratio of area of reinforcement cut off to total area of tension reinforcement at section

Ø = nominal diameter of bar, mm

12.1- Development of reinforcement - General

12.1.1- Calculated tension or compression in reinforcement at each section of reinforced concrete members shall be developed on each side of that section by embedement length hook or mechanical device, or a combination thereof. Hooks may be used in developing bars in tension only.

12.1.2- Hook shall not be considered effective as part of anchorage in compression.

12.2- Development of deformed bars and deformed wire in tension.

12.2.1- Development length ℓ_d in mm for deformed bars in tension shall be computed as the product of the basic development length ℓ_{db} of Section 12.2.2 and the applicable modification factor or factors of Section 12.2.3 but ℓ_d shall not be less than that specified in Section 12.2.4.

12.2.2- Basic development length ℓ_{db} shall be:

Ø 35 bar and smaller	$0.02\,A_b f_y/\sqrt{f_{cu}}$
but not less than	$0.06\,Ø f_y$
Ø42 bar	$30\,f_y/\sqrt{f_{cu}}$
Ø56 bar	$38\,f_y/\sqrt{f_{cu}}$
Deformed wire	$0.4\,Ø f_y/\sqrt{f_{cu}}$

12.2.3- Basic development length ℓ_{db} shall be multiplied by applicable factor or factors for:

a) Reinforcement with f_y greater than 410 N/mm^2 .... $2 - \frac{410}{f_y}$

b) Top horizontal reinforcement where more than 300mm of concrete is cast in the member below the bar 1.4

12.2.4- Basic development length ℓ_{db} modified by appropriate factors of Section 12.2.3 may be multiplied by applicable factor or factors for:

a) Reinforcement in a flexural member in excess of that required .... $\frac{A_s(\text{required})}{A_s(\text{provided})}$

b) Reinforcement being developed in length under consideration and spaced laterally at least 150 mm on center with at least 75mm clear from face of member to edge bar, measured in direction of spacing 0.8

c) Reinforcement enclosed within spiral reinforcement not less than 6 mm diameter and not more than 100mm pitch 0.75

12.2.5- Development length ℓ_d shall not be less than 300 mm except in computation of lap splices by Section 12.14 and development of web reinforcement by Section 12.12.

12.3- Development of standard hooks in tension

12.3.1- Development length ℓ_{dh}, in mm, for deformed bars in tension terminating in a standard hook (Section 5.2) shall be computed as the product of the basic development length ℓ_{hb} of Section 12.3.2 and the applicable modification factors of Section 12.3.3 but ℓ_{dh} shall not be less than 8Ø or 150 mm, whichever is greater.

12.3.2- Basic development length ℓ_{hb} for hooked bar with f_y equal to 410 N/mm^2 shall be .... $\frac{110\,Ø}{\sqrt{f_{cu}}}$

12.3.3- Basic development length ℓ_{hb} shall be multiplied by the applicable factor or factors for:

a) Reinforcement having yield strength other than 410 N/mm^2 .... $\frac{f_y}{410}$

b) For bar diameters 35 mm and smaller, side cover (normal to plane of hook) not less than 65mm, and for 90 deg hook, cover on bar extension beyond hook not less than 50 mm 0.7

c) For bar diameters 35mm and smaller, hook enclosed vertically or horizontally within ties or stirrup - ties spaced along the full development length ℓ_{dh} not greater than 3 Ø where Ø is diameter of hooked bar 0.80

d) where anchorage or development for f_y is not specifically required, reinforcement in excess of that required by analysis .... $\frac{A_s(\text{required})}{A_s(\text{provided})}$

12.4- Development of deformed bars in compression

12.4.1- Development length ℓ_d, in mm, for deformed bars in compression shall be computed as the product of the basic development length ℓ_{db} of Section 12.4.2 and applicable modification factors of Section 12.4.3, but ℓ_d shall be not less than 200 mm.

12.4.2- Basic development length

ℓ_{db} shall be .... $0.3\,Ø\,f_y/\sqrt{f_{cu}}$

but not less than .... $0.045\,Øf_y$

12.4.3- Basic development length ℓ_{db} may be multiplied by applicable factors for:

a) Reinforcement in excess of that required by analysis .... $\frac{A_s(\text{required})}{A_s(\text{provided})}$

b) Reinforcement enclosed within spiral reinforcement not less than 6 mm diameter and not more than 100mm pitch 0.75

12.5- Development of bundled bars

The development length of individual bars within a bundle in tension or compression shall be that for the individual bar, increased by 20 percent for three - bar bundle and 33 percent for four-bar bundle.

12.6- Mechanical Anchorage

12.6.1- Any mechanical device capable of developing the strength of reinforcement without damage to the concrete may be used as anchorage.

12.6.2- Development of reinforcement may consist of a combination of mechanical anchorage plus additional embedment length of reinforcement between the point of maximum bar stress and the mechanical anchorage.

12.7- Development of welded deformed wire fabric in tension

12.7.1- Development length ℓ_d, in mm, of welded deformed wire fabric measured from point of critical section to end of wire shall be computed as the product of the basic development length ℓ_{db} of Section 12.7.2 or 12.7.3 and applicable modification factor or factors of Section 12.2.3 and 12.2.4; but ℓ_d shall not be less than 200mm except in computation of lap splices by Section 12.17 and development of web reinforcement by Section 12.12.

12.7.2- Basic development length ℓ_{db} of welded deformed wire fabric, with at least one cross wire within the development length not less than 50mm from point of critical section, shall be $0.4\,Ø(f_y - 138)/\sqrt{f_{cu}}$ but not less than

$$2.7\,\frac{A_w}{S_w}\,\frac{f_y}{\sqrt{f_{cu}}}$$

12.7.3- Basic development length ℓ_{db} of welded deformed wire fabric, with no cross wires within the development length, shall be determined as for deformed wire.

12.8- Development of welded smooth wire fabric in tension

Yield strength of welded smooth wire fabric shall be considered developed by embedment of two cross wires with the closer cross wire not less than 50mm from point of critical section. However, basic development length ℓ_{db} measured from point of critical section to outermost cross wire shall not be less than

$$3.6\,\frac{A_w}{S_w}\,\frac{f_y}{\sqrt{f_{cu}}}$$

modified by **(As required)/(As provided)** for reinforcement in excess of that required by analysis but ℓ_d shall not be less than 150mm except in computation of lap splices by Section 12.18.

12.9- Development of flexural reinforcement - General

12.9.1- Tension reinforcement may be developed by bending across the web to be anchored or made continuous with reinforcement on the opposite face of member.

12.9.2- Critical section for development of reinforcement in flexural members are at points of maximum stress and at points within the span where adjacent reinforcement terminates, or is bent. Provisions of Section 12.10.3 must be satisfied.

12.9.3- Reinforcement shall extend beyond the point at which it is no longer required to resist flexure, for a distance not less than

a) effective depth of the member or

b) 12Ø

whichever is greater, except at a simply supported end and at free end of a cantilever.

12.9.4- Continuing reinforcement shall have an embedment length not less than the development length ℓ_d beyond the point where bent or terminated tension reinforcement is no longer required to resist flexure.

12.9.5- Flexural reinforcement shall not be terminated in a tension zone unless one of the following conditions is satisfied:

a) Shear at the cutoff point does not exceed two - thirds that permitted, including shear strength of shear reinforcement provided.

b) Stirrup area in excess of that required for shear and torsion is provided along each terminated bar or wire over a distance from the termination point equal to three-fourths the effective depth of member. Excess stirrup area A_v shall be not less than $0.4\ b_w\ s/f_y$. Spacing s shall not exceed $d/8\beta_b$ where β_b is the ratio of area of reinforcement cut off to total area of tension reinforcemtn at the section.

c) For Ø35 mm bar and smaller, continuing reinforcement provides double the area required for flexure at the cutoff point and shear does not exceed three - fourths that permitted.

12.9.6- Adequate anchorage shall be provided for tension reinforcement in flexural members where reinforcement stress is not directly proportional to moment, such as: sloped, stepped, or tapered footings; brackets; deep flexural members; or members in which tension reinforcement is not parallel to compression face.

12.10- Development of positive moment reinforcement

12.10.1- At least one-third the positive moment reinforcement in simple members and one - fourth the positive moment reinforcement in continuous members shall extend along the same face of member into the support. In beams, such reinforcement shall extend into the support at least 150 mm.

12.10.2- When a flexural member is part of a primary lateral load resisting system, positive moment reinforcement required to be extended into the support by Section 12.10.1 shall be anchored to develop the charactresitic yield strength f_y in tension at the face of support.

12.10.3- At simple supports and at points of inflection, positive moment tension reinforcement shall be limited to a diameter such that ℓ_d computed for f_y by Section 12.2 satisfies Eq. (12-1); except, Eq. (12-1) need not be satisfied for reinforcement terminating beyond centerline of simple supports by a standard hook, or a mechanical anchorage at least equivalent to a standard hook.

$$\ell_d \leq \frac{M_u}{V_u} + \ell_a \qquad (12\text{-}1)$$

where:

M_u is ultimate moment capacity at section assuming all reinforcement at the section to be stressed to the charactersitic yield strength f_y.

V_u is factored shear force at the section.

ℓ_a at a support shall be the embedment length beyond center of support.

ℓ_a at a point of inflection shall be limited to the effective depth of member or 12 Ø; whichever is greater.

Value of M_u/V_u may be increased 30 percent when the ends of reinforcement are confined by a compressive reaction.

12.11- Development of negative moment reinforcement

12.11.1- Negative moment reinforcement in a continuous, restrained, or cantilever member, or in any member of a rigid frame, shall be anchored in or through the supporting member by embedment length, hooks, or mechanical anchorage.

12.11.2- Negative moment reinforcement shall have an embedment length into the span as required by Section 12.1 and 12.9.3.

12.11.3- At least one-third the total tension reinforcement provided for negative moment at a support shall have an embedment length beyond the point of inflection not less than effective depth of member, 12Ø, or one-sixteenth the clear span, whichever is greater.

12.12- Development of web reinforcement

12.12.1- Web reinforcement shall be carried as close to compression and tension surfaces of member as cover requirements and proximity of other reinforcement will permit.

12.12.2- Ends of single leg, simple U–, or multiple U–stirrups shall be anchored by one of the following means:

12.12.2.1- A standard hook plus an embedment of $0.5\ell_d$. The $0.5\ell_d$ embedment of a stirrup leg shall be taken as the distance between middepth of member $d/2$ and start of hook (point of tangency).

12.12.2.2- Embedment $d/2$ above or below middepth on the compression side of the member for a full development length ℓ_d but not less than 24Ø or for deformed bars or deformed wire, 30mm.

12.12.2.3- Bending around longitudinal reinforcement through at least 135 deg plus, for stirrups with design stress exceeding 275 N/mm^2, an embedment of $0.33\ell_d$. The $0.33\ell_d$ embedment of a stirrup leg shall be taken as the distance between middepth of member $d/2$ and start of hook (point of tangency).

12.12.2.4- For each leg of welded smooth wire fabric forming simple U–stirrups, either:

a) Two longitudinal wires spaced at a 50mm spacing along the member at the top of the U.

b) One longitudinal wire located not more than $d/4$ from the compression face and a second wire closer to the compression face and spaced not less than 50mm from the first wire. The second wire may be located on the stirrup leg beyond a bend, or on a bend with an inside diameter of bend not less than 8Ø.

12.12.2.5- For each end of a single leg stirrup of welded smooth or deformed wire fabric, two longitudinal wiers at a minimum spacing of 50mm and with the inner wire at least the greater of **d/4** or 50mm from middepth of member **d/2**.Outer longitudunal wire at tension face shall not be farther from the face than the portion of primary flexural reinforcement closest to the face.

12.12.3- Between anchored ends, each bend in the continuous portion of a simple U-stirrup or multiple U-stirrup shall enclose a longitudinal bar.

12.12.4- Longitudinal bars bent to act as shear reinforcement, if extended into a region of tension, shall be continuous with longitudinal reinforcement and if extended into a region of compression, shall be anchored beyond middepth **d/2** as specified for development length in Section 12.2 for that part of f_y required to satisfy Eq. (11-12).

12.12.5- Pairs of U-stirrups or ties so placed as to form a closed unit shall be considered properly spliced when length of laps are $1.7\ell_d$. In members at least 450mm deep, such splices with $A_b f_y$ not more than 40 kN per leg may be considered adequate if stirrup legs extend the full available depth of member.

12.13- Splices of reinforcement - General

12.13.1- Splices of reinforcement shall be made only as required or permitted on design drawings, or in specifications, or as authorized by the Engineer.

12.13.2- Lap splices

12.13.2.1- Lap splices shall not be used for bars larger than Ø35 except as provided in Sections 12.15.2 and 15.8.2.4.

12.13.2.2- Lap splices of bundled bars shall be based on the lap splice length required for individual bars within a bundle, increased 20 percent for a three-bar bundle and 33 percent for a four - bar bundle. Individual bar splices within a bundle shall not overlap.

12.13.2.3- Bars spliced by noncontact lap splices in flexural members shall not be spaced transversely farther apart than one-fifth the required lap splice length, nor 150mm.

12.13.3- Welded splices and mechanical connections

12.13.3.1- A full welded splice shall have bars butted and welded to develop in tension at least 125 percent of characteristic yield strength f_y of the bar.

12.13.3.2- A full mechanical connection shall develop in tension or compression, as required, at least 125 percent of characteristic yield strength f_y of the bar.

12.13.3.3- Welded splices and mechanical connections not meeting requirements of Section 12.13.3.1 or 12.13.3.2 may be used in accordance with Section 12.14.4.

12.14- Splices of deformed bars and deformed wire in tension

12.14.1- Minimum length of lap for tension lap slices shall be as required for Class A, B, or C splice, but not less than 300mm, where:

Class A splice	$1.0\ell_d$
Class B splice	$1.3\ell_d$
Class C splice	$1.7\ell_d$

where ℓ_d is the tensile development length for the charactresitic yield strength f_y in accordance with Section 12.2.

12.14.2- Lap splices of deformed bars and deformed wire in tension shall conform to Table 12.14.

TABLE 12.4- TENSION LAP SPLICES

A_s provided* / A_s required	Maximum percent of A_s spliced within required lap length		
	50	75	100
Equal to or greater than 2	Class A	Class A	Class B
Less than 2	Class B	Class C	Class C

* Ratio of area of reinforcement provided to area of reinforcement required by analysis at splice location.

12.14.3- Welded splices or mechanical connections used where area of reinforcement provided is less than twice that required by analysis shall meet requirements of Section 12.13.3.1 or 12.13.3.2.

12.14.4- Welded splices or mechanical connections used where area of reinforcement provided is at least twice that required by analysis shall meet the following:

12.14.4.1- Splices shall be staggered at least 600mm and in such manner as to develop at every section at least twice the calculated tensile force at that section but not less than 133 N/mm² for total area of reinforcement provided.

12.14.4.2- In computing tensile force developed at each section, spliced reinforcement may be rated at the specified splice strength. Unspliced reinforcement shall be rated at that fraction of f_y defined by the ratio of the shorter actual development length to ℓ_d required to develop the characteristic yield strength f_y.

12.14.5- Splice in "tension tie members" shall be made with a full welded splice or full mechanical connection in accordance with Section 12.13.3.1 or 12.13.3.2 and splices in adjacent bars shall be staggered at least 750mm.

12.15- Splices of deformed bars in compression

12.15.1- Minimum length of lap for compression lap splices shall be the development length in compression computed in accordance with Section 12.4, but not less than $0.075Ø\, f_y$, nor $(0.13\, f_y - 24)Ø$ for f_y greater than 410 N/mm², nor 300mm. For f_{cu} less than 25 N/mm², length of lap shall be increased by one—third.

12.15.2- When bars of different sizes are lap spliced in compression, splice length shall be the larger of: development length of larger bar, or splice length of smaller bar. Bar sizes Ø42 and Ø56 may be lap spliced to Ø35 and smaller bars.

12.15.3- In tied reinforced compression members, where ties throughout the lap splice length have an effective area not less than **0.0015 hs**, lap splice length may be multiplied by 0.83, but lap length shall not be less than 300mm. Tie legs perpendicular to dimension **h** shall be used in determining effective area.

12.15.4- In spirally reinforced compression members, lap splice length of bars within a spiral may be multiplied by 0.75, but lap length shall not be less than 300mm.

12.15.5- Welded splices or mechanical connections used in compression shall meet requirements of Section 12.13.3.1 or 12.13.3.2.

12.15.6- End bearing splices

12.15.6.1- In bars required for compression only, compressive stress may be transmitted by bearing of square cut ends held in concentric contact by a suitable device.

12.15.6.2- Bar ends shall terminate in flat surfaces within 1 ½ deg of a right angle to the axis of the bars and shall be fitted within 3 deg of full bearing after assembly.

12.15.6.3- End bearing splices shall be used only in members containing closed ties, closed stirrups, or spirals.

12.16- Special splice requirements for columns

12.16.1- Where factored load stress in longitudinal bars in a column, calculated for various loading combinations, varies from f_y in compression to ½f_y or less in tension, lap splices, butt welded splices, mechanical connections, or end bearing splices may be used. Total tensile strength provided in each face of the column by splices alone or by splices in combinations with contiunuing unspliced bars at charactresitic yield strength f_y shall be at least twice the calculated tension in that face of the column but not less than required by Section 12.16.3.

12.16.2- Where factored load stress in longitudinal bars in a column calculated for any loading combination, exceeds ½f_y in tension, lap splices designed to develop the characteristic yield strength f_y in tension, or full welded splices or full mechanical connection in accordance with Section 12.13.3.1 or 12.13.3.2 shall be used.

12.16.3- At horizontal cross sections of columns where splices are located, a minimum tensile strength in each face of the column equal to one-quarter the area of vertical reinforcemt in that face multiplied by f_y shall be provided by splices.

12.17- Splices of welded deformed wire fabric in tension

12.17.1- Minimum length of lap for lap splices of welded deformed wire fabric measured between the ends of each fabric sheet shall be not less than **1.7** ℓ_d nor 200mm, and the overlap measured between outermost cross wires of each fabric sheet shall be not less than 50mm. ℓ_d shall be the development length for the characteristic yield strength f_y in accordance with Section 12.7.

12.17.2— Lap splices of welded deformed wire fabric, with no cross wires within the lap splice length, shall be determined as for deforemed wire.

12.18- Splices of welded smooth wire fabric in tension

Minimum length of lap for lap splices of welded smooth wire fabric shall be in accordance with the following.

12.18.1- When area of reinforcement provided is less than twice that required by analysis at splice location, length of overlap measured between outermost cross wires of each fabric sheet shall not be less than one spacing of cross wires plus 50mm, not less than **1.5**ℓ_d, nor 150mm ℓ_d shall be the development length for the characteristic yield strength f_y in accordance with Section 12.8.

12.18.2— When area of reinforcement provided is at least twice that required by analysis at splice location, length of overlap measured between outermost cross wires of each fabric sheet shall not be less than **1.5**ℓ_d nor 50mm. ℓ_d shall be the development length for the characteristic yield strength f_y in accordance with Section 12.8.

13.0- Notation

b_e = width of column strip for transfer of the effective moment between a slab and edge or corner column, mm

C_x = shorter dimension of a column, mm

C_y = longer dimension of a column, mm

d_h = depth of column capital measured from the soffit of slab or drop to bottom of capital, mm

f_{cu} = characteristic compressive strength of concrete, N/mm^2

f_y = characteristic tensile yield strength of steel, N/mm^2

h_c = effective diameter of column capital which shall be taken as the diameter of a circle of the same area as the cross section of the capital, mm

ℓ = span length of one way slab as defined in Section 9.1.1,mm

ℓ_1 = panal length parallal to span measured from center to center,mm

ℓ_c = dimension of the column measured in the same direction as ℓ_h,mm

ℓ_h = effective dimension of a capital,mm

ℓ_{ho} = actual dimension of a capital,mm

ℓ_x = length of the shorter side of a rectangular slab, mm

ℓ_y = length of the longer side of a rectangular slab, mm

M_x = bending moment at midspan per unit width in the short direction, N.mm/mm

M_y = bending moment at midspan per unit width in the long direction, N.mm/mm

M_{tmax} = maximum moment which can be transferred to an edge or corner column, N·mm

x = distance from nearer support to the section under consideration, mm

w_u = total ultimate load per unit area

α_x = moment coefficient in the x—direction as given in Table 13.3(a)

α_y = moment coefficient in the y—direction as given in Table 13.3(a)

β_x = moment corfficient in the x—direction as given in Table 13.3(b)

β_y = moment coefficient in the y—direction as given in Table 13.3(b)

13.1- General provisions

13.1.1- Scope

Provisions of Chapter 13 shall apply for design of slab systems reinforced for flexure in one or more directions with or without beams betweent supports. The relevant provisions of the other chapters of this code shall also apply.

13.1.2- Moments and forces in solid slabs

In addition to the methods used for beams, moments and shear forces resulting from both distributed and concentrated loads may be determined by elastic analysis such as those of Pigeauds and Westergaard. Alternatively, Johansen's yield—line method or Hillerborg's strip method may be used provided the ratio between support and span moments are similar to those obtained by the use of elastic theory; values between 1.0 and 1.5 are recommended.

13.1.3- Crack control in solid slabs

Relevant provisions of Chapter 8 shall apply. However the provisions may be deemed to be satisfied if the requirements of Section 5.7.5 are met.

13.1.4- Minimum area of reinforcement

For structural slabs of uniform thickness, minimum area and maximum spacing of reinforcement in the direction of the span shall be as required for shrinkage and temperature according to Section 5.10

13.2- One — way slabs

13.2.1- Special provisions on the distribution of concentrated loads in one way slabs:

Allowance should be made for the bending moments due to concentrated loads, using methods based either on the elastic theory or on an appropriate plastic approach.

If a slab is simply supported on two opposite edges and carries one or more concentrated loads in a line in the direction of the span, it should be designed to resist the maximum bending moment caused by the loading system. Such bending moment may be assumed to be reisted by an effective width of slab (measured normal to the span) as follows:

a) For soild slabs, the effective width may be taken as the sum of the load width and $2.4x(1-x/\ell)$ where x is the distance from the nearer support to the section under consideration and ℓ is the span.

b) For other slabs, except where specially provided for, the effective width will depend on the transverse and longitudinal flexural rigidities of the slab. When these are approximately equal, the value for the effective width as given for solid slabs may be used, but as the ratio decreases a smaller value should be taken. The minimum value which need be taken, however, is the load width plus $4x/\ell(1-x/\ell)$ meters where x and ℓ have the same meanings as in (a); so that, for a section at mid—span, the effective width is equal to 1 m plus the load width.

c) Where the concentrated load is near an unsupported edge of a slab the effective width should not exceed the value in (a) or (b) above as appropriate, nor half that value plus the distance to the center of the load from the unsupported edge (see Fig 13.2).

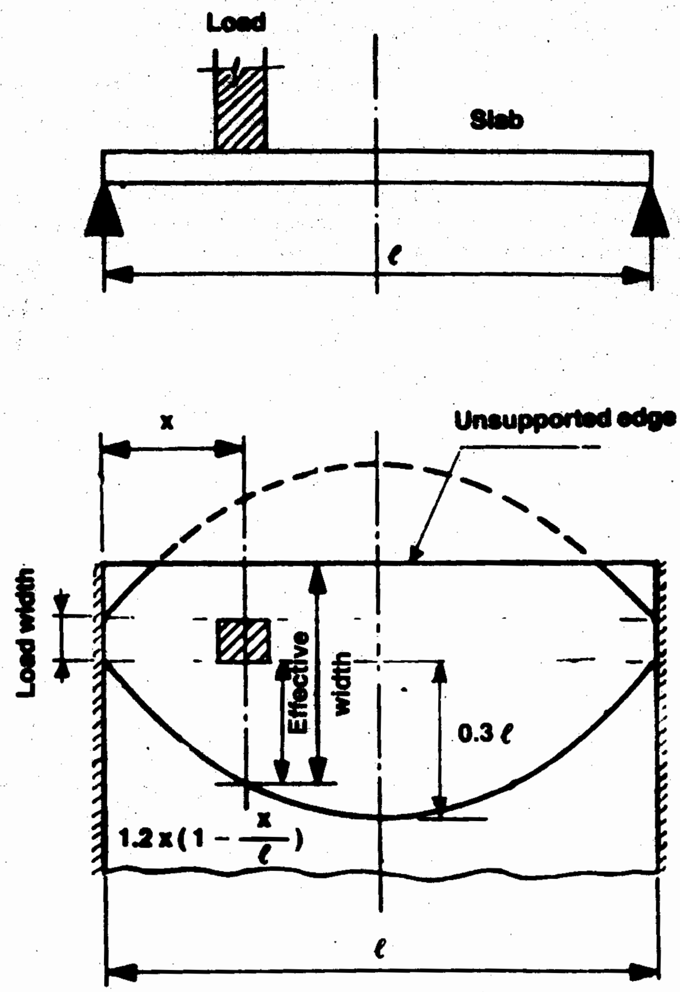


Figure 13-2: EFFECTIVE WIDTH OF SOLID SLAB CARRYING A CONCENTRATED LOAD NEAR AN UNSUPPORTED END.

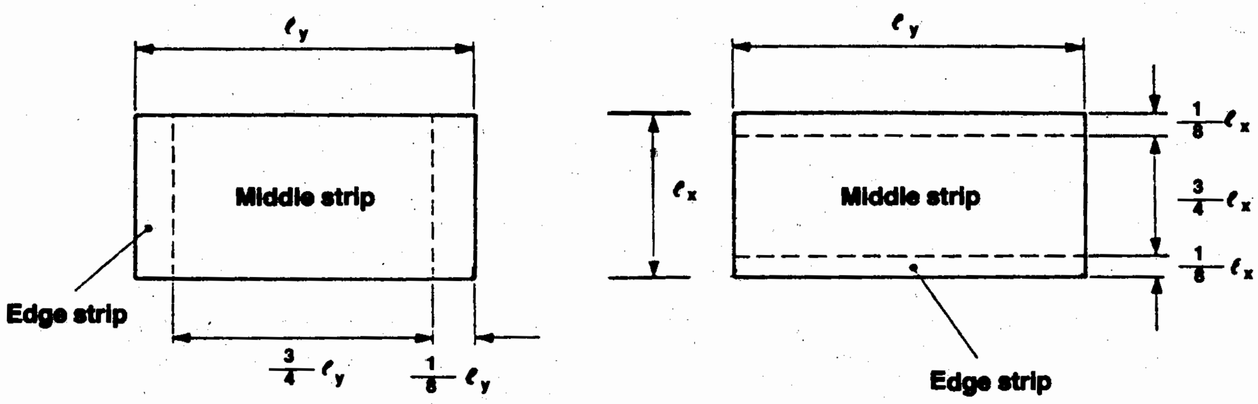


Figure (13.3)- DIVISION OF SLAB INTO MIDDLE AND EDGE STRIPS

13.3- Two way solid slabs supported on four sides

13.3.1- Solid slabs spanning in two directions at right angles: uniformly distributed loads.

In addition to other methods the following may be used for the design of slabs spanning in two directions at right angles and supporting uniformly distributed loads.

13.3.1.1- Simply supported slabs

When simply supported slabs do not have adequate provisions to resist torsion at the corners and to prevent corners from lifting, the maximum moments per unit width are given by the following equations:

$$M_x = \alpha_x w_u \ell_x^2 \quad (13\text{–}1)$$

$$M_y = \alpha_y w_u \ell_y^2 \quad (13\text{–}2)$$

where M_x and M_y are the bending moments at midspan on strips of unit width of spans ℓ_x and ℓ_y respectively. α_x and α_y are bending moment coefficients shown in Table 13.3(a).

TABLE 13.3(a)- BENDING MOMENT COEFFICIENTS FOR SLABS SPANNING IN TWO DIRECTIONS AT RIGHT ANGLES, SIMPLY SUPPORTED ON FOUR SIDES

ℓ_y/ℓ_x	1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0
α_x	0.062	.074	.084	.093	.099	.104	.113	.118
α_y	0.062	.061	.059	.055	.051	.046	.037	.029

At least 50% of the tension reinforcement provided at midspan should extend to the supports. The remaining 50% should extend to within $0.1\ell_x$ or $0.1\ell_y$ of the support as appropritate.

13.3.1.2- Restrained slabs

In slabs where the corners are prevented from lifting, and provisions for torsion are made, the maximum moments per unit width are given by the following equations:

$$M_x = \beta_x w_u \ell_x^2 \quad (13\text{-}3)$$

$$M_y = \beta_y w_u \ell_y^2 \quad (13\text{-}4)$$

where M_x and M_y are the moments at midspan on strips of unit width spanning ℓ_x and ℓ_y respectvely β_x, β_y are coefficients given in Table 13.3(b).

For these slabs the following rules apply:

a) Slabs are considered as divided in each direction into middle strips and edge strips as shown in Fig (13.3), the middle strip being three quarters of the width and each edge strip one eigth of the width.

b) The maximum moments calculated as above apply only to the middle strips and no redistribution should be made.

c) When the negative moment on one side of a support is less than 80 percent of that on the other side, two thirds of the difference shall be distributed in proportion to the relative stiffnesses of the slabs.

d) Provisions of Chapter 12 shall apply for the curtailment of positive moment reinforcement.

e) Over the continuous edge of a middle strip, the tension reinforcement should extend in the upper part of the slab a distance of 0.15ℓ from the support, and at least 50% should extend a distance 0.3ℓ

f) At a discontinuous edge negative moment reinforcement shall extend a minimum of 0.2ℓ into the span.

g) Reinforcement in an edge strip, Parallel to that edge, need not exceed the minimum temperature and shrinkage reinforcement together with the requirements of torsion described below.

h) Torsion reinforcement should be provided at any corner where the slab is simply supported on both edges meeting at that corner. It should consist of top and bottom reinfrocement, each with layers of bars placed parallel to the sides of the slab and extending from the edges a minimum distance of one fifth the shorter span in each direction.

The area of reinforcement per unit width of slab in each of these four layers should be three quarters of the area per unit width required for the maximum mid span moment in the slab.

i) Torsion reinforcement equal to half that described in the preceeding paragraph should be provided at a corner contained by edges over only one of which the slab is continuous.

j) Torsion reinforcement need not be provided at any corner contained by edges over both of which the slab is continuous.

Where ℓ_y/ℓ_x is greater than 2, slab should be designed as spanning one-way only.

13.3.1.3- Supporting beams

The loads on the supporting beams for a two-way rectangular panel may be assumed as the load within the tributary area of the panel bounded by the intersection of 45 deg lines from the corners with the median of the panel parallal to the long side.

The bending moments may be determined approximately by using an equivalent uniform load per unit length of beam for each panel supported as follows:

For the short side: $\dfrac{w_u \ell_x}{3}$

For the long side: $\dfrac{w_u \ell_x}{3}\left(\dfrac{3 - (\ell_x/\ell_y)^2}{2}\right)$

13.3.2- Shear resistance of solid slabs

Relevant provisions of Chapter 11 shall be applied.

13.3.3- Deflection of solid slabs

Relevant provisions of Chapter 8 shall be applied.

13.4- Flat slabs

13.4.1- Definitions

13.4.1.1- flat slab

The term flat slab means a reinforced concrete slab, reinforced in two or more directions with or without drops and supported generally without beams, by columns with or without column capitals. A flat slab may be solid or may have recesses formed on the soffit so that the

TABLE 13.3(b) – BENDING MOMENTS COEFFICIENTS – RECTANGULAR SLABS SUPPORTED AND RESTRAINED ON FOUR SIDES WITH PROVISION FOR TORSION AT CORNERS

Type of slab and moments considered	Short span coefficients β_x								Long span coefficients β_y for all Values of $\frac{\ell_y}{\ell_x}$
	Values of $\frac{\ell_y}{\ell_x}$								
	1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	
Case 1-Interior panels:									
Negative moment at continuous edge	0.033	0.040	0.045	0.050	0.054	0.059	0.071	0.083	0.033
Positive moment at mid-span	0.025	0.030	0.034	0.038	0.041	0.045	0.053	0.062	0.025
Case 2-One short or long edge discontinuous:									
Negative moment at –									
continuous edge	0.041	0.047	0.053	0.057	0.061	0.065	0.075	0.085	0.041
discontinuous edge	0.021	0.024	0.026	0.028	0.030	0.032	0.037	0.042	0.021
Positive moment at mid-span	0.031	0.035	0.040	0.043	0.046	0.049	0.056	0.064	0.031
Case 3-Two adjacent edges discontinuous:									
Negative moment at –									
continuous edge	0.049	0.056	0.062	0.066	0.070	0.073	0.082	0.090	0.049
discontinuous edge	0.025	0.028	0.031	0.033	0.035	0.037	0.040	0.045	0.025
Positive moment at mid-span	0.037	0.042	0.047	0.050	0.053	0.055	0.062	0.068	0.037
Case 4-Two short edges discontinuous:									
negative moment at –									
continuous edge	0.056	0.061	0.065	0.069	0.071	0.073	0.077	0.080	—
discontinuous edge	—	—	—	—	—	—	—	—	0.025
positive moment at mid-span	0.044	0.046	0.049	0.051	0.053	0.055	0.058	0.060	0.044
Case 5-Two long edges discontinuous:									
Negative moment at –									
continuous edge	—	—	—	—	—	—	—	—	0.056
discontinuous edge	0.025	0.028	0.031	0.033	0.035	0.037	0.040	0.045	—
Positive moment at mid-span	0.044	0.053	0.060	0.065	0.068	0.071	0.077	0.080	0.040
Case 6-Three edges discontinuous (one short or long edge continuous):									
Negative moment at –									
continuous edge	0.058	0.065	0.071	0.077	0.081	0.085	0.092	0.098	0.058
discontinuous edge	0.029	0.033	0.036	0.038	0.040	0.042	0.046	0.049	0.029
Positive moment at mid-span	0.044	0.049	0.054	0.058	0.061	0.064	0.069	0.074	0.044
Case 7-Four edges discontinuous:									
Negative moment at –									
discontinuous edge	0.033	0.038	0.041	0.044	0.046	0.049	0.053	0.055	0.033
Positive moment at mid-span	0.050	0.057	0.062	0.067	0.071	0.075	0.081	0.083	0.050

soffit comprises a series of ribs in two directions (waffle or coffered slabs).

13.4.1.2- Column capital

A column capital is a local enlargement of the top of a column providing support to the slab over a larger area than the column section alone. For the purpose of this section, the dimensions of a column capital which may be considered to be effective are limited according to the depth of the capital. In any direction, the effective dimension of a capital ℓ_h, should be taken as the lesser of the actual dimension ℓ_{ho} or ℓ_{hmax}, where ℓ_{hmax} is given by:

$\ell_{hmax} = \ell_c + 2(d_h - 40)$, mm

For a flared capital the actual dimension ℓ_{ho} is that measured 40mm below the soffit of the slab or drop (see Fig. 13.4(a)). The angle of greatest slope of the capital should for the purpose of analysis not exceed 45 deg from the vertical.

13.4.1.3- Effective diameter of a column or column capital, h_c

This is the diameter of a circle whose area equals the cross sectional area of the column or, if column capitals are used, the area of the column capital based on the effective dimensions as defined in Section 13.4.1.2. In no case should h_c be taken greater than one—quareter of the shortest span framing into column.

13.4.1.4- Drops

a) A drop is a thickening of the slab in the region of the column. For the purpose of this section, a drop may only be considered to influence the distribution of moments within the slab where the smaller dimension of the drop is at least one third of the smaller dimension of the surrounding panels. Smaller drops may, however, still be taken into account when assessing the resistance to shear.

b) Projection of drop below the slab shall be at least one—quarter the slab thickness beyond the drop.

c) In computing required slab reinforcement, thickness of drop below the slab shall not be assumed greater than one—quarter the distance from edge of drop to edge of column or column capital.

13.4.1.5— Thickness of panels

The thickness of the slab will generally be controlled by consideration of deflection. However the thickness of the slab should not be less than 125mm nor less than the values given in Table 13.4(a).

Table 13.4(a): MINIMUM SLAB THICKNESS

f_y	with drop at four supports	without drop
250	$\ell_y/40$	$\ell_y/36$
340	$\ell_y/36$	$\ell_y/33$
410	$\ell_y/33$	$\ell_y/30$

13.4.2- Analysis of flat slab structures

13.4.2.1— In lieu of using more rigorous analysis, flat slabs supported by a generally rectangular arrangement of columns may be analysed using the equivalent frame method or the simiplified alternative.

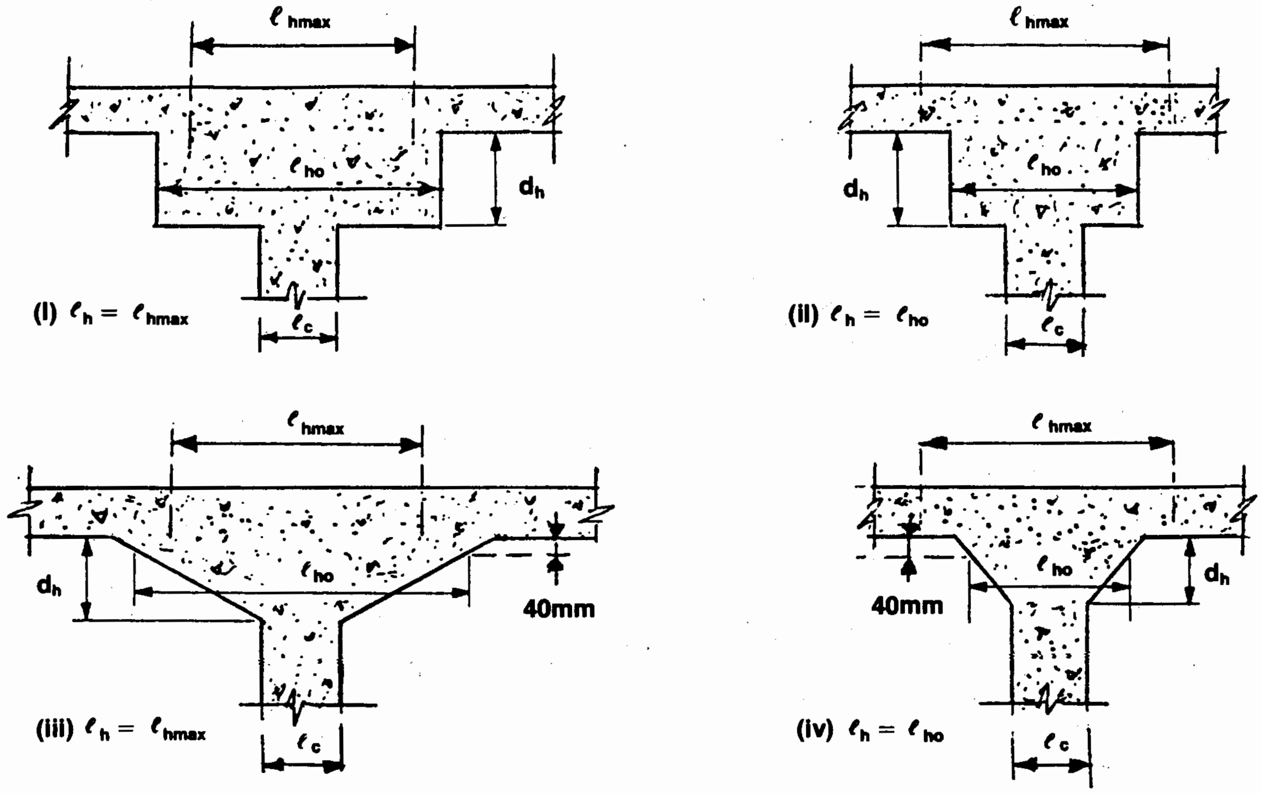


FIGURE 13.4(a) Types of column capital

13.4.2.2— Equivalent frame method

a) Division of flat slab structures into frames

The structure may be assumed to be divided longitudinally and transversely into frames consisting of columns and strips of slab. The width of the slab used to define the effective stiffness of the slab will depend upon the aspect ratio of the panels and the type of loading. In the case of vertical loading the stiffness of rectangular panels may be calculated taking into account the full width of the panel. For horizontal loading half this value shall be taken.

b) Frame stiffness

The moment of inertia of any section of slab or column in calculation the relative stiffness of members may be assumed to be that of the concrete alone. The stiffening effect of drops and column captials may be ignored. In the case of a recessed or coffered slab which is made solid in the region of the columns, the stiffening effect may be ignored provided the solid part of the slab does not extend more than **0.15** ℓ into the span measured from the center line of the columns.

c) Arrangement of loads

It will be sufficient to consider the following arrangements of loads:

1- Alternate spans loaded with the design factored load **(1.4D + 1.7L)** and all other spans loaded with a factored dead load **(1.4D)**.

2 – Any two adjacent spans loaded with design factored load **(1.4D + 1.7L)** and all other spans loaded with a factored dead load **(1.4D)**.

d) Analysis

Each frame may be analysed in its entirety by suitable elastic methods. Alternatively, for vertical loads only, each strip of floor and roof may be analysed as a separate frame with the columns above and below being assumed fixed in position and direction at their extremities. For this method to be valid, the ratio of the larger span to the shorter span shall not exceed 2.

e) Limitation of negative moments

Negative moments greater than those at a distance **h_c/2** from the centerline of a column may be ignored provided the sum of the maximum positive moment and the average of the negative moments in any one span of the slab for the whole panel width is not less than:

$$\frac{w_u \ell^2}{8}(\ell_1 - \frac{2}{3} h_c)^2$$

Where the above condition is not satisfied, the negative moments should be increased accordingly.

13.4.2.3— Simplified method

For flat slab structures whose lateral stability is not dependent on slab — column connections Table 13.4(b) may be used for their design, subject to the following provisos :

a) The design is based on the single load case of all spans loaded with the maximum design factored load provided:

1- The ratio of live load to dead load does not exceed 1.25

2- The charactristic live load does not exceed 5 kN/m^2.

b) The slabs should comprise a series of rectangular panels of approximately constant thickness in at least three continuous spans in each direction and the ratio of the length of a panel to its width should not exceed 4:3

TABLE 13.4. (b)- BENDING MOMENT AND SHEAR FORCE COEFFICIENTS FOR A FULL PANEL OF FLAT SLABS OF THREE OR MORE APPROXIMATELY EQUAL SPANS

	Outer support		Near center	First interior	Center of	Interior
	column	wall	of first span	support	interior span	support
Moment	− .04Fℓ	0.02Fℓ	+0.083Fℓ	−0.063Fℓ	+0.071Fℓ	−0.055Fℓ
Shear	0.45F	0.4F	—	0.6F	—	0.5F
Total column moments	0.04Fℓ	—	—	0.022Fℓ	—	0.022Fℓ

Note 1. F is the design factored on the panel of the slab between adjacent columns considered i.e. **(1.4D + 1.7L)** × area of the panel.

Note 2. The moments in the edge panel marked by an asterisk may have to be adjusted to conform with the provisions of Section 13.4.3.2(c).

Note 3. ℓ = effective span $= \ell_1 - 2/3 h_c$

Note 4. The limitation of Section 13.4.2.2(e) need not be checked.

Note 5. When the negative moment on one side of a support is less than 80 percent of that on the other side two thirds of the difference shall be distributed in proportion to the relative stiffnesses of the slabs.

c) The successive span length in each direction shall not differ by more than 15 percent of the longer span, except that in no case shall an end span be longer than the adjacent interior span.

13.4.2.4- Division of panels (except in the region of edge and comer columns).

a) Flat slab panels should be assumed to be divided into column strips and middle strips [see Fig 13.4(b)].

b) A column strip is a design strip with a width on each side of column center—line equals **0.25** ℓ_x.

c) A middle strip is a design strip bounded by two column strips.

d) In the assessment of widths of the column and middle strips for slabs with drops the width of the coulmn strip shall be taken equal to the width of drop. However drops should be ignored if their smaller lateral dimension is less than one third of the shorter span length.

e) **Column strips of unlike panels**

If widths of two column strips along the same column center — line in two adjacent panels are not equal, the division of the panels over the region of the common support should be taken as that calculated for the panel giving the wider column strip.

13.4.2.5— Division of moment between column and middle strips

The moment obtained from the equivalent frame method or the simplified method should be divided between the column and middle strips in the proportions given in Table 13.4(c).

TABLE 13-4(c): DISTRIBUTION OF MOMENTS IN PANELS OF FLAT SLABS

	Apportionment between column and middle strip expressed as percentages of the total negative or positive moments (see note)	
	Column strip	Middle strip
Negative	75	25
Positive	55	45

Note: For the case where the width of the column strip is taken as equal to that of the drop, and the middle strip is thereby increased in width, the moments to be resisted by the middle strip should be increased in proportion to its increased width. The moments to be resisted by the column strip may then be decreased by an amount such that the total positive and the total negative moments resisted by the column strip and the middle strip together are unchanged.

13.4.3- Design of panels

13.4.3.1— Interior panels

a) **Column and middle strips**

These should be designed to withstand the moments obtained in accordance with Section 13.4.2.2 or 13.4.2.3. In general two thirds of the amount of reinforcement required to resist the negative moment in the column strip should be placed over a width equal to half that of the column strip and central with the column.

b) **Curtailment of bars**

This should be in accordance with Fig 13.4(c).

13.4.3.2— Design of edge panels

a) **Positive moments in span and negative moments over interior edges**

These should be apportioned and designed for exactly as for an internal panel using the same column and middle strips as for an internal panel.

b) **Moments transferable between slab and edge or corner columns**

Moments will only be able to be transferred between a slab and an edge or corner column by a column strip considerably narrower than that appropriate for an internal panel. The breadth of this strip, b_e, for various typical cases is shown in Fig 13.4(d). b_e should never be taken as greater than the column width appropriate for an interior panel. The maximum moment which can be transferred to a column by this strip is given by:

$M_{tmax} = 0.15\, b_e\, d^2\, f_{cu}$

where **d** is the effective depth for the top reinforcement in the column strip.

c) **Limitations of moment transfer**

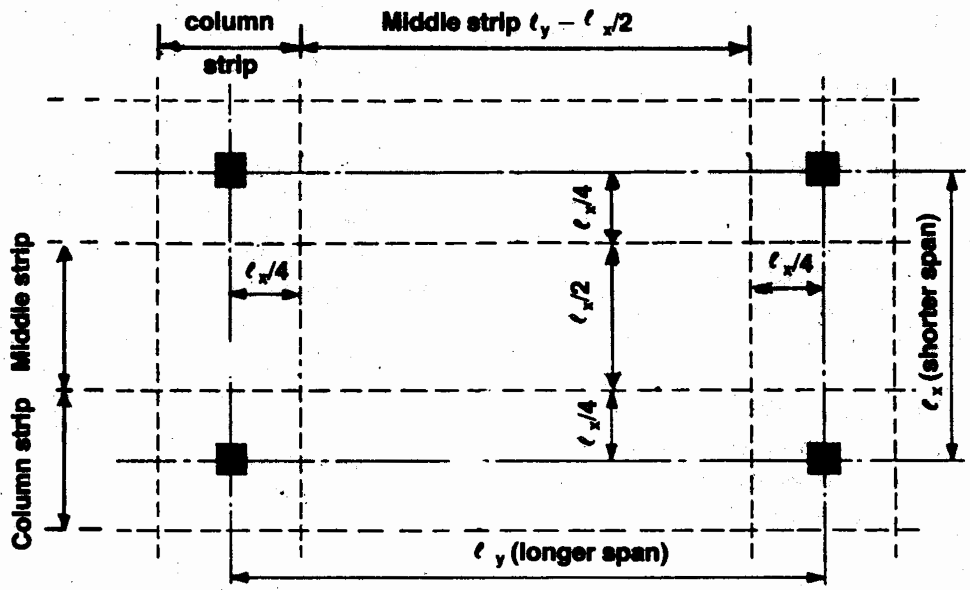
Where analysis of the structure indicates a column moment larger than M_{tmax} the edge moment in the slab should be reduced to a value not greater than M_{tmax} and the positive moment in the span adjusted accordingly. The normal limitations on redistributions and neutral axis depth may be disregarded in this case. Moments in excess of M_{tmax} may only be transferred to a column if an edge beam or strip of slab along the free edge is reinforced in accordance with chapter 11 to carry the extra moment into the column by torsion, In the absence of an edge beam, an appropriate breadth of slab may be assesed by using the principle illustrated in Fig.13.4(d).

13.4.3.3- Panels with marginal beams or walls

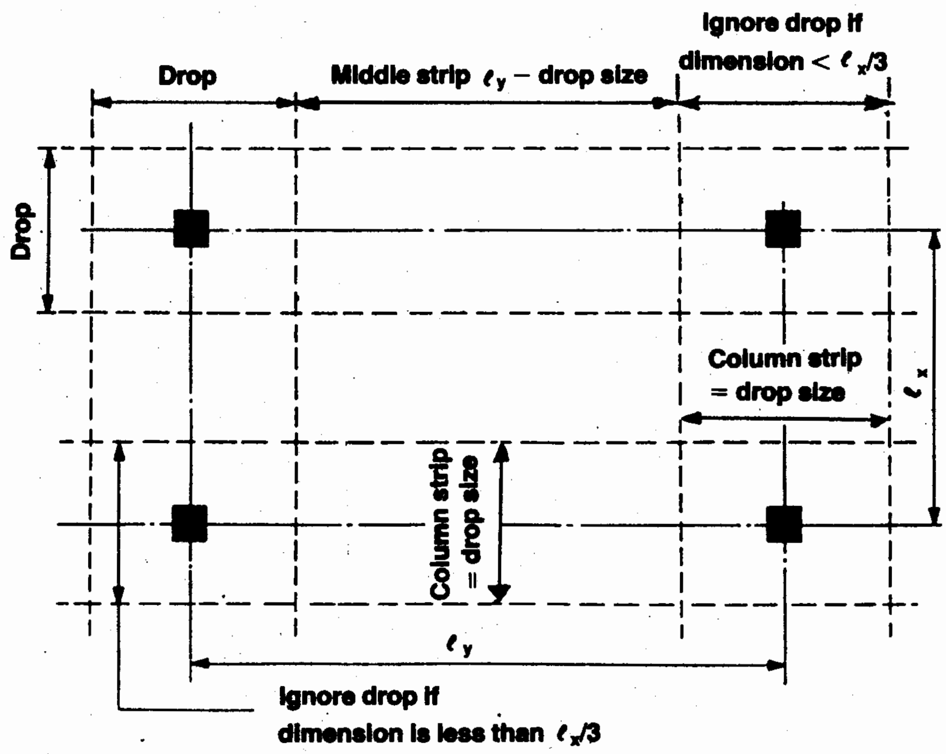
Where the slab is supported by a marginal beam with a depth greater than 1.5 times the thickness of the slab, or by a wall then:

a) The total load to be carried by the beam or wall should comprise those loads directly on the wall or beam plus a uniformly distributed load equal to one — quarter of the total load on the panel, and

b) the moments on the half — column strip adjacent to the beam or wall should be one — quater of the moments obtained from Section 13.4.2.

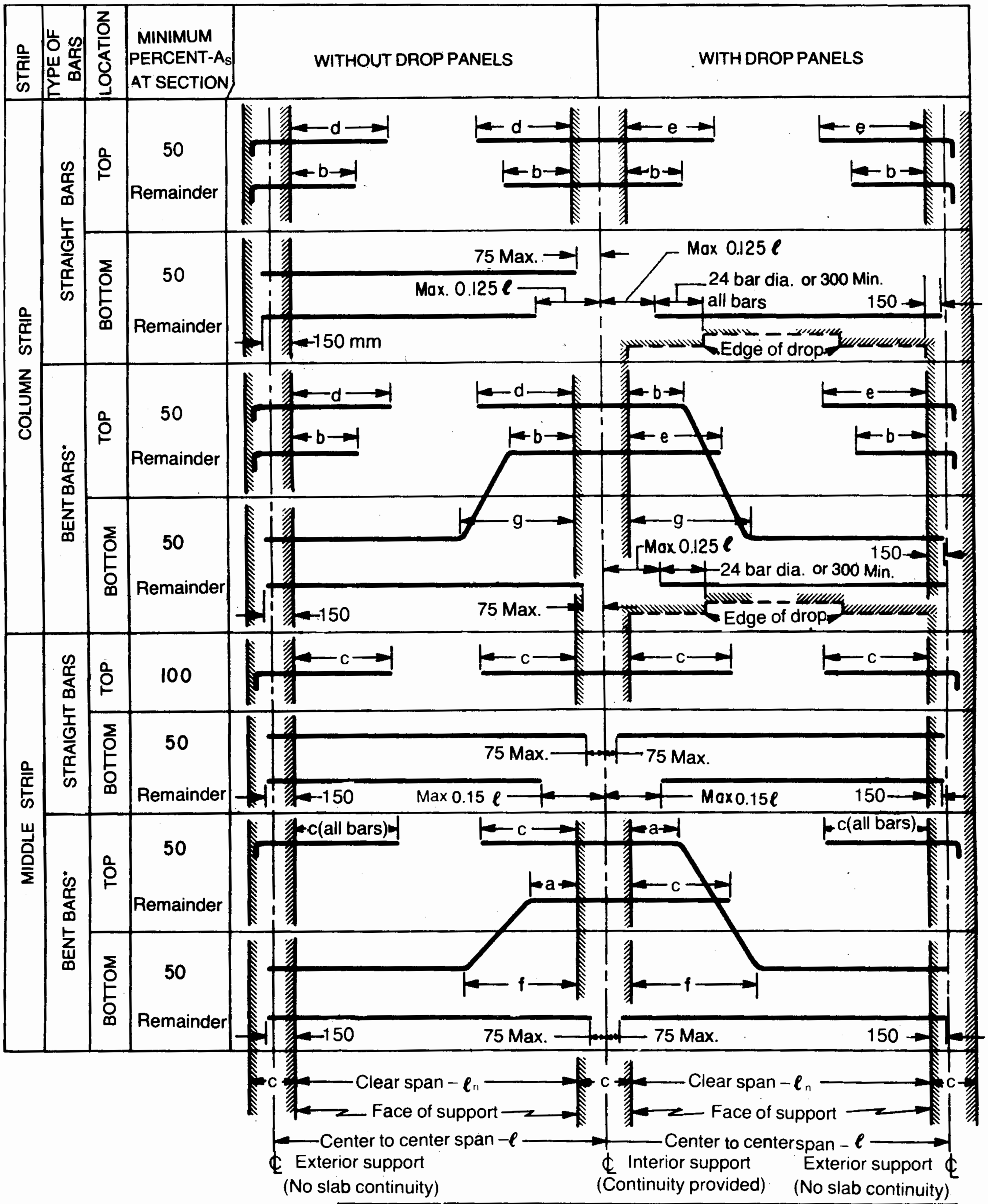


(a) slab without drops



(b) slab with drop

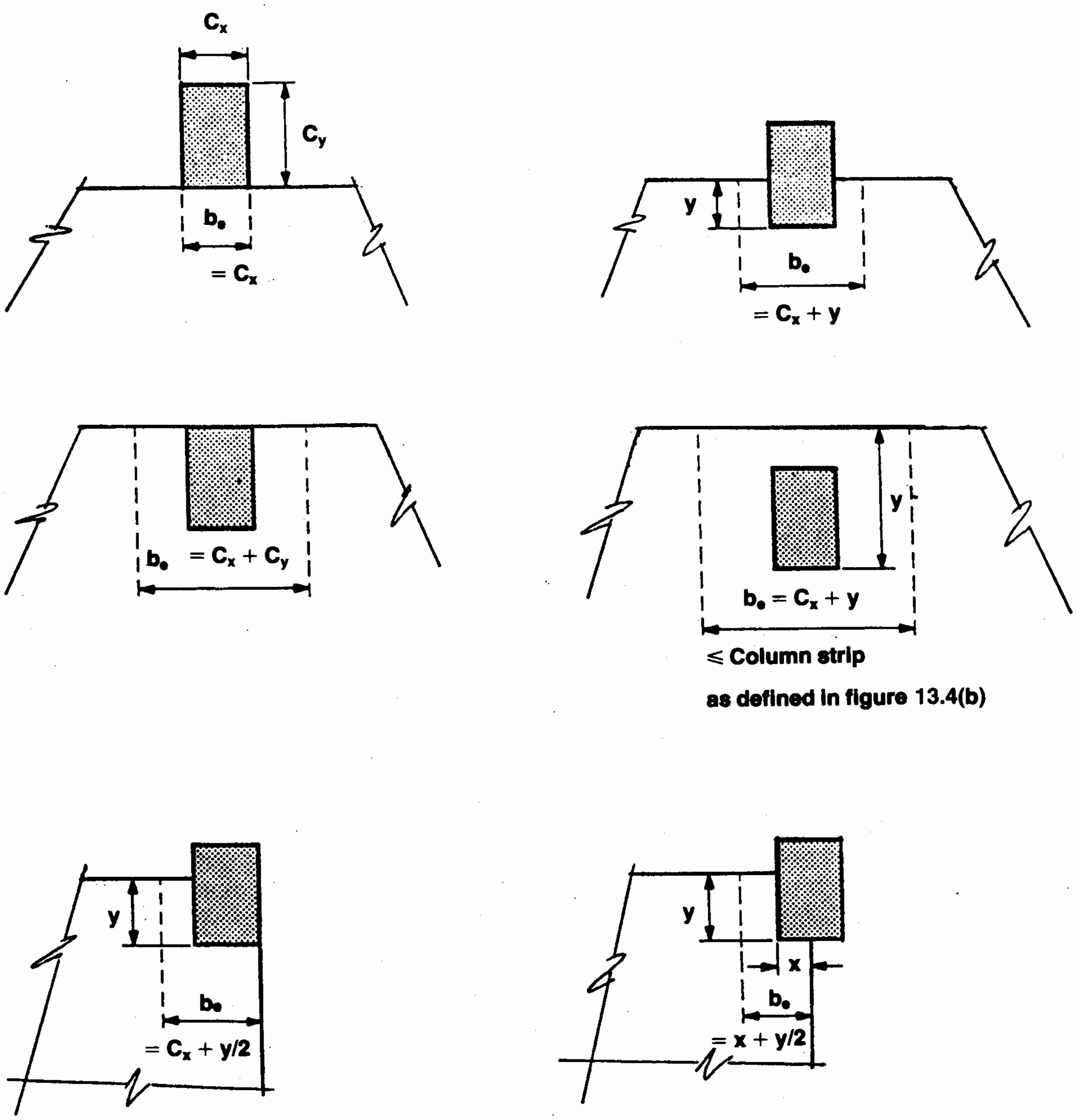
Figure 13.4(b). Division of panels in flat slabs



* Bent bars at exterior supports may be used if a general analysis is made

	BAR LENGTH FROM FACE OF SUPPORT						
	MAXIMUM LENGTH					MINIMUM LENGTH	
MARK	a	b	c	d	e	f	g
LENGTH	$0.14\,\ell_n$	$0.20\,\ell_n$	$0.22\,\ell_n$	$0.30\,\ell_n$	$0.33\,\ell_n$	$0.20\,\ell_n$	$0.24\,\ell_n$

Fig. 13.4(c)- Minimum bend point locations and extensions for reinforcement in slabs without beams



NOTE. y is the distance from the face of the slab to the innermost face of the column.

Figure 13.4(d) Definition of breadth of effective moment transfer strip, b_e, for various typical cases

13.5- Joist construction

13.5.1- Joist construction consists of a monolithic combination of regularly spaced ribs and a top slab arranged to span in one direction or two orthogonal directions.

13.5.2- Ribs shall not be less than 100mm in width and shall have depth of not more than 3 1/2 times the minimum width of rib.

13.5.3- Clear spacing between ribs shall not exceed 800mm.

13.5.4- Joist construction not meeting the limitations of Sections 13.5.1 through 13.5.3 shall be designed as slabs and beams.

13.5.5- When permanent burned clay or concrete tile fillers of material having a unit compressive strength at least equal to that of the specified strength of concrete in the joist are used then:

13.5.5.1- Vertical shells of fillers in contact with the ribs may be included in strength computations for shear and negative moment. Other portions of fillers shall not be included in strength computations.

13.5.5.2- Slab thickness over permanent fillers shall be not less than 1/12 the clear distance between ribs nor less than 40mm.

13.5.5.3- In one way joists, reinforcement normal to the ribs shall be provided in the slab as required in Section 5.11.

13.5.6- When removable forms or fillers not complying with Section 13.5.5. are used then:

a) Slab thickness shall not be less than 1/12 the clear distance between ribs, nor less than 50mm.

b) Reinforcement normal to the ribs shall be provided in the slab as required for flexure, considering load concentrations, if any, but not less than required by Section 5.10.

13.5.7 When conduits and pipes as permitted by Section 4.3 are embedded within the slab, slab thickness shall be at least 25mm greater than the total overall depth of the conduits or pipes at any point. Conduits or pipes shall not impair significantly the strength of the construction.

13.5.8- Shear strength provided by concrete V_c for the ribs may be taken as 10 percent greater than that provided in Section 11.2. Shear strength may be increased by use of shear reinforcement or by widening the ends of the ribs.

13.6- Openings in slab systems

13.6.1- Openings of any size may be provided in slab systems if shown by analysis that the design strength is at least equal to the required strength considering Sections 7.2 and 7.3, and that all serviceability conditions, including the specified limits of deflection, are met.

13.6.2- In lieu of special analysis as required by Section 13.6.1, openings may be provided in slab systems without beams only in accordance with the following:

13.6.2.1- Openings of any size may be located in the area common to intersecting middle strips, provided total amount of reinforcement required for the panel without the opening is maintained.

13.6.2.2- In the area common to intersecting column strips, not more than one-eighth the width of column strip in either span shall be interrupted by openings. An amount of reinforcement equivalent to that interrupted by an opening shall be added on the sides of the opening.

13.6.2.3- In the area common to one column strip and one middle strip, not more than one—quarter the reinforcement in either strip shall be interrupted by openings. An amount of reinforcement equivalent to that interrupted by an opening shall be added on the sides of the opening.

13.6.2.4- Shear requirements of Section 11.9.5 shall be satisfied.

14.0- Notation

A_g = gross area of section, mm^2
b = width of wall, mm
f_{cu} = characteristic compressive strength of concrete, N/mm^2
h = overall thickness of member, mm
k = effective length factor
ℓ_c = vertical distance between supports, mm
P_{uw} = design axial load strength of wall by the empirical method, N
v_u = shear stress due to factored shear force of section, N/mm^2
V_u = factored shear force at section ,N
β_c = ratio of long side to short side of concentrated load or reaction area

14.1- Scope

14.1.1- Provisions of Chapter 14 shall apply for design of both reinforced and plain concrete walls subjected to axial load, with or without flexure.

14.1.2- Design of concrete walls subject to axial loads shall be in accordance with Section 14.2 for reinforced concrete and Section 14.3 for plain concrete.

14.2- Reinforced concrete walls

14.2.1- General

14.2.1.1- Walls shall be designed for eccentric loads and any lateral or other loads to which they are subjected.

14.2.1.2- Unless demonstrated by a detailed analysis, horizontal length of wall to be considered as effective for each concentrated load shall not exceed center — to — center distance between loads, nor width of bearing plus four times the wall thickness.

14.2.1.3- Compression members built integrally with walls shall conform to Section 10.1.8.2.

14.2.1.4- Transfer of force to footing at base of wall shall be in accordance with Section 15.8.

14.2.1.5- Design for shear shall be in accordance with Section 11.8.

14.2.1.6- Walls shall be anchored to intersecting elements such as floors, roofs, or to columns, pilasters, buttresses, and intersecting walls, and footings.

14.2.1.7- Quantity of reinforcement and limits of thickness required by Sections 14.2.2 and 14.2.4 may be waived where structural analysis show adequate strength and stability.

14.2.2- Minimum reinforcement

14.2.2.1- Minimum vertical and horizontal reinforcement shall be in accordance with Section 14.2.2.2 and 14.2.2.3 unless a greater amount is required for shear by Section 11.8.7 and 11.8.8.

14.2.2.2- Minimum ratio of vertical reinforcement area to gross concrete area shall be:

a) 0.0015 for high yield steel of characteristic yield strength not less than 410 N/mm^2 and welded wire fabrics.

b) 0.0020 for other types of steel bars.

14.2.2.3- Minimum ratio of horizontal reinforcement area to gross concrete area shall be:

a) 0.0025 for high yield steel of characteristic yield strength not less than 410 N/mm^2, and welded wire fabric.

b) 0.0030 for other types of steel bars.

14.2.2.4- Walls more than 200mm thick, except basement walls, shall have reinforcement for each direction placed in two layers parallel with faces of wall in accordance with the following:

a) One layer consisting of not less than one— half and not more than two—thirds of total reinforcement required for each direction shall be placed not less than 50mm nor more than one—third the thickness of wall from exterior surface.

b) The other layer, consisting of the balance of required reinforcement in that direction, shall be placed not less than 20mm nor more than one—third the thickness of wall from interior surface.

14.2.2.5- Vertical and horizontal reinforcement shall not be spaced further apart than 3 times the wall thickness, nor 350mm.

14.2.2.6- Vertical reinforcement need not be enclosed by lateral ties if vertical reinforcement area is not greater than 0.01 times the gross concrete area, or where vertical reinforcement is not required as compression reinforcement.

14.2.2.7- In addition to the minimum reinforcement required by Sections 14.2.2.2 and 14.2.2.3, not less than two Ø16 bars shall be provided around all window and door openings. Such bars shall be extended to develop the bar beyond the corners of the openings but not less than 600 mm.

14.2.3- Walls designed as compression members

Except as provided in Section 14.2.4, walls subject to axial load or combined flexure and axial load shall be designed as compression memebers in accordance with provisions of Chapter 10.

14.2.4- Empirical design method

14.2.4.1- Walls of solid rectangular cross section may be designed by the empirical provisions of Section 14.2.4 if resultant of all factored loads is located within the middle—third of the overall thickness of wall and all limits of Sections 14.2.1, 14.2.2, and 14.2.4 are satisfied.

14.2.4.2- Design axial load strength P_{uw} of a wall satisfying limitation of Section 14.2.4.1 shall be computed by Eq. (14.1) unless designed in accordance with Section 14.2.3.

$$P_{uw} = 0.3\, f_{cu}\, A_g\, [1 - (\frac{k\ell_c}{32h})^2] \quad (14\text{-}1)$$

effective length factor **k** shall be:

For walls braced at top and bottom against lateral translation and

a) restrained against rotation at one or both ends (top and/or bottom) 0.8

b) unrestrained against rotation at both ends 1.0

For walls not braced against lateral translation 2.0

14.2.4.3- Minimum thickness of walls designed by empirical design method

a) Thickness of bearing walls shall not be less than 1/25 the unsupported height or length, whichever is shorter, nor less than 120mm.

b) Thickness of exterior basement walls and fondation walls shall not be less than 200mm.

14.2.5- Nonbearing walls

Thickness of nonbearing walls shall not be less than 100mm, nor less than 1/30 the least distance between members that provide lateral support.

14.2.6- Walls as grade beams

14.2.6.1- Walls designed as grade beams shall have top and bottom reinforcement as required for moment in accordance with provisions of Section 9.1 through 9.2.5. Design for shear shall be in accordance with provisions of Chapter 11.

14.2.6.2 Portions of grade beam walls exposed above grade shall also meet requirements of Section 14.2.2.

14.3- Plain concrete walls

14.3.1— General

14.3.1.1- Provisions of Sections 14.2.1.1 and 14.2.1.2 shall apply.

14.3.1.2- Plain concrete walls shall be continuously supported by soil or by footings, fondation walls, grade beams, or other structural members capable of providing continuous vertical support, or where arch action assures compression under all conditions of loading.

14.3.1.3- Plain concrete walls may be designed in accordance with Section 14.3.3 provided the wall is designed for an eccentricity corresponding to the maximum moment that can accompany the axial load but not less than **0.10h**. Otherwise, plain concrete walls shall be designed under provisions of Section 14.3.5.

14.3.1.4- Design for shear shall be in accordance with Section 14.3.4.

14.3.2- Permissible stresses

Maximum fiber stresses in plain concrete due to factored loads and moments shall not exceed the following:

a) Flexure

Extreme fiber stress in compression	$0.5\, f_{cu}$
Extreme fiber stress in tension	$0.24\sqrt{f_{cu}}$

b) Axial compression $\quad 0.3\, f_{cu}\, [1 - (\frac{k\ell_c}{32h})^2]$

c) Shear

Beam action	$0.1\sqrt{f_{cu}}$
Two-way action	$(0.1 + \frac{0.2}{\beta_c})\sqrt{f_{cu}}$
but not greater than	$0.2\sqrt{f_{cu}}$

d) Bearing on loaded area $\quad 0.44\, f_{cu}$

14.3.3- Walls subject to combined flexure and axial load

Walls shall be proportioned such that the sum of the ratios of all calculated to permissible stresses in compression given in Section 14.3.2 (a) and (b) shall be less than or equal to one. Tensile stress resulting from combined flexure and axial load shall not exceed permissible stress in tension given in Section 14.3.2 (a).

14.3.4- Shear strength

14.3.4.1 Shear stress v_u for rectangular sections shall be computed by

$$v_u = \frac{3V_u}{2bh} \quad (14\text{–}2)$$

where **h** is overall thickness of wall. For concrete cast against soil **h** shall be taken as 50mm less than the actual thickness.

14.3.4.2- Maximum shear stress v_u shall be computed at a distance **h** from face of support, and sections located at a lesser distance may be designed for the same shear.

14.3.4.3- Shear stress v_u shall not exceed permissible shear stress for beam action given in Section 14.3.2(c).

14.3.5- Empirical design method

14.3.5.1- Plain concrete walls of solid rectangular cross section may be designed by Eq. (14-3) if resultant of all factored loads is located within the middle-third of the overall thickness of wall.

14.3.5.2- Design axial load strength P_{uw} of a plain concrete wall satisfying limitations of Section 14.3.5.1 shall be computed by:

$$P_{uw} = 0.2\, f_{cu}\, A_g\, [1 - (\frac{K\ell_c}{32h})^2] \quad (14\text{-}3)$$

effective length factor **k** for walls braced at top and bottom against translation shall be taken as in Sections 14.2.4.2 (a) and (b). Laterally unsupported walls shall be designed as reinforced concrete members in accordance with Section 14.2.

14.3.6- Limitations

14.3.6.1- Thickness of bearing walls shall not be less than 1/24 the unsupported height or length, whichever is shorter, nor less than 150mm.

14.3.6.2- Thickness of exterior basement walls and foundation walls shall not be less than 250mm.

14.3.6.3 Walls shall be braced against lateral translation. See Section 14.3.5.2.

14.3.6.4- Not less than two Ø16 bars shall be provided around all window and door openings. Such bars shall extend at least 600mm beyond the corners of openings.

CHAPTER 15- FOOTINGS

15.0— Notation

A_g = gross area of section, mm^2
b = width of member, mm
b_o = perimeter of critical section for footing in two way action, mm
d_p = diameter of circular pile or side dimension of square pile at footing base, mm
f_{cu} = characteristic compressive strength of concrete, N/mm^2
h = overall thickness of member, mm
V_u = facotored shear force at section, N
v_u = shear stress due to V_u, N/mm_2
β = ratio of long side to short side of footing

15.1- Scope

15.1.1- Provisions of Chapter 15 shall apply for design of isolated footings and, where applicable, to combined footings and mats.

15.1.2- Additional requirements of design of combined footings and mats are given in Section 15.10.

15.2- Loads and reactions

15.2.1- Except for base area, footings shall be proportioned to resist the factored loads and induced reactions in accordance with the appropriate design requirements of this code and as provided in Chapter 15.

15.2.2- Base area of footing or number and arrangement of piles shall be determined from unfactored forces and moments transmitted by footing to soil or piles and permissible soil pressure or permissible pile capacity selected through principles of soil mechanics.

15.2.3- For footings on piles, computations for moments and shears may be based on the assumption that the reaction from any pile is concentrated at pile center.

15.3- Footings supporting circular or regular polygon shaped columns or pedestals

Circular or regular polygon shaped concrete columns or pedestals may be treated as square members with the same area for location of critical sections for moment, shear, and development of reinforcement in footings.

15.4- Moment in footings

15.4.1- External moment on any section of a footing shall be determined by passing a vertical plane through the footing, and computing the moment of the forces acting over entire area of footing on one side of that vertical plane.

15.4.2- Maximum factored moment for an isolated footing shall be computed as prescribed in Section 15.4.1 at critical sections located as follows:

a) At face of column, pedestal, or wall, for footings supporting a concrete column, pedestal, or wall.

b) Halfway between middle and edge of wall or column for footings supporting a masonry wall or a masonry column.

c) halfway between face of column and edge of steel base plate, for footings supporting a column with steel base plate.

15.4.3- In one — way footings, and two — way square footings, reinforcement shall be distributed uniformly across entire width of footing.

15.4.4- In two — way rectangular footings, reinforcement shall be distributed as follows:

15.4.4.1- Reinforcemt in long direction shall be distributed uniformly across entire width of footing.

15.4.4.2- For reinforcement in short direction, a protion of the total reinfrocement given by Eq. (15-1) shall be distributed uniformly over a band width (centered on centerline of column or pedestal) equal to the length of short side of footing. Remainder of reinforcement required in short direction shall be distributed uniformly outside center bandwidth of footing:

$$\frac{\text{Reinforcement in band width}}{\text{Total reinforcement in short direction}} = \frac{2}{(\beta+1)} \qquad (15\text{-}1)$$

15.5- Shear in footings

15.5.1- Shear strength of footings shall be in accordance with Section 11.9.

15.5.2- Location of critical section for shear in accordance with Chapter 11 shall be measured from face of column, pedestal, or wall for footings supporting a column, pedestal, or wall. For footings supporting a column or pedestal with steel base plates, the critical section shall be measured from location defined in Section 15.4.2(c).

15.5.3- Computation of shear on any section through a footing supported on piles shall be in accordance with the following:

15.5.3.1- Entire reaction from any piles whose center is located $d_p/2$ or more outside the section shall be considered as producing shear on that section.

15.5.3.2- Reaction from any pile whose center is locatted $d_p/2$ or more inside the section shall be considered as producing no shear on that section.

15.5.3.3- For intermediate positions of pile center, the portion of the pile reaction to be considered as producing shear on the section shall be based on straight-line interpolation between full value at $d_p/2$ outside the section and zero value at $d_p/2$ inside the section.

15.6- Development of reinforcement in footings

15.6.1- Development of reinforcement in footings shall be in accordance with Chapter 12.

15.6.2- Calculated tension or compression in reinforcement at each section shall be developed on each side of that section by embedment length, hook (tension only) or mechanical device, or a combination thereof.

15.6.3- Critical sections for development of reinforcement shall be assumed at the same locations as defined in Section 15.4.2 for maximum factored moment, and at other verical planes where changes of section or reinforcement occur. See also Section 12.9.6.

15.7- Minimum footing depth

Depth of footing above bottom reinforcement shall not be less than 150 mm for footings on soil, nor less than 300 mm for footings on piles. For minimum depth of plain concrete footing see Section 15.11.

15.8- Transfer of force at base of column, wall or reinforced pedestal

15.8.1- Forces and moments at base of column, wall, or pedestal shall be transferred to supporting pedestal or footing by bearing on concrete and by reinforcement, dowels, and mechanical connectors.

15.8.1.1- Bearing on concrete at contact surface between supported and supporting member shall not exceed concrete bearing strength for either surface as given by Section 10.9.

15.8.1.2- Reinforcement, dowels, or mechanical connectors between supported and supporting members shall be adequate to transfer:

a) all compressive force that exceeds concrete bearing strength of either member.

b) any computed tensile force across interface.

In addition, reinforcement, dowels or mechanical conectors shall satisfy Section 15.8.2 or 15.8.3.

15.8.1.3- If calculated moments are transferred to supporting pedestal or footing, reinforcement, dowels or mechanical connectors shall be adequate to satisfy Section 12.16.

15.8.1.4- Lateral forces shall be transferred to supporing pedestal or footing in accordance with shear—friction provisions of Section 11.5, or by other appropriate means.

15.8.2- In cast—in—place construction, reinforcement required to satisfy Section 15.8.1 shall be provided either by extending longitudinal bars into supporting pedestal or footing, or by dowels.

15.8.2.1- For cast—in—place columns and pedestals, area of reinforcement across interface shall be not less than 0.005 times gross area of supported member.

15.8.2.2- For cast—in—place walls, area of reinforcement across interface shall be not less than minimum vertical reinforcement given in Section 14.2.2.2.

15.8.2.3- Diameter of dowels, if used, shall not exceed diameter of longitudinal bars by more than 4mm.

15.8.2.4- At footings, 42mm and 56mm longitudinal bars in compression only, may be lap spliced with dowels to provide reinforcement required to satisfy Section 15.8.1. Dowels shall not ~~be larger~~ than 35mm bar and shall extend into supported member a distance not less than the development length of 42mm or 56mm bars or the splice length of the dowels, whichever is greater, and into the footing a distance not less than the development length of the dowels.

15.8.2.5- If a pinned or rocker connection is provided in cast —in— place construction, connection shall conform to Section 15.8.1 and 15.8.3.

15.8.3- In precast construction, reinforcement required to satisfy Section 15.8.1 may be provided by anchor bolts or suitable mechanical connectors.

15.8.3.1- Connection between precast columns or pedestals and supporting member shall have a tensile strength not less than **1.4** A_g in Newtons, where A_g is area of supported member in mm^2

15.8.3.2- Connection between precast wall and supporting member shall have a tensile strength not less than **0.3**A_g in Newtons, where A_g is cross sectional area of wall.

15.8.3.3- Anchor bolts and mechanical connectors shall be designed to reach their design strength prior to anchorage failure or failure of surrounding concrete.

15.9- Sloped or stepped footings

15.9.1- In sloped or stepped footings, angle of slope or depth and location of steps shall be such that design requirements are satisfied at every section (see also Section 12.9.6).

15.9.2- Sloped or stepped footings designed as a unit shall be constructed to assure action as a unit.

15.10- Combined footings and mats

15.10.1- Except for base areas, footings supporting more than one column pedestal, or wall (combined footings or mats) shall be proportioned to resist the factored loads and induced reactions, in accordance with appropriate design requirements of this code.

15.10.2- Distribution of soil pressure under combined footings and mats shall be consistent with properties of the soil and the structure and with established principles of soil mechanics.

15.11- Plain concrete footings and pedestals

15.11.1- Design stresses in N/mm^2 due to factored loads and moments in plain concrete footings on soil and pedestals shall not exceed the values given in Section 14.3.2.

15.11.2- In sections 11.9.1.1 and 11.9.1.2, **d** shall be replaced by **h** for plain concrete footing.

15.11.3- In computing stresses due to flexure, combined flexure and axial load and shear, the entire cross section of a member shall be considered in design except for concrete cast against soil, overall thickness **h** shall be taken as 50 mm less than actual thickness.

15.11.4- Depth of plain concrete footing shall not be less than 200mm.

15.11.5- Plain concrete shall not be used for footing on piles.

15.11.6- Plain concrete pedestals shall be designed for vertical, lateral, and other loads to which they are subjected.

15.11.7-Ratio of unsupported height to average least lateral dimension of plain concrete pedestals shall not exceed 3.

16.1- Scope

16.1.1- Provisions of Chapter 16 shall apply for design of precast concrete members defined as concrete elements cast elsewhere than their final position in the structure.

16.1.2- All provisions of this code not specifically excluded, and not in conflict with provisions of Chapter 16, shall apply to precast concrete.

16.2- Design

16.2.1- Precast members shall be designed to resist safely stresses induced under all loading and restraint conditions from the start of manufacturing to completion of structure including the least favourable conditions which may arise due to form removal, storage, transport, handling and erection.

16.2.2- In precast construction that does not behave monolithically, effects of all interconnected and adjoining details shall be considered to assure proper performance of the structural system.

16.2.3- Design of joints and bearings shall include effects of all forces to be transmitted, including shrinkage, creep, temperature, elastic deformation, impact, vibration, wind and earthquake.

16.2.4- All details shall be designed to provide for manufacturing and erection tolerances and temporary erection stresses.

16.3- Details

16.3.1- All details of reinforcement, connections, bearing seats, inserts, anchors, concrete cover, openings, lifting devices, fabrication, and erection tolerances shall be shown on the drawings.

16.3.2- Embedded items, (such as dowels or inserts) shall be correctly positioned and properly anchored to develop required factored loads.

16.3.3- Connection joints shall be designed to satisfy safety requirements, manufacturing accuracies and tolerance allowances, and shall be ecnonomical and have acceptable appearance.

16.4- Identification and marking

16.4.1- Each precast member shall be marked in clearly legible form to indicate location and orientation in the structure, top surface, and date of manufacture. Precast components of identical external dimensions but with different reinforcement, concrete characteristic strength or concrete cover shall be provided with distinctive markings.

16.4.2- Identification marks shall correspond to the placing plans.

16.5- Transportation, storage, handling, and erection.

16.5.1- During curing, form removal, storage, handling, transportation and erection, precast members shall not be overstressed, warped or otherwise damaged or have camber adversely affected.

16.5.2- Precast members shall be adequately braced and supported during erection to insure proper alignment and safety until permanent connections are completed.

CHAPTER 17- COMPOSITE CONCRETE FLEXURAL MEMBERS

17.0- Notation

A_c = area of contact surface being investigated for horizontal shear, mm^2.

b_v = width of cross section at contact surface being investigated for horizontal shear.

d = distance from extreme compression fiber to centroid of tension reinforcement for entire composite section, mm.

V_h = horizontal shear strength

V_u = factored shear force at section

17.1- Scope

17.1.1- Provisions of Chapter 17 shall apply for design of composite concrete flexural members defined as precast and/or cast—in—place concrete elements constructed in separate placements but so interconnected that all elements respond to loads as a unit.

17.1.2- All provisions of this code shall apply to composite concrete flexural members, except as specifically modified in Chapter 17.

17.2- General

17.2.1- An entire composite member or portions therof may be used in resisting shear and moment.

17.2.2- Individual elements shall be investigated for all critical stages of loading.

17.2.3- If the specified strength, unit weight, or other properties of the various elements are different, properties of the individual elements or the most critical values, shall be used in design.

17.2.4- In strength computations of composite members, no distinction shall be made between shored and unshored members.

17.2.5- All elements shall be designed to support all loads introduced prior to full development of design strength of composite members.

17.2.6- Reinforcement shall be provided as required to control cracking and to prevent separation of individual elements of composite members.

17.2.7- When used, shoring shall not be removed until supported elements have developed design properties required to support all loads and limit deflections and cracking at time of shoring removal.

17.3- Control of deflection

17.3.1- Shored construction

If composite flexural members are supported during construction so that, after removal of temporary supports, dead load is resisted by the full composite section, the composite member may be considered equivalent to a monolithically cast member for computation of deflection. The portion of the member in compression shall determine whether values in Table 8.1(b) shall apply. If deflection is computed, account should be taken of curvatures resulting from differential shrinkage of precast and cast in—place components.

17.3.2- Unshored construction

If the thickness of a precast flexural member meets the requirements of Table 8.1(b), deflection need not be computed. If the thickness of a composite member meets the requirements of Table 8.1(b), deflection occuring after the member becomes composite need not be computed, but the longtime deflection of the precast member should be investigated for magnitude and duration of load prior to beginning of effective composite action.

17.3.3- Deflection computed in accordance with Sections 17.3.1 and 17.3.2 shall not exceed limits stipulated in Table 8.1(a).

17.4- Vertical shear strength

17.4.1 When an entire composite member is assumed to resist vertical shear, design shall be in accordance with requirements of Chapter 11 as for a monolithically cast member of the same cross—sectional shape.

17.4.2- Shear reinforcement shall be fully anchored into interconnected elements in accordance with Section 12.12.

17.4.3- Extended and anchored shear reinforcement may be included as ties for horizontal shear.

17.5- Horizontal shear strength

17.5.1 In a composite member, full transfer of horizontal shear forces shall be assured at contact surfaces of interconnected elements.

17.5.2- Unless calculated in accordance with Section 17.5.3, design of cross sections subject to horizontal shear shall be based on

$$V_u \leq V_h$$

where **V_u** is factored shear force at section considered and **V_h** is horizontal shear strength in accordance with the following.

17.5.2.1- Shear strength **V_h** shall not be taken greater than **0.5 b_v d** (in Newtons) for beam and slab members without ties and the contact surface has been prepared by the following manner: when the concrete in the precast member has set but not hardened, the surface which will subsequently receive then in –situ concrete should be sprayed with a fine spray of water or brushed with a stiff brush, just sufficient to remove the outer mortar skin and expose the large aggregate without disturbing it.

17.5.2.2- When minimum ties are provided in accordance with Section 17.6, and the contact surface is not as described in Section 17.5.2.1, shear strength V_h shall not be taken greater than $0.4\ b_v\ d$ (in Newtons).

17.5.2.3- When the contact surface has been prepared as described in Section 17.5.2.1 (or where this treatment proved impracticable and the surface skin and laitance has been removed by sand blasting or the use of a needle gun and not by hacking) and minimum ties as described in Section 17.6 are provided, shear strength shall not be taken greater than $1.3\ b_v\ d$ (in Newtons).

17.5.2.4- When minimum ties are provided in accordance with Section 17.6, and contact surfaces are clean, free of laitance, and intentionally roughened to a full amplitude of approximately 6mm, shear strength V_h shall not be taken greater than $2.2\ b_v\ d$ (in Newtons).

17.5.2.5- In composite slabs when ties are not provided, the horizontal shear stresses may be limited as follows:

a) for the contact surface as described in Section 17.5.2.1, the shear strength should not exceed $0.6\ b_v\ d$ (in Newtons).

b) where the top surface of the precast unit has not been treated, in accordance with Section 17.5.2.1, the shear should not exceed $0.3\ b_v\ d$ (in Newtons).

17.5.2.6- When factored shear force V_u at section considered exceeds the limits given in Sections 17.5.2.1 through 17.5.2.5, design for horizontal shear shall be in accordance with Section 11.5.

17.5.3- Horizontal shear may be investigated by computing the actual change in compressive or tensile foce in any segment, and provisions made to transfer that force as horizontal shear to the supporting element. The factored horizontal shear force shall not exceed 85 percent of horizontal shear strength as given in Section 17.5.2.1 through 17.5.2.5, where area of contact surface A_c shall be substituted for $b_v\ d$.

17.5.4- When tension exists across any contact surface between interconnected elements, shear transfer by contact may be assumed only when minimum ties are provided in accordance with Section 17.6.

17.6- Ties for horizontal shear

17.6.1- When ties are provided to transfer horizontal shear, tie area shall not be less than 0.0015 of the contact area, and tie spacing shall not exceed four times the least dimension of the in—situ concrete, nor 600mm.

17.6.2- Ties for horizontal shear may consist of single bars or wire, multiple leg stirrups, or vertical legs of welded wire fabric (smooth or deformed).

17.6.3- All ties shall be fully anchored into interconnected elements in accordance with Section 12.12.

CHAPTER 18- TESTING OF STRUCTURES AND COMPONENTS

18.0- Notation

D = dead loads, or related internal moments and forces
h = overall thickness of member
ℓ= span of member under test load (shorter span of flat slabs and of slabs supported on four sides). Span of member, except as provided in Section 18.2.2.7, is distance between centers of supports or clear distance between supports plus depth of the member, whichever is smaller
L = live loads, or related internal moments and forces

18.1- General

18.1.1- If doubt develops concerning the safety of a structure or a member, the Engineer may order strength and serviceability investigation by means of load tests and/or analyses.

18.1.2- The investigation is intended to assess the soundness of the structure with regards to its ultimate limit state and serviceability limit state of deflection and cracking. Durability of a structure cannot be assessed directly from load test results.

18.2- Load Tests

18.2.1- General

18.2.1.1- Load tests are to be applied to flexural members only.

18.2.1.2- A qualified engineer shall control the test.

18.2.1.3- A load test shall not be made until the portion to be tested is at least 56 days old.

18.2.1.4- Forty-eight hours prior to application of test load, a load to simulate effect of that protion of the dead load not already acting shall be applied and shall remain in place until all testing has been completed.

18.2.1.5 - A load test shall be carried out, as possible, at such times when effects of variation of temperature and humidity are minimum.

18.2.1.6 - Base readings (datum for deflection measurements) shall be made immediately prior to application of test load. More than one deflection reading is preferably taken within the vicinity of maximum deflection in order to obtain more reliable readings of deflections since the exact position of maximum deflection cannot always be determined.

18.2.1.7- All measured deflections shall be the net deflections of the members. i.e. deflections shall be adjusted for any support movement.

18.2.1.8- It is recommended that deflection recovery measurement be taken even if the deflection under test load does not reach the deflection limit below which recovery measurement is not required. This may help in giving a better view of the behavior of the structure and may be of value in case of any doubt concerning some readings.

18.2.1.9- The material used for loading shall not be stored anywhere at the same level or story or near the area to be tested unless it is structurally isolated, but shall directly be placed on the tested area when testing begins.

18.2.1.10- The member to be tested and instrumentation shall, as possible, be protected from direct sunlight, rain, wind or other phenomena that might affect the results of the test.

18.2.2- Strength test requirements and procedure

18.2.2.1- A Load test may be made on the whole structure or a portion therof. An analysis is required to select the questionable portion on the basis that:

a) it is the weakest and/or most affected by the drop in concrete strength.

b) it is subjected to the most severe loading condition.

18.2.2.2- That portion of the structure selected for loading shall be subject to a total load, including dead loads already acting, equivalent to 85 percent of the design factored load. Effect of loadsharing must be taken into consideration, i.e. loading other parts of the structure that may cause additional deflection at the tested portion.

18.2.2.3- Test loads shall be applied uniformly in not less than four approximately equal increments without shock to the structure and in such manner as to avoid arching of load materials. Deflection readings shall be taken for each load increment.

18.2.2.4- After total test load has been in position for 24 hours, deflection reading shall be taken.

18.2.2.5- Test load shall be removed immediately after readings of Section 18.2.2.4 are taken, and a final deflection reading shall be taken 24 hours after removal of test load.

18.2.2.6- In assessing the strength of the portion tested, the following criteria shall be taken as indication of satisfactory behavior:

a) If measured maximum deflection of the member tested is less than ℓ^2**/25000h**, where ℓ is the span length and **h** is the thickness of the member, both having the same units.

b) If measured maximum deflection of the member tested exceeds the limits in (a), deflection recovery with 24hours after removal of the test load shall be at least 75 percent of the maximum deflection.

18.2.2.7- In Section 18.2.2.6, ℓ for cantilevers shall be taken as twice the distance from the face of support to cantilever end.

18.2.2.8- Structures failing to show 75 percent recovery of deflection as required by Section 18.2.2.6(b) may be retested not earlier than 72 hours after removal of the first test load. The portion of the structure tested shall be considered satisfactory if the delection recovery caused by the second test load is at least 75 percent of the deflection in the second test.

18.2.3- Serviceability test requirements and procedures

18.2.3.1- The serviceability test is not to be applied to a structure until a decision is made regarding the load bearing ability of that structure by either analysis or strength load test.

18.2.3.2- Member selected for testing shall be subject to a total load equivalent to **(D+L)**.

18.2.3.3- The live load portion of the test load shall be applied in not less than two approximately equal increments without shock to the structure and in such manner as to avoid arching of load materials.

18.2.3.4- After the application of the full load:

a) maximum deflection readings shall be taken, and

b) maximum crack width shall be measured.

18.2.3.5- In order for a structure to be serviceable the following shall be met:

a) maximum deflection is less than ℓ/**500**.

b) maximum crack width is less than 0.25 mm.

18.2.3.6- No serviceability test is required if the serviceability requirements are met during the strength test load.

18.3- Members other than flexural

Members other than flexural members shall preferably be investigated by analysis.

18.4- Provisions for lower load rating

If structure under investigation does not satisfy conditions or criteria of loading test and analysis, a lower rating for that structure based on results of the load test or analysis may be approved.

18.5- Safety

18.5.1- Load tests shall be conducted in such a manner as to ensure the safety of life and strucutre during the test.

18.5.2- Safety mesurements shall not interfere with load test procedures or affect the test results.

APPENDIX A - RECOGNIZED INTERNATIONAL STANDARDS

1. B.S 5328	:1981	Methods for Specifying Concrete Including Ready-mixed Concrete.
2. ASTM C94-81		Standard Specification For Ready-Mixed Concrete.
3. B.S. 1881	: 1983	Method For Determination Of Tensile Splitting Strength.
4. B.S. 3148	:1980	Method Of Test For Water For Making Concrete (Including Notes On The Suitability Of Water).
5. ASTM C494-82		Specification For Chemical Admixtures For Concrete.
6. B.S. 5075:Part 1	:1982	Specification For Accelerating Admixtures, Retarding Admixtures And Water Reducing Admixtures.
7. ASTM C260-77		Specification For Air-Entraining.
8. B.S. 5075: Part 2	:1982	Admixtures For Concrete, Specification For Air-Entraining Admixtures.
9. ASTM C618-83		Specification For Fly Ash and Raw or Calcined Natural Pozzolana For Use as a Mineral Admixture in Portland Cement Concrete.
10. ASTM A36-81a		Standard Specification For Structural Steel.
11. B.S. 4360	:1979	Specification For Weldable Structural Steels.
12. ASTM A82-79		Standard Specification For Cold-Drawn Steel Wire For Concrete Reinforcement.
13. ASTM A184-79		Standard Specification For Fabricated Deformed Steel Bar Mats For Concrete Reinforcement.
14. B.S. 4483	:1969	Steel Fabric For The Reinforcement Of Concrete AMD 955.1972.
15. ASTM A185-79		Standard Specification For Welded Steel Wire Fabric For Concrete Reinforcement.
16. ASTM A242-81		Standard Specification For High-Strength Low-Alloy Structural Steel.
17. B.S. 4449	:1978	Specification For Hot Rolled Steel Bars For Reinforcement Of Concrete. AMD 3010 - 1979 AMD 3559 - 1981 AMD 4337 - 1983 AMD 4540 - 1984
18. B.S. 4461	:1968 (1984)	Specification For Cold Worked Steel Bars For The Reinforcement Of Concrete. AMD 2987 - 1979 AMD 3560 - 1981 AMD 4338 - 1983
19. ASTM A496-78		Standard Specification For Deformed Steel Wire For Concrete Reinforcement.
20. ASTM A497-79		Standard Specification For Welded Deformed Steel Wire Fabric For Concrete Reinforcement.
21. ANSI A58.1-	:1982	Building Code Requirements For Minimum Design Loads in Buildings and Other Structures.

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