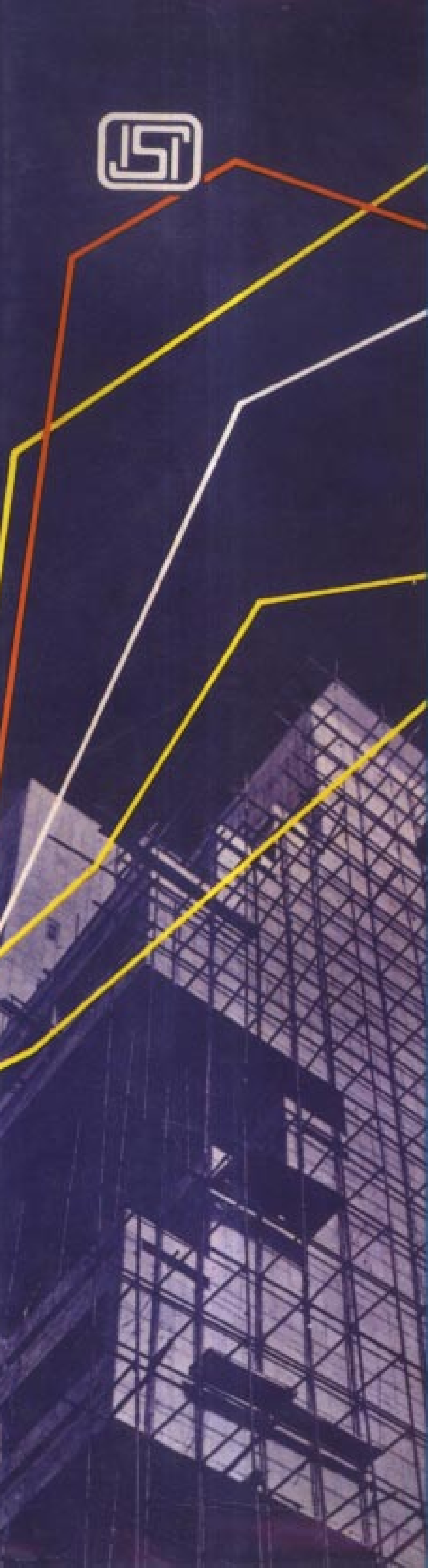

# EXPLANATORY HANDBOOK ON INDIAN STANDARD CODE OF PRACTICE FOR PLAIN AND REINFORCED CONCRETE

IS : 456 - 1978

# EXPLANATORY HANDBOOK
# ON
# INDIAN STANDARD CODE OF PRACTICE
# FOR
# PLAIN AND REINFORCED CONCRETE
# (IS : 456-1978)

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# (IS : 456-1978)

BUREAU OF INDIAN STANDARD
MANAK BHAVAN, 9 BAHADURSHAH ZAFAR MARG
NEW DELHI 110002

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# FOREWORD

Users of various civil engineering codes have been feeling the need for explanatory handbooks and other compilations based on Indian Standards. The need has been further emphasized in view of the publication of the National Building Code of India 1970 and its implementation. In 1972, the Department of Science and Technology set up an Expert Group on Housing and Construction Technology under the Chairmanship of Maj-Gen Harkirat Singh. The Group carried out in-depth studies in various areas of civil engineering and construction practices. During the preparation of the Fifth Five Year Plan in 1975, the Group was assigned the task of producing a Science and Technology plan for research, development and extension work in the sector of housing and construction technology. One of the items of this plan was the production of design handbooks, explanatory handbooks and design aids based on the National Building Code and various Indian Standards and other activities in the promotion of National Building Code. The Expert Group gave high priority to this item and on the recommendation of the Department of Science and Technology, the Planning Commission approved the following two projects which were assigned to the Indian Standards Institution:

a) Development programme on Code implementation for building and civil engineering construction, and
b) Typification for industrial buildings.

A Special Committee for Implementation of Science and Technology Projects (SCIP) consisting of experts connected with different aspects was set up in 1974 to advise the ISI Directorate General in identification and for guiding the development of the work under the chairmanship of Maj-Gen Harkirat Singh, Retired Engineer-in-Chief, Army Headquarters and formerly Adviser (Construction) Planning Commission, Government of India. The Committee has so far identified subjects for several explanatory handbooks/compilations covering appropriate Indian Standards/Codes/Specifications which include the following:

Functional Requirements of Buildings
Functional Requirements of Industrial Buildings
Summaries of Indian Standards for Building Materials*
Building Construction Practices
Foundation of Buildings
Explanatory Handbook on Codes for Earthquake Engineering (IS : 1893-1975, IS : 4326-1976)†
Design Aids for Reinforced Concrete to IS : 456-1978†
Explanatory Handbook on Masonry Code†
Explanatory Handbook on Indian Standard Code of Practice for Plain and Reinforced Concrete (IS : 456-1978)†
Handbook on Concrete Mixes†

*Under print.
†Printed.

Concrete Reinforcement Detailing Handbook
Cracks in Buildings*
Formwork
Timber Engineering
Steel Code (IS : 800)
Loading Code
Fire Safety
Prefabrication
Tall Buildings
Inspection of Different Items of Building Work
Bulk Storage Structures in Steel
Construction Safety Practices

One of the explanatory handbooks identified is on IS : 456-1978 Code of Practice for Plain and Reinforced Concrete (*third revision*). This explanatory handbook provides information on the basis/source, interpretation/explanations to certain Clauses, worked-out examples and sketches to illustrate the application of Codal provisions wherever required.

The 1964 version of IS : 456 was in use for over 15 years. When the revision of this Code was taken up, new features, such as limit state design principles and substantial modifications of important sections were introduced, making it more comprehensive and up-to-date. It was suggested by the committee responsible for the revision of the Code IS : 456-1978 that design aids and explanatory information which could not be covered in the Code should be made available to the users of the Code.

Some of the important points to be kept in view in the use of this handbook are:

a) In this handbook wherever the expression 'the Code' has been used it refers to IS : 456-1978 Code of Practice for Plain and Reinforced Concrete (*third revision*).
b) This handbook is to be read along with IS : 456-1978.
c) The Clause numbers in this handbook refer to the relevant Clause numbers in IS : 456-1978. The Clauses are explained in the same sequence as they occur in IS : 456-1978. Wherever there is no explanation to a particular Clause, only Clause number is mentioned.
d) For convenience, Figures and Tables appearing in the explanatory handbook are identified with a prefix 'E' to distinguish them from those used in the Code. For example, Fig. E-3 refers to the figure in the explanatory handbook whereas Fig. 3 refers to that given in the Code.
e) ***Wherever there is any dispute about the interpretation or opinion expressed in this handbook, the provisions of the Code only shall apply; the provisions of this handbook should be considered as only supplementary and informative.***
f) Notations as per IS : 456-1978 are maintained with additional notations wherever necessary.
g) Reports of committees, papers published in technical periodicals and textbooks referred in the preparation of this handbook have been cited appropriately. These have been listed at the end of this publication. However, reference to a particular work does not necessarily imply that it is the only source or the original source.

The explanatory handbook is based on the first draft prepared by Cement Research Institute of India, New Delhi. The draft handbook was circulated for review to Central Public Works Department, New Delhi; Engineer-in-Chief's

*Under print.

Branch, Army Headquarters, New Delhi; Concrete Association of India, Bombay; Andhra Pradesh Engineering Research Laboratories, Hyderabad; Cement Corporation of India, New Delhi; Tata Consulting Engineers, Bombay; M/s C. R. Narayana Rao, Madras; Research, Designs and Standards Organization, Lucknow; Gammon India Limited, Bombay; Ministry of Shipping and Transport (Roads Wing), New Delhi; and the views received were taken into consideration while finalizing the handbook.

As in the Original Standard, this Page is Intentionally Left Blank

# CONTENTS


| | Page |
|---|---|
| 0. FOREWORD | 3 |
| **SECTION 1 GENERAL** | 5 |
| 1. SCOPE | 7 |
| 2. TERMINOLOGY | 7 |
| 3. SYMBOLS | 7 |
| **SECTION 2 MATERIALS, WORKMANSHIP, INSPECTION AND TESTING** | 9 |
| 4. MATERIALS | 11 |
| 4.1 Cement | 11 |
| 4.2 Aggregates | 12 |
| 4.3 Water | 14 |
| 4.4 Admixtures | 15 |
| 4.5 Pozzolanas | 16 |
| 4.6 Reinforcement | 17 |
| 4.7 Storage of Materials | 19 |
| 5. CONCRETE | 20 |
| 5.1 Grades | 20 |
| 5.2 Properties of Concrete | 20 |
| 6. WORKABILITY OF CONCRETE | 22 |
| 7. DURABILITY | 23 |
| 8. CONCRETE MIX PROPORTIONING | 24 |
| 8.1 Mix Proportion | 24 |
| 8.2 Design Mix Concrete | 25 |
| 8.3 Nominal Mix Concrete | 25 |
| 9. PRODUCTION AND CONTROL OF CONCRETE | 26 |
| 9.1 General | 26 |
| 9.2 Batching | 26 |
| 9.3 Mixing | 26 |
| 10. FORMWORK | 27 |
| 10.1 General | 27 |
| 10.2 Cleaning and Treatment of Forms | 27 |
| 10.3 Stripping Time | 27 |
| 11. ASSEMBLY OF REINFORCEMENT | 27 |
| 12. TRANSPORTING, PLACING, COMPACTING AND CURING | 29 |
| 12.1 Transporting | 29 |

| | | Page |
|---|---|---|
| | 12.2 Placing | 29 |
| | 12.3 Compaction | 29 |
| | 12.4 Construction Joints | 30 |
| | 12.5 Curing | 31 |
| | 12.6 Supervision | 32 |
| 13. | CONCRETING UNDER SPECIAL CONDITIONS | 32 |
| | 13.1 Work in Extreme Weather Conditions | 32 |
| | 13.2 Under-Water Concreting | 32 |
| | 13.3 Concrete in Sea-Water | 33 |
| | 13.4 Concrete in Aggressive Soils and Water | 33 |
| 14. | SAMPLING AND STRENGTH TEST OF CONCRETE | 33 |
| | 14.1 General | 34 |
| | 14.2 Frequency of Sampling | 34 |
| | 14.3 Test Specimens | 34 |
| | 14.4 Test Strength of Sample | 35 |
| | 14.5 Standard Deviation | 35 |
| 15. | ACCEPTANCE CRITERIA | 37 |
| 16. | INSPECTION AND TESTING OF STRUCTURES | 39 |
| SECTION 3 | GENERAL DESIGN REQUIREMENTS | 41 |
| 17. | LOADS AND FORCES | 43 |
| | 17.1 General | 43 |
| | 17.2 Dead Loads | 43 |
| | 17.3 Live Loads and Wind Loads | 43 |
| | 17.4 Earthquake Forces | 43 |
| | 17.5 Shrinkage, Creep and Temperature Effects | 43 |
| | 17.6 Other Forces and Effects | 44 |
| | 17.7 Combination of Loads | 45 |
| | 17.8 Dead Load Counteracting Other Loads and Forces | 45 |
| | 17.9 Design Loads | 45 |
| 18. | BASES FOR DESIGN | 45 |
| | 18.1 General | 45 |
| | 18.2 Methods of Design | 45 |
| | 18.3 Design on Experimental Basis | 45 |
| 19. | STABILITY OF THE STRUCTURE | 45 |
| | 19.1 Overturning | 45 |
| | 19.2 Sliding | 45 |
| 20. | DURABILITY AND FIRE RESISTANCE OF THE STRUCTURE | 46 |
| | 20.1 Durability | 46 |
| | 20.2 Fire Resistance | 46 |
| 21. | ANALYSIS | 46 |
| | 21.1 General | 46 |
| | 21.2 Effective Span | 47 |
| | 21.3 Stiffness | 48 |
| | 21.4 Structural Frames | 49 |
| | 21.5 Moment and Shear Coefficients for Continuous Beams | 51 |
| | 21.6 Critical Sections for Moment and Shear | 51 |
| | 21.7 Redistribution of Moments | 52 |
| 22. | BEAMS | 53 |
| | 22.0 Effective Depth | 53 |
| | 22.1 T-Beams and L-Beams | 53 |

Page


22.2 Control of Deflection 53
22.3 Slenderness Limits for Beams to Ensure Lateral Stability 55
23. SOLID SLABS 55
23.1 General 55
23.2 Slabs Continuous Over Supports 56
23.3 Slabs Monolithic with Supports 56
23.4 Slabs Spanning in Two Directions at Right Angles 56
23.5 Loads on Supporting Beams 56
24. COMPRESSION MEMBERS 57
24.1 Definitions 57
24.2 Effective Length of Compression Members 57
24.3 Slenderness Limits for Columns 57
24.4 Minimum Eccentricity 57
25. REQUIREMENTS GOVERNING REINFORCEMENT AND DETAILING 57
25.1 General 57
25.2 Development of Stress in Reinforcement 58
25.2.1 Development Length of Bars 58
25.2.2 Anchoring Reinforcing Bars 58
25.2.3 Curtailment of Tension Reinforcement in Flexural Members 60
25.2.4 Special Members 63
25.2.5 Reinforcement Splicing 63
25.3 Spacing of Reinforcement 66
25.4 Cover to Reinforcement 67
25.5 Requirements of Reinforcement for Structural Members 68
25.5.1 Beams 68
25.5.2 Slabs 70
25.5.3 Columns 70
26. EXPANSION JOINTS 72

SECTION 4 SPECIAL DESIGN REQUIREMENTS FOR STRUCTURAL MEMBERS AND SYSTEMS 73

27. GENERAL 75
28. DEEP BEAMS 75
28.1 General 75
28.2 Lever Arm 75
28.3 Reinforcement 75
29. RIBBED, HOLLOW BLOCK OR VOIDED SLAB 77
29.1 General 77
29.2 Analysis of Structure 79
29.3 Shear 79
29.4 Deflection 79
29.5 Size and Position of Ribs 79
29.6 Hollow Blocks and Formers 79
29.7 Arrangement of Reinforcement 79
30. FLAT SLABS 79
30.1 General 79
30.2 Proportioning 80
30.3 Determination of Bending Moment 81
30.4 Direct Design Method 82

Page


30.5 Equivalent Frame Method 85
30.6 Shear in Flat Slab 93
30.7 Slab Reinforcement 98
30.8 Openings in Flat Slabs 98
31. WALLS 98
31.1 General 98
31.2 Load Carrying Capacity 99
31.3 Slenderness Effects 99
31.4 Reinforcement 99
32. STAIRS 99
32.1 Effective Span of Stairs 99
32.2 Distribution of Loading on Stairs 100
32.3 Depth of Section 100
33. FOOTINGS 100
33.1 General 100
33.2 Moments and Forces 101
33.3 Tensile Reinforcement 102
33.4 Transfer of Load at the Base of Column 102

SECTION 5 STRUCTURAL DESIGN (LIMIT STATE METHOD) 105

34. SAFETY AND SERVICEABILITY REQUIREMENTS 107
34.1 General 107
34.2 Limit State of Collapse 108
34.3 Limit States of Serviceability 109
34.4 Other Limit States 111
35. CHARACTERISTIC AND DESIGN VALUES AND PARTIAL SAFETY FACTORS 111
35.1 Characteristic Strength of Materials 111
35.2 Characteristic Loads 111
35.3 Design Values 112
35.4 Partial Safety Factors 112
36. ANALYSIS 115
36.1 Analysis of Structure 115
37. LIMIT STATE OF COLLAPSE: FLEXURE 119
37.1 Assumptions 119
38. LIMIT STATE OF COLLAPSE: COMPRESSION 120
38.1 Assumptions 120
38.2 Minimum Eccentricity 120
38.3 Short Axially Loaded Members in Compression 120
38.4 Compression Members with Helical Reinforcement 121
38.5 Members Subject to Combined Axial Load and Uniaxial Bending 122
38.6 Compression Member Subject to Combined Axial Load and Biaxial Bending 122
38.7 Slender Compression Members 122
39. LIMIT STATE OF COLLAPSE: SHEAR 126
39.1 Nominal Shear Stress 126
39.2 Design Shear Strength of Concrete 126
39.3 Minimum Shear Reinforcement 127
39.4 Design of Shear Reinforcement 127

| | | Page |
|---|---|---|
| 40. | LIMIT STATE OF COLLAPSE: TORSION | 128 |
| | 40.1 General | 128 |
| | 40.2 Critical Section | 130 |
| | 40.3 Shear and Torsion | 130 |
| | 40.4 Reinforcement in Members Subjected to Torsion | 130 |
| 41. | LIMIT STATE OF SERVICEABILITY: DEFLECTION | 131 |
| | 41.1 Flexural Members | 131 |
| 42. | LIMIT STATE OF SERVICEABILITY: CRACKING | 132 |
| | 42.1 Flexural Members | 132 |
| | 42.2 Compression Members | 132 |

SECTION 6 STRUCTURAL DESIGN (WORKING STRESS METHOD) 133

| | | |
|---|---|---|
| 43. | GENERAL | 135 |
| | 43.1 General Design Requirements | 135 |
| | 43.2 Redistribution of Moments | 135 |
| | 43.3 Assumptions for Design of Members | 137 |
| 44. | PERMISSIBLE STRESSES | 137 |
| | 44 1 Permissible Stresses in Concrete | 137 |
| | 44.2 Permissible Stresses in Steel Reinforcement | 138 |
| | 44.3 Increase in Permissible Stresses | 139 |
| 45. | PERMISSIBLE LOADS IN COMPRESSION MEMBERS | 139 |
| | 45.1 Pedestals and Short Columns with Lateral Ties | 139 |
| | 45.2 Short Columns with Helical Reinforcement | 139 |
| | 45.3 Long Columns | 139 |
| | 45.4 Composite Columns | 140 |
| 46. | MEMBERS SUBJECT TO COMBINED AXIAL LOAD AND BENDING | 140 |
| | 46.1 Design Based on Uncracked Section | 140 |
| | 46.2 Design Based on Cracked Section | 140 |
| | 46.3 Members Subjected to Combined Direct Load and Flexure | 141 |
| 47. | SHEAR | 141 |
| | 47.1 Nominal Shear Stress | 141 |
| | 47.2 Design Shear Strength of Concrete | 141 |
| | 47.3 Minimum Shear Reinforcement | 141 |
| | 47.4 Design of Shear Reinforcement | 141 |
| 48. | TORSION | 141 |
| | 48.1 General | 141 |
| | 48.2 Critical Section | 141 |
| | 48.3 Shear and Torsion | 141 |
| | 48.4 Reinforcement in Members Subjected to Torsion | 141 |

APPENDICES 143

| | | |
|---|---|---|
| APPENDIX A | REQUIREMENTS FOR DURABILITY | 145 |
| APPENDIX B | CALCULATION OF DEFLECTION | 146 |
| APPENDIX C | SLABS SPANNING IN TWO DIRECTIONS | 148 |
| APPENDIX D | EFFECTIVE LENGTH OF COLUMNS | 150 |
| APPENDIX E | MOMENTS OF RESISTANCE FOR RECTANGULAR AND T-SECTIONS | 154 |

TABLES

| | | Page |
|---|---|---|
| E-1 | Requirements for Reinforcing Bars | 18 |
| E-2 | Deduction for Tolerances in Position of Bent Steel | 28 |
| E-3 | Fire Resistance of Reinforced Concrete Elements | 48 |
| E-4 | Moment Distribution Constants for Slabs Without Drop Panels | 88 |
| E-5 | Moment Distribution Constants for Slabs with Drop Panels | 89 |
| E-6 | Moment Distribution Constants for Slab-Beam Members with Column Capitals | 90 |
| E-7 | Moment Distribution Constants for Slab-Beam Members with Column Capitals and Drop Panels | 91 |
| E-8 | Expression for $A_c$, $\bar{X}$ and $J_{xx}$ for Typical Cases of Support in Flat Slabs | 95 |

FIGURES


| | | Page |
|---|---|---|
| E-1 | Typical Shrinkage Curve for Concrete | 21 |
| E-2 | Bending Dimensions for Stirrups | 27 |
| E-3 | Normal Distribution of Concrete Strength | 35 |
| E-4 | Typical Normal Frequency Curves for Different Levels of Control | 36 |
| E-5 | Sketch for Transformed Sections | 49 |
| E-6 | Permissible Simplification of Frame for Analysis | 50 |
| E-7 | Moments at Interior Supports Due to Restraint at Support | 51 |
| E-8 | Bending Moment at Face of Support | 52 |
| E-9 | Typical Support Conditions for Loading Critical Sections for Shear | 52 |
| E-10 | Effective Width of One Way Solid Slab Carrying a Concentrated Load Near an Unsupported Edge | 56 |
| E-11 | Standard Hook and Bend | 58 |
| E-12 | Anchoring Inclined Bent-Up Bar in Tension Zone | 59 |
| E-13 | Anchoring Inclined Bent-Up Bar in Compression Zone | 59 |
| E-14 | Spalling of Thin Cover Over Stirrups | 59 |
| E-15 | Plan-Bearing Stress at Bends | 60 |
| E-16 | Radial Forces in Reinforcement | 60 |
| E-17 | Bars Stopped at Theoretical Cut-Off Point (E) | 61 |
| E-18 | Development Rule for Section at Support | 62 |
| E-19 | Typical Method of Anchoring Main Tension Reinforcement in Corbels | 64 |
| E-20 | Lap Splices for Flexural Tension Bars | 64 |
| E-21 | Lap Splicing of Welded Wire Fabrics | 65 |
| E-22 | End Bearing Splice | 66 |
| E-23 | Minimum Bar Spacing Between Groups of Bars | 67 |
| E-24 | Shear and Torsional Reinforcement in Flanged Beams | 69 |
| E-25 | Improper Confinement of Core in a Column | 71 |
| E-26 | Splice with Offset Cranked Bar in a Column | 72 |
| E-27 | Reinforcement Detailing in Simply Supported Deep Beams | 76 |
| E-28 | Disposition of Negative Reinforcement in Continuous Deep Beams (Clause 28.3.2) | 77 |
| E-29 | Suspender Bars for Deep Beams | 78 |
| E-30 | Hollow Clay-Block Floor | 78 |
| E-31 | Panels, Column Strips and Middle Strips | 80 |
| E-32 | Transfer of Moments to Columns | 81 |
| E-33 | Equivalent Square Sections for Typical Cross Sections | 83 |
| E-34 | Rules for Dividing the Total Static Moment into Negative and Positive Design Moments | 84 |
| E-35 | Idealization for Equivalent Frame | 85 |
| E-36 | Idealization for Column Types of Slat Elements for Continuous Slab-Beam Stiffness | 86 |
| E-37 | Sections for Calculating Column Stiffness ($K_c$) | 87 |
| E-38 | Reduction of Negative Moments at Interior Supports (For Equivalent Frame Moment) | 92 |

| | | Page |
|---|---|---|
| E-39 | Moment Variation in Flat Slabs | 93 |
| E-40 | Reinforcement for Punching Shear | 97 |
| E-41 | Effective Span of Stairs | 99 |
| E-42 | Face of the Column for Computing Stresses in Round and Octagonal Columns | 101 |
| E-43 | Critical Section for Bending Moment and Shear in Footing | 101 |
| E-44 | Determination of $A_1$ in Footing | 103 |
| E-45 | Crack Width in Beams | 110 |
| E-46 | Moment Rotation Plot for Under-Reinforced Sections | 116 |
| E-47 | Collapse Load for Fixed Beam with Fixed Plastic Moments | 117 |
| E-48 | Redistribution of Bending Moments | 117 |
| E-49 | Assumed Stress Strain Conditions at Limit State of Collapse in Flexural Bending | 119 |
| E-50 | Combined Axial Load and Uniaxial Bending | 121 |
| E-51 | Behaviour of Columns with and Without Helical Reinforcement | 122 |
| E-52 | Moment Curvature Relation for Pin-End Column | 123 |
| E-53 | Moment in Braced Slender Column | 124 |
| E-54 | Additional Moments in Column with Side Sway | 125 |
| E-55 | Imaginary Truss System for Shear with Stirrups | 128 |
| E-56 | Failure Modes for Combined Bending and Torsion | 129 |
| E-57 | Forces Acting on Plane of Failure, Mode 1 | 131 |
| E-58 | Redistribution of Moments | 136 |
| E-59 | Envelope of Bending Moments After and Before Distribution | 137 |
| E-60 | Shrinkage Curvature | 147 |
| E-61 | Assumptions of Creep Deflection | 147 |
| E-62 | Simply Supported Slab Rankine Grashof Formula | 148 |
| E-63 | Support Conditions for Slabs of Table 21 | 149 |
| E-64 | Corner Reinforcement for Torsional Resistance (Showing Top Two Layers only) | 149 |
| E-65 | Buckling Modes for Rectangular Frames | 150 |
| E-66 | Column with No Sway | 151 |
| E-67 | Braced Column in a Frame | 151 |
| E-68 | Limited Substitute Frame | 152 |
| E-69 | Stiffness Values | 153 |
| E-70 | Distribution of Stress in a T-Beam | 155 |

# FOREWORD

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# 0. FOREWORD

*0.4* This clause gives the significant changes that have been made in the 1978 version of the Code. The notable ones are:

a) Introduction of limit state design; this replaces the ultimate load method covered in Appendix B of IS : 456-1964;

b) The Working Stress Method is now aligned with Limit State Method of Design; and

c) SI units have been used in the present Code.

This Code is expected to be used as one package for the design of concrete structures in general building construction. It does not advocate the use of different provisions from different Codes in the design of concrete structures. However, for the design of special structures, such as shell structures, folded plates, arches, bridges, chimneys, blast resistant structures, hydraulic structures and liquid retaining structures, specific requirements as specified in the respective Codes shall be adopted in conjunction with the provisions of the Code as far as they are applicable. Some of the Indian Standards available for the design of reinforced concrete special structures are*:

IS : 3370 (Part I)-1965 Code of practice for concrete structures for the storage of liquids: Part I General requirements

IS : 3370 (Part II)-1965 Code of practice for concrete structures for the storage of liquids: Part II Reinforced concrete structures

IS : 2210-1962 Criteria for the design of reinforced concrete shell structures and folded plates

IS : 4090-1967 Criteria for the design of reinforced concrete arches

IS : 4995 (Part I)-1974 Criteria for design of reinforced concrete bins for storage of granular and powdery materials: Part I General requirements and assessment of bin loads (*first revision*)

IS : 4995 (Part II)-1974 Criteria for design of reinforced concrete bins for storage of granular and powdery materials: Part II Design criteria

IS : 4998 (Part I)-1975 Criteria for design of reinforced concrete chimneys: Part I Design criteria (*first revision*)

IS : 4880 (Part IV)-1971 Code of practice for design of tunnels conveying water: Part IV Structural design of concrete lining in rock

IS : 4880 (Part V)-1972 Code of practice for design of tunnels conveying water: Part V Structural design of concrete lining in soft strata and soils

IS : 7563-1975 Code of practice for structural design of cut and cover concrete conduits

IS : 4247 (Part II)-1978 Code of practice for structural design of surface hydel power stations: Part II Superstructure (*first revision*)

IS : 4247 (Part III)-1978 Code of practice for structural design of surface hydel power stations: Part III Substructure (*first revision*)

IS : 2911 (Part I/Sec 1)-1979 Code of practice for design and construction of pile foundations: Part I Concrete piles, Section 1 Driven cast *in-situ* piles (*first revision*)

IS : 2911 (Part I/Sec 2)-1979 Code of practice for design and construction of pile foundations: Part I Concrete piles, Section 2 Bored cast *in- situ* concrete piles (*first revision*)

*Most of the above codes have been taken up for revision and the limit state design concept will be followed in their next revisions.

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## SECTION 1

# GENERAL

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# SECTION 1 GENERAL

*1.* SCOPE

It is also to be noted that the Code is not directly applicable to precast concrete structures and may be applied with such modifications as found necessary to suit special conditions of each individual case.

*2.* TERMINOLOGY

Some of the additional definitions such as Gross section (*21.3.1*), Transformed section (*12.3.1*), Cracked section (*12.3.1*), Column or strut (*24.1.1*), Short and slender compression member (*24.1.2*), Characteristic load (*35.2*), Characteristic strength (*35.1*), Pedestal (*25.5.3.1*) have been defined in the Code at appropriate places.

*3.* SYMBOLS

It is to be noted from *0.3.4* of the Code that most of the notations have been adopted from International Standards Organization (ISO 3898-1976*) with familiar symbols of the old Code retained to a large extent. The adoption of ISO symbols would facilitate easy comparison between the various National Standards and Codes and also in easy understanding technical literature on the subject from technical journals.

*Bases for design of structures—Notations—General symbols.

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SECTION 2

# MATERIALS, WORKMANSHIP, INSPECTION AND TESTING

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# SECTION 2 MATERIALS, WORKMANSHIP, INSPECTION AND TESTING

*4.* MATERIALS

*4.1 Cement* — The type of cement selected should be appropriate to the intended use. The different types of cements are generally made by the adjustment in relative proportions of chemical compounds and the fineness to suit the particular requirement. The choice of a particular type of cement may also necessitate modifications in other clauses of the Code. For instance, the clauses on workmanship may need modification in the following cases:

a) Stripping time of formwork indicated in Clause *10.3* applies to ordinary Portland cement. Other cements, especially rapid hardening cement and low heat Portland cement may require suitable modification.

b) The minimum cement content for different sulphate attack depends upon the type of cement (*see* Table 20).

RAPID HARDENING PORTLAND CEMENT — Rapid hardening Portland cement has similar properties as that of ordinary Portland cement, except that the former is more finely ground and slightly altered in chemical composition. The so-called 'super fine Portland cement' is also covered under rapid hardening Portland cement by IS : 8041-1978*. Rapid hardening cement gains strength more rapidly at earlier ages, but has a strength comparable to that of ordinary Portland cement at 28 days. Its fineness leads to increase in shrinkage which should be accounted for in design.

PORTLAND SLAG CEMENT — Portland slag cement is manufactured either by intimately intergrinding a mixture of Portland cement clinker and granulated slag with addition of gypsum, or by an intimate and uniform blending of Portland cement and finely ground granulated slag. The resultant product has physical properties similar to those of ordinary Portland cement. In addition, it has low heat of hydration and with favourable slag contents has relatively better resistance to soils and water containing excessive amounts of sulphate, as well as to acid waters and can, therefore, be used for marine works with advantage (*see 13.3*). However, Indian slags are characterized by comparatively higher alumina contents and the slag content may vary between 25 to 65 percent. It is suggested that manufacturer's recommendations or specialists advice should be sought if the Portland slag cement is to be chosen for its resistance to sulphates attack and marine environments.

PORTLAND POZZOLANA CEMENT — Pozzolana used in the manufacture of Portland pozzolana cement may be either a natural material such as diatomaceous earth or materials processed by calcination of soil (for example, burnt clay pozzolana) or artificial material such as fly ash. Portland pozzolana cement can be produced either by grinding together Portland cement clinker and pozzolana in proportions of 10 to 25 percent with the addition of gypsum, or by intimately and uniformly blending ordinary Portland cement and fine pozzolana.

This cement produces less heat of hydration and offers greater resistance to the attack of aggressive waters than ordinary Portland cement. It is particularly useful in marine construction (*see 13.3*) and other mass concrete structures. It can generally be used whenever ordinary Portland cement is usable under normal conditions. However, the addition of pozzolana does not contribute to strength at early ages; strengths similar to those of ordinary Portland cement can be expected only at later ages. The specified 7 days' compressive strength requirement of ordinary Portland cement and Portland pozzolana cement is the same. The compressive strength of Portland pozzolana cement at 28 days also has been specified. The use of Portland pozzolana cement in general building construction under normal conditions would reduce the pressure on the demand for ordinary Portland cement.

HIGH STRENGTH ORDINARY PORTLAND CEMENT — For certain specialized works, such as prestressed concrete and some items of precast concrete requiring consistently high strength concrete, high strength

*Specification for rapid hardening Portland cement (*first revision*).

ordinary Portland cement may be used. It has compressive strength much higher than the minimum compressive strength limits specified for ordinary Portland cement. This type of cement is distinctly different from rapid hardening Portland cement.

It is similar to ordinary Portland cement in many respects, except that the chemical composition is slightly altered (to increase tricalcium silicate) and is ground to an increased fineness, both of which give rise to higher strengths. It can be used wherever ordinary Portland cement may be used, but maximum advantage is likely to be gained when there is a specific requirement for high strength (for example, in the manufacture of prestressed concrete sleepers).

HYDROPHOBIC CEMENT — Hydrophobic cement deteriorates very little during prolonged storage under unfavourable conditions. This cement is obtained by intergrinding ordinary Portland cement clinker with certain hydrophobic agents (such as oleic acid, stearic acid, naphthenic acid, pentachlorphenol, etc) which will impart a water repelling property to the cement. (Hydrophobic cement should not be confused with water proofing cements.)

With the use of hydrophobic cements, a longer period for mixing may be necessary (*see* Note 1, *Clause 9.3*).

*4.1.1* High alumina cement is usually used when durability against extreme sulphate or acid attack is desirable, and when very early development of strength is required. However, this type of cement undergoes conversion or a sudden change of volume under humid and hot environments (>27°C), thereby leading to loss of strength and disintegration. Therefore, in tropical countries, such as India high alumina cement should be used with extreme care and caution, both in workmanship as well as in structural design. High alumina cement should always be used in accordance with the manufacturer's recommendations. It should not be mixed with either other hydraulic cements, lime, calcium chloride or with sea water. Further information on high alumina cement can be obtained from Ref 1.

Supersulphated cement is used where concrete is likely to be attacked by sulphate. It is a product of granulated blast furnace slag, calcium sulphate and a small quantity of Portland cement or Portland cement clinker or any other suitable source of lime. This cement has chemical resistance to most of the aggressive conditions generally encountered in construction and to the attack of sulphates in particular. It is used generally for marine works, mass concrete jobs to resist the attack by aggressive waters, reinforced concrete pipes in ground waters, concrete construction in sulphate-bearing soils, in chemical works involving exposure to high concentration of sulphates of weak solutions of mineral acids, underside of bridges, over railways (steam driven locos) and for concrete sewers carrying industrial effluents. Its use under tropical conditions is recommended by IS : 6909-1973*, provided the prevailing temperature is below 40°C. It should not be used for steam cured concrete products (*see also* comments on *4.1*).

Guidance regarding the use of high alumina cement and supersulphated cement may be obtained from Ref 1 and 2.

*4.2 Aggregates* — The Code gives information and requirements with respect to concretes made with coarse and fine aggregates obtained from natural sources and hence the aggregates should conform to IS : 383-1970†.

IS : 2386 (Parts I to VIII)-1963‡ gives the methods of tests for aggregates for concrete.

The aggregates should be free from deleterious materials such as iron pyrites, coal, mica, shale, clay, alkali, soft fragments, sea shells and organic impurities. IS : 383-1970† gives the limits of deleterious materials, such as coal, clay lumps, soft fragments, shale and material passing 75-micron IS sieve. It draws attention also to the necessity of avoiding use of aggregates containing reactive silica, such as chert and chalcedony. Also soft limestone, soft sandstone or other porous or weak aggregates should not be used for concrete in sea water (*see 13.3.1*).

Fine aggregates should be free from dust, slit and organic impurities. Inadequate

*Specification for supersulphated cement.

†Specification for coarse and fine aggregates from natural sources for concrete (*second revision*).

‡Methods of test for aggregates for concrete, Parts I to VIII.

washing in some cases leaves clay films over the surface of the aggregates. This should be guarded against, as it prevents adhesion of cement to the aggregate and results in a weak concrete. Clay and silt will increase water content because they are fine materials passing through 75-micron IS sieve. Similarly, the dust produced during the crushing of the aggregates, if left adhering to them in sufficient quantities, may be detrimental to concrete.

GRADING — Fine aggregate is defined as aggregate mainly passing 4.75 mm IS sieve and coarse aggregate as that mainly retained on this sieve. The allowance implied by the word 'mainly' is specified for each size and class of aggregate in IS : 383-1970*

IS : 383-1970* divides the grading of fine aggregates into four zones. (Most of the natural sands found in this country have gradings corresponding to one or the other of these zones.) Typical good sands fall in Zone II grading. However, finer or coarser sand may be used with suitable adjustments in the ratio of quantities of coarse to fine aggregates. Very fine sands as included in Zone IV grading should not be used except when the concrete is closely controlled by the use of designed mixes. With nominal mix concrete, it is advisable not to use a Zone IV sand under any circumstance and to avoid the use of Zone I sand, if a lean concrete mix is desired.

For fine aggregates, tolerance amounting to a total of 5 percent are allowed, except on the extreme limits of Zones I and IV and on materials passing a 600-micron IS sieve. Coarse aggregates are specified in terms of 'nominal' maximum size because a fraction of the quantity, say up to 15 percent is allowed to be retained on the sieve designation corresponding to the nominal maximum size. In other words, some materials coarser than 20 mm, but passing through 40 mm sieve are likely to be retained on a 20 mm sieve, when an aggregate of 20 mm maximum nominal size is procured and tested.

*4.2.2* In considering light weight aggregate concrete, the properties of any particular type of aggregate can be established far more accurately than for most naturally occurring materials and the engineer may, therefore, obtain specific data directly from the aggregate producer.

Formula for tensile strength (*5.2.2*) and modulus of elasticity of concrete (*5.2.3*) and the data on creep (*5.2.5*), shrinkage (*5.2.4*) and thermal expansion (*5.2.6*) are likely to be different when lightweight aggregates are used. Among the provisions of the Code, the following may require further consideration where lightweight aggregates are used:

a) Development length (*25.2.1*),
b) Durability, especially cover requirements given in *25.4*;
c) Shear and torsion resistance of beams, especially Table 13, Table 14, Table 17 and Table 18;
d) Deflection of beams: for example, the span/depth ratios given in *22.2.1*; and
e) Additional moments in slender columns, given in *38.7.1*.

It is also to be noted that concrete grades above M 40 are not likely to be achieved in practice with any lightweight aggregates.

Further information on these aspects can be obtained from Ref 3.

*4.2.3* IS : 3812-1981* Covers the extraction and physical and chemical requirements of fly ash for use as a pozzolana for part replacement of cement, for use with lime, for use as an admixture and for the manufacture of Portland pozzolana cement. Part replacement of fine aggregate in mortar and concrete is with a view to improving grading. The recommended magnitude of replacement (of fine aggregate) is up to 20 percent. Fly ash may also be used in making up the deficiency of 'fines' (particularly materials passing the 75-micron IS Sieve) in concrete mixes.

*4.2.4* SIZE OF AGGREGATE — The size limitations on aggregates are given in this clause for plain concrete as well as for lightly reinforced sections, and in *4.2.4.1* for heavily reinforced sections. These limitations are mainly intended to ensure that the bars are encased properly and honeycomb pockets are avoided. The largest possible size, properly graded, should be used in order to reduce the water demand. For high

*Specification for coarse and fine aggregates from natural sources for concrete (*second revision*).

*Specification for fly ash for use as pozzolana and admixture (*first revision*) compressive strengths, it is usually economical to adopt a lower maximum size.

While using plums in plain concrete, it is to be ensured that such plums have no adhering films or coatings, and the crushing value of plums is at least that specified for coarse aggregate.

*4.2.4.1* This clause should be read in conjunction with *25.3* where the spacing between bars is dealt. It will be convenient to decide upon the nominal maximum size of aggregate first and then to decide on the clear spacing between bars or groups of bars.

The minimum cover requirements are given in *25.4* and in many circumstances this requirement may be governed by durability or fire resistance requirements.

*4.2.5* Batching of coarse and fine aggregates separately is essential for design mix concrete (*see 8.2*). Except where it can be ensured that supply of properly graded aggregate of uniform quality can be maintained over the period of work, the grading of aggregate should be controlled by obtaining the coarse aggregates in different sizes and blending them in the right proportions when required, the different sizes being stacked in separate stockpiles.

*4.3 Water*—A detailed account of requirement of mixing water for cement is given in Ref 4.

Marsh waters, mine and colliery waste waters, several industrial waste waters and sea water are not likely to meet the requirements of the Code and, therefore, should be used only after careful consideration. Mineral oil (not mixed with animal or vegetable oils) in concentrations greater than 2 percent by weight of cement may reduce the concrete strength by more than 20 percent. The limits of acids and alkalis given in sub-clauses (a) and (b) of *4.3* are probably on the conservative side. Among the salts which cause durability problems, sulphates and chlorides are more commonly encountered and the limits for the same are given in Table 1.

The strength test requirement of *4.3.1.2* is not sufficient to establish the suitability of water with respect to the limits on sulphates and chlorides. *See also* A-2 (Appendix A) for the limits on total chlorides ($Cl^-$) and total soluble sulphates ($SO_3$) in concrete.

Water containing sugar may not have an adverse effect on concrete strength if the sugar content (in mixing water) is less than 500 ppm. Organic materials, such as algae present in mixing water cause excessive reduction in strength. Sea water is not generally recommended for mixing (*see 4.3.3*). Concretes made with sea water may show efflorescence and in reinforced concrete the sea water used for mixing may corrode the reinforcement. Most sea waters will not meet the limit on chlorides prescribed in Table 1. Municipal waters will generally meet all the requirements of mixing water. Also, waters containing less than 2 000 ppm of total dissolved solids can generally be used satisfactorily for making concrete.

*4.3.1* Should the suitability of water be in doubt, particularly in remote areas or where water is derived from sources not normally utilized for domestic purposes, such water must be tested before use.

The compressive strength alone is not an indicator of the suitability of mixing water. Water passing the strength requirements of *4.3.1.2* may yet contain chlorides and sulphates in excess of the limits given in Table 1.

*4.3.1.2* This requirement is probably intended to preclude the use of waters containing substances which may cause a significant reduction in strength, but not listed as commonly occurring. Examples are salts of zinc, copper and lead, and in particular sodium sulphide.

*4.3.1.3* Setting time is likely to be affected by the presence of sugar, carbonates and bicarbonates of sodium and potassium, salts of sodium, zinc, copper and lead. Waters containing sodium hydroxide may induce quick set.

*4.3.2* This clause puts a limit on the acidity of concrete mixing water.

*4.3.3* SEA WATER (*see also* Ref 4)—Sea water containing up to 35 000 ppm of salt (including sodium chloride and other salts) is generally suitable as mixing water for unreinforced concrete work.

The risk of corrosion due to the use of sea water is not imminent in reinforced concrete permanently coming under sea water, because access to air and alternate wetting and drying are precluded.

*4.3.4* Sea water used for curing is likely to cause efflorescence. Tannic acid is discharged from leather processing industries; iron compounds are likely to be found in excess quantities in acidic mine waters.

*4.4 Admixtures*—Admixtures are added to the concrete mix before or during mixing, in order to modify one or more of the properties of fresh or hardened concrete. The example of admixtures given here are with reference to ordinary Portland cement. It is preferable to evaluate the performance of an admixture with respect to the particular cement, aggregate and conditions expected on the job. The use of admixtures is encouraged where a sound technical reason justifies it, but with the approval of the engineer-in-charge. (For further information *see* Ref 5 and 6.)

IS : 9103-1979* gives only performance requirements for different types of admixtures.

Four main types of admixtures can be distinguished as:

a) Accelerating Admixtures—These are added to increase the rate of early strength development, which in turn facilitates earlier removal of formwork, or reduce the required period of curing or concreting in cold weather or emergency repairs. Common accelerates are calcium chloride, fluosilicates and triethanolamine. If the admixture contains calcium chloride, Clause A-2 (Appendix A) will require consideration, especially when the concrete contains embedded metal, including reinforcements.

b) Retarding Admixtures—Retarders slow down the rate of setting of cement. They are useful in hot weather concreting, for avoiding cold joints in mass concrete works, and for special treatment of concrete surfaces. Common types of retarders are starches and cellulose products, sugars and hydroxyl-carboxylic acids and their salts (for example, sodium tartrate). Many of the retarders excepting sugars and carbohydrates have water reducing properties as well.

c) Water-Reducing or Plasticizing Admixtures—The addition of a plasticiser allows greater workability to be achieved for a given water-cement ratio or alternatively retains the workability or consistency while reducing the water content. Water reducing admixtures frequently function as set-retarders when used in sufficient quantity. If a normal setting time or an accelerated set is required in addition to water reduction, the composition of the admixture is modified by introducing suitable accelerators.

   The basic ingredients of water-reducing admixture are either lignosulphonate salts or polyhydroxy compounds. In the former case, if sugars are present, retardation may occur; alternatively a refined, sugar-reduced product may be used which avoids excessive retardation. Polyhydroxy compounds may be sensitive to over-dosing, in which case excessive retardation may result. Dosages of water-reducing admixtures are usually recommended by manufacturers, but in the case of large jobs it is desirable to conduct laboratory tests for optimum dosages, which may vary with the characteristics of cements and the conditions under which concrete is placed. Another variety, known as 'super plasticizer' may be suitable where an extremely high workability is desired along with no increase in water content, say where concrete is to be pumped. Super plasticizers are based on synthesized condensates, such as melamine-formaldehyde and napthalene sulphonate-formaldehyde. In order to be totally effective, it should be ensured that super plasticizers capable of imparting high fluidity to concrete should not cause undesirable side effects, such as bleeding and segregation.

d) Air-Entraining Admixtures — These are used to intentionally entrain a controlled quantity of air into the

*Specification for admixtures for concrete.

concrete, in the form of discrete, minute bubbles, without significantly altering the setting or hardening of concrete. Their use improves the freeze-thaw durability (that is, resistance to frost damage), workability, and water tightness. Air-entraining agents can be classified as 'surfactants'. In concrete technology, surfactants are used as water-reducing admixtures also. A rough distinction between these two types of surfactants is that air-entraining agents produce stable forms of bubbles whereas water-reducing admixtures need not necessarily produce stable form of bubbles. Commonly used air-entraining agents are: (a) animal and vegetable oils and fats; (b) natural wood resins and their sodium salts, for example, vinsol resin; and (c) alkali salts of sulphated and sulphonated organic compounds. The amount of air entrainment depends on the dosage as well as other factors. More admixture will be required for richer concretes, fine cements and lower temperatures. A forced action pan mixer should preferably be used for mixing. Air entraining agents may be used in conjunction with other admixture, but their compatibility should be ascertained before hand. Excessive entrainment of air may result in reduced strength of concrete.

*4.5 Pozzolanas*—The Code permits the use of pozzolana in four ways:

a) Pozzolana as an ingredient of Portland pozzolana cement [*see 4.1* (d)]. Here the reactivity of pozzolana is made use of to obtain cementing properties.

b) Pozzolana as a part replacement of ordinary Portland cement (*see 4.5.1*).

c) Pozzolana as a fine aggregate (*see 4.2.3*).

d) Pozzolana as an admixture (*see 4.5.2*).

Here attention is confined to (b) and (d) above.

When pozzolana is used as a part replacement of cement, the reactivity of pozzolana is the deciding criterion. When it is used as an admixture, the physical property, such as its fineness rather than the chemical reactivity is important.

*4.5.1* Use of pozzolana as a part replacement of ordinary Portland cement differs from its use as an ingredient of Portland pozzolana cement. In the former case, the pozzolana of the specified quality is mixed along with the other ingredients of the concrete mix whereas in the latter case it is blended intimately with ordinary Portland cement during manufacture. As the Portland pozzolana cement already contains pozzolana, further addition is restricted to only ordinary Portland cement in this Clause.

The Code assumes that the facilities for assessing the quality of pozzolana will be available to the user. Neither this Code nor IS : 3812-1981* specifies the percentage of replacement of cement that will be permissible. Generally, replacements up to 20 or 30 percent will be possible. The major advantage of part replacement by pozzolana lies in the slower rate of heat liberation, and therefore it is most advantageous in massive foundations and dams. The replacement by pozzolana may lower the strength of concrete at early ages and this fact should be taken into account.

Fly ash which is a finely divided material can be used also as a 'mineral admixture' in mixtures deficient in 'fines' to improve the workability, reduce the rate and amount of bleeding and to increase strength (only of a mix deficient in fines). Mineral admixtures like fly ash should not be added to mixtures which are not deficient in 'fines' as this may increase the water demand and thereby lead to a reduction in strength.

The high rate and amount of bleeding of mixes deficient in fines can be brought down by increasing the amount of cement (lowering the water-cement ratio of the paste) or by adding a mineral admixture, such as fly ash. Here the function of the mineral admixture is to increase the paste content of the concrete mix and not necessarily to fill the voids.

Burnt clay pozzolana (covered by IS : 1344-1982†) should be manufactured

*Specification for fly ash for use as pozzolana and admixture (*first revision*).

†Specification for calcined clay pozzolana (*second revision*).

under controlled conditions by calcination of clay at suitable temperature and grinding the resulting product to the required fineness. The chemical requirements for the raw material (clay) are also given in IS : 1344-1981*. The optimum temperature for burning (or calcining the clay) corresponds to that at which the crystal structure of the clay mineral is destroyed and the clay is converted into a fine, active form. Depending on the type of clay, this temperature may vary from 600°C to 1 000°C. The burnt clay should be pulverized to a fineness (specific surface) of not less than 3 200 $cm^2/g$. (A lower value of 2 250 $cm^2/g$ is also acceptable provided the requirements of strength and other properties are met satisfactorily.)

*4.5.2* —

*4.6 Reinforcement*— Plain mild steel bars [IS : 432 (Part I)-1982†] and high strength cold-worked deformed bars (IS : 1786-1979‡) are widely used. The latter should have lugs, ribs or deformations on the surface, so that the bond strength is at least 40 percent more than that of plain bar of the same size.

Cold twisted bars with characteristic strength (that is, 0.2 percent proof stress) of 500 $N/mm^2$ is now included in IS : 1786-1979‡. Therefore, ready-to-use information is given in several clauses, for steels with characteristic strength of 250 $N/mm^2$ (mild steel), 415 $N/mm^2$ and 500 $N/mm^2$ (cold-worked high strength deformed steel) in the Code (*see* Note in *37.1*).

For certain special structures like bins the relevant Indian Standards indicate a preference for deformed bars, either the hot rolled variety conforming to IS : 1139-1966* or the cold-worked bars conforming to IS : 1786-1979†. IS : 4995 (Part II)-1974‡ requires the use of deformed bars in bins and silos for avoiding large cracks and ensuring more uniform distribution of cracks. It also suggests a preference for deformed bars, as these are more convenient for fixing of horizontal bars and operation of sliding formwork in the construction of bins.

Hard-drawn steel wire fabrics are occasionally used for floor slabs (hollow block, ribbed), for secondary reinforcement in developing fire resistance and in some precast concrete products like pipes. The mesh size, weight and size of wires for square and oblong welded wire fabric should be agreed to between the purchaser and the manufacturer. Some details on these are given in IS : 1566-1982§.

The requirements of important mechanical properties for the types of reinforcements covered herein are given in Table E-1.

*Specification for calcined clay pozzolana (*second revision*).

†Specification for mild steel and medium tensile steel bars and hard-drawn steel wire for concrete reinforcement: Part I Mild steel and medium tensile steel bars (*third revision*).

‡Specification for cold-worked steel high strength deformed bars for concrete reinforcement (*second revision*).

*Specification for hot rolled mild steel, medium tensile steel and high yield strength steel deformed bars for concrete reinforcements (*revised*).

†Specification for cold-worked steel high strength deformed bars for concrete reinforcement (*second revision*).

‡Criteria for the design of reinforcement concrete bins for the storage of granular and powdery materials: Part II Design criteria.

§Specification for hard-drawn steel wire fabric for concrete reinforcement (*second revision*).

## TABLE E-1 REQUIREMENTS FOR REINFORCING BARS

| IS No. | Type of Reinforcement | Nominal Size of Bars (mm) | Characteristic Strength ($N/mm^2$) (Yield Stress or 2 Percent Proof Stress) | Ultimate Tensile Stress ($N/mm^2$) | Composition of Steel Conforming to IS No. | *Min* Elongation on Gauge Length of $5.65\sqrt{\text{Area}}$ (Percent) |
|---|---|---|---|---|---|---|
| (1) | (2) | (3) | (4) | (5) | (6) | (7) |
| 432 (Part I)-1982* | Mild steel (Grade I) | 5,6,8,10,12,16,20 | 255 | | | |
| | | 22,25,28,32, 36,40, 45,50 | 236 | 412 | IS : 226-1975† | 20-23 |
| | Mild steel (Grade II) | 5,6,8,10,12,16,20 | 231 | | | |
| | | 22,25,28,32, 36,40, 45,50 | 211 | 373 | Fe 410.0 (St 42.0) of IS : 1977-1975‡ | 20-23 |
| | Medium tensile steel | 5,6,8,10,12,16,20 | 353 | 538 | St 55-HTW of IS : 961-1975§ | 17-20 |
| | | 22,25,28,32,36,40 | 348 | | | |
| | | 45,50 | 323 | | | |
| 1139-1966‖ | Hot-rolled deformed bars | — | 412 | 15 percent higher than measured yield stress | — | 14.5 |
| 1786-1979¶ | Cold twisted bars | 6,8,10,12,16, 18,20,22,25, 28,32,36,40, 45,50 | 415 (for Fe 415) | 15 percent more than the actual 0.2 percent proof stress | C-0.25 percent S-0.055 percent P-0.055 percent | (*Max*) 14.5 |
| | | | 500 (for Fe 500) | 10 percent more than the actual 0.2 percent proof stress | C-0.30 percent S-0.05 percent P-0.05 percent | 12 |
| 1566-1982** | Hard-drawn steel wire fabric | (*See* Note) | As per IS : 432 (Part II)-1982†† | — | — | — |

NOTE — The mesh sizes and sizes of wire for square as well as oblong, welded wire fabric commonly manufactured in the country are given in Appendix A of IS : 1566-1982**.

*Specification for mild steel and medium tensile steel bars and hard-drawn steel wire for concrete reinforcement: Part I Mild steel and medium tensile steel bars (*third revision*).

†Specification for structural steel (standard quality) (*fifth revision*).

‡Specification for structural steel (ordinary quality) (*second revision*).

§Specification for structural steel (high tensile) (*second revision*).

‖Specification for hot-rolled mild steel, medium tensile steel and high yield strength steel deformed bars for concrete reinforcements (*revised*).

¶Specification for cold-worked steel high strength deformed bars for concrete reinforcement (*second revision*).

**Specification for hard-drawn steel wire fabric for concrete reinforcement (*first revision*).

††Specification for mild steel and medium tensile steel bars and hard-drawn steel wire for concrete reinforcement: Part II Hard-drawn steel wire (*third revision*).

Rolled steel sections made from steel conforming to IS : 226-1975* are mainly intended to cover composite construction (*see 45.4*).

Welding of cold worked bars is permitted by the Code (*see 11.4*). Extra care and precautions will be necessary because uncontrolled application of heat to cold worked bars converts them to ordinary mild steel. In those cases where adequate precautions are not possible, only the strength of mild steel should be taken for the purpose of calculation, in the vicinity of the welded connection.

*4.6.1* The surface of the reinforcement should be free from any material which is likely to impair the bond or durability of steel. Loose mill scale and loose rust are removed in the normal course of handling before fixing the reinforcements. Rust firmly adhering to the steel is not harmful. A coat of cement wash may be applied over the reinforcements, if they are to remain in the formwork for more than a few days, say a week, in order to prevent rusting. Also, mould oils or other formwork releasing agents should be applied to the moulds in such a way that they are not smeared over the steel bars.

Research has shown that a normal amount of rust on deformed bars and wire fabric increases bond with concrete. The dividing line between a 'normal' and an 'excessive' amount of rust could be clearly defined in relation to the minimum tolerances on the size of deformed bar or wire fabric as set out in applicable reinforcement specifications. It has been concluded that it is not necessary to clean or wipe the (rusted) bar surface before using it in concrete construction. In the same way, the rust on wire fabric improves the bond quite markedly because of its roughening effect on the very smooth surface of hard-drawn wire. Provided the area of the wire was within the minimum tolerance, research of fabric showed that the more pitted the wire, the greater was the bond development. Qualitative research on plain bars is not available but similar results to wire in fabric could be assumed. It has been found that normal rough handling during fabrication, transportation, and fixing removes the mill-scale and rust which is loose enough to injure bond.

*4.6.2* The value of modulus of elasticity of steel, ($E_s$) specified is the minimum encountered but slightly higher values (say up to 205 kN/m$^2$) are also possible. This variation has negligible effects on design calculation. Note that the specifications for reinforcing steels do not specify the values for modulus of elasticity. Therefore, this clause gives only the design information. The value of $E_s$ is for all practical purposes independent of the types of the steels envisaged in the Code.

*4.7 Storage of Materials*—IS : 4082-1977* provides general guidance for stacking and storage of materials at site and covers cement, aggregate, steel and other construction materials.

Cement should be stored in dry and waterproof sheds and on a platform raised about 20 cm above ground level, and about 30 cm clear off the walls. Cement bags should be stacked in such a manner as to facilitate their removal and use in the order in which they are received. Storage of cement in silos is not mentioned in IS : 4082-1977* but if silos are used, they should be of waterproof and crackproof construction.

Hydrophobic cement conforming to IS : 8043-1978† is permitted by the Code (*see 4.1*) and this may be considered to be used where cement is to be stored for prolonged periods.

Fine aggregates and coarse aggregates should be stacked separately on hard surface or platforms, the stockpiles sufficiently removed from each other, to prevent the material at the edges of the piles from getting intermixed.

Reinforcement bars should be stacked, so that the stack is raised at least 15 cm above ground level. Bars of different classification, size and lengths should be stored separately. In coastal areas or in case of long storage, coat of cement wash is recommended.

*Specification for structural steel (standard quality) (*fifth revision*).

*Recommendations on stacking and storing of construction materials at site (*first revision*).

†Specification for hydrophobic Portland cement (*first revision*).

## 5. CONCRETE

*5.1 Grades*—The grade of concrete is an identifying number, which is numerically equal to the characteristic strength at 28 days expressed in N/mm². The engineer must decide the grade of concrete required for each part of the work and determine the suitable limitations on the constituent materials and mix proportions in accordance with the recommendations of the Code. Generally, grades M 15 and M 20 are used for flexural members.

The use of cube strength at 28 days for specifying the grade designation has arisen out of convenience as major part of the long-term strength of concrete made with normally used cements which is attained at this age. However, if high alumina cement is used, the strength development at earlier ages may also have to be specified.

Nominal mix concrete is permitted for grades M 5, M 7.5, M 10, M 15, and M 20 (*see 8.3*). Design mix concrete (*see 8.2*) is preferred by the Code for all grades of concrete, except for M 5 and M 7.5 as mentioned in Note 2 to Table 2.

Concrete coming in direct contact with sea water or exposed directly along the sea coast should be at least of grade M 15 in the case of plain concrete and M 20 in the case of reinforced concrete (*see 13.3*).

*5.1.1* —

### *5.2 Properties of Concrete*

*5.2.1* INCREASE IN STRENGTH WITH AGE—Sometimes, when the test results for strength of concrete at 28 days are lower than the specified strength, the construction is approved on the basis that the concrete will attain sufficient strength due to continuing hydration of cement by the time the full loads are applied. This approval will not be possible if the increase in strength with age is already allowed for at the design stage.

The age factor will vary with the type of cement. The values specified in Table 5 correspond to concrete made with ordinary Portland cement. The age factors apply to design mix concrete only. Increase in permissible stresses is also applicable to bending, shear, direct compression, etc, as per Note 3 of this clause.

No increase in strength with age is permitted for high alumina cement concrete; the reason being high alumina cement concrete tend to reach their potential strength much more quickly than other cements.

An approximate formula for expressing the strength of concrete at any age $t$ (in days) is, $f_t = \dfrac{t}{a+bt} f_{28}$, where $f_{28}$ is the strength at 28 days; $a$ and $b$ are empirical constants. The age factor given in the Code corresponds to approximately $a=4.7$ and $b=0.833$ in the above formula.

For example, the strength at 7 days for M 15 grade concrete is, $f_7 = \dfrac{7}{4.7+0.833\times 7} \times 15$

$$= 0.664 \times 15 = 9.96 \text{ N/mm}^2$$
$$= 10.0 \text{ N/mm}^2 \text{ (say).}$$

This corresponds to the value specified in Table 5.

*5.2.2* TENSILE STRENGTH OF CONCRETE—Two types of tensile strength are to be distinguished. The flexural tensile strength of concrete corresponds to the modulus of rupture. The split tensile strength is obtained by splitting concrete cylinders by the application of compressive line load along two lines diametrically opposite to each other.

The value corresponding to the former is used in estimating the moment at first crack, which in turn will be required for computation of deflection in flexural members. In Appendix B, this value has to be used straightway without applying any reduction factor. (The Code formula is not applicable to light weight concretes.) The reason for not using a strength reduction factor may lie in the fact that deflection is a cumulative effect rather than a local one.

Split tensile strength is used for estimating the shear strength of beams with unreinforced webs, and a few other similar problems. The Code does not give the value for this, but introduces it indirectly in the appropriate clauses.

*5.2.3* ELASTIC DEFORMATION—Modulus of elasticity of concrete is required for detailed computation of deflection of reinforced concrete flexural members (*see* Appendix B). The elastic property of the aggregate has a greater influence on the modulus of elasticity because the coarse aggregate occupies about 70 percent of the volume of concrete. Thus, the modulus of elasticity is related to the density of concrete as well, but this factor has been ignored by the Code as it deals with only normal weight concretes.

*5.2.3.1* The formula gives the short-term modulus of elasticity of structural concrete for normal weight concretes. It is applicable for computation of deflection (*see* Appendix B). However, this is not applicable for arriving at the modular ratio for other purposes and instead, *43.3*(d) of the Code should be used directly.

Strength reduction factors or safety factors should not be applied to modulus of elasticity.

*5.2.4* SHRINKAGE

*5.2.4.1* The value of shrinkage given here is for concrete made with ordinary Portland cement. In the Code, shrinkage is considered in two places:

a) Clause *25.5.2.1* where minimum reinforcement in slabs is specified to take care of shrinkage and temperature stresses. However, specific values of shrinkage are not required here, as the requirement is indirectly in terms of minimum steel area.

b) Clause *B-3* which deals with the deflection of flexural members due to shrinkage. For detailed computation of the total deflection, the magnitude of shrinkage likely to occur over a long period should be known in advance.

A typical shrinkage curve for concrete is shown in Fig. E-1 for information.

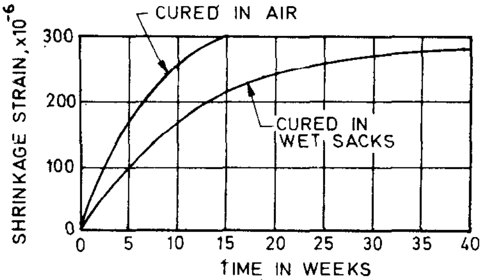


*Fig. E-1 Typical Shrinkage Curve for Concrete*

For the calculation of deformation of concrete at some stage before the full shrinkage is reached, it may be assumed that half of the shrinkage takes place during the first month and about three-quarters of shrinkage takes place in the first six months after commencement of drying.

Sufficient time-lag is to be provided for plastering concrete members to avoid surface cracks due to shrinkage of concrete.

*5.2.5* CREEP OF CONCRETE—Creep is the increase in deformation of concrete under continuous action loading (*see* Note under *5.2.5.1*).

While applying Clause *B-4* (Appendix B) which deals with deflection due to creep, the creep coefficient data given in *5.2.5.1* of the Code will be found necessary. In other places, the effect of creep is allowed for through appropriate modifications without direct reference to creep.

*5.2.5.1* Detailed recommendations for determination of creep coefficients are given in Ref 7 and these may be used whenever appropriate. However, many factors that influence creep, such as environmental condition, actual mix proportions of concrete and the proportion of permanent loads that may come on the structure, are not known with sufficient accuracy at the design stage. Therefore, only the age at loading which is a dominant factor that will be known in advance, is treated as a variable in the Code and the values of creep coefficients have been derived by assuming average condition. The variation of these coefficients with respect to age at loading is in accordance with that recommended in Ref 7, for ordinary Portland cement. The coefficients given in the Code are not applicable to light weight concretes.

Approximate values of creep coefficients for an age at loading different from that given in the Code can be obtained by an interpolation, assuming that the creep coefficient decreases linearly with the logarithm of time in days.

*Example:* Determine the creep coefficient, when the age at loading is 14 days.

| *Age of Loading* $t$ | $log_{10} t$ | *Creep Coefficient* $\theta$ |
|---|---|---|
| 7 | 0.845 1 | 2.2 (from the Code) |
| 28 | 1.447 2 | 1.6 (from the Code) |
| 14 | 1.146 1 | ? |

When $t = 14$,

Creep coefficient

$$= 1.6 + 0.6 \times \frac{(\log 28 - \log 14)}{(\log 28 - \log 7)}$$

$$= 1.90$$

The data given in the form of creep coefficients facilitate a simple analysis for estimating deflections due to creep through the use of an effective modulus of elasticity.

$$E_{ce} = \frac{E_c}{1+\theta} \quad (\textit{see } \text{B-4})$$

*5.2.6* THERMAL EXPANSION — Data on thermal expansion of concrete may be required particularly in estimating the potential rise and distribution of temperature owing to hydration of cement in mass concrete structures. Occasionally, consideration of service load behaviour under a change in temperature may be warranted in special cases of reinforced concrete construction. In *26.3* which deals with expansion joints, attention is drawn to temperature effects but an explicit consideration is required only in buildings which exceed 45 m in length.

The value of coefficient of thermal expansion (per °C), is valid for 0°C to 150°C and for concretes containing aggregates from natural sources. As indicated in the Code, the value may vary between $0.6 \times 10^{-5}$ to $1.2 \times 10^{-5}$, the lowest being for calcareous aggregates and the highest for siliceous aggregates.

For the design of liquid storage structures IS : 3370 (Part I)-1965* recommends a value of $11 \times 10^{-6}/°C$ for coefficient of thermal expansion. The same value is recommended for the design of bins [IS : 4995 (Part II)-1974†] and chimneys [IS : 4998 (Part I)-1975‡].

## 6. WORKABILITY OF CONCRETE

*6.1* Workability is used to cover a variety of characteristics such as cohesiveness, mobility, compactability and finishability of concrete. Concrete design mix should be proportioned such that the required workability is achieved (*see* 8.2.1). Clause *9.3.1* requires that the workability should be controlled by direct measurement of water and it should be checked at frequent intervals.

There are different methods of measuring the workability of fresh concrete. Each of them measures only a particular aspect of it and there is really no unique test which measures workability of concrete in its totality. Although new methods are being developed frequently, IS : 1199-1959* envisages the following three methods:

a) SLUMP TEST — Out of the three methods envisaged for measuring workability, the slump test is perhaps the most widely used, primarily because of the simplicity of apparatus and the test procedure. Apart from some conclusion being drawn regarding the harshness or otherwise of the mix, slump test is essentially a measure of 'consistency' or the 'wetness' of the mix. The test is suitable only for concretes of medium to high workabilities (that is, slump 25 to 100 mm). For very stiff mixes having zero slump, the slump test does not indicate any difference in concrete of different workabilities. It has been pointed out that different concretes having same slump may have indeed different workability under the site conditions. However, when the uniformity between different batches of supposedly similar concretes under field conditions is to be measured, slump test has been found to be suitable.

b) COMPACTING FACTOR TEST — The compactability, that is, the amount of work needed to compact a given mass of concrete, is an important aspect of workability. Strictly speaking, compacting factor test measures workability in an indirect manner, that is, the amount of compaction achieved for a given amount of work. This test has been held to be more accurate than slump test, specially for concrete mixes of 'medium' and 'low' workabilities (that is, compacting factor of 0.9 to 0.8). Its use has been

*Code of practice for concrete structures for the storage of liquids: Part I General requirements.

†Criteria for design of reinforced concrete bins for storage of granular and powdery materials: Part II Design criteria (*first revision*).

‡Criteria for design of reinforced concrete chimneys: Part I Design criteria (*first revision*).

*Methods of sampling and analysis of concrete.

more popular in laboratory conditions. For concrete of very low workabilities (that is, compacting factor of 0.7 and below which cannot be fully compacted for comparison, in the manner described in the test method) this test is not suitable.

This test is sufficiently sensitive to enable differences in workability arising from the initial process of hydration of the cement to be measured. Therefore, each time test should preferably be carried out at a constant time interval after mixing is completed. A convenient time for releasing the concrete from the upper hopper of the compacting factor apparatus would be two minutes after completion of mixing.

c) Vee-bee test is preferable for stiff concrete mixes having 'low' or 'very low' workability. Compared to the other two methods, Vee-Bee test has the advantage that the concrete in the test, receives a similar treatment as it would be in actual practice. In this method, the index vee-bee time is determined as the time taken for the concrete surface to uniformly adhere to the glass disc or rider of the apparatus. This is judged visually and the difficulty of establishing the end point of the test may be a source of an error, especially if the time involved is very short, say 2 to 5 seconds.

## *7.* DURABILITY

Durability of concrete, or lack of it, can be defined and interpreted to mean its resistance (or absence of resistance) to deteriorating influences which may through inadvertence or ignorance reside inside the concrete itself, or which are inherent in the environment to which the concrete is exposed. Under normal circumstances, concrete is generally durable. Problems arise when concrete contains ingredients which were not known beforehand to be deleterious or when it is exposed to harmful environments not anticipated earlier.

The absence of durability may be caused by external agencies like weathering, attack by natural or industrial liquids and gases, bacterial growth, etc, or by internal agencies like harmful alkali aggregate reactions, volume change due to non-compatability of thermal and mechanical properties of aggregate and cement paste, presence of sulphates and chlorides from ingredients of concrete, etc. In case of reinforced concrete, the ingress of moisture or air will facilitate the corrosion of steel, leading to an increase in the volume of steel and cracking and spalling of concrete cover.

*7.1, 7.1.1* and *7.2*—The Code gives the requirements for durable structures in several places and they can be broadly classified under the following topics namely, choice of materials, dense and impermeable concrete, cover to reinforcements, and design and detailing practice.

Clause *20.1* of the Code indicates that compliance with the requirements regarding (a) cover to reinforcements, (b) detailing, (c) cement content (Appendix A), and (d) water cement ratios (Appendix A) will normally meet the durability requirements of most structures. Additional requirements for durability of concrete exposed to sea water and alkali environment are given in *13.3* and *13.4* of the Code.

a) CHOICE OF CONSTITUENT MATERIALS—Clause *4.1.1* of the Code permits the use of high alumina cement and supersulphated cement, only in special circumstances, one of which may be the durability under aggressive condition. Chlorides and sulphates in mixing water (*see* Table 1) and possibly the chlorides in admixtures may be deleterious to concrete. Aggregates containing reactive silica are deleterious to concrete (*see* comment on *4.2*). Use of Portland slag cement or Portland pozzolana cement is advantageous when the concrete is in sea water or is directly exposed along the sea coast.

b) DENSE AND IMPERMEABLE CONCRETE—The minimum cement content and the maximum permitted water-cement ratios are given in Appendix A. Compaction and curing are covered in *12.3* and *12.5* respectively. Starting with water-cement ratio, it is known that the permeability of cement paste increases exponentially with increase in water-cement ratio above 0.45 or so; from the consideration of permeability water-cement ratio is restricted to 0.45 to 0.55 except in mild environment. For a given water-cement ratio, a given cement content in the concrete mix will correspond to a given workability (high, medium or low) and an appropriate value has to be chosen keeping in view the placing condition, cover thickness and reinforcement concentration. In addition cement content is chosen by two other considerations. First, it should ensure sufficient alkalinity to provide a passive environment against corrosion of steel, for example, in concrete in marine environments or in sea water, a minimum cement content of 300 kg/m$^3$ is to be used. Secondly, the cement content and water-cement ratio so chosen should result in sufficient volume of cement paste to overfill the voids in the compacted aggregates. Clearly, this will depend upon the type and nominal maximum size of aggregate employed. For example, crushed rock or rounded river gravel 20 mm maximum size of aggregate will in general, have 27 and 22 percent of voids respectively. The Table 19 in Appendix A is based on this limitation. Therefore, the cement content has to be reduced or increased as the nominal maximum size of aggregate increases or decreases, respectively. Concrete in sea water or exposed directly along the sea coast should be of at least M 15 grade in case of plain concrete and M 20 in case of reinforced concrete.

c) COVER — Increase in cover thickness is specified when the surface of concrete members are exposed to the action of harmful chemical effects (*see 25.4.2*).

d) DESIGN AND DETAILING PRACTICE — For normal condition of exposure, a limiting crack width of 0.3 mm is recommended in *34.3.2*. The detailing requirement regarding the spacing of bars (*see 25.3.2*) is derived on this assumption. If the permissible stresses for steel given in *44.2* for the working stress method of design are adopted, the crack widths are not likely to exceed 0.3 mm. However for severe conditions of exposure, a crack width of 0.3 mm may not be acceptable (*see 34.3.2*) and, therefore, an explicit check on crack width at service loads will be necessary to ensure durability.

## *8.* CONCRETE MIX PROPORTIONING

*8.1 Mix Proportion* — The condition of handling and placing, the amount of reinforcement and the method of compaction together determine the requirements of workability. In one instance, for underwater concreting, the Code specifies the workability of concrete as between 100 mm and 180 mm of slump (*see 13.2.2*). Otherwise apart from the guidance given in *6.1*, workability requirements are left to the judgement of the engineer-in-charge. Also, *8.2.1* of the Code requires that the concrete shall be designed to give required workability and characteristic strength.

It is difficult to determine, within a reasonable time of say one month, whether the hardened concrete will have the required durability. Probably, the intention of the Code is to draw attention to the minimum cement content and the water-cement ratios (*see* Appendix A) before determining the mix proportion.

Requirements of surface finish are not given in the Code. However, if the type of surface finish is specified (Ref 3), this should be taken into account.

*8.1.1* Usually design of concrete mix and adoption of nominal mix are with respect to strength requirements. Therefore, the mix proportion arrived at should be re-appraised with respect to durability and surface finish (*see 8.1* and its comments).

The proportions of nominal mix suggested in *8.3* are on the basis of weights of cement and aggregates.

For the determination of mix proportion for design mix concrete, the target strength should be higher than the specified characteristic strength and is aimed at ensuring that the characteristic strength is attained. The concrete mix is to be proportioned to give an average strength which will be higher than the specified strength by an amount which will ensure that not more than the acceptable percentage of results will fall below the specified strength. The acceptance criteria specified in the Code (*see 15*) is based on a one-in-20 chance that the cube strength of a sample may fall below the specified characteristic strength. Further, assuming a Gaussian or normal distribution of the test results, the following relation holds good:

Target strength = Characteristic strength + 1.65 × standard deviation.

The standard deviation can be assessed from past experience in similar works, or alternatively it may be taken from Table 6, in case sufficient number of test results are not already available for a particular grade of concrete. For example, for Grade M 20 (20 N/mm$^2$, characteristic strength), concrete may be expected to have a standard deviation of 4.6 N/mm$^2$ (*see* Table 6). Thus, the target strength $= 20 + 1.65 \times 4.6 = 27.6$ N/mm$^2$.

Therefore, the mix should be designed for a strength of say 28 N/mm$^2$ and verified by trial mixes. When this designed mix is adopted in the field, the average strength will be around 28 N/mm$^2$ but a few test result (say 1 out of 20 results) are likely to fall below 20 N/mm$^2$.

By the same reasoning, it will be seen that the nominal mix proportions, suggested in *8.3* of the Code, are likely to give average strengths somewhat higher than the characteristic strengths given by the grade designation. Therefore, if the nominal mix corresponding to M 15 gives an average strength of say 25 N/mm$^2$, the grade of concrete should not be assumed as M 25 or M 20 (*see 8.3.2*). Rather, if the test results are consistently high, advantage may be taken of this fact only by changing over to the design mix concrete.

*8.1.2* INFORMATION REQUIRED

*8.1.2.1* —

*8.2 Design Mix Concrete*—(For detailed information reference may be made to SP : 23 Handbook on Concrete Mixes published by ISI).

*8.2.1* Workability must be decided by the engineer and some guidance is given in *6.1* (*see also* comment on *8.1*). To produce consistently a characteristic strength corresponding to the grade designation, the concrete should be designed for target strength which will be higher than the specified characteristic strength.

*8.2.2* In particular, attention may have to be paid to the type of cement (*see 4.1*) and the grading of the aggregate (*see 4.2*).

*8.3 Nominal Mix Concrete* — In Table 3, the quantities of cement and aggregates are given in terms of weight. In the 1964 version of the Code, the quantity of aggregates was given in terms of volume. This shift implies that batching by weight is to be preferred to batching by volume. However, *9.2.2* of the Code permits volume batching of fine and coarse aggregates (but not cement) provided that allowance for bulking of sand is made while batching the wet aggregates and periodic checks are made on mass/volume relationships.

A rough guide for the nominal mix proportions by volume will be 1:3:6, 1:2:4 and 1:1½:3 for M 10, M 15 and M 20 concretes respectively, provided that the nominal maximum size of aggregate is 20 mm. For other sizes of aggregates, adjustments in the ratios of the weight of coarse and fine aggregates will be necessary as indicated in the Note to Table 3. In all cases, fine aggregates should conform to the grading of Zone II or Zone III of IS : 383-1970*. Otherwise, further adjustments may become necessary. In view of these limitations and a number of other reasons the Code prefers a design mix concrete over a nominal mix concrete.

(*See also 13.2.2*). For underwater concreting, the quantity of coarse aggregate should not be less than 1½ times nor more than twice that of the fine aggregate.

The nominal mix concrete should be restricted to works of minor nature in which the strength of concrete is not critical. It is important that the designer and concrete manufacturer should understand clearly their respective areas of responsibility as these are different in the two methods. In design mix concrete, the concrete manufacturer is responsible for ensuring that the mix as supplied meets the performance criteria and any mix limitations specified by the designer. The designer's responsibility is to ensure that the specified criteria are adequate for the expected service conditions. In the nominal mix method, the designer assumes the responsibility for the performance of the specified mix ingredients and

*Specification for coarse and fine aggregates from natural sources for concrete (*second revision*).

proportions. The manufacturer is responsible for ensuring that the materials actually used are as specified and in nominated proportions.

*8.3.1* For durability, cement content should be checked against the requirements of Appendix A also.

*8.3.2* (*See* comment on *8.1.1*).

## 9. PRODUCTION AND CONTROL OF CONCRETE

### *9.1 General*

*9.1.1* (*See* comments on *12.6*).

*9.1.2* —

*9.2 Batching*—Volume batching is permitted both for design mix concrete as well as nominal mix concrete, in *9.2.2* of the Code subject to certain conditions.

IS : 4925-1968* gives the specification of batching and mixing plant for concrete. These plants are usually specially designed to suit the local conditions and output required. Small plants may have an output as low as 30 m³ of mixed concrete per hour, medium plants in the range of 100 to 300 m³ per hour and large plants over 300 m³ per hour. IS : 4925-1968* covers the requirements of manual, semi-automatic or fully automatic central batching and mixing plants capable of producing not less than 100 m³/h of mixed concrete.

*9.2.1* —

*9.2.2* Allowance for bulking of sand is necessary only in case of volume batching. However, allowance for surface water carried by the aggregate should be made in all cases.

IS : 2386 (Part III)-1963† gives a field method for determination of necessary adjustment of bulking of fine aggregate, as the sand may contain an amount of moisture which will cause it (sand) to occupy a larger volume than it would occupy if dry. If the sand is measured by loose volume, it will be necessary to increase the quantity, in order that the amount of sand put into the concrete may be the amount intended for the nominal mix used (based on dry sand). It will be necessary to increase the volume of sand by the percentage of bulking. The correction will be approximate only, because the measurement of volume of loose sand is a rough method at best, but a correction of the right order can easily be determined.

*Specification for concrete batching and mixing plant.

†Methods of test for aggregates for concrete: Part III Specific gravity, density, voids, absorption and bulking.

Two methods, both using the fact that the volume of the inundated sand is the same as for dry sand are suggested in IS : 2386 (Part III)-1963*.

*9.2.3* A field method for determination of surface moisture in fine aggregate is also given in IS : 2386 (Part III)-1963*. The accuracy of the method depends on the accuracy of information on the specific gravity of the material in a saturated surface-dry condition. The Standard suggests that the same procedure, with appropriate change in the volume of the sample and the dimension of the container used for the test, might be applied to coarse aggregates as well.

*9.2.4* —

*9.3 Mixing*—IS : 1791-1968† lays down requirements regarding the drum, water tanks and fittings, loaders, hoppers, power units, discharge height and road worthiness for the free fall (drum) batch type concrete mixers. The continuous mixer, forced action (pan) type mixer and truck mounted mixer are not covered by this standard. Strictly speaking there should be no bar in using the latter type of mixers, but the Code draws attention only to the more common type of mixer (that is, drum type mixer) used at site, for which an Indian Standard is available. Pan type mixers are used more often in precast concrete factories, and truck-mounted mixers for ready mixed concrete.

IS : 4634-1968‡ gives the method of testing the performance of batch type concrete mixers in terms of uniformity of constituents in the concrete mix. The mixing efficiency,

*Methods of test for aggregates for concrete: Part III Specific gravity, density, voids, absorption and bulking.

†Specification for batch type concrete mixers (*first revision*).

‡Method for testing performance of batch type concrete mixers.

that is, an index of the uniformity of the mixed concrete, can be evaluated by finding the percentage variation in quantity of cement, fine aggregate and coarse aggregate in a freshly mixed batch of concrete. According to IS : 1791-1968*, the percentage variation between the quantities of cement, fine aggregate and coarse aggregate (as found by weighing in water) in the two halves of the batch and the average of the two halves of the batch should not exceed the following limits :

| | |
|---|---|
| Cement | 8 percent |
| Fine aggregate | 6 percent |
| Coarse aggregate | 5 percent |

*9.3.1* (*See also* comments on *6.1*).

## *10.* FORMWORK

*10.1 General*—Tolerances on the dimensions of the structures are a practical necessity. Design provision accommodates some of these tolerances. For example, the material strength reduction factor $\gamma_m$ is taken as 1.5 for concrete for ultimate limit state and also minimum eccentricity of load is specified for columns. Tolerances for construction of formwork are given in this clause. These are not to be taken as tolerances on constructed structures for which the Code does not give any information. The tolerances for placing reinforcements are given separately in *11.3* (*see* Fig. E-2). The vibration effect should invariably be taken into account in the design of formwork.

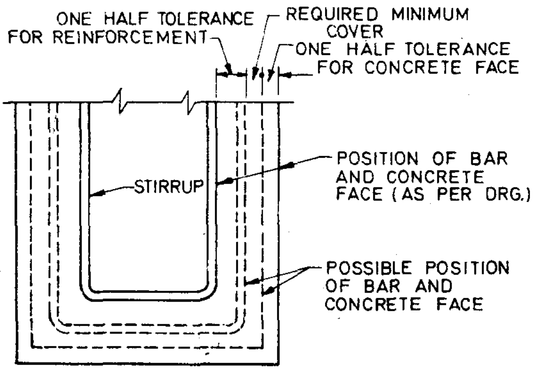


*Fig. E-2 Bending Dimensions for Stirrups*

*10.2 Cleaning and Treatment of Forms*—It is to be ensured that formwork is erected with joints tight enough to prevent leakage of cement mortar. Surfaces that are to come in contact with fresh (wet) concrete must be treated by being coated with a non-staining mineral oil or other approved material. In case of formwork absorbing water from concrete, they shall be thoroughly wetted to prevent absorption of water from wet concrete and to ensure easy release and non-adhesion to formwork during stripping.

*10.3 Stripping Time*—In some cases, such as while using cements other than ordinary Portland cement or when the curing conditions are not normal, it may be necessary to estimate the strength of concrete at the time of removal of formwork. Cubes, if they are cast to determine the strength of concrete at the time of removal of formwork, should be cured along with the structure and not under standard conditions as envisaged in *14.1*. Accordingly, these cubes should not be used for the acceptance of concrete.

For rapid hardening cement, 3/7 of the periods given for ordinary Portland cement will be normally sufficient, except that a minimum period of 24 hours is required.

It is to be ensured that formwork is removed carefully so that shock and damage to the concrete are avoided. Due regard must also be given to curing methods to be employed before the formwork is removed.

*10.3.1* —

## *11.* ASSEMBLY OF REINFORCEMENT

*11.1 Straightening, Cutting and Bending*—Procedures for bending and fixing of reinforcements are given in IS : 2502-1963*.

Reinforcements, especially cold-twisted deformed bars, should be bent cold. Bars larger than 25 mm in size, except cold-twisted ones, may be bent hot at cherry-red heat (not exceeding 850°C).

Bars of 12 mm diameter and under may be bent by using simple tools such as a claw. For bars up to 16 mm a simple hand machine (without gears) is recommended. For larger diameters, a geared bar-bending machine

*Specification for batch type concrete mixers (*first revision*).

*Code of practice for bending and fixing of bars for concrete reinforcement.

(hand operated) will be suitable. For bars 36 mm and above, or where large quantities of bars are to be bent, power-operated benders may be used advantageously. The tolerance for bending and cutting are also specified in IS : 2502-1963*

*11.2* Bars crossing each other should be secured by annealed binding wires (0.9 mm diameter or over) at intervals, in such a manner that they will not slip over each other during concreting.

*11.2.1* The bars should be free to act along with concrete in the direction in which they are provided without any lateral restraint.

*11.3* (*See also* comments on *11.1*). Permissible deviations from specified dimensions of cross-sections of beams and columns are given in *10.1*. Allowances for tolerances for bending and fixing of reinforcements should be based on both *10.1* and *11.3*.

Cover to reinforcement is liable to variation on account of errors in dimensions of formwork and the cutting, bending and fixing of the reinforcement. The dimension corresponding to the cover, shown in the drawing is the nominal cover. All reinforcement should be fixed to this nominal cover, using spacers of the same size as the nominal cover. The term nominal cover implies usually a permissible negative tolerance of 5 mm that is, the actual cover could be 5 mm less than the nominal cover.

When a reinforcing bar is to be bent between two concrete faces (for example, a rectangular stirrup or link), the dimensions on the bar bending schedule may be determined as the nominal cover on each face and due allowance should be given for other errors (member size, bending, etc) as given in Table E-2 (*see also* Fig. E-2).

*11.4 Welding*—IS : 2751-1979* gives the requirements for welding of mild steel round and deformed bars conforming to Grade I of IS : 432 (Part I)-1982† and IS : 1139-1966‡. IS : 9417-1979§ gives requirements for welding of cold-worked steel bars conforming to IS : 1786-1979‖.

Welding close to a bend portion of a large size bar has the effect of accelerating the strain-ageing which follows the bending, and this in turn reduces the ductility of the section. One major area where care is needed is when welding bent bars to base plates and similar devices for use in precast concrete. Welding of any kind may have a deleterious effect on the reinforcement when it is located in a section of the work subject to excessive fatigue or impact forces. The latter forces may occur during erection of precast units. In any type of welding, the ability of the

**TABLE E-2 DEDUCTION FOR TOLERANCES IN POSITION OF BENT STEEL**

| Overall Concrete Dimension (Measured in Direction of Tolerance) | Type of Bar | Deductions to Determine Bending Dimension |
|---|---|---|
| (1) | (2) | (3) |
| mm | | mm |
| Up to 1 000 | Stirrups and other bent bars | 10 |
| 1 000-2 000 | do | 15 |
| Over 2 000 | do | 20 |
| Any length | Straight bar | 40 |

*Code of practice for bending and fixing of bars for concrete reinforcement.

*Code of practice for welding of mild steel plain and deformed bars for reinforced concrete construction (*first revision*).

†Specification for mild steel and medium tensile steel bars and hard-drawn steel wire for concrete reinforcement: Part I Mild steel and medium tensile steel bars (*third revision*).

‡Specification for hot rolled mild steel, medium tensile steel and high yield strength steel deformed bars for concrete reinforcement (*revised*).

§Recommendations for welding cold-worked steel bars for reinforced concrete construction.

‖Specification for cold-worked steel high strength deformed bars for concrete reinforcement (*second revision*).

workman is of the utmost importance in ensuring good quality welds. Therefore, as far as possible welding should be avoided in reinforced concrete structures subjected to large numbers of repetitions of substantial loads. The fatigue strength of beams in which the links have been welded to the main bars can be reduced by as much as 50 percent.

In general, bars up to and including 20 mm in diameter should be lap welded and those larger than 20 mm diameter should be butt welded.

For the requirements concerning welded splices, *see 25.2.5.2.*

*11.5* —

## *12.* TRANSPORTING, PLACING, COMPACTING AND CURING

*12.1 Transporting*—Segregation of concrete occurs because concrete is a mixture of materials differing in particle size and specific gravity. Any lateral movement that occurs when concrete is being deposited at one point and allowed to flow within the forms, or when the concrete is projected forward by the conveying equipment, causes the coarse aggregate and mortar to separate.

Segregation may easily occur unless special attention is given to its prevention. When stiff concrete is transferred from one conveyance to another, long unconfined drops should be avoided, rather the use of hoppers; baffles and short vertical drops through a pipe to the centre of the receiving container is recommended (*see* Ref 10).

*12.1.1*

*12.2 Placing*—For special guidance on placing of concrete under water, *see 13.2.*

Concrete should be deposited at or near its final position in the placement, eliminating the tendency to segregate when it has to flow laterally into its place. On sloping surfaces, concrete should be placed at the lower end of the slope first, progressing upward, and thereby increasing natural compaction of the concrete. High velocity discharge of concrete, which may cause segregation of the concrete, should be avoided. Recommended methods of placing concrete in segregation-prone location are:

a) IN NARROW FORMS—Concrete should be discharged first into a light hopper fitted with a light, flexible drop chute, so that concrete is placed at the surface without striking at the formwork or reinforcements. This avoids segregation and leaves the formwork and reinforcements clean until concrete covers them.

If very wet concrete is to be placed in narrow, deep formwork, water content in the upper layers should be gradually reduced to compensate for water gain.

b) PLACING THROUGH SIDE PORTS IN COLUMN FORMWORK—Concrete should be dropped vertically into the outside pockets under each formwork opening (port) so that concrete stops and then flows easily into the column formwork.

c) PLACING ON SLOPING SURFACES—Concrete should not be discharged from free end of a chute on to a sloping surface, as the heavier coarse aggregates are separated and carried down the slope. The chute should be fitted with a baffle and a drop at its end, so that concrete remains on slope.

d) USE OF WHEEL BARROWS—Concrete from wheel barrows should not be dumped away from the face of concrete already in place. It should be dumped into the face of concrete already in place (*see also* Ref 10).

*12.3 Compaction*

*12.3.1* Compaction should preferably be achieved by mechanical vibration but it can also be achieved (if approved) by manual methods, namely, rodding, spading and tamping. The Code, while not prohibiting the manual methods for isolated instances, recommends mechanical methods of vibration, the latter being the better practice. Also, the Code refers only to vibration and leaves out other methods of compaction like spinning, mechanical tamping, and use of shock, because the latter methods are restricted to special situations.

Immersion vibrators (IS:2505-1980*) are

*General requirements for concrete vibrators, immersion type (*second revision*).

commonly used for consolidation of plain as well as reinforced concrete. They can be either (a) flexible shaft type, powered by different types of motors; or (b) motor-in-head type, electrically or pneumatically driven. The operational frequency of the vibrator should preferably be between 8 000 to 12 000 vibrations per minute. While compacting with internal vibrators, concrete should be deposited in layers of 30 to 45 cm thick and the vibrator inserted vertically at uniform spacing over the entire area of placement. The vibrator should penetrate rapidly to the bottom of the layer and at least 15 cm into the preceding layer, if there is any. It should be held (generally 5 to 15 seconds) until the compaction is considered adequate and then withdrawn slowly at the rate of about 8 cm per second.

Concrete vibrators of screed board type (IS : 2506-1964*) are suitable for compaction of concrete roads, runways, floors, pavements and thin slabs, where the area to be compacted is large or the thickness is too small (less than 200 mm) to allow the use of immersion vibrators. They exert their effects at the top surface and compact the concrete from the top down. The screed vibrator consists of a screed board or plank long enough (generally 3 m or 5 m) to span the width of the slab. One or more eccentrics, depending on the length of the screed, are attached to the top. The eccentrics are driven by suitable power units. Usually the vibrator is provided with two handles, fitted with antivibration packings or springs. An anchor, capable of being embedded into ordinary ground temporarily, is mounted on to one of the handles. The recommended minimum frequency under no-load state is 3 500 vibrations per minute, the amplitude being not less than 1.5 mm.

Formwork vibrators (IS : 4656-1968†) are generally used for compaction of concrete in precast concrete moulds, such as pipes, gullies and deep post-tensioned beams. They are also used for compaction of *in-situ* concrete in small and narrow sections or very heavily reinforced sections where immersion vibrators cannot be used. They are generally powered by electric or air motor and are of two types, namely, (a) the fixed or clamp type, and (b) the manual type. The clamp type is more widely used, and the manual type is used in situations where there are no means of fitting the clamps or where continuous movement along the formwork is desirable. The frequency of vibration under no-load (operation in air) should not be less than 2 800 vibrations per minute for both types of formwork vibrators and the acceleration of vibration under loaded state should not be less than 3 *g*.

*Specification for screed board concrete vibrators.
†Specification for form vibrators for concrete.

Vibrating Table (IS : 2514-1963*) are used for compaction of concrete in moulds for the manufacture of precast products and structural elements. They compact concrete through rapidly alternating horizontal, vertical or circular vibrations which are transmitted to moulds filled with concrete and placed or clamped on the table top. They are to be distinguished from shock tables which pulsate at low frequency and operate on the principle of gravity fall with the help of rotating cans. Usually they have a breadth of 1 m and length varying from 1 to 3 m. The frequency of vibration for the table operating at its maximum load capacity should be between 3 000 to 6 000 cycles per minute, and vibration acceleration should not be less than 4 g.

Under-vibration being harmful should be avoided. Over-vibration can occur if, due to careless operation or use of grossly oversized equipment, vibration is many times the desirable amount. This over-vibration may result in one or any combination of the following damages:

a) Settlement of the coarse aggregate — This condition is more likely to occur with wet mixes and where there is a large difference in specific gravity between the coarse aggregate and mortar.
b) Sand streaks — These are caused by heavy bleeding of the concrete along the formwork. They are most likely to occur with the use of harsh, lean mixes.
c) Excessive formwork deflection or formwork damage — These are most likely with external vibration (Ref 11).

*12.4 Construction Joints* — Construction joints are stopping places in the process of

*Specification for concrete vibrating tables.

placing of concrete and are required because it may be impractical to place concrete in a continuous operation. These should be distinguished from movement joints, including expansion joints dealt with in *26*. The location of joints is controlled by design parameters and by construction limitations, but the joints must nevertheless be kept as few as possible consistent with reasonable precautions against shrinkage.

The spacing of construction joints is determined by the type of work, site conditions and the production capacity. Also, the construction joints should occur only where they may be properly constructed as these joints require careful attention to workmanship. From the point of view of strength of structure, it is desirable to position construction joints at points of minimum shear. Joints in load bearing walls and columns should be located on the underside of floor slabs or beams. In walls, the horizontal length of placement should not exceed 8 to 12 m. If there should be any doubt regarding the adequacy of the bond between the old and new concrete, the reinforcement crossing the construction joint should be supplemented by dowels.

The surface of hardened concrete can be made rough by sand-blasting, chipping it lightly, but the chipping should not be so vigorous that the coarse aggregates are dislodged. Laitance formed on the surface of a construction joint should preferably be removed before the concrete has hardened, but care being exercised not to disturb the young concrete too much as this is likely to leave the concrete in a porous and weak condition.

Although good joints have been produced by using mortar or grout, the technique appears to present more problem than it solves; it is, therefore, best avoided in most situations and emphasis placed instead on a high compactive effort (*see* Ref 12).

The surface of the hardened concrete should be neither too dry nor too wet with puddles of water lying on it. In the former case, excessive quantities of water might be extracted from the fresh concrete and in the latter case a poor bond might result. Through a similar reasoning, it can be inferred that the neat cement slurry should also be of a proper consistency.

In horizontal joints, weakness may occur either at the top of concrete at the underside of the joint, or at the bottom of the concrete of the top layer. In the former case, trouble arises from the formation of laitance, especially when wet concretes incorporating finely ground cements are used. As a result, the top layer is weak, porous and less durable. At the upper side of the joints, troubles arise from an excess of aggregates and a deficiency of cement, both due to segregation.

Vertical joints are more prone to shrinkage, particularly if the joint is weaker than the concrete elsewhere. Vertical stop-boards should always be provided at the ends of each section of work, in order to provide a surface against which the concrete can be compacted properly. Concrete should not be allowed to flow at an angle forming a feather-edge.

Recommendations for construction joints in liquid retaining structures are given in IS : 3370 (Part I)-1965*. In the case of bins and silos, vertical construction joints are not preferred [*see* IS : 4995 (Part II)-1974†]. No construction joints should be allowed within 600 mm below low water level or within 600 mm of the upper and lower planes of wave action when a concrete member is to be constructed in sea water (*see 13.3.3*).

### *12.5 Curing*

*12.5.1* MOIST CURING — The curing period of seven days is applicable to concretes made with ordinary Portland cement and Portland slag cement. It may be reduced to three or four days when rapid hardening Portland cement is used, but greater care should be exercised, particularly at early ages, when the rate of hydration will be high. In the case of low heat Portland cement and super-sulphated cement, longer curing period of say 2 weeks may be necessary. For high alumina cement concrete, great care is required in curing and specialist literature should be consulted (Ref 1 and 2). Continuous curing is also important, to avoid formation of surface cracking due to alternate wetting and drying.

*Code of practice for concrete structures for the storage of liquids: Part I General requirements.

†Criteria for design of reinforced concrete bins for storage of granular and powdery materials: Part II Design criteria (*first revision*).

*12.5.2* MEMBRANE CURING — Membranes, such as polythene sheets or thin films formed by certain liquids (curing compounds) are applied on to the surface of concrete to prevent the loss of water from concrete by evaporation. Curing compounds consist essentially of waxes, resins, chlorinated rubber and solvents of high volatility. The formulation should be capable of providing a seal shortly after being applied and must not be used on surfaces that are to receive additional concrete, paint or tile that require a bond, unless the membrane can be satisfactorily removed, or can serve as a base that can provide a bond. Hence the Code requirement regarding the prior approval by the engineer-in-charge.

Curing compounds must be applied after the free water on the surface has disappeared and no water sheet is seen, but not so late that the compound will be absorbed into the surface pores of the concrete.

On formwork concrete surfaces, the curing compound should be applied immediately after stripping the formwork. In such applications, if the surface is dry, water should be sprayed and it should be allowed to reach a uniformly damp appearance with no free water standing when the compound is applied.

*12.6 Supervision* — The manufacture and placing of concrete and reinforcement should be supervised by a person whose qualification and experience shall be as given in the National Building Code of India.

*12.6.1* —

## 13. CONCRETING UNDER SPECIAL CONDITIONS

*13.1 Work in Extreme Weather Conditions* — Concreting operations done at atmospheric temperatures above 40°C, need special attention. IS : 7861 (Part I)-1975* gives the recommended practices that would result in concrete possessing improved characteristics in the fresh as well as hardened state. Good practices of concreting under hot weather conditions require special care with respect to the following:

a) Temperature control of concrete ingredients.
   1) AGGREGATES — Stored under shade or cooled by water.
   2) WATER — Used in the form of ice or in near freezing temperatures.
   3) CEMENT — Temperature restricted to 77°C.

b) MIX DESIGN — Use low cement content and cements with low heats of hydration. Use approved admixtures for reducing the water demand or for retarding the set.

c) PRODUCTION AND DELIVERY:
   1) Temperature of concrete at the time of placement should be below 40°C.
   2) The mixing time should be held at minimum, subject to uniform mixing.
   3) Period between mixing and delivery should be kept to a minimum.

d) PLACEMENT AND CURING:
   1) Prior to placing concrete formwork, reinforcements and subgrade should be kept cool by spraying with cold water first. If possible, concreting may be restricted to evenings or nights.
   2) Placement and finishing should be speedy.
   3) Immediately after compacting and finishing, concrete should be protected from evaporation of moisture.

*13.2 Under-water Concreting*

*13.2.1* Inspection of concrete during placement under water is difficult. Therefore, it is essential to evaluate the proposed mix proportions, inspect the equipment and review preparation prior to the start of underwater concreting.

*13.2.2* The indicated slump of 100 to 180 mm applies to placing by tremie [*see 13.2.4* (a)].

*13.2.3* Dewatering while the concrete has not hardened sufficiently may disturb the concrete and may lead to undesirable results (*see also 13.2.5*).

*Code of practice for extreme weather concreting: Part I Recommended practice for hot weather concreting.

*13.2.4*

a) TREMIE — The buoyancy of the empty pipe (of the tremie) is frequently a problem when concrete is to be placed through 20 m or more of water. When this problem occurs, it may be desirable to start concreting with water in the pipe. In this case also, the upper end of the pipe should be plugged with a snugly fitting ball made up of gunny sacking before delivering the concrete to the hopper.

b) DROP BOTTOM BUCKET

c) BAGS

d) GROUTING

The void content of the coarse aggregate should be kept as low as possible. The Code assumes a maximum void content of 55 percent.

*13.2.5* —

*13.3 Concrete in Sea-Water*— In addition to the grade of concrete specified, it will be necessary to control the minimum cement content and the maximum water-cement ratio (*see 7* and *also* Table 9).

Portland slag cement may be used but it will be necessary to seek specialists advice (*see* comments on *4.1*).

*13.3.1* —

*13.3.2* Precast members are to be preferred because then it will be possible to achieve dense concrete and eliminate those with porous or defective concrete by inspection before installation. Unreinforced elements should be used if practicable, as reinforcing steels are susceptible to corrosion caused by chlorides present in sea water.

*13.3.3* Construction joints are potentially weak and the problems of durability are accentuated in the zone subject to alternate drying and wetting that is, between upper and lower planes of wave action.

*13.3.4* (*See also* comments on *4.7*). IS : 4082-1977* recommends a coat of cement water over the reinforcing steels stored in coastal areas.

*Recommendations on stacking and storage of construction materials at site (*first revision*).

*13.4 Concrete in Aggressive Soils and Water*

*13.4.1* Clause *13.4* refers primarily to concretes placed in soils and waters, containing sulphates, nitrates and other salts which may cause deterioration of concrete. Naturally occuring aggressive chemicals such as sulphates of sodium and magnesium, are sometimes found in soils and waters. Sea water is mildly aggressive to concrete because of the soluble sulphates it contains (*see 13.3* for precautions). The decomposition of sulphide minerals contained in colliery waters may result in the formation of $F_2SO_4$ which can cause severe sulphate attack. Durability problems may arise also when concrete is exposed to acids. Appendix A of the Code deals with concrete exposed to sulphate attack.

Two types of precautions are given in the Code:

a) Those in which proper attention to the concrete itself will provide sufficient immunity.

b) Those in which additional precautions are to be taken to prevent contact between the aggressive chemicals and the concrete.

Note 4 in Table 20 requires a reduction in water-cement ratio and an increase of cement content for partially immersed structures and for those in contact with aggressive agents on one side only.

*13.4.2* DRAINAGE — High concentrations are those in which the total $SO_3$ in soil is over 1 percent and the sulphate in ground water, expressed as $SO_3$ is over 250 ppm (that is, beyond the upper limits of Table 20). Whenever acids and high concentrations of sulphates are anticipated, Portland cement concrete should be protected from exposure to these aggressive agents.

## *14.* SAMPLING AND STRENGTH TEST OF CONCRETE

*14.0* The sampling scheme given in *14* and the acceptance criteria given in *15* are applicable to both design mix concrete and nominal mix concrete. In the case of the latter, the preliminary tests for establishing the mix proportion are not necessary.

*14.1 General*—IS : 1199-1959* requires that the sample should be of at least 0.02 $m^3$ in volume and that it should be made up by collecting the concrete from three to five intervals or locations. The composite sample of 0.02 $m^3$ volume should be mixed to ensure uniformity. This volume of the sample will be about twice that required for moulding three specimens of 150 mm cubes.

IS : 516-1959† covers tests for determination of compressive strength (required for *14* and *15*) and flexural strength (given as an option in *14.1.1*). A difference regarding the preparation of specimens can be noticed when IS : 516-1959† is compared with *14.3* of the Code. The former requires that three specimens should be made from three different batches whereas the Code stipulates that three specimens should be made from one sample taken from a particular batch. For the purpose of the Code, it will be more logical to follow the requirements of *14.3* without any modification that is, three specimens should be made from one sample drawn from one batch.

IS : 516-1959† stipulates that the individual variation in the compressive strength of cubes in a sample should not be more than ± 15 percent of the average (*see also 14.4*). In other words, the range of the test values should not exceed 30 percent of the average. This within-test-range limitation ensures that the procedures for fabrication, curing and testing of specimens are of an acceptable quality. Whenever this range is exceeded, the procedures for fabrication of specimens and the calibration of the testing machines should be checked.

*14.1.1* —

*14.2 Frequency of Sampling*—Although desirable in theory, strict adherence to statistical random sampling procedure is generally not possible in practice. Sampling should be arranged in such a manner as to avoid conscious bias and provide a reasonable representation of the concrete concerned. Sampling records must indicate, in a suitable manner, which batches were chosen for testing.

*14.2.1* SAMPLING PROCEDURE—The random sampling procedure should ensure that each batch, in a given day or period, should have the same chance of being sampled, as any other of the remaining batches. Therefore, samples drawn at equal intervals of time need not constitute random sampling. A procedure for random sampling is described in IS : 4905-1968*.

*14.2.2* FREQUENCY—The quantity of concrete that may be considered for a single lot of statistical treatment of the results of tests on the samples taken from it must depend upon engineering judgement (*see also* Note under *14.1*).

The rate of sampling may have to be increased at the start of the work to establish the level of quality, quickly [that is, until 30 samples are obtained [*see 14.5.1* (a)] or during periods of production when the quality is in doubt.

For batches of volume less than 1 $m^3$; it will not be necessary to obtain a sample for every batch or every day but it is preferable to ensure that at least one sample (consisting of 3 specimens) is available for every 5 $m^3$ of concrete placed. On the other hand, for highly stressed but isolated members of volume less than 1 $m^3$, it may be desirable to increase the rate of sampling to ensure that the concrete in such members is of adequate strength.

*14.3 Test Specimens*—The term test specimens should be distinguished from samples. In the rest of the Clauses of the Code, the principle followed is that the strength of the sample (that is, the mean of the values of three specimens) represents the strength of concrete and that the range of the values within the sample (that is, the difference between the highest and the lowest values of the three values for the specimen) indicates the uniformity of procedures adopted in fabricating, curing and testing the specimens.

Early strength test results may be used for early remedial measures, such as the changing of mix design, if necessary, but should not be used for assessment of strength in terms of *15* of the Code.

The random sampling procedure (*14.2.1*) does not apply for the additional cubes (*see*

*Methods of sampling and analysis of concrete.
†Methods of tests for strength of concrete.

*Methods for random sampling.

*also* the comments on *14.1*, regarding sampling and testing).

*14.4 Test Strength of Sample*—Even when the concrete is sampled and tested strictly in accordance with the relevant procedures, several factors can influence the strength of a test specimen and the strengths of two or more specimens representing a sample of concrete may not be the same. These factors include the heterogeneous nature of concrete; and the human element in (a) obtaining the sample, (b) in casting, curing and testing of each specimen and subsequent rounding off of the calculated test strength. The difference between the strength of specimens from one sample tested at the same age is to be within the range ± 15 percent.

*14.5 Standard Deviation*—The standard deviation of the results from their mean value is regarded as an index of the scatter and consequently of the degree of site control.

In the Code, it is recognized that when testing a large number of samples of a given grade of concrete manufactured even under stable conditions of control are tested, the results will show some variation. For the strength of concrete, it is usual to assume that the plot of compressive strength (on $x$ axis) against the frequency (No. of samples) of occurrence (on $y$ axis) will follow the normal or Gaussian distribution, which is a bell shaped curve as shown in Fig. E-3.

Throughout the Code, the characteristic strength, which will be less than the mean strength (target strength), is taken as a reference for designating the grade of concrete, as say M 20. The acceptance criteria given in *15* of the Code is also based on the requirement that the dispersion in the test strength of samples should be allowed for.

The average or mean value can be taken as a measure of central tendency and the standard deviation as a measure of dispersion. A small value of standard deviation will result in a curve with a dominant peak, whereas a large value will result in a flatter curve depending upon the level of control exercised in the manufacture of concrete (*see* Fig. E-4).

Reliable value for standard deviation can be obtained only after testing fairly large number of samples, say 30 to 40. Otherwise, it will be necessary to assume a reasonable value of standard deviation, either based on past experience, or by using the values given in Table 6. Also, *see* comments on *8.1.1* regarding the use of standard deviation for arriving at target strength in the design of concrete mixes.

*14.5.1* STANDARD DEVIATION BASED ON TEST RESULTS—It is suggested that standard

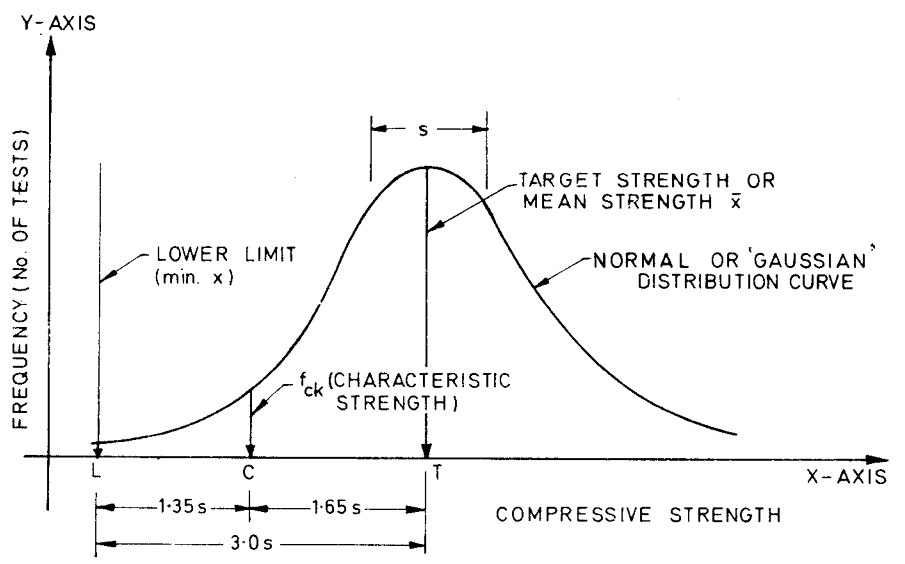


*Fig. E-3 Normal Distribution of Concrete Strength*

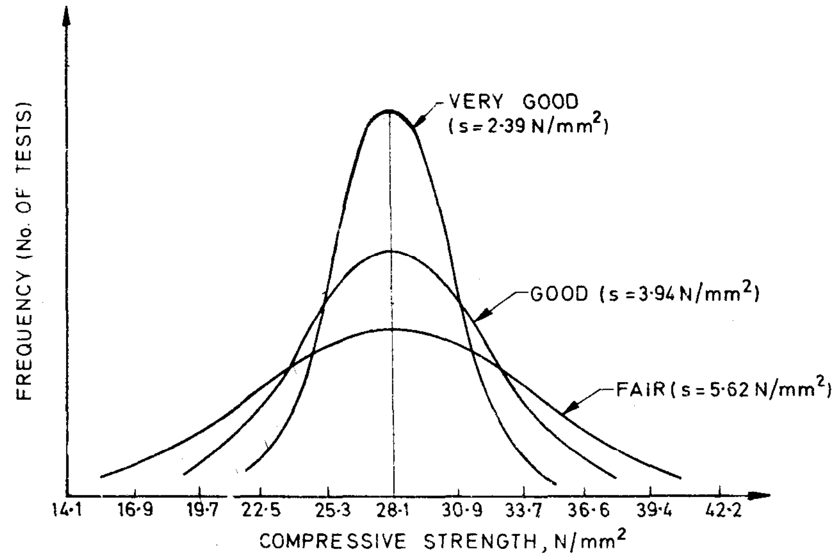


*Fig. E-4 Typical Normal Frequency Curves for Different Levels of Control*

deviation calculations should be begun a fresh and not brought up-to-date, whenever there is a change in mix design.

The sub-para (b) of the Clause with regard to updating the standard deviation refers to minor changes in the mix which might be made at site either to correct consistently higher results or *vice-versa,* where as the materials remain the same, and particularly the water-cement ratio.

*14.5.2* DETERMINATION OF STANDARD DEVIATION — The calculation of standard deviation is illustrated in the following example. For illustration, only ten samples are considered, but note that at least 30 sample should be used for this purpose, according to *14.5.1*, para (a) of the Code. The test results on specimens are obtained from the field as the concreting proceeds, and this will form the data (columns 2, 3, 4 and 5). Grade of concrete is M 15.

$$n = 10, \quad n - 1 = 9$$

$$x = \frac{201.8}{10} = 20.18 = 20.2 \text{ (say)}$$

$$\text{Standard deviation, } s = \sqrt{\frac{\Delta^2}{n-1}} = \sqrt{\frac{75.4}{9}} = 2.89$$

*Example:*

**TEST RESULTS**

| *Sample No.* | *28 Days Strength N/mm²* | | | Average $x_i$ | $(\bar{x}-x_i)$ | $(\bar{x}-x_i)^2$ |
|---|---|---|---|---|---|---|
| | $Cube_1$ | $Cube_2$ | $Cube_3$ | | | |
| (1) | (2) | (3) | (4) | (5) | (6) | (7) |
| 1 | 20.5 | 24.0 | 22.5 | 22.3 | −2.1 | 4.41 |
| 2 | 18.5 | 22.5 | 19.0 | 20.0 | +0.2 | 0.04 |
| 3 | 19.5 | 20.5 | 21.5 | 20.5 | −0.3 | 0.09 |
| 4 | 22.0 | 23.0 | 21.5 | 22.2 | −2.0 | 4.00 |
| 5 | 18.5 | 21.5 | 21.5 | 20.5 | −0.3 | 0.09 |
| 6 | 22.5 | 23.5 | 23.0 | 23.0 | −2.8 | 7.84 |
| 7 | 24.0 | 23.5 | 21.5 | 23.0 | −2.8 | 7.84 |
| 8 | 22.0 | 18.5 | 19.5 | 20.0 | +0.2 | 0.04 |
| 9 | 16.5 | 15.5 | 14.0 | 15.3 | +4.9 | 24.01 |
| 10 | 13.0 | 15.0 | 17.0 | 15.0 | +5.2 | 17.04 |
| | | | | $\Sigma x_i = 201.8$ | | $\Delta^2 = 75.4$ |

*14.5.3* Where insufficient test results (less than 30 samples) are available, an assumed standard deviation has to be used. The values in Table 6 of the Code are provided as initial estimates which should be slightly conservative. (They tend to be more conservative at higher strengths where strength may be more important.)

*15.* ACCEPTANCE CRITERIA—Clause *15* specifies two sets of criteria for demonstrating that the concrete as supplied, either complies or does not comply with the Code requirements for concrete quality:

a) requirements of strength; and
b) requirements of workmanship (*see 15.7*).

These criteria, as well as the sampling scheme given in *14* are applicable to both design mix concrete as well as nominal mix concrete.

*15.1* and *15.2*—The Code specifies the strength requirements in terms of characteristic strength and standard deviation, the former being a reference value preferred for specifications and design. However, it is convenient here to introduce the term target strength.

Characteristic strength is that value of the strength below which not more than 5 percent of the test results are expected to fall (*see 35.1*). Considering the inherent variability of concrete strength during production, it is necessary to design the concrete mix to have a target mean strength which is higher than the characteristic strength by a suitable margin.

$$f_t = f_{ck} + K s$$

where

$f_t$ = target mean strength
$f_{ck}$ = characteristic strength
$s$ = standard deviation
$K$ = constant equal to 1.65 as per the definition of characteristic strength where not more than 5 percent of the test results are expected to fall below the characteristic strength.

The main statistical features of the acceptance criteria are as follows (*see* Fig. E-3):

a) If every sample is to the right of C, accept each of the sample.
b) A sample falls below C, but not beyond L or 0.8 times the characteristic strength—accept if, the mean of all values is above T. This condition is reworded as, actual average strength $\geq$ characteristic strength + 1.65 × standard deviation.

For small groups of samples, a correction is inserted and the expression is,

Actual average strength $\geq$ characteristic strength

$$+\left(1.65 - \frac{1.65}{\sqrt{n}}\right)s$$

c) Strength of sample falls below L or 0.8 times the characteristic strength (whichever is greater)—concrete is deemed not to comply with strength requirements.
d) The average strength of all samples falls below T. That is, the average strength of all the samples is less than the characteristic strength $+\left(1.65 - \frac{3}{\sqrt{n}}\right)s$. Concrete represented by this sample is deemed not to comply with the strength requirements.

*Illustrative Example*

In a concrete work, concrete of grade M 20 ($f_{ck}$ = 20.0 N/mm²) is to be used. The standard deviation for this grade of concrete has been established to be 4.0 N/mm². In the course of testing concrete cubes, the following results are obtained from a week's production (average strength of 3 specimens tested at 28 days in each case expressed in N/mm²):

24.8, 27.0, 28.5, 23.6, 18.0, 21.6, 15.0 N/mm².

a) The first four results are straightway accepted, the sample strength being greater than the characteristic strength (20.0 N/mm²) in each case.
b) The 5th result of 18.0 N/mm² is less than the characteristic strength (20.0 N/mm²) and is compared with:
   i) 0.8 times characteristic strength, that is, 16.0 N/mm²; and
   ii) (20.0 − 1.35 × 4.0) that is, 14.6 N/mm².

   Since 18.0 N/mm² is greater than 16.0 N/mm², we check the average strength of the samples which is,

$$(24.8+27.0+28.5+23.6+18.0) \div 5 = 24.4 \text{ N/mm}^2;$$

$$\text{and } 20.0+\left(1.65-\frac{1.65}{\sqrt{5}}\right)\times 4.0$$

$$= 23.6 \text{ N/mm}^2$$

Since the average of 5 samples is greater than 23.6 N/mm², the 5th sample is acceptable.

c) The 6th result is also acceptable being greater than the characteristic strength. The 7th one (15.0 N/mm²) is lower than 16.0 N/mm² [obtained in step (b)]. The average strength of all the 7 samples is:

$$24.8+27.0+28.5+23.6+18.0+21.6+15.0 \div 7 = 22.6 \text{ N/mm}^2$$ which is greater than

$$20.0+\left(1.65-\frac{3}{\sqrt{7}}\right)\times 4.0$$

$$= 22.1 \text{ N/mm}^2$$

The seventh sample thus does not comply with the requirement as it is less than 16.0 N/mm² (but cannot be deemed not to have complied with the requirement), the acceptance will depend upon the discretion of the designer.

*15.3* —

*15.4* (*See also* comments on *15.1* and *15.2*). In *15.2* concrete is deemed not to comply with the strength requirements, if the strength of sample is 3 × standard deviation below the target strengths which in turn approximately corresponds to a probability of 1 in 1 000. Such strengths are extremely unlikely and, therefore, call for further investigation.

Understandably, the Code leaves to the judgement of the designer the consequential action that should be taken. The following remarks should be taken only as hints on the spirit of the Code and not necessarily as binding on the designer who may have to take other factors into account.

The action in the event of non-compliance of the strength requirements may vary from qualified acceptance in marginal or less severe cases to rejection and removal in extremely severe cases. The Code deals only with technical recommendations and the phrase 'consequential action' refers only to the technical consideration, and not to 'legal actions' which are outside the purview of the Code. In determining the steps to be taken, the designer should consider the technical consequences (for example durability, strength and serviceability and the economic consequences, such as cost of replacement, cost of strengthening the weak point, etc).

In investigating the suspected portions of the structure, and before the remedial steps are taken, the following points should be considered:

a) An appraisal of the sampling and testing procedures, to ensure that they are valid;
b) The mix proportions actually used in concrete. These may effect durability;
c) The influence of any reduction in concrete quality on the strength, serviceability and durability of the affected portion of the structure.

Core tests and load tests (*see 16*) should be used as guides for deciding upon the technical measures to be taken. Favourable results from the above tests will be helpful for avoiding costly replacement or strengthening measures.

*15.5* —

*15.6* —

*15.7* The requirements of this clause relates to workmanship. Porous or honeycombed concrete results from incorrect mix proportion (incorrect consistency of concrete) or improper compaction techniques. This defect can be made good either by patching, if the pockets are located near the surface, or by grouting in other cases.

The defects that are likely to be noticed at the improperly made construction joints are:

a) Horizontal joints:
   1) Laitance formation, along with weak and porous layer at the top of the bottom pour.
   2) Deficiency of cement and excess of aggregates, both due to segregation at the bottom of the top pour.
b) Vertical joints:
   1) Formation of 'feather-edges'
   2) Shrinkage cracks along the jointed surface.

Tolerances for reinforcements are specified in *11.3*. Once the concrete is placed, it is difficult to check the tolerance of reinforcements. However, the position of steel near the surface of concrete (say within 70 mm) can be determined by using a magnetic cover meter.

## *16.* INSPECTION AND TESTING OF STRUCTURES

*16.1 Inspection*

*16.2* (*See also 15.1* and *15.2* and their comments). Concrete will be deemed not to comply with the strength requirements:

a) if there are stray cases of extremely low strength; and

b) if the average of all the samples is extremely low.

In such cases core tests may be required to decide further measures.

*16.3* —

*16.3.1* —

*16.3.2* Core cutting that is, drilling wherever possible, should be avoided in reinforcements. The procedure for preparing and testing cores drilled from concrete are given in IS : 516-1959*. The cores should be undamaged representative of the concrete. The cores should be capped before testing, using the procedure given in IS : 516-1959*.

Standard diameter of core is 150 mm; however cores of 100 mm diameter may also be used and the strength assessed from both cores will be about the same. The length of core should be at least 95 percent of the core diameter. The equivalent cube strength (on 150 mm cubes) may be obtained in the following manner:

a) Strength of core $= f_o$ (on 100 mm diameter core)

b) Apply correction for diameter: if diameter is less than 100 mm, correction factor = 1.08

$$\left.\begin{array}{l}\text{Strength of}\\ \text{150 mm core}\end{array}\right\} = 1.08 \times f_o$$

c) Apply correction for height to diameter ratio, if this is less than 2.0, the factor being obtained from IS : 516-1959*

$$\left.\begin{array}{l}\text{Strength of}\\ \text{standard core}\end{array}\right\} = K \times 1.08 \times f_o$$

d) Convert this result into equivalent cube strength. Compressive strength of 150 mm cube = 1.25 (compressive strength of 150 × 300 mm cylinder) $= 1.25 \times (K \times 1.08 \times f_o)$.

e) Compare the average value of three specimens with the specified strength. Correction for age of concrete may be obtained from *5.2.1* (*see also* comments on *5.2.1*, especially for the influence of the type of cement).

*16.4* The number of test cores should be as large as possible with due regard to economic considerations. In practice, it is usual to secure only three cores (the minimum recommended by the Code) and average the strength results. This sample mean is only an approximation of the strength of concrete in the structure and the results are dependent on the number of cores. It is important, therefore, that structure should not unnecessarily be declared unfit simply because the sample mean may fall below 85 percent of the required strength especially when only a small number of cores are tested. Either further core tests may be carried out or a load test is resorted to or the load-carrying capacity of the structure recalculated in the light of existing core results.

*16.5 Load Tests on Parts of Structures*— (*See* Ref 13). This clause and its sub-clauses are intended to provide reassurance or otherwise as to the adequacy of doubtful structural units. These rules are not to be used as a substitute for normal design procedure.

When it is required that a static load test is to be carried out cognizance should be taken of the effect on the test of the loading pattern. The loading medium should not be stored temporarily on any part of the structure where it could affect the test.

Members other than flexural members should preferably be investigated by analytical procedures due to problems of load application and the difficulty of detecting incipient failure.

The loads are expected to be applied and removed incrementally. The procedure for

*Methods of test for strength of concrete.

load test and the interpretation of results should be under the supervision of an experienced structural engineer.

*16.5.1* Note that 1964 version of the Code specifies that load tests may be carried out after 56 days of effective hardening. Now the requirement is that it should be carried out as soon as possible, after 28 days of the casting. A generous hardening period of 56 days had been suggested earlier which ensured adequate strength developments even in cold climates. It is quite unlikely that a period of 56 days will ever be required. Tests should normally be made at an age of four to six weeks. Testing before 28 days from the date of placing concrete is usually undesirable as it may lead to permanent weakening of the structure.

*16.5.2* Here imposed load means all loads other than the dead load of permanent construction.

The imposed load in a test should include the static equivalent of the appropriate dynamic augment (impact) of any moving load. Compensating loads should be added whenever full dead load is yet to come on the structure.

The increase of 25 percent in imposed load is intended to provide a reasonable overload to assure safety but not so severe as to damage a satisfactory structure. Effects of temperature and humidity on the deformation of the structure is minimized by adopting a 24 hour cycle for measurements.

*16.5.3* During the 24 hours under the test load, a reinforced concrete structure will show a progressively increasing deflection, owing to the creep of concrete. Members constructed with low strength concretes (say M 15) are likely to show a progressive deflection (in 24 hours) almost equal to the initial elastic deflection and may not show a 75 percent recovery immediately on removal of load. However, some subsequent recovery should take place, bringing it within the prescribed limits, and, therefore a recovery period of 24 hours is specified. If within 24 hours this recovery does not take place, the second test should be made.

Creep will be less during the second test, and in a sound structure no difficulty is likely to occur in conforming to the recovery requirements. In unsound members, evidence of weakness will be apparent in other ways, generally in the form of excessive cracking. Any deformation that progresses faster than and not proportional to the loading rate should be viewed with suspicion. For the second test, the datum is the deflection or the permanent set that remains at the end of first test.

*16.5.3.1* The provision that the maximum deflection is less than 40 $l^2/D$, the requirement for recovery do not apply, allows for the behaviour of very stiff structures or those in which significant membrane action develops (for example shells). In such cases, the absolute deflection will be small and of the same order as might be expected from thermal and moisture effects. A true appraisal of the elastic behaviour of the structure then becomes difficult and, therefore, unnecessary.

*16.6* The use of non-destructive tests such as the Schmidt hammer test or ultrasonic tests, can be helpful in comparing the strength of doubtful concrete in the job that is considered to be satisfactory. Hammer test measurements should be taken on smoothed test patches about 150 mm in diameter with about 10 readings being taken uniformly over each test area. Due to variations, such as aggregate, surface smoothness and mode of operation of the impact hammer, a 20 percent coefficient of variation in strength reading is common.

SECTION 3

# GENERAL DESIGN REQUIREMENTS

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# SECTION 3 GENERAL DESIGN REQUIREMENTS

## *17.* LOADS AND FORCES

*17.1 General—(see 17.5.1)* In ordinary structures such as low rise dwellings, effects due to temperature fluctuations, creep and shrinkage can be ignored in the design calculations. In *26.3*, the Code recommends that expansion joints should, in general, be provided in structures exceeding 45 m in length which will normally take care of temperature effects, in ordinary structures.

*17.2 Dead Loads—(see* IS : 875-1964*)— Dead load in a building comprises the weight of all walls, partitions, floors, roofs and the weights of all other permanent constructions in the building. IS : 875-1964* recommends also the value of the equivalent dead load to be assumed when the final positioning of the partitions cannot be assessed in advance.

*17.3 Live Loads and Wind Loads*— IS : 875-1964* lays down the minimum design loads which have to be assumed for the dead loads, live loads, wind loads and other external loads in different types of occupancies. It does not take into consideration loads incidental to constructions and special cases of vibration (such as moving machinery), heavy acceleration from cranes and hoists. Such loads should be assessed and dealt with individually in each case (*see also 17.6*).

*17.4 Earthquake Forces* — IS : 4326-1976† is a necessary adjunct to IS : 1893-1975‡. The former gives some guidelines for earthquake resistant construction of buildings. Broadly, the following general principles are indicated:

a) For less important and relatively small structures, no analysis for earthquake forces need be made, provided certain construction rules/precautions are observed. For example, diagonal bracing in vertical panels add to seismic resistance of frames; in highly seismic areas construction entailing heavy debris may be avoided by observing the precautions given in IS : 4326-1976*;

b) For buildings up to 40 m in height, seismic coefficient method may be used for estimating the earthquake forces.

c) For buildings greater than 40 m in height and up to 90 m, model analysis is recommended. However, the seismic coefficient method may also be used in earthquake Zones I to III.

d) For buildings taller than 90 m in zones other than I and II detailed dynamic analysis shall have to be made based on expected ground motion and model analysis for buildings in Zones I and II.

Load factors (partial safety factors) for earthquake forces are given in *35.4.1* (*see* comments on *35.4.1*). Also *see 44.3* for increase in permissible stresses in working stress method of design.

*17.5 Shrinkage, Creep and Temperature Effects—(See 17.5.1* for exemption). Shrinkage and creep do not normally affect the ultimate strength of a reinforced concrete section. However, they affect the deflection of structures at working load. For normal applications, the use of span/depth ratios given in *22.2.1* will take into account shrinkage and creep in beams and slabs. A more accurate estimate of deflection can be made through the use of Appendix B.

Columns are also normally proportioned by using the ultimate strength methods and creep and shrinkage are not likely to affect the strength of columns. However, these effects along with axial shortening due to external loads are likely to substantially alter the forces in the columns and beams of tall frames (more than 30 storeys high). An approximate procedure of analysis is given in Ref 14.

The ultimate strength of a slender concrete column can be substantially reduced by

*Code of practice for structural safety of buildings: Loading standards (*revised*).

†Code of practice for earthquake resistant design and construction of buildings (*first revision*).

‡Criteria for earthquake resistant design of structures (*third revision*).

*Code of practice for earthquake resistant design and construction of buildings (*first revision*).

creep due to the application of a sustained load leading to creep buckling. Clause *38.7* dealing with design of slender columns recommends either (a) determining the moments by including the effects of deflection, or (b) taking into account additional moments given in *38.7.1*. If the latter simplified approach is adopted, the effect of creep need not be considered. In the former case, creep must also be taken into account (Ref 15 gives a treatment to creep buckling of slender columns).

Composite constructions involving reinforced or prestressed concrete as one of the components will require an analysis for creep and shrinkage effects (*see* IS : 3935-1966*). These topics are discussed in Ref 16 & 17.

Under certain conditions, stresses due to drying shrinkage and temperature changes should be assessed explicitly and provided for in the design of liquid storage structures [*see* IS : 3370 (Part II)-1965†]. Bins intended for storage of hot materials must be analysed for thermal stresses which should be catered for through the provision of additional reinforcement [*see* IS : 4995 (Part II)-1974‡]. According to IS : 4998 (Part I)-1975§, temperature effects both in vertical direction as well as circumferential direction should be considered in the design of chimneys.

*17.6 Other Forces and Effects*

a) FOUNDATION MOVEMENTS — Foundation movements result in three types of distortions: (i) maximum settlement (absolute value); (ii) average tilt (vertical); and (iii) angular distortion. The third type that is, angular change due to differential settlement is responsible for major cracking of buildings. The permissible values of total settlement, differential settlement and angular distortion are given in IS : 1904-1978*. Within these limits, the superstructure can accommodate itself to the movements, without harmful distortions. It is a general practice to design the foundations so that the angular distortion is limited rather than to design the superstructure to withstand the forces caused by differential settlement. For further information, *see* Ref 18.

b) ELASTIC AXIAL SHORTENING — Elastic axial shortening of column is to be considered along with the creep and shrinkage effects, in tall buildings (30 storeys or more high) *see* Ref 14 for further information.

c) SOIL AND FLUID PRESSURE — In the design of structures or parts of structures below ground level, such as basement floors and walls, the pressure exerted by the soil or water or both shall be duly accounted for on the basis of established theory. Due allowance shall be made for possible surcharge from stationary or moving loads.

d) VIBRATION — In most of the concrete structures subject to predominately static loads, the stiffeners provided to comply with the limitations on deflection will be such that no further consideration of vibration is necessary. Effect of vibration should be taken into consideration in the case of columns with crane girders.

e) FATIGUE

1) *Low cycle fatigue* — When repeated loading is not associated with large-scale stress reversal, the mechanisms of failure are usually the same as under static loading and the strength and ductility are not adversely affected. Under large-scale stress reversals, which can be expected during earthquakes, the failure mechanisms themselves may be altered and the reduction in load carrying capacity or ductility may be significant. In earthquake resistant designs, the possibility of changes in failure mechanisms is taken care of by

*Code of practice for composite construction.

†Code of practice for concrete structures for the storage of liquids: Part II Reinforced concrete structures.

‡Criteria for design of reinforced concrete bins for storage of granular and powdery materials: Part II Design criteria.

§Criteria for design of reinforced concrete chimneys: Part I Design criteria (*first revision*).

*Code of practice for structural safety of buildings: Shallow foundations (*second revision*).

changes in detailing rather than by modifications in load factors or in the amount of reinforcement (*see* Ref 19 and 20 for further information).

2) *Fatigue proper*— In case of high cycle fatigue, emphasis is on strength and not on deformations; the loads are applied through say millions of cycles and the stresses causing failure may be considerably below the yield point. This type of fatigue loading is extremely unlikely in most concrete structures. Even in those special case where the primary loading is of a fatigue type (for example, railway sleepers) the fatigue behaviour of a properly designed member will be usually satisfactory, since damages or adverse reduction in strength are not likely until after several millions of cycles of load application. The only significant effects are on the widths of cracks and on deflections; these increasing by 20 to 25 percent compared to the equivalent static loading.

f) IMPACT

1) In certain cases, impact on the main structural component can be avoided through the use of protective barriers, for example, in factories where the impact of lift trucks against columns is possible, the columns can be protected by surrounding them with rails.

2) Sometimes, over design through empirical formulae or *ad-hoc* factors are resorted to.

3) Accidental impacts, as in handling precast products, are avoided by adopting good practices of handling; specific design for accidental impact forces is rarely carried out.

4) Structures, such as forge-hammer foundations and certain harbour works whose main functions are to resist impact must be designed for energy absorption, rather than strength. Design must proceed along the lines that produce a satisfactory earthquake resistant structure.

g) ERECTION LOADS— All loads required to be carried by the structure or any part of it due to placing or storage of construction materials and erection equipment including all loads due to operation of such equipment, shall be considered as 'erection loads'. Proper provision shall be made to take care of all stresses due to such loads.

h) Stress concentration at the application of point loads, re-entrant angles, at opening in slabs shall be taken into account in the design of concrete structures.

*17.7 Combination of Loads — See also 19.1, 19.2, 35.4.1* and *44.3*, for recommendation relating to combination of loads. For frames, the arrangement of live loads is given in *21.4.1.*

*17.8 Dead Load Counteracting Other Loads and Forces*

*17.9 Design Loads*— The term design load has been defined for convenience while dealing with Clauses which are common to both methods of design (working stress method and limit state method).

For definition of characteristic load, *see 35.2*. Partial safety factors for loads are given in *35.4.1* (Table 12). There are separate sets of partial safety factors for the limit state of collapse and limit state of serviceability.

## *18.* BASES FOR DESIGN

*18.1 General*

*18.2 Methods of Design*

*18.3 Design on Experimental Basis*

## *19.* STABILITY OF THE STRUCTURE

*19.1 Overturning*

*19.2 Sliding*— IS : 1904-1978* states as follows with regard to safety against sliding of structures.

The factor of safety against sliding of structures which resist lateral forces (such as

*Code of practice for structural safety of buildings: Shallow foundations (*second revision*).

retaining walls) shall be not less than 1.5 when DL, LL and earth pressure are considered together with wind load or seismic forces. When DL, LL and earth pressures only are considered, the factor of safety shall not be less than 1.75.

*19.3* Here moment connection probably refers to precast construction. For *in-situ* construction, monolithic action should be ensured at the joints.

## *20.* DURABILITY AND FIRE RESISTANCE OF THE STRUCTURE

*20.1 Durability*—The durability requirements shall be determined by the designer and shall be specified in accordance with the Code. Durability of concrete (in particular, resistance to abrasion, sulphate attack, water penetration) shall be suitable for the expected service conditions.

Requirements of cover to the reinforcements with reference to durability are given in *25.4.2* to *25.4.2.4*. Detailing requirements given in *25.5.1.1* (a) (minimum reinforcement) and *25.3.2* (spacing of bars) are important for crack control.

*See also 13.3* (concrete in sea water), *13.4* (concrete in aggressive soils and water), and Appendix A (concrete exposed to sulphate attack).

Reference may also be made to IS : 9077-1979* which deals with the protection of reinforcement from corrosion in reinforced concrete construction.

*20.2 Fire Resistance*—(For comprehensive treatment of this topic *see* Ref 3 and 21). For protection against fire, three aspects are usually considered:

a) Retention of structural strength;
b) Resistance to penetration of flames; and
c) Resistance to heat transmission.

The first criterion is applicable to all elements of construction, while walls and floors which perform a separating function are judged on the other two criteria also. IS : 1642-1960† gives the grading for the fire resistance for walls, columns and beams, floors, roofs and other structural components. The following information is intended to supplement the recommendation of IS : 1642-1960*.

a) CONCRETE AGGREGATES—Usually the data with respect to fire resistance of concrete elements refer to concretes made with siliceous aggregates, which give the lowest fire resistance. Whenever possible, lime stone or blast furnace slag should be used as aggregates which will lead to increased fire resistance.
b) THICKNESS OF MEMBER—IS : 1642-1960* gives the minimum thickness of reinforced concrete walls and floors for different fire resistance gradings, but this does not cover reinforced concrete columns and beams. A summary of the dimensions of reinforced concrete elements (that is, cover and thickness) taken from IS : 1642-1960* and Ref 3 is given in Table E-3.

The fire resistance (of an element of a structure) may be defined as the time during which it fulfils its function of contributing to the fire safety of a building when subjected to prescribed condition of heat and load or restraint.

## *21.* ANALYSIS

*21.1 General*—For either the working stress design method or limit state design method, the internal forces and moments acting in an indeterminate structure may be evaluated by elastic analysis (on the assumption of linear elastic behaviour).

DESIGN LOAD—(*See 17.9* and *35.3.2* for definition). For limit state method of design, factored loads are to be used. Factored loads are service (working/characteristic) loads multiplied by appropriate load factors (*see* Table 12). When working stress method is used, the design loads are service (working/characteristic) loads (with load factors of unity.

LINEAR ELASTIC THEORY—For all cases, the Code permits the usual elastic methods of

*Code of practice for corrosion protection of steel reinforcement in RB and RCC construction.

†Code of practice for fire safety of buildings (general): Materials and details of construction.

*Code of practice for fire safety of buildings (general): Materials and details of construction.

analysis, such as moment distribution method, slope-deflection method, etc, without an explicit consideration of moment-curvature and moment-rotation relationships even while designing frames and continuous structures for limit state of collapse. In other words, while analysing continuous beams and frames it will be sufficient to carry out linear-elastic analysis, be it working stress method of design (Section 6) or limit state method of design (Section 5).

Clause *36.1* implies that the properties of materials that may be required for analysis of the structure may be based on the characteristic strengths. In other words, irrespective of the method of design used, no factors need be applied to the strengths while determining the properties of members (that is, $E$ and $I$) for analysing a structure.

Continuous beams and rigid frames, if designed for adequate ductility, have some reserve strength with respect to ultimate failure which is not revealed by a linear elastic analysis. The Code permits this reserve strength to be taken into account through redistribution of moments (*see 21.7* and *36.1.1* for limit state method, and *43.2* for working stress method of design). However, the redistribution of moments along with a simplified analysis using moment coefficients for continuous beams and slabs is not permitted (*see 21.5.1*).

Simplified analysis are permitted in the Code for the following cases in addition to the recommendations of *21.4* and *21.5* for rigid frames and continuous beams, respectively:

a) Coefficients for the design of 2-way slabs, including hollow, voided and ribbed slabs spanning in two direction (*see 23.4* and *29.2*);
b) A simplified method for design of deep beams without recourse to analysis based on theory of elasticity (*see 28.2*); and
c) Coefficient for bending moments in flat slab structures (*see 30.4*).

In the case of frames containing slender compression members, the Code states (*see 38.7*) that it will be preferable to carry out an analysis which includes the second order effects that is, effect of lateral deflections (sway) of columns on moments and forces. However, this second order analysis need not be done if slender columns are designed for the additional moments given in *38.7.1*.

In analysing buildings built with a skeleton consisting of a series of plane frames connected by transverse beams, it is usual practice to ignore the torsional resistance or stiffness of the transverse beams. In such cases, that is, when the stability of the system does not depend on the torsional strength of certain members, the torsional phenomenon as a whole can be ignored in analysis as well as in design. However, in structure such as beams curved in plan, equilibrium itself is not possible without torsion in the members. In such cases the member should be analysed and provided for torsion (*see 40.1*).

*21.2 Effective Span*—The term 'effective depth' is defined in *22.0*. The points to be noted under this clause are:

a) In general, wherever the term 'clear span' is not specifically mentioned, it can be reasonably assumed that the word 'span' means 'effective span' in the Code;
b) Slenderness limits for beams are given in terms of clear distance (*see 22.3*);
c) In some clauses, owing to special circumstances, the definition of 'effective span' is changed. These clauses are: *28.2* (deep beams) and *32.1* (stairs);
d) Sometimes it may be necessary to determine the effective span of cantilevers. This may be taken as the length of the cantilever to the face of the support plus half its effective depth, except where it forms the end of a continuous beam in which cases, the length to the centre of support should be used (Ref 3);
e) In the analysis of a continuous frame, considering the frame as a whole, the effective span of a flexural member shall be the distance between the centre lines of supporting members; and
f) Even though the moments are computed on the basis of effective spans, in monolithic construction it is permissible to design the section at the support on the basis of the moment at the face of the support (*see 21.6.1*).

TABLE E-3 FIRE RESISTANCE OF REINFORCED CONCRETE ELEMENTS

| SL No. | DESCRIPTION | DIMENSION OF CONCRETE TO GIVE FIRE RESISTANCE IN HOURS | | | | | REMARKS |
|---|---|---|---|---|---|---|---|
| | | 6 (TYPE 1) | 4 (TYPE 2) | 2 (TYPE 3) | 1 (TYPE 4) | ½ (TYPE 5) | |
| (1) | (2) | (3) mm | (4) mm | (5) mm | (6) mm | (7) mm | (8) |
| 1. | Rectangular beam: (siliceous aggregates) | | | | | | |
| | a) Average cover to main reinforcement | — | 65 | 45 | 25 | 15 | (*see* Ref 3) |
| | b) Beam width | — | 280 | 180 | 110 | 80 | (*see* Ref 3) |
| 2. | Solid slabs: (siliceous and calcareous aggregates) | | | | | | |
| | a) Average cover to reinforcement | — | 25 | 20 | 15 | 15 | (*see* Ref 3) |
| | b) Overall depth | 180 | 150 | 125 | 100 | 90 | (*see* IS : 1642-1960*) |
| 3. | Reinforced concrete columns without special protection; overall width (siliceous aggregates) | | | | | | |
| | a) All faces exposed | — | 450 | 300 | 200 | 150 | (*see* Ref 3) |
| | b) One face exposed | — | 180 | 100 | 75 | 75 | (*see* Ref 3) |
| 4. | Reinforced concrete wall with one face exposed, overall depth: | | | | | | |
| | a) Lime stone or blast furnace slag aggregate | 200 | 150 | 100 | 75 | 75 | (*see* IS : 1642-1960*) |
| | b) With siliceous aggregates | 230 | 180 | 100 | 75 | 75 | (*see* IS : 1642-1960*) |

NOTE 1 — When cover to reinforcement exceeds 40 mm, supplementary reinforcement may be necessary to hold the concrete cover in position, especially when using siliceous aggregates.

NOTE 2 — Reinforced concrete walls should be reinforced vertically and horizontally at not more than 150 mm c/c, the reinforcement being not less than 0.2 percent, of volume (and at least 1 percent of which should be in the vertical direction).

NOTE 3 — Concrete cover to the reinforcement in walls should be not less than 15 mm for fire resistance up to one hour and not less than 25 mm for longer periods (*see* Ref 3).

*Code of practice for fire safety of buildings (general): Materials and details of construction.

For effective span of slabs in composite construction (*see* IS : 3935-1966*).

*21.3 Stiffness*

*21.3.1* RELATIVE STIFFNESS—The definition of gross, transformed and cracked sections are self explanatory. Therefore, the points to be considered under this clause are:

a) The Clause speaks of relative stiffness, the consideration of which will be sufficient for analysing indeterminate structures. For explicit calculation of deflection of a beam/slab it will be necessary to consider the effective moment of inertia ($I_{eff}$), moment of inertia of the cracked section ($I_r$), short-term elastic modulus ($E_c$) and long-terms elastic modulus ($E_{ce}$) which are given in Appendix B.

b) Of the three assumptions permitted for determining relative stiffness, the assumption (a) associated with gross section will generally be used because of convenience. Assumptions (b) and (c) may be relevant when checking existing structures for new loadings associated with change of occupational use.

c) The value of modular ratio is given in *43.3* (d).

d) The moment of inertia of rectangular transformed section can be shown to be as (*see* Fig. E-5):

$$I_t = \frac{1}{12} bD^3 + bD\left(x - \frac{D}{2}\right)^2 + mA'_s\left(x - d'\right)^2 + mA_s\,(d-x)^2$$

*Code of practice for composite construction.

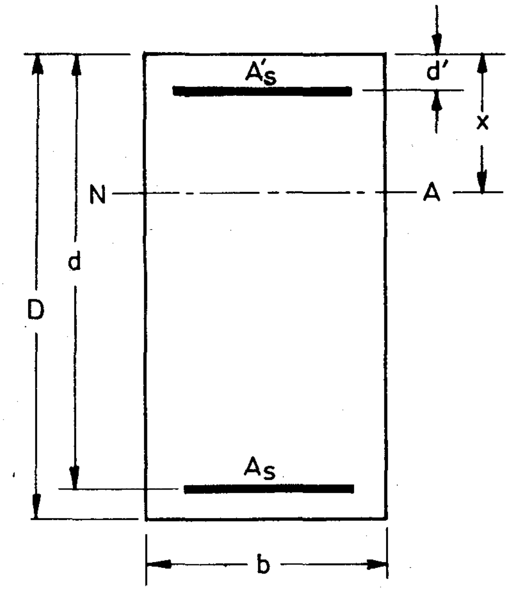



*Fig. E-5 Sketch for Transformed Sections*

where

$$x = d\sqrt{m(p+p')^2 + 2m\left(p'\frac{d'}{d}+p\right)} - md(p+p')$$

$m$ = modular ratio

$$p = \frac{A_s}{bd}$$

$$p' = \frac{A'_s}{bd}$$

e) The moment of inertia of a cracked section (rectangular shape) may be obtained from the following formula. For derivation, *see* any standard text book (for example, Reference 18).

$$I_r = m\,A_s\,d^2\,(1-k)\,(1-k/3)$$

where $k = x/d$, $x$ = depth of neutral axis as defined above.

f) For assessing the relative stiffness of flanged beams (T or L-beams), the effective flange width given in *22.1.2* may be used for computing the moment of inertia.

g) Sometimes it will be necessary to consider the torsional stiffness of members. As a guide, the recommendation given in Ref 3 can be used which is as follows:

The torsional rigidity ($GC$) of a member may be obtained by assuming the shear modulus $G$ equal to 0.4 times the modulus of elasticity of concrete and the torsional constant, $C$ equal to half the St. Venant torsional constant calculated for the plain concrete section. Values of St. Venant torsional constants, $K$, for rectangular section are given below, where $K = k\,bD^3$:

| $D/b$ | $k$ | $D/b$ | $k$ |
|---|---|---|---|
| 1.0 | 0.14 | 2.5 | 0.25 |
| 1.2 | 0.17 | 3.0 | 0.26 |
| 1.5 | 0.20 | 4.0 | 0.28 |
| 2.0 | 0.23 | 5.0 | 0.29 |

Any assumptions that are made for the purpose of calculating the relative stiffness of columns, walls, floors and roof systems shall be consistent throughout the analysis.

### *21.4 Structural Frames*

*21.4.1* ARRANGEMENT OF LIVE LOAD—The points to be noted under this clause are:

a) *21.4.1* (b) is not applicable if redistribution of moments is to be carried out.

b) Definitions of 'design dead load' and 'design live load' can be arrived from *17.9* and *35.3.2*. The Code has been written in such a way as to permit a single analysis for limit state of collapse as well as limit state of serviceability and for working stress method.

c) An envelope for the bending moment covering both cases of sub-para (a) of the clause should be drawn and then the member proportioned for the maximum values. While proportioning the member, redistribution of moments (*see 21.7, 36.1.1* and *43.2*) can be carried out.

d) The load combinations given in para (a) of the clause will give, maximum moment at support (when adjacent spans are loaded) and at mid span (when alternate spans are loaded).

e) The arrangement of live load is normally applicable to buildings. The same is not applicable to frames in liquid storage structures [*see* IS : 3370 (Part II)-1965*].

*Code of practice for concrete structures for the storage of liquids: Part II Reinforced concrete structures.

f) It is clear from the note given in this Clause that the arrangement suggested in *21.4.1* (a) is also applicable to the design of continuous beam supported over columns which may not be monolithic with the beam.

*21.4.2* SUBSTITUTE FRAME—The points to be noted under this clause are:

a) This idealization does not apply for analysing the structure when subjected to lateral loads and is applicable for the purpose of analysis under gravity loads (*see* Fig. E-6).

e) The arrangement of load to produce the most severe effect at any point in the structure theoretically requires consideration of the whole frame. This will not be necessary as the frame can be considered by breaking down into sub-frames; the sub-frames permitted by the Clause are illustrated in Fig. E-6.

f) A two-cycle moment distribution method provides a convenient means for determining moments and shears under these provisions.

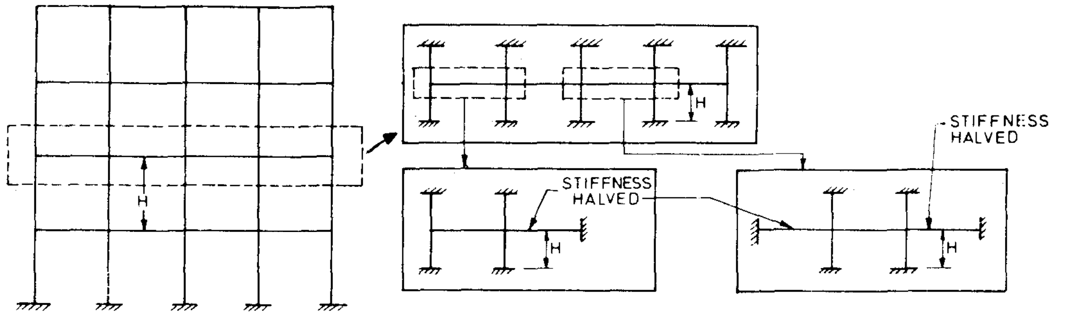



*Fig. E-6 Permissible Simplification of Frame for Analysis*

b) In the 1964 version of the Code, it was permissible to compute the bending moments in the beams of a frame by assuming that the beams are continuous over supports and capable of free rotation at the supports. This assumption will be approximately correct for analysing the effects of vertical loads but is now not permitted.

c) The analysis indicated in the Code constitute a 'first order' method since it does not take into account the effects of deflections and axial deformations that may arise which are generally negligible. However, in case of slender columns, moments in beams and columns must be amplified by additional moment for column as given in *38.7.1*.

d) If flat slabs are designed by the equivalent frame method, and if they are subjected to gravity loads only, a further approximation to the substitute frame will be possible [*see* para (b) of *30.5.1*].

*21.4.3* Considerable uncertainty prevails regarding the magnitude as well as the distribution of winds and earthquakes forces. Therefore, it will be sufficient, in most cases, to use an approximate method which gives an accuracy which is greater than that of the load data and other assumptions. In the analysis for lateral loads, simplified methods such as portal method, may be used to obtain moments, shears and reactions for structures that are symmetrical and satisfy the assumptions used for such simplified methods. For unsymmetrical or very tall structures, more rigorous methods should be used. The portal method is based on the following assumptions:

a) The total horizontal shear in all columns of a given storey is equal and opposite to the sum of all horizontal loads acting above that storey.

b) The horizontal shear is the same in both exterior columns; the horizontal shear in each interior column is twice that of an exterior column.

c) The inflection points of all columns

and beams (girders) are located midway between the joints. (However, the inflection points of the columns in the bottom storey are sometime assumed at 1/3 height from the bottom if they are supported on isolated footings, that is, on supports approaching hinged conditions).

Through these simplifying assumptions, the determination of the forces is reduced to the problem of statics only.

Unsymmetrical structures and frames with mezzanine floors are not amenable to this method of analysis. By 'very tall structure' probably the Code means those having more than twenty storeys. More rigorous method of analysis, recommended for unsymmetrical or very tall structure, are slope deflection method (Kani's method), moment distribution method or their equivalent, considering the effects of side sway.

Therefore, the Code does not permit further moment redistribution when Table 7 is used. It is to be noted that these tables are not applicable to beams carrying two-way slabs as load distribution in them is not uniform.

*21.5.2* BEAMS AND SLABS OVER FREE END SUPPORT — Only partial restraint may develop when the end support of the beam is built into a masonry wall. Depending on the degree of fixity the moment may vary over a range, say $\frac{Wl}{120}$ *to* $\frac{Wl}{12}$. In normal cases it should be sufficient to assume that this negative bending moment is $\frac{Wl}{24}$. Adjustment to the bending moments in the remaining portions of the beam may be made by using the coefficients given in Fig. E-7 and superposing these over the moments obtained from Table 7.

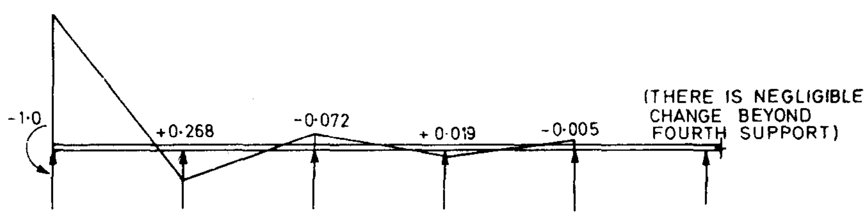



*Fig. E-7 Moments at Interior Supports Due to Restraint at Support*

For flat slab structures, analysis by 'Portal method' is probably not applicable (*see also* comment on *30.5.1*).

*21.5 Moment and Shear Coefficients for Continuous Beams*

*21.5.1* Tables 7 and 8 give the coefficients close to those which would be obtained from accurate analysis of an infinite number of equal spans on point supports. The coefficients will apply mainly to one-way slabs and to secondary beams at fairly close spacing, that is when the loading may be considered as uniformly distributed along their length. The moment coefficients given in Table 7 correspond roughly to those applicable to the worst conditions for equal spans, and the concession for using these coefficients with spans which differ by 15 percent is given by taking advantage of moment redistribution.

*21.6 Critical Sections for Moment and Shear*—Clause *21.2* (a) of the Code permits the representation of a frame by a simple line diagram, based dimensionally on the centreline distance between columns (and by extension, between floor beams). Actually, the sectional dimension of the beams and columns amount to sizable fraction of their respective lengths. The moment diagram for the continuous beams is usually quite steep in the region of the support and there will be a substantial difference between support centre-line moment and the moment at support face. If the former were used in proportioning the member, an unnecessarily large section would result. Therefore, the beam can be designed for the moment at the support face, thereby accounting for the finite width of the supports (*see* Fig. E-8).

However, in the case of columns the moment curve is not very steep, so that the

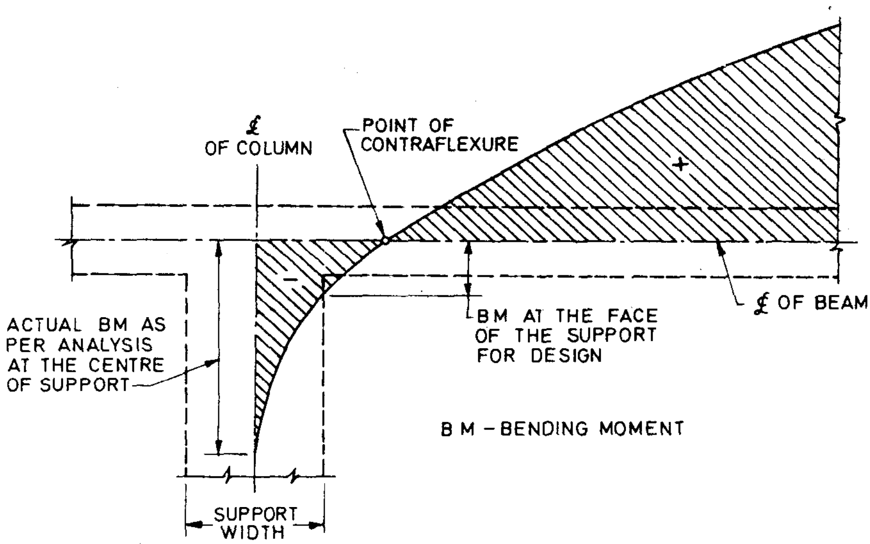



*Fig. E-8 Bending Moment at Face of Support*

differences between the centre-line moment and the moment at the face (top or bottom) of the beam is small and can be ignored.

For non-monolithic construction, obviously, such reductions are not possible at the support. The design will begin with the proportioning of the section in the mid-span regions.

*21.6.2* CRITICAL SECTION FOR SHEAR

*21.6.2.1* Fig. 1A shows an example where the support reaction does not include compression in the end regions. A diagonal (shear) crack is likely to start at the face and therefore, the latter is the critical section. This type of situations are expected in rectangular water tanks. For this case shear within the connection should also be investigated and special corner reinforcement should be provided.

In Fig. 1B, the reaction introduces compression in the end region which has a beneficial effect of displacing the diagonal crack away from the face of the support. Therefore, the Code permits the support sections to be designed for shear computed at a distance '*d*' away from the support.

The critical section for shear in flat slabs should be taken at a distance *d*/2 from the face of the column (*see 30.6.1*). For slabs carrying concentrated loads and for footings, additional considerations given in *33.2.4.1* will be relevant (*see also* Fig. E-9 for critical sections in typical support conditions).

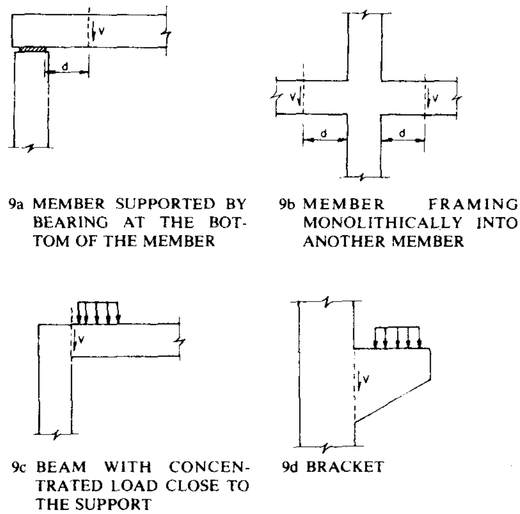



*Fig. E-9 Typical Support Conditions for Loading Critical Sections for Shear*

*21.7 Redistribution of Moments*—A treatment of the theory underlying the concept of moment redistribution can be found in many text books. (For example, Ref 22) (*see also* comments on *36.1.1*).

Redistribution of moments is applicable to continuous beams and beams in monolithic frames. Usually columns are precluded from the application of this principle, and therefore, only beams are considered.

In continuous beams, the support moments are critical and this fact, along with the presence of bars in other members framing into the support, may lead to undesirable congestion of reinforcement in the support region. The concession given in *21.6.1* to reduce the calculated support moment to the value at the face of the support somewhat mitigates the problem, but further advantage can be gained from the redistribution of moments that occur before the complete collapse of a span in continuous beams as a result of inelastic deformations.

In the section on working stress method (*see 43.2*) a redistribution of 15 percent of the maximum moment in the beam is permitted as was done in 1964 version of the Code with no special or additional requirements. However, in the section on limit state design (*see 36.1.1*) moment redistribution up to 30 percent is permitted, but restrictions are placed on the requirements concerning ductility and detailing.

In flat slabs, redistribution of moments is indirectly taken into account through *30.4.3.4* (for direct design method) and *30.5.2.3* (for equivalent frame method). Therefore, *36.1.1* or *43.2* should not be applied to flat slabs in view of the indirect provisions.

In continuous beams with overhanging spans, the moment at exterior support cannot be altered (for redistribution of moments).

## *22.* BEAMS

*22.0 Effective Depth*— In the case of deep beams [*see 28.3.1* (c)], the tension reinforcement is distributed over considerable depth and the basis of design itself is different. Therefore, the definition of 'effective depth' given in this Clause does not apply to deep beams.

### *22.1 T-Beams and L-Beams*

*22.1.1* General — The effective bonding of the web and the slab, by methods other than integral casting, is with reference to composite construction. IS : 3935-1966* gives the details on the methods for effective bonding of the web and the slab. In essence it recommends that the composite structures, in which the *in-situ* concrete is assumed to act integrally with the precast beam, should be inter-connected, in order to transfer the horizontal shear along the contact surfaces and to prevent the vertical separation of these units. Transfer of shear should be by shear connectors (bars), castellation and by bond. Also, the units (that is, slab and web) should be tied together by the extension of web reinforcement. IS : 3935-1966* gives the effective width of flanges of composite beams, which are different from those given in *22.1.2*.

*22.1.2* Effective Width of Flange — A more accurate determination can be made by using the CEB/FIP Recommendation (Ref 7). However, for design office use, the values suggested will be adequate since the stresses in flanges are seldom critical in design.

A single T-beam with a constant flange width (discontinuous at the sides) forms an isolated T-beam.

*22.2 Control of Deflection*— Reinforced concrete members subject to bending shall be designed to have adequate stiffness to limit deflections at service loads.

The allowable limits of deflection are set with respect to:

a) The final deflection (including the effects of temperature, creep and shrinkage) measured below the *as-cast* level of floors, roofs and other horizontal members $\not> \frac{\text{Span}}{250}$. This limitation is based on crack limitation with which the Code is very much concerned and to avoid psychological upsetting of the occupants or affect the appearance of the structure.

b) Part of the deflection (including the effects of temperature, creep and shrinkage) that take place after the construction of partitions or application of finishes should not normally

*Code of practice for composite construction.

$\not> \frac{\text{Span}}{350}$ or 20 mm whichever is less. This limit is intended to avoid damage to partitions and finishes.

The deflection limits specified in para (a) and (b) of this Clause are to be regarded as reasonable but, higher deflections may be permitted provided that partitions and finishes can accommodate them and do not adversely affect the appearance or efficiency of the structure. However, the design must satisfy that deflections are not excessive having regard to the requirements of the particular structure.

Some of the points to be noted under this Clause are:

a) The serviceability requirement of deflection as given in this Clause and its sub-clauses is to be met irrespective of whether the working stress design method is used or limit state design method is used;

b) These provisions are applicable to rectangular beams/slabs of uniform cross section only;

c) The deflection of column members are not covered;

d) The provisions are concerned only with deflections which may occur at service loads; and

e) In the 1964 version of IS : 456, span/overall depth ratios were specified for controlling deflection. No further modifications were allowed for the type and the amount of steel provided, which necessitated the use of minimum dimensions for beams/slabs. The present Code is more rational because it allows any reasonable dimensions for a beam/slab with varying amount of reinforcement to control deflection.

f) However these provisions may not be complied for *CHAJJAS* and lintal projections.

*22.2.1* Explicit or direct computation of deflection (*see* Appendix B) is lengthy, laborious for normal building design. Moreover, the conditions of service loading and many other factors are not known with precision at design stage. Therefore, the Code recommends that the limiting deflection of $\frac{\text{Span}}{250}$ will be satisfactory, if members conform to limiting $\frac{\text{Span}}{\text{Effective depth}}$ ratios as obtained from various modification factors specified in para (a) to (d) of this Clause. These factors can easily be determined at the design stage. Therefore, the $\frac{\text{Span}}{\text{Effective depth}}$ ratios will be used in majority of the cases and explicit calculation of deflection will be carried out in special cases such as:

a) When the designer wishes to exceed the span/effective depth ratio;

b) Where particularly stringent deflection control is required; and

c) Where the structure is abnormal (due to nature of loading or behaviour).

A brief discussion of the principles involved in arriving at the basic values of span to effective depth ratios is given below.

Consider a fully elastic, simply supported rectangular beam supporting a uniformly distributed load $w$ per unit length. If the permissible bending stress is $f$, the section can withstand a moment $M$ given by,

$$M = fZ = \frac{fbD^2}{6} = \frac{wl^2}{8} \qquad \ldots (1)$$

The deflection of the beam is,

$$\delta = \frac{5}{384}\left(\frac{wl^4}{EI}\right) \qquad \ldots (2)$$

From equation (1)

$$w = \frac{4}{3} fb\left(\frac{D}{l}\right)^2 \qquad \ldots (3)$$

Combining (2) and (3)

$$\frac{\delta}{l} = \frac{5}{24}\,\frac{f}{E}\left(\frac{l}{D}\right) \qquad \ldots (4)$$

Equation can be generalized for other types of end conditions and loads as:

$$\frac{\delta}{l} = K\left(\frac{l}{D}\right) \qquad \ldots (5)$$

Thus for a given elastic material, if the ratio $(l/D)$ is kept constant, the ratio of deflection to span will remain constant. By setting a limit to the ratio of span to depth, the deflection will be limited to a given fraction of the span. This is what is specified in the Code.

Span to depth ratios are satisfactory for controlling deflection as long as the material from which the beam is made is elastic. Unfortunately, the stiffness of reinforced concrete depends on several factors, such as steel percentage and state of cracking. Thus if span to depth ratios are to be used for reinforced concrete, some way of correcting for its actual behaviour has to be found. This is being corrected by the use of effective depth instead of overall depth and by the use of appropriate modification factors specified in the Code.

It is recommended that actual deflection calculation should be made for beams/slabs when the span exceeds 10 m length; even though the Code gives a modification factor.

MULTIPLICATION FACTOR FOR TENSION REINFORCEMENT — Deflection is influenced by the amount of tension reinforcement and the service stress, and the basic span to effective depth ratios are to be modified accordingly. The area of tension reinforcement is taken at the centre of the span for beams and slabs and at the support for a cantilever. The curves for different grades of steel specified in Fig. 3 are based on the assumption that:

$$f_s = 0.58 f_y$$

where

$f_s$ = service stress in steel, and

$f_y$ = characteristic strength of steel.

The multiplication factor is based on the empirical formula (Ref 23) given below:

Multiplication factor =

$$\frac{1}{0.225 + 0.003\ \ 22 f_s - 0.625 \log_{10}\left(\dfrac{bd}{100\ A_s}\right)}$$

The curves in Fig. 3 have been plotted for particular values of $f_s$ (N/mm²) $\left(\dfrac{100\ A_s}{bd}\right)$.

If one wishes to use a shallow member, the deflection can be kept within the required limit by providing more tension reinforcement than that is required from strength consideration or working stress considerations, thereby reducing the service stress in steel. The multiplier can then be arrived at by using the formula given above. However, while increasing the amount of tension steel to decrease deflection, the limitation of *37.1(f)* should be kept in view in case of sections proportioned for limit state of collapse.

MULTIPLICATION FACTOR FOR COMPRESSION REINFORCEMENT — The effect of compression reinforcement is to reduce shrinkage and creep effects thereby reducing long-term deflection. As the concrete dries out it tends to shrink, but this effect is partially restrained in the vicinity of compression reinforcement. The multiplication factor for compression reinforcement is based on the following formula (Ref 23):

$$\left.\begin{matrix}\text{Multiplying}\\ \text{factor}\end{matrix}\right\} = \frac{1.6 p_c}{p_c + 0.275}$$

where

$$p_c = \frac{100\ A'_s}{bd} \quad \left(= \begin{matrix}\text{percentage of compres-}\\ \text{sion reinforcement}\end{matrix}\right)$$

Increase in percentage of compression reinforcement is the best method to control deflection in critical cases without decreasing the strain in tension steel beyond a certain limit at collapse load as required by *37.1(f)* of the Code.

FLANGED BEAMS — The method given for flanged beams may sometimes give anomolous results. If the flanges are ignored and the beam is considered as a rectangular section, the value of span to effective depth ratio thus obtained (percentage of steel being based on $b_w d$) should always be on the safer side.

*22.3 Slenderness Limits for Beams to Ensure Lateral Stability* — These rules preclude failure by sideways bending and buckling. The recommendations of this Clause are based on a simplification (Ref 24) of the classical solution for lateral buckling of beams.

Lateral restraint, if required, should normally be provided by construction attached to the compression zone of the beam. In the case of parapet beams lateral restraint may be assumed to be provided by slabs attached to the tension zone of the beam, but the slab thickness should be at least one-tenth of the beam depth and the beam should not project above the slab more than ten times the width of the beam (Ref 21).

## *23.* SOLID SLABS

*23.1 General* — Rules for the design of ribbed, hollow block and voided slabs are given in *29*. Design of flat slab structures is dealt in *30*.

For applying para (c) of *22.2.1* to the design of two-way slabs, the reinforcement at mid span, in the direction of the shorter span should be used. In the case of slabs spanning in one direction only the reinforcement at the mid span is to be considered.

*See also 25.3.2* for spacing of reinforcements in slabs; para (d) of *25.4.1* for cover to steel in slabs and *25.5.2* for minimum reinforcement.

*23.2 Slabs Continuous Over Supports*—From a comparison with the 1964 version, it appears that this Clause is intended for slabs spanning in one direction. Two-way slabs with one or more edges continuous over supports should be designed by using Appendix C of the Code.

*23.3 Slabs Monolithic with Supports*

*23.3.1* The intension of this Clause is mainly to assume that beams supporting monolithic slabs are rigid and do not deform in relation to slabs.

*23.3.2* SLABS CARRYING CONCENTRATED LOAD—The effective width of slab due to concentrated load is illustrated in Fig. E-10.

If the edge of the slab is within $\frac{1}{2}b_{ef}$ from the centre of the load point, then effective width is to be measured from the edge of the slab (*see* Fig. E-10).

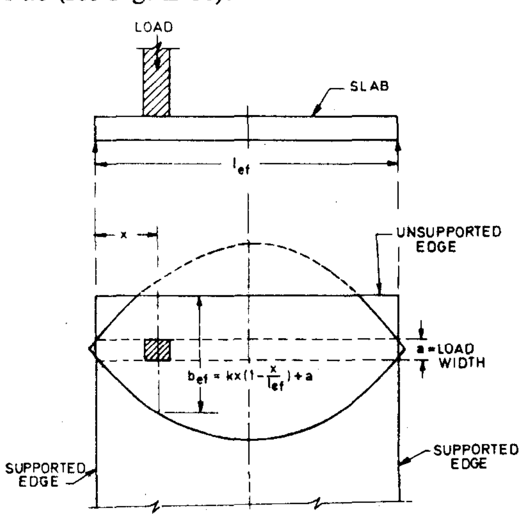



*Fig. E-10 Effective Width of One Way Solid Slab Carrying a Concentrated Load Near an Unsupported Edge*

Where the effective width of slab assumed to resist the concentrated load is less than the actual width of the slab, allowance shall be made for bending moments due to concentrated loads using methods based on elastic theory such as Pigeaud's or Westergaard's or other acceptable method. Alternately, Johanson's yield-line theory or Hillerborg's strip theory based on limit state of collapse may also be used.

*23.3.2.1* The effective width for solid slabs (supported on two opposite sides) is based on elastic theory.

*23.3.2.2* Slabs other than solid slabs include ribbed, hollow block or voided slabs covered in *29* of the Code.

*23.3.2.3* —

*23.3.2.4* —

*23.4 Slabs Spanning in Two Directions at Right Angles*—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 Pigeaud's and Westergaard's theory. Alternately, collapse theory such as Johanson's yield-line theory or Hillerborg's strip method may be used provided that the ratio between support and span moments is similar to those obtained by elastic theory; values between 1.0 and 1.5 are recommended.

The Code gives coefficients for bending moments for simply supported slabs and restrained slabs spanning in two directions and carrying uniformly distributed load in *Appendix B*. For 'yield line method' and 'stip method' of analysis *see* Ref 25 and 26. Pigeaud's method and Westergaard's method mentioned in the Note are suitable for determining the bending moments due to concentrated loads. Details are given in Ref 27.

*23.5 Loads on Supporting Beams*—The method of estimating the loads carried by the beams is based on an approximate yield-line pattern in which the slab may fail. The total loads on the short and long spans due to one loaded panel are given by:

$$\text{load on short beam} = \frac{wl_x^2}{4}$$

$$\text{load on long beam} = \frac{wl_x l_y}{2} - \frac{wl_x^2}{4}$$

where

$l_x$ = length of short span
$l_y$ = length of long span
$w$ = load per unit area

The corresponding bending moments in the beams may be determined with sufficient accuracy by assuming that the loading is equivalent to a uniform load per unit length of the beam of the following amounts:

On the short span, $\frac{wl_x}{3}$

On the long span, $\frac{wl_x}{6}\left[3 - \left(\frac{l_x}{l_y}\right)^2\right]$

However, the shearing forces should be determined from the load distribution recommended in Fig. 6.

## *24.* COMPRESSION MEMBERS

*24.1 Definitions*—This definition is applicable to rectangular and circular sections only and is not applicable to I-section and channel sections. When the length of the compression member is less than three times the least lateral dimension it is called a pedestal (Ref 28) Clause *45.1* of the Code says that reinforced concrete pedestals can be designed as short columns.

Pedestals can be of plain concrete also (*see 33.1.3*).

*24.1.1* The upper limit of 12 on the ratio of effective length to least lateral dimension, assures that the secondary effects of loads will be negligible in short columns. Applying this limit in the expression for additional moments due to slenderness effects (*38.7.1*), it is seen that the column may have to be designed for an additional eccentricity $e$.

$$e = \frac{M_{ax}}{P} = \frac{D}{2\,000}(12)^2 = 0.072D$$

which is in someway similar to that assumed in *38.3* of the Code.

*24.1.2* —

*24.1.3* UNSUPPORTED LENGTH

*24.2 Effective Length of Compression Members*—The Code requires the use of 'effective length' for computing slenderness effect (*see 38.7.1* and *45.3* of the Code).

*24.3 Slenderness Limits for Columns*—Slenderness limits are introduced to prevent the development of significant torsional deformation and to rule out the possibility of failure by lateral torsional buckling under bending only. Columns outside the limits become, because of the excessive additional moments which must be considered in design, very inefficient.

*24.3.1* —

*24.3.2* These limits are similar to those given for beams in *22.3.* Definition of unsupported length is given in *24.2.1*

*24.4 Minimum Eccentricity*—All concrete columns are subject to some moment due to eccentricity of loading which may be due to the following:

a) Inaccuracies in construction;
b) Lateral deflection of column; and
c) Inaccuracies in loading, etc.

Therefore, the Code requires all columns to be designed for a minimum eccentricity of load even if the computed moment is less.

The Code treats, in most Clauses, only rectangular columns and minimum eccentricity for other shapes of cross section is not given. In the absence of any other guideline, a value of $\frac{l_{ef}}{300}$ is suggested as a simple expedient (*see* German Code DIN 1045).

## *25.* REQUIREMENTS GOVERNING REINFORCEMENT AND DETAILING

*25.0* For detailed information regarding reinforcement detailing reference may be made to handbook on reinforcement detailing (*under preparation*).

*25.1 General*—Different types of reinforcing bars, such as plain bars and deformed bars of various grades say Fe 415 (N/mm²) and Fe 500 (N/mm²), should not be used side by side as this practice will lead to confusion and errors at site. However, secondary reinforcement such as ties and stirrups, may be of mild steel throughout even though the main steel may be of high strength deformed bars.

*25.1.1* Detailing requirements for bundled bars are covered in *25.2.1.2, 25.2.3.5, 25.2.5.1(g)* and *25.3.1(b)* of the Code.

Bundling of bars larger than 36 mm especially in flexural members may lead to unsatisfactory crack widths. However, such bundles do not adversely effect the crack width in columns.

*25.1.2* IS : 4326-1976* makes it obligatory that in all cases where the design seismic coefficient is ≥ 0.05, greater ductility provisions as specified therein shall be followed considering the following points:

a) Reversal of stresses in beams and columns should be adequately estimated and provided for; an example being the provision of closed stirrups in beams. Columns should be designed to take up the shear caused by lateral forces due to earthquake.

b) Continuity of construction should be ensured by adopting monolithic beam-column joints and by continuing sufficient number of bars beyond the joints.

c) Ductile behaviour of structural members should be ensured by providing the minimum reinforcements, through confinement of concrete especially in exterior columns (near the joints), and by precluding shear failures in beams as well as in columns.

*25.2 Development of Stress in Reinforcement*—This forms a common Clause to both methods of design, namely, limit state method and working stress method. The check for bond stress specified in the earlier Code is now replaced by the concept of development length. The development length may be defined as the length of the bar required on either side of the section to develop the required stress in steel at that section.

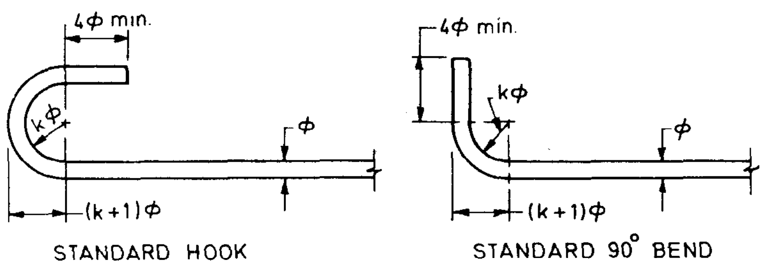
*25.2.1* Development Length of Bars—In the formula for development length, the appropriate values for $\sigma_s$ and $\tau_{bd}$ are to be used depending upon the method of design adopted. If the amount of steel provided at a section is more than that required from design considerations, the development length may be modified as:

$$L_{dm} = \left(\frac{A_s\text{, required}}{A_s\text{, provided}}\right) L_d$$

The values of development length obtained for the two methods of design may not coincide but will be approximately equal.

Development length of a bend/hook bar is equal to the length of the reinforcement between the point of tangency of the bend/hook and the critical section plus the anchorage value of bend/hook. The anchorage values of standard hooks and bends (*see* Fig. E-11) are specified in *25.2.2.1.*

*25.2.1.1* Extra development length is required in case of bundled bars than that required for an individual bar because the grouping makes it more difficult to mobilize bond resistance from the core between the bars.




NOTE MINIMUM VALUE OF k FOR MILD STEEL IS 2 AND FOR COLD WORKED STEEL IS 4

*Fig. E-11 Standard Hook and Bend*

*25.2.2* Anchoring Reinforcing Bars

*25.2.2.1* Any deficiency in the required development length can be made up by anchoring the reinforcing bars suitably. Deformed bars have superior bond properties owing to mechanical bearing and, therefore, provision of hooks is not

*Code of practice for earthquake resistant design and construction of buildings (*first revision*).

absolutely essential. However, plain bars should preferably end in hooks, as there may be some uncertainty regarding the full mobilization of bond strength through adhesion and friction.

*25.2.2.2* ANCHORING BARS IN COMPRESSION — While considering the development length in compression zone, the projected length of hooks, bends and straight lengths beyond the bends should be considered and not as per mentioned in *25.2.2.1.*

*25.2.2.3* MECHANICAL DEVICES FOR ANCHORAGES — The following information is of relevance to this Clause (*see* German Code DIN 1045): Anchorages formed with the aid of attachments should, if possible, be located only at or in the immediate vicinity of the end face of a structural component. The permissible loading on anchorage attachments should be verified by calculation or by means of tests.

For the purpose of verification by calculation, a factor of 0.80 must be applied to the ultimate strength of the anchorage. The bearing pressures excerted on the concrete at the anchorage surface must not exceed the values allowed under partial area loading (*see 33.4*).

If the capacity of anchorage attachments for use with static loads is determined by means of tests, the average of three test results may be adopted. The value to be adopted for permissible load under working condition must not exceed half the average values of the failure load.

If welded-on transverse bars are used as anchorage attachments (*see* comments on *25.2.4*), the capacity should be determined experimentally on test specimens. The amount of slip occuring at the non-loaded end of the bar (average value) should not exceed 0.01 mm under the working load envisaged and should not exceed 0.1 mm under 1.75 times the working load. In addition, the shearing strength of welded connections should be determined in comparison with specimen not embeded in concrete.

*25.2.2.4* ANCHORING SHEAR REINFORCEMENT

a) *Inclined bars* — The rules given in this Clause are illustrated by Fig. E-12 and Fig. E-13.

b) Thin concrete cover over the 90° hook in a stirrup may lead to spalling of cover concrete as the 90° hook has a tendency to straighten out under overloads (*see* Fig. E-14). To avoid this type of failure, it is suggested that the cover be at least twice the diameter of the stirrup bar. Where this is impracticable, the hook should have a 135° bend (Ref 21).

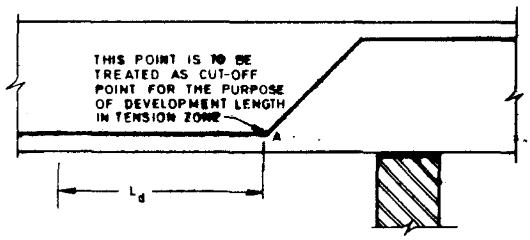



*Fig. E-12 Anchoring Inclined Bent-Up Bar in Tension Zone*

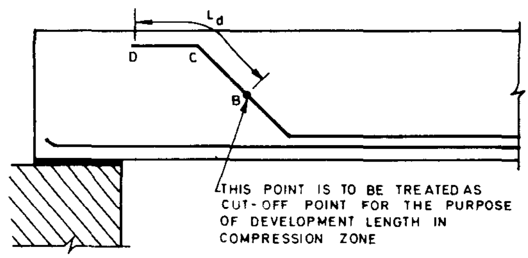



*Fig. E-13 Anchoring Inclined Bent-Up Bar in Compression Zone*

*25.2.2.5* BEARING STRESSES AT BENDS — Standard hooks and bends are those which conform to the dimensions given in

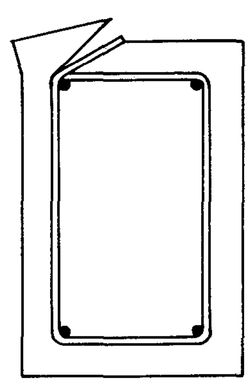


*Fig. E-14 Spalling of Thin Cover Over Stirrups*

IS : 2502-1963* (*see* Fig. E-12).

While considering the first two sentences of the Clause, three cases may be distinguished:

a) A bend or hook, which is terminated at 4 times the bar size beyond the end of the bend;

b) A bend, in which it is assumed (and provided in design) that the bar is not stressed at portions beyond a distance of 4 times the bar size beyond the end of the bend; and

c) A bend or hook, in which the bar is continuous beyond the end of the bend and is also stressed along the length in the continued portions (*see* Fig. E-15).

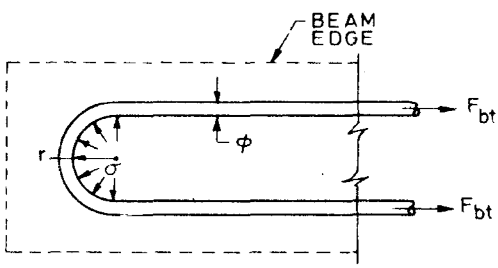



*Fig. E-15 Plan-Bearing Stress at Bends*

If the radius of the bend or hook, conforms to that of the standard one, cases (a) and (b) above will need no further consideration, as experience shows that the arrangement will be satisfactory. However, in (c), whether the radius of the bend (or hook) corresponds to that of the standard one or not, a check on the bearing stress of concrete within the bend will be required.

Referring to Fig. E-15, let $F_{bt}$ be the tensile force in the steel bar at the start of the bend with a radius '*r*' and let the internal compressive stress (radial) required to resist this force be $\sigma$. Resolving forces along the direction of $F_{bt}$:

$$2F_{bt} = 2\,\sigma\, r\, \phi$$

$$\text{or } \sigma = \frac{F_{bt}}{r\phi}$$

The concrete within a bend is subjected to a triaxial stress field and, therefore, can withstand high stresses locally. Accordingly, the bearing stress $\sigma$ is allowed to reach high magnitudes. Empirically arrived allowable values are given in the Code for both methods of design.

*25.2.2.6* In structural components with curved or angled soffits, or those formed with bends or corners, it should be ensured that the radial tensile forces due to changes in direction of the reinforcement are resisted by additional links. A specific application of this requirement is specified in *25.5.3.3.* Three typical cases are illustrated in Fig. E-16.

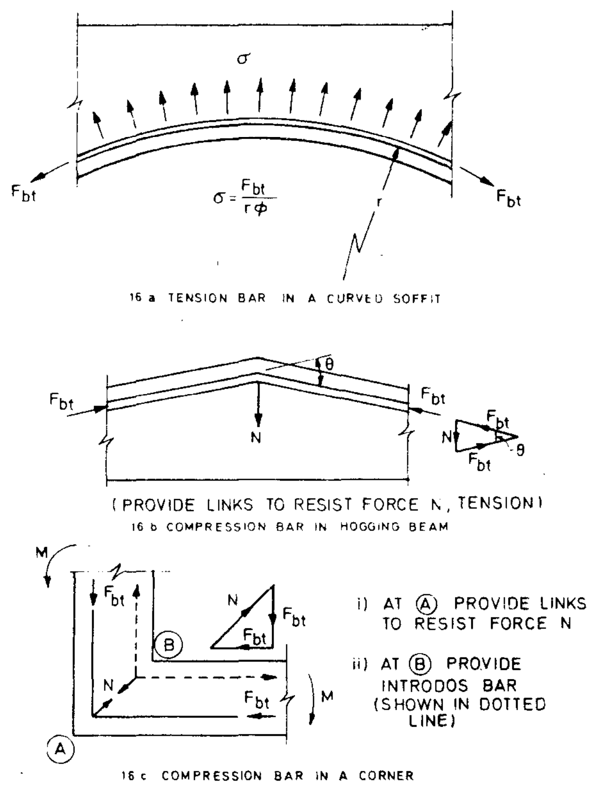



*Fig. E-16 Radial Forces in Reinforcement*

*25.2.3* CURTAILMENT OF TENSION REINFORCEMENT IN FLEXURAL MEMBERS—Since the publication of the 1964 version of the Code, experimental evidence has become available which indicates that the bar curtailment, especially when deformed reinforcements are used, may adversely affect the shear strength of beams. Curtailment rules given in the Code take into account this fact. (For curtailment of bars in slabs, *see Appendix C*).

*25.2.3.1* Rules for anchoring the bars at

*Code of practice for bending and fixing of bars for concrete reinforcement.

simple supports are given in *25.2.3.3*.

The Code prohibits the cutting-off of a bar at the theoretically determined point for a number of reasons:

a) The bar should extend to a distance further from the theoretical cut-off point, in order to avoid stress concentration which may lead to moment cracks even at working loads. These cracks may well be of above average size which may locally reduce the shear strength.

b) The recommended extension provides for slight deviations in inaccuracies in the analysis. For example, the loading may well not be absolutely uniformly distributed, in which case the shape of the bending moment diagram will be different from that assumed.

c) In the presence of shear, diagonal cracks may form which in the absence of stirrups, will cause the longitudinal steel stress to be that corresponding to the moment at a section roughly an effective depth closer to the supports. Referring to Fig. E-17, it can be seen that the force in the steel bar at E is not proportional to the moment at E, but will be proportional to a higher moment that exists at F.

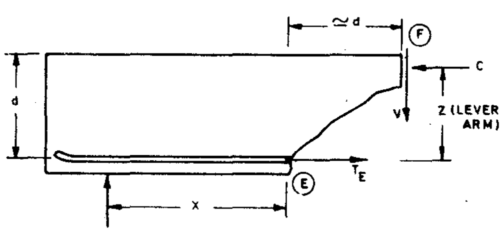



*Fig. E-17 Bars Stopped at Theoretical Cut-Off Point (E)*

*25.2.3.2* Termination of flexural tensile reinforcement gives rise to a sharp discontinuity in the steel, causing early opening of flexural cracks which in turn may change into a diagonal crack prematurely. Fulfilment of any one of the conditions of the Clause will mean that either the shear stresses are kept low (para a), or extra shear reinforcement is provided (para b) or excess flexural steel is available along with excess shear capacity (para c).

Application of para (c) is straight forward when the diameter of main steel bars is equal to or less than 36 mm. If bars larger than 36 mm diameter are used, they cannot be curtailed by involving para (c); rather paras (a) and (b) of the Clause will apply. In the rare case when paras (a) or (b) cannot be satisfied and also the bars are greater than 36 mm diameter, no curtailment will be possible.

*25.2.3.3* POSITIVE MOMENT REINFORCEMENT— Regarding para (a): Extension by $L_d/3$ is not applicable to all cases. For exception, *see* para (b) of this Clause and also *28.3.1* (b). Requirements in case of slabs are also different (*see* Appendix C and also *30.7.3*).

Specified amounts of the positive moment reinforcement are required to be carried into the supports to provide for some shifting of the moment diagrams due to changes in loading, settlement of supports and other causes.

Regarding para (b): When a flexural member (beam) is part of a primary lateral load resisting system (that is, when the frame is unbraced) loads greater than those anticipated in design may cause reversal of moment at supports. Full anchorage of the indicated amount of positive moment reinforcement assures ductility of response in the event of serious overstress, such as from earthquakes.

The required proportion of reinforcement should be anchored so as to develop the design stress in full. This implies that: (a) it is not enough if more reinforcement is provided at lower stress levels; (b) the design stress is 0.87 $f_y$ for limit state metho and the allowable stress for the working stress method; and (c) the full anchorage requirement will not apply to any excess reinforcement (over that required) provided at the face of the support.

Regarding para (c): Requirements indicated in this para are equivalent to the flexural (local) bond check of earlier version of IS : 456, as shown later. The provisions have been recast in terms of development length:

a) *25.2* of the Code says that:
'The calculated tension or compression in any bar at any section shall be

developed on each side of the section by an appropriate development length or end anchorage or by a combination thereof.' Now, consider the case of a uniformly loaded simply supported beam, proportioned in such a way that the design stress, say 0.87 $f_y$, is developed at the mid section, which is required to carry a moment $M_u$ (*see* Fig. E-18). However, at quarter-span, that is, at 1/4 from the support, the bar would develop only half the design stress but would have to cater for 0.75 $M_u$, because the bending moment diagram is a parabola. Clearly, *25.2.1* cannot be satisfied at the quarter-span, merely through assuring the development of stress at mid-span. Nor will it be enough to ensure a development length for 0.75 $M_u$ at quarter-span, because it can be shown that variation will still occur by considering a section $L/8$ away from the support.

b) Extending the argument given above, and considering an infinitesimal length of the beam very near the support it is apparent that if the development length requirement is satisfied at the point (*see* Fig. E-18), *25.2.1* will always be satisfied in the region to the left of P. The location of P is determined easily, by noting that $\frac{dM}{dx} = V$, or the slope of AB is $V$.

Therefore, P is at a distance $\frac{M_1}{V}$ from the support A. While computing $M_1$, it is necessary to calculate the moment of resistance of the section at $A$ (and not the externally imposed moment), assuming that the bars are stressed to the design value (that is, 0.87 $f_y$ in limit state method and the permissible stress in the working stress method). At point P, the development of stress should be assured. That is,

AP > Development length

$$L_d \ngtr \frac{M_1}{V} \qquad (1)$$

However, the bar may extend beyond A (towards the left), to Q and the 'anchorage' $L_o$ may be counted for development, but not

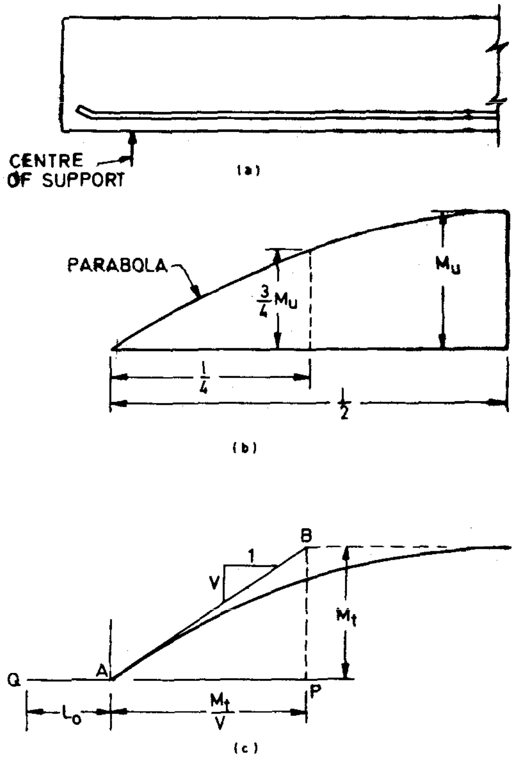



*Fig. E-18 Development Rule for Section at Support*

to an indefinite extent. For point of inflexion, the Code limits $L_o$ to effective depth or 12 times the diameter of bar, whichever is greater. Accordingly,

$$L_d - L_o \ngtr \frac{M_1}{V}$$

$$\text{or } L_d \ngtr \frac{M_1}{V} + L_o \qquad (2)$$

Which is the form given in the Code. Whenever inequality is obtained in (2), the following three options may be adopted:

1) Bring in more bars to the support A, so that $M_1$ is increased. This is not always possible without increasing the reinforcement cost.
2) Increase $L_o$, but this cannot be done indefinitely, owing to the upper limit on $L_o$.
3) Obviously, the best course is to reduce the diameter of the bar, so that $L_d$ will be reduced and the same is recommended in the Code.

Extension of this requirement to points of inflexion is obvious as similar conditions exist there for bending moment as that at simple supports.

*25.2.3.4* NEGATIVE MOMENT REINFORCEMENT — Clauses *25.2.3.1*, *25.2.3.3*(a) and *25.2.3.5* follow the same general principles. Here anchoring of one-third of the total reinforcement for a distance of effective depth or one-sixteenth clear span, provides for the shifting of bending moment diagram, owing to change in loading, settlement of foundations and other unanticipated causes.

At the face of the support for a continuous beam, it should be ensured that the required development length is available for the remaining two-thirds of the bars (*see 25.2.1*).

Note that the Code does not insist on the condition that $L_d \ngtr \frac{M_1}{V} + L_0$ at the interior support of a continuous beam. The reason is that the shape of the bending moment diagram near the interior supports is such that the development length at any sections away from the support is always satisfied, if the requirements at the face of the support are fulfilled.

*25.2.3.5* CURTAILMENT OF BUNDLED BARS — Three conditions need consideration:

a) Anchoring of a bundle at a support;
b) Bundles terminating at support; and
c) Bundles terminating inside the span.

Anchoring of bundle at a support — *25.2.3.3*(a), and *25.2.3.4* will govern the anchoring requirements. While applying *25.2.3.3*(a), the development length must be increased according to *25.2.1.2*. In *25.2.3.4*, the diameter of the bundle should be taken as that of an equivalent bar of the total area of the bundle.

Termination of bundles inside the span or at the support — Note that two bars in contact do not constitute a bundle (*see 25.1*), and if desired, a pair of bars may be cut-off at the same point, but the perimeter reduction appropriate to a pair would then apply.

When all the bars in a bundle are carried to the support, the increased development length (based on single bar) for bundles (*see 25.2.1.2*) should be used for checking of the development of stress.

*25.2.4* SPECIAL MEMBERS — The stress in reinforcement does not decrease linearly in proportion to a decrease in moment, in the case of sloped or tapered footings, brackets and deep beams. The development length requirements need special consideration in such cases:

a) *Footings* — In the case of stepped footings, the development length and end anchorage requirement should be checked wherever there is an abrupt change of cross-section. In sloped footings, the additional checks must be carried out at a few intermediate points (*see also 33.2.4.3*).
b) *Brackets and corbels* — Referring to the bracket shown in Fig. E-19, the stress at ultimate loads in the reinforcement is almost constant (equal to yield stress) from the face of the support to the load point. In such cases, the stress development check will be critical at the load end itself. A welded cross-bar, of diameter equal to that of the main steel provides an effective end anchorage near the loaded end. An end hook (standard) in the vertical plane near the loaded end will not be effective. However, if the bracket is wide (perpendicular to the plane of figure) and if the load is not applied very near the corner, 180° hooks in the horizontal plane may provide adequate anchorage.
c) *Deep beams* — [*See 28.3.1*(b)].

*25.2.5* REINFORCEMENT SPLICING — Careful detailing is necessary when reinforcements are to be spliced. Therefore, location and details of splices should be determined at the design stage and shown in the detailing drawings. Preferably, splicing details should not be left to be decided at the site.

Note that the recommendations given in this Clause will not apply to bars in compression and also that they will not be adequate for main bars in concrete ties.

When splicing of more bars at critical sections cannot be avoided altogether, the Code recommends that special precautions should be taken, but it does not specify the extra lengths or details of spirals. The scheme

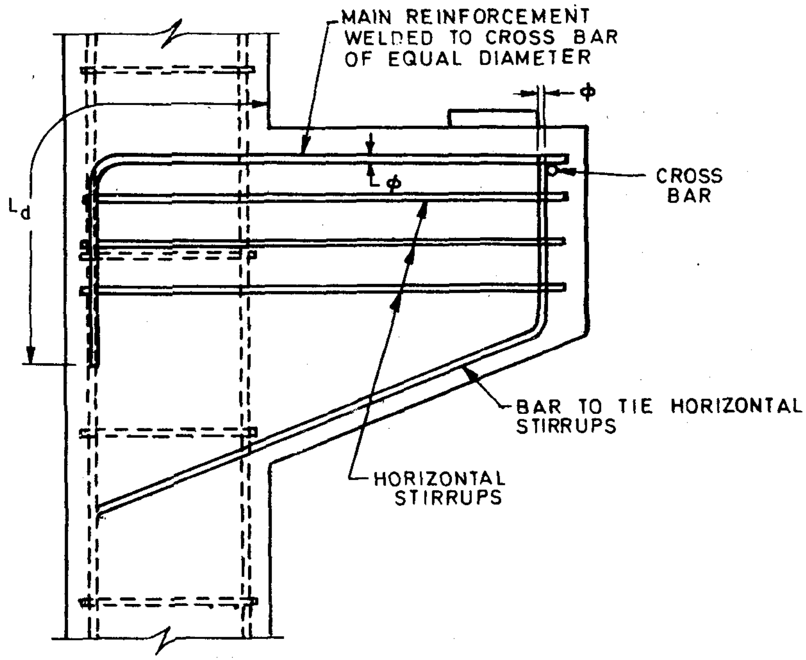



*Fig. E-19 A Typical Method of Anchoring Main Tension Reinforcement in Corbels*

shown in Fig. E-20 based on Ref 28, may be used in such situations.

The Code requires that the splices should be staggered. This requirement is applicable specially to bars which are loaded in tension, such as those in the tension zone of flexural members and in ties. In compression members, all the compression bars may be lapped at the same section (*see* Ref 7).

*See also 25.5.3.3* for splicing of bars (with off-set splice) in columns.

*25.2.5.1* LAP SPLICES

a) Lap splices are not recommended for

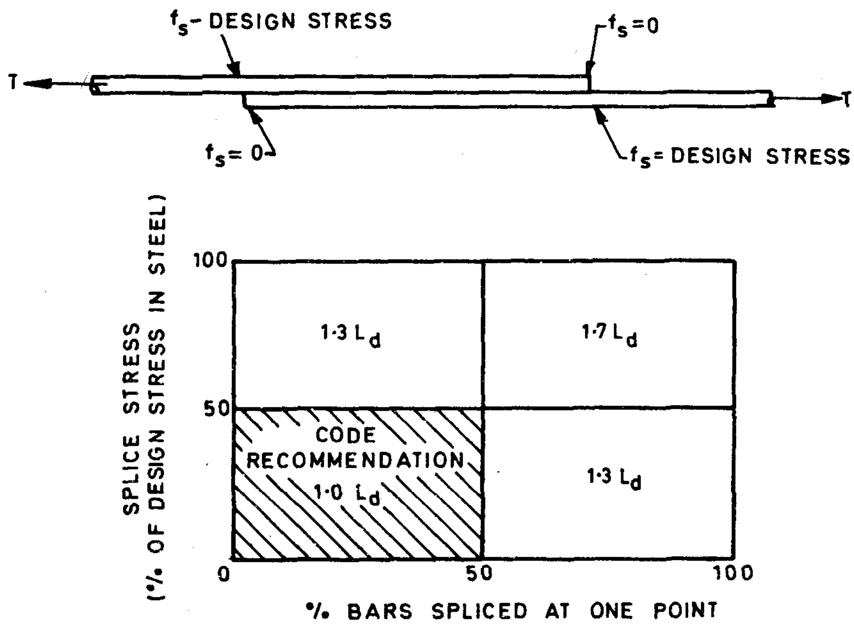



*Fig. E-20 Lap Splices for Flexural Tension Bars* large diameter bars (over 36 mm diameter), because of insufficient data on development length as well as on behaviour of bars at spliced locations. Lap splices for large diameter bars, if the splices are unavoidable, may be detailed in accordance with recommendations given in Ref 29.

b) *Staggering of lap splices*—The free ends of spliced bars being sources of discontinuity, act as crack initiators across a tension zone. This transverse crack in turn may develop into splitting cracks, along the axial direction of the bars, especially when deformed bars are used. When several highly stressed bars are terminated across the same cross-section, the splitting effects at their free ends are cumulative. It is beneficial to stagger lapped splices in such a way that the free ends do not line up at the same section. Therefore, the Code prefers staggered splice layouts, located away from sections of maximum tension.

c) *Lap lengths*—*See also* Fig. E-20 and comments on *25.2.5*, regarding the situations where increased lap lengths might become necessary.

With the use of plain bars in tension, provision of hooks at the ends of splices will be mandatory, in all cases [*see 25.2.2.1*(a)].

In concrete ties (concrete members carrying axial tension), the splices should invariably be provided with hooks where the bars are stopped, even in the case of deformed or ribbed bars. However, the lap lengths in bars in concrete ties are likely to be large and cumbersome to fabricate. In general, it is preferable to use welded connections or other positive mechanical connectors (*see 25.2.5.2*) for joining the bars in concrete ties.

While calculating the development lengths ($L_d$) for the purposes of determining the lap lengths in splices, the full strength or maximum permissible stress in bar should be used along with the appropriate bond stress indicated in *25.2.1*. No reductions in development length is permitted, even when the splices are located away from points of maximum stresses.

d) *Lap length in compression*—Lapped splices for compression bars need not be staggered. The lap length in compression is specified to be less than that in tension, because of the beneficial effects of transverse ties, the absence of transverse cracking at the ends of bars in compression regions, and the ability of the bars to transmit the compressive stresses by end bearing into the concrete.

e) *Bars of different diameter*—At the lap splice, the force to be transmitted is governed by the thinner bar and, therefore, it will be permissible to determine the lap length on the basis of the diameter of the smaller bar.

f) *Splices in welded wire fabric*—(*see* Fig. E-21).

g) *Splices of bundled bars*—(*see also 25.1.1* and *25.2.1.2*.).

While calculating the lap lengths, the development lengths should be increased to the values indicated in *25.2.1.2*. In the absence of additional information, the individual splices of bars within the bundle should be staggered by 1.3 times the increased lap lengths, as required in *25.2.5.1*(b).

*25.2.5.2* WELDED SPLICES AND MECHANICAL CONNECTIONS

Welded splices—(*See also 11.4* and its

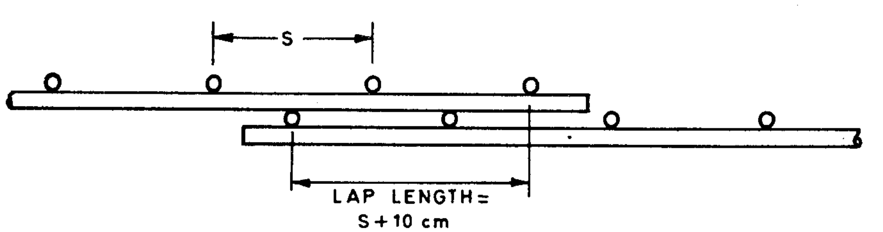



*Fig. E-21 Lap Splicing of Welded Wire Fabrics* comments.) Welded joints should not occur at bends in reinforcements. Where possible, joints in parallel bars of principal tensile reinforcement should be staggered in the longitudinal direction.

The design strength of the welded connection should be based on results of tests, which should prove that the strength of the weld is as strong as that of the parent bar.

According to IS : 2751-1979*, mild steel bars up to and including 20 mm diameter should be lap welded and those larger than 20 mm diameter should be butt welded. In the welds of a lapped joints, the shear strength of the filler material should be taken as 0.38 times its yield or proof stress as given in the appropriate standards. The length of weld should be sufficient to transmit the design load in the bar that is, the cross sectional area of (parent) bar × 0.87 $f_y$ should be equal to the effective lengths of weld × throat thickness × the shear strength of filler material. The length of a run of weld should not normally exceed five times the size of the bar. If a longer length of weld is required, it should be divided into sections and the space between runs made not less than five times the size of the bar (Ref 3).

Construction problems should be considered in the location of welded joints. It is the responsibility of the designer to give details of splicing requirements which include lengths as well as location.

*Mechanical connections*—The use of mechanical splices, for example by means of screws or clamps should be agreed to in advance. The soundness of the method to be used should be verified by means of tests.

*25.2.5.3* END-BEARING SPLICES—Force transfer by end bearing can be used only when the designer is certain that under the most adverse load combination the bars are never required to carry tension. Bar ends should terminate in flat outer faces within 1½ degrees of a right angle to the axis of the bars and should be fitted within 3 degrees of full bearing after assembly (*see* Fig. E-22). End bearing splices should be used only in members containing closed ties, closed stirrups or spirals (Ref 28).

The Code requirement regarding suitable devices, such as say a sleeve, to ensure concentric bearing of the ends is important.

*25.3 Spacing of Reinforcement*

*25.3.0* Clause *25.1.1* of the Code distinguishes between pairs of bars in contact and bundles of three or four bars grouped in contact. Some Codes (Ref 3) maintain this distinction while specifying the equivalent diameter. As the Code is silent on this issue, it is recommended that this rule be applied to pairs of bars in contact as well.

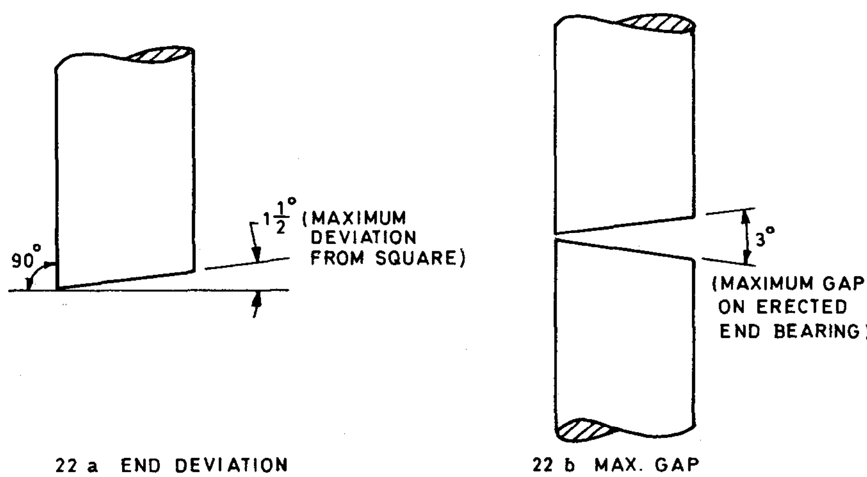



*Fig. E-22 End Bearing Splice*

*25.3.1* MINIMUM DISTANCE BETWEEN INDIVIDUAL BARS—Clause *4.2.4.1* which

*Code of practice for welding of mild steel plain and deformed bars reinforced concrete construction (*first revision*).

deals with maximum size of aggregates is partially complementary to *25.3.1*. It is convenient to fix the nominal maximum size of the aggregate first and then to detail the spacing between reinforcements.

Note that minimum spacing is measured clear between two adjacent bars, or between a single bar and an adjacent lap splice. The clear distance limitations betwen bars are based on successful practical consideration and remain essentially unchanged from that of the 1964 version. The minimum limits are intended to permit concrete to flow readily around the bars and to obtain adequate compaction.

Note that the entire Clause, speaks of only individual bars, and the mention of group in para (b) does not apply for bars bundled in contact or pair of bars in contact. As pairs and bundles are subject to high bond stresses, special precautions should be taken to ensure that there is adequate spacing between the pairs or bundles. The vertical spacing between bars is subjected to the same spacing as that of horizontal spacing subject to a minimum of 15 mm.

In this connection it is to be mentioned here that bundling of bars may be particularly useful in reducing congestion (*see* Fig. E-23).

*25.3.2* Maximum Distance Between Bars in Tension — Clause *34.3.2* suggests that, for structures exposed to mild or moderate environment (described in Table 19), the surface width of cracks should not exceed 0.3 mm. The maximum clear distances given in Table 10 are based on this requirement.

*25.4 Cover to Reinforcement* — The cover requirements specified herein are applicable to cast *in-situ* reinforced structures and are not applicable to precast construction. Lesser thickness than those specified herein may be permissible for precast construction.

*25.4.1* to *25.4.2.3* — Clear cover requirements are essentially the same as in 1964 version of the Code, but the additional cover required for concrete exposed to aggressive agents is dealt with more elaborately in the Clauses that follow *25.4.1*. In hollow block slabs, the side cover to reinforcement may be 10 mm [*see* *29.7*(c)].

Use of bundled bars is permitted in *25.1.1* and when minimum concrete cover is based on bar diameter, a group of bars bundled in contact should be treated as a single bar of diameter derived from the total equivalent area (*see* *25.3.0*).

Cover to reinforcement is provided to

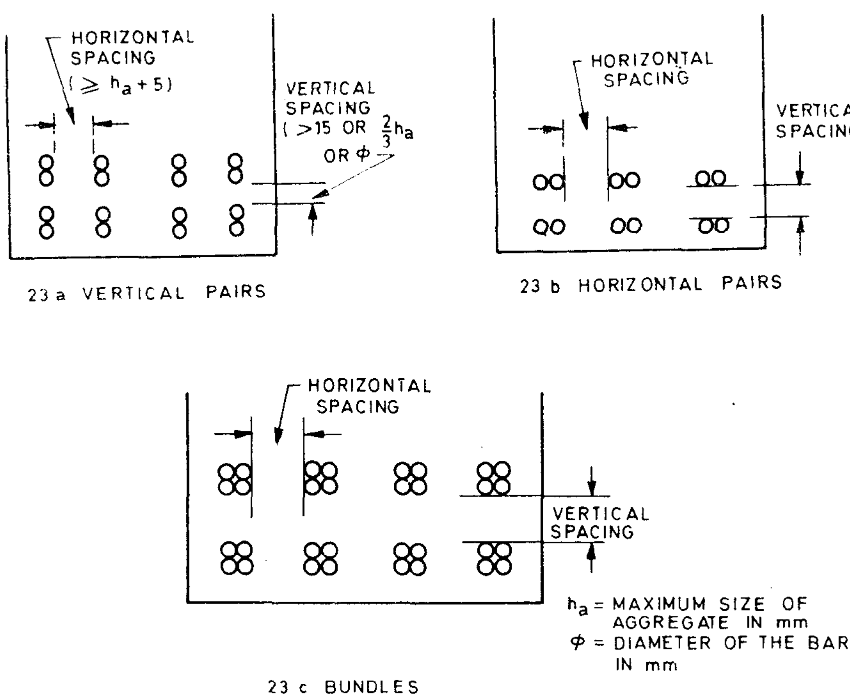



*Fig. E-23 Minimum Bar Spacing Between Groups of Bars* develop the required bond strength and to protect the reinforcement against corrosion. Susceptibility of corrosion of the steel is governed by the cover provided and the permeability of the concrete (*see 7.1*). Therefore, in addition to providing the required cover, it should also be ensured that the concrete is well compacted, dense and impermeable. Also it is necessary to specify concrete mix details to provide the required durability (*see* Table 19). When high strength deformed bars are used, the development length (or bond strength) may be governed by the cover, especially with large diameter bars. Consideration of cover in relation to bond strength of steel may become necessary when bars of large diameters, that is, greater than 36 mm are lapped (Ref 29).

The values of cover suggested in the Code and which will be shown in the drawings are nominal. Clause *11.3* permits tolerances from these nominal values, that is, the cover shall in no case be reduced by more than one-third of specified cover or 5 mm, whichever is less. During construction it should be ensured that these tolerances are met with (*see also* comments on *11.3*).

Where a member is required to have resistance to fire, the nominal cover may have to be increased to the main bars to prevent premature spalling of concrete cover. (*see* comments on *20.2*).

In general, the risk of corrosion of steel is more when a reinforced concrete member is periodically immersed in sea water than when continuously submerged in sea water. Hence greater cover is required for reinforced concrete members subject to alternate wetting and drying of sea water.

Clauses *25.4.1* to *25.4.2.3* give the nominal cover for concretes made with dense natural aggregates conforming to IS : 383-1970*. The cover may have to be increased where lightweight or porous aggregates are used.

'Any other reinforcement' specified in *25.4.1* sub-para (e) of the Code includes shear reinforcement (stirrups) in beams and lateral ties in columns.

*25.5 Requirements of Reinforcement for Structural Members*—For requirements concerning reinforced concrete walls, *see 31.4*.

*25.5.1* BEAMS

*25.5.1.1* TENSION REINFORCEMENT—The provisions for a minimum amount of tensile reinforcement applies to those beams which for architectural or other reasons are much larger in cross-section than required by strength considerations alone. With a very small amount of tensile reinforcement, the strength computed by assuming a cracked reinforced concrete section becomes less than that of a corresponding plain concrete section computed from its modulus of rupture. In such beams, failure will be sudden. To prevent such a possibility the Code requires a minimum amount of steel. This requirement will give tension reinforcement as 0.34 percent for mild steel ($f_y = 250$ N/mm²) and 0.20 percent for cold-worked deformed bars ($f_y = 415$ N/mm²). The requirement gives approximately the same 0.3 percent minimum (for mild steel) as required in the 1964 version of the Code.

The minimum reinforcement required for slabs (*see 25.5.2.1*) is less than that required for beams, since an overload will be distributed laterally and a sudden failure will be less likely, and, therefore, is based on shrinkage and temperature effects.

MAXIMUM REINFORCEMENT—The restriction indicated here arises from convenience in construction and is similar to the considerations for limiting the longitudinal reinforcement in column given in *25.5.3.1* (a) of the Code.

Note that a steel area of 4 percent will be on the high side for a singly reinforced section. However, it will be possible to reach steel areas of about 3 percent to 4 percent when doubly reinforced sections are designed.

For earthquake resistant designs, IS : 4326-1976* recommends that the maximum tensile steel ratio, $\frac{A_s}{bd}$ should be, $p_c + 0.15\frac{f_{cu}}{f_y}$, where $p_c$ is the actual steel

*Specification for coarse and fine aggregates from natural sources for concrete (*second revision*).

*Code of practice for earthquake resistant design and construction of buildings (*first revision*).

ratio $\frac{A_{sc}}{bd}$ on the compression face.

*25.5.1.2* COMPRESSION REINFORCEMENT— [*see also 25.5.1.1* and *25.5.3.1*(a)]. The restriction indicated here arises from requirements of convenience in construction.

Whenever compression reinforcement is introduced in the beam, in order to take up additional moments (and not merely as stirrup hangers), it should be ensured that this reinforcement is effectively restrained just as in columns. Application of *25.5.3.2* will in essence mean that the diameter of the stirrup should be at least one-fourth of the diameter of the largest longitudinal bar and the spacing should not exceed sixteen times the diameter of the smallest longitudinal bar.

*25.5.1.3* SIDE FACE REINFORCEMENT—This rule takes into consideration crack width limitation as well as lateral buckling of web in beams.

*25.5.1.4* TRANSVERSE REINFORCEMENT IN BEAMS FOR SHEAR AND TORSION—Thin cover to stirrup should be avoided [*see* comments on *25.2.2.4*(b)].

For flanged beams the Code favours the arrangement shown in Fig. E-24, especially if *48.4.3* is considered.

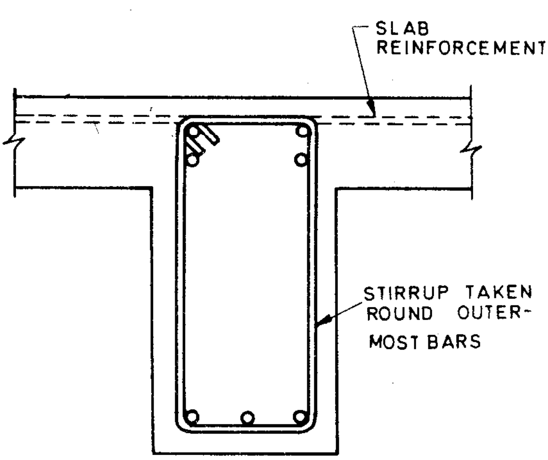



*Fig. E-24 Shear and Torsional Reinforcement in Flanged Beams*

*25.5.1.5* MAXIMUM SPACING OF SHEAR REINFORCEMENT—For vertical stirrups, a spacing of 0.75 $d$ ensures that a potential shear crack (inclined crack) is crossed by at least one stirrup. By a similar consideration, inclined stirrups (*see 39.4* and *47.4*) should be so arranged that a potential shear crack assumed at 45° inclination to the axis of the member is crossed at least by one of such stirrups.

*25.5.1.6* MINIMUM SHEAR REINFORCEMENT—Shear reinforcement restrains the growth of inclined shear cracks and hence increases ductility and provides a warning of the impending failure. In an unreinforced web, the sudden formation of an inclined crack might lead to abrupt and sudden failures without warning. Accordingly, a minimum amount of shear reinforcement is specified. Many times this provision will govern design requirement and not *39.4* and *47.4*.

'The permissible value in shear' refers to the shear strength of concrete, given in *39.2.1* for limit state method and *47.2.1* for working stress method.

Stirrups may be omitted in slabs and footings also, provided that the punching shear stress is sufficiently low.

*25.5.1.7* DISTRIBUTION OF TORSION REINFORCEMENT—When torsion arises from the requirements of compatibility, and when equilibrium can be maintained without torsion in the member under consideration, specific provision need not be made to resist torsional stresses. In any beam, the minimum shear reinforcements should be provided.

a) *Stirrups*—Note that single legged stirrups, and U-shaped stirrups are not to be used for resisting torsion, as these will not be effective. Also inclined stirrups and bent up bars are not recognised for torsion reinforcement.

   The limits on spacing of stirrups are intended to ensure the development of the torsional strength of the beam, to prevent excessive loss of torsional stiffness after cracking and to control crack widths.

b) *Longitudinal reinforcement*—Bars are required at each corner to form a closed stirrup and to provide satisfactory anchorage to the stirrup.

*25.5.1.8* In major T-beams and L-beams, distribution of the negative reinforcement for control of cracking must take into account two considerations:

a) Wide spacing of the reinforcement across the full effective width of flange may cause some wide cracks to form in the slab near the web; and

b) Close spacing of reinforcement near the web leaves the outer regions of the flange unprotected. The 1/10 of the span (effective span) limitation is to guard against too wide a spacing with some additional reinforcement required to protect the outer portions of the flange.

*25.5.2* SLABS

*25.5.2.1* This rule applies principally to solid slabs and not to ribbed, hollow block or voided slabs. The minimum amounts recommended are empirical and are intended to take care of shrinkage and temperature stresses.

*25.5.2.2* —

*25.5.3* COLUMNS

*25.5.3.1* LONGITUDINAL REINFORCEMENT

a) *Minimum reinforcement*—The design method for columns suggested in *38.3* and *45.1* incorporate separate terms for the load carried by concrete and by longitudinal reinforcement to ensure that only reinforced concrete columns are designed by these procedures. Reinforcement is necessary to provide resistance to bending, which may exist whether or not computations show that bending exists, and to reduce the effect of creep and shrinkage of concrete under sustained loading. It has been observed that creep and shrinkage tend to transfer load to the reinforcement, with a consequent increase in stress in reinforcement, and that this increase is greater as the ratio of reinforcement decreases. Unless lower limit is placed on this ratio, the stress in reinforcement may increase to yield level under sustained load.

For nominal reinforcement in pedestals, *see 25.5.3.1*(b). Requirements of minimum reinforcement in wall are given in *31.4*.

*Maximum reinforcement*—The indicated limit can be considered as a practical maximum in terms of requirements of placing of concrete.

Further restrictions are sometimes necessary on longitudinal steel; to say 2½ percent of gross area in the case of mild steel bars and 4 percent in the case of cold-worked bars, whenever heavy live loads are likely to come on the columns for a long period and are subsequently removed, as in the case of columns supporting bins and silos;

b) Larger column sections may be adopted either for meeting local building regulations (say with respect to fire resistance) or to use standard formworks or moulds. In such cases of lightly loaded columns, the Code permits the calculation of minimum steel on the basis of the area of concrete required to resist the direct stress. Two points may be mentioned:

   1) A minimum limit in such cases will be that corresponding to pedestals, that is, 0.15 percent of gross area [*see 25.5.3.1*(h)].

   2) If the column is a part of a framework, accidental removal of the column in the lower storey may convert the column under consideration into a tie carrying tension. Reference 3 suggests a minimum steel area of $\frac{0.15\,P}{f_y}$ to take care of this contingency. (where $P$ is the ultimate axial load imposed on the column, expressed in Newtons, and $f_y$ is expressed in N/mm²).

   In all cases the minimum number of bars and the minimum diameter of longitudinal bars must be provided.

c) For shapes other than rectangular or circular, at least one bar should be provided at each apex or corner with proper lateral reinforcement provided.

d) The minimum diameter is based on the requirements of stiffness and, therefore, is independent of the strength or type of the steel.

e) —

f) —

g) When columns carry large bending moments and small axial forces, they can be approximated to beams. In such cases, reinforcements at the sides will be necessary to control crack width, and the considerations will be similar to those for side face reinforcement in beams (*see 25.5.1.3*).

h) (*see* comment on *24.1.1*).

*25.5.3.2* TRANSVERSE REINFORCEMENT

a) *General*—The design recommendations in the Code are based on the assumption that the members will be so detailed that the participation of steel in the reinforced concrete will be such that:

1) reinforcing bars will carry their intended portion of the load until ultimate conditions are reached; and

2) members will have adequate ductility so that abrupt failures under overload conditions will be avoided.

With reference to columns, these requirements may be restated as:

1) the longitudinal bars should be restrained adequately by means of ties properly spaced and enclosed to avoid premature buckling of longitudinal bars; and

2) the concrete in the core should be adequately confined, so that reasonable ductility is ensured.

The rest of the rules in sub-clauses of *25.5.3.2* follow from these requirements.

In earthquake resistant design of reinforced concrete columns, special confining reinforcements and shear reinforcements will be required.

b) *Arrangement of transverse reinforcement*—This rule ensures that the concrete in the core is adequately confined, resulting in ductile behaviour under overloads. The effectiveness of transverse reinforcement comes from the location at which the longitudinal steel is held rigidly in position and not from the straight portion of ties extending between corner bars (*see* Fig. E-25). Therefore, the Code indicates the requirement of supplementary cross ties for effective confining of concrete.

c) *Pitch and diameter of lateral ties*

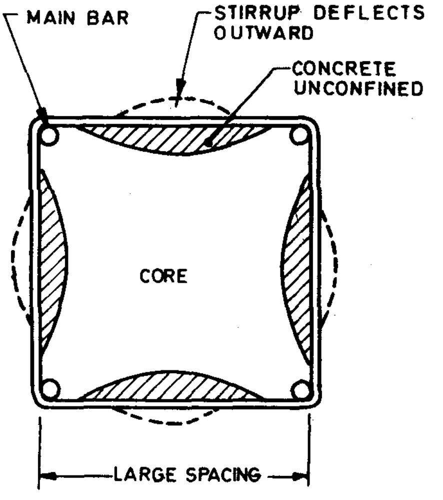



*Fig. E-25 Improper Confinement of Core in a Column*

1) *Pitch*—Column bars carrying compression loads are liable to buckle when ultimate failure conditions are approached. Therefore, transverse ties must provide adequate lateral support to each longitudinal bar to prevent instability that is, outward movement of the bar. From theoretical considerations, it has been shown that the critical unsupported length for buckling of compression bars will be in excess of sixteen times the bar diameter (Ref 31).

2) *Diameter*—Ties must be stiff enough to prevent lateral displacement of the longitudinal bars when ultimate failure conditions are reached. Here it is not the strength but rather the stiffness of the ties that govern. Therefore, the size of the ties is independent of the type or grade of steel used.

d) *Helical reinforcement*

*Pitch*—An increase in load capacity (5 percent) is allowed for columns with helical transverse reinforcements, according to *38.4* and *45.2,* provided that the volume of helical reinforcement satisfies *38.4.1.* In such cases, the load capacity of the column depends on the area of concrete available within the core and in order that the full capacity of the core is mobilised it should be confined effectively, thereby ensuring a triaxial state of stress inside the core. Hence the requirement of small pitch for the helical reinforcement.

*25.5.3.3* This rule is a specific application of *25.2.2.6*. Wherever the steel force changes its direction, such as at the offset bends, transverse forces are required to maintain equilibrium. Additional ties should be placed at such bends and they should have adequate strength to take up the outward transverse component of the vertical force at the bend when the longitudinal bar is stressed to its design value (*see* Fig. E-26). Note that in the column face illustrated in Fig. E-26, additional stirrup will be required only near the bottom crank.

## *26.* EXPANSION JOINTS

*26.1* The width of expanison joints between adjacent structures or parts of the same structure which are dissimilar in mass or stiffness should provide for the maximum amplitudes of the motion of each structure or part in earthquake resistant design of buildings.

*26.2* —

*26.3* Joints are classified as:

a) Construction joints (*see* 12.4)
b) Movement joints
   1) Contraction joints
   2) Expansion joints

Guidance for location, design and construction of different types of movement joints in buildings is given in

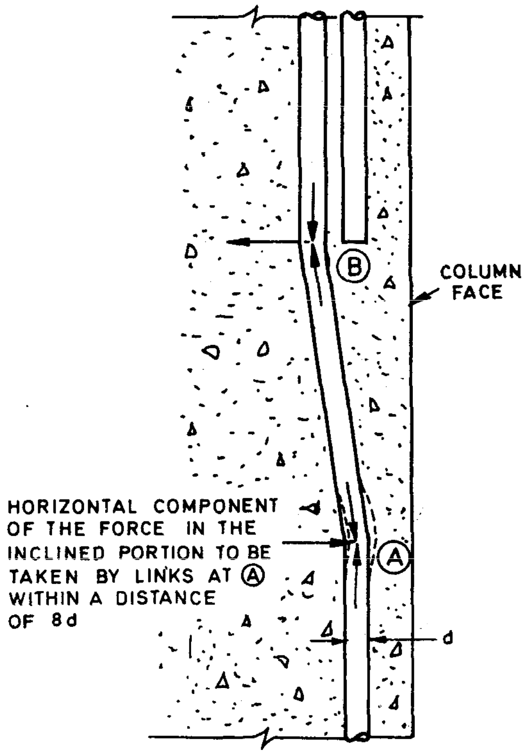



*Fig. E-26 Splice with Offset Cranked Bar in a Column*

IS : 3414-1968*, which is meant for general building construction including concrete buildings. When earthquake forces are to be considered in design, parts of the structures may have to be separated from adjacent ones and in such cases the location and gaps of the separations may have to be different from that for ordinary building (*see* IS : 4326-1976†). Joints in concrete structures for storage of liquids are dealt separately in IS : 3370 (Part I)-1965‡.

*Code of practice for design and installation of joints in buildings.

†Code of practice for earthquake resistant design and construction of building (*first revision*).

‡Code of practice for concrete structures for the storage of liquids: Part I General requirements.

SECTION 4

# SPECIAL DESIGN REQUIREMENTS FOR STRUCTURAL MEMBERS AND SYSTEMS

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# SECTION 4 SPECIAL DESIGN REQUIREMENTS FOR STRUCTURAL MEMBERS AND SYSTEMS

*27.* GENERAL

*27.1* —

*28.* DEEP BEAMS

*28.1 General*

a) The provisions given in *28* are based on the work reported in Reference 32. It has been observed that, for deep beams, having small values of effective span to overall depth ratio (that is $\frac{l}{D} < 2.5$), the assumption of plane sections before bending remain plane after bending is not valid and shall be designed taking account of non-linear distribution of stress and lateral buckling. In such a situation, special requirements regarding disposition and detailing of reinforcement are necessary. The provisions of *28* and its Subclauses of the Code are applicable mainly when the deep beam is uniformly loaded from the top. In addition, special detailing provisions for deep beams loaded from bottom face are given in *28.3.3*. For other types of loadings, specialist literature may be referred to (Ref 7 and 20).

b) —

*28.2 Lever Arm*—In deep beams, the requirement for flexural reinforcement is not large, and a high degree of accuracy for its determination is not necessary. Therefore, reinforcement is determined using the approximate values of lever arms given here which have been derived from experiments. These expressions for lever arm take into account the distribution of the reinforcements as defined in *28.3.1*(c) and *28.3.2*(b), respectively, for simply supported beams and continuous beams.

It may be noted that the definition of effective span $l$, given in *28* is different from that of the usual definition of effective span for normal beams given in *21.2*.

*28.3 Reinforcement*—The detailing rules given in *28.3* and its sub-clauses are such that a separate check for shear is not required [*see 28.1*(b)]. The area of flexural reinforcement provided in a deep beam should be at least equal to that required for a normal beam subjected to the given imposed moment, the width of beam being same as that for the deep beam and lever arms as given in *28.2*.

*28.3.1* Positive Reinforcement—Positive reinforcement is the reinforcement provided in the mid-span of a deep beam on its tension side. The negative reinforcement (*see 29.3.2*) refers to the reinforcement provided on the tension face of a deep beam at the supports.

a) The loading applied at top of beam is carried to the supports, primarily by arch action. This requires very good anchorages and the extensions of the entire flexural reinforcement to the supports. Accordingly, curtailment of flexural reinforcements recommended for normal beams (*see 25.2.3.3*) is not permitted.

b) Smaller diameter bars with sufficient development length or mechanical anchorages should be provided to prevent anchorage failure before the attainment of required strengths of the flexural reinforcement (*see* Fig. E-27).

c) It has been observed (Ref 32) that the tension zone in the bottom of the beam is relatively small and accordingly the Code stipulates that the principal flexural reinforcement should be placed in the zone equal to $0.25D - 0.05l$ adjacent to the tension face of the beam. The arrangement is illustrated in Fig. E-27.

*28.3.2* Negative Reinforcement—Negative reinforcement in deep beams refers to the tensile reinforcement to resist the hogging moment over supports in case of continuous spans. Figure E-28 shows the disposition of this reinforcement in case of deep beams, having $l/D$ varying from less than unity (< 1) up to 2.5. Note that here the symbol $l$ refers to the clear span of deep beams and not the effective span, as in case

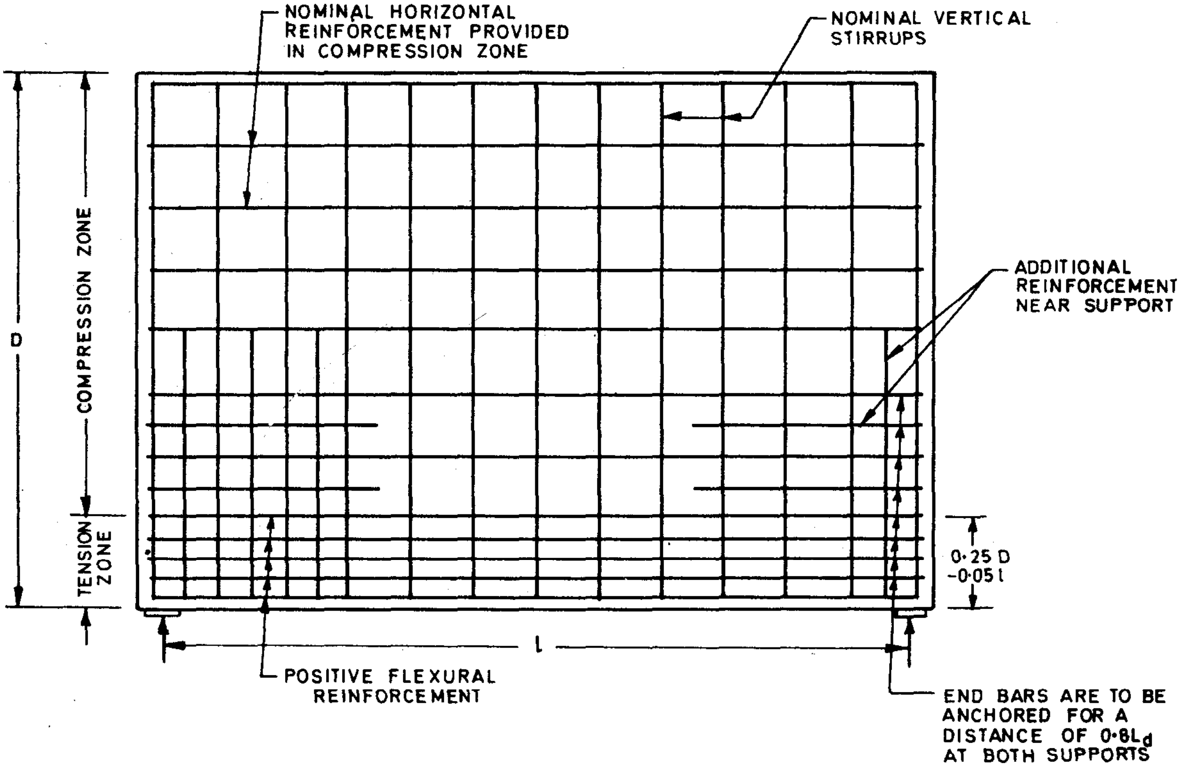



*Fig. E-27 Reinforcement Detailing in Simply Supported Deep Beams*

of other clauses of the Code (namely *28.1*, *28.2* and *28.3.1*). In Fig. E-28 the following points may be noted:

a) Obviously, negative reinforcement can be terminated [*see 28.3.2 (a)*] only in such deep beams which have $l/D >$ 1.0. This curtailment is to be done probably throughout for all the negative reinforcement.

b) If $l/D \geq 2.5$, no negative reinforcement need to be placed in the lower (web) zone (depth $= 0.6D$), [*see* E-28 (a)] the beam acts as a shallow beam.

c) For deep beams, having $l/D \leq 1.0$, the negative reinforcement is placed uniformly over the whole compression zone of depth $0.8D$ [(*see* Fig. E-28(c)].

d) Other cases, which are likely to be met with, can be detailed as shown in Fig. E-28 (b).

*28.3.3* VERTICAL REINFORCEMENT — When a deep beam is loaded at its bottom edge, the load is carried by vertical or inclined tension towards the supports. To enable the compression arch to develop, the whole of the suspended load is to be transferred by means of vertical reinforcement into the compression zone of the beam. Care must be exercised not to exceed the design strength of steel in the stirrups (or suspension reinforcement). This will ensure that the deep beam would not fail by splitting longitudinally along the flexural tensile reinforcement and also will ensure a satisfactory crack control in the web region under service loads. Suspender stirrups should completely surround the bottom flexural reinforcement and extend into the compression zone of the wall beam as shown in Fig. E-29. The spacing of vertical reinforcement should not exceed 150 mm. Another case where suspension reinforcement is provided is when loads or reactions are introduced along the full depth of a beam, for example, when deep wall beams support each other. (For details *see* Ref 20).

Separate consideration of shear reinforcements, as in the case of shallow beams, will not be necessary for deep beams.

*28.3.4* SIDE FACE REINFORCEMENT — The minimum requirements are intended to

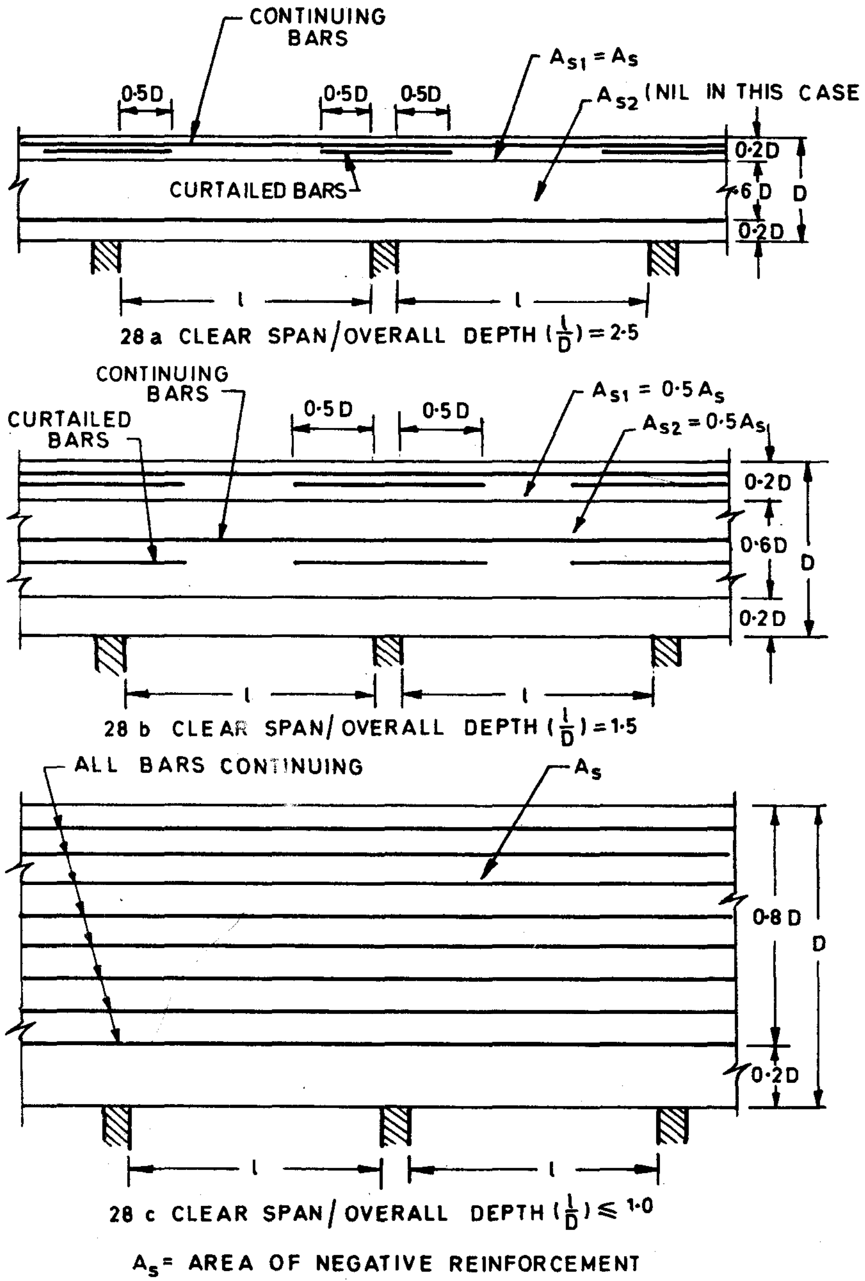



*Fig. E-28 Disposition of Negative Reinforcement in Continuous Deep Beams (Clause 28.3.2)*

control cracking due to shrinkage and temperature and also to resist lateral buckling. Then the deep beam is loaded at the bottom edge, *28.3.3* should be applied for determination of vertical reinforcement (suspender bars).

## *29.* RIBBED, HOLLOW BLOCK OR VOIDED SLAB

*29.1 General*—(*See* Ref 3 and *also 29.8*). Clauses *29.1* to *29.7* are intended for *in-situ* construction, constructed in one of the ways

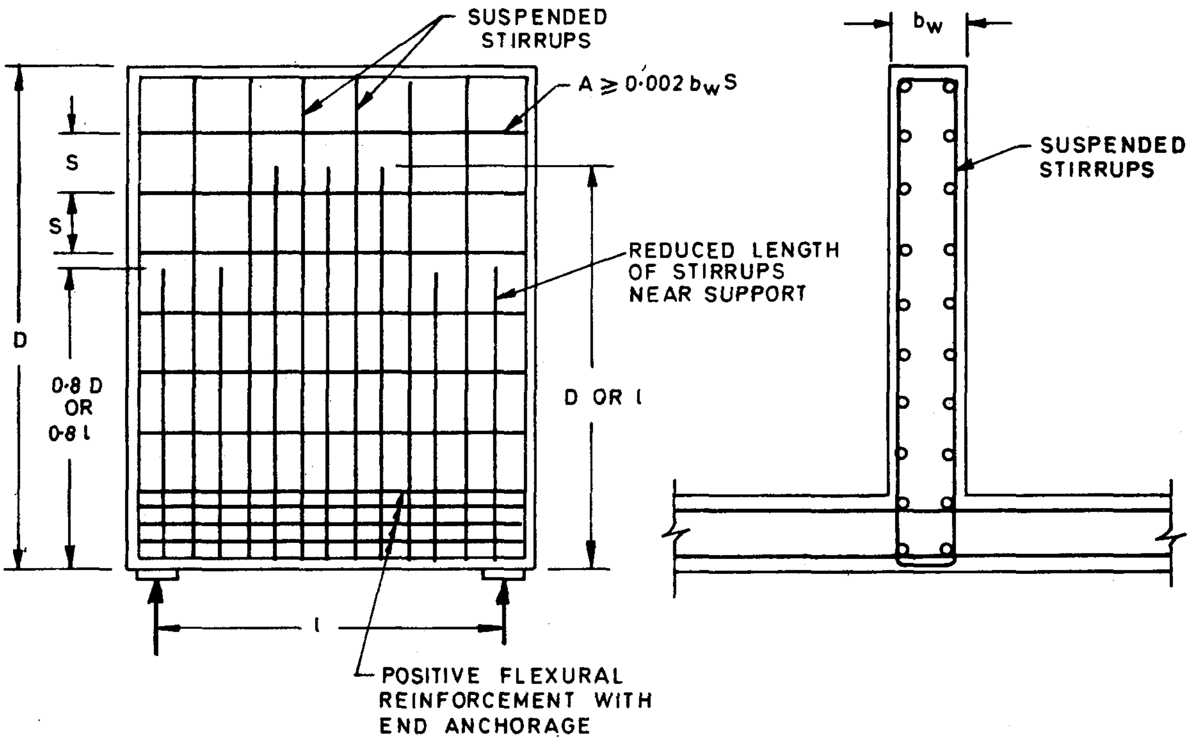



*Fig. E-29 Suspender Bars for Deep Beams*

described in para (a), (b) and (c) and Clause *29.8* for construction with precast joists. The sub-paras (a), (b) and (c) correspond to ribbed slabs, hollow block slabs and voided slabs, respectively.

Ribbed slabs and hollow block slabs (*see* Fig. E-30) are provided with a concrete topping. The Code is silent on the issue whether this topping can be used for computing the structural strengths. As the topping transfers the load by arching action, its depth should be at least one-tenth of the clear distance between the ribs in slabs with or without permanent filler blocks whenever the topping is reckoned in structural strengths

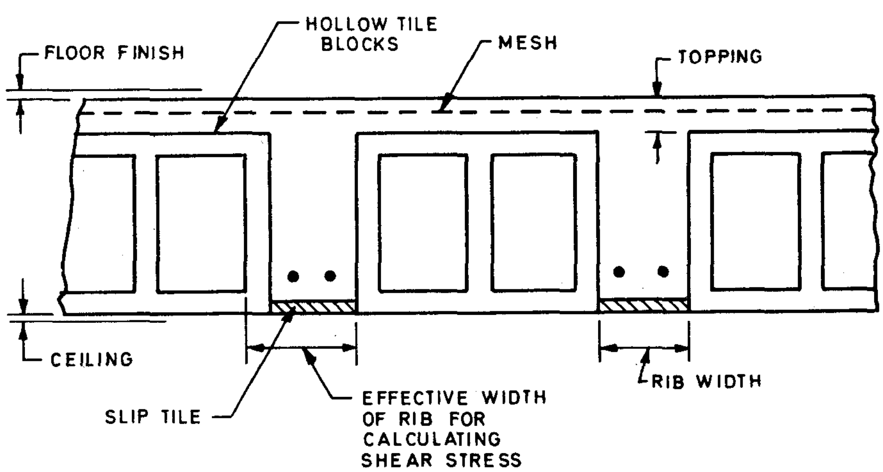



*Fig. E-30 Hollow Clay-Block Floor*

calculations. IS : 6061 (Part II)-1971* suggests a minimum topping thickness of 50 mm for cement concrete and this value may be taken as a guide. Further information on the thickness of topping may be obtained from Ref 3.

*29.2 Analysis of Structure*— It is assumed in the Code that the size and position of ribs will be according to *29.5*.

Ribbed, hollow block and voided slabs are essentially designed as slabs spanning one-way and for these cases *23.2* will be relevant. *23.4* speaks of ribbed slabs spanning in two directions and probably refers to waffle slabs.

The designer is permitted to idealize continuous spans as a series of simple spans because it may not be feasible to arrange the reinforcements over the supports within the restricted space available in the ribs. In such cases, however, crack control reinforcements required in para (b) of *29.7* should be provided over the support regions.

*29.3 Shear*—(*See* Fig. E-30).

*29.4 Deflection*— If a slab, through continuous over supports, is designed as simply supported [*see 29.2* and para (b) of *29.7*] then it should be treated as simply supported for the purpose of checking the ratio of span to effective depth given in *22.2.1*.

*29.5 Size and Position of Ribs*— The conditions laid down are intended to ensure that the methods of analysis permitted in *29.2* are justified. Where the slab is arranged to span in one direction, it is suggested that, in addition to the condition specified in the clause, a minimum of five ribs be provided.

*29.6 Hollow Blocks and Formers*

*29.7 Arrangement of Reinforcement:*

a) As a general rule, 50 percent of the mid span reinforcements in slabs should extend into the supports. Anchorage requirements will be in accordance with para (a) of *25.2.3.3*.

b) The system of treating continuous slabs as simply supported has arisen in practice because of the difficulty of providing enough top steel in the ribs over supports to resist the moments which would arise from treating the slabs as continuous. From the point of view of safety, this is likely to be satisfactory. However, from the point of view of serviceability, its sufficiency is more doubtful. Effectively designing this way is asking for a very large redistribution in the support section. This means that, even under dead load, the support steel will yield if the concrete cracks, and it cannot, therefore, act effectively as anti-crack reinforcement. It may well be that cracks in the top surface of slabs over the supports are often not serious, the cracks being covered by floor finishes or partitions. The designer should nevertheless be aware that this method of design does have risks of serious cracking associated with it.

c) It is suggested to provide a single layer of mesh having cross-sectional area of not less than 0.12 percent of the topping in each direction.

## *30.* FLAT SLABS

*30.1 General*

a) (*See also 23.4*)— Ribbed slabs spanning in two directions at right angles may also be treated as solid slabs provided that the spacing of the ribs is not more than 12 times the flange thickness.

b) The two design methods, namely the empirical method and the continuous frame method given in the 1964 version of IS : 456 have been redesignated now as 'direct design method' and 'equivalent frame method' respectively, but have been brought into closer agreement, especially through *30.5.4*.

c) The Code now incorporates substantial revisions. The methods of analysis and design follow closely those of which were based on the extensive research at the University of Illinois, USA referred to in the commentry to ACI 318-1977.

*Code of practice for construction of floor and roof with joists and hollow filler blocks: Part II With hollow clay filler blocks.

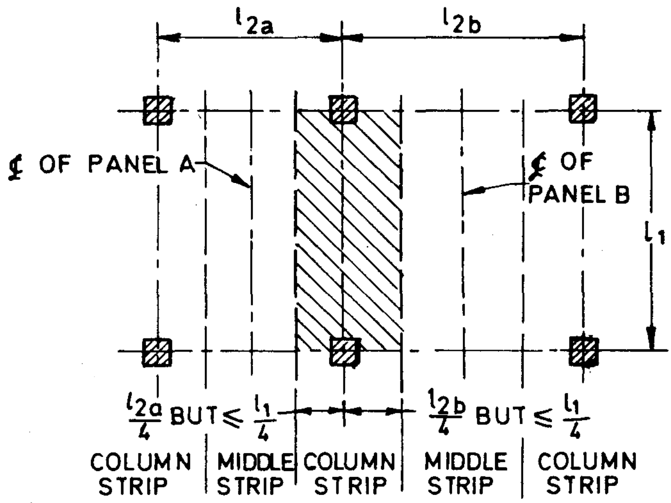



*Fig. E-31 Panels, Column Strips and Middle Strips*

*30.1.1* The definitions of column strip, middle strip and panel are illustrated in Fig. E-31.

*Panel*—An unambiguous definition of 'Panel' is as follows:

Panel means that part of a slab bounded on each of its four sides by the centre lines of the columns.

*30.2 Proportioning*

*30.2.1* THICKNESS OF FLAT SLAB—While referring to *22.2.1* and Fig. 3 of the Code, the average percentage of steel across the whole width of the panel at mid span should be used for computing the modification factor for tension reinforcement.

For the span/depth ratios of flat slabs, the Code stipulates that the longer span should be considered and this is in contrast to the Note given in *23.1* which recommends the consideration of short span for slabs spanning in two directions.

*30.2.2* DROPS—Drops are usually provided for the purpose of reducing the shear stresses around the column supports and also for reducing the amount of negative reinforcements in the support regions.

The Code does not specify the minimum thickness (projection below the slab) for the drop. According to Ref 28, this should be at least 1/4 the thickness of slab wherever provided.

*See also 30.7.2* (b), for the limit on drop thickness for the purpose of computing negative reinforcement over supports.

*30.2.3* COLUMN HEADS—Column heads and similar arrangements at the top of a column are intended primarily to increase the capacity of the slab to resist punching shear. On this basis, a theoretical 45 degree failure plane can be defined, outside of which enlargements of the support would be ineffective in transferring shear into the column.

The proportions of Column head affect the following points in design:

a) In calculating the absolute sum, $M_o$ of the bending moments, the clear span $l_n$ is to be taken as the distance from face to face of columns or capitals (*see 30.4.2.2*). From this it could also be inferred that the maximum height of the column head is limited to 0.175 $l$, for design purposes.

b) The stiffening effect of flared column heads may be ignored [*see* comments on *30.5.1* (d)].

c) The critical section for shear should be taken at a distance $d/2$ from the column/capital (*see 30.6.1*).

d) Openings should not encroach on column heads (*see 30.6.1.2*).

*30.3 Determination of Bending Moment*— The Code permits two specific methods of design, the equivalent frame method and the direct design method, both of which represent progressive simplifications of the general statement and can be adopted provided the limitations on these methods specified in *30.5.1* and *30.4.1* respectively are met. Both methods depend on the representation of a three dimensional structure as a two dimensional system.

*30.3.1* METHODS OF ANALYSIS AND DESIGN— The two methods of design specified here, namely, direct design method, and equivalent frame method, correspond to the empirical method and continuous frame method of the 1964 version of the Code. Apart from the change in terminology; an important change in approach is that design requirements are now given for rectangular columns and if the columns are of other shapes, equivalent rectangular or square sections should be considered. The reasons for changing over to rectangular columns are:

a) rectangular shapes are more widely used; and
b) more experimental data are available for flat slabs on rectangular columns and information is meagre for other cases.

Any deviations from the limitations of proportions indicated should be justified by more rigorous methods of analysis and design in accordance with basic principles of structural mechanism and all serviceability conditions, including the specified limit on deflection are met.

It is to be noted that the detailing rules given in Fig. 15 are applicable only when the analysis is done by direct design method.

*30.3.2* BENDING MOMENTS IN PANELS WITH MARGINAL BEAMS OR WALLS—The bending moments in the slab adjacent to the marginal beam, refers to those of *30.5.5*. *See also* para (c) of *30.5.5.4* for moments in the middle strip adjacent and parallel to an edge supported by wall.

*30.3.3* TRANSFER OF BENDING MOMENTS TO COLUMNS—This provision is to be complied by both the methods of design.

Frequently slabs have to transfer moments in addition to shear in the case of (a) exterior columns of flat slabs, (b) interior columns in flat slabs when the adjacent spans are different in length or in loading, and (c) flat slab systems which resist lateral loads. Note that this clause will be relevant only to flat slabs (which contains no beams) which are required to transfer unbalanced moments to columns directly.

Research on slab system without beams and in which the columns were square indicates that 60 percent of the unbalanced moment can be assumed to be transferred to the column by flexure of the slab and 40 percent by eccentricity of the shear forces about the centroid of the critical section (*see* Fig. E-32). While transfer through flexural action is covered here, the complementary portion for transfer by eccentricity of shear is given in *30.6.2.2*

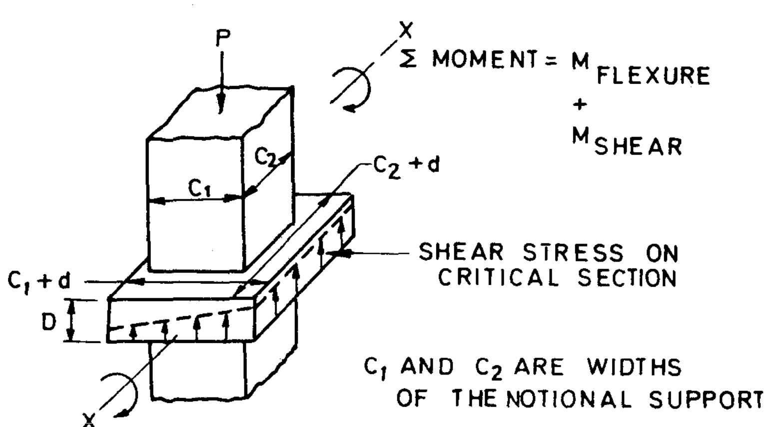



*Fig. E-32 Transfer of Moments to Columns*

This clause is based on a study described in Ref 33 wherein columns with square section only were considered and little information is available on other shapes of cross-section of columns. The expression for $\alpha$ given in the Code has been obtained as an extension for rectangular shaped columns and accounts for the fact that the amount of moment transferred by flexure increases with an increase in the width of the face that resists the flexural action.

Critical section for shear is defined in *30.6.1*. If $C_1$ and $C_2$ are the dimensions of the column support, the corresponding dimension for critical section for shear will be $C_1+d$ and $C_2+d$ respectively (*see* Fig. E-32). Note that $d$, the effective depth should be taken with respect to the total depth of drop or slab for flat slabs with and without drops respectively. The additional steel required for the transfer of moment through flexure may be distributed over a width of $C+3D$, and not the dimension $C+D$ alone.

The total unbalanced moment is to be distributed to the columns above and below the joint in proportion to their stiffness. The appropriate moment capacity must be provided in the columns.

When the equivalent frame method is adopted, the unbalanced moments should be computed with respect to the critical sections for negative moments, specified in *30.5.3.1* and *30.5.3.2*.

*30.4 Direct Design Method*—The direct design method gives a set of simplified rules including those of Fig. 15 of the Code that finally leads to the proportioning of slab sections to resist flexural and shear stresses.

The limitations imposed on the applicability of the direct design method arise from the analytical and experimental studies from which it was developed, together with the precedents provided by structural systems designed and built in conformity with the earlier Code.

The following steps are involved in this method:

a) Determination of the total design moment for the span (that is, the iso-static moment *M*o of a simply supported beam), with respect to each direction (*30.4.2.2*).

b) Determination of negative and positive design moments along a span, as fractions of *M*o (*30.4.3*).

c) Apportioning of these negative and positive design moments to column strips and middle strips (*30.4.4* and *30.4.5*).

In addition, the columns must have a minimum stiffness and these should be proportioned to resist the moment arising from the effects of patterned loadings. The corresponding requirements are given in *30.4.5* and *30.4.6* respectively.

*30.4.1* Limitations—An important limitation, not mentioned in the Code, is that the direct design method can be applied only for gravity loads which are uniformly distributed over the panels. When lateral loads are considered, shear walls or other bracing elements designed to resist the entire lateral loads must be provided, if the direct design method is to be applied (*see* Ref 3 and 28).

a) At least three spans are required because the negative bending moment at an interior support in a two-bay structure can easily exceed the values specified in the method.

b) If the ratio of panel length to panel width exceeds two, the panel becomes virtually a one-way slab and the direct design method cannot be extended to such cases.

c) Provision of 10 percent offset of columns is considered a reasonable practical limit. If the column offsets result in variation of spans in the transverse direction, the adjacent transverse spans should be averaged while carrying out the analysis (*see 30.4.2.4*).

d) Maximum ratio between the successive spans than specified may develop negative bending moment in regions for which design is made only for positive moment. When the successive spans differ in the transverse direction, the average value should be considered for determination of *M*o (*see 30.4.2.4*).

e) The limitation on the ratio of live load to dead load is imposed in consideration of the effects of pattern loading. If the ratio of live load to dead load exceeds three, moments produced by pattern loading would be more severe than those calculated according to the direct design method.

*30.4.2* TOTAL DESIGN MOMENT FOR A SPAN

*30.4.2.1* —

*30.4.2.2* The formula for total moment is based on the assumption that the panel is supported at the edges of the support. This is justified by the success of the method used in the earlier Code in producing safe structures. The values of clear span shall not be taken less than 0.65 $l_1$ to prevent undue reductions in design moment when the columns are long and narrow in cross-section or have large brackets or capitals.

*30.4.2.3* As the expression for *M*o in *30.4.2.2* is meant for rectangular column support, supports that are not rectangular/square are replaced by equivalent rectangular/square supports having the same area. This device simplifies the use of design methods and is generally conservative. Equivalent square sections for a few typical shapes of cross-sections are given in Fig. E-33.

the design practice that has been in vogue. Figure E-34 provides a summary of the provisions in *30.4.3.1* to *30.4.3.3*. The design moments obtained are for the entire width of the panel and these should be distributed to column strips and middle strips as specified in *30.5.5*.

*30.4.3.1* —

*30.4.3.2* —

*30.4.3.3* It will be adequate to assume the gross concrete cross-section for computing the flexural stiffnesses ($K = \frac{I}{l}$), since in the equivalent frame method also this value is permitted [*see 30.5.1* (c)].

Since the use of the direct design method is limited by the requirements of *30.4.1*, it is permissible to use uniform cross sections between column centre-lines for computing $K_s$ (*see* Ref 34).

The detailing of the reinforcement for transferring the moment from the slab to the exterior column is important and should be done according to *30.3.3* and *30.6.2.2*.

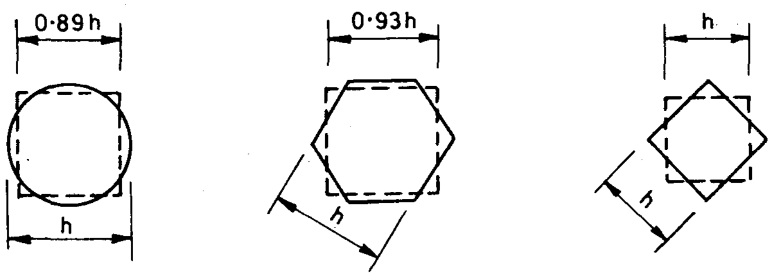



*Fig. E-33 Equivalent Square Sections for Typical Cross Sections*

*30.4.2.4* —

*30.4.2.5* —

*30.4.3* NEGATIVE AND POSITIVE DESIGN MOMENTS—The coefficients recommended for obtaining the negative and positive design moments from the iso-static bending moment *M*o, are based on analytical studies on the distribution of elastic moments in different slab configurations, tempered by the design practice that has been in vogue.

*30.4.3.4* The reduction in moments is based on the redistribution of moments (*see 21.7, 36.1.1* and *43.2*). Though redistribution up to 30 percent is permitted in *36.1.1*, here it is restricted to 10 percent because of the approximations and limitations inherent in the idealization and the simplified method of analysis recommended. Clauses *21.7*, *36.1.1* and *43.2* on redistribution of moments should not be applied for flat slabs.

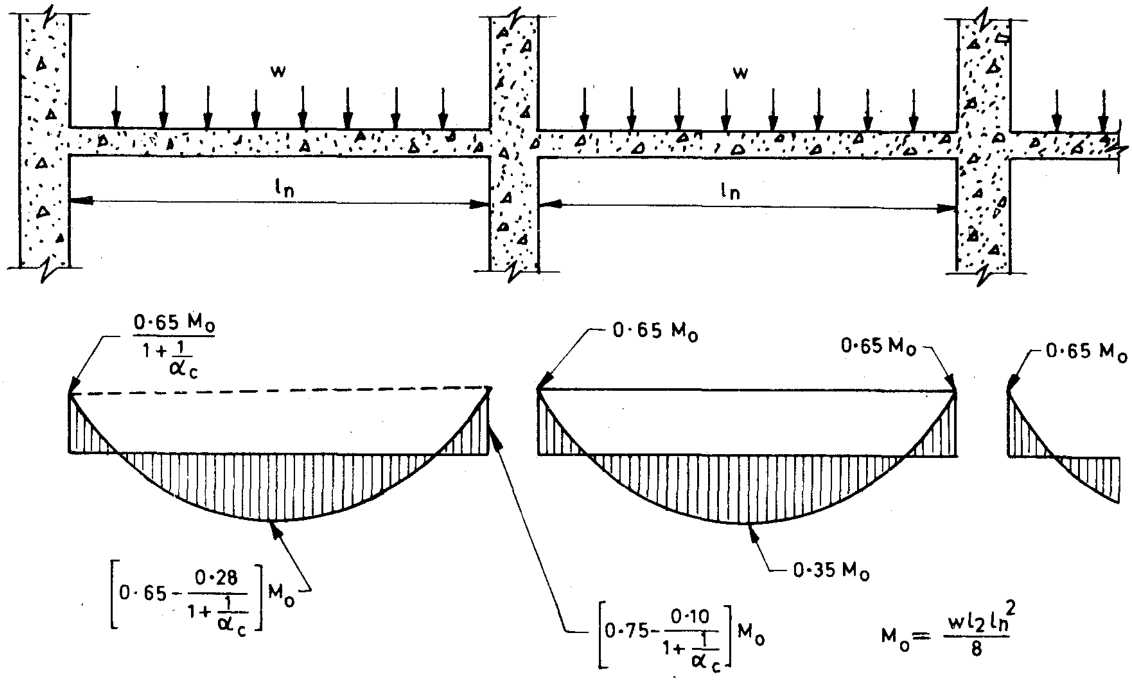



*Fig. E-34 Rules for Dividing the Total Static Moment into Negative and Positive Design Moments*

*30.4.3.5* If no further analysis is made to distribute the unbalanced moment, the entire unbalanced moment should be transferred to the columns in accordance with *30.3.3* and *30.6.2.2*. If an analysis is made to distribute unbalanced moments, flexural stiffnesses may be obtained on the basis of gross concrete sections of the members involved.

*30.4.4* The critical cross-sections for bending moments are the midspan section and support faces for each span, since *30.4.3.1* states that the negative design moments should be located at the face of rectangular supports.

*30.4.5* MOMENTS IN COLUMNS

*30.4.5.1* and *30.4.5.2* This rule provides for the design of interior columns for a specific level of unbalanced moment based on the application of half the live load to the longer of the adjacent spans. (For exterior columns the design moment is derived directly from the negative moment in the column strip). Application of the formula requires the determination of $\alpha_c$ for interior columns.

The expression for *M* has been derived for the case of two adjacent unequal spans, by placing full dead load plus half live load on the longer span, and dead load only on the shorter span, thereby accounting partially for the effects of pattern loading.

*30.4.6* EFFECTS OF PATTERN LOADING—The direct design method makes no direct provision for unbalanced live loads, except in *30.4.5.2* which gives the moments in columns arising from the effects of pattern loading. In the absence of further limitations, it is likely that the slab bending moments could exceed the prescribed values for full loading (on all spans) by as much as 100 percent. This clause limits the possible increases in moment that results from pattern loading at service load level. If the flexural stiffness of the columns above and below the slab are such that the value of $\alpha_c$ falls below the permissible minimum, moments due to pattern loading are likely to exceed the calculated values by more than one-third and the positive design moments must be increased according to the formula. A similar relief for unbalanced live load condition is given in the equivalent frame method, in *30.5.2.3* (Ref 35).

*30.5 Equivalent Frame Method*—The equivalent frame method (which corresponds to the continuous frame method of the 1964 version of the Code) is intended for the cases that fall outside the limitations of *30.4* given for the direct design method and will be compulsory when the frame is required to resist lateral loads. It differs from the direct design method up to the stage of determining the negative and positive design moments (*30.4.3*) and thereafter, the apportioning of the moments to column strips and middle strips across a panel width (*30.5.5*) is common to both the methods. Herein, the three-dimensional slab-and-column system is represented by a series of two dimensional frames taken longitudinally and transversely through the structure and which are analysed for loads acting in the plane of the frames [*see* para (b) of *30.5.1*]. Each frame comprises a slab-beam section of width equal to the distance between panel centrelines and the columns above and below the beam. Figure E-35 shows the idealization for this method of analysis for one direction.

The equivalent frame method gives the design moments and shear forces in the two directions at right angles, which must be provided for, in order to satisfy the equilibrium requirements for slabs. However, the loads on columns can be estimated from the analysis of longitudinal and transverse series of frames and designed accordingly.

The simplified detailing rules of *30.7.3* and Fig. 15 of the Code are not applicable when equivalent frame method of design is used.

*30.5.1* Assumptions—Figure E-35 shows the idealization of the frame with respect to one direction. A similar consideration will apply for the other direction.

In the real slab-and-column system, slab moment can flow around the columns by torsional rotation. Actual columns are, therefore, replaced in line diagrams by equivalent columns with flexural stiffnesses that reflect the torsional rotations possible in three-dimensional systems.

In para (b) *30.5.1*, the Code requires that the frame should be analysed in its entirety when lateral loads are to be resisted by the frame. Though simplified methods, such as portal method, are permitted for lateral loads on the normal types of structural frames (*see 21.4.3*), it appears that these simplified methods should not be applied to

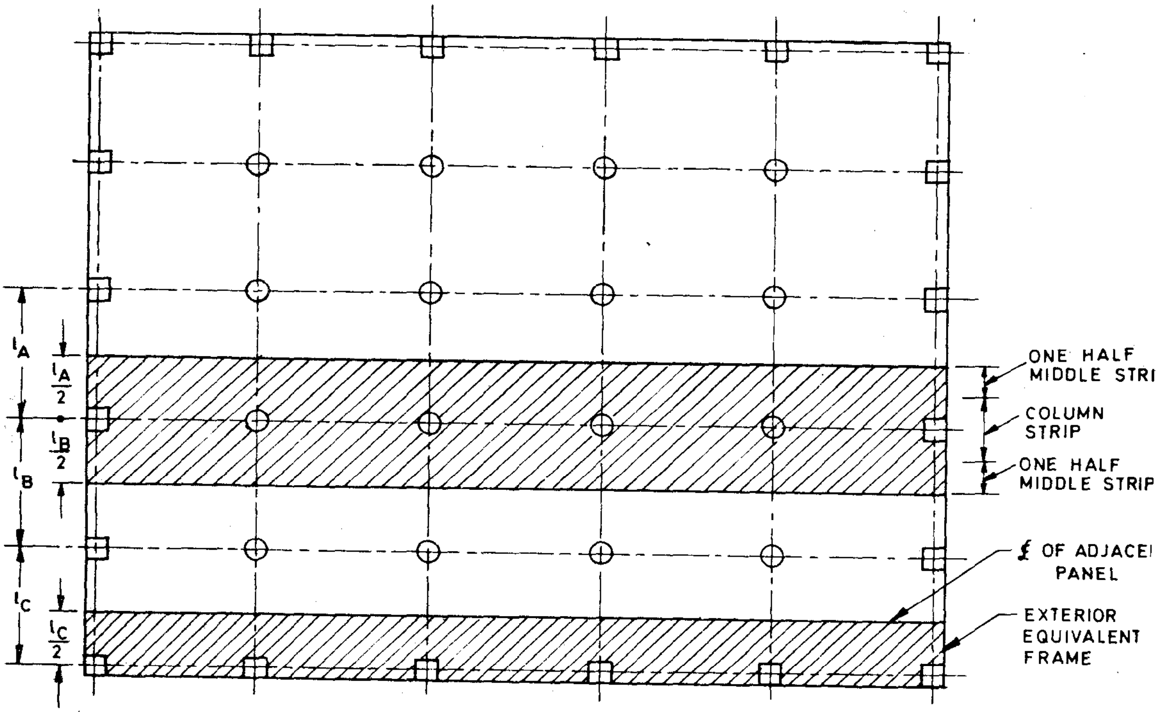
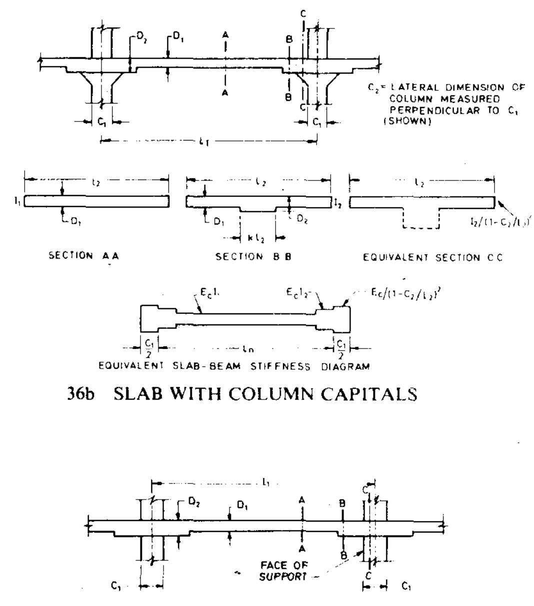


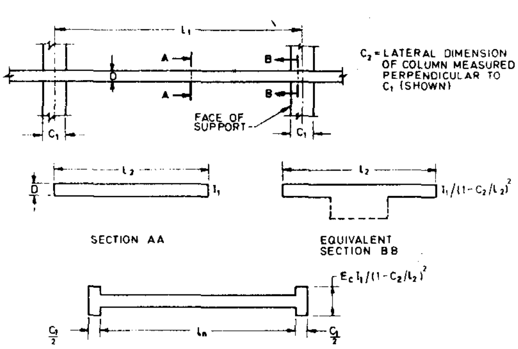

*Fig. E-35 Idealization for Equivalent Frame* the equivalent frames constituting the flat slabs. Thus the equivalent frame should be analysed in its entirety by Hardy cross method or other suitable elastic methods. When lateral loads are not required to be resisted by the frame (that is, when lateral bracings and shear walls are provided to take up the lateral loads), only vertical loads (gravity loads) will require consideration and the Code permits the use of a substitute frame (*see 21.4.2*), with the further simplification that the slab is fixed at a support two panels away. The spans used for analysis should be the distance between the centre-lines of supports as in *21.2(c)* and not the clear span $l_n$ as in *30.4.2.2*. However after the moments are determined, the slab may be proportioned for negative moment at a critical section which will be the face of the support (*see 30.5.3.1*). Redistribution of moments is indirectly provided for in *30.5.2.2* and *30.5.2.3* and, therefore, *21.7* should not be applied again to the flat slab.

While applying para (d), it appears desirable to follow a consistent approach by either (a) ignoring the variation of moment of inertia along the axis of the slab and also the stiffening effects of flared column heads as in Ref 3 or (b) considering the variations in slabs as well as columns, as recommended in Ref 28. However, since the majority of the Code recommendations follow those given in Ref 28, it will be desirable to include the stiffening effects of flared column heads whenever possible. The following supplementary information on the effect of flared column head and allowance for the rigidity of slab-column joints have been included.

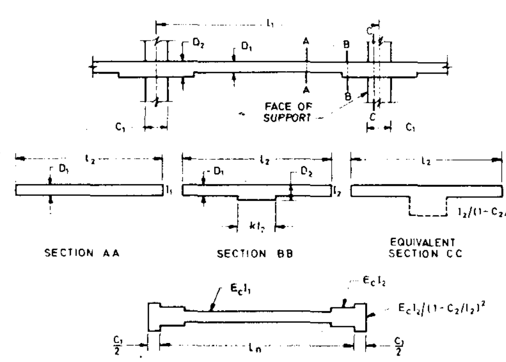
STIFFNESS OF SLABS—Figure E-36 (a) to E-36 (e) show the idealization for common types of slab elements. Coefficients for fixed end moments (FEM), Stiffness ($K$), and carry-over factors (COF) are given in Tables E-4 to E-7. In these Tables, the moment of inertia for the slab between two faces of supports is based on gross concrete section and the variation in the moment of inertia along the axis of the slab-beam between supports is taken into account. The moment of inertia of the slab-beam from the face of support to the centre line of support is assumed equal to the moment of inertia of the slab-beam at the face of support divided by $(1-C_2/l_2)^2$. Where $C_2$ is the width of rectangular support and $l_2$ is the span, both referring to the direction perpendicular to that in which the analysis is made. Note that the conventional factors for FEM, $K$ and COF derived for prismatic members will not be appropriate




36b SLAB WITH COLUMN CAPITALS




36a SIMPLE SLABS




36c SLAB WITH DROP PANELS

*Fig. E-36 Idealization for Column Types of Slab Elements for Continuous Slab-Beam Stiffness* when the moment of inertia varies along the axis of the member.

STIFFNESS OF COLUMNS—Figure E-37 shows the idealizations for different types of column elements. Note that in Fig. E-37 (b) the moment of inertia in the flared column head is assumed to vary linearly along the axis which is an approximation. If the stiffening effect due to flares is neglected, as permitted in the Code, Fig. E-37(a) and (c) will be relevant and the factors for FEM, *K* and COF will correspond to that of a prismatic member.

*30.5.2* LOADING PATTERN

*30.5.2.1* In the case of water reservoirs supported on flat slabs, it may be expected that the load will be applied on all the panels, and therefore, the loading pattern is known. In such cases, no advantage can be taken from *30.5.2.3.* When the live load is variable but does not exceed three-quarters of the dead load, only one load case need be considered, namely, full live load plus dead load on all spans.

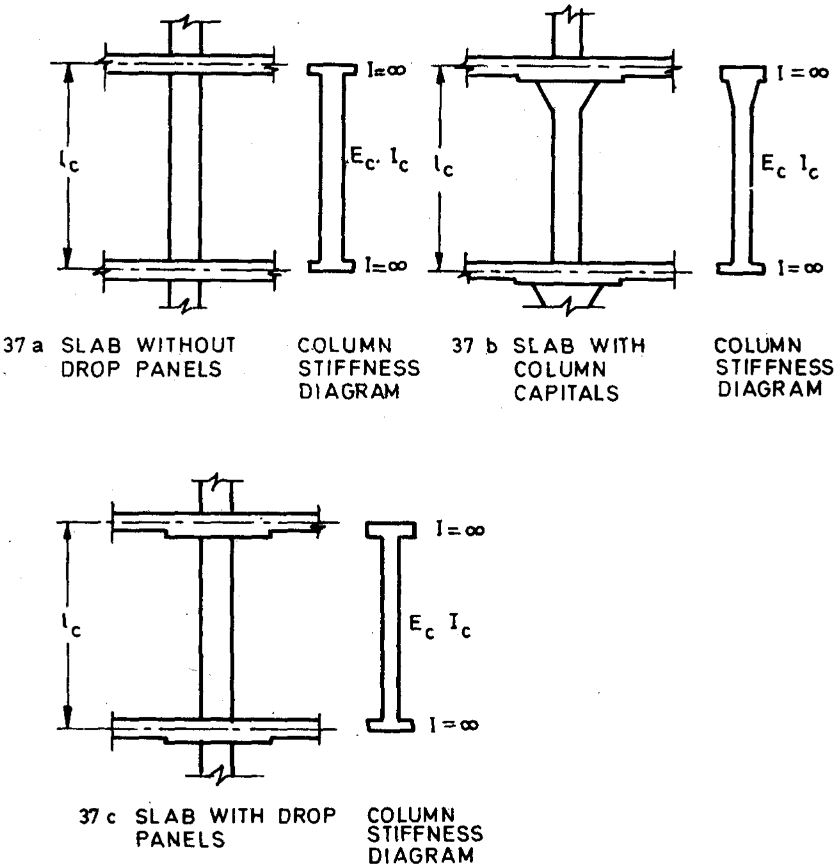
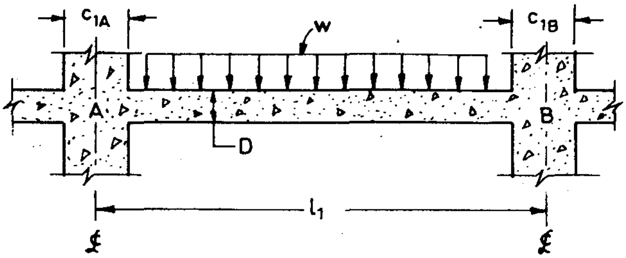



*Fig. E-37 Sections for Calculating Column Stiffness ($K_c$)*

**TABLE E-4 MOMENT DISTRIBUTION CONSTANTS FOR SLABS WITHOUT DROP PANELS***

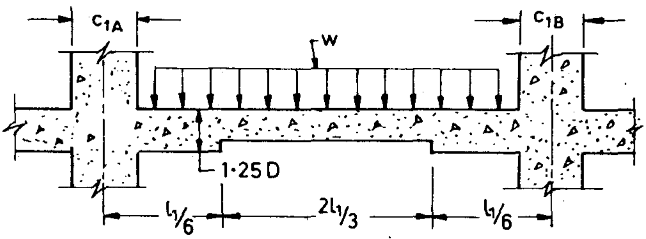



| Column Dimension | | Uniform Load FEM = Coef. $(wl_2l^2_1)$ | | Stiffness Factor† | | Carryover Factor | |
|---|---|---|---|---|---|---|---|
| $\frac{C_{1A}}{l_1}$ | $\frac{C_{1B}}{l_1}$ | $M_{AB}$ | $M_{BA}$ | $K_{AB}$ | $K_{BA}$ | $COF_{AB}$ | $COF_{BA}$ |
| | 0.00 | 0.083 | 0.083 | 4.00 | 4.00 | 0.500 | 0.500 |
| | 0.05 | 0.083 | 0.084 | 4.01 | 4.04 | 0.504 | 0.500 |
| | 0.10 | 0.082 | 0.086 | 4.03 | 4.15 | 0.513 | 0.499 |
| 0.00 | 0.15 | 0.081 | 0.089 | 4.07 | 4.32 | 0.528 | 0.498 |
| | 0.20 | 0.079 | 0.093 | 4.12 | 4.56 | 0.548 | 0.495 |
| | 0.25 | 0.077 | 0.097 | 4.18 | 4.88 | 0.573 | 0.491 |
| | 0.30 | 0.075 | 0.102 | 4.25 | 5.28 | 0.603 | 0.485 |
| | 0.35 | 0.073 | 0.107 | 4.33 | 5.78 | 0.638 | 0.478 |
| | 0.05 | 0.084 | 0.084 | 4.05 | 4.05 | 0.503 | 0.503 |
| | 0.10 | 0.083 | 0.086 | 4.07 | 4.15 | 0.513 | 0.503 |
| | 0.15 | 0.081 | 0.089 | 4.11 | 4.33 | 0.528 | 0.501 |
| 0.05 | 0.20 | 0.080 | 0.092 | 4.16 | 4.58 | 0.548 | 0.499 |
| | 0.25 | 0.078 | 0.096 | 4.22 | 4.89 | 0.573 | 0.494 |
| | 0.30 | 0.076 | 0.101 | 4.29 | 5.30 | 0.603 | 0.489 |
| | 0.35 | 0.074 | 0.107 | 4.37 | 5.80 | 0.638 | 0.481 |
| | 0.10 | 0.085 | 0.085 | 4.18 | 4.18 | 0.513 | 0.513 |
| | 0.15 | 0.083 | 0.088 | 4.22 | 4.36 | 0.528 | 0.511 |
| 0.10 | 0.20 | 0.082 | 0.091 | 4.27 | 4.61 | 0.548 | 0.508 |
| | 0.25 | 0.080 | 0.095 | 4.34 | 4.93 | 0.573 | 0.504 |
| | 0.30 | 0.078 | 0.100 | 4.41 | 5.34 | 0.602 | 0.498 |
| | 0.35 | 0.075 | 0.105 | 4.50 | 5.85 | 0.637 | 0.491 |
| | 0.15 | 0.086 | 0.086 | 4.40 | 4.40 | 0.526 | 0.526 |
| | 0.20 | 0.084 | 0.090 | 4.46 | 4.65 | 0.546 | 0.523 |
| 0.15 | 0.25 | 0.083 | 0.094 | 4.53 | 4.98 | 0.571 | 0.519 |
| | 0.30 | 0.080 | 0.099 | 4.61 | 5.40 | 0.601 | 0.513 |
| | 0.35 | 0.078 | 0.104 | 4.70 | 5.92 | 0.635 | 0.505 |
| | 0.20 | 0.088 | 0.088 | 4.72 | 4.72 | 0.543 | 0.543 |
| 0.20 | 0.25 | 0.086 | 0.092 | 4.79 | 5.05 | 0.568 | 0.539 |
| | 0.30 | 0.083 | 0.097 | 4.88 | 5.48 | 0.597 | 0.532 |
| | 0.35 | 0.081 | 0.102 | 4.99 | 6.01 | 0.632 | 0.524 |
| | 0.25 | 0.090 | 0.090 | 5.14 | 5.14 | 0.563 | 0.563 |
| 0.25 | 0.30 | 0.088 | 0.095 | 5.24 | 5.58 | 0.592 | 0.556 |
| | 0.35 | 0.085 | 0.100 | 5.36 | 6.12 | 0.626 | 0.548 |
| 0.30 | 0.30 | 0.092 | 0.092 | 5.69 | 5.69 | 0.585 | 0.585 |
| | 0.35 | 0.090 | 0.097 | 5.83 | 6.26 | 0.619 | 0.576 |
| 0.35 | 0.35 | 0.095 | 0.095 | 6.42 | 6.42 | 0.609 | 0.609 |

*Applicable when $c_1/l_1 = c_2/l_2$. For other relationships between these ratios, the constants will be slightly in error.

†Stiffness is, $K_{AB} = k_{AB}\,E\,\frac{l_2D^3}{12\,l_1}$ and $K_{BA} = k_{BA}\,E\,\frac{l_2D^3}{12\,l_1}$.

## TABLE E-5 MOMENT DISTRIBUTION CONSTANTS FOR SLABS WITH DROP PANELS*



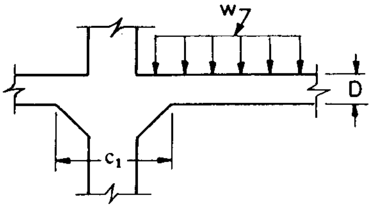

| Column Dimension | | Uniform Load FEM = Coef. $(w l_2 l_1^2)$ | | Stiffness Factor† | | Carryover Factor | |
|---|---|---|---|---|---|---|---|
| $\frac{C_{1A}}{l_1}$ | $\frac{C_{1B}}{l_1}$ | $M_{AB}$ | $M_{BA}$ | $K_{AB}$ | $K_{BA}$ | $COF_{AB}$ | $COF_{BA}$ |
| | 0.00 | 0.088 | 0.088 | 4.78 | 4.78 | 0.541 | 0.541 |
| | 0.05 | 0.087 | 0.089 | 4.80 | 4.82 | 0.545 | 0.541 |
| | 0.10 | 0.087 | 0.090 | 4.83 | 4.94 | 0.553 | 0.541 |
| 0.00 | 0.15 | 0.085 | 0.093 | 4.87 | 5.12 | 0.567 | 0.540 |
| | 0.20 | 0.084 | 0.096 | 4.93 | 5.36 | 0.585 | 0.537 |
| | 0.25 | 0.082 | 0.100 | 5.00 | 5.68 | 0.606 | 0.534 |
| | 0.30 | 0.080 | 0.105 | 5.09 | 6.07 | 0.631 | 0.529 |
| | 0.05 | 0.088 | 0.088 | 4.84 | 4.84 | 0.545 | 0.545 |
| | 0.10 | 0.087 | 0.090 | 4.87 | 4.95 | 0.553 | 0.544 |
| 0.05 | 0.15 | 0.085 | 0.093 | 4.91 | 5.13 | 0.567 | 0.543 |
| | 0.20 | 0.084 | 0.096 | 4.97 | 5.38 | 0.584 | 0.541 |
| | 0.25 | 0.082 | 0.100 | 5.05 | 5.70 | 0.606 | 0.537 |
| | 0.30 | 0.080 | 0.104 | 5.13 | 6.09 | 0.632 | 0.532 |
| | 0.10 | 0.089 | 0.089 | 4.98 | 4.98 | 0.553 | 0.553 |
| | 0.15 | 0.088 | 0.092 | 5.03 | 5.16 | 0.566 | 0.551 |
| 0.10 | 0.20 | 0.086 | 0.094 | 5.09 | 5.42 | 0.584 | 0.549 |
| | 0.25 | 0.084 | 0.099 | 5.17 | 5.74 | 0.606 | 0.546 |
| | 0.30 | 0.082 | 0.103 | 5.26 | 6.13 | 0.631 | 0.541 |
| | 0.15 | 0.090 | 0.090 | 5.22 | 5.22 | 0.565 | 0.565 |
| 0.15 | 0.20 | 0.089 | 0.094 | 5.28 | 5.47 | 0.583 | 0.563 |
| | 0.25 | 0.087 | 0.097 | 5.37 | 5.80 | 0.604 | 0.559 |
| | 0.30 | 0.085 | 0.102 | 5.46 | 6.21 | 0.630 | 0.554 |
| | 0.20 | 0.092 | 0.092 | 5.55 | 5.55 | 0.580 | 0.580 |
| 0.20 | 0.25 | 0.090 | 0.096 | 5.64 | 5.88 | 0.602 | 0.577 |
| | 0.30 | 0.088 | 0.100 | 5.74 | 6.30 | 0.627 | 0.571 |
| 0.25 | 0.25 | 0.094 | 0.094 | 5.98 | 5.98 | 0.598 | 0.598 |
| | 0.30 | 0.091 | 0.098 | 6.10 | 6.41 | 0.622 | 0.593 |
| 0.30 | 0.30 | 0.095 | 0.095 | 6.54 | 6.54 | 0.617 | 0.617 |

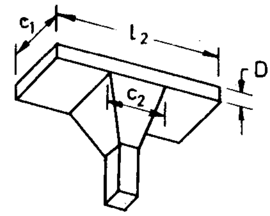
*Applicable when $c_1/l_1 = c_2/l_2$. For other relationships between these ratios, the constants will be slightly in error.

†Stiffness is, $K_{AB} = k_{AB} E \frac{l_2 D^3}{12 l_1}$, and $K_{BA} = k_{BA} E \frac{l_2 D^3}{12 l_1}$.

## TABLE E-6 MOMENT DISTRIBUTION CONSTANTS FOR SLAB-BEAM MEMBERS WITH COLUMN CAPITALS

FEM (uniform load $w$) = $Mwl_2(l_1)^2$
$K$(stiffness) = $kEl_2D^3/12l_1$
Carryover factor = $C$

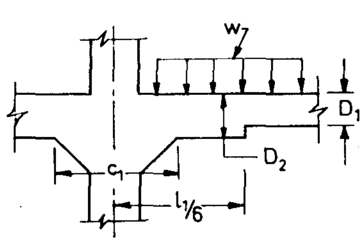


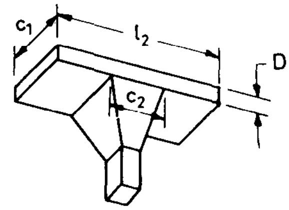




| $c_1/l_1$ | $c_2/l_2$ | $M$ | $k$ | $C$ |
|---|---|---|---|---|
| | 0.00 | 0.083 | 4.000 | 0.500 |
| | 0.05 | 0.083 | 4.000 | 0.500 |
| | 0.10 | 0.083 | 4.000 | 0.500 |
| | 0.15 | 0.083 | 4.000 | 0.500 |
| | 0.20 | 0.083 | 4.000 | 0.500 |
| 0.00 | 0.25 | 0.083 | 4.000 | 0.500 |
| | 0.30 | 0.083 | 4.000 | 0.500 |
| | 0.35 | 0.083 | 4.000 | 0.500 |
| | 0.40 | 0.083 | 4.000 | 0.500 |
| | 0.45 | 0.083 | 4.000 | 0.500 |
| | 0.50 | 0.083 | 4.000 | 0.500 |
| | 0.00 | 0.083 | 4.000 | 0.500 |
| | 0.05 | 0.084 | 4.047 | 0.503 |
| | 0.10 | 0.084 | 4.093 | 0.507 |
| | 0.15 | 0.084 | 4.138 | 0.510 |
| | 0.20 | 0.085 | 4.181 | 0.513 |
| 0.05 | 0.25 | 0.085 | 4.222 | 0.516 |
| | 0.30 | 0.085 | 4.261 | 0.518 |
| | 0.35 | 0.086 | 4.299 | 0.521 |
| | 0.40 | 0.086 | 4.334 | 0.523 |
| | 0.45 | 0.086 | 4.368 | 0.526 |
| | 0.50 | 0.086 | 4.398 | 0.528 |
| | 0.00 | 0.083 | 4.000 | 0.500 |
| | 0.05 | 0.084 | 4.091 | 0.506 |
| | 0.10 | 0.085 | 4.182 | 0.513 |
| | 0.15 | 0.085 | 4.272 | 0.519 |
| | 0.20 | 0.086 | 4.362 | 0.524 |
| 0.10 | 0.25 | 0.087 | 4.449 | 0.530 |
| | 0.30 | 0.087 | 4.535 | 0.535 |
| | 0.35 | 0.088 | 4.618 | 0.540 |
| | 0.40 | 0.088 | 4.698 | 0.545 |
| | 0.45 | 0.089 | 4.774 | 0.550 |
| | 0.50 | 0.089 | 4.846 | 0.554 |
| | 0.00 | 0.083 | 4.000 | 0.500 |
| | 0.05 | 0.084 | 4.132 | 0.509 |
| | 0.10 | 0.085 | 4.267 | 0.517 |
| | 0.15 | 0.086 | 4.403 | 0.526 |
| | 0.20 | 0.087 | 4.541 | 0.534 |
| 0.15 | 0.25 | 0.088 | 4.680 | 0.543 |
| | 0.30 | 0.089 | 4.818 | 0.550 |
| | 0.35 | 0.090 | 4.955 | 0.558 |
| | 0.40 | 0.090 | 5.090 | 0.565 |
| | 0.45 | 0.091 | 5.222 | 0.572 |
| | 0.50 | 0.092 | 5.349 | 0.579 |
| | 0.00 | 0.083 | 4.000 | 0.500 |
| | 0.05 | 0.085 | 4.170 | 0.511 |
| | 0.10 | 0.086 | 4.346 | 0.522 |
| | 0.15 | 0.087 | 4.529 | 0.532 |
| | 0.20 | 0.088 | 4.717 | 0.543 |
| 0.20 | 0.25 | 0.089 | 4.910 | 0.554 |
| | 0.30 | 0.090 | 5.108 | 0.564 |
| | 0.35 | 0.091 | 5.308 | 0.574 |
| | 0.40 | 0.092 | 5.509 | 0.584 |
| | 0.45 | 0.093 | 5.710 | 0.593 |
| | 0.50 | 0.094 | 5.908 | 0.602 |
| | 0.00 | 0.083 | 4.000 | 0.500 |
| | 0.05 | 0.085 | 4.204 | 0.512 |
| 0.25 | 0.10 | 0.086 | 4.420 | 0.525 |
| | 0.15 | 0.087 | 4.648 | 0.538 |
| | 0.20 | 0.089 | 4.887 | 0.550 |
| | 0.25 | 0.090 | 5.138 | 0.563 |
| | 0.30 | 0.091 | 5.401 | 0.576 |
| | 0.35 | 0.093 | 5.672 | 0.588 |
| 0.25 | 0.40 | 0.094 | 5.952 | 0.600 |
| | 0.45 | 0.095 | 6.238 | 0.612 |
| | 0.50 | 0.096 | 6.527 | 0.623 |
| | 0.00 | 0.083 | 4.000 | 0.500 |
| | 0.05 | 0.085 | 4.235 | 0.514 |
| | 0.10 | 0.086 | 4.488 | 0.527 |
| | 0.15 | 0.088 | 4.760 | 0.542 |
| | 0.20 | 0.089 | 5.050 | 0.556 |
| 0.30 | 0.25 | 0.091 | 5.361 | 0.571 |
| | 0.30 | 0.092 | 5.692 | 0.585 |
| | 0.35 | 0.094 | 6.044 | 0.600 |
| | 0.40 | 0.095 | 6.414 | 0.614 |
| | 0.45 | 0.096 | 6.802 | 0.628 |
| | 0.50 | 0.098 | 7.205 | 0.642 |
| | 0.00 | 0.083 | 4.000 | 0.500 |
| | 0.05 | 0.085 | 4.264 | 0.514 |
| | 0.10 | 0.087 | 4.551 | 0.529 |
| | 0.15 | 0.088 | 4.864 | 0.545 |
| | 0.20 | 0.090 | 5.204 | 0.560 |
| 0.35 | 0.25 | 0.091 | 5.575 | 0.576 |
| | 0.30 | 0.093 | 5.979 | 0.593 |
| | 0.35 | 0.095 | 6.416 | 0.609 |
| | 0.40 | 0.096 | 6.888 | 0.626 |
| | 0.45 | 0.098 | 7.395 | 0.642 |
| | 0.50 | 0.099 | 7.935 | 0.658 |
| | 0.00 | 0.083 | 4.000 | 0.500 |
| | 0.05 | 0.085 | 4.289 | 0.515 |
| | 0.10 | 0.087 | 4.607 | 0.530 |
| | 0.15 | 0.088 | 4.959 | 0.546 |
| | 0.20 | 0.090 | 5.348 | 0.563 |
| 0.40 | 0.25 | 0.092 | 5.778 | 0.580 |
| | 0.30 | 0.094 | 6.255 | 0.598 |
| | 0.35 | 0.095 | 6.782 | 0.617 |
| | 0.40 | 0.097 | 7.365 | 0.635 |
| | 0.45 | 0.099 | 8.007 | 0.654 |
| | 0.50 | 0.100 | 8.710 | 0.672 |
| | 0.00 | 0.083 | 4.000 | 0.500 |
| | 0.05 | 0.085 | 4.311 | 0.515 |
| | 0.10 | 0.087 | 4.658 | 0.530 |
| | 0.15 | 0.088 | 5.046 | 0.547 |
| | 0.20 | 0.090 | 5.480 | 0.564 |
| 0.45 | 0.25 | 0.092 | 5.967 | 0.583 |
| | 0.30 | 0.094 | 6.517 | 0.602 |
| | 0.35 | 0.096 | 7.136 | 0.621 |
| | 0.40 | 0.098 | 7.836 | 0.642 |
| | 0.45 | 0.100 | 8.625 | 0.662 |
| | 0.50 | 0.101 | 9.514 | 0.683 |
| | 0.00 | 0.083 | 4.000 | 0.500 |
| | 0.05 | 0.085 | 4.331 | 0.515 |
| | 0.10 | 0.087 | 4.703 | 0.530 |
| | 0.15 | 0.088 | 5.123 | 0.547 |
| | 0.20 | 0.090 | 5.599 | 0.564 |
| 0.50 | 0.25 | 0.092 | 6.141 | 0.583 |
| | 0.30 | 0.094 | 6.600 | 0.603 |
| | 0.35 | 0.096 | 7.470 | 0.624 |
| | 0.40 | 0.098 | 8.289 | 0.645 |
| | 0.45 | 0.100 | 9.234 | 0.667 |
| | 0.50 | 0.102 | 10.329 | 0.690 |

## TABLE E-7 MOMENT DISTRIBUTION CONSTANTS FOR SLAB-BEAM MEMBERS WITH COLUMN CAPITALS AND DROP PANELS

FEM (uniform load $w$) = $Mwl_2l_1^2$
$K$(stiffness) = $kEl_2D^3/12l_1$
Carryover factor = $C$




| $C_1/l_1$ | $C_2/l_2$ | Constants for $D_2 = 1.25D_1$ | | | Constants for $D_2 = 1.5D_2$ | | |
|---|---|---|---|---|---|---|---|
| | | $M$ | $k$ | $C$ | $M$ | $k$ | $C$ |
| | 0.00 | 0.088 | 4.795 | 0.542 | 0.093 | 5.837 | 0.589 |
| | 0.05 | 0.088 | 4.795 | 0.542 | 0.093 | 5.837 | 0.589 |
| | 0.10 | 0.088 | 4.795 | 0.542 | 0.093 | 5.837 | 0.589 |
| 0.00 | 0.15 | 0.088 | 4.795 | 0.542 | 0.093 | 5.837 | 0.589 |
| | 0.20 | 0.088 | 4.795 | 0.542 | 0.093 | 5.837 | 0.589 |
| | 0.25 | 0.088 | 4.795 | 0.542 | 0.093 | 5.837 | 0.589 |
| | 0.30 | 0.088 | 4.795 | 0.542 | 0.093 | 5.837 | 0.589 |
| | 0.00 | 0.088 | 4.795 | 0.542 | 0.093 | 5.837 | 0.589 |
| | 0.05 | 0.088 | 4.846 | 0.545 | 0.093 | 5.890 | 0.591 |
| | 0.10 | 0.089 | 4.896 | 0.548 | 0.093 | 5.942 | 0.594 |
| 0.05 | 0.15 | 0.089 | 4.944 | 0.551 | 0.093 | 5.993 | 0.596 |
| | 0.20 | 0.089 | 4.990 | 0.553 | 0.094 | 6.041 | 0.598 |
| | 0.25 | 0.089 | 5.035 | 0.556 | 0.094 | 6.087 | 0.600 |
| | 0.30 | 0.090 | 5.077 | 0.558 | 0.094 | 6.131 | 0.602 |
| | 0.00 | 0.088 | 4.795 | 0.542 | 0.093 | 5.837 | 0.589 |
| | 0.05 | 0.088 | 4.894 | 0.548 | 0.093 | 5.940 | 0.593 |
| | 0.10 | 0.089 | 4.992 | 0.553 | 0.094 | 6.042 | 0.598 |
| 0.10 | 0.15 | 0.090 | 5.039 | 0.559 | 0.094 | 6.142 | 0.602 |
| | 0.20 | 0.090 | 5.184 | 0.564 | 0.094 | 6.240 | 0.607 |
| | 0.25 | 0.091 | 5.278 | 0.569 | 0.095 | 6.335 | 0.611 |
| | 0.30 | 0.091 | 5.368 | 0.573 | 0.095 | 6.427 | 0.615 |
| | 0.00 | 0.088 | 4.795 | 0.542 | 0.093 | 5.837 | 0.589 |
| | 0.05 | 0.089 | 4.938 | 0.550 | 0.093 | 5.986 | 0.595 |
| | 0,10 | 0.090 | 5.082 | 0.558 | 0.094 | 6.135 | 0.602 |
| 0.15 | 0.15 | 0.090 | 5.228 | 0.565 | 0.095 | 6.284 | 0.608 |
| | 0.20 | 0.091 | 5.374 | 0.573 | 0.095 | 6.432 | 0.614 |
| | 0.25 | 0.092 | 5.520 | 0.580 | 0.096 | 6.579 | 0.620 |
| | 0.30 | 0.092 | 5.665 | 0.587 | 0.096 | 6.723 | 0.626 |
| | 0.00 | 0.088 | 4.795 | 0.542 | 0.093 | 5.837 | 0.589 |
| | 0.05 | 0.089 | 4.978 | 0.552 | 0.093 | 6.027 | 0.597 |
| | 0.10 | 0.090 | 5.167 | 0.562 | 0.094 | 6.221 | 0.605 |
| 0.20 | 0.15 | 0.091 | 5.361 | 0.571 | 0.095 | 6.418 | 0.613 |
| | 0.20 | 0.092 | 5.558 | 0.581 | 0.096 | 6.616 | 0.621 |
| | 0.25 | 0.093 | 5.760 | 0.590 | 0.096 | 6.816 | 0.628 |
| | 0.30 | 0.094 | 5.962 | 0.590 | 0.097 | 7.015 | 0.635 |
| | 0.00 | 0.088 | 4.795 | 0.542 | 0.093 | 5.837 | 0.589 |
| | 0.05 | 0.089 | 5.015 | 0.553 | 0.094 | 6.065 | 0.598 |
| | 0.10 | 0.090 | 5.245 | 0.565 | 0.094 | 6.300 | 0.608 |
| 0.25 | 0.15 | 0.091 | 5.485 | 0.576 | 0.095 | 6.543 | 0.617 |
| | 0.20 | 0.092 | 5.735 | 0.587 | 0.096 | 6.790 | 0.626 |
| | 0.25 | 0.094 | 5.994 | 0.598 | 0.097 | 7.043 | 0.635 |
| | 0.30 | 0.095 | 6.261 | 0.600 | 0.098 | 7.298 | 0.644 |
| | 0.00 | 0.088 | 4.795 | 0.542 | 0.093 | 5.837 | 0.589 |
| | 0.05 | 0.089 | 5.048 | 0.554 | 0.094 | 6.099 | 0.599 |
| | 0.10 | 0.090 | 5.317 | 0.567 | 0.095 | 6.372 | 0.610 |
| 0.30 | 0.15 | 0.092 | 5.601 | 0.580 | 0.096 | 6.657 | 0.620 |
| | 0.20 | 0.093 | 5.902 | 0.593 | 0.097 | 6.953 | 0.631 |
| | 0.25 | 0.094 | 6.219 | 0.605 | 0.098 | 7.258 | 0.641 |
| | 0.30 | 0.095 | 6.550 | 0.618 | 0.099 | 7.571 | 0.651 |

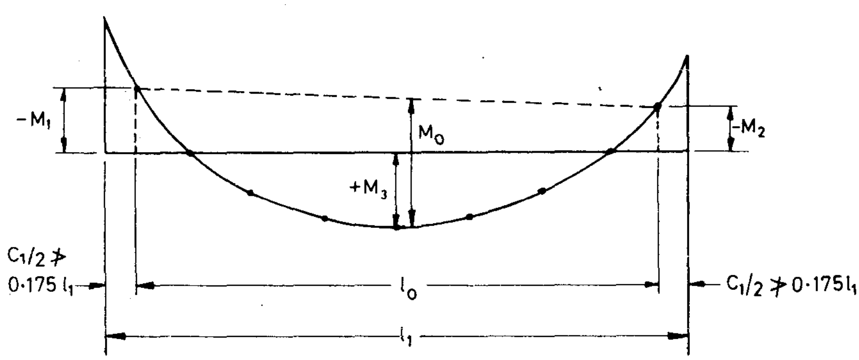
*30.5.2.2* [*See also 21.4.2* (b)]. The use of uniform live load on all panels is based on the relief provided by redistribution of moments in continuous structures. When the building or structure has to carry live loads which may or may not come on a panel, the maximum values of negative and positive moments are not likely to occur simultaneously in a span and advantage may be taken of redistribution of moments. Therefore, the Code does not permit any further redistribution of moments in the case of equivalent frame method.

*30.5.2.3* and *30.5.2.4*—For larger live-load ratios, it is acceptable to design only for three-quarters of full live load on alternate spans (for maximum positive moment) and on adjacent spans (for maximum negative moment). The use of only three-quarters of the full design live-load for maximum-moment loading patterns is based on the fact that maximum negative and maximum positive live load moments cannot occur simultaneously and that redistribution of maximum moments is thus possible before failure occurs. This procedure in effect permits some local overstress under the full design live load if it is distributed in the prescribed manner, but still ensures that the ultimate capacity of the slab system after redistribution of moment is not less than that required to carry the full design dead and live loads on all panels.

*30.5.3* NEGATIVE DESIGN MOMENT

*30.5.3.1* Note *21.6.1*, where a similar reduction in the value of negative moment is to be assumed for beams for proportioning the section at support.

Though the negative moments are determined at the centre line of supports when the equivalent frame method is used, the section at support may be proportioned for the reduced moment at the face of support. This will give rise to the some advantage as that given in *30.4.2.2* for the direct design method. The limitation on the distance between critical section for negative moment and the centre line of support (0.175 $l_1$) also corresponds to that in *30.4.2.2*, the latter being stated as a limit on $l_n$. As elsewhere in the Code, the recommendations are given with respect to rectangular supports of flat slabs, and other shapes of sections should be converted to equivalent rectangles as stated in *30.5.3.3*. Figure E-38 explains the application of this Clause and $-M_1$ and $-M_2$ shown in this figure should be distributed over the column strips and middle strips of the panel.




$M_0 = [(M_1+M_2)/2]+M_3$ need not be greater than $(wl_2 l_0^2)/8$. Permissible reduction for moments $M_1$, $M_2$ and $M_3 = [wl_2 l_0^2/8]\ [(M_1+M_2)/2+M_3]$ for cases qualifying under the clause *30.5.4* only.

*Fig. E-38 Reduction of Negative Moments at Interior Supports (For Equivalent Frame Moment)*

*30.5.3.2* This Clause is intended to exclude the possibility of undue reduction of the design moments at exterior support.

*30.5.3.3* *See also 30.4.2.3,* and Fig. E-33.

*30.5.4* MODIFICATION OF MAXIMUM MOMENT — Since the Code gives two separate methods of design, it is likely that the more rigorous equivalent frame method may lead to more robust section and larger amounts of steel in a few cases although the direct design method was equally applicable and would have led to lesser requirements. As the direct design method has led to satisfactory results in the past, the Code invokes the principle that if two different methods are prescribed to obtain a particular answer, it should not require a value greater than the least acceptable and removes any possible anomoly that may arise.

For those cases which satisfy the limitations of *30.4.1*, but analysed by the equivalent frame method, the permissible reduction is shown in Fig. E-38.

*30.5.5* DISTRIBUTION OF BENDING MOMENT ACROSS THE PANEL WIDTH — (*See also 30.4.4*). The procedures for distribution of the moments across the panel width is common to the direct design method and the equivalent frame method. In the 1964 version of the Code separate criteria for apportioning the moments across the panel width were given for the two methods of design; now these criteria are replaced with a single set which is applicable equally to either of the methods of design.

The factors given in *30.5.5* are based on a linear elastic analysis of moments in slabs (Ref 36). The actual distribution of the moments across a panel is not constant, but resembles that shown by full lines in Fig. E-39, and the exact variation is controlled by presence or absence of drops and column heads, as well as the intensity of loads. However, for design purposes it is a convenient simplification to treat a panel as composed of column strips and middle strips across which the moments are assumed to have constant values.

*30.5.5.1* —

*30.5.5.2* —

*30.5.5.3* —

*30.5.5.4* With reference to para (c), *see also 30.3.2(b)* for panels with marginal beams or walls.

## *30.6* *Shear in Flat Slab*

*30.6.1* Shear strength of flat slabs, pile caps, and two way slabs carrying concentrated loads or reactions is governed by punching shear failure. Two-way action footings [(*see 33.2.4.1* (b)] should also be designed for shear by using *30.6.*

The critical section for punching shear should be taken at a distance $d/2$ from the periphery of the column/capital/drop panel

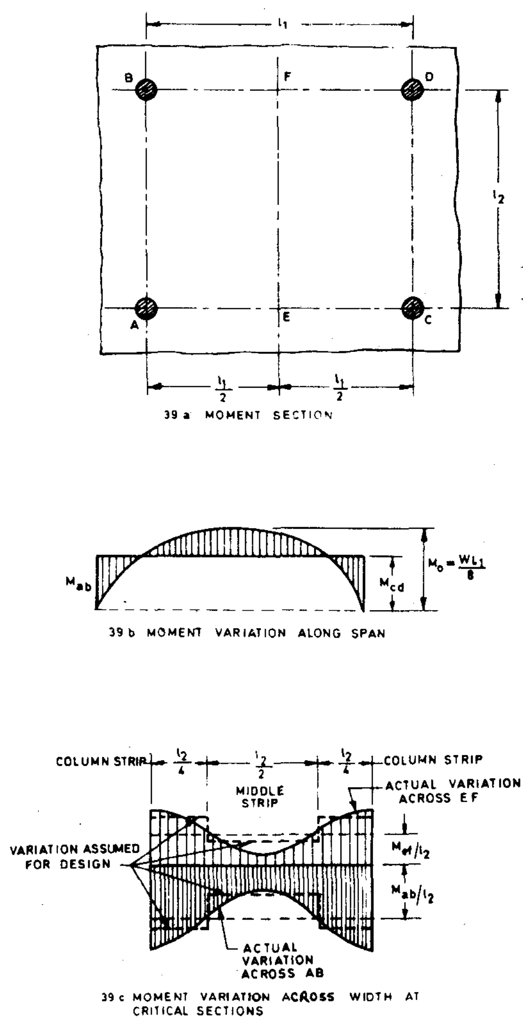



*Fig. E-39 Moment Variation in Flat Slabs* and not a distance $d$ from support as in the case of flexural shear in beams. When flat slabs are provided with drops, two critical sections for shear, one within the drop and the other at a distance $d/2$ from the face of the drop will have to be considered (*see* Fig. 11 B of the Code).

*30.6.1.1* —

*30.6.1.2* The definition of critical section near openings is based on the recommendations of the ACI-ASCE Committee 426 report (Ref 37) and the adequacy of this provision has been confirmed in Ref 38.

*30.6.2* CALCULATION OF SHEAR STRESS

*30.6.2.1* The expression for nominal shear stress in flat slabs is similar to that given in *39.1* except that here $b_o$ is the perimeter of the critical section.

*30.6.2.2* (*See also 30.3.3* and its comment). When slabs are required to transfer moments to columns, a fraction of the moment may be assumed to be transferred by flexural action. The remaining portion $(1-\alpha)$ of the moment is transferred by eccentricity of shear about the centroid of the critical section for shear (*see* Fig. E-32). According to the Code, the shear stresses may be taken as linearly varying about the centroid of the section. The shear stress at any point in the periphery arises from two effects, namely, that due to the shear force $V$ (*see 30.6.2.1*) and that arising from transfer of unbalanced moment to column. Combining these two, the maximum shear stress $\tau$ is given by,

$$\tau = \frac{V}{b_o d} + M_{(\text{shear})} \frac{C_1 + d}{2 J_{xx}}$$

where

$V$ = shear force on the critical section,

$M_{(\text{shear})}$ = portion of moment transferred between slab and column by eccentricity of shear,

$d$ = effective depth of slab,

$b_o$ = periphery of the critical section,

$J_{xx}$ = similar to polar moment of inertia of the critical section about xx (Axis about which the moment acts),

Expressions for $b_o d$ and $J_{xx}$ for typical support configurations are given in Table E-8.

For the purpose of this Clause, the shear force, $V$ should be obtained by considering the loads acting on the panel on each face of the critical section. The forces acting in the area within the critical section need not be considered for computing $V$. If the loads are distributed uniformly in all the panels, the load coming from the tributary area, but ignoring that acting within the critical section, gives $V$ directly.

*30.6.3* PERMISSIBLE SHEAR STRESS

*30.6.3.1* The limiting value of shear stress in the case of flat slabs is determined from consideration of punching shear and will be considerably higher than the value of shear stress recommended in the case of flexural action, as in say Table 13 of the Code.

The 'punching' shear strength of concrete $\tau_c$ depends on the following (Ref 37, 38) factors:

a) The ultimate shear strength $f_{ck}$;

b) The ratio of the dimension of the column support to be effective depth $d$ of the slab; and

c) The ratio, $\beta_c$ (the ratio of shorter side to the longer side of the column support).

Tests for punching shear (Ref 38) and also observations of distress in flat slab buildings indicate that an unmodified or simple limit of $\tau_c = 0.25\sqrt{f_{ck}}$ (N/mm²) may be unconservative if the column support is narrow. For columns with $\beta_c$ less than 0.5, the shear stress at failure may vary from a maximum of $0.25\sqrt{f_{ck}}$ (N/mm²) near the shorter sides to $0.15\sqrt{f_{ck}}$ (N/mm²) or less along the longer sides of the section. The factor $k_s$, given in the Code, has been introduced to reflect this reduction in the shear strength. For non-rectangular shapes, such as the L-shaped column (*see* Fig. 12) $\beta_o$ may be taken as the ratio of the shortest overall dimension to the longest overall dimension of the effective loaded area, formed by enclosing all the re-entrant corners.

*30.6.3.2* The principles for the design of reinforcement for punching shear are broadly similar to those for shear associated with flexural action. The following supplementary information from Ref 13 and 21 and

**TABLE E-8 EXPRESSION FOR $A_c$, $\bar{X}$ AND $J_{xx}$ FOR TYPICAL CASES OF SUPPORT IN FLAT SLABS**

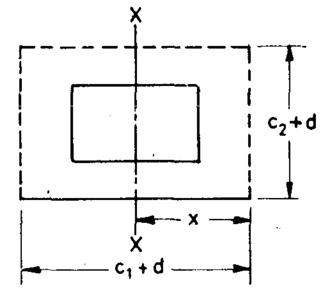
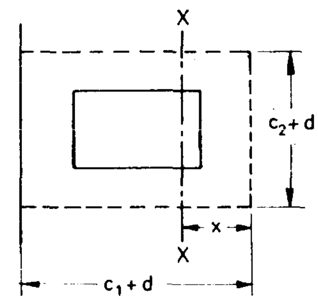
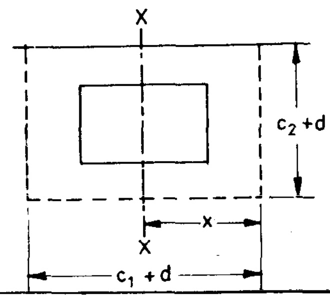
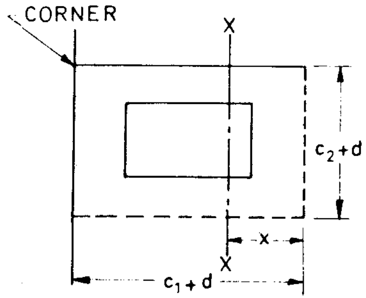
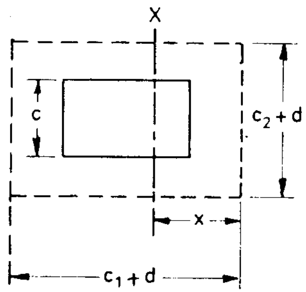
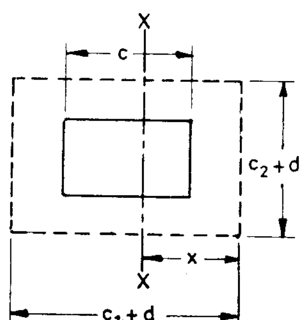
| Sl No. | Case | Geometric Representation | $b_0 d$ | $\bar{X}$ | $J_{xx}$ |
|---|---|---|---|---|---|
| (1) | (2) | (3) | (4) | (5) | (6) |
| 1) | Interior column with no interference from openings |  | $2d(C_1+C_2+2d)$ | $(C_1+d)/2$ | $\frac{d(C_1+d)^3}{6}+\frac{(C_1+d)d^3}{6}+\frac{d(C_2+d)(C_1+d)^2}{2}$ |
| 2) | Exterior column at an edge or interior column with an opening of width more than $C_2+d$ (Fig. 13A or 14D in the Code) |  | $d(2C_1+C_2+3d)$ | $\frac{(C_1+d)^2}{2[2C_1+C_2+3d]}$ | $\frac{d(C_1+d)^3}{6}+\frac{(C_1+d)d^3}{6}+d(C_2+d)\bar{x}^2$ $+2d(C_1+d)[(C_1+d)½-\bar{x}]^2$ |
| 3) | Exterior column at an edge or interior column with an opening of width more than $C_2+d$ (Fig. 13A or 14D in the Code) but free edge parallel in the other direction. |  | $d(C_1+2C_2+3d)$ | $\frac{(C_1+d)}{2}$ | $\frac{d(C_1+d)^3}{12}+\frac{(C_1+d)d^3}{12}+\frac{d(C_2+d)(C_1+d)^2}{12}$ |

TABLE E-8 EXPRESSION FOR $A_c$, $\bar{X}$ AND $J_{xx}$ FOR TYPICAL CASES OF SUPPORT IN FLAT SLABS — *Contd*

| Sl No. | Case | Geometric Representation | $b_0 d$ | $\bar{X}$ | $J_{xx}$ |
|---|---|---|---|---|---|
| (1) | (2) | (3) | (4) | (5) | (6) |
| 4) | Exterior corner column (Fig. 13B in the Code) |   | $d(C_1+C_2+2d)$ | $\dfrac{(C_1+d)^2 . \frac{1}{2}}{C_1+C_2+2d}$ | $\dfrac{d(C_1+d)^3}{12}+\dfrac{(C_1+d)d^3}{12}+d(C_2+d)\bar{x}^2$ $+d(C_1+d)[(C_1+d)/2-\bar{x}]^2$ |
| 5) | Interior column with a small area in effective (Fig. 14A but symmetrical) |   | $d(2C_1+2C_2+4d-C)$ | $\dfrac{\dfrac{(C_1+d)^2}{2}+(C_2+d-C)\times(C_1+d)}{(2C_1+2C_2+4d-C)}$ | $\dfrac{d(C_1+d)^3}{6}+\dfrac{(C_1+d)d^3}{6}+d(C_2+d)\bar{x}^2$ $+d(C_2+d-C)(C_1+d-\bar{x})^2$ $+2d(C_1+d)[(C_1+d)/2-\bar{x}]^2$ |
| 6) | Similar to V but in other orientation |   | $d(2C_1+2C_2+4d-C)$ | $(C_1+d)/2$ | $\dfrac{d(C_1+d)^3}{6}+\dfrac{(C_1+d)d^3}{6}$ $+\dfrac{d(C_2+d)(C_1+d)^2}{2}$ $-\dfrac{dC^3}{12}-\dfrac{Cd^3}{12}$ |

modifications appropriate to the Code have been indicated below:

a) Starting with the critical section for punching shear, the shear stress $\tau$ is computed by using *30.6.2.1* and *30.6.2.2*.

b) If $\tau$ exceeds $(0.5 + \beta_c) \times 0.25\sqrt{f_{ck}}$ (limit state method), shear reinforcement will be necessary. Referring to *39.4*, a suitably arranged expression for area of shear reinforcement is,

$$A_{sv} = \frac{(V - 0.5\, k_s\, \tau_c\, b_o d)}{0.87 f_y}$$

where $A_{sv}$ in the total cross-sectional area of all stirrup legs in the perimeter.

c) This shear reinforcement must be provided along the perimeter of the column.

d) Stirrups may be detailed in one of the ways shown in Fig. E-40. Stirrups may be closed or castellated and should pass around one row of tension steel running perpendicular to them, at each face of the relevant section.

e) The Code requires that the shear

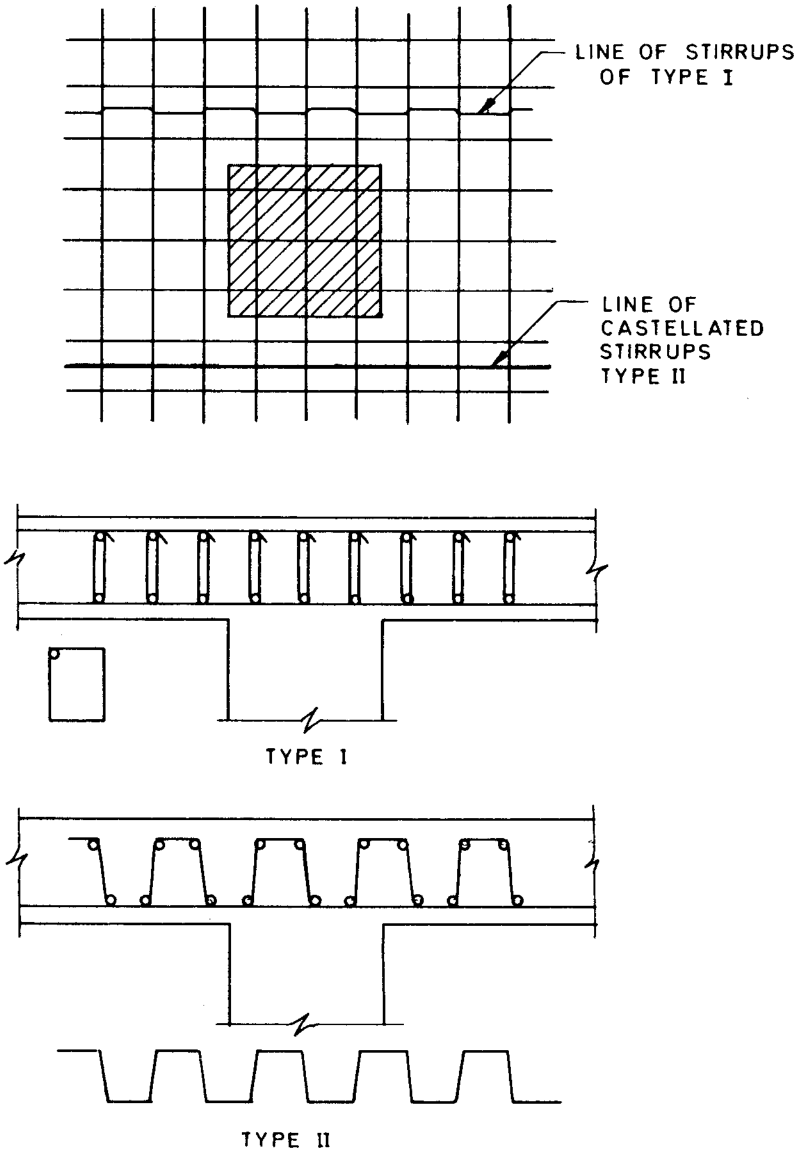



*Fig. E-40 Reinforcement for Punching Shear* stresses should be investigated at successive sections away from the support, but does not indicate suitable intervals. It is suggested that these successive sections may be taken at intervals of ¾ *d*.

f) Practical difficulties preclude the use of stirrups in slabs less than 200 mm thick and, therefore, they have to be designed without shear reinforcement.

g) The spacing of stirrups should not exceed ¾ *d* and must be continued to a distance '*d*' beyond the section at which the shear stress is within allowable limits.

*30.7 Slab Reinforcement*—The four-way reinforcement of flat slabs now has been discontinued in the present version of the Code, probably due to a preference for simplified procedures of detailing and construction. The rules for detailing have now been changed slightly and the locations for bends and curtailment lengths (*30.7.3* and Fig. 15) are defined more clearly.

*30.7.1* SPACING—The upper limit on bar spacing is more stringent in the case of flat slab constructions (with solid slabs) than that of ordinary slabs specified in *25.3.2*. In the 1964 version of the Code, the bar spacing in solid slabs was restricted to 3 times the effective depth or 600 mm and now it is more stringent. The limitation is necessary to ensure slab action and to provide against the effects of concentration of loads on small areas. It also provides control on cracking.

*30.7.2* AREA OF REINFORCEMENT—(*See 30.4.3.1*). The negative design moment should be located at the face of rectangular supports. Accordingly, it will be possible to proportion the reinforcement over the supports by including the depth of the drops, if provided. In some cases, the thickness of the drop may be large, mainly from consideration of punching shear. There is also a remote possibility that very thick drops might be provided in order to economize on the support reinforcements. However, it is not desirable to allow undue advantage to arise from the provision of very thick drops and, therefore, the Code limits the consideration of drop panel thickness for the purpose of computing the support reinforcements.

*30.7.3* MINIMUM LENGTH OF REINFORCEMENT—It is tacitly assumed that the flat-slab structure is subjected to gravity loads only and the minimum lengths are indicated in Fig. 15 of the Code on this basis. When the frame is intended to resist the lateral loads as well, the equivalent frame method should be used and the more rigorous requirements of *25* (requirements governing reinforcement and detailing) should be applied.

*30.7.4* ANCHORING REINFORCEMENT—These anchorage requirements are intended to cater for the probability that, at a discontinuous edge, conditions will be different from those assumed to occur at a spandrel beam or a wall.

*30.8 Openings in Flat Slabs*—In general a rational analysis is required of the effects of openings in slabs. Provided, however, the effect of the moment capacity of the slab system is not likely to be serious, it is permissible to maintain the overall moment capacity in the vicinity of the opening by placing extra reinforcement on each side of the opening in accordance with the specific requirements of this Clause. Additional reinforcements might be required to control possible cracking at the re-entrant corners. It is to be noted that the use of reinforcement according to this Clause is to provide specially for the maintenance of moment capacity. Design for shear is covered specifically under *30.6.3.2*.

## *31.* WALLS

*31.1 General*—If the design of reinforced concrete walls is based on the principles recommended for columns, para (a) of *25.5.3.1* should also be taken into account. It follows that the cross-sectional area of the longitudinal (vertical) reinforcement in a reinforced concrete wall should not be less than 0.8 percent of the cross section; otherwise the wall should be treated as a plain concrete wall. However, the Code is silent on this issue, leaving it to the designer to decide the minimum amount of structural reinforcement in a reinforced concrete wall. It is suggested that this structural reinforcement, which should not be confused with the nominal reinforcements given in *31.4* (a), should be at least 0.4 percent (Ref 3).

Detailing rules given in *25.5.3.2* with regard to transverse reinforcement in compression members are essentially meant for columns. For walls, the following rules (Ref 3) are suggested as a guide:

When in a wall the percentage of vertical reinforcement resisting compression exceeds 2 percent, links at least 6 mm or one quarter the size of the largest compression bar should be provided through the thickness of the wall. The spacing of these links should not exceed twice the wall thickness in either the horizontal or vertical directions and in the vertical direction should be not greater than 16 times the bar size. Any vertical compression bar not enclosed by a link should be within 200 mm of a restrained bar. However, the Handbook on the Unified Code (Ref 21) suggests as follows:

> 'The necessity for links in walls with more than 2 percent of steel will make for considerable practical difficulties. It is recommended that the cross-sectional area of the wall be increased to avoid this if at all possible. Links are unnecessary if reinforcement is arranged in one layer'.

Therefore, a maximum vertical steel of 2 percent of the cross-section seems to be appropriate.

It is to be noted that in a load bearing wall, the ratio of greater lateral dimension to least lateral dimension must be greater than four.

*31.2 Load Carrying Capacity*— Because of the increase in stability against vertical loads, the load carrying capacity of walls increases with decrease in height to length ratio.

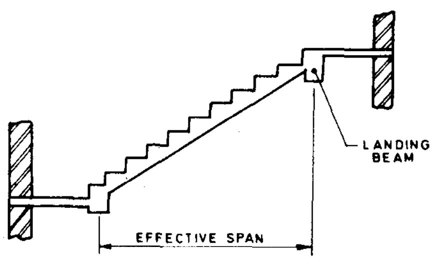
*31.3 Slenderness Effects*— Where cross walls are provided as stiffners sometimes advantage may arise (*see* IS : 1905-1980*) which permits that the slenderness ratio may be taken as the effective length divided by the thickness. In other words, if the cross walls are spaced sufficiently close, the horizontal distance between the cross walls may lead to an effective length smaller than the usual effective height of wall.

*31.4 Reinforcement*—The minimum reinforcements required are nominal and intended for walls in ordinary buildings. When walls form a part of special structures, such as chimneys, silos and water tanks, the minimum steel requirements will be different and the appropriate Codes will govern the design. The minimum reinforcement in the horizontal direction is greater, probably to provide for stability against lateral buckling of walls.

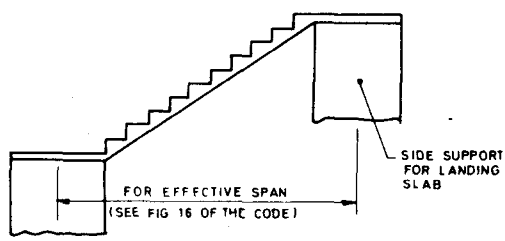
## *32.* STAIRS

*32.0* The recommendations as for beams and slabs may be used for staircases when strength, deflection and crack control is being considered.

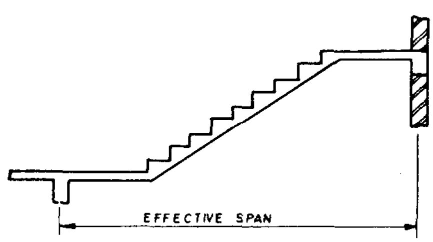
*32.1 Effective Span of Stairs*— *See* Fig. E-41.




41 a SUPPORT AT TOP AND BOTTOM RISERS




41 b SUPPORTING ON THE EDGE OF A LANDING SLAB




41 c LANDING SLAB SPANNING IN THE SAME DIRECTION AS STAIRS

*Fig. E-41 Effective Span of Stairs*

*Code of practice for structural safety of buildings: Masonry walls (*second revision*).

*32.2 Distribution of Loading on Stairs*— The value of live load to be considered on stairs and landings are specified in IS : 875-1964*. The load is taken as uniformly distributed over the plan area of a staircase.

*32.3 Depth of Section*

## 33. FOOTINGS

*33.1 General*—The Code recommendations are confined to the design of footings that support isolated columns or walls and rest directly on soil or on a group of piles. Combined footings, multiple column bases and raft foundation, and pile foundations are not covered here and they will require careful consideration of soil conditions. Relevant Indian Standards and specialist literature should be consulted for design of machine foundations and other foundations subjected to vibratory or impact loads. The following Indian Standards are useful in the design of different types of foundations:

IS : 1080-1980 Code of practice for design and construction of simple spread foundations (*first revision*)

IS : 2911 (Part I/Sec 1)-1979 Code of practice for design and construction of pile foundations: Part I Concrete piles, Section 1 Driven cast *in-situ* concrete piles (*first revision*)

IS : 2911 (Part I/Sec 2)-1979 Code of practice for design and construction of pile foundations: Part I Concrete piles, Section 2 Bored cast *in-situ* piles (*first revision*)

IS : 2911 (Part I/Sec 3)-1979 Code of practice for design and construction of pile foundations: Part I Concrete piles, Section 3 Driven precast concrete piles (*first revision*)

IS : 2911 (Part III)-1980 Code of practice for design and construction of pile foundations: Part III Under-reamed piles (*first revision*)

IS : 2950 (Part I)-1982 Code of practice for design and construction of raft foundations: Part I Design (*first revision*)

In the case of columns or walls subjected to eccentric loading conditions, it may be necessary to locate the geometric centre of the footing deliberately away from the axis of the supported elements, so that the soil pressure and the expected settlement will be nearly uniform. The safe bearing capacity of soils is usually given with respect to service load conditions.

Foundations are not easily accessible for periodic inspection and maintenance and, therefore, careful considerations must be given to the durability aspects. The recommendations on durability (*7.1* and *7.2*) and especially minimum cement content (Appendix A) are relevant. While deflection control, (*see 22.2*) may be neglected in the case of footings, the control of crack widths may have to be considered. Clause *34.3.2* recommends that for particularly aggressive environments, such as the severe category in Table 19, the assessed surface crackwidth should not exceed 0.004 times the nominal cover to main steel. As Table 19 places "buried concrete in soil" in the moderate category of exposure, it will be sufficient to restrict the crackwidth to 0.3 mm (*see 34.3.2*) in a majority of footings, and accordingly the simplified detailing rules given in *25.3.2* will serve the purpose of crackwidth control, provided the footing is not exposed to aggressive chemicals in the soil. Increased cover thickness may also be necessary if the soil is contaminated with harmful chemicals (*see 25.4.2*).

*33.1.1* Though the Code states here that the design requirements (that is, flexural strength, shear strength and development lengths) should be satisfied at every section, specific locations for considering the bending strength, shear strength and development lengths are indicated in the clauses following this clause.

The footings on slopes and at different levels shall satisfy the requirements specified in IS : 1905-1980*.

*33.1.2* THICKNESS AT THE EDGE OF FOOTING

*33.1.3* This requirement is based on the dispersion of internal pressure in the concrete pedestal.

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*Code of practice for structural safety of buildings: Loading standards (*revised*).

*Code of practice for structural safety of buildings: Masonry walls (*second revision*).

*33.2 Moments and Forces*—For footings supported on soil, it is reasonable to assume, constant or uniformly varying soil reaction for axial and eccentric loading conditions, respectively.

Safe bearing capacity of soil corresponds to service loads. Appropriate factors should be applied to the moments and shears arising from loads and soil reactions when the footing is proportioned by using the limit state methods.

*33.2.1* For computation of shear in footings supported on piles (*see also 33.2.4.2*).

*33.2.2* Recommendations for the inscribed square section in the place of circular or octagonal section is conservative (*see* Fig. E-42).

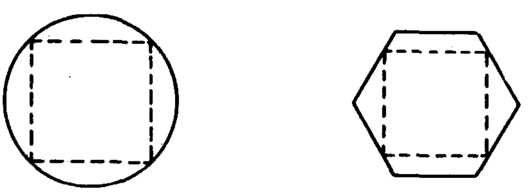

*Fig. E-42 Face of the Column for Computing Stresses in Round and Octagonal Columns*

*33.2.3* BENDING MOMENT

*33.2.3.1* The moments acting at a given section may be determined by assuming constant or uniformly varying pressure distribution from the soil, in the case of footings resting directly on soils. For footings supported on piles, the moments may be computed by assuming that the reaction from any pile is concentrated at the centre of the pile (*see 33.2.1*).

*33.2.3.2* For the purpose of design, the largest value of bending moments may be computed for the critical sections indicated in the Code. The critical sections specified for bending moments reflect the likely flexural behaviour of the footing under the particular type of structure being supported (*see* Fig. E-43). However, *33.1.1* and *33.2.4.3* read along with this clause imply that additional checks will be required whenever there is an abrupt change in the section.

*33.2.4* SHEAR AND BOND

*33.2.4.1* Design for shear in footings is covered by this Clause. It is envisaged that shear failure can occur either in an inclined cracking mode across the full width of the footing or by a punching shear failure of truncated cone or pyramid around the column, stanchion or pier.

The effect of punching shear around the periphery of the loaded area must be investigated in a similar manner to that described in *30.6* for flat slabs (excepting *30.6.2.2*) and, if the shear resistance of the concrete alone is insufficient, suitable reinforcements must again be provided to resist the balance of the shearing force. The critical plane for punching shear is located at a distance $d/2$ from the loaded area on the basis that the critical section for bending moment defined in *33.2.3.2* encloses the reaction area (*see* Fig. E-43).

*33.2.4.2* The critical section for shear (as in a flexurally loaded element) may be taken at a distance $d/2$ from the face of the column, when the footing rests on piles. The following cases must be reckoned in investigating shear stresses in pile cap:

a) Centre of pile is $\frac{D_p}{2}$ away from the section—the entire reaction from pile is reckoned;

b) Centre of pile is $\frac{D_p}{2}$ inwards to the section—the entire reaction is ignored; and

c) For intermediate positions of pile centres—straight line interpolation of pile reaction between full value at $\frac{D_p}{2}$ outside the section and zero value at $\frac{D_p}{2}$ inside the section is to be considered.

*33.2.4.3* Development length must be checked at the face of column and elsewhere where bars are terminated or the effective depth changes. (Normally a further single check midway down the slope of a splayed base will suffice).

The anchorage requirements for curtailed bars are as follows:

For curtailment, reinforcement shall

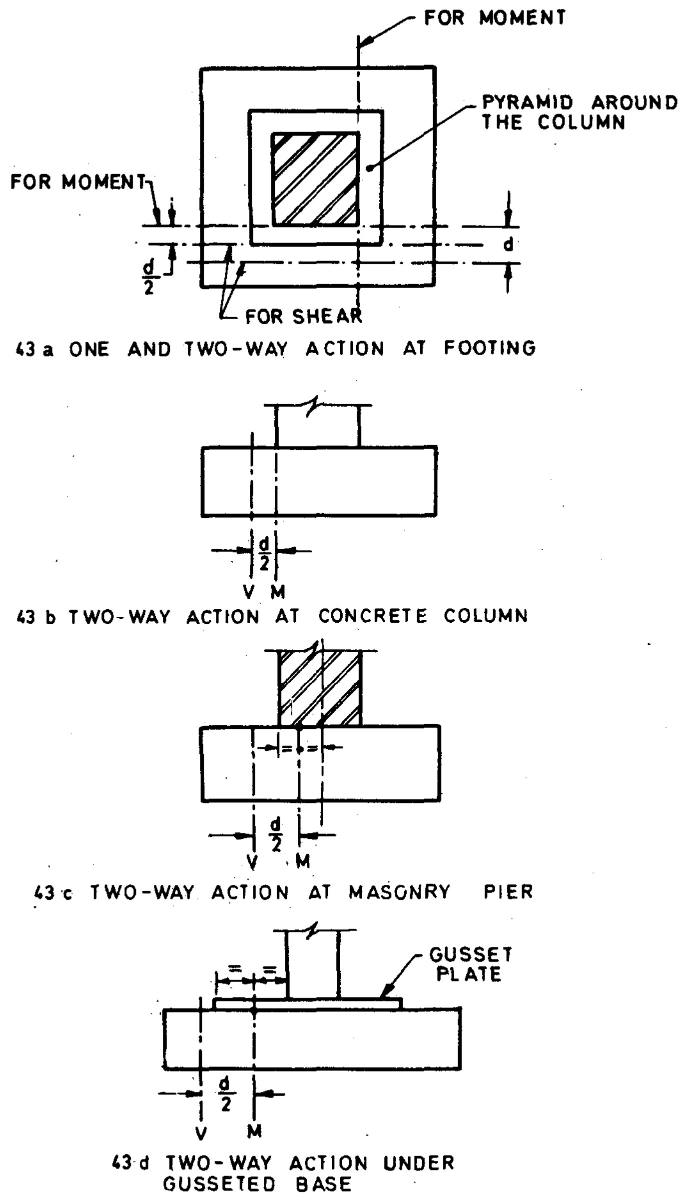


*Fig. E-43 Critical Section for Bending Moment and Shear in Footing*

extend beyond the point at which it is no longer required to resist flexure, for a distance equal to the effective depth of the member or 12 times the bar diameter (*see 25.2.3.1*).

(*See also 25.2.4*). Adequate end achnorage should be provided for tension reinforcement in flexural members where reinforcement stress is not directly proportional to moment, such as sloped, stepped or tapered footings. Where end anchorage is not adequate, cross-bars may be welded to the main steel near the edge or vertical face of the footing.

*33.3 Tensile Reinforcement*

*33.3.1* REGARDING SUB-PARA (c) — In the case of rectangular footings designed for two-way action, the curvature in the direction of short span at a section is sharper in the region near the column. To account for this effect, the reinforcement in the central band is to be increased by an amount $\frac{2}{\beta+1}$.

*33.4 Transfer of Load at the Base of Column*—In general, forces may be transmitted from column to footing in the following manner:

a) Compressive forces—by bearing on concrete surface and also by reinforcements;
b) Tensile forces uplift—by reinforcements properly anchored into column as well as footing; and
c) Lateral forces—by shear friction, shear keys etc.

Though all the above types of forces should be transferred from column to footing, the Code recommendations are confined to compressive forces only.

The compressive stresses in the concrete at the base of the column must be transferred by bearing. If the design values of bearing stresses are exceeded, the reinforcements or dowels should be detailed so as to take up the remaining forces and should be extended across the interface into the column as well as footing so that sufficient development lengths are provided on each side (*see 33.4.2*).

The basic design bearing stress on the footing may be increased by $\sqrt{\frac{A_1}{A_2}}$, taking advantage of confinement of the bearing area (triaxial state of compressive stresses) in the immediate vicinity of the loaded area of the footing. The concrete around the loaded area in the footing provides considerable support through confinement. The Code criteria for the determination of $A_1$ is shown in Fig. E-44.

Usually the depth of the footing will be governed by punching shear (*see* 30.6) or other consideration but not by the bearing

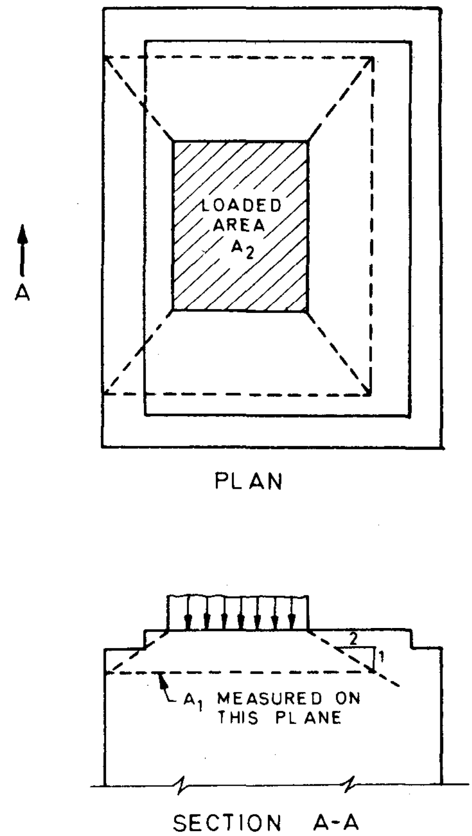


*Fig. E-44 Determination of $A_1$ in Footing*

stresses. However, the factor $\sqrt{\frac{A_1}{A_2}}$, cannot be increased indefinitely by providing very thick footings and the upper limit of $\sqrt{\frac{A_1}{A_2}} = 2$ will impose a corresponding limit on the bearing strength.

*33.4.1* —

*33.4.2* For anchoring of bars in compression, *see 25.2.2.2.* Anchoring of tension bars will be governed by *25.2.2.1.*

*33.4.3* —

*33.4.4* —

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SECTION 5

# STRUCTURAL DESIGN
## (LIMIT STATE METHOD)

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# SECTION 5 STRUCTURAL DESIGN (LIMIT STATE METHOD)

*34.* SAFETY AND SERVICEABILITY REQUIREMENTS

*34.1 General*—The object of limit state design is stated in the Code and can be paraphrased as 'the achievement of an acceptable probability that a part or whole of the structure will not become unfit for its intended use during its life time owing to collapse, excessive deflection, cracking, etc, under the action of all loads and load effects'.

A state is any set of conditions pertaining to a structure. A limit state is a state at which one condition pertaining to a structure has reached a limiting or critical value.

A limit state is attained when the loads or actions of a structure are such that their effects (in terms of stresses in the members of the structure, or of deflection or deformation of the structure or member) exceed a specified limiting value.

The limit states can be placed in the following categories:

a) The ultimate limit states, which are those corresponding to the maximum load carrying capacity or where exceedance results in complete unserviceability; examples are:
   1) loss of equilibrium of a part or the whole of the structure when considered as a rigid body;
   2) rupture of critical sections of the structure due to exceedance of material strength (in some cases reduced by repeated loading) or by deformations;
   3) transformation of the structure into a mechanism collapse;
   4) loss of stability (buckling etc);
   5) elastic, plastic or creep deformation, or cracking leading to a change of geometry which necessitates replacement of the structure.

   The ultimate limit state may equally be caused by the sensitivity of the structure to the effects of a repetition of loads, fire, explosive pressure, etc, it is then necessary to consider such effects in the structural concept.
   (*see* also comments on *34.2*)
b) Serviceability limit states; examples:
   1) excessive deformation with respect to normal use of structure;
   2) premature or excessive cracking;
   3) excessive displacement without loss of equilibrium; and
   4) excessive vibrations.

In a structure, a limit state may be reached as a result of a number of random factors affecting the safety factors which combine or originate. Examples are:

a) In the uncertainty of values taken into account for the strength of the materials used;
b) In the uncertainty of the realization of the assumed geometry of the structure and sections;
c) In the uncertainty of action—permanent loads or variable imposed loads, or imposed deformations impossible to foresee accurately for the entire intended life of the structure; and
d) In the departure of the actual loading effects from the calculated values.

The Code gives the design requirements into two groups, namely:

a) those concerning safety (ultimate limit state) (*34.2*), and
b) those concerning serviceability limit state (*34.3*).

Deflection and cracking limitations, being generally applicable for serviceability requirements are covered in full, but other limit states depending upon the speciality of the structure must be foreseen and provided for in design.

In the Code, care has been taken at present to ensure that the structures designed in accordance with the provisions of limit state design have sensibly the same degree of safety as those designed by using the working stress method.

*34.1.1* It is normally adequate to consider three limit states, namely, collapse, deflection and cracking for the design of normal reinforced concrete structures in general building construction. The most critical limit state for design of reinforced concrete structures is found from experience to be the limit state of collapse. Therefore, normally such structures may be designed initially on the basis of ultimate strength (that is, safety against collapse) and then checked for deflection and cracking limit (that is, serviceability criteria). In the Code, indirect checks for ensuring control of deflection and crack widths are given and these simple rules will be adequate for most cases. In addition to the three limit states mentioned above, it may be necessary to consider a few more in the case of structures with unusual or special functional requirements (*see 34.4*).

*34.1.2* The terms 'characteristic values' and 'design values' have a bearing on the uncertainties in establishing the intended strength of the structure and the anticipated loads. These uncertainties arise out of the following:

a) Inherent variability of loading and in the properties of the material, the latter arising in the manufacture. The characteristic values for material strengths and loads are introduced to take care of this type of uncertainty or variability.
b) Deviations may arise from construction faults, errors in mix proportions and inaccuracies in design assumptions and analysis procedures. This type of uncertainty is taken care of through the use of separate partial safety factors for loads and materials (*see 35.3* and *35.4*). The 'design values' are obtained by applying partial safety factors to the characteristic value for loads and material strength.

Characteristic strengths and characteristic loads are used as reference values in the limit state design. The definitions given in statistical terms in *35.1* and *35.2* are somewhat rigorous and are intended to account for the known, inherent variability. However, in the absence of adequate statistical data, the values of characteristic loads are taken as the nominal loads given in the relevant Indian Standard Specifications for loads. For the same reason, the characteristic strength of steel is taken as the minimum yield stress or 0.2 percent proof stress whichever is less specified in the relevant specifications. However, for concrete, the characteristic strength as defined in *35.1*, has been used in the Code (with required quality control and acceptance criteria).

Design strengths and design loads are obtained from characteristic strengths and characteristic loads, respectively, through the use of a set of partial safety factors (*see 35.3* and *35.4*), which are intended to account for the uncertainties due to deviations from intended construction practices, the approximations introduced in analysis and design, etc. Note that instead of a single load factor used in the ultimate strength method in the 1964 version of the Code, now the Code uses separate partial safety factors on loads and on material strengths.

*34.2 Limit State of Collapse*

a) Rupture Due to Overloads—Inadequate margin for the strength of members may result in the rupture of one or more critical sections or in the collapse of the whole structure. The appropriate partial safety factors for material strengths and loads (*see 35.4.1* and *35.4.2*) should be applied to the respective characteristic values, in order that the structure might have the desired margin of safety. In *36.1.1*, the Code permits the redistribution of moments in indeterminate structures, as they possess some reserve strength which can be taken into account.
b) Elastic Instability and Plastic Instability—Failure resulting from elastic instability in beams and columns can be avoided by using the rules given in *22.3* and *24.3* respectively. (Plastic instability may occur in slender columns (*see 24.1.1* for definition of slender column). Designs incorporating 'additional moments' in slender columns, in accordance with *38.7*, will be sufficient to avert plastic instability in a majority of low-rise building frames.
c) Overturning—The stability with respect to the overturning of the structure as a whole is ensured through the application of *19.1*.

The combinations of loads to be considered in design and the recommended values of partial safety factors for those combinations are given in Table 12.

The Code states elsewhere (*see 34.1*) that the structure should be designed to withstand safely all loads liable to act on it throughout its life, and in Table 12 lists the combinations of dead load, live load, wind load and earthquake load to be taken into account, and these will be adequate in most cases. However, in a few isolated instances the possibility of collapse due to causes other than those listed in Table 12 can be foreseen, for example, unusual or excessive loadings, localized damage, dynamic response or conditions during construction. In these cases, the probability of collapse may be minimized by the following methods:

a) By adopting a structural scheme which can accept a decrease, or even complete loss of the structural effectiveness of cer in members although with a reducr.d degree of safety (*see* Ref 3).

b) By the provision of appropriate devices to limit the effects of these accidental occurrences to acceptable levels, for example, controlled venting where explosions may occur, and crash barriers where impact of vehicles can be anticipated.

It is to be noted even if the structures were to collapse due to unforeseen overloading, it should have a ductile failure [this point is taken care of in *37.1(f)*] so as to avoid excessive loss to life and property.

*34.3 Limit States of Serviceability*

*34.3.1* Deflection — Excessive deflection at service load may cause damage to non-load bearing elements, such as partition walls, discomfort to the users and may adversely affect the appearance of the structure or functioning of sensitive equipment (*see also* comments on *22.2*). The Code recommendations are confined to flexural members (*see 41.1*) as deflection is not likely to be critical in other cases. Even in the case of flexural members, deflection control through the use of span to depth ratios, given in *22.2*, will be adequate in normal cases. In critical cases, deflections may be computed in accordance with Appendix E and compared with the limiting values given in *22.2*.

*34.3.2* Cracking — Design for the serviceability limit state of cracking involves an analysis of the section, in which concrete is considered to have no tensile strength, and the design requirement for exposure. The evidence on the effect of crack width on the corrosion of reinforcing steel is conflicting. The limiting surface crack width of 0.3 mm is generally accepted from aesthetic considerations and this limit is also accepted as adequate for the purpose of durability when the member is completely protected against weather or aggressive conditions.

The recommended limit of 0.3 mm (surface crack width) for general use in building construction is related to a reference value for the cover, and this reference value can be taken as 25 mm. For severely aggressive environment, the crack width is preferably limited to 0.1 mm, but in general more cover is also provided. Therefore, the Code recommends a limiting crack width of 0.004 times the nominal cover nearest to the main reinforcement, which leads to 0.1 mm when nominal cover is 25 mm.

The acceptable limits of cracking indicated in the Code are intended for general use. Crack width limitation depends upon functional requirements of different structures:

a) *Bins and Silos* — The crackwidths in walls of bins shall not exceed the following:

| | |
|---|---|
| Where watertightness is required | = 0.1 mm |
| Otherwise | = 0.2 mm |

b) *Tanks for Storage of Water and Other Liquids* — According to IS : 3370 (Part II)-1965*, resistance to cracking should be ensured by limiting the tensile stresses in concrete assuming the section to be uncracked. In addition, the widths of possible cracks are restricted indirectly through a normal strength check, but with reduced values of permissible stresses in reinforcements. Limiting values of crack widths have not yet been given explicitly.

*Code of practice for concrete structures for the storage of liquids: Part II Reinforced concrete structures.

Although several suggested approaches are available for computation of crack widths, a wholly satisfactory or generally acceptable procedure is not available. A reinforcement detailing guidance was therefore considered more practical for this purpose and no procedure is given for calculating crack width.

The control on crack width is exercised through the reinforcing detailing requirements (spacing of bars) specified in *25* and its sub-clauses. This indirect control is applicable when the limiting crack width is 0.3 mm (mild conditions of exposure). For particularly aggressive environments (severe category in Table 19), *see* the Note in Table 10 of the Code.

Absolute limits to the widths of cracks cannot be predicated easily, because cracking is influenced by many factors and is a variable phenomenon. The limits given in the Code merely require an acceptable probability of the crack widths not being exceeded. Keeping this in view, the possibility of some cracks (say 20 percent of the total number) being wider than the limiting value must be accepted, unless special precautions like prestressing is undertaken.

*Calculation of Crack-Width (Reference 3)*— The widths of flexural cracks at a particular point on the surface of a member depend primarily on three factors:

a) The proximity to the point being considered of reinforcing bars perpendicular to the cracks;

b) The proximity of the neutral axis to the point being considered; and

c) The average surface strain at the point being considered.

The following formula gives the relationship between crack width and the three principle variables which give acceptably accurate results in most normal design circumstances. However, the formula needs to be used with caution for members subjected to dominately to an axial tension.

Design surface crack width (*see* Fig. E-45)

$$W_{cr} = \frac{3a_{cr}\,\epsilon_m}{1 + 2\left(\dfrac{a_{cr} - C_{min}}{D - x}\right)}$$

where

$a_{cr}$ = distance from the point considered to the surface of the nearest longitudinal bar,

$C_{min}$ = minimum cover to the longitudinal bar,

$\epsilon_m$ = average strain at the level considered,

$D$ = overall depth of the member,

$x$ = depth of the neutral axis.

The average strain at the level at which cracking is being considered is given by:

$$\epsilon_m = \epsilon_1 - \frac{0.7 b_t D(a' - x)}{A_s(D - x) f_s} \times 10^{-3}$$

where

$\epsilon_1$ (strain at the level being considered)

$$= (D - x)/(d - x) \left(\frac{f_s}{E_s}\right)$$

$b_t$ = The width of the section at the centroid of the tension steel;

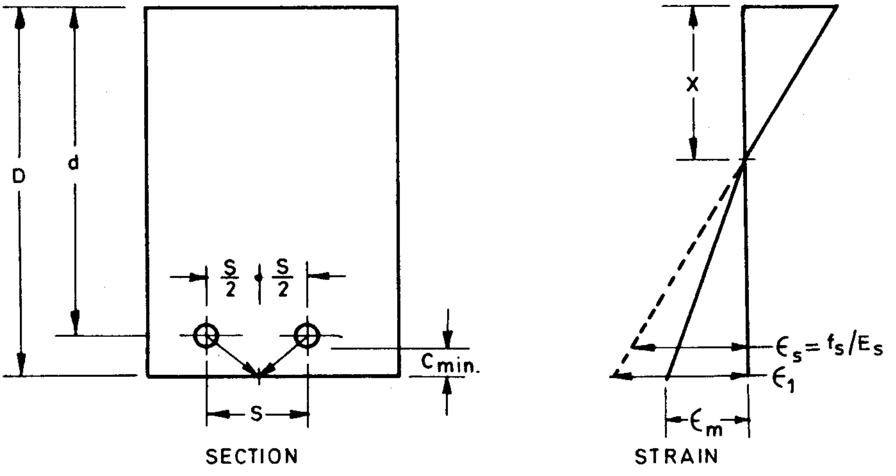



*Fig. E-45 Crack Width in Beams*

$a'$ = The distance from the compression face to the point at which the crack width is being calculated;

$A_s$ = The area of tension reinforcement;

$f_s$ = Service stress in tension reinforcement (N/mm²)

$E_s$ = Modulus of elasticity of steel; and

$d$ = Effective depth.

*34.4 Other Limit States*—On a few occasions, vibration or fatigue may have to be considered explicitly (*see also* comments on *17.6*).

Lateral drift limitations for tall buildings subjected to earthquake forces and wind loads are given in IS : 4326-1976*, IS : 1893-1975† and Reference 40, respectively. Limits on lateral drift may be necessary to minimize:

a) damage to non-structural parts, such as partitions, finishes and window panes; and

b) discomfort or alarming effects to the occupants of the buildings.

## *35.* CHARACTERISTIC AND DESIGN VALUES AND PARTIAL SAFETY FACTORS

*35.1 Characteristic Strength of Materials*—Characteristic strength has been introduced in order to provide for the inherent variability of the material property, arising during its manufacture (*see also* comments on *34.1*).

Throughout the Code, it is recognized that when a large number of samples say, a given grade of concrete manufactured even under stable conditions of control are tested, the results will show some variation. For the strength of concrete, it is usual to assume that the plot of compressive strength (on $X$-axis) against the frequency of occurrence (on $Y$-axis) will follow the 'normal' or 'Gaussian' distribution, shown in Fig. E-4.

Characteristic strength of concrete,
$f_{ck} = f_t - k.s$

where

$f_t$ = arithmetic mean of the different test results

$k$ = (coefficient depending on the probability) = 1.65

$s$ = standard deviation (*see* Table 6).

It is to be noted that the concrete mix has to be designed for the target mean strength greater than the specified characteristic strength.

The characteristic strength is used as a reference value for specifications as well as design purpose throughout the Code, and as far as possible, all requirements are given in terms of characteristic strength. Some examples follow:

a) In Table 2, the grade designation of concrete is related directly to the characteristic strength of concrete.

b) The entire section on acceptance criteria for concrete (*see 15.1* to *15.7*) has been written, taking the characteristic strength of concrete as a reference value.

c) Derived quantities, such as flexural strength of concrete (*see 5.2.2*) and modulus of elasticity of concrete (*see 5.2.3.1*) are expressed in terms of the characteristic strength $f_{ck}$.

d) Design formulae and other related information for important topics, such as flexure (*see 37.1*) and short columns (*see 38.3*) are also given in terms of the characteristic strengths.

According to *34.1*, 'design values' should be obtained from the characteristic values through the use of partial safety factors. However, when the design formula is given directly in terms of characteristic strength, the recommended partial safety factors would have been incorporated already. For example, partial safety factors have already been incorporated in the short-column formula given in *38.3*.

*35.2 Characteristic Loads*—As in the case of characteristic strength, the term characteristic load has been introduced in order to provide for the inherent variability associated with the assessment of maximum loads. It represents reasonably upper or lower limits to the expected range of pattern of loads.

The values of characteristic loads should be multiplied by relevant partial safety

*Code of practice for earthquake resistant design and construction of buildings (*first revision*).

†Criteria for earthquake resistant design of structures (*third revision*).

factors (*35.3.2, 35.4.1,* and Table 12) to obtain the design loads.

When statistical data are not available, the loading values specified in appropriate Code (IS : 1911-1967*, IS : 875-1964† and IS : 1893-1975‡) may be taken as characteristic loads.

*35.3 Design Values*

*35.3.1* MATERIALS — The characteristic strength of a material has been defined in such a way that the uncertainties due to the inherent variability of the property of a material arising in the manufacture is accounted in a systematic manner. However, the strength of a material (say concrete or reinforcing steel) in the actual structure may be somewhat less than the characteristic strength, this being mostly due to errors or deviations arising in construction.

The strength of concrete, for example, might be reduced by errors in the mix proportions, by the presence of impurities or by inadequate compaction or curing. The partial safety factor $\gamma_m$ relates the possible reduction in the material strength in the actual structure to the values of the characteristic strengths. The values of $\gamma_m$ are fixed separately for each material and each limit state (*see 35.4.2*). The recommended values of $\gamma_m$ are based on the assumption that adequate standards of construction (say, tolerance etc) specified in Section 3 of the Code would be ensured.

When a design formula is given in the Code directly in terms of characteristic strength, the partial safety factors would have been built into the formula, and it will be unnecessary to apply partial safety factors again, unless specifically demanded by the Code.

Some relaxation of these load factors may be allowed when considering the effects of unusual or excessive loadings, localized damage, dynamic response or conditions during erection.

*35.3.2* LOADS — The term 'Characteristic load' (*see 35.2*) has been introduced in the Code in order to provide for the inherent variability associated with the assessment of maximum loads. However, during the expected life span of the structure the actual maximum may be larger than the characteristic load due to the following reasons:

a) The lack of dimensional accuracy in actual construction may lead to larger or smaller loads which in turn may increase the load effect on critical sections;

b) Approximations and simplifying assumptions are made in the analysis and the proportioning of structure which may in turn affect the computed values of the forces, moments or stresses in the structures;

c) There may be an unforeseen increase in the loads though there may not be any change in the use of the building; and

d) The load on a member may be increased due to the effect of creep, shrinkage and temperature, but the Code permits the designer to ignore such effects in normal building types (*see 17.5.1*).

Therefore, the characteristic load must be multiplied by the partial safety factor $\gamma_f$, to obtain the design load, which may be greater or lesser than unity. The values of $\gamma_f$ are specified separately for each load combination and limit state (*see* Table 12).

*35.3.3* CONSEQUENCES OF ATTAINING LIMIT STATE — Usually, the limit state of collapse (*34.2*) will be the one that might possibly lead to disastrous consequences, such as loss of life and property. The amount of increase in the partial safety factor should be determined by the designer, based on judgement and past experience. The partial safety factors may be increased either singly or together, but the usual practice is to adopt an increased value of $\gamma_f$ (*see* IS : 1893-1975* which suggests increased values of the load factors for various magnitudes of expected damages with respect to earthquake resistant designs).

*35.4 Partial Safety Factors* — By the introduction of two partial safety factors, one

*Schedule of unit weights of building materials (*first revision*).

†Code of practice for structural safety of buildings: Loading standards (*revised*).

‡Criteria for earthquake resistant design of structures (*third revision*).

*Criteria for earthquake resistant design of structures (*third revision*).

applied to the material strength, the other to loads, it has been possible to develop a design method which ensures that the probability is acceptable that the limit states will not be reached. The main reason for adopting two partial safety factors instead of one overall factor is to enable that the uncertainties in assessing the loads and their influence on the structure to be considered separately in design from the uncertainties associated with performance of construction materials. Subsequently it will simplify the incorporation of improvements to the Code as and when new knowledge becomes available with regard to loads and materials.

*35.4.1* PARTIAL SAFETY FACTORS $\gamma_f$ FOR LOADS—The partial safety factor for loads, $\gamma_f$, is required for obtaining the design load from the characteristic load (*see 35.3.2*). The values listed in Table 12 have been arranged in such a way that the structure designed in accordance with the provisions for limit state design (Section 5), will have sensibly the same degree of safety and serviceability as structures designed by using the working stress method (Section 6). The values of $\gamma_f$ have been arranged in such a way that only a minimum number of analysis will be required for carrying out the design.

LOAD COMBINATIONS—Clause *17.7* which states that the combination of loads shall be as given in IS : 875-1964* (*see also 19.1* and *19.2* and their comments).

For frames, the arrangement of live loads is given in *21.4.1*. Although all the relevant load combinations must be considered in design, the first one in Table 12, that is, ($DL + LL$) is most likely to govern the design in ordinary buildings; the second combination ($DL + WL$) in structure, such as chimneys and cooling towers where the lateral loading (wind or earthquake) is the primary imposed load.

VALUES OF SAFETY FACTORS $\gamma_f$—A uniform load factor of 1.5 is recommended by the Code for the load combination ($DL + LL$) when collapse is to be considered. The Code Panel went into this question and decided to adopt this value on the following basis:

a) The factor 1.5 may be interpreted as a device for ensuring the desired margin of safety against collapse, with respect to the combined effects of dead load and live load taken together, instead of providing distinct factors for dead load taken separately. This will also go in conformity with the recommendation of the European Concrete Committee (Reference 7).

b) As the designs are based on "elastic analysis" of the structure (*see 4.1*), one analysis will be sufficient for the limit state of collapse as well as the limit state of serviceability when ($DL + LL$) is considered; the design moments are obtained simply through a multiplications by a suitable constant; and

c) Sometimes, a lower factor, say 1.4, is suggested for the dead load and a higher one, say 1.6, for the live load, the ostensible justification being that more is known about the magnitude of dead load and therefore, a lower factor of safety will be appropriate for that. However, there is some evidence (*see* Ref 41) that a uniform factor for ($DL + LL$) ensures a better degree of safety. The Code recommends a uniform factor for the combination ($DL + LL$), also to simplify the design procedures.

In those structures where stability against overturning may become critical (*see also 19.1* and its comments), the unfavourable case corresponding to the application of minimum value of dead load should be considered. Here, the Code recommends $\gamma_f = 0.9$ for dead load, for the following reasons:

a) The size of the member may be smaller than the nominal values shown in drawings; and

b) The densities of materials may be less than those assumed in design calculations.

In the third load combination, that ($DL + LL + WL$), there is only a smaller probability that all the imposed loads will reach their characteristic values simultaneously, and this is reflected in the reduced value of $\gamma_f$. As it is also unlikely that both wind loads and earthquake forces will occur simultaneously, it will be sufficient to consider each of them in turn, in combination with dead load and live load (*see* Note 2 to

*Code of practice for structural safety of buildings: Loading standards (*revised*).

Table 12). However, it is to be remembered that the reduced live load (*LL*) in accordance with IS : 1893-1975* needs to be used when (*DL* + *LL* + *EL*) are combined.

For earthquake effects, IS : 1893-1975* must also be considered (*see 17.4*), and it recommends a similar approach, though the value of the load factor there is higher. This is because the safety factor given in IS : 1893-1975* is applied only on loads, whereas the Code has separate safety factors for loads ($\gamma_f$) and materials ($\gamma_m$). While applying either of these Codes, this difference should be kept in mind and the safety factors should be applied in a consistent manner.

For computing the deflections due to creep, it will be appropriate to consider only that portion of the live load that is likely to be of a permanent nature (*see* Appendix E). The Code does not give a specific recommendation in this regard, but it is suggested that the following proportion of the specified characteristic imposed loads may be taken as a guide as being of long duration depending upon the type of the occupancy:

| | |
|---|---|
| Residential floors | 0.7 |
| Office floors | 0.6 |
| Hospital floors (wards and corridors) | 0.5 |
| Theatres and library | 0.8 |
| School floors | 0.5 |
| Assembly floors | 0.5 |
| Storage and industrial floors | 1.0 |

However, the possibility of later changes of occupancy or use should be kept in mind. The intention of Note 2 to Table 12 is to draw specific attention to this aspect and it will not be necessary to change $\gamma_f$ applicable to serviceability limit states.

*Cases not listed in Table 12*

a) SHRINKAGE, CREEP AND TEMPERATURE EFFECTS — The Code requires that the effects of shrinkage, creep and temperature should be taken into account if these are likely to affect the safety and serviceability of the structure (*see 17.5*), but permits the designer to ignore these effects in ordinary structures (*see 17.5.1*). In those few instances where such effects are considered explicitly it is suggested that the following recommendations, (Ref 28) be adopted in the absence of any other Code provisions:

Where structural effects ($T$) of differential settlement, creep, shrinkage or temperature change may be significant in design, required strength, $U$ shall at least be equal to:

$U = 0.75\ (1.4\ DL + 1.4\ T + 1.7\ LL)$,

but $U$ shall not be less than

$1.4\ (DL + T)$.

For serviceability limit states, $\gamma_f$ can be taken as unity while considering creep, shrinkage and temperature effects.

b) LOADS CAUSED BY MISUSE OR ACCIDENT — In a few isolated cases the possibility of collapse due to causes other than those listed in Table 12 can be foreseen for example, explosive pressures and vehicle impact. Though the Code states in *34.1* that the structure should be designed to withstand safely all loads liable to act on it throughout its life, it does not give recommendations regarding the design concepts for loads caused by misuse or accident. The following information (Ref 3) may be used as a guide:

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 forces that could arise due to an extreme cause, but it should not be damaged to an extent disproportionate to the original cause. In addition, due to the nature of a particular occupancy or use of a structure (for example, flour mill, chemical plant) it may be necessary in the design concept or a design reappraisal to consider the effect of a particular hazard and to ensure that, in the event of an accident, there is an acceptable probability of the structure surviving after the event, even if in a damaged condition. If in the design it is necessary to consider the probable

*Criteria for earthquake resistant design of structures (*third revision*).

effects of excessive loads caused by misuse or accident, the $\gamma_f$ factor (for the ultimate limit state) should be taken as 1.05 on the defined loads, and only those loads likely to be acting simultaneously need be considered. Again, when considering the continued stability of a structure after it has sustained localised damage, the $\gamma_f$ factor (for the ultimate limit state) should be taken as 1.05. The loads considered should be likely to occur before temporary or permanent measures are taken to repair or offset the effect of the damage. When considering the effect of excessive loads or localized damage, the values of $\gamma_m$ for ultimate limit state may be taken as 1.3 for concrete and 1.0 for steel.

*35.4.2* PARTIAL SAFETY FACTOR $\gamma_m$ FOR MATERIAL STRENGTH

*35.4.2.1* For definitions relating to the characteristic strength, design strength and partial safety factor $\gamma_m$, *see 35.3.1.*

The reasons for introducing $\gamma_m$ for the strength of concrete are mentioned in the comments on *35.3.1.* In the case of reinforcing steel, $\gamma_m = 1.15$ accounts for:

a) reduction in the strength of the cross-section of the member as a result of inaccurate positioning of steel; and

b) reduction in the strength of the steel bar due to manufacturing defects, such as deviations from the nominal diameter.

A higher value of $\gamma_m$ is recommended for concrete as it can be expected that the actual strength of concrete in the structure may deviate from that determined from tests by an amount more than that can be expected for reinforcing steel, owing to several construction operations, such as transporting, placing, compacting and curing of concrete.

*35.4.2.2* For the analysis of structures, it may be necessary to determine the elastic constants such as $E_c$, the modulus of elasticity. Irrespective of the limit state being considered during the analysis, the characteristic strength of concrete should be used in the expression for $E_c$, and the factor $\gamma_m$ should not be introduced (*see 36.1*).

When properties other than compressive strength are derived from cube test results, the factor $\gamma_m$ should be applied to the derived values. For example, if the tensile strength of concrete is taken as $f_{cr}$, the design tensile strength should be $\frac{f_{cr}}{\gamma_m}$.

## *36.* ANALYSIS

*36.1 Analysis of Structure—See* commentry on *21* and its sub-clauses.

Redistribution of moments may be carried out for rigid frames, statically indeterminate frames and continuous beams if the linear-elastic analysis or the substitute frame and portal methods are used for determining the internal forces and actions in these structures. If the moment and shear coefficients of Tables 7 and 8 are used for the design of continuous beam, redistribution of moments is not permitted (*see 21.7*) as the same has already been considered.

For the determination of elastic properties of concrete, *see also* Comments on *35.4.2.2.*

The relative stiffnesses of members, required for the analysis of indeterminate structure, may be determined on the basis of any one of three assumptions given in *21.3.1.* The moduli of elasticity of steel and concrete are given in *4.6.3* and *5.2.3.1* respectively.

*36.1.1* REDISTRIBUTION OF MOMENTS IN CONTINUOUS BEAMS AND FRAMES—From the title of the Clause, it can be inferred that redistribution of moments is applicable to continuous beams and frames, that is, statically indeterminate structures and not to simply supported beams.

In statically indeterminate structures, redistribution of bending moment occurs before failure as a result of inelastic deformation of the reinforced concrete sections. This redistribution permits a reduction in the support moments of a continuous beam which in turn would reduce the congestion of reinforcements at the supports. Such redistribution of moments occurs whenever incipient failure is reached whether at a support or in the span as explained further below.

In the limit state design method, moment redistribution is permitted up to a reduction of 30 percent of the numerically largest moment in a member [*see para* (c) of *36.1.1*] whereas in the working stress method redistribution up to ± 15 percent is allowed (*see 43.2*). This liberalized provision for limit state method is not based on any new concept in the method of design but should be interpreted as an increase justified on theoretical grounds and on the basis of corresponding restrictions on relevant details.

According to *21.5.1* redistribution of moments is not permissible when the coefficients given in Table 7 are used for calculation of bending moments as the same has already been included.

BASIS AND JUSTIFICATION — An under-reinforced section subjected to a gradually increasing bending moment behaves elastically at the initial stages but displays a pronounced non-linear and somewhat plastic behaviour at high values of bending moments. The magnitudes of deformations at yield and at failure would be determined by the relative quantities of steel and concrete in the section and their stress-strain characteristics. However, for an under-reinforced section the moment *versus* rotation plotted will resemble the dotted line shown in Fig. E-46 which is usually idealized by a bi-linear diagram shown by the full lines. In other words, the behaviour of an under-reinforced section can be idealized with a linear plotted of moment *versus* rotation up to a stage when the plastic moment capacity $M_p$ is reached and thereafter by a line of limited length up to failure parallel to the $X$-axis.

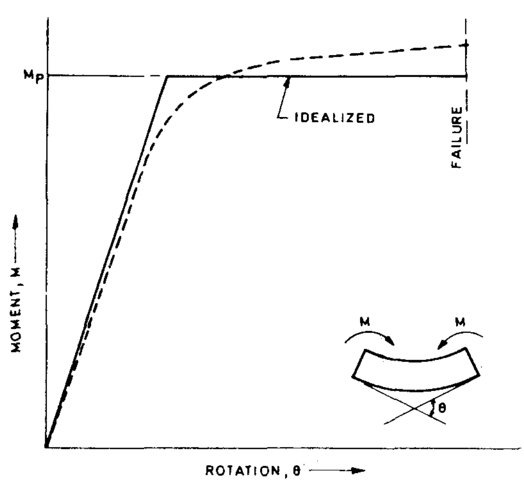



*Fig. E-46 Moment Rotation Plot for Under-Reinforced Sections*

Consider an indeterminate beam, say a fixed beam, restrained at both ends and with a point load at the centre. As the load increases, the beam behaves elastically until the plastic moment on one or more critical sections is reached, that is where the steel has yielded (at support). Assume that the ultimate moment capacity of the beam at supports, $M_p$ is half of that at mid span. As the load is increased gradually, the beam will behave elastically until the steel at the critical section (that is, support) yields, or in other words, the plastic moment $M_p$ is reached at the support. Referring to Fig. E-47, it can be seen that this condition is reached when the load $P$ attains a value $P_1 = \frac{8\,M_p}{l}$. At this stage, even though the support moment has attained the value $M_p$ the beam can sustain a further increase in load as the support sections will support a moment $M_p$ with increased rotations, that is, the plastic moment $M_p$ can be sustained at supports at a constant moment, but the rotation will increase along the horizontal (idealized) line shown in Fig. E-46. As the load is increased further and if sufficient rotation capacity was available at support, the fixed beam will act as a beam with hinged ends. Eventually, as the moment at mid span reaches the full capacity 2 $M_p$, the beam will not be able to sustain any further increase in load since its full moment resistance capacity is now reached. Referring to Fig. E-47, statical calculations indicate that total collapse will take place when the load reaches a value of $\frac{12\,M_p}{l}$. Note that, in a strictly elastic design based on an elastic analysis, the load $P$ would have been restricted to $P_1 = \frac{8\,M_p}{l}$, corresponding to the stage when one of the critical sections would have reached its elastic limit. But a consideration of the total collapse of an indeterminate structure reveals the existence of some reserve strength in most cases.

From design point of view, the reserve strength in statically indeterminate beams and frames can be taken advantage of, simply by calculating the bending moments for a load of, say $P_u = \frac{12\,M_p}{l}$ for the fixed beam

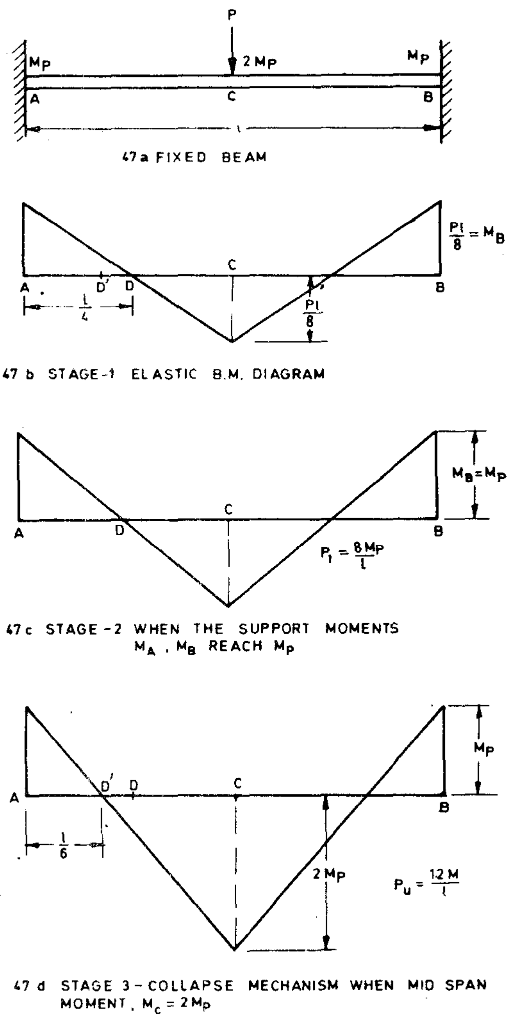



*Fig. E-47 Collapse Load for Fixed Beam with Fixed Plastic Moments*

example, assuming the entire beam to be elastic and then reducing the support moments. The mid span moment is increased by the corresponding amount required to maintain equilibrium. This procedure, explained in Fig. E-48, is called 'redistribution of moments'. Comparison of Fig. E-48 with Fig. E-47 shows that redistribution of moments is equivalent to utilizing the reserve strength in indeterminate beams. However, the advantage of the concept of redistribution of moments is that it is a design procedure and the designer may choose any set of statically equilibrating values for the moment capacities at A, B and C; in other words, one is free to choose the amount of redistribution of moments, say reduction in $M_A$ and $M_B$ provided that statical equilibrium is maintained through the structure.

Paras (a) to (e) of *36.1.1* lay down the procedure and limitations for redistribution of moments. The condition at para (a) is a basic one. The condition at para (b) is intended to ensure that the sections, especially those near the points of contraflexure, do not crack at working loads. Referring to Fig. E-47, for the example considered therein, the point of contraflexure marked D, is at a distance 1/4 from support A, when the beam behaves elastically [*see* Fig. E-47 (b)]. However, at the collapse stage, [*see* Fig. E-47 (c)] the point of contraflexure moves to D′ which is only 1/6 away from the support A. Therefore, when the service loads are placed on the structure, it will be necessary to provide for the negative moments up to a distance AD, which is greater than AD′, the latter being sufficiently away from

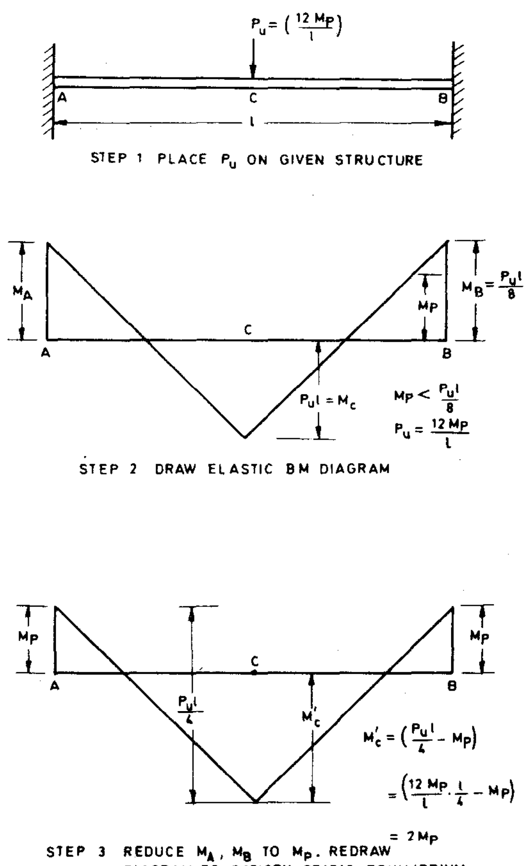



*Fig. E-48 Redistribution of Bending Moments* the redistributed moment diagram. Thus, if the negative reinforcement is provided on the basis of redistributed moment, cracks are likely to appear over the length DD′ when service loads are placed, even though collapse has not taken place. The value of 70 percent, mentioned in para (b) is the ratio of working loads to ultimate loads, with respect to the first combination ($DL + LL$) given in Table 12. The partial safety factor $\gamma_f$ in this case is 1.5 and, therefore, the elastic moment at working load will be $\frac{1}{1.5} = 0.67$, say 0.70 percent of the moment at collapse condition. Accordingly, the envelope of elastic maximum moments must be drawn first, this envelope reduced by 70 percent throughout and then the redistributed moment diagram drawn. The proportioning and detailing should cover the more critical of the two cases at any section namely, (i) elastic moment envelope (with no redistribution) reduced by 70 percent, and (ii) envelope of the redistributed moment diagram.

The reduction of 30 percent of the elastic moment [*see* para (c)] is valid for sagging moments as well as hogging moments, but the equilibrium with external forces must be maintained as required in para (a). The limit of 30 percent has been set from practical consideration, mainly to restrict the demand for rotation capacities of concrete structures at critical sections, although theoretically a reduction of 100 percent would be possible provided equilibrium requirements are satisfied (for example, a span in continuous beam could be reduced to a simply supported span by reducing the support moment by 100 percent and then designed as a simply supported beam). Even on economic considerations, it would rarely be desirable to reduce the bending moments by more than 30 percent. Note that no limit is placed on the amount by which moments can be increased (either at midspan or at supports). The moments can be reduced anywhere along the beam regardless of whether the section is at support or near midspan. Finally, the reduction coefficient of 70 percent referred in para (c) are not complementary. There are independent reasons for choosing them.

Redistribution of the moment at a section can be achieved only if the section has sufficient ductility, that is, rotation capacity. As the rotation capacity of reinforced concrete sections is rather limited, a check has been introduced in para (d). The depth of neutral axis at failure provides a reasonable albeit approximate estimate of the rotation capacity of the section. Also, the demand on the rotation capacity required at a section would increase with the percentage redistribution. These two considerations form the basis of the inequality condition in para (d).

In para (d), $\frac{X_u}{d}$ is the neutral axis depth factor of the section resisting the reduced moment (that is, after redistribution). The inequality should be applied only when the moment at a section is reduced and only the magnitude of $\frac{\delta M}{100}$ without any sign attached, should be inserted. Referring to the formulae for $\frac{X_u}{d}$ given in Appendix E of Code, it can be seen that this factor increases with the steel ratio $\frac{A_s}{bd}$ for singly reinforced sections and with $\left(\frac{A_s}{bd} - \frac{A'_s}{bd}\right)$ for doubly reinforced sections. Therefore, appreciable redistribution of moments will be possible only with under-reinforced sections.

For various types of steel, the limiting values of $\frac{X_u}{d}$ are given in the Note to para (f) of *37.1*. If the sections are designed in such a way that these limiting values are closely approached, the moment reduction ratio $\frac{\delta M}{100}$ cannot exceed 7 percent for mild steel and 12 percent for cold-worked deformed bars ($f_y$ = 415 N/mm²), as against 30 percent maximum, permitted in the Code. However, in a general case, generous overall dimensions with lesser amount of tension steel can be chosen, $\frac{X_u}{d}$ values determined through compatibility conditions for limiting strain values indicated in *37.1* and then the inequality requirement of para (d) of *36.1.1* may be checked.

Finally, the condition in para (d), involving the depth of neutral axis ($X_u$) will rule out the possibility of redistribution (that is, reduction) in moments in a column unless the axial load is very small.

*36.1.2* ANALYSIS OF SLABS SPANNING IN TWO DIRECTIONS AT RIGHT ANGLES—(*see* comments on *23.4* and Appendix B).

## *37.* LIMIT STATE OF COLLAPSE: FLEXURE

*37.1 Assumptions*—The assumptions given in this Clause are valid only for shallow beams (span to depth ratio greater than 2.5). A separate set of assumptions is given for deep beams (*see 28*). Additional assumptions are given for members subjected to axial thrust with or without bending (*see 38.1*). In other words the assumptions given in *38.1* provide a general theory for design of reinforced concrete members covering the entire range between simple bending and direct compression.

ASSUMPTION (a)—It implies linear variation of strain across the depth of the section (*see* Fig. E-49) and has been verified by numerous tests on reinforced concrete beams.

ASSUMPTION (b)—The maximum strain in concrete as defined in assumption (b) is the strain at which the section reaches its maximum moment capacity (*see* Fig. 20). The value of such a strain is liable to large variations depending upon rate of deformation, loading time, grade of concrete, shape of cross-section, percentage of reinforcement etc (Ref 42 and 43). The Code adopts a value of 0.003 5 which will give conservative results in most cases of pure bending and also for compression members subjected to axial loads with large eccentricities that is, which cause tension on one face. For compressive members subjected to axial thrust only or with small eccentricity keeping the whole section under compression, appropriate values of maximum strains are suggested in *38.1*.

ASSUMPTION (c)—The concrete stress distribution in the compression zone may be obtained by using the stress-strain curve of concrete given in the Code (*see* Fig. 21). The shape of the compressive stress distribution in concrete (*see* Fig. E-49) follows the shape of assumed stress-strain curve of concrete (*see* Fig. 21) which is a combination of a parabola and a rectangle. The assumed compressive stress distribution is referred to as stress block. In other words the concrete stress block is parabolic-rectangular in shape which is used for deriving design formulae given in Appendix E. The maximum stress (*see* Fig. 21) is related to characteristic strength $f_{ck}$ by a factor 0.67. This factor has been introduced to account for the observed fact that the apparent strength of concrete in the compression zone of beam or column at failure is approximately 0.85 times the cylinder strength of the same concrete or 0.67 times the cube strength taking cylinder strength as approximately equal to 0.8 times the cube strength. The design

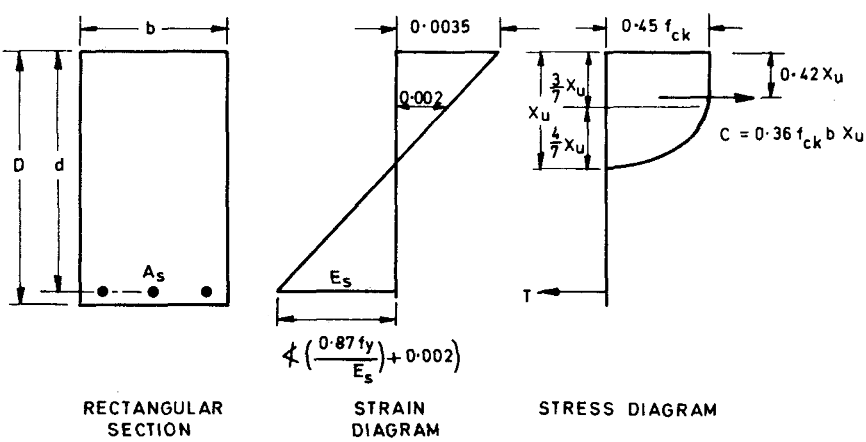



*Fig. E-49 Assumed Stress Strain Conditions at Limit State of Collapse in Flexural Bending* strength equals $0.67 f_{ck}/\gamma_m = 0.45 f_{ck}$ since $\gamma_m = 1.5$. The design stress block is shown in Fig. E-48.

A rigorous analysis of a section from first principles may be made by utilising the set of assumptions given in this Clause. The Code, however, provides design formulae for rectangular and flanged sections (Appendix E), obtained by using the assumption stated in this clause.

ASSUMPTION (d)—Calculation can be made taking the tensile strength of concrete into account, but such refinements are rarely justified.

ASSUMPTION (e)—Cold twisted steel does not have a definite yield point, as can be seen in the representative stress-strain diagram (*see* Fig. 22 A). It is also valid for hard drawn steel wire fabric reinforcement conforming to IS : 1566-1982*. The stress-strain curve for mild steel which has definite yield point is approximated by two straight lines (*see* Fig. 22 B). This diagram can also be used for hot rolled deformed bars conforming to IS : 1139-1966† and rolled steel made from structural steel conforming to IS : 226-1975‡.

ASSUMPTION (f)—This assumption is intended to ensure ductile failure, that is, the tensile reinforcement at the critical section has to undergo a certain degree of inelastic deformation before the concrete fails in compression. In the Code limit on maximum strain, the first term, $\frac{f_y}{1.15 E_s}$ corresponds to the strain at the yield stress $f_y$, for the case of mild steel and other types which have a well defined yield point. However, the cold twisted steel permitted in the Code does not have a well defined yield point and the yield stress $f_y$ is taken as the conventional value of 0.2 percent proof stress. 0.002 (strain) is added so that there is sufficient yielding of steel before failure at a constant stress.

Assumptions (b) and (f) govern the maximum depth of neutral axis in flexural members. The strain distribution across the member corresponding to those limiting conditions is shown in Fig. E-48. The maximum depth of neutral axis $X_{u,max}$ is obtained directly from the strain diagram by considering similar triangles.

$$\frac{X_{u,max}}{d} = \frac{0.003\ 5}{(0.005\ 5 + 0.87 f_y/E_s)}$$

The values of $\frac{X_{u,max}}{d}$ for three grades of reinforcing steel based on the formula are given in the Note under sub-clause (f).

## *38.* LIMIT STATE OF COLLAPSE: COMPRESSION

*38.1 Assumptions*—Assumptions (a), (c), (d) and (e) of *38.1* for flexural members are also applicable to members subjected to combined axial load and bending. The assumption (b) that the maximum strain in concrete at the outermost compression fibre is 0.003 5 is also applicable when the neutral axis lies within the section and in limiting case when the neutral axis lies along the edge of the section; in the later case the strain varies from 0.003 5 at the highly compressed edge to zero at the opposite edge. For purely axial compression, the strain is assumed to be uniformly equal to 0.002 across the section. The strain distribution lines for these two cases intersect each other at a depth of $\frac{3D}{7}$ from the highly compressed edge. This point is assumed to act as a fulcrum for the strain distribution line when the neutral axis lies outside the section (*see* Fig. E-50). This leads to the assumption that the strain at the highly compressed edge is 0.003 5 minus 0.75 times the strain at the least compressed edge.

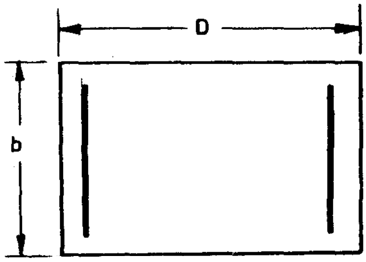
*38.2 Minimum Eccentricity*—In case of structural frames, the calculated eccentricity refers to the eccentricity obtained from analysis using *21* that is, $\frac{M}{P}$ (For additional information *see* comments on *24.2* also).

*38.3 Short Axially Loaded Members in Compression*—The strain distribution will be as shown in Fig. E-50 according to assumption (a) of *38.1*. At failure, the strain

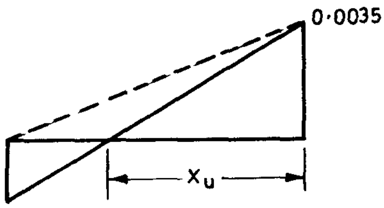
*Specification for hard-drawn steel wire fabric for concrete reinforcement (*second revision*).

†Specification for hot rolled mild steel, medium tensile steel and high yield strength steel deformed bars for concrete reinforcements (*revised*).

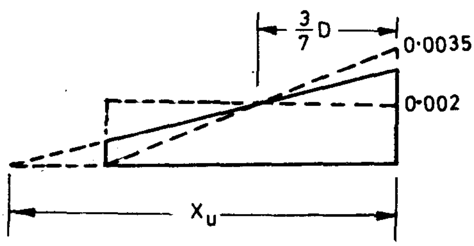
‡Specification for structural steel (standard quality) (*fifth revision*).




50 a COLUMN SECTION




50 b STRAIN DIAGRAM—NEUTRAL AXIS WITHIN THE SECTION




50 c STRAIN DIAGRAM - NEUTRAL AXIS OUTSIDE THE SECTION

*Fig. E-50 Combined Axial Load and Uniaxial Bending*

across the section is uniform at 0.002. The stress in concrete will be $0.67 f_{ck}/\gamma_m$.

The steel may develop full design stress in the case of mild steel reinforcement when concrete attains the limiting strain of 0.002. However, with cold twisted reinforcement $f_y = 415$ N/mm² the full design stress will not develop at a strain of 0.002. The stress corresponding to this strain will amount only to $\frac{0.85 f_y}{1.15}$ as can be read from Fig. 22A.

Therefore, the capacity of the member subjected to only axial load will be given by the equation

$$P = \frac{0.67 f_{ck}}{1.5} A_c + \frac{0.85 f_y}{1.15} A_{sc}$$

or $P = 0.45 f_{ck} A_c + 0.74 f_y A_{sc}$, which has been designated as $P_{uz}$ in *38.6*.

The formula given in *38.3* is obtained by reducing this capacity by approximately 10 percent thereby allowing for the minimum eccentricity of 0.05 *D*, where *D* is the lateral dimension of the section. If minimum eccentricity is greater than 0.05 *D*, the design may be done in accordance with *38.5*.

The classification of short column avoids superfluous calculations of moment due to additional eccentricity. It will be found that, if the additional eccentricities in short columns are calculated (*24.4*), they are nearly always less than the minimum of 0.05 *D*.

*38.4 Compression Members with Helical Reinforcement*—The Code permits larger loads in compression members with helical reinforcement because columns with helical reinforcement have greater ductility or toughness (Ref 34) when they are loaded concentrically or with small eccentricities. The ductility is ensured by the provisions in *38.4.1* and *25.5.3.2* (d).

*38.4.1* It has been observed experimentally that up to a certain stage ($P_u$ in Fig. E-51), the columns with or without sufficient lateral reinforcement behave almost identically. After reaching this stage, the column without adequate helical reinforcement or without closely spaced rectangular ties fails immediately, accompanied by the breakdown of concrete and buckling of bars between the ties. On the other hand, the column with adequate helical reinforcement continues to deform, almost at the same load. The ultimate load $P_u$ can be significantly larger than $P_u$ if sizable helical steel is provided although there is a small drop in load capacity just when the concrete shell outside the helical spiral spalls off at load $P_u$. The increased ductility and load carrying capacity in case of helically reinforced columns are due to the following reasons:

a) The spacing of helical spiral is usually small enough to prevent buckling of longitudinal steel; and

b) Owing to large deformation, the concrete core inside the helical spiral bears against the helical reinforcement causing it to exert confining reaction on the core. This confining reaction increases the load capacity and strain bearing capacity of the core concrete.

The Code does not intend to make use of increase in capacity beyond the spalling load

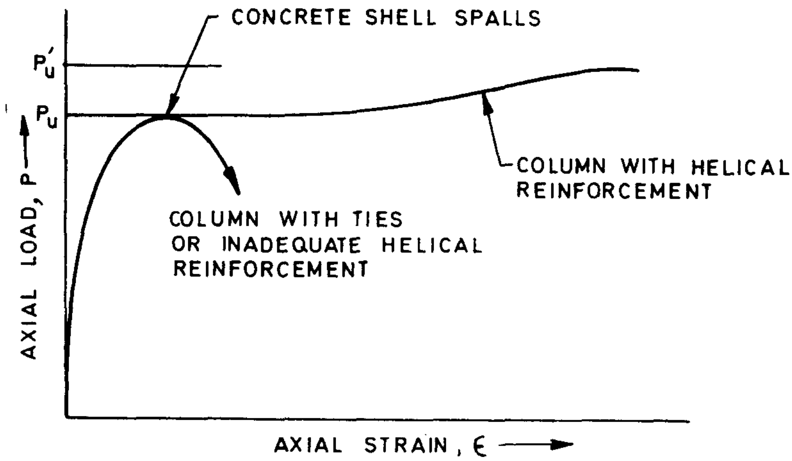



*Fig. E-51 Behaviour of Columns with and Without Helical Reinforcement*

in case of helically reinforced columns and the permitted increase in design capacity of such columns (*see 38.4*) is because the failure will be gradual and ductile. The criterion of minimum volume of helical reinforcement is to ensure that the load $P'_u$ carried by the column when the helical reinforcement yields after the spalling of the shell concrete, just exceeds the yield load of the column before spalling. In determining the amount of minimum helical reinforcement the partial safety factors $\gamma_m$ have not been introduced because a more severe condition would arise when the actual strengths of concrete and steel are equal to the respective characteristic strength (*see also* comments on *35.3.1*).

It has been established from experimental results (Ref 20) that the volume of steel in helical reinforcement is approximately twice as effective as the same volume longitudinal steel in contributing to the strength of column. The limit on the ratio of the volume of helical reinforcement to the volume of the core is based on this concept.

The characteristic strength of helical reinforcement is limited to 415 N/mm². This is to limit the crack width and also it becomes difficult to bend higher grades helically or they may get damaged during fabrication.

*38.5 Members Subject to Combined Axial Load and Uniaxial Bending*—The design of member subject to combined axial load and uniaxial bending will involve lengthy calculations by trial and error. In order to overcome these difficulties, interaction diagrams are used for the purpose of design. A set of interaction diagrams for columns have been prepared and published by ISI in 'SP : 16 (S&T) Design aids for reinforced concrete to IS : 456-1978'. This serves as a companion handbook to the Code for design engineers.

*38.6 Compression Member Subject to Combined Axial Load and Biaxial Bending*—Design and analysis of columns subject to biaxial bending are difficult because a trial and adjustment procedure is necessary. Also, interaction diagrams, similar to those for uniaxial bending, to cover all possible design cases cannot be provided without a large number of charts as they involve a large number of variables.

The formula given in the Code is derived by approximating the interaction surface for a given axial load (Ref 44). The value of $\alpha_n$ as given in the Code gives results close to those which would be obtained from a rigorous analysis in accordance with *38.1* (Ref 21). It may be noted $P_{uz}$ is the design load capacity for a column when the load is applied concentrically and not the load capacity which takes into account an eccentricity of 0.05 *D* (*see* comments on *38.3*).

In designing for biaxial bending, a section and reinforcement pattern could be assumed and the reinforcement area successively corrected till the condition given in *38.6* is satisfied.

*38.7 Slender Compression Members*—The Code recommends the use of a second order frame analysis, also called *P*-$\Delta$ analysis, which includes the effect of sway deflections on the axial loads and moments in a frame. For an adequate and rational analysis, realistic moment curvature or moment rotation relationships should be used to provide accurate values of deflections and forces. The analysis will also usually include the effect of foundation rotation and sustained loads.

Because of the complexity in the general second order analysis of frames, the Code provides an approximate design method which takes into account the 'additional moments' due to lateral deflections in columns. The accuracy of the 'additional moment' method has been established through a series of comparisons of analytical and experimental results (Ref 45) over the total range of slender columns as defined in *24.1.1*.

*38.7.1* The cross-section is designed for a moment equal to the sum of moments obtained by (a) first order analysis (*21*), and (b) additional moment due to the simultaneous action of lateral deflection (buckling) and axial load. The equations given in *38.7.1* for computing additional moment are derived for the case of a no-sway pin-ended column subjected to axial load and symmetric bending as shown in Fig. E-52. Because of the symmetric bending, the maximum moment and maximum deflection will occur at the mid length of the column.

The total moment, $M_t$ at mid length is equal to the sum of moments $M_1$ and additional moment $M_a$. The column will be capable of resisting axial load $P$ until $M_t$ is

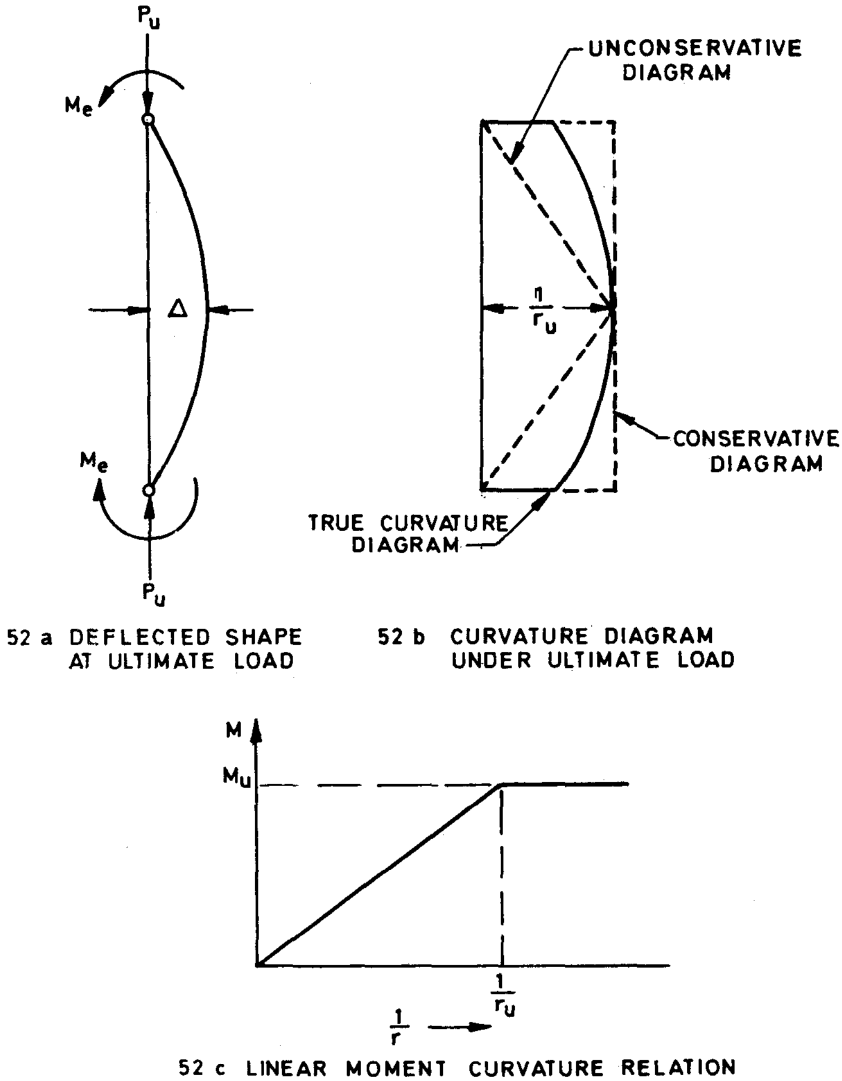



*Fig. E-52 Moment Curvature Relation for Pin-End Column* equal to the ultimate moment capacity $M_u$. The additional moment $M_a$ is given by the product of the axial load $P$ and central deflection, that is, $M_a = P\triangle$. The central deflection depends upon the ultimate curvature and the curvature distribution along the length of the column. If the moment-curvature relationship is known the curvature distribution along the length of the column can be determined using mathematics. For an idealized case of linear moment curvature relationship (the curvature diagram may have the shape shown in full line in Fig. E-52(b).

As an approximation, the curvature diagram for design purposes may be assumed to be somewhere between the triangular distribution (unconservative) and rectangular distribution (conservative), maximum curvature being equal to $1/r_u$.

By integrating the curvature diagram, deflection, $\triangle = \frac{l^2}{12r_u}$ (for triangular distribution) and for (rectangular distribution) $= \frac{l^2}{8r_u}$. It is, therefore, reasonable to take a value of $\frac{l^2}{10}\left(\frac{1}{r_u}\right)$ for design purposes (Ref 45). The additional moment $M_q$ is then given by

$$M_a = P_u\triangle = P_u\frac{l^2}{10}\left(\frac{1}{r_u}\right)$$

The ultimate curvature $\frac{1}{r_u}$ can be determined if the strain in concrete ($\epsilon_c$) and in tension steel ($\epsilon_s$) are known at collapse,

$$\frac{1}{r_u} = \frac{\epsilon_c + \epsilon_s}{D}$$

assuming that the effective depth is approximately equal to the distance between compression and tension faces.

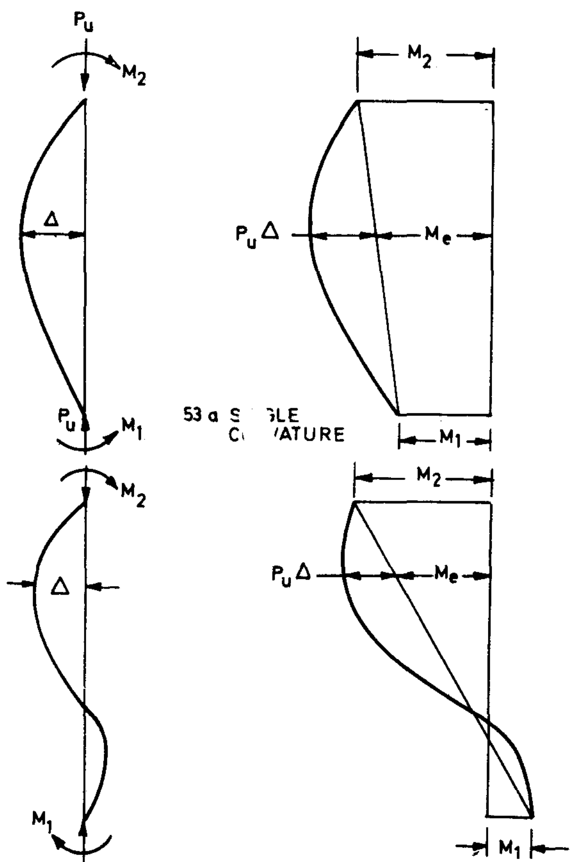
For the balanced condition defined in *38.7.1.1*, the values of $\epsilon_c$ and $\epsilon_s$ are 0.003 5 and 0.002, respectively. The ultimate curvature for balanced condition may then be taken as

$$\frac{1}{r_u} = \frac{0.003\,5 + 0.002}{D} = \frac{1}{182\,D}$$

$$\therefore M_a = \frac{PD}{1820}(L/D)^2 = \frac{PD}{1820}\left(\frac{L}{D}\right)^2$$

The expression given in the Code is a modified version of the above equation. In the equations given in *38.7.1*, the column length $l$ is replaced by effective length $l_e$ to allow for the effects of various end conditions occurring in practical columns (*see* 24.2 and Appendix D). Here, $l_e$ and $D$ will assume suitable values depending upon the direction of bending. The Code expressions for $M_a$ will yield conservative results in most cases even if the failure is not a balanced one.

In case of symmetric bending the total moment $M_t$ is maximum at mid length of column. When loading conditions give rise to unsymmetric bending, $M_t$ may not be maximum at or near mid length (*see* Fig. E-53 and Fig. E-54). It is required, in such cases, to find an appropriate value of moment $M_a$ which when added to $M_1$ (*38.7.1*) gives maximum moment $M_t$. The value of initial moment given in Note 1 of *38.7.1* is based on Ref 45.




53 a S LE C ATURE

53 b DOUBLE CURVATURE

*Fig. E-53 Moment in Braced Slender Column*

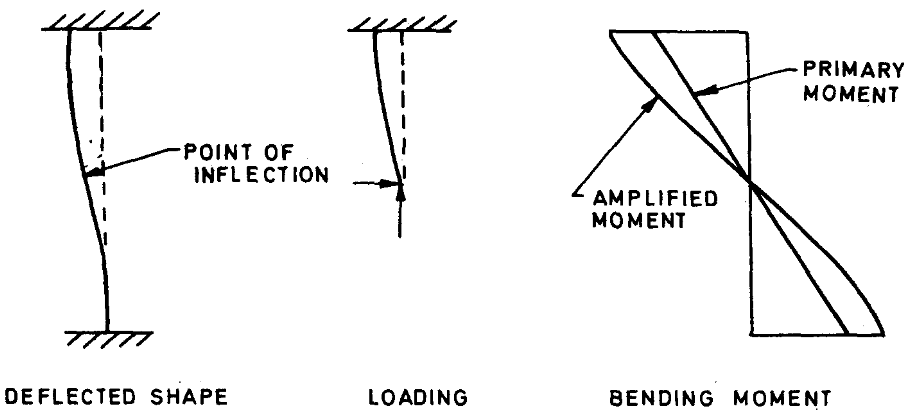



*Fig. E-54 Additional Moments in Column with Side Sway*

For the design of column sections on the above basis four cases may be distinguished:

a) Braced columns which are bent in single-curvature over a major portion of their lengths, shown in Fig. E-53 are subjected to the additional moment near the mid height. In such cases, the initial moment $M$ is taken as the value near the mid height and computed as $0.4\,M_1 + 0.6\,M_2$ as suggested in the Code. Further $M_1$ is usually taken to be atleast $0.4\,M_2$ (Ref 3 and 45).

b) In the case of columns (either braced or unbraced) which are bent in double curvature [*see* Fig. E-53 (b)], it is possible that the total moment $M_t = M_e + M_a$, computed for a point near mid height of column, is less than the initial moment $M_2$ applied at the end. In such a case, the column section should be designed to resist the moment $M_2$.

c) An unbraced column may be subjected to end moments which are nearly equal in magnitude, but opposite in sign (*see* Fig. E-54). Here, the total moment $M_t$ should be taken as the sum of the initial moment $M_i (= M_2)$ and the additional moment $M_a$, since the critical section for design will be that over which $M_2$ acts.

d) In unbraced frames, if the loading is confined to an individual column, it can be treated separately as a column braced against sidesway, for computing the effective length $l_e$ and the additional moment $M_a$ (Ref 34).

*38.7.1.1* The factor $k$ is intended as a correction for the ultimate curvature for various intensities of axial load and its use is optional. It varies from 1.0 at an axial load corresponding to the balanced condition, to zero as the applied axial load increases to the pure axial load capacity, $P_{uz}$ defined in *38.6*. This is in accordance with the fact that as the axial load increases on a section, beyond the balanced point, the curvature gets on reducing till it is zero at $P = P_{uz}$ when the strain distribution is constant. If the design axial load $P$ is less than $P_{bal}$ the value of $k$ is to be taken as unity only. In general, $P_{uz}$ and $P_{bal}$ required for computing $k$ will be functions of amount and arrangement of reinforcement, cover to reinforcement in addition to grades of concrete and steel. Since $k$ is not known before hand, a trial and error procedure is required to reach a solution wherever the design ultimate load $P$ is greater than $P_{bal}$. As a first trial $k$ may be taken as unity which can be modified once the area of steel reinforcement is fixed. Though this modification is optional according to the Code, it should always be taken advantage of, since the value of $k$ could be substantially less than unity.

The foregoing procedure for obtaining design moment in slender columns does not enable the designer to establish the bending moments throughout the column length. This is not necessary in general, since it is usual to provide a uniform cross-section with symmetrically arranged steel. If desired, however, reference can be made to the bending moment diagram given in Ref 3, and the cross-section or amount of steel varied along the length.

## 39. LIMIT STATE OF COLLAPSE: SHEAR

The recommendation given in the Code are framed in such a way that sufficient shear resistance is provided in all regions of a member so that upon overloading, the load carrying capacity of the member is governed mostly by its flexural strength. This is done to ensure ductile failure of the member as the shear failures are brittle and catastrophic in nature. The rules for curtailment of tension reinforcement (*25.2.3.2*) and those for bond and anchorage (*25.2.1* and *25.2.2*) also help in keeping the chances of shear failure to a minimum.

The shear provision in this Code limit the nominal shear stress $\tau_v$ (*39.1*) at design load $V_u$, for beams which do not contain shear (web) reinforcement. If $\tau_v$ exceeds design shear strength of concrete $\tau_c$ (Table 13) shear reinforcement should be provided as given in *39.4*. The section should be redesigned if $\tau_v$ exceeds $\tau_{c\,max}$ given in Table 14.

The design shear force $V_u$ is obtained by an elastic analysis of structure, (*36.1*) using appropriate partial safety factors on loads for collapse. The design shear force $V_u$ does not include the shear force due to torsion, for which separate provisions are given in *40*. The critical section for shear is determined from *21.6.2*. It may be necessary to provide for a possible increase in shear forces resulting from moment redistribution in indeterminate structure (*36.1.1*). However, the Code does not make any specific mention of it, as the resulting changes in shear forces are not likely to be substantial.

*39.1 Nominal Shear Stress*—The nominal shear stress is a measure of the shear resistance offered by the concrete. Note that the expression for nominal shear stress is similar to the one used in the working stress method of the 1964 version of the Code, except that the lever arm factor has been dropped. This simplification is reasonable as the nominal shear stress is only an indication of the shear resistance offered by concrete and does not necessarily represent the actual stress conditions.

Where hollow blocks are used, the rib width may be increased to take into account the wall thickness of the block on one side of the rib (*see* Fig. E-30).

*39.1.1* BEAMS OF VARYING DEPTH

### *39.2 Design Shear Strength of Concrete*

*39.2.1* WITHOUT SHEAR REINFORCEMENT—The values given in Table 13 are the limiting values on the nominal shear stress $\tau_v$. For computing the values of $\tau_c$ it is assumed that the diagonal cracking strength in shear is reached whenever the principal tensile stress in the neutral axis of a flexurally cracked beam reaches the tensile strengths of concrete. A semi-emperical formula for, $\tau_c$, based on this criterion is given in Reference 47 and has been used here:

$$\tau_c = \frac{0.85\sqrt{0.8 f_{ck}}\,(\sqrt{1.+5\beta}-1)}{6\beta}$$

where $\beta = 0.8\, f_{ck}/6.89\, p_t$, but not less than 1, and

$$P_t = \frac{100\, A_s}{b_w d}$$

The value of $\tau_c$ corresponding to $p_t$ varying from 0.2 to 3.0 at intervals of 0.25 are given in Table 13 of the Code for different grades of concrete. The factor 0.8 in the formula is for converting cylinder strength to cube strength and 0.85 is a reduction factor similar to partial safety factor $\gamma_m$ for materials as relevant to the formula.

The design shear strength, $\tau_c$ serves a dual purpose. In addition to imposing a limit on the shear carried by beams without shear reinforcement, it serves as a measure of the shear carried by concrete in a member with shear reinforcement (*see 39.4* and its comments).

It is explained elsewhere (*see* comment on *25.2.3.2*) that the Shear capacity of a beam may be reduced substantially at a section where the tension reinforcement is curtailed. The Code requirement, given in the Note to Table 13 arises from this consideration. Sometimes, it may be desirable to put additional stirrups locally near the curtailment points [*see 25.2.3.2*(b)].

*39.2.1.1* For flat slabs and two way action footings, the recommendations for shear design (*see 30.6*) are based on punching shear and therefore *39.2.1.1* should not be applied.

The increased shear strengths are based on test results (Ref 48) which show that shallow beams and slabs would fail at loads corresponding to a higher nominal shear stress. The values of $k$ in the Code have been obtained by linearly interpolating between the values at the ends of the range.

*39.2.2* SHEAR STRENGTH OF MEMBERS UNDER AXIAL FORCE—The effect of an axial compressive force is to delay the formation of both flexural and inclined cracks and to decrease the angle of inclination of the inclined cracks to the longitudinal axis when they do form. The same has been recognised in the Code and the multiplication factor as specified in the Code is to be applied. It is also to be noted that a tensile force acting on the section decreases the cracking load and increases the inclination of the inclined cracks to the longitudinal axis.

*39.2.3* WITH SHEAR REINFORCEMENT—The Code recommendation for the design of shear reinforcements (*see 39.4*) allow the yielding of shear reinforcements at the ultimate load conditions and thus the member will have ductile failure characteristics. However, the shear strengths of a beam cannot be increased indefinitely by the addition of unlimited amounts of web steel. This is because where large shears are carried it is possible for the diagonal compressive stresses to cause crushing of web concrete. Such failures are brittle. It has been shown that such failures can be avoided by imposing an upper limit on $\tau_c$ (Ref 38 and 49).

The values of $\tau_{c\,max}$ given in Table 14 of the Code have been obtained from the following expression:

$$\tau_{c\,max} = 0.83\sqrt{f_c}\ \text{N/mm}^2$$

where $f_c$ = cylinder strengths of concrete. The numerical values in the Code have been obtained after converting the cylinder strength to cube strength of concrete and thereafter applying a factor of 0.85, the latter being the partial safety factor for material strength $\frac{1}{\gamma_m}$.

*39.2.3.1* —

*39.3 Minimum Shear Reinforcement*—Although *39.4* requires that shear reinforcement be provided to carry the excess shear force $V_u - \tau_c bd$, this clause requires that a minimum amount must be provided in accordance with *25.5.1.6*. This minimum requirement is designed to allow for any sudden transfer of tensile stress from the web concrete to the shear reinforcement. This clause does not apply to solid slabs.

*39.4 Design of Shear Reinforcement*—(*see also* comments on *30.6.3.2* regarding shear reinforcement for resisting punching shear).

In the Code, the shear resistance $V_u$ of a reinforced concrete beam is taken as the sum of the shear force $V_c$ carried by concrete and the shear force $V_s$ carried by the shear reinforcement.

$$V_u = V_c + V_s$$

The shear force resisted by concrete, $V_c$ is obtained by using the design shear strength $\tau_c$ given in Table 13.

$$V_c = \tau_c bd$$

As the quantities $V_u$ *and* $V_c$ are known, the normal design procedure will involve the determination of the shear reinforcement for resisting the shear force $V_s$ contributed by the web reinforcement. Although various theories have been proposed to explain the action of shear reinforcement, the truss analogy theory is fairly generally accepted. The truss analogy assumes that the tension in the imaginary pin jointed truss are carried by longitudinal bars and stirrup reinforcement, and the concrete carries the thrust in the compression zone and the diagonal thrust across the web. The same is illustrated in Fig. E-55.

The tensions in the truss are carried by longitudinal and stirrup reinforcement and the concrete carried the thrust in the compression zone and the diagonal thrust across the web. Fig. E-55 shows the general case of stirrups at a longitudinal spacing $s_v$ and inclined at an angle $\alpha$ to the beam axis. The stirrups which intercept a diagonal crack, say AB will be able to resist the shear force $V_s$. Test results indicate (Ref 38, 49), that the potential diagonal crack AB may be represented for design purposes by a 45° line occurring anywhere in the region where the shear exists. Considering the equilibrium across the section AB,

$V_s = A_{sv} f_s \sin\alpha$ (number of stirrups crossed by the crack)

$$= A_{sv} f_s \sin\alpha\,(\cot 45° + \cot\alpha)\,\frac{d}{S_v}$$

$$\text{or } V_s = A_{sv} f_s\,(\sin\alpha + \cos\alpha)\,\frac{d}{S_v}$$

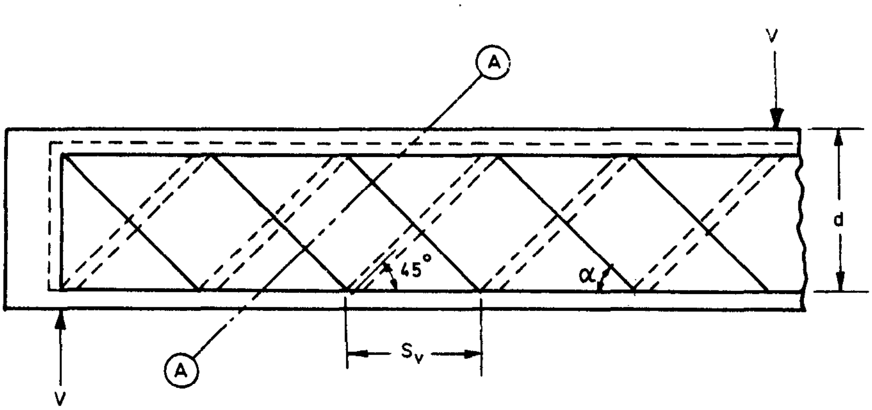



*Fig. E-55 Imaginary Truss System for Shear with Stirrups*

Introducing $f_s = 0.87 f_y$,

$$V_s = (0.87 f_y) A_{sv} \frac{(\text{Sin } \alpha + \text{Cos } \alpha)}{S_v} d$$

Three cases of stirrups are distinguished in the Code:

a) For the more common case of vertical stirrups,
$\alpha = 90°$ and (Sin $\alpha$ + Cos $\alpha$) = 1, and the Code formula follows.

$$V_s = (0.87 f_y) \frac{A_{sv} d}{s_v}$$

b) When the shear reinforcement consists of a series of bent-up bars as shown in Fig. E-55, or similar inclined stirrups, the original derivation holds good

$$V_s = (0.87 f_y) \frac{A_{sv} d}{S_v} (\text{Sin } \alpha + \text{Cos } \alpha)$$

c) The above equation is valid if the number of bars crossed by the diagonal crack AB is correctly given by the expression (Cot 45° + Cot $\alpha$) $d/S_v$. If a single stirrup or single group of parallel bars, all bent-up at the same cross-section are used,

$$V_s = (0.87 f_y) A_{sv} \text{ Sin } \alpha$$

The contribution from stirrups $V_s$ cannot be increased indefinitely by adding $A_{sv}$. The limit on $V_s$ and consequently $A_{sv}$ also is imposed by *39.2.3.1*.

There are two considerations which limit the shear contribution of bent-up bars:

1) The question regarding the exact behaviour of bent-up bars in resisting shear in reinforced concrete beams is still somewhat controversial.

2) Bent-up bars do not contribute to reversal of shear force.

The reasons for limiting $f_y$ value to 415 N/mm² for shear reinforcement are as follows:

i) The Code is concerned with crack width limitation and crack width and grade of steel are interrelated.

ii) It becomes difficult to bend higher strength steel bars and the sharp edges may get damaged during bending.

The maximum spacing of stirrups should not exceed 0.75 d or 45 cm whichever is less (*see 25.5.1.5*). For anchorage requirements of stirrups [*see 25.2.2.4*(b)]. The stirrups should be taken round the outermost tension and compression bars for torsion (*25.5.1.4*). Extra stirrups may be required in the case of (a) termination of flexural reinforcement in some cases [*see 25.2.3.2* (b)], (b) change in direction of tension or compression reinforcement (*see 25.2.2.6*), and (c) in splicing of reinforcement in certain cases (*see 25.2.5*).

It is to be noted that many times minimum reinforcement in accordance with *25.5.1.6* will govern design.

The clear cover to stirrups should not be less than 15 mm [*see* (d) and (e) of *25.4.1*].

## *40.* LIMIT STATE OF COLLAPSE: TORSION

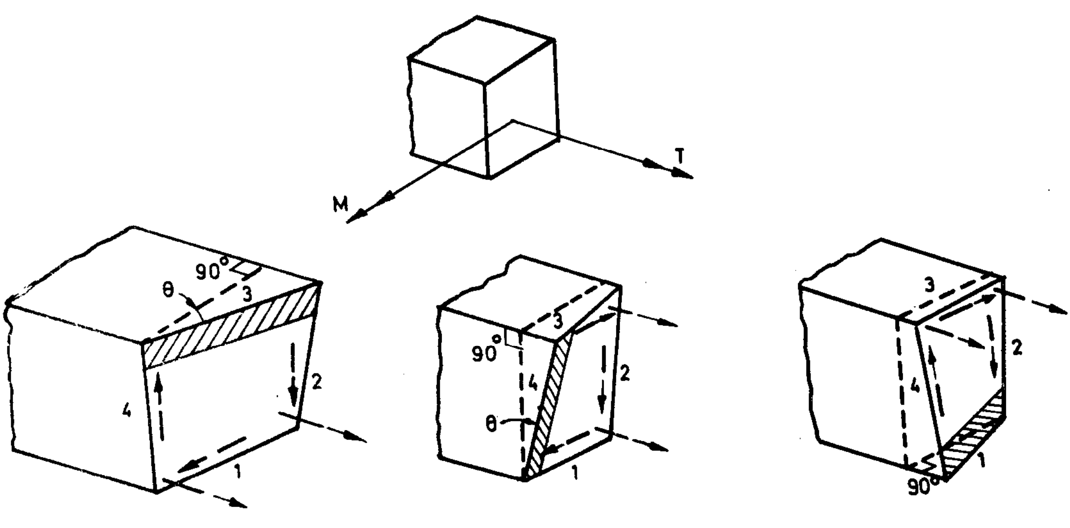
*40.1 General*—The Code implicitly distinguishes between two types of torsion encountered in structures:

a) Equilibrium Torsion — which is required to maintain equilibrium in the structure; and

b) Compatibility Torsion — which is required to maintain compatibility between the members of the structure. In statically determinate structures only equilibrium torsion exists, while in indeterminate structure both types are possible. A given load produces compatibility torsion in an indeterminate structure if the torsion can be eliminated by releasing redundant restraints, and equilibrium maintained despite this release.

The Code implies that for a member subjected to equilibrium torsion it is necessary to provide enough reinforcement to ensure that the member is capable of resisting the full torsion required by statics. The compatibility torsion may, at the discretion of the designer, be neglected in the design calculations by ignoring the torsional resistance of the members. If it is desired to consider the compatibility torsion, the torsional rigidity (*G C*) may be calculated by assuming the modulus of rigidity *G* equal to *0.4* $E_c$ and torsional stiffness *C* equal to half of the St. Venant value calculated for the plain concrete section (Ref 3). St. Venant torsional constants for rectangular sections are given in the explanatory comments on *21.3.1*. The design provisions for torsion given in the Code (*40.3* and *40.4*) are derived from the 'Skew bending theory' for rectangular beams. Three modes of failure (Fig. E-56) established experimentally for beams subjected to combined bending and torsion are considered. Each mode has a compression face and a tension face as in the case of pure bending, but the compression face becomes skewed to the normal cross-section due to the presence of torsion. Whether beam will fail in mode 1, mode 2 or mode 3, depends upon the imposed loading, amount of longitudinal and transverse reinforcement and cross-sectional dimensions. Mode 1 failure where compression zone becomes skewed but remains in the top surface of beam occurs in the region where bending is predominant. It may also occur in regions primarily subjected to torsion provided the section is not narrow. It is by far the most common type of failure since usually bending predominates over torsion and forces a slightly modified bending failure. If the cross-section is narrow (*D* $\gg$ *b*), predominant torsion compared with bending force initiate mode 2 failure where the compression face is skewed on to the sides of the section [*see* Fig. E-56 (b)].

If the longitudinal top steel is much less than the bottom steel the beam may fail in mode 3 with the compression face at the bottom [*see* Fig. E-56 (c)].




56 a MODE 1    56 b MODE 2    56 c MODE 3

*Fig. E-56 Failure Modes for Combined Bending and Torsion*

In mode 2 failure, the beam is failing by lateral bending, and in mode 3 failure by negative bending that is, a bending failure opposite in sign to the actual (small) bending moment present. In a square beam with equal longitudinal steel in all faces, under pure tension, there would be equal possibility of failure in mode 1, 2 and 3.

The presence of shear may cause a beam to fail at a strength below that predicted by one of the three skew bending modes. The possibility of shear type failure is checked by the introduction of 'equivalent shear' (*40.3*).

*40.1.1* Studies have shown that treating flanged section as cases of rectangular section will yield conservative results [(Ref 50), *see also 25.5.1* and its comments]. For hollow box beams, tests (Ref 51) have indicated that their strengths in the skew bending failure modes may be calculated as for solid beams of the same outside dimensions, provided the wall thickness is not less than $D/4$ or $b/4$ whichever is greater. For more slender walls it may be necessary to consider the warping effects also.

*40.2 Critical Section*—This clause is also analogous to *21.6.2* for shear (*see* comments on *21.6.2*).

*40.3 Shear and Torsion*—When both shear and torsion act together the resulting stresses are additive on same faces of the member. Consequently the shear and torsion which the member can resist simultaneously is less than that when either shear or torsion acts alone. In combined shear and torsion two types of failure modes are possible. In the presence of torsion combined with moderate shear forces a mode 2 failure (*see* Fig. E-56) will occur. The second type of failure occurs in the presence of high shear forces and is similar to shear failure in bending alone. Clause *40.4.3* is intended to avert both types of failures. The value of $\tau_{ve}$ computed on the basis given here should be used for determination of the lower limit on $A_{sv}$ in *40.4.3*. The lower limit on $A_{sv}$ is intended to preclude the possibility of a failure caused by predominant shear.

*40.3.1* Whether a beam requires transverse reinforcement against shear failure or not depends upon the magnitude of both $T$ and $V$. The formula given in this Clause enables the check to be made in single step. The formula developed empirically gives results well on the conservative side (Ref 52). The upper limit on $\tau_{ve}$ is to avoid web crushing of concrete before the yielding of transverse reinforcement takes place. If $\tau_{ve}$ exceeds $\tau_{c,\ max}$ the section should be redesigned.

*40.3.2* Minimum reinforcement is required to improve the ductility of the member and to preclude failure due to shear.

*40.3.3* If $\tau_{ve}$ lies between $\tau_c$ and $\tau_{c,\ max}$ the amount of transverse reinforcement required to avoid shear failure is to be calculated in accordance with *39.4*. The amount of transverse reinforcement required to check mode 2 failure can be calculated from the formula given in *40.4.3*. The larger of the above values will govern the amount of transverse reinforcement to be provided.

*40.4 Reinforcement in Members Subjected to Torsion*

*40.4.1* (*See also* comments on *40.1*) For each of the failure modes (*see* Fig. E-56), it is assumed in the Code that the intersections of the failure planes with the three sides of the beam will be three straight lines spiralling at a constant angle around the beam. The fourth side of the failure surface has compression zone of uniform depth making an angle with the normal cross-section. Moment equilibrium condition is employed for developing the formulae given in *40.4.2* and *40.4.3*. It has been assumed that both the longitudinal and transverse steel reach design strength before failure occurs.

*40.4.2* LONGITUDINAL REINFORCEMENT—It is implied that if a beam is designed to have flexural strength $M_{e1}$ then a probable failure is by mode 1 [*see* Fig. E-56 (a)].

Equating the internal and external moments and by using a few approximation (*see* Fig. E-57) the following equation may be obtained (Ref 53):

$$A_{s1}\ (0.87 f_y)\ (jd)_1 = M + T\left(\frac{1+\frac{D}{b}}{1.7}\right) Cot\ \alpha$$

where

$A_{s1}$ = Area of longitudinal steel on face 1 that is, flexure tension face; and

$(jd)_1$ = Lever arm when longitudinal steel $A_{s1}$ yields.

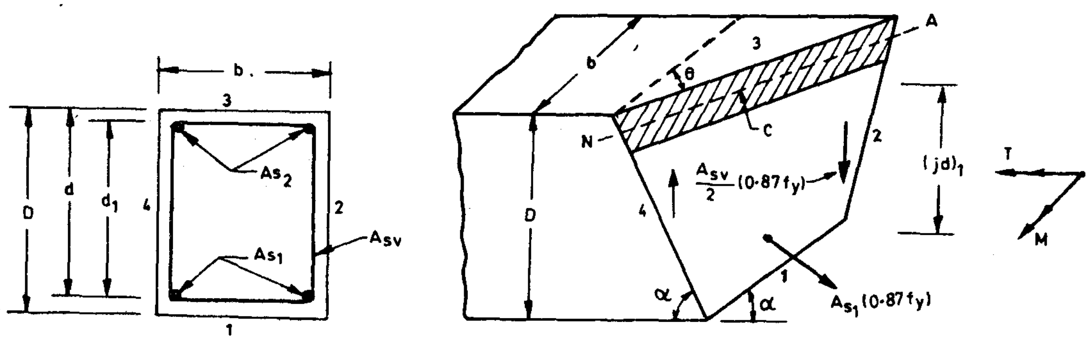



*Fig. E-57 Forces Acting on Plane of Failure, Mode 1*

The lever arm $(jd)_1$, is taken as that corresponding to flexure without torsion. Thus $A_{s1}$ $(0.87f_y)$ $(jd)_1$, is taken as the equivalent bending moment $M_{e1}$ to be resisted. The angle $\alpha$ varies from 45° to 90°. Since test data on crack angle ($\alpha$) is quite inadequate, a conservative value of 45° is taken for $\alpha$. These substitutions result in the formula given in this clause.

*40.4.2.1* (*See also* comments on *40.1*) This Clause precludes the occurance of mode 3 failure (Fig. E-56 with longitudinal steel at flexure compression face reaching the design strength before failure). Such a failure occurs when beam is predominantly subjected to torsion that is, $M_t > M_u$. The formula given for equivalent moment in this clause can be derived by using mode 1 equation and recognizing that the longitudinal steel at flexure compression face has reached design strength before failure. Once again from moment equilibrium conditions the following equation results:

$$A_{s3}\,(0.87\,f_y)\,(jd)_3 = \left[\,T\left(\frac{1+\frac{D}{b}}{1.7}\right) - M\right]$$

where

$A_{s3}$ = Longitudinal steel on face 3 (*see* Fig. E-56),

$(jd)_3$ = Lever arm when $A_{s3}$ starts yielding, and

$\therefore\ M_{e2}$ = $A_{s3}\,(0.87\,f_y)\,(jd)_3 = M_t - M.$

In other words the section should resist an equivalent bending moment $M_{e2}$ to avoid mode 3 failure. Once again it has been assumed that crack angle is at 45°. Mode 3 occurs, in general, under predominantly torsion loading and it has been observed in such cases that the crack angle is about 45° (Ref 52).

*40.4.3* TRANSVERSE REINFORCEMENT (*see also* comments on *40.1* and *40.3*)—Note that only the outer two legs of the closed stirrups should be considered for computing the torsional resistance contribution by web steel. If the stirrup consists of more than two legs, the interior legs should be ignored.

This clause consists of two requirements. The first one corresponds to the expression given for computing $A_{sv}$, and is intended to take care of a mode 2 failure [*see* Fig. E-56 (b)] which is likely with the application of a large amount of torsion and small shear forces. The second requirement imposes a minimum limit so that the shear stress $(\tau_{ve}-\tau_c)$ will be carried by the stirrup. The latter is intended for avoiding the normal type of shear failure associated with large shears together with a small amount of torsion.

Clause *40.4.3*, together with *25.5.1.7(a)* and *25.5.1.4* ensures that at least one stirrup would intersect the shear surface of failure.

## *41.* LIMIT STATE OF SERVICEABILITY : DEFLECTION

*41.1 Flexural Member* (*see 22.2* and its comments)—The Code does not provide any specific check for deflection in case of compression members, since deflection control will be achieved if the compression member satisfies the slenderness limits (*24.3*). The British Code of Practice (Ref 3) however, puts an additional check on sway for unbraced columns. It states that the deflection for an unbraced column may be considered to be acceptable if in that direction the average value of $\frac{l_{ef}}{D}$ for all columns at that level is less than or equal to 30.

## *42.* LIMIT STATE OF SERVICEABILITY : CRACKING

*42.1 Flexural Members* (*see 34.3.2* and its comments) — Side face reinforcement also may be needed for controlling cracks in accordance with *25.5.1.3* in addition to the provision in *25.3.2*.

Separate provision for crack control are given elsewhere for flat slabs (*30.7.1*) and hollow, ribbed and voided slabs [*29.7* (b)].

*42.2 Compression Members* — This clause requires a check for cracking in members subjected to combined bending moment and axial load if the ultimate axial load is less than $0.2 f_{ck} A_c$. This value of axial load is approximately equal to the value at the balanced condition (*see also 38.7.1.1* and its comments).

SECTION 6

# STRUCTURAL DESIGN
## (WORKING STRESS METHOD)

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# SECTION 6 STRUCTURAL DESIGN (WORKING STRESS METHOD)

*43.* GENERAL

*43.0* Clause *0.4.5* of the Code gives the major changes effected with respect to the working stress method provisions of the 1964 version of the Code.

*43.1 General Design Requirements*— Those design rules which are generally applicable to the structures irrespective of the design method adopted (that is, limit state method or working stress method) have been grouped together in Section 3. Therefore, reference to Section 3 will become essential in the case of methods of analysis and detailing requirements which are common to both the methods of design.

*43.2 Redistribution of Moments* (*see also 36.1.1* and its comments)—This Clause is applicable to continuous beams and indeterminate frames.

In statically indeterminate structures, redistribution of bending moment occurs before failure as a result of inelastic deformation of the reinforced concrete at critical sections. This redistribution permits a reduction in the support moments of a continuous beam which in turn would reduce the congestion of reinforcements at the supports. Such redistribution of bending moments occurs whenever incipient failure (plastic hinge) is reached whether at a support or in the span.

The arrangements of loadings to be assumed are given in *21.4.1*(a). Note that the simplified arrangement of *21.4.1*(b) should not be used along with redistribution of moments.

The procedure for carrying out moment redistribution is illustrated by the following example of a symmetrical three span continuous beam (*see* Fig. E-58):

STEP 1—Plot the elastic bending moment diagram for the continuous beam shown for the following three cases:

a) Spans AB and BC loaded with dead load plus live load ($DL + LL$) and span CD with dead load only } Adjacent span loaded fully

b) Span BC loaded with $DL + LL$ and the remaining with $DL$ only
c) Spans AB and CD loaded with $DL + LL$ and span BC with $DL$ only. } Alternate spans loaded fully

These plots are shown by full lines in Fig. E-58 for each of the three cases. Other combinations of loads can be eliminated from considerations of symmetry.

STEP 2—From inspection, determine the maximum value of the moment considering all the three cases. Suppose that this is at support B, for the loading case (a) and let this be $M_{B1}$. For the working stress method of design, the Code permits that this value can be reduced by 15 percent to, say $M'_{B1}$. At support C, in the example shown, the elastic moment $M_{C1}$ is less than the redistributed support moment at B, that is, $M'_{B1}$. It is permissible to increase the value of $M_{C1}$ by 15 percent. As it is desirable to have the same moment of resistance at all supports, $M_{C1}$, can be increased to $M'_{C1}$, so that $M'_{B_1} = M'_{C1}$.

STEP 3—With the adjusted support moments $M'_{B1}$ and $M'_{C1}$ redraw the bending moment diagram for the given loading, in such a way that equilibrium is maintained with respect to external loads. This diagram is shown by the chain-dotted line in Fig. E-58. Note that the mid-span moments are increased by '*a*' and '*b*' for the spans AB and BC respectively, as a consequence of redistribution.

STEP 4—Consider the bending moment diagram for case (b). Increase or decrease the bending moment at the supports, so that the adjusted values of case (b) are approximately equal to the critical value of $M'_{B1}$ of case (a). In the example the

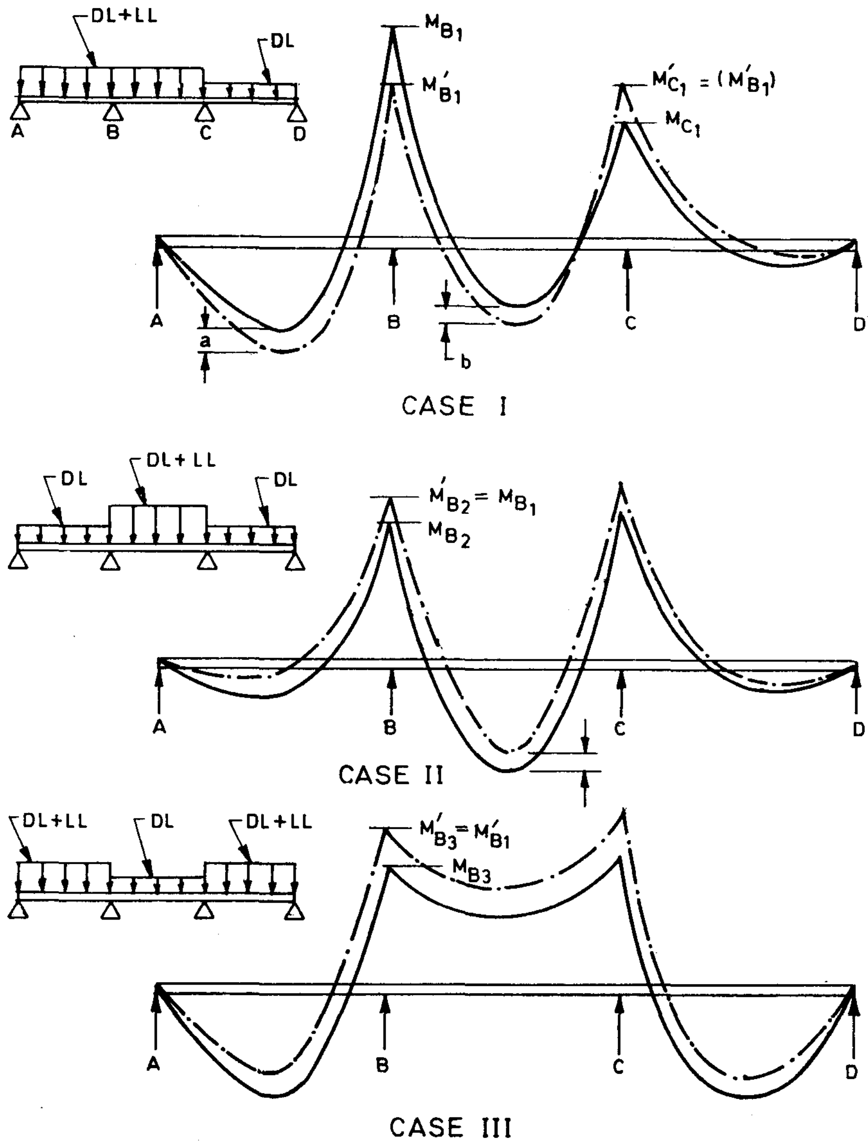


*Fig. E-58 Redistribution of Moments*

elastic moments at each support is shown less than $M'_{B1}$ and, therefore, are increased to $M'_{B2}$ so that $M'_{B2} = M'_{B1}$. Of course this increase should be limited to 15 percent. Note that the mid span moment in the span BC is reduced by an amount '*c*', when the redistributed bending moment diagram is drawn in such a way that equilibrium is maintained with respect to external loads. Repeat the procedure for case (c) also. Again, the redistributed bending moment diagram are shown by chain-dotted lines.

STEP 5 — Draw the envelope of the redistributed bending moments (shown in Fig. E-59). The continuous beam can now be proportioned for the redistributed values of the bending moments obtained from the envelope.

Note that the amount of redistribution permitted is only 15 percent in the working stress method whereas redistribution up to 30 percent is allowed in the limit state method with additional checks on depth of neutral axis as well as ductility of the section in the latter method (*see 36.1.1*). In the working stress method, the checks on detailing and ductility are not included because of the reduced magnitude of redistribution permitted.

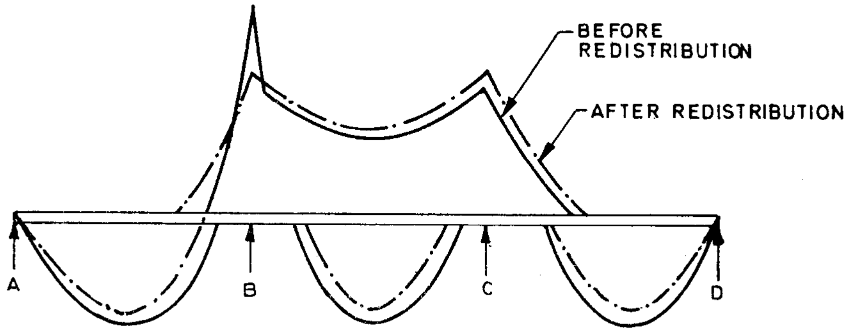


*Fig. E-59 Envelope of Bending Moments After and Before Distribution*

*43.3 Assumptions for Design of Members*—The phrase elastic theory here means the method of proportioning the section through the use of permissible stress in concrete and steel.

Assumption (a) regarding plane sections is traditional and confirmed by numerous observations and experiments.

Assumption (b) is meant primarily for beams and other elements subject to predominant bending. There are instances where the Code stipulates that the tensile stress in concrete are calculated, but the stipulation is aimed more often towards ensuring that the concrete is not unduly cracked rather than towards conferring any advantage in design by permitting the tensile strength of concrete to be taken into account. Some of the instances where tensile stresses in concrete are to be considered are:

a) Clause *44.1.1*, members in direct tension.
b) Clause *46.1,* para (b), members with combined bending and axial thrust (this is optional).

Assumption (c) regarding the linear relationship between stress and strain in concrete is justified for the low levels of stress (up to ⅓ of the cube crushing strength) permitted in the working stress method.

Assumption (d) is based on long-term experiments.

The modular ratio indicated in the Code is for concretes made with aggregates from natural sources and should not be applied for light-weight concretes.

## *44.* PERMISSIBLE STRESSES

*44.1 Permissible Stress in Concrete*—Following are the salient changes with respect to permissible stresses in concrete given in 1964 version of the Code:

a) The bearing stress inside a bend (other than standard bends and hooks) of reinforcing bar should not exceed the permissible value specified in *25.2.2.5*.
b) The permissible bearing pressure for concrete in pedestal footing is governed by *33.4*.
c) The permissible values of tensile stress in concrete, in members subject to direct tension, are given separately in *44.1.1*.
d) Permissible stress for 'average bond' are given in Table 15, which are the same as given in the 1964 version of the Code. However, in using the present version of the Code, it must be borne in mind that the detailing rules have been made more comprehensive.
e) Permissible stresses for local bond have been deleted in the present version, since an equivalent check given in para (c) of *25.2.3.3* takes this point into consideration.

The values indicated in Tables 15, 16 and 17 may be increased by 33⅓ percent while considering the combined effects of wind, temperature and shrinkage along with dead, live and impact loads (*see 44.3*).

*44.1.1* DIRECT TENSION—In direct tension, strength requirement is governed by steel alone. The limit on the tensile stress is to limit crack width and to maintain proper bond between steel and concrete.

*44.1.2* BOND STRESS FOR DEFORMED BARS (*see also* comments on *25.2.1*).

*44.2 Permissible Stresses in Steel Reinforcement*—The following information is with respect to mild steel bars and high yield strength deformed bars which are in wider use, medium tensile steel bars are not discussed here. Further restriction may be necessary in the case of special structures, such as water tanks:

a) Permissible stress for tension ($\sigma_{st}$ or $\sigma_{sv}$): In relation to the partial safety factors given in the limit state method of design, it can be shown that the permissible stresses given in Table 16 are consistent with the requirements of limit state method as well. Consider a beam subjected to a working load moment of $M_w$, caused by dead load and live load. Let the stress in tension steel at the working load moment be $\sigma_{st}$, the permissible stress. Corresponding to this case, the design moment $M_u$ for the limit state method is obtained by multiplying $M_w$ by partial safety factor $\gamma_f = 1.5$. Under the design load $M_u$, the steel may be stressed to $\frac{f_y}{1.15}$. Ignoring the slight variation in the lever arms calculated by using the two methods, a condition for obtaining approximately the same area of steel can be written down as:

FOR WORKING STRESS METHOD:

$$A_{st} = \frac{M_w}{\sigma_{st}\ (jd)}$$

FOR LIMIT STATE METHOD:

$$A_{st} = \frac{M_u}{\frac{f_y}{1.15} \times jd} = \frac{1.15 \times (1.5\ M_w)}{f_y \times jd}$$

Equating the areas of steel,

$$\sigma_{st} = \frac{f_y}{1.5 \times 1.15} = \frac{f_y}{1.73}$$

Referring to Table 16, it can be found that the permissible stresses are obtained by applying a factor of about 1.8.

b) COMPRESSION IN COLUMN BARS—For mild steel, the specified value of permissible stress can be explained on the basis of the partial safety factors similar to the basis given for permissible stress in steel in tension.

The reason for adopting a lower value of permissible stress in compression for high yield strength deformed bars is as follows:

As a basis for design, the Code stipulates elsewhere [*see* para (a), *38.1*] that the maximum compressive strain in concrete, in axial compression under ultimate load is 0.002. Consider the case of cold twisted bar of high yield strength (415 N/mm²), which has no definite yield point. Referring to the appropriate stress-strain diagram (*see* Fig. 22 A), the strain in steel when full yield is permitted will be about $0.002 + \frac{0.87\,f_y}{200 \times 10^3} = 0.0038$ (when $f_y = 415$ N/mm²). Clearly this is in excess of the design assumption of 0.002 as maximum compressive strain in concrete. A simple expedient to avoid this anamoly will be to limit the stress in steel to $0.80\,f_y$, the limit of proportionality under ultimate load condition. Applying a factor of 1.73 (that is, $1.15 \times 1.5$), the corresponding permissible stress is obtained as:

$$\sigma_{sc} = \frac{0.80 \times 415}{1.73} = 190 \text{ N/mm}^2$$

Obviously, this consideration will not be necessary for mild steel.

c) COMPRESSION IN BARS IN DOUBLY REINFORCED BEAM—If both steel and concrete are completely elastic, it will be reasonable to assume that the stress in the compression steel is equal to the compressive stress in the surrounding concrete multiplied by the modular ratio (that is, $m \times \sigma_c$), and this practice, being conservative, was followed earlier. However, the stresses and strains in the concrete are proportional only at relatively low strains; at higher strains the stresses in concrete no longer increase proportionately with respect to strain. Since the strains in the compression steel and the adjacent concrete must be equal the stress in steel, which remains in elastic state, will be larger than that estimated by assuming a linear elastic behaviour of concrete. To approximate the effects of non-linear stress-strain of concrete and of creep, the Code specified, albeit empirically, that the stress in compression reinforcement in beams and eccentrically loaded columns [*see* para (b), *46.1*] be taken as 1.5 times the modular ratio multiplied by the stress in the surrounding concrete.

*44.2.1* In a simplified treatment of design or analysis of beams, it is found expedient to assume that the reinforcing steel is concentrated at its centre of gravity and then estimate the steel stress, especially when more than one layer of steel is present. However, reasonable as this assumption is for most cases, the stresses in the outer layer of steel will be found slightly in excess of the permissible values if an accurate analysis were to be carried out. This Clause provides for such nominal discrepancies in stresses computed by the approximate method and a rigorous analysis.

Obviously, the practice of lumping of several layers of steel into a single mass at its centroid should not be carried too far.

*44.3 Increase in Permissible Stresses*— While dealing with the combinations of the effects of dead and live loads with those due to wind or earthquake loads, the Code assumes that the coincidence of full horizontal force (that is, wind or earthquake loads) with the most unfavourably distributed live load is rather unlikely. Also, the infrequent occurrence and temporary nature of the maximum horizontal loads are not likely to cause severe damage and these can be treated as some what secondary effects, in comparison with the main effects of dead load and live load. In working stress method, an increase in allowable stress is permitted; in the limit state method (or ultimate strength method) reduced load factors are recommended when combination of loads are considered (*see 35.4.1*).

## *45.* PERMISSIBLE LOADS IN COMPRESSION MEMBERS

*45.1 Pedestals and Short Columns with Lateral Ties*—Pedestal is defined as a compression member the effective length of which does not exceed three times the lateral dimensions. According to *24.1.1*, a compression member may be considered as short when both the slenderness ratios $\frac{l_{ex}}{D}$ and $\frac{l_{ey}}{b}$ are less than 12. Reinforced columns should have a minimum longitudinal steel area of 0.8 percent of the gross sectional area of the column [*see* para (a) of *25.5.3.1*]. Requirements for lateral ties are given in *25.5.3.2*.

The formula for permissible load $P$ is similar to the formula given in *38.3* for limit state collapse of compression members. This formula is applicable when the minimum eccentricity in accordance with *24.4* does not exceed 0.05 times the lateral dimension.

*45.2 Short Columns with Helical Reinforcement* (*see also 38.4* and its comments)—A column that is reinforced with the amount of helical reinforcement indicated in this clause is more ductile when compared to one that is reinforced with lateral ties. That is to say, a helically reinforced column gives ample warning of the approaching failure. Therefore, the Code allows a higher load on helically reinforced columns.

*45.3 Long Columns*—Slenderness effects in long columns can be taken into account by either of the two following methods:

a) Reduce the axial load on the columns to such a value that it corresponds to the safe load on slender columns; and

b) Design the critical section for an additional moment $P \times e_{add}$, where $e_{add}$ is the additional eccentricity due to the lateral deflection of the long column.

The latter approach has been introduced in the limit state design method (*38.7.1*). The former method, adopted in the 1964 version has been retained now for the working stress method. Though the Code speaks only of reduction in permissible stresses, it will be more convenient in design practice to determine the permissible load as for a short column and then to apply the reduction factor to this load to account for the slenderness effect.

Sample derivation for the reduction factor for long columns can be found in Reference 54. The formula given in the 1964 version was an approximation which has been changed now so that a reduction factor of 1.0 is obtained when $\frac{l_{ef}}{b}$ is 12, corresponding to the present definition of short column. The values of $C_r$ given in *45.3* are slightly on the liberal side (resulting in higher load capacities).

*45.4 Composite Columns* — The formula in para (a) is for 'squash load' that is, axial load on stocky or short columns. When moments and slenderness effects are to be considered, reference should be made to specialist literature (Ref 55).

## *46.* MEMBERS SUBJECT TO COMBINED AXIAL LOAD AND BENDING

*46.1 Design Based on Uncracked Section* — When a short column is subjected to loads with small eccentricities, it will be sufficient to limit the stresses in the transformed section. As the Code permits a small amount of tension in the section, the check based on uncracked section will be applicable when the eccentricity is less than 0.3 times the width of the column section.

To derive the design equation given in para (a), consider the extreme fibre stress in any section subject to eccentric loading.

$$\sigma > \frac{P}{A} + \frac{P.e}{Z}$$

where

$\sigma$ = permissible stress in concrete
$P$ = the axial load
$e$ = eccentricity
$A$ = transformed area of the uncracked section
$Z$ = Section modulus for transformed uncracked section

The equality can be rewritten as

$$\sigma_{cc,\,cal} + \sigma_{cbc,\,cal} < \sigma$$

$$\text{or } \frac{\sigma_{cc,\,cal}}{\sigma} + \frac{\sigma_{cbc,\,cal}}{\sigma} < 1$$

The value of $\sigma$ for the two terms on the left-hand side of the last equation is set from the requirement that this should be equal to $\sigma_{cc}$ under purely axial load and $\sigma_{c,\,cbc}$ under pure bending. Therefore,

$$\frac{\sigma_{cc,\,cal}}{\sigma_{cc}} + \frac{\sigma_{cbc,\,cal}}{\sigma_{cbc}} \leq 1$$

For computing the values of $\sigma_{cc,\,cal}$ and $\sigma_{cbc,\,cal}$, the entire concrete section and the area of steel multiplied by 1.5 times the modular ratio may be taken as the transformed section (*see* Note 1 in the clause). The value of modular ratio suggested in para (d) of *43.3* partially takes into account the long term effects. However, as columns are likely to carry sustained loads, the modular ratio is increased further by 50 percent for this rule, as this is likely to agree more with long-term loading conditions.

In Note 2, of the clause, the basis for computing the section modulus Z is not explicitly stated. However, it can be inferred from Note 1 of the clause that the entire concrete section along with the area of steel transformed on the basis of 1.5 times modular ratio will be appropriate for calculating Z.

*46.2 Design Based on Cracked Section* — If the eccentricity or moment is large, the section is likely to crack and then it will be appropriate (though not mentioned in the Code) to transform the area of compression steel on the basis of 1.5 times the modular ratio, in line with Note 1 in *46.1*. The area of tension steel should be transformed on the basis of modular ratio.

For columns bent about one axis (that is, axial load with uniaxial bending), condition (a) and (b) of the Note under the clause will be sufficient to determine the stresses in steel and concrete. However, a third condition will be necessary to fix the position and orientation of the no-stress line when the column is subjected to biaxial bending. In the latter case, it may be expedient to solve for the position and orientation of the no-stress line by equating the direct forces in the relevant direction and the moments on two mutually perpendicular axes.

The term 'no-stress line' is used in the Code, in lieu of 'neutral axis', to avoid the confusion resulting from the possible determination of neutral axis from the geometrical properties of a section.

*46.3 Members Subjected to Combined Direct Load and Flexure*— According to this clause, members subject to combined axial load and bending designed by methods based on elastic theory should be further checked for their strength at ultimate load conditions. Therefore, it would be advisable to design such members directly by limit state method to avoid lengthy calculations.

*47.* SHEAR—(*See* explanatory comments on *39*).

*47.1 Nominal Shear Stress*—(*See* explanatory comments on *39.1*).

*47.2 Design Shear Strength of Concrete*—(*See* explanatory comments on *39.2*).

*47.3 Minimum Shear Reinforcement*—(*See* explanatory comments on *39.3*).

*47.4 Design of Shear Reinforcement*—(*See* explanatory comments on *39.4*).

*48.* TORSION—(*See* explanatory comments on *40*).

*48.1 General*—(*See* explanatory comments on *40.1*).

*48.2 Critical Section*—(*See* explanatory comments on *40.2*).

*48.3 Shear and Torsion*—(*See* explanatory comments on *40.3*).

*48.4 Reinforcement in Members Subjected to Torsion*—(*See* explanatory comments on *40.4*).

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# APPENDICES

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# APPENDIX A

## REQUIREMENTS FOR DURABILITY

*A-1.* The minimum cement content given in Table 19 should be complied with, irrespective of the quantity of cement needed for the strength of the concrete. Where concrete is liable to sulphate attack, Table 20, should also be considered and the more stringent of the requirements given in Table 19 or Table 20 should be ensured. Independent of these two requirements, it should also be ensured that the materials used for concrete do not contain chlorides and soluble sulphates in excess of the quantities mentioned in *A-2*.

Regarding Table 19 (Durability, but not involving sulphates): The main intention here is to ensure reasonable durability either by limiting the water content or by specifying minimum cement content, since one of the stipulations will take care of the other, provided the concrete is workable. However, the Code gives the requirements for both cement content as well as water content perhaps in order to ensure tighter control on production. The cement contents indicated are with respect to a maximum aggregate size of about 20 mm which is in common use. If smaller size aggregates are used the cement contents may have to be increased so that the maximum water-cement ratio is not exceeded.

Referring to the Note 1 to Table 19, a 'strict control' of water-cement ratio implies that the free water in the mix should be controlled by making due allowance for the surface water carried by the aggregate. For this strict control, use of Table 4 in the Code may not be sufficient.

*A-2. Regarding Table 20 (Durability under Sulphate Attack)*: The concentrations of sulphates, indicated in columns 2, 3 and 4 of Table 20, should be used as a guide for site classification, serialized in column 1. It is desirable to determine the sulphate concentrations both in soil as well as ground water and take the worse of the results for specifying the requirements for concrete (*see* Note 3 of Table 20). As these concentrations are likely to vary abruptly even over small distance (say 20 m), site classification should be based on as large a number of samples as possible, especially in areas known to contain alkali soils and sulphates.

Even though Note 1 says that super sulphated cement can be used when mineral acids are encountered and Note 6 permits the use of ordinary Portland cement (with additional limits on $C_3A$ and 2 $C_3A$ + $C_4AF$) in place of super sulphated cement, the latter should not be used when concrete is likely to be exposed to acids. Rather, the statement in Note 1 that ordinary Portland cement would not be recommended in acidic conditions should be extended to Note 6 also.

Table 20 should be applied to concretes of grades M 20 and above, M 20 being the minimum for structures exposed to sulphate attack. Reference 21 suggests that the following minimum cement contents for the lower grades of concrete:

| | |
|---|---|
| 120 kg/m$^3$ | for grade M 7 |
| 150 kg/m$^3$ | for grade M 10 |
| 180 kg/m$^3$ | for grade M 15 |

The cement contents given in column (6) are for coarse aggregates of maximum size of about 20 mm. If smaller aggregates are used, the cement content may have to be increased.

The Code limits the total amount of chloride in terms of $Cl^-$ (and not in terms of $Cl_2$, Ca $Cl_2$, etc), since the chloride ions are responsible for corrosion of embedded metal in concrete. However, soluble sulphates are expressed as $SO_3$ (and not $SO_4$) following the conventions used in cement chemistry.

Compliance with the limits on chloride would require considerations of chlorides entering from mixing water, aggregates (especially from marine sources) and admixture.

# APPENDIX B

## CALCULATION OF DEFLECTIONS

*B-1.* TOTAL DEFLECTION

*B-1.1* —

*B-2.* SHORT TERM DEFLECTION

*B-2.1* The value for short term modulus of elasticity, $E_c$ is given in *5.2.3.1*.

Three definitions for moment of inertia are given in *21.3.1* and any of these may be used only for computing the relative stiffness of members for the analysis of rigid frames. However, while estimating the absolute values of deflections, it should be noted that, depending on the load level, the beam will be cracked at a few sections and will remain uncracked in the portions between these cracks. In other words, the effective moment of inertia $I_{eff}$ for the entire beam will be somewhere between the value $I_{gr}$ for the gross section [*21.3.1*(a)] and that of $I_r$ for the fully cracked section [*21.3.1*(c)], the exact value being dependent on the applied moment $M$ corresponding to the service loads.

The expression for $I_{eff}$ given in the Code is an empirical fit to the results of several deflection tests on reinforced concrete beams. In the case of slabs, normally it will be sufficient to restrict the span/depth ratio. As the columns in a building are usually braced (restrained against side sway) and likely to be stocky, specific check on deflection of columns will be unnecessary in most cases and, therefore, the Code does not give recommendations in this regard.

CONTINUOUS BEAMS—The expression for $I_{eff}$, the effective moment of inertia, is valid for simply supported beams. When the end moment in a beam element arises due to restraints (for example, interior supports in continuous beams, encastre ends in fixed beams), the deflections will be less and this should be accounted for by modifying the value of $I_{eff}$ as indicated in the Code. The basis of the method is given in Reference 60.

*B-3.* DEFLECTION DUE TO SHRINKAGE

*B-3.1* Completely unrestrained sections, that is, unrestrained plain sections and unrestrained symmetrical sections ('symmetry' applies to reinforcements as well) will be free from shrinkage curvature. The approach given in the Code is applicable for beams with unequal amounts of compression and tension reinforcements. From the Code recommendations, it will be clear that the introduction of compression reinforcement will reduce the deflection due to shrinkage, and due allowance has been made in the span/depth requirements also [*see 22.2.1*(d) and Fig. 4].

The empirical method given in the Code avoids the complications of computing $E$ and $I$ for shrinkage curvature, but is accurate enough for practical purposes. Considering a simply reinforced beam, Fig. E-60 assume that the compression fibre shrinks to $\epsilon_{cs}$, the unrestrained shrinkage (that is, the ultimate shrinkage strain of concrete). From geometry (Fig. E-60), the shrinkage curvature $\psi_{cs}$ is,

$$\psi_{cs} = \frac{\epsilon_{cs}}{d} \left(1 - \frac{\epsilon_s}{\epsilon_{cs}}\right)$$

Where $\epsilon_s$ is strain at the steel level. As the presence of compression steel will reduce the shrinkage curvature, it will be logical to relate $\psi_{cs}$ to $(p-p')$, $p$ and $p'$ being defined as the percentages of tension and compression reinforcements, in the Code.

$$\psi_{cs} = \frac{\epsilon_{cs}}{D} \text{ [function } (p-p')]$$

Here the overall depth $D$, instead of the effective depth $d$ has been introduced for convenience. This function is designated as $K_4$ in the Code and the recommended expressions are empirical fits to test data.

*B-4.* DEFLECTION DUE TO CREEP

*B-4.1* An approximate method for computing the total deflection (that is, the initial plus creep deflection) under permanent load is to use the effective modulus of elasticity $E_{ce}$. For computing $I_r$ and subsequently $I_{eff}$ (*see B-2.1*), the effective modular ratio,

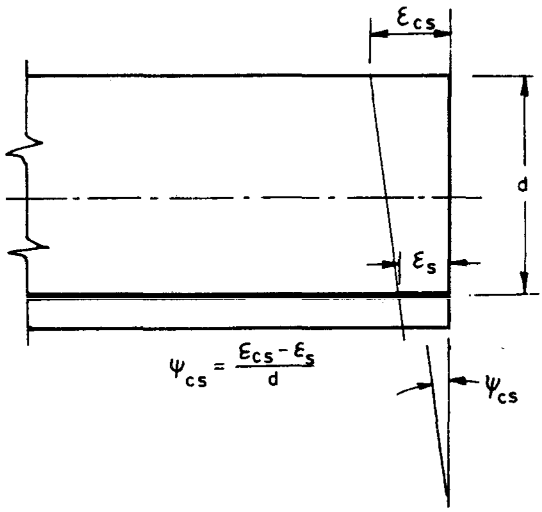



*Fig. E-60 Shrinkage Curvature*

$m_e = \frac{E_s}{E_{ce}} = \frac{E_s}{E_c}$ $(1 + \theta)$ should be used, though this point is only implied (and not explicit) in the Code.

For convenience of computations, it is desirable to consider the total deflection occurring after a long time in three parts:

a) the instantaneous or short-term deflection under permanent loads, $a_{i\,(perm)}$;

b) the creep deflection due to permanent loads; and

c) the short-term deflection under the total load.

The short-term deflection under total load and under the permanent loads can be computed, using the short-term modulus of elasticity $E_c$. A separate analysis will be required to calculate $a_{i,\,cc\,(perm)}$, using the effective modulus $E_{ce}$. Referring to Fig. E-61, total deflection under total load

= Instantaneous deflection under total load
+ (Total deflection under permanent load

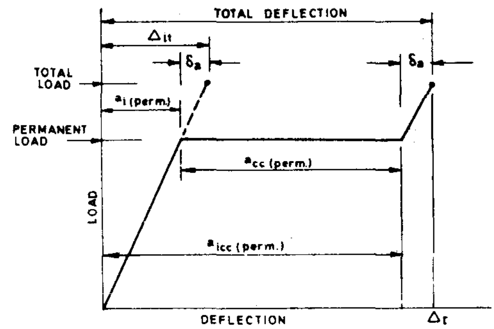



*Fig. E-61 Assumptions of Creep Deflection*

– short-term deflection under total load)
$= \delta_{it} + [a_{i,\,cc\,(perm)} - a_{i(perm)}]$

The expression within the square brackets is the creep deflection due to permanent loads $a_{cc(perm)}$, given in the Code. Now, the procedure for calculation can be summarised as:

a) Place the characteristic load on the beam, calculate the short term deflection using short term modulus $E_c$. This will correspond to $\delta_{it}$;

b) Place only the permanent load on the beam, calculate the corresponding short term deflection $a_{i\,(perm)}$;

c) Place the permanent load on the beam and compute the initial plus creep deflection $a_{i,cc\,(perm)}$, using the effective modulus $E_{ce}$;

d) Calculate creep deflection under permanent load as

$$a_{cc,\,(perm)} = a_{i,\,cc\,(perm)} - a_{i,\,(perm)};$$

e) Compute the deflection due to shrinkage $a_{cs}$; and

f) The total deflection, which includes the effects of creep and shrinkage, is computed as the sum of the values obtained in steps (a), (d) and (e) above.

# APPENDIX C

## SLABS SPANNING IN TWO DIRECTIONS

*C-1.* RESTRAINED SLABS

*C-1.0* Simply supported two-way slabs whose corners are not restrained from lifting or not reinforced to resist torsion at the corners are to be designed according to *C-2.1* and *C-2.1.1*.

In slabs where the corners are prevented from lifting and contain reinforcement to resist torsion, they may be designed as specified in *C-1.1* to *C-1.11*.

*C-1.1* The Code gives bending moment coefficients in Table 22 for uniformly loaded rectangular panels that are supported on all sides and are continuous over one, two, three or four edges. The coefficients have been derived from yield line analysis and adjusted to take into account the fact that, as the slab is divided into middle strips and edge strips, the steel is not spaced uniformly across the slab. Arbitrary parameters, such as the ratio of support to span moment, have been selected to give moments that correspond to those which would be obtained from an elastic analysis. The support conditions for various cases given in Table 21 are described in Fig. E-62.

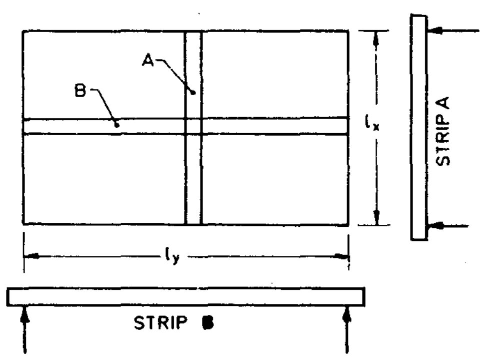



*Fig. E-62 Simply Supported Slab — Rankine Grashof Formula*

*C-1.2* —

*C-1.3* The negative and positive bending moments as calculated by the Code coefficients are those that occur in the middle strips which should then be reinforced accordingly. Redistribution of moments is not allowed because the coefficients in Table 22 already include redistribution.

*C-1.4* —

*C-1.5* —

*C-1.6* A negative bending moment may occur at the end of a slab where it is not continuous over the support but is cast monolithically with the support. The Code recommends that in ordinary cases this moment should be assumed to be one-half of the positive bending moment at mid-span of the middle strip at right angles to the support.

*C-1.7* The minimum reinforcement for slabs is given in *25.5.2.1*. The minimum quantity should be calculated separately with respect to the cross sectional area of the edge strip and provided within the edge strip.

*C-1.8* This clause is illustrated in Fig. E-63. The value of $A_s$ is calculated as that required for maximum mid-span moment per unit width.

*C-1.9* —

*C-1.10* —

*C-1.11* For design of one way slabs, *see 23.1, 23.2, 23.3,* and *23.3.1.*

*C-2.* SIMPLY SUPPORTED SLABS

*C-2.1* The coefficients given in Table 23 are derived from Rankine-Grashof formulae which are based on the equality of deflection of two strips of the slabs, one each along the directions parallel to the supports at their intersection. If the slabs shown in Fig. E-62 were supported only on the longer sides, the centre strip of unit width would have a bending moment of $M_x = \frac{W_x l_x^2}{8}$, and if it were supported only at the shorter sides, the bending moment would be $M_y = \frac{W_y l_y^2}{8}$. For the actual case of the slab

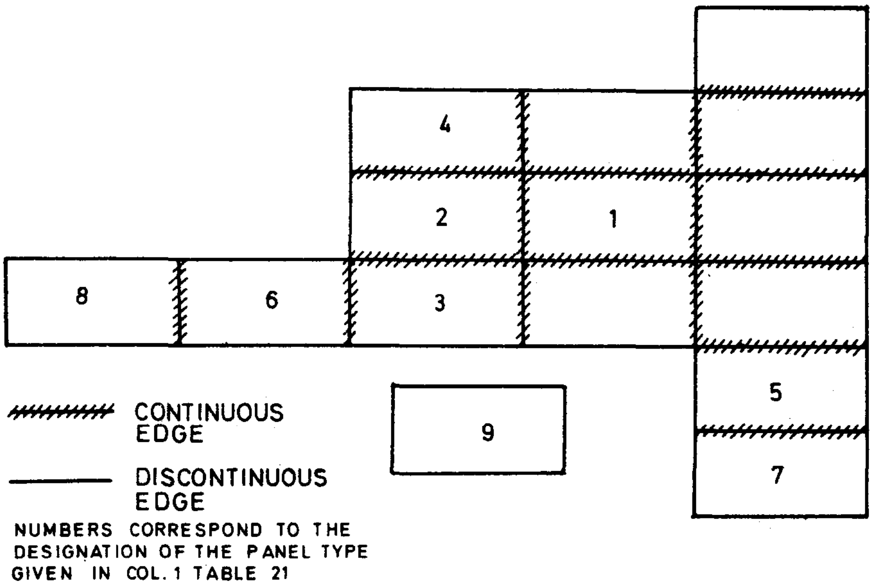



*Fig. E-63 Support Conditions for Slabs of Table 21*

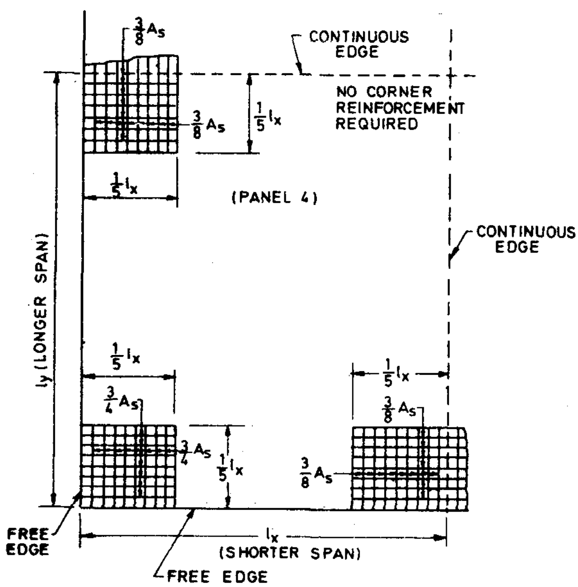



*Fig. E-64 Corner Reinforcement for Torsional Resistance (Showing Top Two Layers Only)*

supported on all four sides, assume that the load is divided between the two strips in such a way that the deflection of the two strips are equal at their point of intersection. Then,

$W = W_x + W_y$ and

$$\frac{5}{384}\,\frac{W_x l_x^4}{EI} = \frac{5}{384}\,\frac{W_y l_y^4}{EI}$$

Therefore $W_x = \dfrac{W\,l_y^4}{l_x^4 + l_y^4}$

and $W_y = \dfrac{W\,l_x^4}{l_x^4 + l_y^4}$

Accordingly, along the x and y direction the bending moments are,

$$M_x = \frac{W_x\,l_x^2}{8}$$

$$= \frac{1}{8}\,\frac{(l_y/l_x)^4}{1+(l_y/l_x)^4}\,.\,Wl_x^2$$

$$= \alpha_x\,W\,l_x^2$$

where $\alpha_x = \dfrac{1}{8}\,\dfrac{(l_y/l_x)^4}{1+(l_y/l_x)^4}$

Similarly, $M_y = \alpha_y\,W\,l_x^2$

where $\alpha_y = \dfrac{1}{8}\,\dfrac{(l_y/l_x)^2}{1+(l_y/l_x)^4}\left(\dfrac{l_y}{l_x}\right)^2$

Special corner reinforcements (*see C-1.8*) need not be provided for slabs which are free to lift at the corners, and are designed by the above rule.

*C-2.1.1* This clause is intended to facilitate a simple detailing procedure, without having to adopt the bar curtailment rules given in *25.2.3*.

# APPENDIX D

## EFFECTIVE LENGTH OF COLUMNS

*D-1.* The Code recommends three methods for calculating effective length of columns.

a) *Exact Analysis* — This will require a second order analysis of an entire frame of which the column may form a part, including therein the effects of deflections and inelastic behaviour. The complexity of an exact analysis of an entire frame renders it almost unsuitable for routine design application.

b) *Use of Fig. 24 and 25* — In rigid frames, the effective lengths of columns should be determined by using Fig. 24 and 25 of the Code, which in turn are based on the concept of vanishing stiffness method (Ref 58).

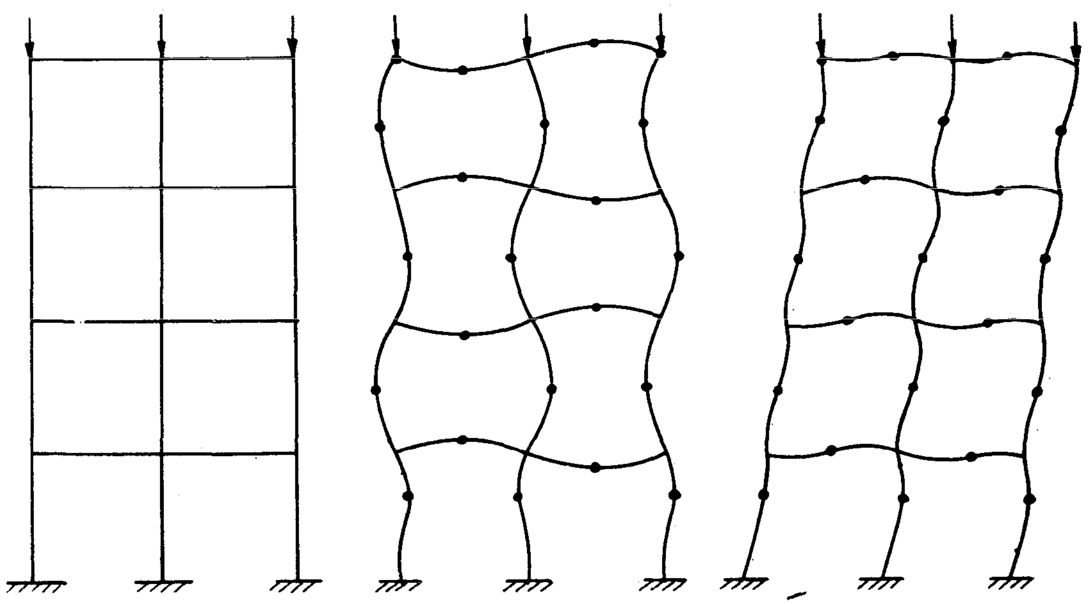
c) *Use of Table 24* — In normal usage, the columns may be idealized as falling into any one of the conditions specified in Table 24. Table 24 should then be used to assess the effective lengths of such compression members.

The Code distinguishes two cases of columns, namely, braced and unbraced columns.

a) Columns, whose ends are prevented from lateral displacements (also known as braced columns or no-sway case). Fig. 24 of the Code should be used for such columns.

b) Columns whose ends are not prevented from lateral displacements (also known as unbraced column or no-shear case). Figure 25 of the Code should be used for this type of columns.

Figure E-65 shows the buckling modes associated with the two cases. Because of the difference in behaviour between a braced and an unbraced frame, it is necessary to have two separate graphs (Fig. 24 and 25) for the determinations of effective lengths. However, in an actual frame, the columns are seldom completely braced or completely unbraced. Thus it is necessary first to determine whether the columns in a given storey are braced or unbraced. Roughly, a compression member may be assumed to be braced if the bracing elements (shear walls, shear trusses or other types of lateral bracing) in that storey have a total



65 a FRAME　　65 b FRAME WITH NO-SWAY　　65 c FRAME WITHOUT RESTRAINT AGAINST SWAY

*Fig. E-65 Buckling Modes for Rectangular Frames* stiffness, for resisting lateral movements, at least six times the sum of the stiffness of all columns within that storey.

Alternatively, a more accurate way of determining the effective length is to use the stability index $Q$. When the stability index $Q$ is less than 0.04, the storey can be considered as braced (Ref 34 and 59).

$$Q \ngtr 0.04$$

where

$$Q = \frac{\Sigma P_u \Delta_u}{H_u h_s}$$

$\Sigma P_u$ = Sum of axial loads on all columns in the storey.

$\Delta_u$ = Elastically computed first-order lateral deflection.

$H_u$ = Total lateral force acting within the storey.

and $h_s$ = Height of the storey.

*Basis of the Method*—Complete derivations are given in Ref 58. A brief account of the basis of the vanishing stiffness method is given below. First consider the simple braced column shown in Fig. E-66, with an applied moment $M_A$ at end A, the corresponding rotation being $\theta_A$. Following the conventional practice, let the rotational stiffness be $K = \frac{I}{L}$ where $L$ is the length of the column.

At end A, the applied moment and the rotation are related as:

$$M_A = E\,K\,s\,\theta_A \qquad (1)$$

where $s$ is a 'stiffness function' expressed in terms of $\frac{P}{P_E}$

$P$ = Axial load on column; and

$P_E$ = Euler load = $\frac{\pi^2 EI}{L^2}$ for a pin-ended column.

$$\text{Stiffness of column} = \frac{M_A}{4E\theta_A} = \frac{K\,s}{4}$$

At the base B, a moment $M_B$ would have been carried over, with $\theta_B = 0$

$$M_B = C.M_A = C.E\,K\,s\,\theta_A \qquad (2)$$

where $C$ is the carry-over factor, which

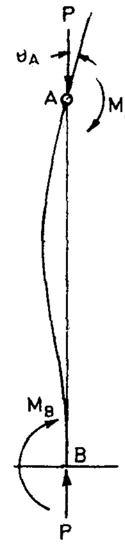



*Fig. E-66 Column with No Sway*

depends on $\frac{P}{P_E}$ as in the case of the stiffness function $s$.

Referring to Fig. E-67, the column in a frame generally frames into beams, with stiffness, say $\Sigma K_{bt}$ and $\Sigma K_{bb}$ at top and bottom respectively. Moment M is applied at the top end A as before. At the far end B which is clamped initially, the column will have a carry-over moment ($CM$) as before. Now clamp the end A at the original rotation of $\theta$ and release the far end B. After release,

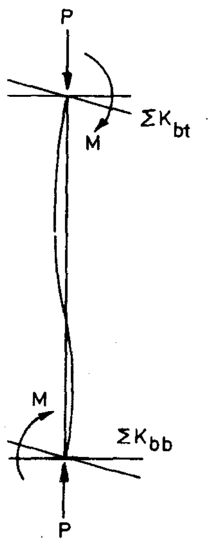



*Fig. E-67 Braced Column in a Frame*

the end B will have a moment equal to:

$$M'_B = C.M \times \frac{\text{Stiffness of column}}{\text{Total stiffness of members at B}}$$

$$M'_B = C.M \quad \frac{K\,s/4}{K\,s/4 + \Sigma K_{bb}} \tag{3}$$

Consequently, a moment of $-C.M'_B$ is carried back to joint A, which is still clamped at rotation $\theta$. Therefore, the next applied moment at A is,

$$M'_A = M\left(1 - C^2 \frac{K\,s/4}{K\,s/4 + \Sigma K_{bb}}\right) \tag{4}$$

Hence the modified stiffness of the column at A is (noting $K = \frac{I}{L} = \frac{M}{4E\Theta}$)

$$K'_A = \frac{M'_A}{4E\Theta}$$

$$= \frac{M}{4E\Theta}\left(1 - \frac{C^2\,K\,s/4}{K\,s/4 + \Sigma K_{bb}}\right)$$

or, modified stiffness of column end A is,

$$K'_A = \frac{K\,s}{4}\left(1 - \frac{C^2\,K\,s/4}{K\,s/4 + \Sigma K_{bb}}\right) \tag{5}$$

The column end A frames into beams of total stiffness $\Sigma K_{bt}$. Therefore, when axial load $P$ is carried by the column, the stiffness of the joint A is,

$$K''_A = K'_A + \Sigma K_{bt} \tag{6}$$

When the axial load $P$ is increased gradually, it reaches a critical value $P_{cr}$ when a very small lateral load causes an indefinitely large lateral displacement. In other words, the total stiffness $K''_A$ at end A vanishes at the critical load $P_{cr}$. Inserting this condition in Equation (6)

$$\frac{K\,s}{4}\left(1 - \frac{C^2\,K\,s/4}{K\,s/4 + \Sigma K_{bb}}\right) + \Sigma K_{bt} = 0 \tag{7}$$

By a few simple manipulation of Equation (7), it can be shown that the total stiffness $K''_B$ also vanishes at joint B simultaneously, that is

$$\frac{K\,s}{4}\left(1 - \frac{C^2\,K\,s/4}{K\,s/4 + \Sigma K_{bt}}\right) + \Sigma K_{bb} = 0 \tag{8}$$

Therefore, for any given column, it will be sufficient to consider one of the ends only.

In equation (7), the coefficient $s$ and $C$ depend on the ratio $\frac{P}{P_E}$. For given values of stiffness $\Sigma K_{bb}$, $\Sigma K_{bt}$ and $K$, Equation (7) is used first to determine the ratio $\frac{P_{cr}}{P_E}$, if the column is braced. As the Euler critical load $P_E = \frac{\pi^2 EI}{L^2}$ refers to the pin-ended column, the effective length of braced column with other fixing condition can be determined as:

$$l_{ef} = L\sqrt{\frac{P_E}{P_{cr}}} \tag{9}$$

A similar derivation will be appropriate for an unbraced column. The effective length ratios $\frac{l_{ef}}{l}$ are plotted in Fig. 23 and 24 of the Code for braced and unbraced columns respectively.

While using the charts in Fig. 23 and 24 of the Code, the following points should be kept in view:

a) For braced frames, the beam stiffness $K_b$ should modified as,

$$K_b = \frac{1}{2}\cdot\frac{I}{L}$$

b) In unbraced frames, the beams are bent in double curvature. Therefore the beam stiffness in such frames should be modified as

$$K_b = 1.5 \times \frac{I}{L}$$

c) For determining $\beta_1$ and $\beta_2$ the 'limited substitute frame' (*see* Fig. E-68) should be used.

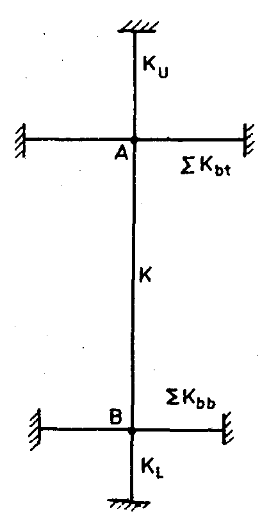



*Fig. E-68 Limited Substitute Frame*

At top joint,

$$\beta_1 = \frac{K+K_u}{K+K_u+\Sigma K_{bt}}$$

At bottom joint,

$$\beta_2 = \frac{K+K_1}{K+K_1+\Sigma K_{bb}}$$

Where $K_u$ and $K_1$ are the stiffness of columns framing above the top joint A and below the bottom joint B (*see* Fig. E-67).

*Illustrative Example*

Frame braced against sidesway, column AB in Fig. E-69 is under consideration. The stiffnesses ($K = \frac{I}{L}$) are shown adjacent to each element. Determine the effective length of AB.

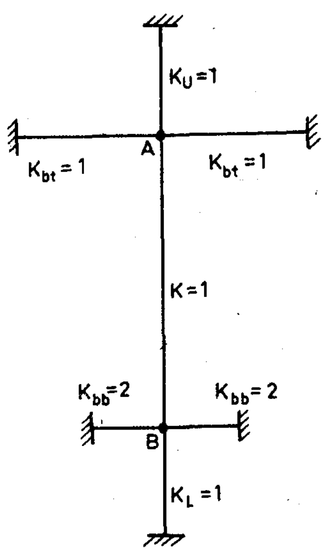



*Fig. E-69 Stiffness Values*

*Top joint A:*

$K = 1$

$K_u = 1$

$K_{bt} = \frac{1}{2}(1 + 1)$ (Factor ½ is introduced because the column is braced)

$$\beta_1 = \frac{K+K_u}{K+K_u+\Sigma K_{bt}}$$

$$= \frac{2}{2+1} = 0.67$$

*Bottom joint B:*

$K = 1$

$K_1 = 1$

$K_{bb} = \frac{1}{2}(2 + 2) = 2$

$$\beta_2 = \frac{K+K_1}{K+K_1+\Sigma K_{bb}}$$

$$= \frac{2}{2+2} = 0.5$$

From Fig. 24 of the Code,

$$\frac{l_{ef}}{l} = 0.73$$

Effective length = 0.73 × unsupported length

If the frame is *not* braced against sidesway in Fig. E-68:

*Top joint A:*

$K = 1$

$K_u = 1$

$K_{bt} = 1.5 (1 + 1) = 3.0$ (Factor 1.5 accounts for the frame being unbraced)

$$\beta_1 = \frac{K+K_u}{K+K_u+\Sigma K_{bt}}$$

$$= \frac{2}{2+3} = 0.4$$

*Bottom joint B:*

$K = 1$

$K_u = 1$

$K_{bt} = 1.5 (2 + 2) = 6.0$

$$\beta_2 = \frac{K+K_L}{K+K_L+\Sigma K_{bb}}$$

$$= \frac{2}{2+6} = 0.25$$

From Fig. 25 of the Code (for unbraced frames)

$$\frac{l_{ef}}{l} = 1.25$$

Therefore, effective length $l_{ef} = 1.25 \times$ unsupported length.

# APPENDIX E

## MOMENTS OF RESISTANCE FOR RECTANGULAR AND T-SECTIONS

### *E-1.* RECTANGULAR SECTIONS

*E-1.1 Sections without Compression Reinforcement*

a) In underreinforced sections, the depth of neutral axis $X_u$ will be smaller than $X_{u,\,max}$. The strain in steel at the limit state of collapse will be more than $\frac{0.87 f_y}{E_s} + 0.002$ and, the design stress in steel will be $0.87\, f_y$.

The depth of neutral axis is obtained by equating the forces of tension and compression (*see* Fig. E-49).

$$0.36\, f_{ck}.X_u.b = 0.87\, f_y.A_{st}$$

$$\text{or } \frac{X_u}{d} = \frac{0.87\, f_y\, A_{st}}{0.36\, f_{ck}\, b.d}$$

b) The moment of resistance of the section is equal to the product of the tensile force and level arm.

$$M_u = TZ$$

$$TZ = A_{st}(0.87\, f_y)\,(d - 0.416\, x_u)$$

$$= A_{st}(0.87\, f_y)\, d \left(1 - 0.416 \frac{x_u}{d}\right)$$

$$\therefore M_u = 0.87\, f_y\, A_{st}\, d \left(1 - \frac{A_{st}\, f_y}{bd\, f_{ck}}\right)$$

c) The limiting moment on a section is obtained by considering the maximum allowed depth of neutral axis ($X_{u,\,max}$) given in *37.1*.

Taking the moment of compressive force $C$ about the level of tension steel,

$$M_{u,\,lim} = C \times \text{Lever arm}$$

$$= 0.36\, f_{ck}.X_{u,\,max}.b\,(d - 0.42\, X_{u,\,max})$$

$$= 0.36\, f_{ck} \frac{X_{u,\,max}}{d} \left(1 - \frac{0.42\, X_{u,\,max}}{d}\right) bd^2$$

d) As shown above, the limits on $\frac{X_{u,\,max}}{d}$ restrict the moment capacity of a singly reinforced section to $M_{u,\,lim}$. If the applied moment exceeds this value, the section is to be redesigned either (1) by changing the cross-sectional dimensions of the member, or (2) by designing the section as a doubly reinforced section according to *E-1-2*.

*E-1.2 Section with Compression Reinforcement*—Doubly reinforced sections are generally adopted when the dimensions of the beam have been predetermined from other considerations and the design moment exceeds the moment of resistance of a single reinforced section. The additional moment of resistance needed is obtained by providing compression reinforcement and additional tensile reinforcement. The moment of resistance of a doubly reinforced section is thus the sum of the limiting moment of resistance $M_{u,\,lim}$ of a singly reinforced section and the additional moment of resistance $M_{u2}$. Given the value of $M_u$ which is greater than $M_{u,\,lim}$, the value of $M_{u2}$ can be calculated.

$$M_{u2} = M_u - M_{u,\,lim}$$

The lever arm for the additional moment of resistance is equal to the distance between centroids of tension reinforcement, that is $(d - d')$ where $d'$ is the distance from the extreme compression fibre to the centroid of compression steel.

$$\therefore M_{u2} = A_{st2}\,(0.87\, f_y)\,(d - d')$$

$$\text{Also } M_{u2} = A_{sc}\,(f_{sc} - f_{cc})\,(d - d')$$

$$\therefore A_{sc}\,(f_{sc} - f_{cc}) = A_{st2}\,(0.87\, f_y)$$

where

$A_{st2}$ = Area of additional tensile reinforcement

$A_{sc}$ = Area of compression reinforcement

$f_{sc}$ = Stress in compression reinforcement

$f_{cc}$ = Compressive stress in concrete at the level of the centroid of compression reinforcement

Any two of the above three equations may be used for finding $A_{st2}$ and $A_{sc}$.

Total reinforcement $A_{st} = A_{st1} + A_{st2}$

where

$$A_{st1} = P_{t,\,lim}\frac{bd}{100}$$

Values of $f_{sc}$ and $f_{cc}$ are to be calculated before calculating $A_{sc}$. This can be approached by taking the depth of neutral axis as equal to $X_{u,\,max}$ and the strain at the level of compression reinforcement will be equal to $0.0035\left(1-\dfrac{d'}{X_{u,\,max}}\right)$

## *E-2.* FLANGED SECTION

*E-2.1, E-2.2* and *E-2.3* The moment of resistance of a T-beam can be considered as the sum of moment of resistance of the concrete in the web of width $b_w$ and the contribution due to the flanges. The maximum moment of resistance is obtained when depth of neutral axis is $X_{u,max}$. When the thickness of the flange is small, that is 0.2 *d,* the stress in the flange will be uniform or nearly uniform and the centroid of the compressive force in the flange can be taken at $D_f/2$ from the extreme compression fibre. Therefore, the following expression is obtained for limiting moment of resistance of T-beams with small values of $D_f/d$:

$$M_{u,\,lim} = M_{u,\,lim,\,web} + 0.446\,f_{ck} \times (b_f - b_a)\,D_f\,(d - \frac{d_f}{2})$$

where $M_{u,\,lim,\,web} = 0.36\ f_{ck}b_wX_{u,\,max}\,(d-0.416\,X_{u,\,max})$.

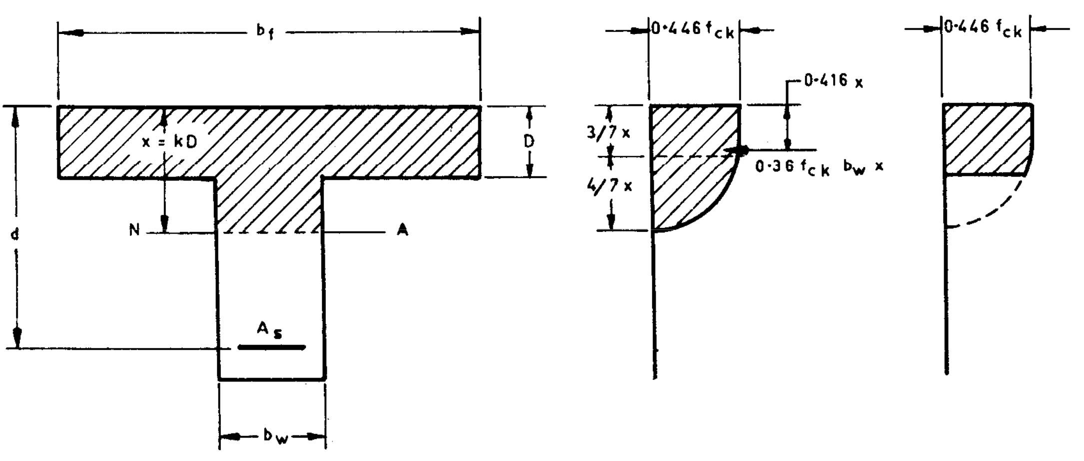
This equation is same as given in *E-2.2.* When the flange thickness is greater than about 0.2d, the above equation is not correct because the stress distribution in the flange would not be uniform. The expression given in *E-2.2.1* is an approximation which makes allowance for the variation of stress in the flange.

If the actual design moment ($M_u$) is less than the limiting moment, calculate the moment of resistance ($M_r$) for the condition when the neutral axis is at the bottom of the flange.

$$M_r = 0.36\,f_{ck}\,b_fD_f\,(d-0.416\,D_f)$$

If the actual design moment is less than $M_r$, design the T-beam as a rectangular beam of width $b_f$ and effective depth *d.* The reinforcement can then be calculated as per a rectangular beam.

*Method of Calculating Tension Reinforcement in T-Beams (by Exact Method)*




a) STRESS DISTRIBUTION IN WEB PORTION WHEN NA LIES BELOW THE FLANGE

b) STRESS DISTRIBUTION IN FLANGE PORTION WHEN NA LIES BELOW THE FLANGE

*Fig. E-70 Distribution of Stress in a T-Beam*

*Case*(i) when the neutral axis lies between the bottom of the flange and the position in the web, when the stress block rectangular portion just reaches the bottom of the flange:

$$\frac{M_u}{b_w d^2 f_{ck}} = 0.446\left(\frac{D}{d}\right)\left[1-\frac{1}{3}\left(1-\frac{3}{7}k\right)\left(\frac{1-\frac{3k}{7}}{k-\frac{3k}{7}}\right)^2\right]\left(\frac{b_f}{b_w}-1\right)$$

$$\times\left\{1-\frac{\frac{D}{d}\left[1-2\left(1-\frac{3k}{7}\right)\left(\frac{1-\frac{3k}{7}}{k-\frac{3k}{7}}\right)^2\left\{\frac{k}{7}+\frac{1}{4}\left(1-\frac{3k}{7}\right)\right\}\right]}{2\left[1-\frac{1}{3}\left(1-\frac{3k}{7}\right)\left(\frac{1-\frac{3k}{7}}{k-\frac{3k}{7}}\right)^2\right]}\right\}$$

$$+0.36k\frac{D}{d}\left[1-0.416k\frac{D}{d}\right] \quad ..\ (1)$$

$$\frac{p_t(100)}{f_{ck}} = \frac{4460}{0.87}\left(\frac{1}{f_y}\right)\left(\frac{D}{d}\right)\times\left\{\left[1-\frac{1}{3}\left(1-\frac{3k}{7}\right)\left(\frac{1-\frac{3k}{7}}{k-\frac{3k}{7}}\right)^2\right]\left(\frac{b_f}{b_w}-1\right)+\frac{0.36k}{0.446}\right\} \quad ..\ (2)$$

*Case*(ii) when the neutral axis lies below the limit specified in case(i):

$$\frac{M_u}{b_w d^2 f_{ck}} = 0.446\left(\frac{D}{d}\right)\left(\frac{b_f}{b_w}-1\right)\left(1-\frac{D}{2d}\right)$$

$$+0.36k\left(\frac{D}{d}\right)\left(1-0.416k\frac{D}{d}\right) \quad ..\ (3)$$

$$\frac{p_t(100)}{f_{ck}} = \frac{4460}{0.87 f_y}\left(\frac{D}{d}\right)\left(\frac{b_f}{b_w}-1\right)$$

$$+\frac{3600}{0.87}\left(\frac{k}{f_y}\right)\left(\frac{D}{d}\right) \quad ..\ (4)$$

where,

$M_u$ = Design moment for limit state design,

$f_{ck}$ = Characteristic compressive strength of concrete (N/mm²),

$f_y$ = Characteristic yield strength of steel, (N/mm²)

$p_t = \dfrac{A_s(100)}{b_w d}$

$b_f$, $b_w$, $D$, $d$ are as indicated in Fig. E-70.

Using Equations (1), (2), (3) and (4), Tables E-9, E-10 and E-11 have been prepared for three grades of steel, namely $f_y = 250$ N/mm², $f_y = 415$ N/mm² and $f_y = 500$ N/mm² which will be convenient in the design of T-beams.

**Notes for Tables E-9, E-10 and E-11**

NOTE 1—The first value of $\frac{p_t(100)}{f_{ck}}$ in each column correspond to the case when neutral axis is at the bottom of the flange. The corresponding value of $\frac{M_u}{f_{ck} b_w d^2}$ are also given above the first value of $\frac{p_t(100)}{f_{ck}}$. For lower values of $\frac{M_u}{f_{ck} b_w d^2}$ the section is to be considered as a rectangular section of width $b_f$ and depth $d$.

NOTE 2—The value of $\frac{p_t(100)}{f_{ck}}$ in each column correspond to the value of the limiting moment of resistance of the beam or to the case when $\frac{p_t(100)}{f_{ck}}$ approaches a value of 26.6. This would be close to the maximum permissible reinforcement when $f_{ck} = 15$ N/mm². For higher grades of concrete the maximum permissible reinforcement would be reached at lower values than 26.6. The value of $\frac{M_u}{f_{ck} b_w d^2}$ corresponding to the last value of $\frac{p_t(100)}{f_{ck}}$ in each column are given under the last value of $\frac{p_t(100)}{f_{ck}}$.

**TABLE E-9 REINFORCEMENT PERCENTAGE FACTOR $\frac{p_t(100)}{f_{ck}}$ FOR SINGLY REINFORCED T-BEAMS ($f_y$ = 250 N/mm²)**

| $\frac{M_u}{b_w d^2 f_{ck}}$ | $D_f/d$ = [illegible], $b_f/b_w$ = 10 | 8 | 7 | 6 | 5 | 4 | 3 | 2 | 1 |
|---|---|---|---|---|---|---|---|---|---|
| 0.075 | | | | | | | | 0.083 / 4.14 | |
| 0.090 | | | | | | | | 4.36 | |
| 0.105 | | | | | | | | 5.15 | |
| 0.120 | | | | | | | 0.120 / 6.22 | 5.95 | |
| 0.135 | | | | | | | 6.79 | 6.80 | |
| 0.150 | | | | | | | 7.32 | 7.71 | |
| 0.165 | | | | | | 0.171 / 8.29 | 8.12 | 8.67 | |
| 0.180 | | | | | | 8.79 | 8.94 | 9.60 | |
| 0.195 | | | | | | 9.57 | 9.82 | 10.82 | |
| 0.210 | | | | | 0.214 / 10.40 | 10.50 | 10.71 | 11.56 / 0.204 | |
| 0.225 | | | | | 10.96 | 11.11 | 11.89 | | |
| 0.240 | | | | | 11.70 | 11.91 | 12.82 | | |
| 0.255 | | | | 0.237 / 12.44 | 12.40 | 12.69 | 13.93 / 0.234 | | |
| 0.270 | | | | 13.12 | 13.30 | 13.82 | | | |
| 0.285 | | | | 13.91 | 14.50 | 14.80 | | | |
| 0.300 | | | 0.290 / 14.52 | 14.60 | 15.00 | 15.99 | | | |
| 0.315 | | | 15.30 | 15.46 | 15.91 | 16.40 / 0.300 | | | |
| 0.330 | | | 16.00 | 16.30 | 16.96 | | | | |
| 0.345 | | 0.342 / 16.50 | 16.80 | 17.11 | 17.99 | | | | |
| 0.360 | | 17.40 | 17.68 | 18.00 | 19.40 / 0.350 | | | | |
| 0.375 | | 18.26 | 18.40 | 18.90 | | | | | |
| 0.390 | | 19.09 | 19.29 | 20.17 | | | | | |
| 0.405 | | 19.80 | 20.10 | 21.14 | | | | | |
| 0.420 | 0.420 / 20.74 | 20.61 | 21.50 | 21.64 / 0.410 | | | | | |
| 0.435 | 21.14 | 21.42 | 22.00 | | | | | | |
| 0.450 | 21.90 | 22.26 | 22.31 | | | | | | |
| 0.465 | 22.60 | 23.22 | 24.17 / 0.461 | | | | | | |
| 0.480 | 23.40 | 24.19 | | | | | | | |
| 0.495 | 24.19 | 25.17 | | | | | | | |
| 0.510 | 24.99 | 26.31 | | | | | | | |
| 0.525 | 25.70 | 26.6 / 0.511 | | | | | | | |
| 0.540 | 26.80 / 0.540 | | | | | | | | |

| $\frac{M_u}{b_w d^2 f_{ck}}$ | $D_f/d$ = [illegible], $b_f/b_w$ = 10 | 8 | 7 | 6 | 5 | 4 | 3 | 2 | 1 |
|---|---|---|---|---|---|---|---|---|---|
| 0.075 | | | | | | | | | |
| 0.090 | | | | | | | | 0.097 / 4.74 | |
| 0.105 | | | | | | | | 5.15 | |
| 0.120 | | | | | | | | 5.99 | |
| 0.135 | | | | | | | 0.146 / 7.11 | 6.81 | |
| 0.150 | | | | | | | 7.35 | 7.70 | |
| 0.165 | | | | | | | 8.17 | 8.90 | |
| 0.180 | | | | | | | 8.92 | 9.57 | |
| 0.195 | | | | | | 0.194 / 9.49 | 9.79 | 10.66 | |
| 0.210 | | | | | | 10.31 | 10.60 | 11.71 / 0.200 | |
| 0.225 | | | | | | 11.11 | 11.56 | | |
| 0.240 | | | | | 0.243 / 11.66 | 11.89 | 12.56 | | |
| 0.255 | | | | | 12.51 | 12.79 | 13.64 | | |
| 0.270 | | | | | 13.27 | 13.60 | 14.06 / 0.267 | | |
| 0.285 | | | | 0.290 / 14.24 | 14.09 | 14.35 | | | |
| 0.300 | | | | 14.70 | 14.99 | 15.55 | | | |
| 0.315 | | | | 15.49 | 15.73 | 16.63 | | | |
| 0.330 | | | | 16.37 | 16.66 | 17.90 / 0.330 | | | |
| 0.345 | | | 0.340 / 16.61 | 17.06 | 17.33 | | | | |
| 0.360 | | | 17.67 | 17.85 | 18.35 | | | | |
| 0.375 | | | 18.44 | 18.73 | 19.66 | | | | |
| 0.390 | | 0.390 / 18.90 | 19.25 | 19.62 | 20.92 / 0.386 | | | | |
| 0.405 | | 19.85 | 20.00 | 20.54 | | | | | |
| 0.420 | | 20.66 | 20.82 | 21.11 | | | | | |
| 0.435 | | 21.49 | 21.70 | 22.01 | | | | | |
| 0.450 | | 22.32 | 22.60 | 23.40 / 0.445 | | | | | |
| 0.465 | | 23.00 | 23.49 | | | | | | |
| 0.480 | 0.495 / 23.71 | 23.80 | 24.31 | | | | | | |
| 0.495 | 24.34 | 24.60 | 25.62 | | | | | | |
| 0.510 | 25.00 | 25.37 | 26.19 / 0.506 | | | | | | |
| 0.525 | 25.70 | 26.40 | | | | | | | |
| 0.540 | 26.60 / 0.540 | 26.80 / 0.527 | | | | | | | |

| $\frac{M_u}{b_w d^2 f_{ck}}$ | $D_f/d$ = [illegible], $b_f/b_w$ = 8 | 7 | 6 | 5 | 4 | 3 | 2 | 1 |
|---|---|---|---|---|---|---|---|---|
| 0.075 | | | | | | | | |
| 0.090 | | | | | | | | |
| 0.105 | | | | | | | 0.112 / 5.34 | |
| 0.120 | | | | | | | 5.97 | |
| 0.135 | | | | | | | 6.80 | |
| 0.150 | | | | | | | 7.65 | |
| 0.165 | | | | | | 0.166 / 8.51 | 8.56 | |
| 0.180 | | | | | | 8.94 | 9.51 | |
| 0.195 | | | | | | 9.76 | 10.54 | |
| 0.210 | | | | | | 10.60 | 11.96 | |
| 0.225 | | | | | 0.264 / 11.60 | 11.47 | 12.21 / 0.217 | |
| 0.240 | | | | | 11.91 | 12.40 | | |
| 0.255 | | | | | 12.76 | 13.39 | | |
| 0.270 | | | | | 13.57 | 14.00 | | |
| 0.285 | | | | 0.280 / 13.05 | 14.40 | 15.04 / 0.263 | | |
| 0.300 | | | | 14.98 | 15.32 | | | |
| 0.315 | | | | 15.71 | 16.26 | | | |
| 0.330 | | | 0.336 / 16.21 | 16.56 | 17.28 | | | |
| 0.345 | | | 17.10 | 17.37 | 18.40 | | | |
| 0.360 | | | 17.91 | 18.23 | 19.00 / 0.350 | | | |
| 0.375 | | | 18.60 | 19.56 | | | | |
| 0.390 | | 0.401 / 19.40 | 19.50 | 20.64 | | | | |
| 0.405 | | 20.09 | 20.56 | 21.30 | | | | |
| 0.420 | | 20.90 | 21.19 | 22.56 | | | | |
| 0.435 | | 21.66 | 22.06 | 22.49 / 0.432 | | | | |
| 0.450 | 0.440 / 22.17 | 22.40 | 23.00 | | | | | |
| 0.465 | 23.00 | 22.33 | 24.00 | | | | | |
| 0.480 | 23.07 | 24.14 | 25.11 | | | | | |
| 0.495 | 24.07 | 25.01 | 25.91 / 0.498 | | | | | |
| 0.510 | 25.49 | 25.07 | | | | | | |
| 0.525 | 26.30 | 26.40 / 0.521 | | | | | | |
| 0.540 | 26.61 / 0.531 | | | | | | | |

| $\frac{M_u}{b_w d^2 f_{ck}}$ | $D_f/d$ = [illegible], $b_f/b_w$ = 7 | 6 | 5 | 4 | 3 | 2 | 1 |
|---|---|---|---|---|---|---|---|
| 0.075 | | | | | | | |
| 0.090 | | | | | | | |
| 0.105 | | | | | | | |
| 0.120 | | | | | | | |
| 0.135 | | | | | | 0.132 / 6.63 | |
| 0.150 | | | | | | 7.64 | |
| 0.165 | | | | | | 8.50 | |
| 0.180 | | | | | | 9.42 | |
| 0.195 | | | | | 0.190 / 9.95 | 10.10 | |
| 0.210 | | | | | 10.58 | 11.45 | |
| 0.225 | | | | | 11.49 | 12.50 | |
| 0.240 | | | | | 12.30 | 12.90 / 0.229 | |
| 0.255 | | | | 0.246 / 13.17 | 13.20 | | |
| 0.270 | | | | 13.99 | 14.19 | | |
| 0.285 | | | | 14.41 | 15.15 | | |
| 0.300 | | | | 15.34 | 16.20 | | |
| 0.315 | | | | 16.11 | 16.99 / 0.300 | | |
| 0.330 | | | 0.339 / 16.59 | 16.99 | | | |
| 0.345 | | | 17.35 | 17.99 | | | |
| 0.360 | | | 18.20 | 18.90 | | | |
| 0.375 | | | 19.00 | 19.98 | | | |
| 0.390 | | 0.397 / 20.06 | 19.92 | 21.09 / 0.380 | | | |
| 0.405 | | 20.54 | 20.77 | | | | |
| 0.420 | | 21.10 | 21.07 | | | | |
| 0.435 | | 22.00 | 22.61 | | | | |
| 0.450 | | 22.07 | 23.07 | | | | |
| 0.465 | 0.463 / 23.23 | 23.00 | 24.01 | | | | |
| 0.480 | 24.14 | 24.50 | 24.10 / 0.478 | | | | |
| 0.495 | 24.97 | 25.49 | | | | | |
| 0.510 | 25.99 | 26.39 | | | | | |
| 0.525 | 26.60 / 0.529 | 26.60 / 0.515 | | | | | |
| 0.540 | | | | | | | |

| $\frac{M_u}{b_w d^2 f_{ck}}$ | $D_f/d$ = [illegible], $b_f/b_w$ = 6 | 5 | 4 | 3 | 2 | 1 | $D_f/d$ = [illegible], $b_f/b_w$ = 4 | 3 | 2 | 1 | [illegible] |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.075 | | | | | | | | | | | |
| 0.090 | | | | | | | | | | | |
| 0.105 | | | | | | | | | | | |
| 0.120 | | | | | | | | | | | |
| 0.135 | | | | | | | | | | | |
| 0.150 | | | | | | | | | | | |
| 0.165 | | | | | 0.162 / 8.28 | | | | | | |
| 0.180 | | | | | 9.40 | | | | | | |
| 0.195 | | | | | 10.32 | | | | 0.207 / 11.04 | | |
| 0.210 | | | | | 11.21 | | | | 11.36 | | |
| 0.225 | | | | | 12.30 | | | | 12.22 | | |
| 0.240 | | | | 0.242 / 12.60 | 13.40 / 0.240 | | | | 13.21 | | |
| 0.255 | | | | 13.16 | | | | | 14.07 | | |
| 0.270 | | | | 14.00 | | | | | 14.60 / 0.271 | | |
| 0.285 | | | | 14.70 | | | | | | | |
| 0.300 | | | | 15.80 | | | | 0.310 / 16.57 | | | |
| 0.315 | | | 0.321 / 16.50 | 16.91 | | | | 16.80 | | | |
| 0.330 | | | 16.90 | 17.97 | | | | 17.03 | | | |
| 0.345 | | | 17.01 | 18.17 / 0.330 | | | | 18.79 | | | |
| 0.360 | | | 18.71 | | | | | 19.79 | | | |
| 0.375 | | | 19.65 | | | | | 20.00 | | | |
| 0.390 | | | 20.50 | | | | | 21.40 | | | |
| 0.405 | | 0.408 / 20.74 | 21.53 | | | | 0.394 / 23.00 | 22.20 / 0.400 | | | |
| 0.420 | | 21.65 | 22.36 | | | | 23.47 | | | | |
| 0.435 | | 22.56 | 23.44 / 0.435 | | | | 25.44 | | | | |
| 0.450 | | 23.42 | | | | | 24.79 | | | | |
| 0.465 | | 24.34 | | | | | 25.19 | | | | |
| 0.480 | 0.481 / 24.99 | 25.21 | | | | | 26.45 | | | | |
| 0.495 | 25.40 | 26.06 | | | | | 26.65 / 0.490 | | | | |
| 0.510 | 26.31 | 26.46 / 0.500 | | | | | | | | | |
| 0.525 | 26.80 / 0.517 | | | | | | | | | | |
| 0.540 | | | | | | | | | | | |

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TABLE E-10 REINFORCEMENT PERCENTAGE FACTOR $\frac{A_s(100)}{f_{ck}}$ FOR SINGLY REINFORCED T-BEAMS ($f_y$ = 415 N/mm²)

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**TABLE E-11 REINFORCEMENT PERCENTAGE FACTOR $\frac{p_t(100)}{f_{ck}}$ FOR SINGLY REINFORCED T-BEAMS ($f_y$ = 500 N/mm²)**

**$d/D_f$ = 8**

| $\frac{M_u}{b_w d^2 f_{ck}}$ | $b_f/b_w$ = 10 | 8 | 7 | 6 | 5 | 4 | 3 | 2 | 1 |
|---|---|---|---|---|---|---|---|---|---|
| 0.080 | | | | | | | | 0.085/2.07 | |
| 0.100 | | | | | | | | 2.46 | |
| 0.120 | | | | | | | 0.128/3.11 | 3.00 | |
| 0.140 | | | | | | | 3.42 | 3.57 | |
| 0.160 | | | | | | 0.171/4.15 | 3.95 | 4.19 | |
| 0.180 | | | | | | 4.37 | 4.50 | 5.06/0.185 | |
| 0.200 | | | | | 0.214/5.18 | 4.91 | 5.09 | | |
| 0.220 | | | | | 5.33 | 5.44 | 5.73 | | |
| 0.240 | | | | | 5.86 | 6.00 | 6.34/0.230 | | |
| 0.260 | | | | 0.275/6.22 | 6.38 | 6.61 | | | |
| 0.280 | | | | 6.83 | 6.93 | 7.28 | | | |
| 0.300 | | | 0.299/7.26 | 7.35 | 7.51 | 7.62/0.290 | | | |
| 0.320 | | | 7.79 | 7.86 | 8.14 | | | | |
| 0.340 | | 0.342/8.30 | 8.31 | 8.43 | 8.64 | | | | |
| 0.360 | | 8.74 | 8.86 | 9.01 | 8.90/0.342 | | | | |
| 0.380 | | 9.26 | 9.37 | 9.60 | | | | | |
| 0.400 | | 9.80 | 9.96 | 10.18/0.394 | | | | | |
| 0.420 | 0.420/10.37 | 10.12 | 10.56 | | | | | | |
| 0.440 | 10.88 | 10.87 | 11.26 | | | | | | |
| 0.460 | 11.20 | 11.45 | 11.67/0.447 | | | | | | |
| 0.480 | 11.71 | 12.09 | | | | | | | |
| 0.500 | 12.34 | 12.54/0.499 | | | | | | | |
| 0.520 | 12.77 | | | | | | | | |
| 0.540 | 13.31 | | | | | | | | |
| 0.560 | 13.86 | | | | | | | | |
| 0.580 | 14.51 | | | | | | | | |
| 0.600 | 15.20 | | | | | | | | |
| 0.620 | 15.31/0.605 | | | | | | | | |
| 0.640 | | | | | | | | | |
| 0.660 | | | | | | | | | |
| 0.680 | | | | | | | | | |
| 0.700 | | | | | | | | | |
| 0.720 | | | | | | | | | |
| 0.740 | | | | | | | | | |
| 0.760 | | | | | | | | | |
| 0.780 | | | | | | | | | |
| 0.800 | | | | | | | | | |
| 0.820 | | | | | | | | | |
| 0.840 | | | | | | | | | |
| 0.860 | | | | | | | | | |

**$d/D_f$ = 7**

| $\frac{M_u}{b_w d^2 f_{ck}}$ | $b_f/b_w$ = 10 | 8 | 7 | 6 | 5 | 4 | 3 | 2 | 1 |
|---|---|---|---|---|---|---|---|---|---|
| 0.080 | | | | | | | | 0.097/2.37 | |
| 0.100 | | | | | | | | 2.46 | |
| 0.120 | | | | | | | | 2.99 | |
| 0.140 | | | | | | | 0.143/3.56 | 3.55 | |
| 0.160 | | | | | | | 3.94 | 4.16 | |
| 0.180 | | | | | | 0.194/4.74 | 4.48 | 4.81 | |
| 0.200 | | | | | | 4.90 | 5.05 | 5.25/0.193 | |
| 0.220 | | | | | | 5.43 | 5.65 | | |
| 0.240 | | | | | 0.243/5.93 | 5.96 | 6.31 | | |
| 0.260 | | | | | 6.39 | 6.53 | 6.72/0.252 | | |
| 0.280 | | | | 0.291/7.12 | 6.91 | 7.14 | | | |
| 0.300 | | | | 7.35 | 7.46 | 7.80 | | | |
| 0.320 | | | | 7.86 | 8.02 | 8.18/0.311 | | | |
| 0.340 | | | 0.340/8.30 | 8.41 | 8.63 | | | | |
| 0.360 | | | 8.83 | 8.94 | 9.10 | | | | |
| 0.380 | | 0.368/[illegible] | 9.37 | 9.51 | 9.65/0.370 | | | | |
| 0.400 | | 9.81 | 9.89 | 10.11 | | | | | |
| 0.420 | | 10.32 | 10.41 | 10.76 | | | | | |
| 0.440 | | 10.85 | 10.99 | 11.12/0.429 | | | | | |
| 0.460 | | 11.38 | 11.81 | | | | | | |
| 0.480 | 0.485/11.67 | 11.92 | 12.18/0.469 | | | | | | |
| 0.500 | 12.29 | 12.48 | | | | | | | |
| 0.520 | 12.77 | 13.10 | | | | | | | |
| 0.540 | 13.29 | 13.79 | | | | | | | |
| 0.560 | 13.81 | 14.09/0.548 | | | | | | | |
| 0.580 | 14.35 | | | | | | | | |
| 0.600 | 14.89 | | | | | | | | |
| 0.620 | 15.47 | | | | | | | | |
| 0.640 | 16.08 | | | | | | | | |
| 0.660 | 16.70 | | | | | | | | |
| 0.680 | 16.99/0.666 | | | | | | | | |
| 0.700 | | | | | | | | | |
| 0.720 | | | | | | | | | |
| 0.740 | | | | | | | | | |
| 0.760 | | | | | | | | | |
| 0.780 | | | | | | | | | |
| 0.800 | | | | | | | | | |
| 0.820 | | | | | | | | | |
| 0.840 | | | | | | | | | |
| 0.860 | | | | | | | | | |

**$d/D_f$ = 6**

| $\frac{M_u}{b_w d^2 f_{ck}}$ | $b_f/b_w$ = 10 | 8 | 7 | 6 | 5 | 4 | 3 | 2 | 1 |
|---|---|---|---|---|---|---|---|---|---|
| 0.080 | | | | | | | | | |
| 0.100 | | | | | | | | 0.112/2.77 | |
| 0.120 | | | | | | | | 2.98 | |
| 0.140 | | | | | | | | 3.54 | |
| 0.160 | | | | | | | 0.168/4.15 | 4.12 | |
| 0.180 | | | | | | | 4.47 | 4.73 | |
| 0.200 | | | | | | | 5.02 | 5.45 | |
| 0.220 | | | | | | 0.224/5.54 | 5.59 | 5.49/0.201 | |
| 0.240 | | | | | | 5.95 | 6.20 | | |
| 0.260 | | | | | | 6.51 | 6.87 | | |
| 0.280 | | | | | 0.280/6.92 | 7.06 | 7.30/0.276 | | |
| 0.300 | | | | | 7.46 | 7.66 | | | |
| 0.320 | | | | | 7.99 | 8.29 | | | |
| 0.340 | | | | 0.336/8.31 | 8.54 | 8.92/0.338 | | | |
| 0.360 | | | | 8.96 | 9.11 | | | | |
| 0.380 | | | 0.369/9.20 | 9.48 | 9.74 | | | | |
| 0.400 | | | 9.88 | 10.04 | 10.42 | | | | |
| 0.420 | | | 10.44 | 10.60 | 10.63/0.406 | | | | |
| 0.440 | | 0.440/11.00 | 10.97 | 11.19 | | | | | |
| 0.460 | | 11.40 | 11.51 | 11.84 | | | | | |
| 0.480 | | 11.91 | 12.07 | 12.34/0.474 | | | | | |
| 0.500 | | 12.46 | 12.64 | | | | | | |
| 0.520 | | 13.00 | 13.27 | | | | | | |
| 0.540 | | 13.55 | 14.09/0.541 | | | | | | |
| 0.560 | 0.561/13.86 | 14.12 | | | | | | | |
| 0.580 | 14.37 | 14.72 | | | | | | | |
| 0.600 | 14.91 | 15.30 | | | | | | | |
| 0.620 | 15.45 | 15.77/0.611 | | | | | | | |
| 0.640 | 15.98 | | | | | | | | |
| 0.660 | 16.52 | | | | | | | | |
| 0.680 | 17.07 | | | | | | | | |
| 0.700 | 17.64 | | | | | | | | |
| 0.720 | 18.26 | | | | | | | | |
| 0.740 | 18.83 | | | | | | | | |
| 0.760 | 19.19/0.748 | | | | | | | | |
| 0.780 | | | | | | | | | |
| 0.800 | | | | | | | | | |
| 0.820 | | | | | | | | | |
| 0.840 | | | | | | | | | |
| 0.860 | | | | | | | | | |

**$d/D_f$ = 5**

| $\frac{M_u}{b_w d^2 f_{ck}}$ | $b_f/b_w$ = 10 | 8 | 7 | 6 | 5 | 4 | 3 | 2 | 1 |
|---|---|---|---|---|---|---|---|---|---|
| 0.080 | | | | | | | | | |
| 0.100 | | | | | | | | | |
| 0.120 | | | | | | | | 0.132/3.31 | |
| 0.140 | | | | | | | | 3.53 | |
| 0.160 | | | | | | | | 4.11 | |
| 0.180 | | | | | | | | 4.71 | |
| 0.200 | | | | | | | 0.190/4.97 | 5.36 | |
| 0.220 | | | | | | | 5.57 | 5.82/0.213 | |
| 0.240 | | | | | | | 6.15 | | |
| 0.260 | | | | | | 0.265/6.63 | 6.76 | | |
| 0.280 | | | | | | 7.05 | 7.41 | | |
| 0.300 | | | | | | 7.62 | 7.87/0.294 | | |
| 0.320 | | | | | 0.331/8.29 | 8.19 | | | |
| 0.340 | | | | | 8.54 | 8.79 | | | |
| 0.360 | | | | | 9.10 | 9.45 | | | |
| 0.380 | | | | | 9.65 | 9.93/0.374 | | | |
| 0.400 | | | | 0.397/9.96 | 10.25 | | | | |
| 0.420 | | | | 10.59 | 10.94 | | | | |
| 0.440 | | | | 11.13 | 11.48 | | | | |
| 0.460 | | | 0.463/11.62 | 11.72 | 11.98/0.454 | | | | |
| 0.480 | | | 12.07 | 12.29 | | | | | |
| 0.500 | | | 12.63 | 12.89 | | | | | |
| 0.520 | | 0.529/13.27 | 13.20 | 13.53 | | | | | |
| 0.540 | | 13.56 | 13.76 | 14.03/0.534 | | | | | |
| 0.560 | | 14.12 | 14.35 | | | | | | |
| 0.580 | | 14.67 | 14.95 | | | | | | |
| 0.600 | | 15.24 | 15.57 | | | | | | |
| 0.620 | | 15.80 | 16.08/0.615 | | | | | | |
| 0.640 | | 16.37 | | | | | | | |
| 0.660 | 0.663/16.59 | 16.97 | | | | | | | |
| 0.680 | 17.09 | 17.43 | | | | | | | |
| 0.700 | 17.64 | 18.13/0.695 | | | | | | | |
| 0.720 | 18.20 | | | | | | | | |
| 0.740 | 18.75 | | | | | | | | |
| 0.760 | 19.32 | | | | | | | | |
| 0.780 | 19.89 | | | | | | | | |
| 0.800 | 20.47 | | | | | | | | |
| 0.820 | 21.05 | | | | | | | | |
| 0.840 | 21.70 | | | | | | | | |
| 0.860 | 22.23/0.856 | | | | | | | | |

**$d/D_f$ = 4**

| $\frac{M_u}{b_w d^2 f_{ck}}$ | $b_f/b_w$ = 10 | 8 | 7 | 6 | 5 | 4 | 3 | 2 | 1 |
|---|---|---|---|---|---|---|---|---|---|
| 0.080 | | | | | | | | | |
| 0.100 | | | | | | | | | |
| 0.120 | | | | | | | | | |
| 0.140 | | | | | | | | | |
| 0.160 | | | | | | | | 0.162/4.14 | |
| 0.180 | | | | | | | | 4.70 | |
| 0.200 | | | | | | | | 5.31 | |
| 0.220 | | | | | | | | 5.97 | |
| 0.240 | | | | | | | 0.242/6.22 | 6.13/0.230 | |
| 0.260 | | | | | | | 6.74 | | |
| 0.280 | | | | | | | 7.34 | | |
| 0.300 | | | | | | | 7.95 | | |
| 0.320 | | | | | | 0.321/8.29 | 8.62 | | |
| 0.340 | | | | | | 8.78 | 8.89/0.328 | | |
| 0.360 | | | | | | 9.37 | | | |
| 0.380 | | | | | | 9.97 | | | |
| 0.400 | | | | | 0.406/10.37 | 10.64 | | | |
| 0.420 | | | | | 10.82 | 11.27 | | | |
| 0.440 | | | | | 11.41 | 11.64/0.425 | | | |
| 0.460 | | | | | 12.01 | | | | |
| 0.480 | | | | 0.485/12.44 | 12.62 | | | | |
| 0.500 | | | | 12.80 | 13.25 | | | | |
| 0.520 | | | | 13.49 | 13.93 | | | | |
| 0.540 | | | | 14.09 | 14.08/0.532 | | | | |
| 0.560 | | | 0.546/14.32 | 14.65 | | | | | |
| 0.580 | | | 14.93 | 15.26 | | | | | |
| 0.600 | | | 15.49 | 15.89 | | | | | |
| 0.620 | | | 16.10 | 16.37/0.610 | | | | | |
| 0.640 | | 0.647/16.59 | 16.69 | | | | | | |
| 0.660 | | 17.00 | 17.29 | | | | | | |
| 0.680 | | 17.55 | 17.99 | | | | | | |
| 0.700 | | 18.15 | 18.54 | | | | | | |
| 0.720 | | 18.73 | 19.10/0.717 | | | | | | |
| 0.740 | | 19.34 | | | | | | | |
| 0.760 | | 19.95 | | | | | | | |
| 0.780 | | 20.55 | | | | | | | |
| 0.800 | 0.809/20.74 | 21.19 | | | | | | | |
| 0.820 | 21.10 | 21.66/0.814 | | | | | | | |
| 0.840 | 21.66 | | | | | | | | |
| 0.860 | 22.25 | | | | | | | | |

**$d/D_f$ = 3**

| $\frac{M_u}{b_w d^2 f_{ck}}$ | $b_f/b_w$ = 8 | 7 | 6 | 5 | 4 | 3 | 2 | 1 |
|---|---|---|---|---|---|---|---|---|
| 0.080 | | | | | | | | |
| 0.100 | | | | | | | | |
| 0.120 | | | | | | | | |
| 0.140 | | | | | | | | |
| 0.160 | | | | | | | | |
| 0.180 | | | | | | | | |
| 0.200 | | | | | | | 0.207/5.52 | |
| 0.220 | | | | | | | 5.95 | |
| 0.240 | | | | | | | 6.62 | |
| 0.260 | | | | | | | 7.06/0.255 | |
| 0.280 | | | | | | | | |
| 0.300 | | | | | | 0.310/8.28 | | |
| 0.320 | | | | | | 8.60 | | |
| 0.340 | | | | | | 9.23 | | |
| 0.360 | | | | | | 9.90 | | |
| 0.380 | | | | | | 10.36/0.375 | | |
| 0.400 | | | | | | | | |
| 0.420 | | | | | 0.416/11.05 | | | |
| 0.440 | | | | | 11.87 | | | |
| 0.460 | | | | | 12.52 | | | |
| 0.480 | | | | | 13.10 | | | |
| 0.500 | | | | | 13.63/0.497 | | | |
| 0.520 | | | | 0.518/13.81 | | | | |
| 0.540 | | | | 14.55 | | | | |
| 0.560 | | | | 15.18 | | | | |
| 0.580 | | | | 15.82 | | | | |
| 0.600 | | | | 16.49 | | | | |
| 0.620 | | | 0.621/16.57 | 16.91/0.612 | | | | |
| 0.640 | | | 17.22 | | | | | |
| 0.660 | | | 17.72 | | | | | |
| 0.680 | | | 18.47 | | | | | |
| 0.700 | | | 19.12 | | | | | |
| 0.720 | | 0.723/19.34 | | | | | | |
| 0.740 | | 19.84 | | | | | | |
| 0.760 | | 20.40 | | | | | | |
| 0.780 | | 21.10 | | | | | | |
| 0.800 | | 21.76 | | | | | | |
| 0.820 | 0.820/22.10 | 22.41 | | | | | | |
| 0.840 | 22.48 | 23.06 | | | | | | |
| 0.860 | 23.11 | 23.40/0.853 | | | | | | |

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