

DESIGN OF
REINFORCED
CONCRETE
STRUCTURES

N SUBRAMANIAN

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BRIEF CONTENTS

Features of the Book iv

Foreword vi

Preface vii

Detailed Contents xi

List of Symbols xvii

1. Introduction to Reinforced Concrete	1
2. Structural Forms	45
3. Loads and Load Combinations	70
4. Basis of Structural Design	102
5. Flexural Analysis and Design of Beams	142
6. Design for Shear	214
7. Design for Effective Bond between Concrete and Steel	262
8. Design for Torsion	306
9. Design of One-way Slabs	334
10. Design of Two-way Slabs	362
11. Design of Flat Plates and Flat Slabs	415
12. Serviceability Limit States: Deflection and Crack Control	467
13. Design of Axially Loaded Short Columns	506
14. Design of Columns with Moments	546
15. Design of Footings and Pile Caps	585
16. Design of RC Walls and Structural Walls	644
17. Design of Staircases	702
18. Design of Tension Members	726
19. Design of Joints	743
20. Design of Multi-storey Buildings	790

Appendix A: Properties of Soils 809

Appendix B: Design Using Strut-and-tie Model 813

Appendix C: Analysis and Design Aids 822

Appendix D: Practical Tips and Some Rules of Thumb 839

Appendix E: Conversion Factors 850

Bibliography 852

Index 853

About the Author 857

DETAILED CONTENTS

Features of the Book iv

Foreword vi

Preface vii

Brief Contents x

List of Symbols xvii

1. Introduction to Reinforced Concrete	1		
1.1 Introduction	1		
1.1.1 Brief History	2		
1.1.2 Advantages and Disadvantages of Concrete	3		
1.2 Concrete-making Materials	4		
1.2.1 Cement (Portland Cement and Other Cements)	4		
1.2.2 Aggregates	8		
1.2.3 Water	10		
1.2.4 Admixtures	11		
1.3 Proportioning of Concrete Mixes	13		
1.4 Hydration of Cement	16		
1.5 Types of Concrete	17		
1.5.1 Ready-mixed Concrete	17		
1.5.2 High-performance Concrete	18		
1.5.3 Structural Lightweight Concrete	20		
1.5.4 Fibre-reinforced Concrete	21		
1.5.5 Ductile Fibre-reinforced Cementitious Composites	21		
1.5.6 Ferrocement	23		
1.6 Reinforcing Steel	24		
1.6.1 Corrosion of Rebars	26		
1.7 Concrete Mixing, Placing, Compacting, and Curing	27		
1.8 Properties of Fresh and Hardened Concrete	29		
1.8.1 Workability of Concrete	29		
1.8.2 Compressive Strength	29		
1.8.3 Stress–Strain Characteristics	32		
1.8.4 Tensile Strength	33		
1.8.5 Bearing Strength	33		
1.8.6 Modulus of Elasticity and Poisson’s Ratio	33		
1.8.7 Strength under Combined Stresses	35		
1.8.8 Shrinkage and Temperature Effects	35		
1.8.9 Creep of Concrete	36		
1.8.10 Non-destructive Testing	36		
1.9 Durability of Concrete	36		
2. Structural Forms	45		
2.1 Introduction	45		
2.2 Basic Structural Elements	46		
2.2.1 Footings	47		
2.2.2 Columns	47		
2.2.3 Beams	48		
2.2.4 Slabs	48		
2.2.5 Walls	48		
2.2.6 Trusses	49		
2.3 Floor and Roof Systems	51		
2.3.1 Bearing Wall Systems	51		
2.3.2 One-way and Two-way Slab Systems	52		
2.3.3 Two-way Flat Plates and Flat Slabs	54		
2.3.4 Grid Floors	56		
2.3.5 Composite Floors	56		
2.4 Precast and Prestressed Concrete Buildings	56		
2.5 Lateral Load Resisting Systems	57		
2.5.1 Rigid or Moment-resisting Frames	58		
2.5.2 Shear-walled Frame Systems	58		
2.5.3 Outrigger and Belt Truss Systems	59		
2.5.4 Framed-tube Systems	60		
2.5.5 Braced-tube Systems	60		
2.5.6 Tube-in-tube and Bundled-tube Systems	61		
2.5.7 Diagrid Systems	62		
2.5.8 Other Systems	62		
2.5.9 Transfer Girders	62		
2.6 Structural Integrity	64		
2.7 Slip-form and Jump-form Constructions	66		
2.7.1 Slip-form Construction	66		
2.7.2 Jump-form Construction	66		
3. Loads and Load Combinations	70		
3.1 Introduction	70		
3.2 Characteristic Actions (Loads)	70		
3.3 Dead Loads	72		
3.4 Imposed Loads	72		
3.4.1 Consideration of Slab Loads on Beams	73		
3.4.2 Consideration of Wall Loads on Beams	74		
3.5 Impact Loads	75		
3.6 Snow and Ice Loads	75		
3.7 Wind Loads	75		
3.7.1 Vortex Shedding	77		
3.7.2 Dynamic Effects	78		
3.7.3 Wind Effects on Tall Buildings	79		
3.8 Earthquake Loads	80		
3.8.1 Natural Frequencies	82		
3.8.2 Equivalent Static Method	83		
3.8.3 Rules to be Followed for Buildings in Seismic Areas	84		
3.8.4 Devices to Reduce Earthquake Effects	85		
3.9 Other Loads and Effects	86		
3.9.1 Foundation Movements	87		
3.9.2 Thermal and Shrinkage Effects	87		
3.9.3 Soil and Hydrostatic Pressure	88		
3.9.4 Erection and Construction Loads	89		
3.9.5 Flood Loads	89		
3.9.6 Axial Shortening of Columns	89		

3.10	Pattern Loading	90	5.5.6	Slenderness Limits for Rectangular Beams	157
3.11	Load Combinations	90	5.5.7	Guidelines for Choosing Dimensions and Reinforcement of Beams	158
4. Basis of Structural Design		102	5.5.8	Procedure for Proportioning Sections for Given Loads	160
4.1	Introduction	102	5.5.9	Design of Over-reinforced Sections	162
4.2	Steps Involved in Construction	102	5.5.10	Design Using Charts and Design Aids	162
4.3	Roles and Responsibilities of Designers	103	5.6	Doubly Reinforced Rectangular Beams	162
4.4	Design Considerations	105	5.6.1	Behaviour of Doubly Reinforced Beams	164
4.4.1	Safety	105	5.6.2	Analysis of Doubly Reinforced Rectangular Beams	165
4.4.2	Stability	108	5.6.3	Limiting Moment of Resistance and Compression Steel	166
4.4.3	Serviceability	109	5.6.4	Design of Doubly Reinforced Rectangular Beams	166
4.4.4	Economy	109	5.6.5	Design Using Charts and Design Aids	168
4.4.5	Durability	109	5.7	Flanged Beams	168
4.4.6	Aesthetics	116	5.7.1	Effective Width of Flange	169
4.4.7	Environment Friendliness	116	5.7.2	Behaviour of Flanged Beams	170
4.4.8	Functional Requirements	118	5.7.3	Analysis of Flanged Beams	171
4.4.9	Ductility	120	5.7.4	Minimum and Maximum Steel	174
4.5	Analysis and Design	121	5.7.5	Doubly Reinforced Flanged Beams	176
4.5.1	Relative Stiffness	122	5.7.6	Design of Flanged Beams	178
4.5.2	Redistribution of Moments	124	5.7.7	Design of Flanged Beams Using Charts and Design Aids	179
4.6	Codes and Specifications	125	5.7.8	Design of L-beams	180
4.7	Design Philosophies	126	5.8	Minimum Flexural Ductility	180
4.7.1	Working Stress Method	127	5.9	Deep Beams	181
4.7.2	Ultimate Load Design	129	5.10	Wide Shallow Beams	185
4.7.3	Principles of Limit States Design	129	5.11	Hidden Beams	185
4.7.4	Sampling and Acceptance Criteria	131	5.12	Lintel and Plinth Beams	186
4.8	Limit States Method	132	5.13	High-strength Steel and High-strength Concrete	187
4.8.1	Limit States of Strength	133	5.14	Fatigue Behaviour of Beams	188
4.8.2	Serviceability Limit States	135	6. Design for Shear		
4.9	Design by Using Model and Load Tests	136	6.1	Introduction	214
4.10	Strut-and-tie Model	136	6.2	Behaviour of Reinforced Concrete Beams under Shear	215
4.11	Performance-based Design	136	6.2.1	Behaviour of Uncracked Beams	215
5. Flexural Analysis and Design of Beams		142	6.2.2	Behaviour of Beams without Shear Reinforcement	218
5.1	Introduction	142	6.2.3	Types of Shear or Web Reinforcements	220
5.2	Behaviour of Reinforced Concrete Beams in Bending	143	6.2.4	Behaviour of Beams with Shear/Web Reinforcements	227
5.2.1	Uncracked Section	143	6.3	Factors Affecting Shear Strength of Concrete	229
5.2.2	Cracking Moment	145	6.4	Design Shear Strength of Concrete in Beams	231
5.2.3	Cracked Section	145	6.4.1	Maximum Shear Stress	233
5.2.4	Yielding of Tension Reinforcement and Collapse	146	6.4.2	Effects Due to Loading Condition	234
5.3	Analysis of and Design for Flexure	146	6.5	Critical Section for Shear	234
5.3.1	Effective Span	146	6.6	Minimum and Maximum Shear Reinforcement	236
5.4	Analysis of Singly Reinforced Rectangular Sections	146	6.6.1	Maximum Spacing	236
5.4.1	Assumptions Made to Calculate Ultimate Moment of Resistance	146	6.6.2	Upper Limit on Area of Shear Reinforcement	237
5.4.2	Design Bending Moment Capacity of Rectangular Section	149	6.7	Design of Shear Reinforcement	237
5.4.3	Balanced, Under-reinforced, and Over-reinforced Sections	150	6.7.1	The Ritter–Mörsch Truss Model	238
5.4.4	Depth of Neutral Axis	151	6.7.2	Modified Compression Field Theory	240
5.4.5	Resisting Moment Strength for Balanced Sections	152	6.7.3	Design Procedure for Shear Reinforcement	241
5.5	Design of Singly Reinforced Rectangular Sections	153	6.7.4	Transverse Spacing of Stirrups in Wide Beams	242
5.5.1	Minimum Depth for Given M_u	153	6.7.5	Design Aids	242
5.5.2	Limiting Percentage of Steel	153	6.7.6	Anchoring of Shear Stirrups	242
5.5.3	Factors Affecting Ultimate Moment Capacity	154	6.8	Shear Design of Flanged Beams	243
5.5.4	Minimum Tension Reinforcement	154	6.9	Shear Design of Beams with Varying Depth	243
5.5.5	Maximum Flexural Steel	156	6.10	Shear Design of Beams Located in Earthquake Zones	244

6.11	Shear in Beams with High-strength Concrete and High-strength Steel	245	8.5.3	Consideration of Flanged Beams	315
6.12	Shear Design Beams with Web Opening	246	8.5.4	Area of Stirrups for Torsion	315
6.13	Shear Strength of Members with Axial Force	247	8.5.5	Area of Longitudinal Reinforcement for Torsion	316
6.14	Design of Stirrups at Steel Cut-off Points	248	8.5.6	Limiting Crack Width for Combined Shear and Torsion	317
6.15	Shear Friction	248	8.6	Skew Bending Theory	318
7. Design for Effective Bond between Concrete and Steel		262	8.6.1	Interaction Curves for Combined Flexure and Torsion	319
7.1	Introduction	262	8.6.2	Interaction Curves for Combined Shear and Torsion	320
7.2	Local or Flexural Bond Stress	263	8.7	Indian Code Provisions for Design of Longitudinal and Transverse Reinforcements	320
7.3	Anchorage Bond	264	8.7.1	Equivalent Shear and Moment	321
7.4	Bond Behaviour	266	8.7.2	Minimum Longitudinal Reinforcement for Torsion	321
7.4.1	Test Specimens	266	8.7.3	Design of Transverse Reinforcement	321
7.4.2	Factors Affecting Bond Strength	267	8.7.4	Distribution of Torsional Reinforcement	322
7.5	Development Length	269	8.8	Design and Detailing for Torsion as per IS 456 Code	323
7.5.1	Critical Sections for Development of Reinforcement	270	8.9	Graphical Methods	323
7.5.2	Derivation of Analytical Development Length	270	8.10	Other Considerations	324
7.5.3	Codal Provisions for Development Length	271	9. Design of One-way Slabs		
7.5.4	Design Aids	273	9.1	Introduction	334
7.5.5	Recent Research	273	9.2	Types of Slabs	334
7.5.6	Reduction in Development Length due to Excess Reinforcement	274	9.3	Behaviour of One-way Slabs	335
7.5.7	Development Length of Bars in Compression	274	9.4	General Considerations for Design of Slabs	335
7.5.8	Development Length of Bundled Bars	274	9.4.1	Effective Span	335
7.5.9	Development of Welded Deformed Wire Reinforcement in Tension	275	9.4.2	Minimum Thickness	336
7.6	Anchoring Reinforcing Bars	276	9.4.3	Concrete Cover	338
7.6.1	Behaviour of Hooks or Bends in Tension	276	9.4.4	Analysis of Continuous One-way Slabs	338
7.6.2	Headed and Mechanically Anchored Bars in Tension	279	9.4.5	Calculation of Reinforcements	340
7.6.3	Anchoring Bent-up Bars and Shear Reinforcement	281	9.4.6	Detailing of Reinforcement in One-way Slabs	341
7.7	Splicing of Reinforcement	282	9.4.7	Use of High-strength Steel and High-strength Concrete in Slabs	342
7.7.1	Indirect Splices	282	9.5	Design of Slabs for Shear and Size Effect	343
7.7.2	Direct Splices	288	9.6	Design Procedure for One-way Slabs	344
7.8	Curtailement of Reinforcement	293	9.7	Concentrated Load on One-way Slabs and Design of Culverts	345
7.8.1	Curtailement of Tension Reinforcement in Flexural Members	293	9.7.1	Effective Width of Cantilever Slabs	346
7.8.2	Curtailement of Positive Moment Reinforcement	293	9.7.2	Design of Culvert	346
7.8.3	Curtailement of Negative Moment Reinforcement	294	9.8	Ribbed, Hollow Block, or Voids Slabs	347
7.8.4	Curtailement of Bundled Bars	294	9.9	Earthquake Considerations	349
7.8.5	Special Members	295	9.10	Slabs-on-grade	350
7.9	Seismic Considerations	295	10. Design of Two-way Slabs		
8. Design for Torsion		306	10.1	Introduction	362
8.1	Introduction	306	10.2	Behaviour of Two-way Slabs	362
8.2	Equilibrium and Compatibility Torsion	306	10.3	Minimum Thickness of Slabs	364
8.2.1	Torsion in Curved Beams	308	10.4	Wall- and Beam-supported Two-way Slabs	367
8.2.2	Semicircular Beams Supported on Three Columns	309	10.4.1	Analysis of Wall-supported Slabs	367
8.3	Behaviour of Beams in Torsion	309	10.4.2	Beam-supported Two-way Slabs	378
8.3.1	Torsional Analysis	309	10.4.3	Design Procedure for Two-way Slabs	378
8.3.2	Behaviour of Plain Concrete Members	311	10.4.4	Use of Design Aids	379
8.3.3	Behaviour of Beams with Torsional Reinforcement	311	10.4.5	Detailing of Reinforcements	379
8.4	Theoretical Models for Torsion	312	10.5	Design of Non-rectangular Slabs	381
8.5	Plastic Space Truss Model	313	10.5.1	Circular Slabs	382
8.5.1	Design Strength in Torsion	313	10.5.2	Triangular Slabs	384
8.5.2	Cracking Torque	313	10.5.3	Trapezoidal and Polygonal Slabs	385

10.6	Concentrated Loads on Two-way Slabs	386	12.5	Deflection of Two-way Slabs	483
10.7	Openings in Two-way Slabs	387	12.6	Cracking in Reinforced Concrete Members	484
10.8	Yield-line Analysis for Slabs	388	12.6.1	Types of Cracks	485
10.8.1	Upper and Lower Bound Theorems	389	12.6.2	Mechanism of Cracking	486
10.8.2	Characteristics of Yield Lines	389	12.6.3	Limiting the Crack Width	486
10.8.3	Orthotropic Reinforcement and Skewed Yield Lines	390	12.6.4	Crack Control Provisions in Codes	487
10.8.4	Ultimate Load on Slabs	391	12.6.5	Distribution of Tension Reinforcement in Flanges of I-beams	489
10.8.5	Yield-line Analysis by Virtual Work Method	391	12.7	Side Face Reinforcement in Beams	490
10.8.6	Yield-line Analysis by Equilibrium Method	394	12.8	Frame Deflections	490
10.8.7	Limitations of Yield-line Theory	394	12.9	Vibration Control	491
10.9	Sloped and Pyramidal Roofs	395	12.10	Fatigue Control	492
10.10	Earthquake Considerations	396	12.11	Slenderness Limits for Beams for Stability	493
10.11	Simplified Design	397	12.12	Deflection of Cantilevers	493
11. Design of Flat Plates and Flat Slabs		415	13. Design of Axially Loaded Short Columns		506
11.1	Introduction	415	13.1	Introduction	506
11.2	Proportioning of Flat Slabs	416	13.2	Classification of Columns	506
11.2.1	Thickness of Slab	416	13.2.1	Based on Cross Section	506
11.2.2	Drop Panel	416	13.2.2	Based on Type of Reinforcement	507
11.2.3	Column Heads	416	13.2.3	Based on Type of Loading	507
11.2.4	Shear Caps	417	13.2.4	Based on Slenderness Ratio	507
11.3	Behaviour of Flat Slabs	417	13.3	Unsupported and Effective Lengths of Columns	508
11.4	Methods of Analysis	419	13.3.1	Unsupported Length	508
11.4.1	Direct Design Method	420	13.3.2	Buckling of Columns and Effective Length	509
11.4.2	Equivalent Frame Method	424	13.4	Determination of Effective Length of Columns in Frames	511
11.4.3	Transfer of Moments to Columns	425	13.4.1	Sway and Non-sway Frames	513
11.5	Shear in Flat Plates and Flat Slabs	427	13.4.2	Fixity at Footing	514
11.5.1	One-way or Beam Shear	427	13.5	Behaviour of Short Columns	515
11.5.2	Two-way or Punching Shear	428	13.5.1	Confining Reinforcement for Circular Columns	516
11.5.3	Reinforcement for Punching Shear	435	13.5.2	Confining Reinforcement for Rectangular and Square Columns	517
11.6	Design Procedure for Flat Slabs and Plates	441	13.6	Practical Provisions on Reinforcement Detailing	517
11.7	Detailing of Reinforcements	441	13.6.1	Dimensions	518
11.7.1	Detailing at Edge Columns	443	13.6.2	Concrete Cover	518
11.7.2	Structural Integrity Reinforcement	444	13.6.3	Longitudinal Reinforcement	518
11.8	Openings in Flat Slabs	445	13.6.4	Transverse Reinforcement	519
11.9	Earthquake Effects	446	13.6.5	Columns in Multi-storey Frames	521
11.9.1	Diaphragm Action	447	13.7	Other Codal Provisions	522
11.9.2	Effective Slab Width for Earthquake Loads	447	13.7.1	Slenderness Limit	522
11.9.3	Deformation Capacity of Slab–Column Connections	448	13.7.2	Minimum Eccentricity	522
11.10	Waffle Slabs	449	13.8	Design of Short Columns Under Axial Compression	523
11.11	Grid Slabs	450	13.8.1	Design of Rectangular Columns with Axial Loading	523
11.12	Hollow-core Slabs	450	13.8.2	Design of Circular or Square Columns with Spiral Reinforcement	524
12. Serviceability Limit States: Deflection and Crack Control		467	13.8.3	Steps in the Design of Short Columns	525
12.1	Introduction	467	13.8.4	Design Aids	525
12.2	Design for Limit State of Deflection	468	13.9	Design of Pedestals	525
12.2.1	Limiting Deflection	469	13.10	Earthquake and other Considerations	526
12.3	Short-term Deflections	469	13.10.1	Strong Column–Weak Beam Concept	526
12.3.1	Moment of Inertia of Member	470	13.10.2	Detailing of Longitudinal Reinforcement	527
12.3.2	Load–Deflection Behaviour of RC Beam	472	13.10.3	Detailing of Transverse Reinforcement	528
12.3.3	Other Factors that Influence Deflection	476	13.10.4	Special Confining Reinforcement	530
12.3.4	Deflection of Continuous Beams	476			
12.3.5	Design Aids	477			
12.4	Long-term Deflections	477			
12.4.1	Deflection due to Creep	477			
12.4.2	Deflection due to Differential Shrinkage	481			
12.4.3	Deflection due to Temperature	482			

14. Design of Columns with Moments	546		
14.1	Introduction	546	
14.2	Design of Column with Axial Load and Uniaxial Bending	547	
14.2.1	Assumptions Made in Limit States Design for Columns	547	
14.2.2	Derivation of Basic Equations	547	
14.2.3	Sections with Asymmetric Reinforcement and Plastic Centroid	551	
14.2.4	Analysis of Circular Columns	551	
14.2.5	Interaction Curves	552	
14.2.6	Design Aids	553	
14.2.7	Design Procedure	554	
14.2.8	Splicing of Reinforcement	555	
14.2.9	Transverse Reinforcement	556	
14.3	Design of Columns with Axial Load and Biaxial Bending	557	
14.3.1	Methods of Superposition	559	
14.3.2	Methods of Equivalent Uniaxial Eccentricity	559	
14.3.3	Methods Based on Approximations for Shape of Interaction Surface	560	
14.3.4	Design Procedure for Columns with Biaxial Moments	561	
14.3.5	Design Aids	561	
14.3.6	Design of L, T, and + Columns	562	
14.4	Slender Columns	562	
14.4.1	Definition	562	
14.4.2	Behaviour	563	
14.4.3	Design Approaches	565	
14.4.4	Slender Columns Bent about Both Axes	569	
14.4.5	Design Procedure	570	
14.5	Earthquake Considerations	570	
15. Design of Footings and Pile Caps	585		
15.1	Introduction	585	
15.2	Types of Footings	585	
15.3	Soil Pressure under Footings	587	
15.3.1	Soil Pressure under Footings Subjected to Lateral Moments	587	
15.3.2	Safe Bearing Capacity	589	
15.3.3	Settlement of Foundation	589	
15.3.4	Depth of Foundation	590	
15.3.5	Gross and Net Soil Pressures	591	
15.4	Design Considerations	592	
15.5	Structural Design of Individual Footings	594	
15.5.1	Shear Design Considerations	595	
15.5.2	Bending Moment Considerations	596	
15.5.3	Providing Development Length	597	
15.5.4	Transfer of Load at Base of Column	597	
15.5.5	Design of Wall Footings	599	
15.5.6	Design of Square Column Footings	599	
15.5.7	Design of Rectangular Footings	600	
15.5.8	Design of Sloped Footings	601	
15.6	Design of Combined Footings	602	
15.6.1	Two-column Footings	604	
15.6.2	Design of Combined Slab and Beam Footing	606	
15.6.3	Design of Combined Footing with Strap Beam	608	
15.7	Design of Plain Concrete Footings	609	
15.8	Design of Piles	610	
15.8.1	Behaviour of Piles	610	
15.8.2	Static Formula for Pile Capacity	611	
15.8.3	Dynamic Pile Formula	612	
15.8.4	Pile Groups	612	
15.8.5	Structural Design of Piles	613	
15.8.6	Design of Under-reamed Piles	615	
15.9	Design of Pile Caps	617	
15.9.1	Sectional Method of Design of Pile Cap	618	
15.9.2	Strut-and-tie Model for Pile Caps	619	
15.9.3	Detailing of Pile Caps	622	
15.10	Earthquake Considerations	622	
15.10.1	Use of Grade Beams	623	
15.10.2	Failures due to Liquefaction	623	
16. Design of RC Walls and Structural Walls	644		
16.1	Introduction	644	
16.2	Types of Reinforced Concrete Walls	645	
16.3	Load-bearing Walls	645	
16.3.1	Braced and Unbraced Walls	645	
16.3.2	Eccentricities of Vertical Load	646	
16.3.3	Slenderness Ratio and Effective Height of Walls	646	
16.3.4	Empirical Design Method	647	
16.3.5	Design of Walls Subjected to Combined Horizontal and Vertical Loads	647	
16.3.6	Detailing of Concrete Walls	649	
16.4	Design of Retaining Walls	650	
16.4.1	Types of Retaining Walls	650	
16.4.2	Theories of Earth Pressure	652	
16.4.3	Earth Pressure during Earthquakes	656	
16.4.4	Preliminary Proportioning of Retaining Walls	657	
16.4.5	Drainage and Compaction of Backfill	658	
16.4.6	Stability Requirements	659	
16.4.7	Soil Bearing Pressure Requirements	660	
16.4.8	Procedure for Design	660	
16.4.9	Behaviour and Design of Cantilever-retaining Walls	661	
16.4.10	Behaviour and Design of Counterfort-retaining Walls	662	
16.4.11	Bridge Abutments or Basement Walls	664	
16.5	Structural Walls	665	
16.5.1	Types of Structural Walls	665	
16.5.2	Behaviour of Structural Walls	666	
16.5.3	Design of Structural Walls	668	
16.5.4	Detailing of Structural Walls	673	
16.5.5	Procedure for Design of Reinforced Concrete Structural Walls	675	
16.5.6	Coupling Beams	676	
16.5.7	Openings in Structural Walls	680	
16.5.8	Construction Joints	680	
17. Design of Staircases	702		
17.1	Introduction	702	
17.2	Types of Staircases	703	
17.2.1	Structural Classifications	704	
17.2.2	Effective Span	706	
17.3	Loads on Stair Slabs	707	
17.4	Design of Stair Slabs Spanning Transversely	707	
17.5	Design of Stair Slabs Spanning Longitudinally	709	
17.6	Helicoidal Staircases	711	
17.7	Earthquake Considerations	712	

18. Design of Tension Members	726	19.5.10 Concrete Pryout Strength of Anchor in Shear	770
18.1 Introduction	726	19.5.11 Bond Strength of Adhesive Anchor in Tension	770
18.2 Behaviour of Tension Members	726	19.5.12 Required Strength of Anchors	772
18.3 Design Methods for Members in Direct Tension	727	19.5.13 Interaction of Tensile and Shear Forces	773
18.3.1 Minimum Concrete Grade and Cover of Reinforcement for Liquid Retaining Structures	728	19.5.14 Seismic Design Requirements	773
18.3.2 Minimum Reinforcement	728	19.5.15 Influence of Reinforcements to Resist Shear	774
18.3.3 Spacing of Reinforcement	729	19.5.16 Required Edge Distances and Spacing to Prevent Splitting of Concrete	775
18.4 Design Procedure for Direct Tension	729	19.6 Obtuse-angled and Acute-angled Corners	775
18.5 Design of Members Subjected to Tension and Bending	729		
18.5.1 Tension with Small Eccentricity	729	20. Design of Multi-storey Buildings	790
18.5.2 Tension with Large Eccentricity	730	20.1 Introduction	790
18.5.3 Checking for Crack Width	731	20.2 Example Frame	791
18.6 Interaction Curves for Bending and Tension	732	20.3 Detailed Structural Layouts	792
18.7 Design for Bending, Shear, and Tension	733	20.4 Estimation of Loads	794
18.8 Detailing for Tension Members	733	20.4.1 Calculation of Dead Load—STAAD.Pro Load Case 11	794
18.9 Earthquake and Other Considerations	733	20.4.2 Calculation of Live Load—STAAD.Pro Load Case 12	794
18.10 Design of Water Tanks	734	20.4.3 Calculation of Earthquake Load	794
		20.5 Analysis of Structure	795
19. Design of Joints	743	20.5.1 Gravity Load Analysis	796
19.1 Introduction	743	20.5.2 Lateral Load Analysis	796
19.2 Beam-column Joints	744	20.6 Load Combinations	796
19.2.1 Requirements of Beam-column Joints	745	20.7 Reinforced Concrete Design Using STAAD.Pro for Indian Codes	796
19.2.2 Design and Detailing of Joints	745	20.7.1 Design Parameters as per IS 456	796
19.2.3 Corner Joints	746	20.7.2 Design Parameters as per IS 13920	797
19.2.4 T-joints	747	20.7.3 Design Parameters for Building under Consideration	798
19.2.5 Beam-column Joints in Frames	748	20.8 Serviceability Checks	798
19.2.6 Design of Beam-column Joints	748	20.9 Strength Design of Columns	798
19.2.7 Anchorage of Bars at Joints	753	20.10 Strength Design of Beams	799
19.2.8 Constructability Issues	755	20.11 Design of Foundations	801
19.3 Beam-to-beam Joints	756	20.11.1 Design Parameters	801
19.4 Design of Corbels	758	20.11.2 Design Forces	801
19.5 Design of Anchors	761	20.11.3 Summary of Foundation Design	801
19.5.1 Types of Anchors	761	20.12 Design of Slabs	802
19.5.2 Code Provisions for Design	763	20.12.1 Summary of Loading on Slab	802
19.5.3 Steel Strength of Anchor in Tension	764	20.12.2 Design Data	802
19.5.4 Concrete Breakout Strength of Anchor in Tension	764	20.12.3 Summary of Slab Reinforcement	802
19.5.5 Pull-out Strength in Tension	766	20.13 STAAD.Pro Input File	802
19.5.6 Concrete Side-face Blowout Strength in Tension	767		
19.5.7 Failure Modes in Shear Loading	767		
19.5.8 Steel Strength of Anchor in Shear	767		
19.5.9 Concrete Breakout Strength of Anchor in Shear	768		

Appendix A: Properties of Soils 809

Appendix B: Design Using Strut-and-tie Model 813

Appendix C: Analysis and Design Aids 822

Appendix D: Practical Tips and Some Rules of Thumb 839

Appendix E: Conversion Factors 850

Bibliography 852

Index 853

About the Author 857

INTRODUCTION TO REINFORCED CONCRETE

1.1 INTRODUCTION

The word ‘concrete’ comes from the Latin word *concretus* (meaning compact or condensed), the perfect passive participle of *concrecere*, from *con* (together) and *crescere* (to grow). This name was chosen perhaps due to the fact that this material *grows together*, due to the process of *hydration*, from a visco-elastic, moldable liquid into a hard, rigid, solid rock-like substance. The Romans first invented what is today known as *hydraulic cement-based concrete* or simply *concrete*. They built numerous concrete structures, including the 43.3 m diameter concrete dome, the *Pantheon*, in Rome, which is now over 2000 years old but is still in use and remains the world’s largest non-reinforced concrete dome (see case study in Chapter 2 for more details about the Pantheon).

Concrete is used in nearly every type of construction. Traditionally, concrete has been primarily composed of cement, water, and aggregates (aggregates include both coarse and fine aggregates). Although aggregates make up the bulk of the mix, it is the hardened cement paste that binds the aggregates together and contributes to the strength of concrete, with the aggregates serving largely as low-cost fillers (though their strength also is important).

Concrete is not a homogeneous material, and its strength and structural properties may vary greatly depending upon its ingredients and method of manufacture. However, concrete is normally treated in design as a homogeneous material. Steel reinforcements are often included to increase the tensile strength of concrete; such concrete is called *reinforced cement concrete* (RCC) or simply *reinforced concrete* (RC).

As of 2006, about 7.5 billion cubic metres of concrete were produced each year—this equals about one cubic metre per year for every person on the earth (see Table 1.1). The National Ready Mixed Concrete Association (NRMCA) estimates that ready-mixed concrete production in 2005 was

about 349 million cubic metres in the USA alone, which is estimated to have about 6000 ready-mixed concrete plants.

TABLE 1.1 Annual consumption of major structural materials in the world

Material	Unit Weight (kg/m ³)	Million Tonnes	Tonnes/Person
Structural steel	7850	1244	0.18
Cement	1440	3400	0.48
Concrete	2400	~18,000	2.4 (990 litres)
Timber	700	277	0.04
Drinking water ⁺	1000	5132	0.73 (730 litres)

Notes: The estimated world population as of August 2012 is 7.031 billion.

⁺Assumed as two litres/day/person

Concrete technology has advanced considerably since its discovery by the Romans. Now, concrete is truly an engineered material, with a number of ingredients, which include a host of mineral and chemical admixtures. These ingredients should be precisely determined, properly mixed, carefully placed, vibrated (not required in self-compacting concretes), and properly cured so that the desired properties are obtained; they should also be inspected at regular intervals and maintained adequately until their intended life. Even the cement currently being used has undergone a number of changes. A variety of concretes is also being used, some tailored for their intended use and many with improved properties. Few specialized concretes have compressive strength and ductility matching that of steel. Even though this is a book on RC design, it is important for the designers to know about the nature and properties of the materials they are going to specify for the structures designed by them. As concrete technology has grown in parallel with concrete design, it is impossible to describe all the ingredients, their chemistry, the different kinds of concretes, and their properties in this chapter. Hence, only a brief introduction is given about them, and interested readers should consult a book on concrete technology (many references are given at the end) for further details.

1.1.1 Brief History

Many researchers believe that the first use of a truly cementitious binding agent (as opposed to the ordinary lime commonly used in ancient mortars) occurred in southern Italy around second century BC. Volcanic ash (called *pozzuolana*, found near Pozzuoli, by the Bay of Naples) was a key ingredient in the Roman cement used during the days of the Roman empire. Roman concrete bears little resemblance to modern Portland cement concrete. It was never put into a mould or formwork in a plastic state and made to harden, as is being done today. Instead, Roman concrete was constructed in layers by packing mortar by hand in and around stones of various sizes. The Pantheon, constructed in AD 126, is one of the structural marvels of all times (Shaeffer 1992).

During the Middle Ages, the use of concrete declined, although isolated instances of its use have been documented and some examples have survived. Concrete was more extensively used again during the Renaissance (14th–17th centuries) in structures like bridge piers. Pozzolanic materials were added to the lime, as done by the Romans, to increase its hydraulic properties (Reed, et al. 2008).

In the eighteenth century, with the advent of new technical innovations, a greater interest was shown in concrete. In 1756, John Smeaton, a British Engineer, rediscovered hydraulic cement through repeated testing of mortar in both fresh and salt water. Smeaton's work was followed by Joseph Aspdin, a bricklayer and mason in Leeds, England, who, in 1824, patented

the first 'Portland' cement, so named since it resembled the stone quarried on the Isle of Portland off the British coast (Reed, et al. 2008). Aspdin was the first to use high temperatures to heat alumina and silica materials, so that cement was formed. It is interesting to note that cement is still made in this way. I.K. Brunel was the first to use Portland cement in an engineering application in 1828; it was used to fill a breach in the Thames Tunnel. During 1959–67, Portland cement was used in the construction of the London sewer system.

The small rowboats built by Jean-Louis Lambot in the early 1850s are cited as the first successful use of reinforcements in concrete. During 1850–1880, a French builder, Francois Coignet, built several large houses of concrete in England and France (Reed, et al. 2008). Joseph Monier of France, who is considered to be the first builder of RC, built RC reservoirs in 1872. In 1861, Monier published a small book, *Das System Monier*, in which he presented the applications of RC. During 1871–75, William E. Ward built the first landmark building in RC in Port Chester, NY, USA. In 1892, François Hennebique of France patented a system of steel-reinforced beams, slabs, and columns, which was used in the construction of various structures built in England between 1897 and 1919. In Hennebique's system, steel reinforcement was placed correctly in the tension zone of the concrete; this was backed by a theoretical understanding of the tensile and compressive forces, which was developed by Cottançin in France in 1892 (Reed, et al. 2008).

CASE STUDY

The Ingalls Building

The Ingalls Building, built in 1903 in Cincinnati, Ohio, is the world's first RC skyscraper. This 15-storey building was designed by the Cincinnati architectural firm Elzner & Anderson and engineer Henry N. Hooper. Prior to 1902, the tallest RC structure in the world was only six storeys high. Since concrete possesses very low tensile strength, many at that time believed that a concrete tower as tall as the Ingalls Building would collapse under wind loads or even its own weight. When the building was completed and the supports removed, one reporter allegedly stayed awake through the night in order to be the first to report on the building's failure.

Hooper designed a monolithic concrete box of 200 mm walls, with the floors, roof, beams, columns, and stairs all made of concrete. Columns measured 760 mm by 860 mm for the first 10 floors and 300 mm² for the rest. It was completed in eight months, and the finished building measured 15 m by 30 m at its base and was 64 m tall.

Still in use, the building was designated a National Historic Civil Engineering Landmark in 1974 by the American Society of Civil Engineers; in 1975, it was added to the American National Register of Historic Places.



15-storey Ingalls Building in Cincinnati, Ohio
(Source: http://en.wikipedia.org/wiki/Ingalls_Building)

Earnest L. Ransome patented a reinforcing system using twisted rods in 1884; he also built the first RC framed building in Pennsylvania, USA, in 1903. In 1889, the first concrete reinforced bridge was built. The Ingalls building, which is the first concrete skyscraper, was built in 1904 using the Ransome system and is still in use.

By the 1900s, concrete was generally used in conjunction with some form of reinforcement, and steel began to replace wrought iron as the predominant tensile material. A significant advance in the development of RC was the pre-stressing of steel reinforcing, which was developed by Eugène Freyssinet, in the 1920s, but the technique was not widely used until the 1940s. Victoria skyscraper in Montreal, constructed in 1964, with a height of 190 m and utilizing 41 MPa concrete in the columns, paved way for high-strength concretes (HSCs) (Shaeffer 1992).

In 1908, Prof. Mörsch and Bach of the University of Stuttgart conducted a large number of tests to study the behaviour of RC elements. Prof. Mörsch's work can be considered to be the starting point of modern theory of RC design. Thaddeus Hyatt, an American, was probably the first to correctly analyse the stresses in an RC beam and in 1877 published a small book. In 1895, A. Considère of France tested RC beams and columns and in 1897 published the book *Experimental Researches on Reinforced Concrete*. Several early studies of RC members were based on ultimate strength theories, for example, flexure theory of Thullie in 1897 and the parabolic stress distribution theory of Ritter in 1899. However, the straight line (elastic) theory of Coignet and Tedesco, developed in 1900, was accepted universally because of its simplicity. The ultimate strength design was adopted as an alternative to the working stress method only in 1956–57. Ecole des Ponts et Chaussées in France offered the first teaching course in RC design in 1897. The first British code was published in 1906 and the first US code in 1916. The first Indian code was published in 1953 and revised in 1957, 1964, 1978, and 2000.

1.1.2 Advantages and Disadvantages of Concrete

Reinforced concrete has been used in a variety of applications, such as buildings, bridges, roads and pavements, dams, retaining walls, tunnels, arches, domes, shells, tanks, pipes, chimneys, cooling towers, and poles, because of the following advantages:

Moulded to any shape It can be poured and moulded into any shape varying from simple slabs, beams, and columns to complicated shells and domes, by using *formwork*. Thus, it allows the designer to combine the architectural and structural functions. This also gives freedom to the designer to select any size or shape, unlike steel sections where the designer is constrained by the standard manufactured member sizes.

Availability of materials The materials required for concrete (sand, gravel, and water) are often locally available and are relatively inexpensive. Only small amounts of cement (about 14% by weight) and reinforcing steel (about 2–4% by volume) are required for the production of RC, which may have to be shipped from other parts of the country. Moreover, reinforcing steel can be transported to most construction sites more easily than structural steel sections. Hence, RC is the material of choice in remote areas.

Low maintenance Concrete members require less maintenance compared to structural steel or timber members.

Water and fire resistance RC offers great resistance to the actions of fire and water. A concrete member having sufficient cover can have one to three hours of fire resistance rating without any special fire proofing material. It has to be noted that steel and wood need to be fireproofed to obtain similar rating—steel members are often enclosed by concrete for fire resistance. If constructed and cured properly, concrete surfaces could provide better resistance to water than steel sections, which require expensive corrosion-resistant coatings.

Good rigidity RC members are very rigid. Due to the greater stiffness and mass, vibrations are seldom a problem in concrete structures.

Compressive strength Concrete has considerable compressive strength compared to most other materials.

Economical It is economical, especially for footings, basement walls, and slabs.

Low-skilled labour Comparatively lower grade of skilled labour is required for the fabrication, erection, and construction of concrete structures than for steel or wooden structures.

In order to use concrete efficiently, the designer should also know the weakness of the material. The disadvantages of concrete include the following:

Low tensile strength Concrete has a very low tensile strength, which is about one-tenth of its compressive strength and, hence, cracks when subjected to tensile stresses. Reinforcements are, therefore, often provided in the tension zones to carry tensile forces and to limit crack widths. If proper care is not taken in the design and detailing and also during construction, wide cracks may occur, which will subsequently lead to the corrosion of reinforcement bars (which are also termed as *rebars* in the USA) and even failure of structures.

Requires forms and shoring Cast in situ concrete construction involves the following three stages of construction, which are not required in steel or wooden structures: (a) Construction of formwork over which concrete will be

poured—the formwork holds the concrete in place until it hardens sufficiently, (b) removal of these forms, and (c) propping or shoring of new concrete members until they gain sufficient strength to support themselves. Each of these stages involves labour and material and will add to the total cost of the structure. The formwork may be expensive and may be in the range of one-third the total cost of an RC structure. Hence, it is important for the designer to make efforts to reduce the formwork cost, by reusing or reducing formwork.

Relatively low strength Concrete has relatively low strength per unit weight or volume. (The compressive strength of normal concrete is about 5–10% steel, and its unit density is about 31% steel; see Table 1.2.) Hence, larger members may be required compared to structural steel. This aspect may be important for tall buildings or long-span structures.

TABLE 1.2 Physical properties of major structural materials

Item	Mild Steel	Concrete ¹ M20 Grade	Wood
Unit mass, kg/m ³	7850 (100) ³	2400 (31) ³	290–900 (4–11) ³
Maximum stress in MPa			
Compression	250 (100)	20 (8)	5.2–23 ² (2–9)
Tension	250 (100)	3.13 (1.3)	2.5–13.8 (1–5)
Shear	144 (100)	2.8 (1.9)	0.6–2.6 (0.4–1.8)
Young's modulus, MPa	2 × 10 ⁵ (100)	22,360 (11)	4600–18,000 (2–9)
Coefficient of linear thermal expansion, °C × 10 ⁻⁶	12	10–14	4.5
Poisson's ratio	0.3	0.2	0.2

Notes:

¹ Characteristic compressive strength of 150mm cubes at 28 days

² Parallel to grain

³ The values in brackets are relative percentage values as compared to steel.

Time-dependent volume changes Concrete undergoes drying shrinkage and, if restrained, will result in cracking or deflection. Moreover, deflections will tend to increase with time due to creep of the concrete under sustained loads (the deflection may possibly double, especially in cantilevers). It has to be noted that both concrete and steel undergo approximately the same amount of thermal expansion or contraction; see Table 1.2.

Variable properties The properties of concrete may widely vary due to variation in its proportioning, mixing, curing, and curing. Since cast in situ concrete is site-controlled, its quality may not be uniform when compared to materials such as structural steel and laminated wood, which are produced in the factory.

CO₂ emission Cement, commonly composed of calcium silicates, is produced by heating limestone and other

ingredients to about 1480°C by burning fossil fuels, and it accounts for about 5–7 per cent of CO₂ emissions globally. Production of one ton of cement results in the emission of approximately one ton of CO₂. Hence, the designer should specify cements containing cementitious and waste materials such as fly ash and slags, wherever possible. Use of fly ash and other such materials not only reduces CO₂ emissions but also results in economy as well as improvement of properties such as reduction in heat of hydration, enhancement of strength and/or workability, and durability of concrete (Neville 2012; Subramanian 2007; Subramanian 2012).

1.2 CONCRETE-MAKING MATERIALS

As already mentioned, the present-day concrete is made up of cement, coarse and fine aggregates, water, and a host of mineral and chemical admixtures. When mixed with water, the cement becomes adhesive and capable of bonding the aggregates into a hard mass, called *concrete*. These ingredients are briefly discussed in the following sections.

1.2.1 Cement—Portland Cement and Other Cements

The use of naturally occurring limestone will result in natural cement (*hydraulic lime*), whereas carefully controlled computerized mixing of components can be used to make manufactured cements (*Portland cement*). Portland cements are also referred to as *hydraulic cements*, as they not only harden by reacting with water but also form a water-resistant product. The raw materials used for the manufacture of cement consist of limestone, chalk, seashells, shale, clay, slate, silica sand, alumina and iron ore; lime (calcium) and silica constitute about 85 per cent of the mass.

The process of manufacture of cement consists of grinding the raw materials finely, mixing them thoroughly in certain proportions, and then heating them to about 1480°C in huge cylindrical steel rotary kilns 3.7–10m in diameter and 50–150m long and lined with special firebrick. (The rotary kilns are inclined from the horizontal by about 3° and rotate on its longitudinal axis at a slow and constant speed of about 1–4 revolutions/minute.) The heated materials sinter and partially fuse to form nodular shaped and marble- to fist-sized material called *clinker*. (It has to be noted that at a temperature range of 600–900°C, *calcination* takes place, which results in the release of environmentally harmful CO₂). The clinker is cooled (the strength properties of cement are considerably influenced by the cooling rate of clinker) and ground into fine powder after mixing with 3–5 per cent gypsum (*gypsum* is added to regulate the setting time of the concrete) to form Portland cement. (In modern plants, the heated air from the coolers is returned to the kilns, to save fuel and to increase the burning efficiency). It is then loaded into bulk carriers or packaged into bags; in India, typically 50kg bags are used.

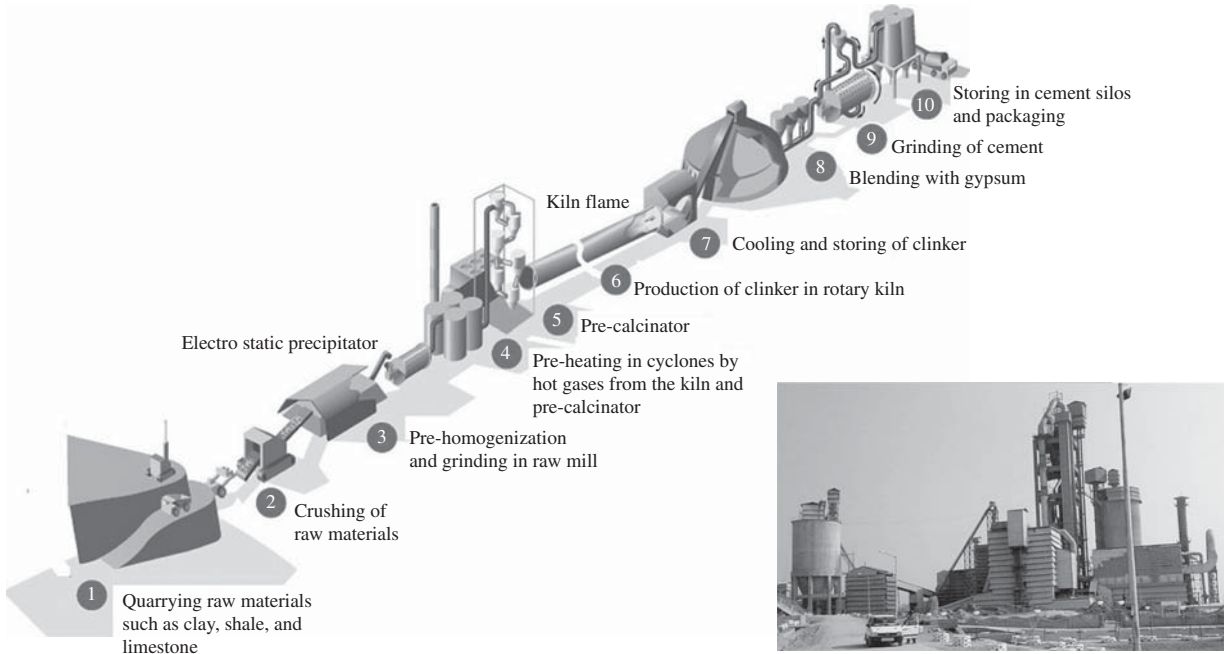


FIG. 1.1 Dry process of cement manufacture (a) Schematic representation (b) View of MCL Cement plant, Thangskai, Meghalaya

Sources: www.cement.org/basics/images/flashtour.html and http://en.wikipedia.org/wiki/File:Cement_Plant_MCL.jpg (adapted)

Two different processes, known as *dry* and *wet*, are used in the manufacture of Portland cement, depending on whether the mixing and grinding of raw materials is done in dry or wet conditions. In addition, a semi-dry process is also sometimes employed in which the raw materials are ground dry, mixed with water, and then burnt in the kilns. Most modern cement factories use either a dry or a semi-dry process. The schematic representation of the dry process of cement manufacture is shown in Fig. 1.1.

Portland Cement

Portland cement (often referred to as *ordinary Portland cement* or *OPC*) is the most common type of cement in general use around the world. The different types of cements covered by the Indian and US standards and their chemical compounds are shown in Table 1.3. Cement production in India consists mainly of the following three types (see Fig. 1.2): OPC ~39 per cent, Portland pozzolana cement (PPC) ~52 per cent, and Portland slag cement (PSC) ~8 per cent. All other varieties put together comprise only 1 per cent of the total production (Mullick 2007).

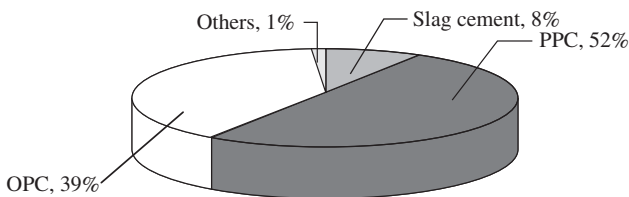


FIG. 1.2 Production trend of different varieties of cement in India

Source: Mullick 2007

TABLE 1.3 Types of Portland cements

India/UK	USA (ASTM)	Typical Compounds ³
OPC (IS 269, IS 8112 and IS 12269)	Type I ¹	C ₃ S 55%, C ₂ S 19%, C ₃ A 10%, C ₄ AF 7%, MgO 2.8%, SO ₃ 2.9%, Ignition loss 1.0%, and free CaO 1.0% (C ₃ A < 15%)
	Type II ¹	C ₃ S 51%, C ₂ S 24%, C ₃ A 6%, C ₄ AF 11%, MgO 2.9%, SO ₃ 2.5%, Ignition loss 1.0%, and free CaO 1.0% (C ₃ A < 8%)
Rapid hardening Portland cement (IS 8041:1990)	Type III ¹	C ₃ S 57%, C ₂ S 19%, C ₃ A 10%, C ₄ AF 7%, MgO 3.0%, SO ₃ 3.1%, Ignition loss 0.9%, and free CaO 1.3% Its seven day compressive strength is almost equal to Types I and II 28 day compressive strengths.
Low heat Portland cement (IS 12600:1989)	Type IV	C ₃ S 28%, C ₂ S 49%, C ₃ A 4%, C ₄ AF 12%, MgO 1.8%, SO ₃ 1.9%, Ignition loss 0.9%, and free CaO 0.8% (C ₃ A < 7% and C ₃ S < 35%)
Sulphate resisting Portland cement (IS 12330:1988)	Type V	C ₃ S 38%, C ₂ S 43%, C ₃ A 4%, C ₄ AF 9%, MgO 1.9%, SO ₃ 1.8%, Ignition loss 0.9%, and free CaO 0.8% [C ₃ A < 5% and (C ₄ AF) + 2(C ₃ A) < 25%]
PSC (IS 455:1989, IS 12089:1987)	Type IS	Made by grinding granulated high-quality slag with Portland cement clinker

(Continued)

TABLE 1.3 (Continued)

India/UK	USA (ASTM)	Typical Compounds ³
PPC [IS 1489-Part 1:1991 (fly ash based), IS 1489-Part 2:1991 (calcined clay based)]	Type IP	A blended cement made by inter-grinding Portland cement and pozzolanic materials without burning
Ternary blended cement	Type IT(SX) (PY) ²	A blended cement made by inter-grinding Portland cement, slag, and pozzolana without burning

Notes:

¹ Types Ia, Iia, and IIIa have the same composition as Types I, II, and III, but have an air-entraining agent ground into the mix.

² The letters X and Y stand for the percentage of supplementary cementitious material (SCM) included in the blended cement, and S and P are the types of SCMs, where S is for slag and P for pozzolan. For example, Type IT(S25) (P15) contains 25 per cent slag and 15 per cent pozzolans.

³ See Table 1.5 for explanation of these compounds.

There are other types, such as high alumina cement (IS 6452:1989), super sulphated cement (IS 6909:1990), hydrophobic Portland cement (IS 8043: 1991), white cement (IS 8042:1989), concrete sleeper grade cement (IRS-T 40:1985), expanding cements, and masonry cement (IS 3466:1988), which are used only in some special situations. (Refer to Mehta and Monteiro (2006) and Shetty (2005) for details regarding these cements.) Geopolymer cements are inorganic hydraulic cements that are based on the polymerization of minerals (see Section 4.4.7 of Chapter 4).

Ordinary Portland cement is the most important cement and is often used, though the current trend is to use PPC (see Fig. 1.2). Most of the discussions to follow in this chapter pertain to this type of cement. The Bureau of Indian Standards (BIS) has classified OPC into the following three grades:

1. 33 grade OPC, IS 269:2013
2. 43 grade OPC, IS 8112:2013
3. 53 grade OPC, IS 12269:2013

TABLE 1.4 Physical properties of various types of cements

S. No.	Type of Cement	IS Code	Fineness m ² /kg (min.)	Setting Time in Minutes		Soundness		Compressive Strength in MPa		
				Initial (min.)	Final (max.)	Le Chatelier (max.) (mm)	Auto Clave, for MgO, (max.) (%)	3 days	7 days	28 days
1.	OPC 33	269:2013	225	30	600	10	0.8	16	22	33
2.	OPC 43	8112:2013	225	30	600	10	0.8	23	33	43
3.	OPC 53	12269:2013	225	30	600	10	0.8	27	37	53
4.	PPC (fly ash-based)	1489:1991 (Part 1)	300	30	600	10	0.8	16	22	33
5.	PSC (slag)	455:1989	225	30	600	10	0.8	16	22	33
6.	SRC	12330:1988	225	30	600	10	0.8	10	16	33

TABLE 1.5 Chemical composition of OPC (Bogue's Compounds)

S. No.	Compound	Cement Chemist Notation (CCN) [*]	Typical Composition as %	Mineral Phase
1.	Tricalcium silicate 3(CaO)·SiO ₂	C ₃ S	45–65	Alite
2.	Dicalcium silicate 2(CaO)·SiO ₂	C ₂ S	15–30	Belite
3.	Tricalcium aluminate 3(CaO)·Al ₂ O ₃	C ₃ A	5–10	Aluminate
4.	Tetracalcium aluminato ferrite 4(CaO)·Al ₂ O ₃ ·Fe ₂ O ₃	C ₄ AF	5–15	Ferrite
5.	Gypsum CaSO ₄ ·2 H ₂ O		2–10	

*Cement chemists use the following shorthand notation:

C = CaO, S = SiO₂, A = Al₂O₃, F = Fe₂O₃, M = MgO,

H = H₂O, N = Na₂O, K = K₂O, \bar{S} = SO₃.

The number in the grade indicates the compressive strength of the cement in N/mm² at 28 days. The 33 grade cement is suitable for producing concrete up to M25. Both 43 grade and 53 grade cement are suitable for producing higher grades of concrete. The important physical properties of the three grades of OPC and other types of cements are compared in Table 1.4. The chemical composition of OPC is given in Table 1.5 and Fig. 1.3.

Approximately 95 per cent of cement particles are smaller than 45 micrometres, with the average particle being around 15 micrometres. The overall particle size distribution of cement is called *fineness*. Fineness affects the heat released and the rate of hydration; greater fineness causes greater early strength (especially during the first seven days) and more rapid generation of heat. *Soundness* refers to the ability of the cement paste to retain its volume after setting and is related to the presence of excessive amounts of free lime or

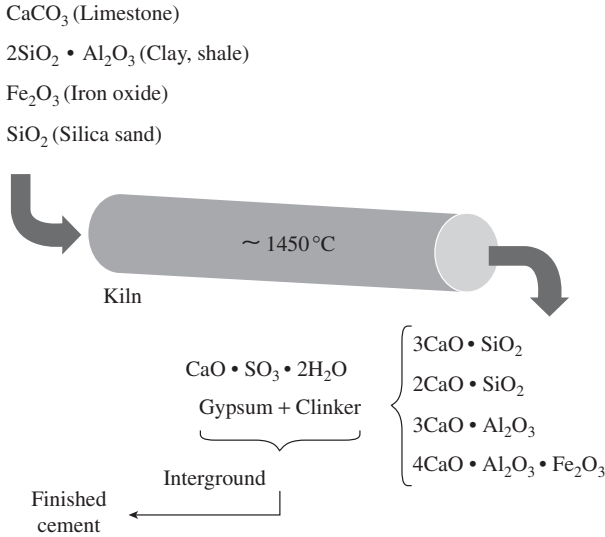


FIG. 1.3 Chemical compounds of cement

magnesia in the cement. *Consistency* indicates the degree of density or stiffness of cement. *Initial setting* of cement is that stage when the paste starts to lose its plasticity. *Final setting* is the stage when the paste completely loses its plasticity and attains sufficient strength and hardness. The specific gravity of Portland cement is approximately 3.15.

As seen in Table 1.5 and Fig. 1.3, there are four major compounds in cement and these are known as tricalcium silicate (C_3S), dicalcium silicate (C_2S), tricalcium aluminate (C_3A), and tetracalcium aluminoferrite (C_4AF). Their composition varies from cement to cement and plant to plant. (The levels of the four clinker minerals can be estimated using a method of calculation first proposed by Bogue in 1929 or by the X-ray diffraction analysis, which gives the exact measurement.) In addition, to these compounds, there are other minor compounds such as MgO , Na_2O , K_2O , SO_3 , fluorine, chloride, and trace metals, which are present in small quantities (Moir 2003). Of these K_2O and Na_2O are called *alkalis* and are found to react with some aggregates, resulting in *alkali-silica reaction* (ASR), which causes disintegration of concrete at a later date.

The silicates C_3S and C_2S are the most important compounds and are mainly responsible for the strength of the cement paste. They constitute the bulk of the composition. C_3A and C_4AF do not contribute much to the strength, but in the manufacturing process they facilitate the combination of lime and silica and act as a flux. The role of the different compounds on different properties of cement is shown in Table 1.6.

Portland Pozzolana Cement

As mentioned already, the Romans and Greeks were aware that the addition of volcanic ash results in better performance of concrete. The name *pozzolan* is now frequently used to describe a range of materials both natural and artificial. [A *pozzolan*

TABLE 1.6 Role of different compounds on properties of cement

Characteristic	Different Compounds in Cement			
	C_3S	C_2S	C_3A	C_4AF
Setting	Quick	Slow	Rapid	–
Hydration	Rapid	Slow	Rapid	–
Heat liberation (Cal/g) 7 days	Higher	Lower	Higher	Higher
Early strength	High up to 14 days	Low up to 14 days	Not much beyond 1 day	Insignificant
Later strength	Moderate at later stage	High at later stage after 14 days	–	–

may be defined as a siliceous or siliceous and aluminous material, which in itself possesses little or no cementitious value. However, in finely divided form and in the presence of water, it reacts chemically with calcium hydroxide released by the hydration of Portland cement, at ordinary temperature, to form calcium silicate hydrate and other cementitious compounds possessing cementitious properties (Mehta 1987)]. Fly ash, ground granulated blast furnace slag (GGBS), silica fume, and natural pozzolans, such as calcined shale, calcined clay or metakaolin, are used in conjunction with Portland cement to improve the properties of the hardened concrete. The latest amendment (No. 3) to IS 1489 requires that PPC be manufactured by the inter-grinding of OPC clinker with 15–35 per cent of pozzolanic material. The generally used pozzolanic materials in India are fly ash (IS 1489-Part 1) or calcined clay (IS 1489-Part 2). Mixtures using three cementitious materials, called *ternary mixtures*, are becoming common, but no Indian specification regarding this has been developed yet. UltraTech PPC, Suraksha, Jaypee Cement (PPC) are some of the brand names of PPC in India. As of now, in India, PPC is considered equivalent to 33 grade OPC.

PPC offers the following advantages:

1. Economical than OPC as the costly clinker is replaced by cheaper pozzolanic material
2. Converts soluble calcium hydroxide into insoluble cementitious products, thus improving permeability and durability
3. Consumes calcium hydroxide and does not produce as much calcium hydroxide as OPC
4. Improves pore size distribution and reduces micro-cracks at the transition zone due to the presence of finer particles than OPC
5. Reduces heat of hydration and thermal cracking
6. Has high degree of cohesion and workability in concrete and mortar

The main disadvantage is that the rate of development of strength is initially slightly slower than OPC. In addition, its

effect of reducing the alkalinity may reduce the resistance to corrosion of steel reinforcement. However, as PPC significantly lowers the permeability, the risk of corrosion is reduced. The setting time is slightly longer.

Portland Slag Cement

Blast furnace slag is a non-metallic product consisting essentially of silicates and aluminosilicates of calcium developed in a molten condition simultaneously with iron in a blast furnace. *GGBS* is obtained by rapidly cooling the molten slag, which is at a temperature of about 1500°C, by quenching in water or air to form a glassy sand-like granulated material. Every year about nine million tons of blast furnace slag is produced in India. The *GGBS* should conform to IS 12089:1987. *PSC* is obtained either by intimate inter-grinding of a mixture of Portland cement clinker and granulated slag with the addition of gypsum or calcium sulphate or by an intimate and uniform blending of Portland cement and finely ground granulated slag. Amendment No. 4 of IS 455 requires that the slag constituent not be less than 25 per cent or more than 70 per cent of the *PSC*. It has to be noted that *PSC* has physical properties similar to those of *OPC*.

The following are some advantages of *PSC*:

1. Utilization of slag cement in concrete not only lessens the burden on landfills; it also conserves a virgin manufactured product (*OPC*) and decreases the embodied energy required to produce the cementitious materials in concrete. Embodied energy can be reduced by 390–886 million Joules with 50 per cent slag cement substitution. This is a 30–48 per cent reduction in the embodied energy per cubic metre of concrete (<http://www.slagcement.org>).
2. By using a 50 per cent slag cement substitution less CO₂ is emitted (amounting to about 98 to 222 kg per cubic metre of concrete, a 42–46% reduction in greenhouse gas emissions) (<http://www.slagcement.org>).
3. Using slag cement to replace a portion of Portland cement in a concrete mixture is a useful method to make concrete better and more consistent. *PSC* has a lighter colour, better concrete workability, easier finishability, higher compressive and flexural strength, lower permeability, improved resistance to aggressive chemicals, and more plastic and hardened consistency.
4. The lighter colour of slag cement concrete also helps reduce the heat island effect in large metropolitan areas.
5. It has low heat of hydration and is relatively better resistant to soils and water containing excessive amounts of sulphates and is hence used for marine works, retaining walls, and foundations.

Both *PPC* and *PSC* will give more strength than *OPC* at the end of 12 months. UltraTech Premium, Super Steel (Madras Cement), and S 53 (L&T) are some of the brand names of *PSC* available in India.

Storage of Cement

Cement is very finely ground and readily absorbs moisture; hence, care should be taken to ensure that the cement bags are not in contact with moisture. They should be stored in airtight and watertight sheds and used in such a way that the bags that come in first are the first to go out. Cement stored for a long time tends to lose its strength (loss of strength ranges from 5–10% in three months to 30–40% in one year). It is better to use the cement within 90 days of its production. In case it is used at a later date, it should be tested before use.

Tests on Cement

The usual tests carried out for cement are for chemical and physical requirements. They are given in IS 4031 (different parts) and IS 4032. Most of these tests are conducted at a laboratory (Neville 2012).

Fineness is measured by the *Blaine air permeability test*, which indirectly measures the surface area of the cement particles per unit mass (m²/kg), or by actual sieving (IS 4031-Part 1:1996 and Part 2:1999). Most cement standards have a minimum limit on fineness (in the range 225–500 m²/kg). Soundness of cement is determined by Le-Chatelier and autoclave tests, as per IS 4031-Part 3:1988. *Consistency* is measured by *Vicat apparatus*, as per IS 4031-Part 4:1988. The paste is said to be of standard consistency when the penetration of plunger, attached to the *Vicat apparatus*, is 33–35 mm. The initial and final setting times of cement are measured using the *Vicat apparatus* with different penetrating attachments, as per IS 4031-Part 5:1988. It has to be noted that the setting time decreases with increase in temperature; the setting time of cement can be increased by adding some admixtures. The compressive strength of cement is the most important of all the properties. It is found using a cement–sand mortar (ratio of cement to sand is 1:3) cube of size 70.6 mm, as per IS 4031-Part 6:1988. The compressive strength is taken as the average of strengths of three cubes. The heat of hydration is tested in accordance with IS 4031-Part 9:1988 using vacuum flask methods or by conduction calorimetry.

A web-based computer software called *Virtual Cement and Concrete Testing Laboratory* (eVCCTL) has been developed by scientists at the National Institute of Standards and Technology (NIST), USA, which can be used to explore the properties of cement paste and concrete materials. This software may be found at http://www.nist.gov/el/building_materials/evcctl.cfm.

1.2.2 Aggregates

The fine and coarse aggregates occupy about 60–75 per cent of the concrete volume (70–85% by mass) and hence strongly influence the properties of fresh as well as hardened concrete, its mixture proportions, and the economy. Aggregates used in concrete should comply with the requirement of

IS 383:1970. Aggregates are commonly classified into fine and coarse aggregates. *Fine aggregates* generally consist of natural sand or crushed stone with particle size smaller than about 5 mm (materials passing through 4.75 mm IS sieve). Coarse aggregates consist of one or a combination of gravels or crushed stone with particle size larger than 5 mm (usually between 10 mm and 40 mm). Aggregates can also be classified in two more ways. Depending on the source, they could be either naturally occurring (gravel, pebbles, sand, etc.) or synthetically manufactured (bloomed clay aggregates, sintered fly ash aggregate, etc.). Moreover, depending on the bulk density, aggregates can either be normal weight (1520–1680 kg/m³), lightweight (less than 1220 kg/m³), or heavyweight (more than 2000 kg/m³). The normal weight aggregates—sand, gravel, crushed rock (e.g., granite, basalt, and sand stone), and blast furnace slag—are used to produce normal weight concrete with a density of 2200–2400 kg/m³. Aggregates such as expanded shale, clay, slate, slag, pumice, perlite, vermiculite, and diatomite are used to produce structural lightweight concrete (SLWC) with density ranging from about 1350 kg/m³ to 1850 kg/m³. Heavyweight aggregates consist of hematite, steel, or iron and are used in special applications such as providing radiation shielding and abrasion resistance (ACI 301M:10 2010, ACI Committee E-701 2007).

The factors of aggregates that may directly or indirectly influence the properties of concrete are given in Table 1.7 (Ambuja technical booklets 5:1996, 125:2007). Only normal weight aggregates are discussed here and should confirm to IS 383:1970. The coarse aggregates form the main matrix of the concrete and hence provide strength to the concrete, whereas the fine aggregates form the filler matrix and hence reduce the porosity of concrete. Some properties of aggregates are shown in Table 1.8.

TABLE 1.7 Factors of aggregates that may affect properties of concrete

S. No.	Factors	Influence on Concrete Property
1.	Specific gravity/Porosity	Strength/Absorption of water
2.	Crushing strength	Strength
3.	Chemical stability	Durability
4.	Surface texture	Bond grip
5.	Shape (see Fig. 1.4)	Water demand (strength)

S. No.	Factors	Influence on Concrete Property
6.	Gradation or particle size distribution	Water demand (strength), cohesion, bleeding, and segregation
7.	Maximum size of aggregate	Strength and water demand
8.	Presence of deleterious materials such as dust, clay, silt, or mud	Water demand (strength), cohesion, bond, and durability

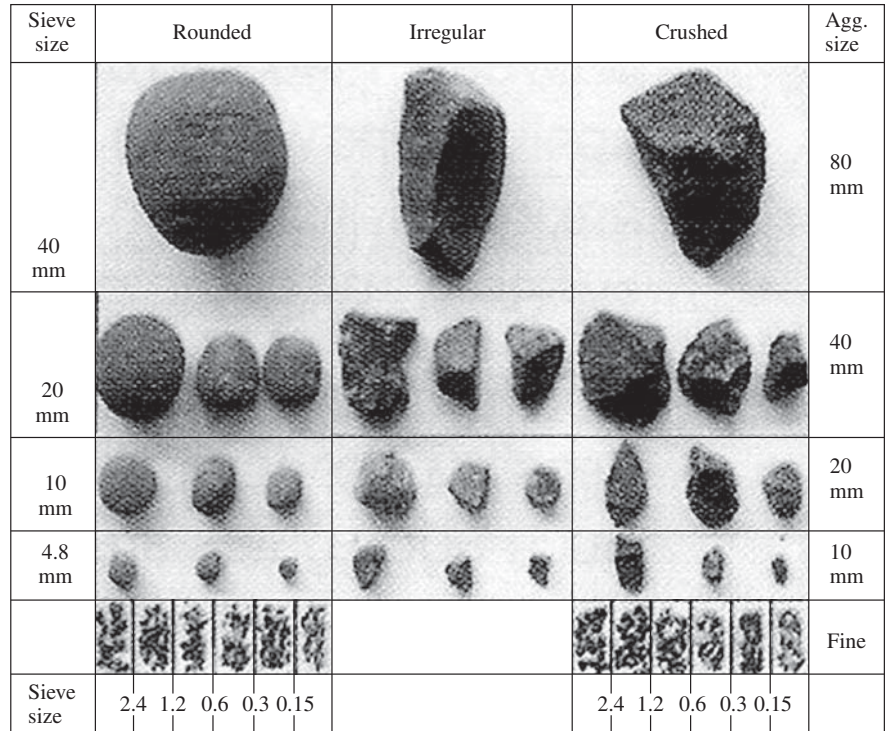


FIG. 1.4 Different shapes and sizes of aggregates

Source: Ambuja technical booklet 125:2007

TABLE 1.8 Properties of aggregates

Aggregate	Property	Aggregate	Property
	Specific Gravity		Minimum Voids (%)
Gravel	2.67	River sand	
Granite	2.80	Fine	43
Sand	2.65	Coarse	35
Basalt	2.85	Mixed and moist	38
Bottom ash	1.57	Mixed and dry	30
	<i>Bulk density (kg/l)</i>		
Broken granite	1.68	Broken stone, graded	
Broken stone	1.60	Maximum size: 25 mm	46
Stone screening	1.45	Maximum size: 50 mm	45

(Continued)

TABLE 1.8 (Continued)

Aggregate	Property	Aggregate	Property
	Specific Gravity		Minimum Voids (%)
Beach or river Shingle	1.60	Maximum size: 63 mm	41
River sand		Stone screening	48
Fine	1.44		Fineness Modulus
Medium	1.52	Sand	2.70
Coarse	1.60	Bottom ash	2.08

In several countries including India, natural coarse aggregates and river sand are scarce; at the same time, the waste from the demolition of buildings is escalating. The amount of construction waste in India alone is estimated to be around 12–14.7 million tons per annum (Rao, et al. 2011). In such places, *recycled coarse aggregates* (RCA) could be used profitably. More details about RCA and their use in concrete may be found in the works of Dhir and Paine (2010), Rao, et al. (2011), and Subramanian (2012). In general, mechanical properties such as compressive strength, split and tensile strengths, and modulus of elasticity are reduced with increasing percentage of RCA. It is suggested that 25 per cent of RCA may be used in concrete, as it will not affect the properties significantly (Rao, et al. 2011). Other substitutes for coarse aggregate include incinerator bottom ash aggregate and sintered fly ash pellets. Recycled glass aggregates, bottom ash from thermal power plants, and quarry dust have significant potential for use as fine aggregates in concrete (Dhir and Paine 2010; Mullick 2012). Clause 5.3.1 of IS 456 stipulates that such aggregates should not contain more than 0.5 per cent of sulphates as SO_3 and should not absorb more than 10 per cent of their own mass of water. Before using these materials, it is better to study their effect on the properties of concrete. For example, manufactured sand, often referred to as *M-sand*, from crushed gravel or rock is cubical in shape and results in increased water demand of the concrete mix.

Aggregates must be clean, hard, strong, and durable; they should be free from coatings of clay, absorbed chemicals, and other fine materials that could affect the hydration and bond of the cement paste. Aggregates are usually washed to remove impurities and graded at the site or plant. Grading or particle size distribution of aggregates is a major factor determining the workability, segregation, bleeding, placing, and finishing characteristics of concrete. The grading of fine aggregates has been found to influence the properties of green (fresh) concrete more than those of coarse aggregates. The grading requirements recommended by the Indian and US standards for fine aggregates is given in Table 1.9. Combined gradation

of fine and coarse aggregate may result in better control of workability, pumpability, shrinkage, and other properties of concrete (Kosmatka, et al. 2011). In general, aggregates that do not have a large deficiency or excess of any size and give a smooth grading curve will produce the most satisfactory results (Kosmatka, et al. 2011). Coarse and fine aggregates should be batched separately.

TABLE 1.9 Grading requirements for fine aggregates

IS Sieve Designation	Percentage Passing by Weight for Grading Zone				ASTM Standard C 33
	I	II	III	IV	
10 mm	100	100	100	100	100
4.75 mm	90–100	90–100	90–100	95–100	95–100
2.36 mm	60–95	75–100	85–100	95–100	80–100
1.18 mm	30–70	55–90	75–100	90–100	50–85
600 μm	15–34	35–59	60–79	80–100	25–60
300 μm	5–20	8–30	12–40	15–50	5–30
150 μm	0–10	0–10	0–10	0–15	0–10

The *fineness modulus* (FM) of either fine or coarse aggregate is calculated by adding the cumulative percentages by mass retained on each of the series of sieves and dividing the sum by 100. The higher the FM, the coarser will be the aggregate. The maximum size of coarse aggregate should not be greater than the following: one-fourth of the maximum size of member, 5 mm less than the maximum clear distance between the main bars, or 5 mm less than the minimum cover of the reinforcement. For RCC works, 20mm aggregates are preferred. In thin concrete members with closely spaced reinforcement or small cover and in HSC, Clause 5.3.3 of IS 456 allows the use of 10 mm nominal maximum size. Rounded aggregates are preferable to angular or flaky aggregates, as they require minimum cement paste for bond and demand less water. Flaky and elongated aggregates are also susceptible to segregation and low strength.

It should be noted that the amount of water added to make concrete must be adjusted for the moisture conditions of the aggregates to accurately meet the water requirement of the mix design. Various testing methods for aggregates to concrete are described in IS 2386-Parts 1 to 8:1963.

1.2.3 Water

Water plays an important role in the workability, strength, and durability of concrete. Too much water reduces the concrete strength, whereas too little will make the concrete unworkable. The water used for mixing and curing should be clean and free from injurious amounts of oils, acids, alkalis, salts, sugars, or organic materials, which may affect the concrete or steel. As per Clause 5.4 of IS 456, potable water is considered satisfactory for mixing as well as curing concrete; otherwise, the water to be used should be tested as per IS 3025-Parts 1 to 32 (1984

to 1988). In general, sea water should not be used for mixing or curing concrete. The permissible limits for impurities as per Clause 5.4 of IS 456 are given in Table 1.10. The pH value of water used for mixing should be greater than six.

TABLE 1.10 Permissible limits for impurities in mixing water

Impurity	Maximum Permissible Limit	
	IS 456 (mg/l)	ASTM C 94 (ppm)
Organic	200	–
Inorganic	3000	–
Sulphates (such as SO ₃)	400	3000
Chlorides (such as Cl)	2000 (for plain concrete work) 500 (for RCC)	1000 ¹
Suspended matter	2000	50,000
Alkalis (such as Na ₂ O + 0.658K ₂ O)	–	600

Note:

¹ Prestressed concrete or concrete in bridge decks 500 ppm (ppm and mg/l are approximately equal)

In general, the amount of water required to be added for cement hydration is less compared to that required for workability. For complete hydration of Portland cement, only about 36–42 per cent water (this is represented by the water/cement or water/cementitious ratio, usually denoted by *w/c ratio* or *w/cm ratio*), that is, *w/c* of 0.36–0.42, is needed. If a *w/c* ratio greater than about 0.36 is used, the excess water, which is not required for cement hydration, will remain in the capillary pores or may evaporate in due course. This process leads to *drying shrinkage* (drying shrinkage is destructive as it leads to micro-cracking and may eventually weaken concrete). Similarly, when a *w/c* ratio of less than about 0.36 is used, some cement will remain unhydrated. The space initially taken up by water in a cementitious mixture will be partially or completely replaced over time by the hydration products. If a *w/c* ratio of more than 0.36 is used, then porosity in the hardened material will remain, even after complete hydration. This is called *capillary porosity* and will lead to corrosion of reinforcement.

1.2.4 Admixtures

It is interesting to note that the Romans were the first to use admixtures in concrete in the form of blood, milk, and lard (pig fat). Present-day admixtures may be classified as chemical and mineral admixtures.

Chemical Admixtures

Chemical admixtures are materials in the form of powder or fluids that are added to the concrete immediately before or

during mixing in order to improve the properties of concrete. They should comply with the requirements of IS 9103:1999. Admixtures are used for several purposes, such as to increase flowability or pumpability of fresh concrete, obtain high strength through lowering of *w/c* ratio, retard or accelerate time of initial setting, increase freeze–thaw resistance, and inhibit corrosion (Krishnamurthy 1997). Normal admixture dosage is about 2–5 per cent by mass of cement. The effectiveness of an admixture depends upon factors such as type, brand, and amount of cementing materials; water content; aggregate shape, gradation, and proportions; mixing time; slump; and temperature of the concrete (Kosmatka, et al. 2011).

The common types of admixtures are as follows (Rixom and Mailvaganan 1999; Aïtcin, et al. 1994; Kosmatka, et al. 2011):

1. *Accelerators* enhance the rate of hydration of the concrete and, hence, result in higher early strength of concrete and early removal of formwork. Typical materials used are calcium chloride, triethanolamine, sodium thiocyanate, calcium formate, calcium nitrite, and calcium thiosulphate. Typical commercial products are Mc-Schnell OC and Mc-Schnell SDS. Typical dosage is 2–3 per cent by weight of cement. As the use of chlorides causes corrosion in steel reinforcing, they are not used now.
2. *Retarders* slow down the initial rate of hydration of cement and are used more frequently than accelerators. They are often combined with other types of admixtures like water reducers. Typical retarders are sugars, hydroxides of zinc and lead, calcium, and tartaric acid. Typical dosage is 0.05 per cent to 0.10 per cent by weight of cement. Commonly used retarders are lignosulphonic acids and hydroxylated carboxylic acids, which act as water-reducing and water-retarding admixtures; they delay the initial setting time by three to four hours when used at normal ambient temperatures.
3. *Water-reducing admixtures* are used to reduce the quantity of mixing water required to produce concrete. Water-reducing admixtures are available as *ordinary water-reducing admixtures* (WRA) and *high-range water-reducing admixtures* (HRWRA). WRA enable up to 15 per cent water reduction, whereas HRWRA enable up to 30 per cent. Popularly, the former are called *plasticizers* and the latter *superplasticizers*. In modern day concreting, the distinction seems to be disappearing. Compounds used in India as superplasticizers include sulphonated naphthalene formaldehyde condensates (SNF), sulphonated melamine formaldehyde condensates (SMF), and modified lignosulphonates (MLS). Some new generation superplasticizers include acrylic polymer based (AP) superplasticizers, copolymers of carboxylic acid with acrylic ether (CAE), polycarboxylate ethers (PCs), and multi-polycarboxylate ethers (MCEs). The naphthalene

and melamine types of superplasticizers or HRWRA are typically used in the range 0.7–2.5 per cent by weight of cement and give water reductions of 16–30 per cent. The PCs are more powerful and are used at 0.3–1.0 per cent by weight of cement to typically give 20 per cent to over 40 per cent water reduction. Use of superplasticizers with reduced water content and w/c ratio can produce concretes with (a) high workability (in fresh concretes), with increased slump, allowing them to be placed more easily, with less consolidating effort, (b) high compressive strengths, (c) increased early strength gain, (d) reduced chloride ion penetration, and (e) high durability. It has to be noted that it is important to consider the compatibility of superplasticizers with certain cements (Jayasree, et al. 2011; Mullick 2008).

4. *Air entraining admixtures* are used to entrain tiny air bubbles in the concrete, which will reduce damage during freeze–thaw cycles, thereby increasing the concrete’s durability. Furthermore, the workability of fresh concrete is improved significantly, and segregation and bleeding are reduced or eliminated. However, entrained air entails a trade off with strength, as each 1 per cent of air may result in 5 per cent decrease in compressive strength. The materials used in such admixtures include salts of wood resins, some synthetic detergents, salts of petroleum acids, fatty and resinous acids and their salts, and salts of sulphonated hydrocarbons.
5. *Corrosion inhibitors* are used to minimize the corrosion of steel and steel bars in concrete.

The other chemical admixtures include foaming agents (to produce lightweight foamed concrete with low density), alkali–aggregate reactivity inhibitors, bonding admixtures (to increase bond strength), colouring admixtures, shrinkage reducers, and pumping aids. It is important to test all chemical admixtures adequately for their desired performance. It is also desirable to prepare trial mixes of concrete with chemical admixtures and test their performance before using them in any large construction activity (see also Clause 5.5 of IS 456). They should not be used in excess of the prescribed dosages, as they may be detrimental to the concrete.

Mineral Admixtures

Mineral admixtures are inorganic materials that also have pozzolanic properties. These very fine-grained materials are added to the concrete mix to improve the properties of concrete (mineral admixtures) or as a replacement for Portland cement (blended cements). Pozzolanic materials react with the calcium hydroxide (lime) released during the hydration process of cement to form an additional C-S-H gel. This can reduce the size of the pores of crystalline hydration products, make the microstructure of concrete more uniform, and improve the impermeability and durability of concrete. These improvements can lead to an increase in strength and service life of concrete. Some of the mineral admixtures are briefly described here:

1. *Fly ash* is a by-product of coal-fired thermal power plants. In India, more than 120 million tons of fly ash is produced every year, the disposal of which poses a serious environmental problem. Any coal-based thermal power station may produce four kinds of ash: fly ash, bottom ash, pond ash, and mound ash. The quality of fly ash to be used in concrete is governed by IS 3812 (Parts 1 and 2):2003, which groups all these types of ash as *pulverized fuel ash* (PFA). PFA is available in two grades: Grade I and grade II (Class F—siliceous fly ash and Class C—calcareous fly ash, as per ASTM). Both these grades can be used as admixtures. Up to 35 per cent replacement of cement by fly ash is permitted by the Indian codes. Fly ash is extracted from flue gases through electrostatic precipitator in dry form. It is a fine material and possesses good pozzolanic properties. The properties of fly ash depend on the type of coal burnt. The lower the loss on ignition (LOI), the better will be the fly ash. The fineness of individual fly ash particles range from 1 micron to 1 mm in size. The specific gravity of fly ash varies over a wide range of 1.9 to 2.55. For a majority of site-mixed concrete, fly ash-based blended cement is the best option. Fly ash particles are generally spherical in shape and reduce the water requirement for a given slump. The use of fly ash will also result in reduced heat of hydration, bleeding, and drying shrinkage.
2. *Ground granulated blast furnace slag* is a by-product of steel production and has been used as a cementitious material since the eighteenth century. It is currently inter-ground with Portland cement to form blended cement, thus partially replacing Portland cement. It reduces the temperature in mass concrete, permeability, and expansion due to alkali–aggregate reaction and improves sulphate resistance. See Section 1.2.1 for more details on PSC.
3. *Silica fume* is also referred to as *micosilica* or *condensed silica fume*. It is a by-product of the production of silicon and ferrosilicon alloys. Silica fume used in concrete should conform to IS 15388:2003; as per Clause 5.2.1.2 of IS 456, its proportion is 5–10 per cent of cement content of a mix. Silica fume is similar to fly ash, with spherical shape, but has an average particle size of about 0.1 micron, that is, it is 100 times smaller than an average cement particle. This results in a higher surface to volume ratio and a much faster pozzolanic reaction. Silica fume addition benefits concrete in two ways: (a) The minute particles physically decrease the void space in the cement matrix—this phenomenon is known as packing. (b) Silica fume is an extremely reactive pozzolan; it increases the compressive strength and improves the durability of concrete. Silica fume for use in concrete is available in wet or dry form. It is usually added during concrete production at a concrete plant. However, it generally requires the use of superplasticizers for workability.
4. *Rice husk ash* (RHA) is produced by burning rice husk in controlled temperature, without causing environmental

pollution. (India produces about 125 million tons of paddy and 30 million tons of rice husk.) It exhibits high pozzolanic characteristics and its use in concrete results in high strength and impermeability. Water demand and drying shrinkage should be studied before using rice husk.

5. *High-reactivity Metakaolin* (HRM) is obtained by calcination of pure or refined *kaolinitic* clay at a temperature between 650 °C and 850 °C followed by grinding to achieve a fineness of 700–900 m²/kg. The strength and durability of concrete produced with the use of HRM is similar to that produced with silica fume. Whereas silica fume is usually dark grey or black in colour, HRM is usually bright white in colour, making it the preferred choice for architectural concrete, where appearance is important.

More details about mineral admixtures may be found in the works of Bapat (2012) and Ramachandran (1995).

1.3 PROPORTIONING OF CONCRETE MIXES

Concrete mix design is the process of proportioning various ingredients such as cement, cementitious materials, aggregates, water, and admixtures optimally in order to produce a concrete at minimal cost and will have specified properties of workability and homogeneity in the green state and strength and durability in the hardened state (SP 23:1982).

Earlier mix design procedures such as minimum voids method, Fuller's maximum density method, Talbot–Richart method, and fineness modulus method are based on the principles of minimum voids and maximum density (Krishna Raju 2002). The modern mix design methods include the Road Note No. 4 method, the ACI (American Concrete Institute) method, the USBR (United States Bureau of Reclamation) method, the Bolomeya model, the British mix design method, and the BIS method (Krishna Raju 2002; Nataraja and Reddy 2007). All these methods are mostly based on empirical relations, charts, graphs, and tables developed through extensive experiments using locally available materials. Although the older BIS code (IS 10262:1982) differed from the ACI method (ACI 211.1, 1991) in some aspects, the present BIS code (IS 10262:2009) is in line with the ACI code method (Nataraja and Das 2010). In all these mix proportioning methods, the ingredients are proportioned by weight per unit volume of concrete.

The main objective of any concrete mix proportioning method is to make a concrete that has the following features:

1. Satisfies workability requirements in terms of slump for easy placing and consolidating
2. Meets the strength requirements as measured by the compressive strength
3. Can be mixed, transported, placed, and compacted as efficiently as possible
4. Will be economical to produce

5. Fulfills durability requirements to resist the environment in which the structure is expected to serve

Changes in Procedure for Mix Proportioning in IS 10262:2009

As per Clause 9.1.1 of IS 456, the minimum grade of concrete to be used in an RCC should not be less than M20. Moreover, all concretes above M20 grade for RCC work must be *design mixes*. Concrete grades above M60 fall under the category of HSC and hence should be proportioned using the guidelines given in specialist literature, such as ACI 211.4-93 and the work of Krishna Raju (2002) and de Larrard (1999).

The 2009 version of the code does not contain the graph of w/c ratio versus 28-day compressive strength. Now, the relationship between w/c ratio and the compressive strength of concrete needs to be established for the materials actually used or by using any other available relationship based on experiments. The maximum w/c ratio given in IS 456:2000 for various environmental conditions may be used as a starting point. The water content per cubic metre of concrete in the earlier version of the standard was a constant value for various nominal maximum sizes of aggregates. However, in the revised version, the maximum water content per cubic metre of concrete is suggested. Another major inclusion in the revised standard is the estimation of volume of coarse aggregate per unit volume of total aggregate for different zones of fine aggregate. As air content in normal (non-air entrained) concrete will not affect the mix proportioning significantly, it is not considered in the revised version; it is also not considered in IS 456:2000.

Data for Mix Proportioning

The following basic data is required for concrete mix proportioning of a particular grade of concrete:

1. Exposure condition of the structure under consideration (see Table 3 of IS 456:2000 and Table 4.4 in Chapter 4 of this book for guidance)
2. Grade designation—The minimum grade of concrete to be designed for the type of exposure condition under consideration (see Tables 3 and 5 of IS 456:2000 and Table 4.4 in Chapter 4 and Table 1.11 of this book for guidance)
3. Type of cement (OPC, PPC, PSC, etc.)

TABLE 1.11 Grades of concrete

Group	Grade Designation	Specified Characteristic 28-day Compressive Strength of 150 mm cube, N/mm ²
Ordinary concrete	M10–M20	10–20
Standard concrete	M25–M60	25–60
High-strength concrete	M65–M100	65–100

4. Minimum and maximum cement content (see Tables 3, 4, 5, and 6 of IS 456:2000 and Tables 4.4 and 4.5 in Chapter 4 of this book for guidance)
5. Type of aggregate (basalt, granite, natural river sand, crushed stone sand, etc.)
6. Maximum nominal size of aggregate to be used (40 mm, 20 mm, or 12.5 mm)
7. Maximum w/c ratio (see Tables 3 and 5 of IS 456:2000 and Tables 4.4 and 4.5 in Chapter 4 of this book for guidance)
8. Desired degree of workability (see Table 1.12, which is based on Clause 7 of IS 456)
9. Use of admixture, its type, and conditions of use
10. Maximum temperature of concrete at the time of placing
11. Method of transporting and placing
12. Early age strength requirements, if required

TABLE 1.12 Workability of concrete

Placing Conditions	Degree of Workability	Slump, mm
Mud mat, shallow section, pavement using pavers	Very low	0.70–0.80 (compacting factor)
Mass concrete; lightly reinforced slabs, beams, walls, columns; strip footings	Low	25–75
Heavily reinforced slabs, beams, walls, columns	Medium	50–100
Slip formwork, pumped concrete	Medium	75–100
In situ piling, trench fill	High	100–150
Tremie concrete	Very high	150–200 (flow test as per IS 9103:1999)

Note: Internal (needle) vibrators are suitable for most of the placing conditions. The diameter of the needle should be determined based on the density and spacing of reinforcements and the thickness of sections. Vibrators are not required for tremie concrete.

The step-by-step mix proportioning procedure as per IS 10262 is as follows (IS 10262:2009; Nagendra 2010):

Step 1 Calculate the target mean compressive strength for mix proportioning. The 28-day target mean compressive strength as per Clause 3.2 of IS 10262 is

$$f'_{ck} = f_{ck} + 1.65 \times s \quad (1.1)$$

where f'_{ck} is the target mean compressive strength at 28 days (N/mm^2), f_{ck} is the characteristic compressive strength at 28 days (N/mm^2), and s is the standard deviation (N/mm^2).

Standard deviation should be calculated for each grade of concrete using at least 30 test strength of samples (taken from

site), when a mix is used for the first time. In case sufficient test results are not available, the values of standard deviation as given in Table 1.13 may be assumed for proportioning the mix in the first instance. As soon as sufficient test results are available, actual standard deviation shall be calculated and used to proportion the mix properly.

TABLE 1.13 Assumed standard deviation

S. No.	Grade of Concrete	Assumed Standard Deviation, N/mm^2
1.	M10	3.5
2.	M15	
3.	M20	4.0
4.	M25	
5.	M30	5.0
6.	M35	
7.	M40	
8.	M45	
9.	M50	
10.	M55	

Note: These values correspond to strict site control of storage of cement, weigh batching of materials, controlled addition of water, and so on. The values given in this table should be increased by 1 N/mm^2 when the aforementioned are not practised.

Step 2 Select the w/c ratio. The concrete made today has more than four basic ingredients. We now use both chemical and mineral admixtures to obtain concretes with improved properties both in fresh and hardened states. Even the qualities of both coarse and fine aggregates in terms of grading, shape, size, and texture have improved due to the improvement in crushing technologies. As all these variables will play a role, concretes produced with the same w/c ratio may have different compressive strengths. Therefore, for a given set of materials, it is preferable to establish the relationship between the compressive strength and free w/c ratio. If such a relationship is not available, maximum w/c ratio for various environmental exposure conditions as given in Table 5 of IS 456 (Table 4.5 in Chapter 4 of this book) may be taken as a starting point. Any w/c ratio assumed based on the previous experience for a particular grade of concrete should be checked against the maximum values permitted from the point of view of durability, and the lesser of the two values should be adopted.

Step 3 Select the water content. The quality of water considered per cubic metre of concrete decides the workability of the mix. The use of water-reducing chemical admixtures in the mix helps to achieve increased workability at lower water contents. The water content given in Table 1.14 (Table 2 of IS 10262) is the maximum value for a particular nominal maximum size of (angular) aggregate, which will achieve a slump in the range of 25 mm to 50 mm. The water content per

TABLE 1.14 Maximum water content per cubic metre of concrete for nominal maximum size of (angular) aggregate

S. No.	Nominal Maximum Size of Aggregate, mm	Maximum Water Content*, kg
1.	10	208
2.	20	186
3.	40	165

Note: These quantities of mixing water are for use in computing cementitious material contents for trial batches.

*Water content corresponding to saturated surface dry aggregate

unit volume of concrete can be reduced when increased size of aggregate or rounded aggregates are used. On the other hand, the water content per unit volume of concrete has to be increased when there is increased temperature, cement content, and fine aggregate content.

In the following cases, a reduction in water content is suggested by IS 10262:

1. For sub-angular aggregates, a reduction of 10 kg
2. For gravel with crushed particles, a reduction of 20 kg
3. For rounded gravel, a reduction of 25 kg

For higher workability (greater than 50 mm slump), the required water content may be established by trial, an increase by about 3 per cent for every additional 25 mm slump, or alternatively by the use of chemical admixtures conforming to IS 9103:1999.

Use of water reducing admixture Depending on the performance of the admixture (conforming to IS 9103:1999) that is proposed to be used in the mix, a reduction in the assumed water content can be made. Water-reducing admixtures will usually decrease water content by 5–10 per cent and superplasticizers decrease water content by 20 per cent and above at appropriate dosages. As mentioned earlier, the use of PC-based superplasticizers results in water reduction up to 30–40 per cent.

Step 4 Calculate the content of cementitious material. The cement and supplementary cementitious material content per unit volume can be calculated from the free w/c ratio of Step 2. The total cementitious content so calculated should be checked against the minimum content for the requirements of durability and the greater of the two values adopted. The maximum cement content alone (excluding mineral admixtures such as fly ash and GGBS) should not exceed 450 kg/m³ as per Clause 8.2.4.2 of IS 456.

Step 5 Estimate the proportion of coarse aggregate. Table 1.15 (Table 3 of IS 12062) gives the volume of coarse aggregate for unit volume of total aggregate for different zones of fine aggregate (as per IS 383:1970) for a w/c ratio of 0.5, which requires to be suitably adjusted for other w/c ratios. This table is based on ACI 211.1:1991. Aggregates of

TABLE 1.15 Volume of coarse aggregate per unit volume of total aggregate for different zones of fine aggregate

Nominal Maximum Size of Aggregate, mm	Volume of Coarse Aggregate* Per Unit Volume of Total Aggregate for Different Zones of Fine Aggregate (for w/c Ratio = 0.5)			
	Zone IV	Zone III	Zone II	Zone I
10	0.50	0.48	0.46	0.44
20	0.66	0.64	0.62	0.60
40	0.75	0.73	0.71	0.69

Note: The volume of coarse aggregate per unit volume of total aggregate needs to be changed at the rate of ± 0.01 for every ± 0.05 change in w/c ratio.

*Volumes are based on aggregate in saturated surface dry condition.

essentially the same nominal maximum size, type, and grading will produce concrete of satisfactory workability when a given volume of coarse aggregate per unit volume of total aggregate is used. It can be seen that for equal workability, the volume of coarse aggregate in a unit volume of concrete is dependent only on its nominal maximum size and the grading zone of fine aggregate.

Step 6 Identify the combination of different sizes of coarse aggregate fractions. Coarse aggregates from stone crushes are normally available in two sizes, namely 20 mm and 12.5 mm. Coarse aggregates of different sizes can be suitably combined to satisfy the gradation requirements (cumulative per cent passing) of Table 2 in IS 383:1970 for the given nominal maximum size of aggregate.

Step 7 Estimate the proportion of fine aggregate. The absolute volume of cementitious material, water, and the chemical admixture is found by dividing their mass by their respective specific gravity, and multiplying by 1/1000. The volume of all aggregates is obtained by subtracting the summation of the volumes of these materials from the unit volume. From this, the total volume of aggregates, the weight of coarse and fine aggregate, is obtained by multiplying their fraction of volumes (already obtained in Step 5) with the respective specific gravities and then multiplying by 1000.

Step 8 Perform trial mixes. The calculated mix proportions should always be checked by means of trial batches. The concrete for trial mixes shall be produced by means of actual materials and production methods. The trial mixes may be made by varying the free w/c ratio by ± 10 per cent of the pre-selected value and a suitable mix selected based on the workability and target compressive strength obtained. Ribbon-type mixers or pan mixers are to be used to simulate the site conditions where automatic batching and pan mixers are used for the production of concrete. After successful laboratory trials, confirmatory field trials are also necessary.

The guidelines for mix proportioning for HSC are provided by ACI 211.4R:93, for concrete with quarry dust by Nataraja,

et al. (2001), and for concrete with internal curing by Bentz, et al. (2005). Rajamane (2004) explains a procedure of mix proportioning using the provisions of IS 456:2000. Optimal mixture proportioning for concrete may also be performed using online tools such as COST (Concrete Optimization Software Tool) developed by NIST, USA (<http://ciks.cbt.nist.gov/cost/>).

1.4 HYDRATION OF CEMENT

When Portland cement is mixed with water, a series of chemical reactions takes place, which results in the formation of new compounds and progressive setting, hardening of the cement paste, and finally in the development of strength. The overall process is referred to as *cement hydration*. Hydration involves many different reactions, often occurring at the same time. When the paste (cement and water) is added to aggregates (coarse and fine), it acts as an adhesive and binds the aggregates together to form concrete. Most of the hydration and about 90 per cent strength development take place within 28 days; however, the hydration and strength development continues, though more slowly, for a long time with adequate moisture and temperature (50% of the heat is liberated between one and three days, 75% in seven days, and about 90% in six months). Hydration products formed in hardened cement pastes are more complicated, and the chemical equations are shown in Table 1.16. More details

of the chemical reactions may be found in the works of Johansen, et al. (2002), Lea (1971), Powers (1961), and Taylor (1997).

As shown in Fig. 1.5, tricalcium silicate (C_3S) hydrates and hardens rapidly and is mainly responsible for the initial set and early strength of concrete. Thus, OPC containing increased percentage of C_3S will have high early strength. On the other hand, dicalcium silicate (C_2S) hydrates and hardens slowly and contributes to strength increase only after about seven days. Tricalcium aluminate (C_3A) is responsible for the large amount of heat of hydration during the first few days of hydration and hardening. It also contributes slightly to the strength development in the first few days. Cements with low percentages of C_3A are more resistant to soils and waters containing sulphates. Tetracalcium aluminoferrite (C_4AF) contributes little to strength. The grey colour of cement is due to C_4AF and its hydrates. As mentioned earlier, gypsum (calcium sulphate dihydrate) is added to cement during final grinding to regulate the setting time of concrete and reacts with C_3A to form *ettringite* (calcium trisulphoaluminate or AFt). In addition to controlling setting and early strength gain, gypsum also helps control drying shrinkage (Kosmatka, et al. 2003). Figure 1.5 shows the relative reactivity of cement compounds. The 'overall curve' has a composition of 55 per cent C_3S , 18 per cent C_2S , 10 per cent C_3A , and 8 per cent C_4AF .

TABLE 1.16 Portland cement compound hydration reactions

Basic Cement Compounds		Hydrated Compounds	
$2(C_3S)$ Tricalcium silicate	+11H Water	= $C_3S_2H_8$ Calcium silicate hydrate (C-S-H)	+3 (CH) Calcium hydroxide
$2(C_2S)$ Dicalcium silicate	+9H Water	= $C_3S_2H_8$ Calcium silicate hydrate (C-S-H)	+CH Calcium hydroxide
C_3A Tricalcium aluminate	+3($C\bar{S}H_2$) Gypsum	+26H Water	= $C_6A\bar{S}_3H_{32}$ Ettringite (AFt)
$2(C_3A)$ Tricalcium aluminate	+ $C_6A\bar{S}_3H_{32}$ Ettringite (AFt)	+4H Water	= $3(C_4A\bar{S}H_{12})$ Calcium mono-sulphoaluminate (AFm)
C_3A Tricalcium aluminate	+CH Calcium hydroxide	+12H Water	= C_4A13H Tetracalcium aluminate hydrate
C_4AF Tetracalcium aluminoferrite	+10H Water	+2(CH) Calcium hydroxide	= $6CAF12H$ Calcium aluminoferrite hydrate

\bar{S} = SO_3 (Sulfur trioxide)

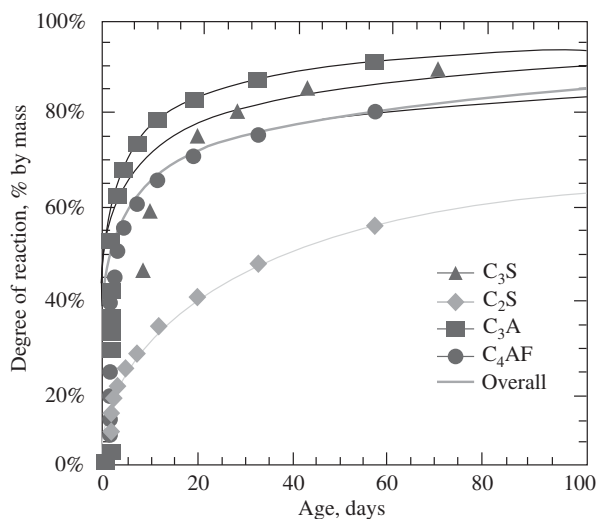


FIG. 1.5 Relative reactivity of cement compounds

Source: Reprinted from Tennis, P.D. and H.M. Jennings 2000, 'A Model for Two Types of Calcium Silicate Hydrate in the Microstructure of Portland Cement Pastes', *Cement and Concrete Research*, Vol. 30, No. 6, pp. 855–63, with permission from Elsevier.

Heat of hydration When Portland cement is mixed with water, heat is liberated as a result of the exothermic chemical reaction. This heat is called the *heat of hydration*. The heat generated by the cement's hydration raises the temperature

of concrete; temperature rises of 55°C have been observed with mixes having high cement content. Such a temperature rise will result in the cracking of the concrete. As a rule of thumb, the maximum temperature differential between the interior and exterior concretes should not exceed 20°C to avoid crack development. ACI 211.1:91 states that as a rough guide, hydration of cement will generate a concrete temperature rise of about 4.7–7.0°C per 50 kg of cement per cubic metre of concrete. Usually, the greatest rate of heat of hydration occurs within the first 24 hours and a large amount of heat evolves within the first three days. Factors influencing heat development in concrete include the cement content (cements with higher contents of tricalcium silicate (C₃S) and tricalcium aluminate (C₃A) and higher fineness have higher rates of heat generation), w/c ratio, placing and curing temperature, the presence of mineral and chemical admixtures, and the dimensions of the structural element. Higher temperatures greatly accelerate the rate of hydration and the rate of heat liberation at early stages (less than seven days). Kulkarni (2012) observed that over the years there is a large increase in the C₃S content and fineness of cement, both of which speed up the hydration reaction and provide high early strength and accompanying side effect of higher heat of hydration (for example, in 1920s, the cement in the USA contained 21% of C₃S and 48% of C₂S; now their proportion is completely reversed and it is 56% of C₃S and 17% of C₂S). In view of these changes in the cement characteristics, design strengths could be achieved with low cement content and higher w/c ratio.

Mineral admixtures (e.g., fly ash), can significantly reduce the rate and amount of heat development. The methods to minimize the rise in concrete temperature include cooling the mixing water, using ice as part of the mixing water, using a moderate-heat Portland cement or moderate- or low-heat blended cement, using chemical admixtures (water-reducer or water-retarder), keeping cement contents to a minimum level, or cooling the aggregate. Moreover, curing with water helps to control temperature increases and is better than other curing methods.

1.5 TYPES OF CONCRETE

Depending on where it is mixed, concrete may be classified as *site-mixed concrete* or *ready-mixed (factory-mixed) concrete (RMC)*. Site mixing is not always recommended as the mixing may not be thorough and the control on the w/c or w/cm ratio cannot be strictly maintained. Hence, it is used only in locations where RMC is not readily available. Concrete without reinforcement is called plain concrete and with reinforcement is called RCC or RC. Even though concrete is strong in compression, it is weak in tension and tends to crack when subjected to tensile forces; reinforcements are

designed to resist these tensile forces and are often provided in the tension zones. Hence, only RCC is used in structures. Depending on the strength it may attain in 28 days, concrete may be designated as ordinary concrete, standard or *normal strength concrete (NSC)*, HSC, and *ultra-high-strength concrete (UHSC)*. In IS 456, the grades of concrete are designed as per Table 1.11. Clause 6.1.1 of IS 456 defines the *characteristic strength* as the strength of the concrete below which not more than five per cent of the test results will fall (refer to Section 4.7.3 and Fig. 4.25 of Chapter 4). The minimum grade for RC as per IS 456 is M20; it should be noted that other international codes specify M25 as the minimum grade. In general, the usual concretes fall in the M20 to M50 range. In normal buildings M20 to M30 concretes are used, whereas in bridges and prestressed concrete construction, strengths in the range of M35 to M50 are common. Very high-strength concretes in the range of M60 to M70 have been used in columns of tall buildings and are normally supplied by ready-mix concrete companies (Kumar and Kaushik 2003).

Concrete with enhanced performance characteristics is called *high-performance concrete (HPC)*. *Self-compacting concrete (SCC)* is a type of HPC, in which maximum compaction is achieved using special admixtures and without using vibrators. Structural engineers should aim to achieve HPC through suitable mix proportioning and the use of chemical and mineral admixtures.

When fibres are used in concrete, it is called *fibres-reinforced concrete (FRC)*. (Fibres are usually used in concrete to control cracking due to plastic shrinkage and drying shrinkage.) High-performance FRCs are called *ductile fibre-reinforced cementitious composites (DFRCCs)*; they are also called *ultra-high-performance concretes (UHPCs)* or *engineered cementitious composites (ECCs)*. Due to the non-availability of standard aggregates or to reduce the self-weight, lightweight aggregates may be used; such concretes are called SLWCs or *autoclaved aerated concretes (AACs)*. A brief description of these concretes is given in the following sections.

1.5.1 Ready-mixed Concrete

Ready-mixed concrete is a type of concrete that is manufactured in a factory or batching plant, based on standardized mix designs, and then delivered to the work site by truck-mounted transit mixers. This type of concrete results in more precise mixtures, with strict quality control, which is difficult to follow on construction sites. Although the concept of RMC was known in the 1930s, this industry expanded only during the 1960s. The first RMC plant started operating in Pune, India, in 1987, but the growth of RMC picked up only after 1997. Most of the RMC plants are located in seven large cities of India,

and they contribute to about 30–60 per cent of total concrete used in these cities. (Even today, a substantial proportion of concrete produced in India is volumetrically batched and site-mixed, involving a large number of unskilled labourers in various operations.) The fraction of RMC to total concrete being used is 28.5 per cent. RMC is being used for bridges, flyovers, and large commercial and residential buildings (Alimchandani 2007).

The RMC plants should be equipped with up-to-date equipment, such as transit mixer, concrete pump, and concrete batching plant. RMC is manufactured under computer-controlled operations and transported and placed at site using sophisticated equipment and methods. The major disadvantage of RMC is that since the materials are batched and mixed at a central plant, travelling time from the plant to the site is critical over longer distances. It is better to have the ready mix placed within 90 minutes of batching at the plant. (The average transit time in Mumbai is four hours during daytime.) Though modern admixtures can modify that time span, the amount and type of admixture added to the mix may affect the properties of concrete.

1.5.2 High-performance Concrete

High-performance concrete may be defined as any concrete that provides enhanced performance characteristics for a given application. It is difficult to provide a unique definition of HPC without considering the performance requirements of the intended use. ACI has adopted the following broad definition of HPC: 'A concrete meeting special combinations of performance and uniformity requirements that cannot always be achieved routinely by using only conventional materials and normal mixing, placing, and curing practices. The requirements may involve enhancements of characteristics such as easy placement, compaction without segregation, long-term mechanical properties, early-age strength, permeability, density, heat of hydration, toughness, volume stability, and long service life in severe environments' (ACI 363 R-10). Table 1.17 lists a few of these characteristics. Concretes possessing many of these characteristics often achieve higher strength (HPCs usually have strengths greater than 50–60 MPa). Therefore, HPCs will often have high strength, but a HSC need not necessarily be called HPC (Mullick 2005; Muthukumar and Subramanian 1999).

The HPCs are made with carefully selected high-quality ingredients and optimized mixture designs (see Table 1.18). These ingredients are to be batched, mixed, placed, compacted, and cured with superior quality control to get the desired characteristics. Typically, such concretes will have a low water–cementitious materials ratio of 0.22 to 0.40.

TABLE 1.17 Desired characteristics of HPCs

Property	Criteria that may be specified
High strength	70–140 MPa at 28–91 days
High early compressive strength	20–28 MPa at 3–12 hours or 1–3 days
High early flexural strength	2–4 MPa at 3–12 hours or 1–3 days
High modulus of elasticity	More than 40 GPa
Abrasion resistance	0–1 mm depth of wear
Low permeability	500–2000 Coulombs
Chloride penetration	Less than 0.07% Cl at 6 months
Sulphate attack	0.10% or 0.5% maximum expansion at 6 months for moderate or severe sulphate exposures
Low absorption	2–5%
Low diffusion coefficient	1000×10^{-14} m/s
Resistance to chemical attack	No deterioration after 1 year
Low shrinkage	Shrinkage strain less than 0.04% in 90 days
Low creep	Less than normal concrete

TABLE 1.18 Typical HPC mixtures used in some structures

Ingredients	Structure				
	Two Union Square, Seattle, 1988	Great Belt Link, East Bridge, Denmark, 1996	Kaiga Atomic Project Unit 2, India, 1998	Petronas Tower, Malaysia, 1999	Urban Viaduct, Mumbai, India, 2002
Water kg/m ³	130	130	136	152	148
Portland cement, kg/m ³	513	315	400	186	500
Fly ash, kg/m ³	–	40	–	345*	–
Silica fume, kg/m ³	43	23	25	35	50
Coarse aggregates, kg/m ³	1080	1140	1069	1000	762 (20 mm) + 384 (10 mm)
Fine aggregates, kg/m ³	685	710	827	725	682
Water reducer, L/m ³	–	1.5	–	–	–
Air content %	–	5.5	2	–	1.5
Superplasticizer, L/m ³	15.7	5.0	5.82	9.29	8.25

(Continued)

TABLE 1.18 (Continued)

Ingredients	Structure				
	Two Union Square, Seattle, 1988	Great Belt Link, East Bridge, Denmark, 1996	Kaiga Atomic Project Unit 2, India, 1998	Petronas Tower, Malaysia, 1999	Urban Viaduct, Mumbai, India, 2002
W/cm ratio	0.25	0.34	0.32	0.25–0.27	0.269
Slump, mm	–	–	175 + 25	180–220	130–180 (at plant) 80–120 (at site)
Strength at 28 days, MPa	119	–	75.9	80	79.6–81.3
Strength at 91 days, MPa	145	–	81.4 (180 days)	100 (56 days)	87.2–87.4

*Mascrete, which is a cement–fly ash compound in the ratio 20:80

Superplasticizers are usually used to make these concretes fluid and workable. It should be noted that without superplasticizers, the w/cm ratio cannot be reduced below a value of about 0.40. Typically, 5–15 L/m³ of superplasticizer can effectively replace 45–75 L/m³ of water (Aïtcin and Neville 1993). This drastic reduction in mixing water reduces the distance between cement particles, resulting in a much denser cement matrix than NSC. The optimal particle-packing mixture design approach may be used to develop a workable and highly durable design mixture (with cement content less than 300 kg/m³), having compressive strength of 70–80 MPa (Kumar and Santhanam 2004).

As the crushing process takes place along any potential zones of weakness within the parent rock, and thus removes them, smaller particles of coarse aggregates are likely to be stronger than the large ones. Hence, for strengths in excess of 100 MPa, the maximum size of aggregates should be limited to 10–12 mm; for lesser strengths, 20 mm aggregates can be used (Aïtcin and Neville 1993; Aïtcin, 1998). Strong and clean crushed aggregates from fine-grained rocks, mostly cubic in shape, with minimal flaky and elongated shapes are suitable for HPC. In order to have good packing of the fine particles in the mixture, as the cement content increases, the fine aggregates should be coarsely graded and have fineness modulus of 2.7–3.0.

As the HPC has very low water content, it is important to effectively cure HPC as early as possible. Membrane curing is not suitable for HPC, and hence fogging or wet curing should be adopted to control plastic and autogenous shrinkage cracking (see Section 1.7).

HPC has been primarily used in tunnels, bridges, pipes carrying sewage, offshore structures, tall buildings, chimneys, and foundations and piles in aggressive environments for its strength, durability, and high modulus of elasticity. It has also been used in shotcrete repair, poles, parking garages, and agricultural

applications. It should be noted that in severe fires, HPC results in bursting of the cement paste and spalling of concrete. More information on HPC may be obtained from ACI 363R-10 and IS 9103:1999 codes and the works of Zia, et al. (1991), Zia, et al. (1993), Aïtcin and Neville (1993), and Aïtcin (1998).

Self-compacting Concrete

Self-compacting concrete, also known as *high-workability concrete*, *self-consolidating concrete*, or *self-levelling concrete*, is a HPC, developed by Prof. Okamura and associates at the University of Tokyo (now Kochi Institute of Technology), Japan, in 1988 (Okamura and Ouchi 2003). SCC is a highly workable concrete that can flow through densely reinforced and complex structural elements under its own weight and adequately fill all voids without segregation, excessive bleeding, excessive air migration, and the need for vibration or other mechanical consolidation. The highly flowable nature of SCC is due to very careful mix proportioning, usually replacing much of the coarse aggregate with fines and cement, and adding chemical admixtures (EFNARC 2005). SCC may be manufactured at a site batching plant or in an RMC plant and delivered to site by a truck mixer. It may then be placed by either pumping or pouring into horizontal or vertical forms. To achieve fluidity, new generation superplasticizers based on polycarboxylic esters (PCE) are used nowadays, as it provides better water reduction and slower slump loss than traditional superplasticizers. The stability of a fluid mix may be achieved either by using high fines content or by using viscosity-modifying agents (VMA).

Several new tests have evolved for testing the suitability of SCC (see Fig. 1.6). They essentially involve testing the (a) flowability (slump flow test), (b) filling ability (slump flow test, V-funnel, and Orimet) (It may be noted that in the slump flow test, the average spread of flattened concrete is measured horizontally, unlike the conventional slump test, where vertical slump is measured.), (c) passing ability (L-box, J-ring, which is a simpler substitute for U-box), (d) robustness, and (e) segregation resistance or stability (simple column box test, sieve stability test). The details of these test methods may be found in the works of Okamura and Ouchi (2003) and Hwang, et al. (2006).

The SCC has been used in a number of bridges and precast projects in Japan, Europe, and USA (Ouchi 2003). Recently, SCC has been used in a flyover construction in Mumbai, India (ICJ, August 2009). The various developments in SCC undertaken in India may be found in ICJ (2004, 2009). An amendment (No. 3, August 2007) in the form of Annex J was added to IS 456, which prescribes the following for SCC:

1. Minimum slump flow: 600 mm
2. Amount of fines (< 0.125 mm) in the range of 400–600 kg/m³, which may be achieved by having sand content more than 38 per cent and using mineral admixture to the order of 25–50 per cent by mass of cementitious materials
3. Use of HRWRA and VMA

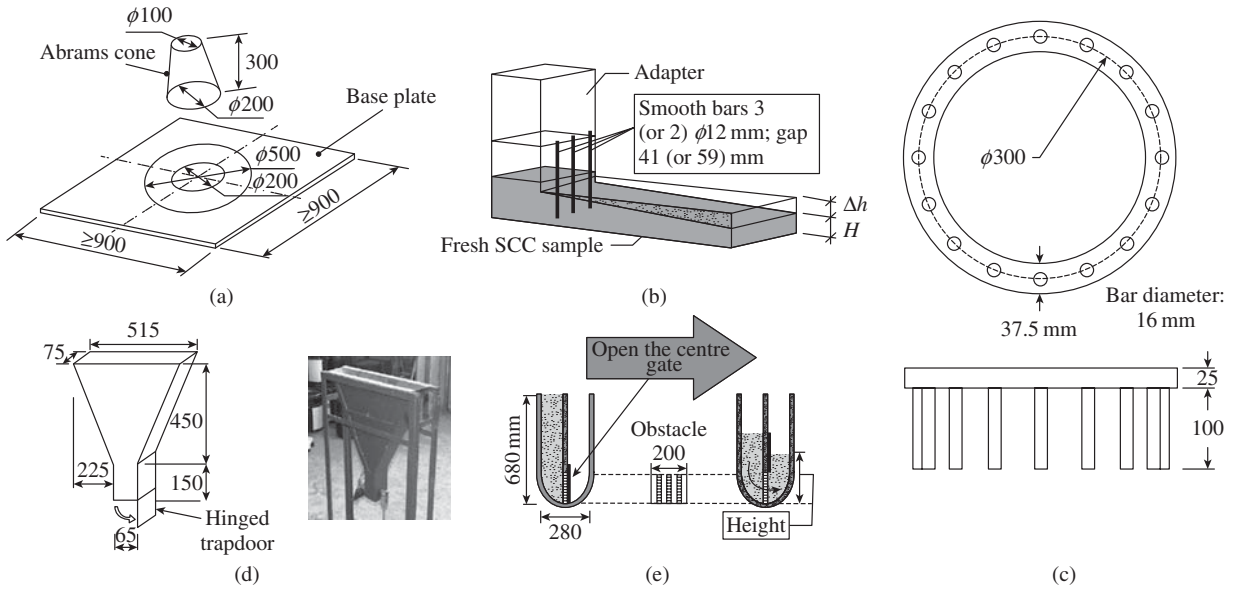


FIG. 1.6 Tests on self-consolidating concrete (a) Slump flow test (b) L-box (c) J-ring (d) V-funnel (e) U-flow test

Source: Okamura and Ouchi 2003, reprinted with permission from JCI.

1.5.3 Structural Lightweight Concrete

Some of the early structures from the Roman Empire that still survive today, including the Pantheon, have elements that were constructed with lightweight concrete. The use of lightweight concrete in modern times started when Steven J. Hayde, a brick-maker from Kansas City, Missouri, developed a rotary kiln method for expanding clays, shales, and slates in the early 1900s. SLWC is made with lightweight coarse aggregates such as natural *pumice* or *scoria* aggregates and expanded slags; sintering-grate expanded shale, clay, or fly ash; and rotary-kiln expanded shale, clay, or slate (ACI E1-07). The in-place density (unit weight) of such SLWC will be of the order of 1360–1850 kg/m³, compared to the density of normal weight concrete of 2240–2400 kg/m³. For structural applications, the strength of such SLWC should be greater than 20 MPa. The use of SLWC allows us to reduce the deadweight of concrete elements, thus resulting in overall economy. In most cases, the slightly higher cost of SLWC is offset by reductions in the weight of concrete used. Seismic performance is also improved because the lateral and horizontal forces acting on a structure during an earthquake are directly proportional to the inertia or mass of a structure. Companies like Lafarge produce varieties of industrial lightweight aggregates; examples include Aglite™, Haydite™, Leca™, Litex™, Lytag™, True Lite™, and Vitrex™ (www.escsi.org). As a result of these advantages, SLWC has been used in a variety of applications in the past 80 years. The reduced strength of SLWC is considered in the design of the ACI code by the factor λ .

An effective technique developed to help mitigate and overcome the issues of autogenous shrinkage and self-desiccation is *internal curing*; *autogenous shrinkage* is defined as a concrete volume change occurring without moisture transfer to the environment, as a result of the internal chemical and structural reactions (Holt 2001). Autogenous shrinkage is accompanied by self-desiccation during hardening of the concrete, which is characterized by internal drying. *Self-desiccation*, or internal drying, is a phenomenon caused by the chemical reaction of cement with water (Persson and Fagerlund 2002). The reaction leads to a net reduction in the total volume of water and solid (Persson, et al. 2005). The porosity of lightweight aggregates provides a source of water for internal curing, resulting in continued enhancement of the strength and durability of concrete. However, this does not prevent the need for external curing. More details about the mix design, production techniques, properties, and so on may be found in the ACI 213R-03 manual and the works of Neville (1996), Clarke (1993), Bentz, et al. 2005, and Chandra and Berntsson (2002).

Autoclaved Aerated Concrete

Autoclaved aerated concrete, also known as *autoclaved cellular concrete* (ACC) or *autoclaved lightweight concrete* (ALC) with commercial names Siporex, e-crete, and Ytong, was invented in the mid-1920s by the Swedish architect Johan Axel Eriksson. It is a lightweight, strong, inorganic, and non-toxic precast building material that simultaneously provides strength, insulation, and fire, mould, and termite resistance. Though relatively unknown in countries such as the USA,

India, Australia, and China, AAC now accounts for over 40 per cent of all construction in the UK and more than 60 per cent of construction in Germany.

Autoclaved aerated concrete is a precast product manufactured by combining silica (either in the form of quartz/silica sand or recycled fly ash), cement, lime, water, and an expansion agent—aluminium powder—at the rate of 0.05–0.08 per cent (it has to be noted that no coarse aggregates are used). Aluminium powder reacts with calcium hydroxide and water to form numerous hydrogen bubbles, resulting in the expansion of concrete to roughly two to five times its original volume. The hydrogen subsequently evaporates, leaving a highly closed-cell aerated concrete.

When the forms are removed from the material, it is solid but still soft. It is then cut into either blocks or panels and placed in an autoclave chamber for 12 hours. AAC blocks (typically 600 mm long, 200 mm high, and 150–300 mm thick) are stacked one over the other using thin-set mortar, as opposed to the traditional concrete masonry units (CMU) construction.

1.5.4 Fibre-reinforced Concrete

Fibres are added to concrete to control cracking caused by plastic shrinkage and drying shrinkage. The addition of small closely spaced and uniformly dispersed fibres will act as crack arresters and enhance the tensile, fatigue, impact, and abrasion resistance of concrete. They also reduce the permeability of concrete. Though the flexural strength may increase marginally, fibres cannot totally replace flexural steel reinforcement (the concept of using fibres as reinforcement is not new; fibres have been used as reinforcement since ancient times, for example, horsehair in mortar and asbestos fibres in concrete).

Clause 5.7 (Amendment No. 3) of IS 456:2000 permits the use of fibres in concrete for special applications to enhance its properties. Steel, glass, polypropylene, carbon, and basalt fibres have been used successfully; steel fibres are the most common (see Fig. 1.7). Steel fibres may be crimped, hooked, or flat. This type of concrete is known as FRC.

The amount of fibres added to a concrete mix is expressed as a percentage of the total volume of the composite (concrete and fibres) and termed *volume fraction*, which is denoted by V_f and typically ranges from 0.25 per cent to 2.5 per cent (of

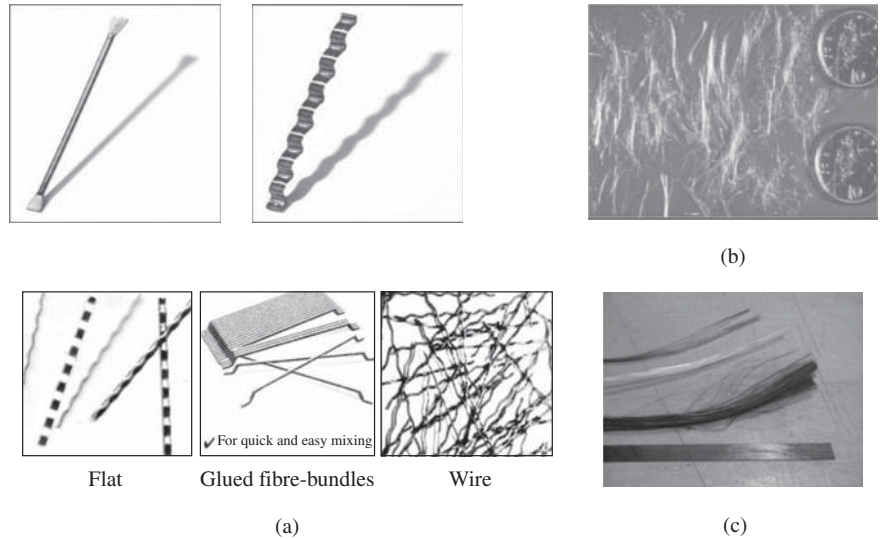


FIG. 1.7 Fibres used in concrete (a) Different types and shapes of steel fibres (b) Fine fibrillated polypropylene fibres (c) Glass fibres

Courtesy: Dr V.S. Parameswaran

which 0.75–1.0 is the most common fraction). The aspect ratio of a fibre is the ratio of its length to its diameter. Typical aspect ratio ranges from 30 to 150. The diameter of steel fibres may vary from 0.25 mm to 0.75 mm. Increasing the aspect ratio of the fibre usually increases the flexural strength and toughness of the matrix. However, fibres that are too long tend to ‘ball’ in the mix and create workability problems (Subramanian 1976b). To obtain adequate workability, it is necessary to use superplasticizers. The ultimate tensile strength of steel fibres should exceed 350 MPa. More information on FRC may be had from the works of Parameswaran and Balasubramanian (1993) and Bentur and Mindess (2007) and ACI 544.1R-96 report.

1.5.5 Ductile Fibre-reinforced Cementitious Composites

Ductile fibre-reinforced cementitious composite is a broader class of materials that has properties and superior performance characteristics compared to conventional cementitious materials such as concrete and FRC. DFRCCs have unique properties including damage reduction, damage tolerance, energy absorption, crack distribution, deformation compatibility, and delamination resistance (*delamination* is a mode of failure in composite materials—splitting or separating a laminate into layers) (Matsumoto and Mihashi 2003). The various subgroups of DFRCC are shown in Fig. 1.8 and Table 1.19 (Matsumoto and Mihashi 2003). It should be noted that DFRCC encompasses a group of high-performance fibre-reinforced cementitious composites (HPFRCC). UHPC, also known as *ultra-high performance fibre-reinforced concrete* (UHPRFC) or *reactive powder concrete* (RPC), developed in France in the late 1990s, is a new class

of DFRCCs that have ultra-strength and ultra-performance characteristics.

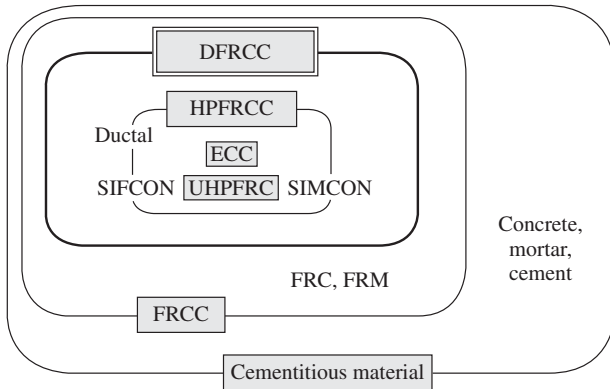


FIG. 1.8 Classification of cementitious materials

Source: Matsumoto and Mihashi 2003, reprinted with permission from JCI

TABLE 1.19 Characteristics of different cementitious materials

Characteristics	Cement, Mortar	Concrete, FRC	DFRCC	HPFRCC
Material response	Brittle	Quasi brittle	Quasi-brittle (tension) or ductile (flexure)	Ductile
Strain softening or hardening (see Fig. 1.9)	–	Strain softening	Strain softening (tension) or hardening (flexure)	Strain hardening
Cracking behaviour (flexure)*	Localized cracking	Localized cracking	Multiple cracking	Multiple cracking
Cracking behaviour (tension)	Localized cracking	Localized cracking	Localized cracking	Multiple cracking

*Cracking behaviour in flexure is dependent on specimen size. This comparison is based on specimen size of 100 × 100 × 400 mm

Source: Matsumoto and Mihashi 2003, reprinted with permission from JCI

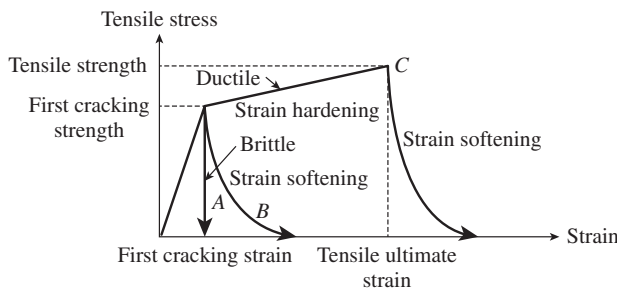


FIG. 1.9 Definitions of brittle, ductile, strain softening, and strain hardening under uniaxial tensile loading

Source: Matsumoto and Mihashi 2003, reprinted with permission from JCI

Engineered Cementitious Composites

Engineered cementitious composites are a special type of HPFRCC that has been micro-structurally tailored based on micro-mechanics. ECC is systematically engineered to achieve high ductility under tensile and shear loading. By employing material design based on micro-mechanics, it can achieve maximum ductility in excess of three per cent under uniaxial tensile loading with only two per cent fibre content by volume. Experiments have shown that even at the ultimate load (5% strain), the crack width remains at about 60µm and is even lower at strain below one per cent.

As shown in Fig. 1.10, extensive experimental studies have demonstrated superior seismic response as well as minimum post-earthquake repair (Fischer and Li 2002). It should be noted that even at high drift level of 10 per cent, no spalling of the reinforced ECC was observed; in contrast, the RCC column lost the concrete cover after bond splitting and being subjected to heavy spalling. The test results also illustrated the potential reduction or elimination of steel stirrups by taking advantage of the shear ductility of ECC. The tensile ductility in ECC also translates into shear ductility since the material undergoes diagonal tensile multiple cracking when subjected to shear (Li, et al. 1994).

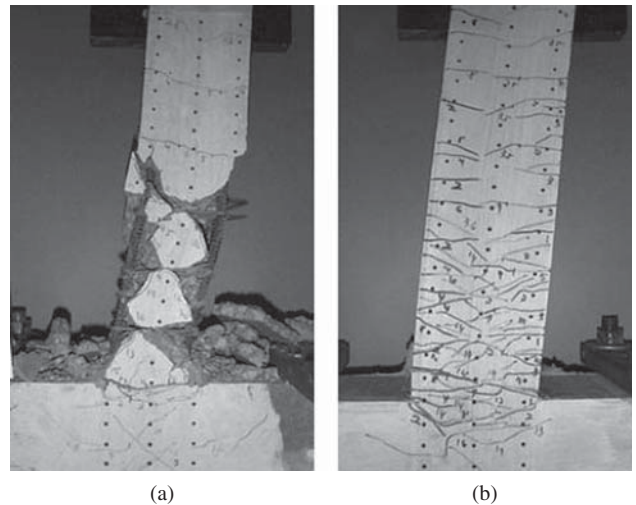


FIG. 1.10 Damage of column at 10% drift after reverse cyclic loading (a) RCC (b) ECC without stirrups

Source: Fischer and Li 2002, reprinted with permission from ACI

Life cycle cost comparison showed that ECC slab systems provide 37 per cent cost efficiency, consume 40 per cent less total primary energy, and produce 39 per cent less carbon dioxide compared to conventional RCC systems (Li 2003). More details about the behaviour and application of ECC may be found in the study of Li (2003).

Ultra-high-performance Concrete

Ultra-high-performance concrete is a high-strength, high-stiffness, self-consolidating, and ductile material, formulated by combining Portland cement, silica fume, quartz flour, fine silica sand, high-range water reducer, water, and steel or organic fibres. Originally it was developed by the Laboratoire Central des Ponts et Chaussées (LCPC), France, containing a mixture of short and long metal fibres and known as *multi-scale fibre-reinforced concrete* (Rossi 2001). It has to be noted that there are no coarse aggregates, and a low w/cm ratio of about 0.2 is used in UHPC compared to about 0.4–0.5 in NSC. The material provides compressive strengths of 120–240 MPa, flexural strengths of 15–50 MPa, and post-cracking tensile strength of 7.0–10.3 MPa and has modulus of elasticity from 45 GPa to 59 GPa [Ductal® (Lafarge, France), CoreTUFF® (US Army Corps of Engineers), BSI®, Densit® (Denmark), and Ceracem® (France and Switzerland) are some examples of commercial products]. The enhanced strength and durability properties of UHPC are mainly due to optimized particle gradation that produces a very tightly packed mix, use of steel fibres, and extremely low water to powder ratio (Nematollahi, et al. 2012).

Some of the potential applications of UHPC are in prestressed girders and precast deck panels in bridges, columns, piles, claddings, overlays, and noise barriers in highways. The 60 m span Sherbrooke pedestrian bridge, constructed in 1997 at Quebec, Canada, is the world's first UHPC bridge without any bar reinforcement. More details of this bridge may be had from the works of Blais and Couture (1999) and Subramanian (1999). The 15 m span Shepherds Creek Road Bridge, New South Wales, Australia, built in 2005 is the world's first UHPC bridge for normal highway traffic. Since then, a number of bridges and other structures have been built utilizing UHPC all over the world (see www.fhwa.dot.gov).

The materials for UHPC are usually supplied by the manufacturers in a three-component premix: powders (Portland cement, silica fume, quartz flour, and fine silica sand) pre-blended in bulk bags; superplasticizers; and organic fibres. Care should be exercised during mixing, placing, and curing. The ductile nature of this material makes concrete to deform and support flexural and tensile loads, even after initial cracking. The use of this material for construction is simplified by the elimination of reinforcing steel and its ability to be virtually self-placing. More details about UHPC may be found in the works of Schmidt, et al. (2004), Fehling, et al. (2008), and Schmidt, et al. (2012). A comparison of stress–strain curves in concretes is provided in Fig. 1.11. The influence of fibres and confinement on the ductility of RPC should be noted.

Slurry Infiltrated Fibrous Concrete and Slurry Infiltrated Mat Concrete

Slurry infiltrated fibrous concrete (SIFCON), invented by Lankard in 1979, is produced by infiltrating cement slurry

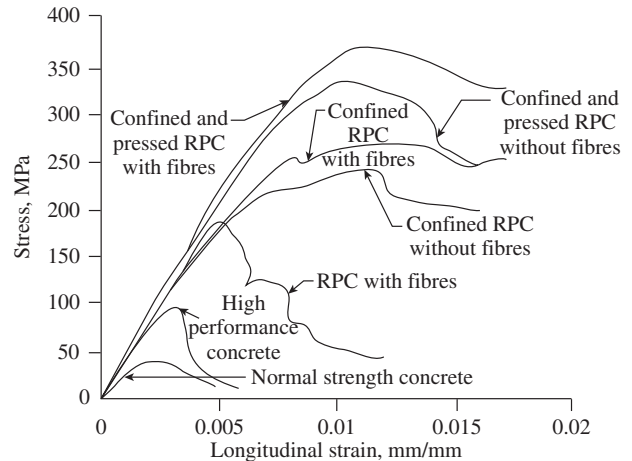


FIG. 1.11 Comparison of stress–strain curves of NSC, HPC, and RPC

Source: Blais and Couture 1999

(made of cement and sand in the proportion 1:1, 1:1.5, or 1:2, with fly ash and silica fume equal to 10–15% by weight of cement, w/cm ratio of 0.3–0.4, and superplasticizer equal to 2–5% by weight of cement) into pre-placed steel fibres (single plain or deformed fibres) in a formwork. It has to be noted that it does not contain any coarse aggregates but has a high cementitious content. Due to the pre-placement of fibres, its fibre volume fraction may be as high as 6–20 per cent. The confining effect of numerous fibres yields high compressive strengths from 90 MPa to 210 MPa, and the strong fibre bridging leads to tensile strain hardening behaviour in some SIFCONs. Slurry infiltrated mat concrete (SIMCON) is similar to SIFCON, but uses pre-placed fibre mat instead of steel fibres. SIFCON and SIMCON are extremely ductile and hence ideally suitable for seismic retrofit of structures (Dogan and Krstulovic-Opara 2003). They also have improved uniaxial tensile strength, flexural, shear, impact strengths, and abrasion resistance (Parameswaran 1996). They are best suited for the following applications: pavement rehabilitation, safety vaults, strong rooms, refractory applications, precast concrete products, bridge decks and overlays, repair and rehabilitation of structures, especially in seismic zones, military applications, and concrete mega-structures, such as offshore platforms and solar towers. More details about SIFCON and SIMCON may be found in the works of Parameswaran, et al. (1990), Parameswaran (1996), Lankard (1984), Naaman, et al. (1992), Sashidhar, et al. (2010, 2011) and Hackman, et al. (1992).

1.5.6 Ferrocement

Ferrocement also known as *ferrocete*, invented by Jean Louis Lambot of France, in 1848, is a composite material like RCC. In RCC, the reinforcement consists of steel bars placed in the tension zone, whereas ferrocement is a thin RC made

of rich cement mortar (cement to sand ratio of 1:3) based matrix reinforced with closely spaced layers of relatively small diameter wire mesh, welded mesh, or chicken mesh. (The diameter of wires range from 4.20 mm to 9.5 mm and are spaced up to 300 mm apart.) The mesh may be metallic or synthetic (Naaman 2000). The mortar matrix should have excellent flow characteristics and high durability. The use of pozzolanic mineral admixtures such as fly ash (50% cement replacement with fly ash is recommended) and use of superplasticizers will not only permit the use of water–binder ratio of 0.40–0.45 by mass but will also enhance the durability of the matrix. A mortar compressive strength of 40–50 MPa is recommended.

During the 1940s, Pier Luigi Nervi, an Italian engineer, architect, and contractor, had used ferrocement for the construction of aircraft hangars, boats and buildings. It has to be noted that though Nervi used a large number of meshes in his structures, in many present-day applications, only two layers of mesh reinforcement are used. Applications of ferrocement include boats, barges, water tanks, pipes, biogas digesters, septic tanks, toilet blocks, and monolithic or prefabricated housing (Subramanian 1976a). Recently, Spanos, et al. (2012) studied the use of ferrocement panels as permanent load bearing formwork for one-way and two-way slabs. Such panels provide economic advantages and the slabs incorporating them will provide superior serviceability performance. At the new Sydney Opera House, the sail-shaped roofs (built of conventional RC) have been covered with tile-surfaced panels of ferrocement, which serve as waterproofs for the concrete underneath. More information about the design and construction of ferrocement may be had from the study of Naaman (2000) and ACI 549.1R-93 manual.

Polymer concrete Polymer concrete is obtained by impregnating ordinary concrete with a monomer material and then polymerizing it by radiation, by heat and catalytic ingredients, or by a combination of these two techniques. Depending on the process by which the polymeric materials are incorporated, they are classified as (a) polymer concrete (PC), (b) polymer impregnated concrete (PIC), and (c) polymer modified concrete (PMC). Due to polymerization, the properties are much enhanced and polymer concrete is also used to repair damaged concrete structural members (Subramanian and Gnana Sambanthan 1979).

In addition to these types of concrete, *prestressed concrete* is often used in bridges and long-span structures; however, it is outside the scope of this book. A prestressed concrete member is one in which internal stresses (compressive in nature) are introduced, which counteract the tensile stresses resulting from the given external service level loads. The prestress is commonly introduced by tensioning the high-strength steel reinforcement (either by using the pre-tensioning or the

post-tensioning method), which applies pre-compression to the member. The design of prestressed concrete members should conform to IS 1343:1980.

1.6 REINFORCING STEEL

As stated earlier, steel reinforcements are provided in RCC to resist tensile stresses. The quality of steel used in RCC work is as important as that of concrete. Steel bars used in concrete should be clean and free from loose mill scales, dust, loose rust and any oily materials, which will reduce bond. Sand blasting or similar treatment may be done to get clean reinforcement.

As per Clause 5.6 of IS 456, steel reinforcement used in concrete may be of the following types (see Table 1.1 of SP 34 (S&T):1987 for the physical and mechanical properties of these different types of bars):

1. Mild steel and medium tensile steel bars (MS bars) conforming to IS 432 (Part 1):1982
2. High-yield strength-deformed steel bars (HYSD bars) conforming to IS 1786:2008
3. Hard drawn steel wire fabric conforming to IS 1566:1982
4. Structural steel conforming to Grade A of IS 2062:2006

It should be noted that different types of rebars, such as plain and deformed bars of various grades, say grade Fe 415 and Fe 500, should not be used side by side, as this may lead to confusion and error at site. Mild steel bars, which are produced by hot rolling, are not generally used in RCC as they have smooth surface and hence their bond strength is less compared to deformed bars (when they are used they should be hooked at their ends). They are used only as ties in columns or stirrups in beams. Mild steel bars have characteristic yield strength ranging from 240 N/mm² (grade I) to 350 N/mm² (medium tensile steel) and percentage elongation of 20–23 per cent over a gauge length of $5.65\sqrt{\text{area}}$.

Hot rolled *high-yield strength-deformed bars* (HYSD bars) were introduced in India in 1967; they completely replaced mild steel bars except in a few situations where acute bending was required in bars greater than 30 mm in diameter. They were produced initially by cold twisting (CTD bars) and later by heat treatment (TMT bars) and micro-alloying. They were introduced in India by Tata Steel as Tistrong bars and later as Tiscon/Torsteel bars. *Cold twisted deformed bars* (CTD bars or *Torsteel bars*) are first made by hot rolling the bars from high-strength mild steel, with two or three parallel straight ribs and other indentations on it. After cooling, these bars are cold twisted by a separate operation, so that the steel is strained beyond the elastic limit and then released. As the increase in strength is due to cold-working, this steel should not be normally welded. In CTD bars, the projections will form a helix around the bars; if they are over-twisted, the pitch of the helixes will be too close. Cold twisting introduces residual stresses in steel,

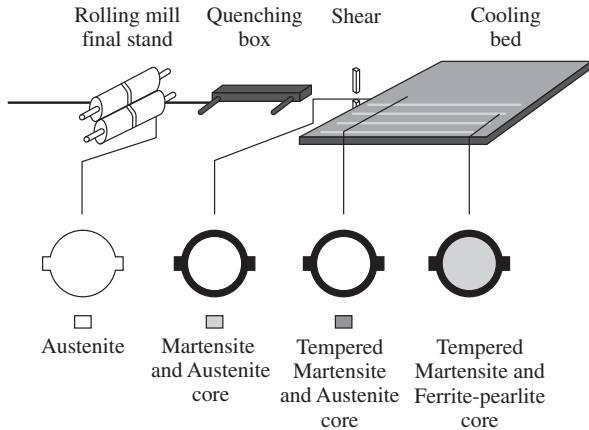


FIG. 1.12 Manufacturing process of TMT bars

Source: Tata steel-TISCON CRS-Corrosion Resistant Steel Bars, Tata Steel Catalog, Tata Steel, Kolkata, August 2004.

and as a result, these bars corrode much faster than other bars; hence, these are not recommended in many advanced countries.

Thermo-mechanically treated reinforcement bars (TMT Bars) are a class of hot rolled HYSD bars, which are rapidly cooled to about 450°C by a controlled quenching process using water when they are leaving the last stand of the rolling mill at a temperature of about 950°C . This sudden partial quenching, along with the final cooling, transforms the surface layer of the bars from austenite to tempered martensite, with a semi-tempered middle ring of martensite and bainite and a fine-grained ferrite-pearlite core (see Fig. 1.12). TMT bars can be welded as per IS 9417 using ordinary electrodes and no extra precautions are required. Strength, weldability, and ductility are the main advantages of TMT bars; in addition, they are also economical. TMT bars produced by SAIL or Tata are known as SAIL-TMT or TISCON-TMT. Bars produced by RINL are called REBARS. As it is visually difficult to distinguish TMT bars from mild steel deformed bars, the following procedure is suggested in IS 1786: A small piece

(about 12 mm long) can be cut and the transverse face lightly ground flat on progressively finer emery papers up to '0' size. The sample can be macro-etched with nital (five % nitric acid in alcohol) at ambient temperature for a few seconds to reveal a darker annular region corresponding to martensite or bainite microstructure and a lighter core region.

By micro-alloying with elements such as copper, phosphorus, and chromium, *thermo-mechanically treated corrosion resistant steel bars* (TMT CRS bars) are produced, which have better corrosion resistance than ordinary TMT bars.

It is better to adopt precautions against corrosion even while using such bars, as they are not 100% corrosion-resistant. Though IS 1786 specifies four grades for these HYSD bars, namely Fe 415, Fe 500, Fe 550, and Fe 600, and additional three grades with a suffix D, denoting that they are ductile, the availability of Fe 550, Fe 600, Fe 415D, Fe 500D, and Fe 550D grades are limited (the numbers after Fe denoting the 0.2% proof or yield stress, in N/mm^2).

The most important characteristic of the reinforcing bar is its stress-strain curve; the important property is its characteristic yield strength or 0.2 per cent proof stress as the case may be (see Fig. 1.13 and Table 1.20), and as per Clause 5.6.3 of IS 456, the modulus of elasticity E_s for these steels may be taken as 200 kN/mm^2 . (For HYSD bars the yield point is not easily defined based on the shape of the stress-strain curve; hence an offset yield point is arbitrarily defined at 0.2% of the strain. Thus by drawing a line parallel to the elastic portion of the stress-strain curve from the 0.2% strain, the yield point stress is located on the stress-strain curve as shown in Fig. 1.13b.) The design stress-strain curves for steel reinforcements (both mild steel and HYSD bars) are given in Fig. 5.5 of Chapter 5. The inelastic strains in HYSD bars for some design stress values, as per IS 456, are given in Table 5.1 (see Section 5.4). The chemical composition of various grades of steel is given in IS 1786:2008 specifications.

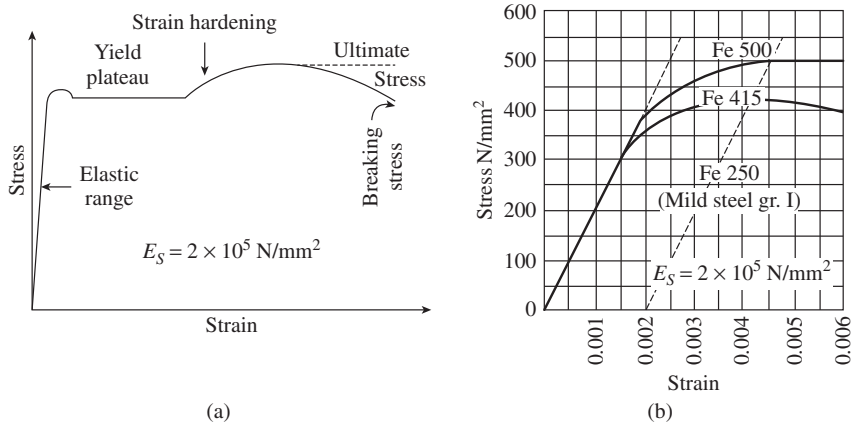


FIG. 1.13 Stress-Strain curve (a) Mild steel (b) HYSD bars (only initial portions of the curves are shown)

Clause 5.3 of IS 13920 stipulates that steel reinforcements of grade Fe 415 or less should be used in structures situated in earthquake zones. However, TMT bars of grades Fe 500 and Fe 550, having elongation more than 14.5 per cent, are also allowed. For providing sufficient bond between the bars and the concrete, the area, height, and pitch of ribs should satisfy Clause 5 of IS 1786 (see Fig. 1.14). The nominal size (in millimetres) of the available bars as per IS 1786 are 4, 5, 6, 8, 10, 12, 16, 20, 25, 28, 32, 36, and 40. A density of $7,850 \text{ kg/m}^3$ may be taken for calculating the nominal mass.

TABLE 1.20 Mechanical properties of high-strength deformed bars as per IS 1786:2008

S. No.	Property	Fe 415	Fe 415D	Fe 500	Fe 500D	Fe 550	Fe 550D	Fe 600
1.	0.2% proof stress/yield stress, min. N/mm ²	415.0	415.0	500.0	500.0	550.0	550.0	600.0
2.	Elongation, percentage, min. on gauge length $5.65\sqrt{A}^*$	14.5	18.0	12.0	16.0	10.0	14.5	10.0
3.	Tensile strength, min.	10% more than the actual 0.2% proof stress/yield stress but not less than 485.0 N/mm ²	12% more than the actual 0.2% proof stress/yield stress but not less than 500.0 N/mm ²	8% more than the actual 0.2% proof stress/yield stress but not less than 545.0 N/mm ²	10% more than the actual 0.2% proof stress/yield stress but not less than 565.0 N/mm ²	6% more than the actual 0.2% proof stress/yield stress but not less than 585.0 N/mm ²	8% more than the actual 0.2% proof stress/yield stress but not less than 600.0 N/mm ²	6% more than the actual 0.2% proof stress/yield stress but not less than 660.0 N/mm ²
4.	Total elongation at maximum force, percentage, min. on gauge length $5.65\sqrt{A}^*$	—	5%	—	5%	—	5%	—

*A is the cross-sectional area of the test piece.

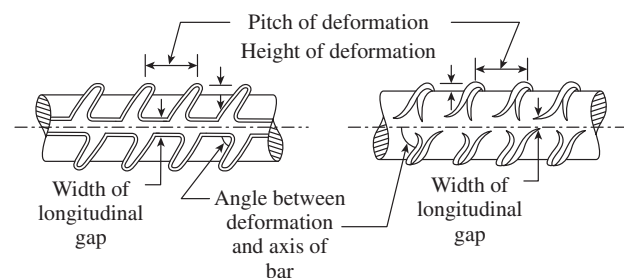


FIG. 1.14 Deformation on bars

Welded wire fabrics (WWF) consist of hard drawn steel wire mesh made from medium tensile steel, drawn out from higher diameter steel bars. As they undergo cold-working, their strength is higher than that of mild steel. WWF consists of longitudinal and transverse wires (at right angles to one another) joined by resistant spot welding using machines. They are available in different widths and rolls and as square or oblong meshes; see Table C-1 of SP 34 (S&T):1987 and SP 1566:1982. Their use in India is limited to small size slabs.

1.6.1 Corrosion of Rebars

Corrosion of steel rebars is considered the main cause of deterioration of numerous RCC structures throughout the world. In fact, the alkaline environment of concrete (pH of 12–13) provides a thin oxide passive film over the surface of steel rebars and reduces the corrosion rate considerably.

For steel bars surrounded by sound concrete, the passive corrosion rate is typically 0.1 μm per year. Without the passive film, the steel would corrode at rates at least 1000 times higher (ACI 222R-01). The destruction of the passive layer occurs when the alkalinity of the concrete is reduced or when the chloride concentration in concrete is increased to a certain level. In many cases, exposure of RC to chloride ions is the primary cause of premature corrosion of steel reinforcement. Although chlorides are directly responsible for the initiation of corrosion, they appear to play only an indirect role in the rate of corrosion after initiation. The primary factors controlling the corrosion rate are the availability of oxygen, electrical resistivity and relative humidity of the concrete, pH, and prevailing temperature. *Carbonation* is another cause for corrosion. Carbonation-induced corrosion often occurs in building facades that are exposed to rainfall, are shaded from sunlight, and have low concrete cover over the reinforcing steel. Carbonation occurs when carbon dioxide from the air penetrates the concrete and reacts with hydroxides (e.g., calcium hydroxide), to form carbonates. In the reaction with calcium hydroxide, calcium carbonate is formed. This reaction reduces the pH of the pore solution to as low as 8.5, destroying the passive film on steel rebars. It has to be noted that carbonation is generally a slow process. In high-quality concrete, carbonation is estimated to proceed at a rate up to 1.0 mm per year. The highest rates of carbonation occur

when the relative humidity is maintained between 50 per cent and 75 per cent. The amount of carbonation is significantly increased in concrete with a high water-to-cement ratio, low cement content, short curing period, low strength, and highly permeable paste. Corrosion can also occur when two different metals are in contact within concrete. For example, *dissimilar metal corrosion* can occur in balconies where embedded aluminium railings are in contact with the reinforcing steel.

Conventional concrete contains pores or micro-cracks. Detrimental substances or water can penetrate through these cracks or pores, leading to corrosion of steel bars. When corrosion takes place, the resulting rust occupies more than three times the original volume of steel from which it is formed. This drastic expansion creates tensile stresses in the concrete, which can eventually cause cracking, delamination, and spalling of cover concrete (see Fig. 4.5 of Chapter 4). The presence of corrosion also reduces the effective cross-sectional area of the steel reinforcement and leads to the failure of a concrete element and subsequently the whole structure. Mitigation measures to reduce the occurrence of corrosion include (a) decreasing the w/c or w/cm ratio of concrete and using pozzolans and slag to make the concrete less permeable (pozzolans and slag also increase the resistivity of concrete, thus reducing the corrosion rate, even after it is initiated), (b) providing dense concrete cover, as per Table 16 of IS 456, using controlled permeability formwork (CPF), thus protecting the embedded steel rebars from corrosive materials (see Section 4.4.5 for the details of CPF), (c) including the use of corrosion-inhibiting admixtures, (d) providing protective coating to reinforcement, and (e) using of sealers and membranes on the concrete surface. It should be noted that the sealers and membranes, if used, have to be reapplied periodically (Kerckhoff 2007).

As mentioned, one of the corrosion mitigation methods is by using the following reinforcements:

Fusion-bonded epoxy-coated reinforcing bars (IS 13620:1993) Typical coating thickness of these bars is about 130–300 μm . Damaged coating on the bars, resulting from handling and fabrication and the cut ends, must be properly repaired with patching material prior to placing them in the structure. These bars have been used in RC bridges from the 1970s and their performance is found to be satisfactory (Smith and Virmani 1996). They may have reduced bond strength.

Galvanized reinforcing bars (IS 12594:1988) The precautions mentioned for epoxy-coated bars are applicable to these bars as well. The protective zinc layer in galvanized rebars does not break easily and results in better bond.

Stainless steel bars Stainless steel is an alloy of nickel and chromium. Two types of stainless steel rods, namely SS304 and SS316, are used as per BS 6744:2001. Though the initial cost of

these bars is high, life cycle cost is lower and they may provide 80–125 years of maintenance-free service. The Progreso Pier in Yucatan, Mexico, was built during 1937–41 using stainless rebar and has not required maintenance until now.

Fibre-reinforced polymer bars (FRP bars) These are aramid fibre (AFRP), carbon fibre (CFRP) or glass fibre (GFRP) reinforced polymer rods. They are non-metallic and hence non-corrosive. Although their ultimate tensile strength is about 1500 MPa, their stress–strain curve is linear up to failure. In addition, they have one-fourth the weight of steel reinforcement and are expensive. The modulus of elasticity of CFRP is about 65 per cent of steel bars and the bond strength is almost the same. As the Canadian Highway Bridge Design Code, CSA-S6-06, has provisions for the use of GFRP rebars, a number of bridges in Canada are built using them. More details about them may be obtained from the work of GangaRao, et al. (2007) and the ACI 440R-07 report.

Basalt bars These are manufactured from continuous basalt filaments and epoxy and polyester resins using a pultrusion process. It is a low-cost, high-strength, high-modulus, and corrosion-resistant alternative to steel reinforcement. More information about these bars may be found in the study of Subramanian (2010).

In addition, *Zbar*, a pretreated high-strength bar with both galvanizing and epoxy coating, has been recently introduced in the USA. High-strength *MMFX steel bars*, conforming to ASTM A1035, with yield strength of 827 MPa and having low carbon and 8–10 per cent chromium have been introduced in the USA recently, which are also corrosion-resistant, similar to TMT CRS bars (www.mmf.com).

Clause 5.6.2 of IS 456 suggests the use of coating to reinforcement, and Amendment No. 3 of this clause states that the reduction of design bond strength of coated bars should be considered in design, but it does not elaborate. See Sections 7.4.2 and 7.5.3 of Chapter 7 for the reduction of design bond strength based on the ACI code provisions.

Viswanatha, et al. (2004), based on their extensive experience of testing rebars, caution about the availability of substandard rebars in India, including rerolled bars and inadequately quenched or low carbon content TMT bars. Hence, it is important for the engineer to accept the rebars only after testing them in accordance with IS 1608:2005 and IS 1786:2008. Basu, et al. (2004) also provide an overview of the important characteristics of rebars and a comparison of specifications of different countries.

1.7 CONCRETE MIXING, PLACING, COMPACTING, AND CURING

The measurement of materials for making concrete is called *batching* (see also Clause 10.2 of IS 456). Though volume

batching is used in small works, it is not a good method and weigh batching should always be attempted (fully automatic weigh batching equipment are used in RMC plants). The mixing of materials should ensure that the mass becomes homogeneous, uniform in colour and consistency. Again, hand mixing is not desirable for obvious reasons and machine mixing is to be adopted for better quality. Several types of mixtures are available; pan mixtures with revolving star blades are more efficient (Shetty 2005; IS 1791:1985; IS 12119:1987). Clause 10.3 of IS 456 stipulates that if there is segregation after unloading from the mixer, the concrete should be remixed. It also suggests that when using conventional tilting type drum mixtures, the mixing time should be at least two minutes and the mixture should be operated at a speed recommended by the manufacturer (normal speeds are 15–20 revolutions/minute). Clause 10.3.3 of IS 456 restricts the dosage of retarders, plasticizers, superplasticizers, and polycarboxylate-based admixtures to 0.5 per cent, 1.0 per cent, 2.0 per cent, and 1.0 per cent, respectively, by weight of cementitious materials.

Concrete can be transported from the mixer to the formwork by a variety of methods and equipment such as mortar pans, wheel barrows, belt conveyors, truck-mixer-mounted conveyor belts, buckets used with cranes and cable ways, truck mixer and dumpers, chutes or drop chutes, skip and hoist, transit mixer (in case of RMC), tremies (for placing concrete under water) or pumping through steel pipes. As there is a possibility of segregation during transportation, care should be taken to avoid it. More details about the methods of transportation may be found in the works of Panarese (1987), Kosmatka (2011), and Shetty (2005).

It is also important that the concrete is placed in the formwork properly to yield optimum results. Prior to *placing*, reinforcements must be checked for their correctness (location and size), cover, splice, and anchorage requirements, and any loose rust must be removed. The formwork must be cleaned, its supports adequately braced, joints between planks or sheets effectively plugged, and the inside of formwork applied with mould-releasing agents for easy stripping. Details of different kinds of formwork and their design may be found in the work of Hurd (2005) and IS 14687:1999 guidelines. It is necessary to thoroughly clean the surface of previous lifts with a water jet and treat them properly. Concrete should be continuously deposited as near as possible to its final position without any segregation. In general, concrete should be placed in thicker members in horizontal layers of uniform thickness (about 150 mm thick for reinforced members); each layer should be thoroughly consolidated before the next is placed. Chutes and drop chutes may be used when the concrete is poured from a height, to avoid segregation. Though Clause 13.2 of IS 456 suggests a permissible free fall of 1.5 m, it has been found that a free fall of even high-slump concrete of

up to 46 m directly over reinforcing steel does not result in segregation or reduction of compressive strength (Suprenant 2001).

Concreting during hot or cold weather should conform to the requirements of IS 7861(Part 1):1975 and IS 7861(Part 2):1981. More guidance on *hot weather concreting* is given in the work of Venugopal and Subramanian (1977) and ACI 305R-10 manual. Guidance for underwater concreting is provided in Clause 14 of IS 456.

Right after placement, concrete contains up to 20 per cent entrapped air. Vibration consolidates concrete in two stages: first by moving the concrete particles and then by removing entrapped air. The concrete should be deposited and compacted before the commencement of initial setting of concrete and should not be disturbed subsequently. Low-slump concrete can be consolidated easily, without adding additional water, by the use of superplasticizers. High frequency power driven internal or external vibrators (as per IS 2505, IS 2506, IS 2514, and IS 4656) also permit easy consolidation of stiff mixes having low w/cm ratio (manual consolidation with tamping rod is suitable only for workable and flowing mixtures). The internal vibrator or needle vibrator is immersed in concrete and the external vibrator is attached to the forms. (The radius of action of a needle vibrator with a diameter of 20–40 mm ranges between 75 mm and 150 mm; ACI 309R:05 provides more data on consolidation.) Good compaction with vibrators prevents honeycombing and results in impermeable and dense concrete, better bond between concrete and reinforcement, and better finish. Guidance on construction joints and cold joints is provided in Clause 13.4 of IS 456.

All newly placed and finished concrete slabs should be cured and protected from drying and from extreme changes in temperature. *Wet curing* should start as soon as the final set occurs and should be continued for a minimum period of 7–15 days (longer curing is required in case of concretes with fly ash). It has to be noted that in concretes without the use of retarders or accelerators, final set of cement takes place at about six hours. Concreting in hot weather conditions requires special precautions against rapid evaporation and drying due to high temperatures. More information on curing is provided in Clause 13.5 of IS 456 and also in Section 4.4.5 of Chapter 4.

Removal of forms It is advantageous to leave forms in place as long as possible to continue the curing period. As per Clause 11.3 of IS 456, the vertical supporting members of formwork (shoring) should not be removed until the concrete is strong enough to carry at least twice the stresses to which the concrete may be subjected to at the time of removal of formwork. When the ambient temperature is above 15°C and where Portland cement is used and adequate curing is done, the vertical formwork to columns, walls, and beams can be

removed in 16–24 hours after concreting. Beam and floor slab forms and supports (props) may be removed between 3 and 21 days, depending on the size of the member and the strength gain of the concrete (see Clause 11.3.1 of IS 456). If high early strength concrete is used, these periods can be reduced. Since the minimum stripping time is a function of concrete strength, the preferred method of determining stripping time in other cases is to be determined based on the tests of site-cured cubes or concrete in place. More details including shoring and reshoring of multi-storey structures may be found in ACI 347-04 guide.

1.8 PROPERTIES OF FRESH AND HARDENED CONCRETE

A designer needs to have a thorough knowledge of the properties of concrete for the design of RC structures. As seen in the previous sections, present-day concrete is much complicated and uses several different types of materials, which considerably affect the strength and other properties. Complete knowledge of these materials and their use and effects on concrete can be had from the works of Gambhir (2004), Mehta and Monteiro (2006), Mindess, et al. (2003), Neville (2012), Neville and Brooks (2010), Santhakumar (2006), and Shetty (2005). An introduction to some of the properties, which are important for the designer and construction professionals, is presented in this section.

1.8.1 Workability of Concrete

As discussed in Section 1.2.3, water added to the concrete mix is required not only for hydration purposes but also for workability. *Workability* may be defined as the property of the freshly mixed concrete that determines the ease and homogeneity with which it can be mixed, placed, compacted, and finished. The desired degree of workability of concrete is provided in Table 1.12. The main factor that affects workability is the water content (in the absence of admixtures). The other interacting factors that affect workability are aggregate type and grading, aggregate/cement ratio, presence of admixtures, fineness of cement, and temperature. It has to be noted that finer particles require more water to wet their large specific surface, and the irregular shape and rough texture of angular aggregate demand more water. Workability should be checked frequently by one of the standard tests (*slump*, *compacting factor*, *Vee Bee consistency*,

or *flow table*) as described in IS 1199:1955. Although it does not measure all factors contributing to workability, *slump test* is the most commonly used method to measure the consistency of the concrete, because of its simplicity. This test is carried out using an open-ended cone, called the *Abrams cone*. This cone is placed on a hard non-absorbent surface and filled with fresh concrete in four layers, and each time the concrete is tamped using a rod of standard dimensions. At the end of the fourth stage, the concrete is struck off level with a trowel at the top of the mould. Now, the mould is carefully lifted vertically upwards without disturbing the concrete in the cone, thereby allowing the concrete to subside. This subsidence is termed as *slump* and is measured to the nearest 5 mm. Figure 1.15 shows the slump testing mould, measurement, and types of slumps. If a shear slump (indicates concrete is non-cohesive) or collapse slump (indicates a high workability mix) is achieved, a fresh sample should be taken and the test repeated. A slump of about 50–100 mm is used for normal RC (see Table 1.12).

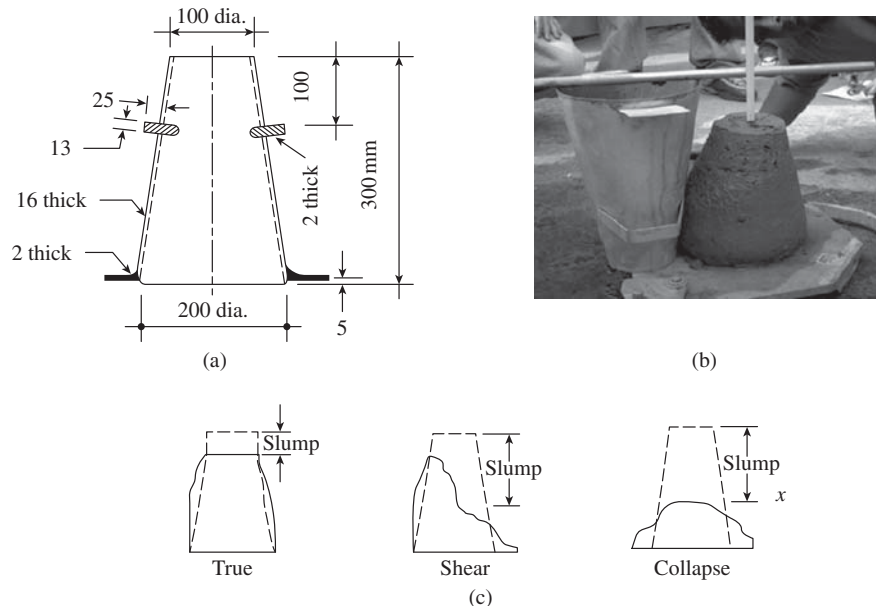


FIG. 1.15 Slump testing (a) Typical mould for slump test (b) Measuring slump (c) Types of slump

1.8.2 Compressive Strength

Compressive strength at a specified age, usually 28 days, measured on standard cube or cylinder specimens, has traditionally been used as the criterion for the acceptance of concrete. It is very important for the designer because concrete properties such as stress–strain relationship, modulus of elasticity, tensile strength, shear strength, and bond strength are expressed in terms of the uniaxial compressive strength. The compressive strengths used in structural applications vary from 20 N/mm^2 to as high as 100 N/mm^2 . (In One, World Trade Center, New York, USA, a concrete with a compressive strength of 96.5 MPa was used with a modulus of elasticity of $48,265 \text{ MPa}$).

Cube and Cylinder Tests

In India, the UK, and several European countries, the characteristic compressive strength of concrete (denoted by f_{ck}) is determined by testing to failure 28-day-old concrete cube specimens of size 150 mm × 150 mm × 150 mm, as per IS 516:1959. When the largest nominal size of aggregate does not exceed 20 mm, 100 mm cubes may also be used. However, in the USA, Canada, Australia, and New Zealand, the compressive strength of concrete (denoted by f'_c) is determined by testing to failure 28-day-old concrete cylinder specimens of size 150 mm diameter and 300 mm long. Recently, 70 mm cube or 75 mm cylinder HSC or UHSC specimen is being recommended for situations in which machine capacity may be exceeded (Graybeal and Davis 2008).

The concrete is poured in the cube or cylinder mould in layers of 50 mm and compacted properly by either hand or a vibrator so that there are no voids. The top surface of these specimens should be made even and smooth by applying cement paste and spreading smoothly on the whole area of the specimen. The test specimens are then stored in moist air of at least 90 per cent relative humidity and at a temperature of $27^\circ\text{C} \pm 2^\circ\text{C}$ for 24 hours. After this period, the specimens are marked and removed from the moulds and kept submerged in clear fresh water, maintained at a temperature of $27^\circ\text{C} \pm 2^\circ\text{C}$ until they are tested (the water should be renewed every seven days). The making and curing of test specimen at site is similar (see also Clause 3.0 of IS 516).

These specimens are tested by a compression testing machine after 7 days of curing or 28 days of curing. Load should be applied gradually at the rate of 140 kg/cm² per minute until the specimen fails. Load at the failure divided by the area of specimen gives the compressive strength of concrete. A minimum of three specimens should be tested at each selected age. If the strength of any specimen varies by more than ± 15 per cent of average strength, results of such a specimen should be rejected (Clause 15.4 of IS 456). The average of *three* specimens gives the compressive strength of concrete. Sampling and acceptance criteria for concrete strength, as per IS 456, are provided in Section 4.7.4 of Chapter 4. (In the USA, the evaluation of concrete strength tests is done as per ACI 214R-02.) Figure 1.16 shows the cube testing and various failure modes of concrete cubes. The ideal failure mode, with almost vertical cracks (see Fig. 1.16b) is rarely achieved due to the rough contact surface between the concrete cube and the plate of testing machine. When the stress level reaches about 75–90 per cent of the

maximum, internal cracks are initiated in the mortar throughout the concrete mass, parallel to the direction of the applied load. The concrete tends to expand laterally due to Poisson's effect, and the cube finally fails leaving two truncated pyramids one over the other (see Fig. 1.16b). Sometimes the failure may be explosive, especially in cubes of HSC; to avoid injuries, proper precautions should be taken to contain the debris using high resistance and transparent polycarbonate or steel mesh shields around the testing machine.

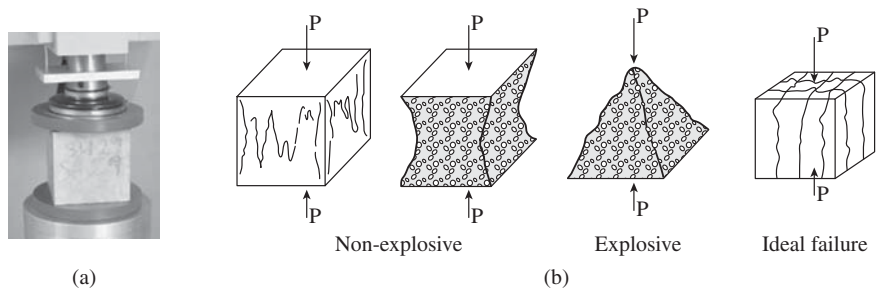


FIG. 1.16 Cube testing and failure of concrete cubes (a) Cubes in testing machine (b) Failure of concrete cubes

Factors Affecting Compressive Strength

The compressive strength of concrete is affected by the following important factors: w/c or w/cm ratio, type of cement, use of supplementary cementitious materials, type of aggregates, quantity and quality of mixing water, moisture and temperature conditions during curing, age of concrete, rate of loading during the cube or cylinder test (the measured compressive strength of concrete increases with increasing rate of loading), and the size of specimen.

The w/c ratio is inversely related to concrete strength: the lower the ratio, the greater the strength. It is also directly linked to the spacing between cement particles in the cement paste. When the spacing is smaller, cement hydrates fill the gaps between the cement particles faster and the links created by the hydrates will be stronger, resulting in stronger concrete (Bentz and Aitcin, 2008). Various mathematical models have been developed to link strength to the porosity of the hydrates. In 1918, Abrams presented his classic law of the following form (Shetty 2005):

$$f_{c,28} = \frac{k_1}{k_2^{wc}} \quad (1.2a)$$

where $f_{c,28}$ is the 28-day compressive strength, k_1 and k_2 are the empirical constants, and w/c is the w/c ratio by volume. For 28-day strength of concrete recommended by ACI 211.1-91, the constants k_1 and k_2 are 124.45 MPa and 14.36, respectively. Popovics (1998) observed that these values are conservative and suggested the values 187 MPa and 23.07, respectively, for k_1 and k_2 . Abrams' w/c ratio law states that

the strength of concrete is dependent only upon the w/c ratio, provided the mix is workable. Abram's law is a special case of the following Feret formula developed in 1897 (Shetty 2005):

$$f_{c,28} = k \left(\frac{V_c}{V_c + V_w + V_a} \right) \quad (1.2b)$$

where V_c , V_w , and V_a are the absolute volumes of cement, water, and entrained air, respectively, and k is a constant. In essence, strength is related to the total volume of voids and the most significant factor in this is the w/c ratio. The graph showing the relationship between the strength and w/c ratio is approximately hyperbolic in shape (see Fig. 1.17).

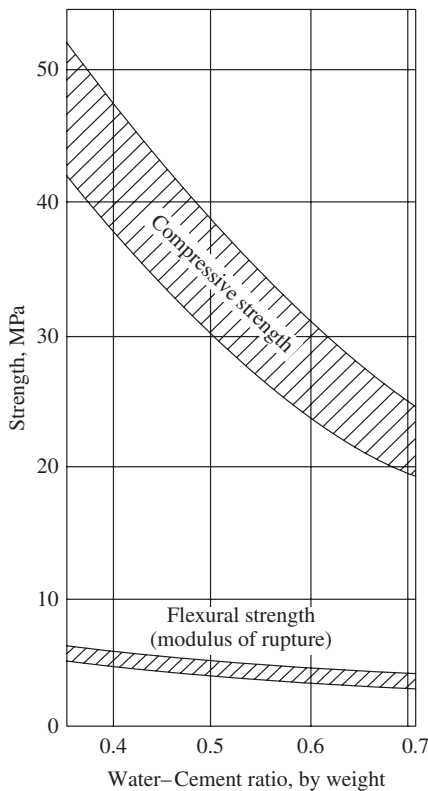


FIG. 1.17 Relation between strength and w/c ratio of normal concrete

At a more fundamental level, this relation can be expressed as a function of the gel/space ratio (x), which is the ratio of the volume of the hydrated cement paste to the sum of the volumes of the hydrated cement and the capillary voids. The data from Powers (1961) gives the following relationship:

$$f_{c,28} = 234x^3 \text{ MN/m}^2 \quad (1.2c)$$

where x is the gel/space ratio and 234 is the intrinsic strength of the gel in MPa for the type of cement and specimen used by

Powers. It has to be noted that this relation is independent of the age of the concrete and the mix proportions. This equation is valid for many types of cement, but the values of the numerical coefficients vary a little depending on the intrinsic strength of the gel. Such models that focus only on the cement paste ignore the effects of the aggregate characteristics on strength, which can be significant. A comparison of these mathematical models is provided by Popovics (1998). Based on the strength vs w/c ratio curves provided in the earlier version of IS 10262, Rajamane (2005) derived the following equation.

$$f_{c,28} = 0.39f_{cem}[(1/wc) - 0.50] \quad (1.2d)$$

where f_{cem} is the 28-day compressive strength of cement tested as per IS 4031 (MPa) and wc is the w/c ratio by weight.

Many researchers have also attempted to estimate the strength of concrete at 1, 3, or 7 days and correlate it to the 28-day strength. This relationship is useful for formwork removal and to monitor early strength gain; however, it depends on many factors such as the chemical composition of cement, fineness of grinding, and temperature of curing. The 7-day strength is often estimated to be about 75 per cent of the 28-day strength (Neville 2012). Neville, however, suggests that if the 28-day strength is to be estimated using the strength at 7 days, a relationship between the 28-day and 7-day strengths has to be established experimentally for the given concrete. For concrete specimens cured at 20°C, Clause 3.1.2(6) of Eurocode 2 (EN 1992-1-1:2004) provides the following relationship.

$$f_{cm}(t) = \exp \left[s \left(1 - \left(\frac{28}{t} \right)^{0.5} \right) \right] f_{cm} \quad (1.3a)$$

where $f_{cm}(t)$ is the mean compressive strength at age t days, f_{cm} is the mean 28-day compressive strength, and s is a coefficient depending on the type of cement; $s = 0.2, 0.25,$ and 0.38 for high early strength, normal early strength, and slow early strength cement, respectively. ACI Committee 209.2R-08 recommends the relationship for moist-cured concrete made with normal Portland concrete as given here:

$$f_{cm}(t) = \left(\frac{t}{a + bt} \right) f_{c28} \quad (1.3b)$$

The values of constants a and b are 4.0 and 0.85, respectively, for normal Portland cement and 2.3 and 0.92, respectively, for high early strength cement. The 1978 version of IS 456 specified an 'age factor', based on Eq. (1.3b), using $a = 4.7$ and $b = 0.833$, but that provision has been deleted in the 2000 version of the code.

Influence of Size of Specimen

The pronounced effect of the height/width ratio and the cross-sectional dimension of the test specimen on the compressive

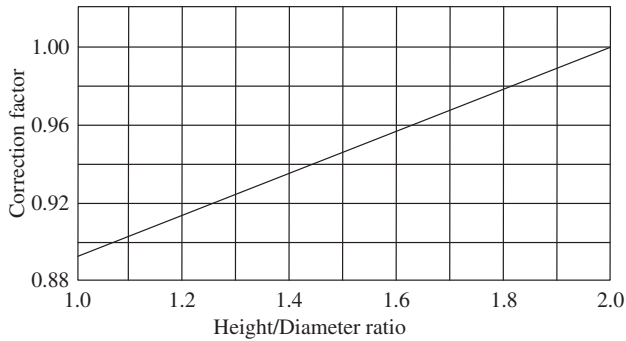


FIG. 1.18 Correction factor for height/diameter ratio of cylinder

strength has been observed by several researchers. The difference in compressive strength of different sizes of specimens may be due to several factors such as St Venant's effect, size effect, or lateral restraint effect due to the testing machine's platen (Pillai and Menon 2003). In addition, the preparation of the end conditions (cappings) of the concrete cylinder can significantly affect the measured compressive strength. When the height/diameter ratio of cylinders is less than 2.0, IS 516:1959, suggests a correction factor as shown in Fig. 1.18. Standard cubes with height/width ratio of 1.0 have been found to have higher compressive strength than standard cylinders with height/diameter ratio of 2.0. The ratio of standard cylinder strength and standard cube strength is about 0.8–0.95; higher ratio is applicable for HSC. Similarly 100 mm × 200 mm cylinders exhibit 2–10 per cent higher strengths than 150 mm × 300 mm cylinders; the difference is less for higher strength concrete (Graybeal and Davis 2008). It has to be noted that the ACI code formulae, which are based on standard cylinder strength, f'_c have been converted to standard cube strength, f_{ck} , for easy comparison, by using the relation $f'_c = 0.8f_{ck}$ throughout this book. A more precise coefficient R to convert cylinder strength to cube strength is $R = 0.76 + 0.2 \log(f'_c/20)$.

In the case of cubes, the specimens are placed in the testing machine in such a way that the load is applied on opposite sides of the cube as cast, that is, not to the top and bottom. On the other hand, cylinders are loaded in the direction in which they are cast. Due to this reason and also because the standard cylinders have height/width ratio of two, the compressive strengths predicted by cylinders are more reliable than cubes.

1.8.3 Stress–Strain Characteristics

Typical stress–strain curves of normal weight concrete of various grades, obtained from uniaxial compression tests, are shown in Fig. 1.19(a) and a comparison of normal weight and lightweight concrete is shown in Fig. 1.19(b). (The idealized stress block adopted in IS 456 are given in Fig. 5.4 in Section 5.4 of Chapter 5). It has to be noted that, for design, the value of maximum compressive strength of concrete in structural elements is taken as 0.85 times the cylinder strength, f'_c , which is approximately equal to $0.67f_{ck}$.

It is seen from Fig. 1.19 that the curves are initially linear and become non-linear when the stress level exceeds about 40 per cent of the maximum stress. The maximum stress is reached when the strain is approximately 0.002; beyond this point, the stress–strain curve descends. IS 456 limits the maximum failure strain in concrete under direct compression to 0.002 (Clause 39.1a) and under flexure to 0.0035 (Clause 38.1b). The shape of the curve is due to the formation of micro-cracks within the structure of concrete. The descending branch of the curve can be fully traced only with rigid testing machines. In axially flexible testing machines, the test cube or cylinder will fail explosively when the maximum stress is reached.

Numerical approximations of stress–strain curves of concretes have been provided by various researchers, and a comparison of these formulae is provided by Popovics (1998).

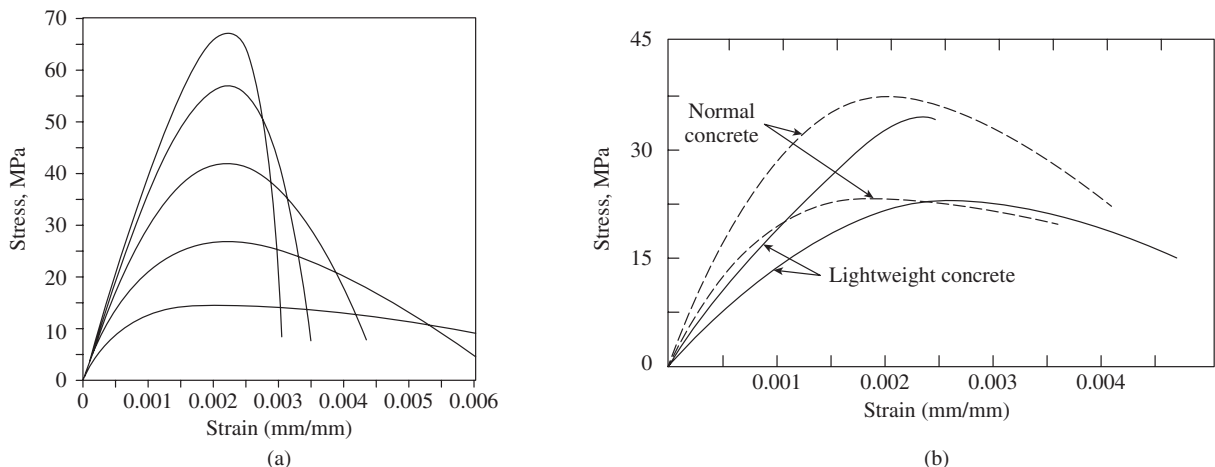


FIG. 1.19 Typical stress–strain curves of concrete in compression (a) Concrete with normal weight aggregates (b) Normal weight vs lightweight aggregate concrete

Such a mathematical definition of stress–strain curve is required for non-linear analysis of concrete structures. HSCs exhibit more brittle behaviour, which is reflected by the shorter horizontal branch of stress–strain curves.

1.8.4 Tensile Strength

As mentioned earlier, concrete is very weak in tension, and direct tensile strength is only about 8–11 per cent of compressive strength for concretes of grade M25 and above (Shetty 2005). The use of pozzolanic admixtures increases the tensile strength of concrete. Although the tensile strength of concrete increases with an increase in compressive strength, the rate of increase in tensile strength is of the decreasing order (Shetty 2005). The tensile strength of concrete is generally not taken into account in the design of concrete elements. Knowledge of its value is required for the design of concrete structural elements subject to transverse shear, torsion, and shrinkage and temperature effects. Its value is also used in the design of prestressed concrete structures, liquid retaining structures, roadways, and runway slabs. Direct tensile strength of concrete is difficult to determine. The *splitting (cylinder) tensile test* on 150 mm × 300 mm cylinders, as per IS 5816:1999, or the *third-point flexural loading test* on 150 mm × 150 mm × 700 mm concrete beams, as per IS 516:1959, is often used to find the tensile strength. The splitting tensile test is easier to perform and gives more reliable results than other tension tests; though splitting strength may give 5–12 per cent higher value than direct tensile strength (Shetty 2005). According to Mehta and Monteiro (2010), the third-point flexural loading test tends to overestimate the tensile strength of concrete by 50–100 per cent.

The theoretical maximum flexural tensile stress occurring in the extreme fibres of RC beams, which causes cracking, is referred to as the *modulus of rupture*, f_{cr} . Clause 6.2.2 of IS 456 gives the modulus of rupture or flexural tensile strength as

$$f_{cr} = 0.7\sqrt{f_{ck}} \quad (1.4)$$

It should be noted that Clause 9.5.2.3 of ACI 318 code suggests a lower, conservative value for the modulus of rupture, which equals $\lambda 0.55\sqrt{f_{ck}}$, where λ is the modification factor for lightweight concrete and equals 1.0 for normal weight concrete, 0.85 for sand-lightweight concrete, and 0.75 for all lightweight concrete. IS 456 does not provide an empirical formula for estimating the direct tensile strength, f_{ct} . Clause R8.6.1 of ACI 318 suggests an average splitting tensile strength of

$$f_{ct} = 0.5\sqrt{f_{ck}} \quad (1.5)$$

Shear strength Pure shear is a rare occurrence; usually a combination of flexural and shear stresses exists, resulting in a

diagonal tension failure. The design shear strength of concrete is given in Table 19 of IS 456 as a function of percentage flexural reinforcement. The maximum shear stress in concrete with shear reinforcement is restricted in Clause 40.2.3 to the following value:

$$\tau_{c,max} = 0.63\sqrt{f_{ck}} \quad (1.6)$$

More discussions on shear strength of concrete are provided in Chapter 6.

Bond strength The common assumption in RC that plane sections remain plane after bending will be valid only if there is perfect bond between concrete and steel reinforcement. Bond strength depends on the shear stress at the interface between the reinforcing bar and the concrete and on the geometry of the reinforcing bar. Clause 26.2.1.1 of IS 456 provides a table for design bond stress and is approximately represented by

$$\tau_{bd} = 0.16(f_{ck})^{2/3} \quad (1.7)$$

More discussions on bond strength of concrete are provided in Chapter 7.

1.8.5 Bearing Strength

The compressive stresses at supports, for example, at the base of column, must be transferred by bearing (Niyogi 1974). Clause 34.4 of IS 456 stipulates that the permissible bearing stress on full area of concrete in the working stress method can be taken as $0.25f_{ck}$ and for limit state method it may be taken as $0.45f_{ck}$. According to Clause 10.4.1 of ACI 318, the design bearing strength of concrete should not exceed $\phi 0.85 f'_c$, where ϕ is the strength reduction factor, which is taken as 0.65. Thus, it is approximately equal to $0.442f_{ck}$.

1.8.6 Modulus of Elasticity and Poisson's Ratio

Concrete is not an elastic material, that is, it will not recover its original shape on unloading. In addition, it is non-linear and exhibits a non-linear stress–strain curve. Hence, the elastic constants such as *modulus of elasticity* and *Poisson's ratio* are not strictly applicable. However, they are used in the analysis and design of concrete structures, assuming elastic behaviour. The modulus of elasticity of concrete is a key factor for estimating the deformation of buildings and members as well as a fundamental factor for determining the *modular ratio*, m . The use of HSC will result in higher modulus of elasticity and in reduced deflection and increased tensile strengths. The modulus of elasticity is primarily influenced by the elastic properties of the aggregates and to a lesser extent by the curing conditions, age of the concrete, mix proportions, porosity of concrete, and the type of cement. It is normally related to the compressive strength of concrete and may be determined by means of an extensometer attached to the compression test specimen as described in IS 516:1959.

The Young’s modulus of elasticity may be defined as the ratio of axial stress to axial strain, within the elastic range. When the loading is of low intensity and of short duration, the initial portion of the stress–strain curve of concrete in compression is linear, justifying the use of modulus of elasticity. However, when there is sustained load, inelastic creep occurs even at relatively low stresses, making the stress–strain curve non-linear. Moreover, the effects of creep and shrinkage will make the concrete behave in a non-linear manner. Hence, the initial tangent modulus is considered to be a measure of *dynamic modulus of elasticity* (Neville and Brooks 2010).

When linear elastic analysis is used, one should use the *static modulus of elasticity*. Various definitions of modulus of elasticity are available: initial tangent modulus, tangent modulus (at a specified stress level), and secant modulus (at a specified stress level), as shown in Fig. 1.20. Among these, the *secant modulus*, which is the slope of a line drawn from the origin to the point on the stress–strain curve corresponding to 40 per cent of the failure stress, is found to represent the average value of E_c under service load conditions (Neville and Brooks 2010). Clause 6.2.3.1 of IS 456 suggests that the short-term static modulus of elasticity of concrete, E_c , may be taken as

$$E_c = 5000 \sqrt{f_{ck}} \text{ N/mm}^2 \quad (1.8a)$$

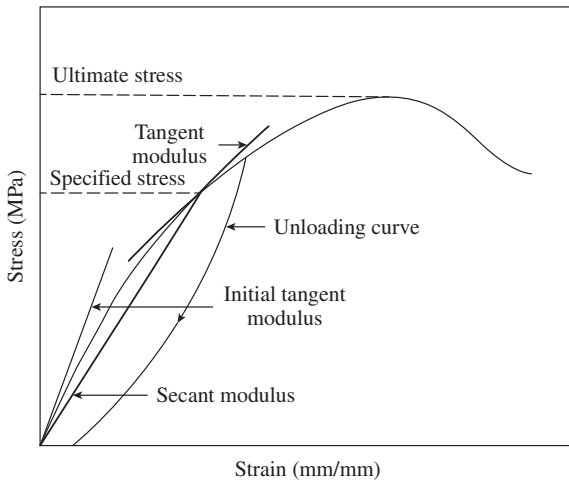


FIG. 1.20 Various definitions of modulus of elasticity of concrete

As per Clause 8.5.1 of ACI 318, the modulus of elasticity for concrete may be taken as

$$E_c = \rho_c^{1.5} 0.038 \sqrt{f_{ck}} \text{ N/mm}^2 \quad (1.8b)$$

where ρ_c is the unit weight of concrete (varies between 1440 kg/m³ and 2560 kg/m³). For normal weight concrete, ACI code allows it to be taken as (assuming $\rho_c = 2300 \text{ N/mm}^2$)

$$E_c = 4200 \sqrt{f_{ck}} \text{ N/mm}^2 \quad (1.8c)$$

Both IS 456 and ACI 318 caution that the actual measured values may differ by ± 20 per cent from the values obtained from Eq. (1.8). Moreover, the US code value is 16 per cent less than the value specified by the Indian code. It has to be noted that the use of lower value of E_c will result in a conservative (higher) estimate of the short-term elastic deflection.

The ACI committee report on HSC (ACI 363R-92) suggests the following equation, which has been adopted by NZS 3101-Part 1:2006 and CSA A23.3-04:

$$E_c = (2970 \sqrt{f_{ck}} + 6900) (\rho_c/2300)^{1.5} \text{ N/mm}^2 \quad (1.8d)$$

for 26 MPa < f_{ck} < 104 MPa

Noguchi, et al. (2009) proposed the following equation, which is applicable to a wide range of aggregates and mineral admixtures used in concrete.

$$E_c = k_1 k_2 \times 3.36 \times 10^4 (\rho_c/2400)^2 (f_{ck}/75)^{1/3} \text{ N/mm}^2 \quad (1.8e)$$

where the correction factors k_1 and k_2 are given in Tables 1.21 and 1.22.

TABLE 1.21 Values of correction factor k_1

Type of Coarse Aggregate	Value of k_1
Crushed limestone, calcined bauxite	1.20
Crushed quartzite aggregate, crushed andesite, crushed basalt, crushed clay slate, crushed cobblestone	0.95
Coarse aggregate other than above	1.0

TABLE 1.22 Values of correction factor k_2

Type of Mineral Admixture	Value of k_2
Silica fume, GGBS, fly ash fume	0.95
Fly ash	1.10
Mineral admixture other than the above	1.0

The *dynamic modulus of elasticity* of concrete, E_{cd} , corresponds to a small instantaneous strain. It can be determined by the non-destructive electro-dynamic method, by measuring the natural frequency of the fundamental mode of longitudinal vibration of concrete prisms, as described in IS 516:1959. The dynamic modulus of elasticity has to be used when concrete is used in structures subjected to dynamic loading (i.e., impact or earthquake). The value of E_{cd} is generally 20 per cent, 30 per cent, and 40 per cent higher than the secant modulus for high-, medium-, and low-strength concretes, respectively (Mehta and Monteiro 2006).

Poisson’s ratio is defined as the ratio of lateral strain to the longitudinal strain, under uniform axial stress. Experimental studies have predicted widely varying values of Poisson’s ratio, in the range of 0.15–0.25. A value of 0.2 is usually suggested for design for both NSCs and HSCs. For lightweight concretes, the Poisson’s ratio has to be determined from tests.

1.8.7 Strength under Combined Stresses

Structural members are usually subjected to a combination of forces, which may include axial force, bending moments, transverse shear, and twisting moments. Any state of combined stress acting at any point in a member may be reduced to three principal stresses acting at right angles to each other on an appropriately oriented elementary cube in the material. Any or all of the principal stresses can be either compression or tension. When one of these three principal stresses is zero, a state of *biaxial* stress exists; if two of them are zero, the state of stress is *uniaxial*. In most of the situations, only the uniaxial strength properties are known from simple tests described in this chapter. The failure strength under combined stresses is usually defined by an appropriate failure criterion. Until now, neither a general theory of strength of concrete under combined stresses nor a universally accepted failure criterion has been proposed.

However, the strength of concrete for biaxial state of stress has been established experimentally by Kupfer, et al. (1969) (see Fig. 1.21). This figure shows that under biaxial tension, the strength is close to that of uniaxial tension. When one principal stress is tension and other is compressive, the concrete cracks at a lower stress than it would have in uniaxial tension or compression. Under biaxial compression, the strength is greater than the uniaxial compression by about 27 per cent.

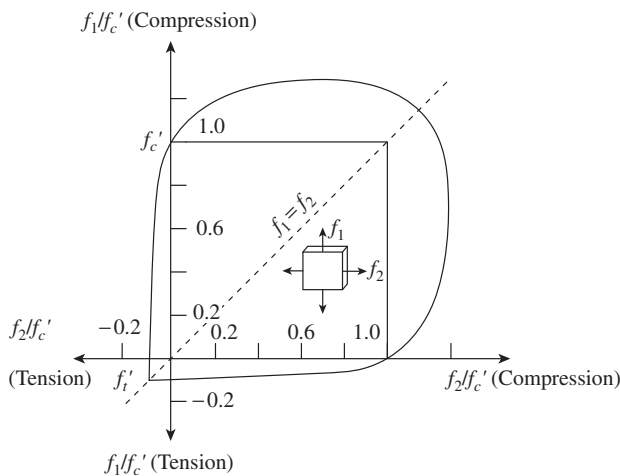


FIG. 1.21 Strength of concrete in biaxial stress

Source: Kupfer, et al. 1969, reprinted with permission from ACI

1.8.8 Shrinkage and Temperature Effects

As shrinkage and temperature effects are similar, they are both considered in this section.

Shrinkage Effects

Shrinkage and creep are not independent phenomena. For convenience, their effects are treated as separate, independent,

and additive. The total shrinkage strain in concrete is composed of the following:

1. *Autogenous shrinkage*, which occurs during the hardening of concrete (Holt 2001)
2. *Drying shrinkage*, which is a function of the migration of water through hardened concrete

Drying shrinkage, often referred to simply as *shrinkage*, is caused by the evaporation of water from the concrete. Both shrinkage and creep introduce time-dependent strains in concrete. However, shrinkage strains are independent of the stress conditions of concrete. Shrinkage can occur before and after the hydration of the cement is complete. It is most important, however, to minimize it during the early stages of hydration in order to prevent cracking and to improve the durability of the concrete. Shrinkage cracks in RC are due to the differential shrinkage between the cement paste, the aggregate, and the reinforcement. Its effect can be reduced by the prolonged curing, which allows the tensile strength of the concrete to develop before evaporation occurs. The most important factors that influence shrinkage in concrete are (a) type and content of aggregates, (b) w/c ratio, (c) effective age at transfer of stress, (d) degree of compaction, (e) effective section thickness, (f) ambient relative humidity, and (f) presence of reinforcement (ACI 209R-92).

Shrinkage strain is expressed as a linear strain (mm/mm). In the absence of reliable data, Clause 6.2.4.1 of IS 456 recommends the approximate value for the total shrinkage strain for design as 0.0003. (ACI 209R-92 suggests an average value of 780×10^{-6} mm/mm for the ultimate shrinkage strain, ϵ_{sh}). Different models for the prediction of creep under compression and shrinkage induced strains in hardened concrete are presented and compared in ACI 209.2R-08. Long-term deflection calculations considering the effects of shrinkage and creep are covered in Chapter 12.

Temperature Effects

Concrete expands with rise in temperature and contracts with fall in temperature. The effects of thermal contraction are similar to the effects of shrinkage. To limit the development of temperature stresses, expansion joints are to be provided, especially when there are marked changes in plan dimensions. In addition, when the length of the building exceeds 45 m, expansion joints are to be provided, as per Clause 27 of IS 456. Temperature stresses may be critical in the design of concrete chimneys and cooling towers. Roof slabs may also be subjected to thermal gradient due to solar radiation. In large and exposed surfaces of concrete such as slabs, nominal reinforcements are usually placed near the exposed surface to take care of temperature and shrinkage stresses. The coefficient of thermal expansion depends on the type of cement and aggregate, cement content, relative humidity, and

the size of section. Clause 6.2.6 of IS 456 provides a table to choose the value of coefficient of thermal expansion based on the aggregate used. However, SP 24:1983 recommends a value of 11×10^{-6} mm/mm per degree Celsius for the design of liquid storage structures, bins and chimneys, which is close to the thermal coefficient of steel (about 11×10^{-6} mm/mm per degree Celsius). The calculation of deflection due to temperature effects is discussed in Section 12.4.3 of Chapter 12. More discussions on thermal and shrinkage effects are provided in Section 3.9.2 of Chapter 3.

Fire design of concrete structures is outside the scope of this book. When exposed to fire, both concrete and steel reinforcement of RC members lose 60 per cent of their characteristic strength at a temperature of 500°C. Where HSCs are used, consideration should be given to mitigate the effects of spalling (e.g., use of fibre reinforcement, sacrificial concrete layers, thermal barriers, and fire-resisting concrete.). More information on fire design may be found in fib reports (2007, 2008).

1.8.9 Creep of Concrete

Creep in concrete is the gradual increase in deformation (strain) with time in a member subjected to sustained loads. The creep strain is much larger than the elastic strain on loading (creep strain is typically two to four times the elastic strain). If the specimen is unloaded, there is an immediate elastic recovery and a slower recovery in the strain due to creep (see Fig. 1.22). Both amounts of recovery are much less than the original strains under load. If the concrete is reloaded at a later date, instantaneous and creep strains develop again. Creep occurs under both compressive and tensile stresses and always increases with temperature. HSCs creep less than NSCs. When the stress in concrete does not exceed one-third of its characteristic compressive strength, creep may be assumed proportional to the stress (Clause 6.2.5 of IS 456). It has to be noted that, unlike concrete, steel will creep only above 700°F.

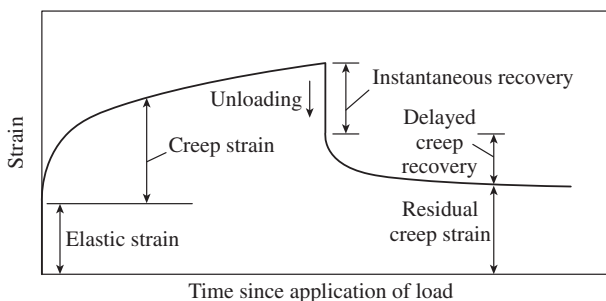


FIG. 1.22 Typical creep curve

The main factors affecting creep strain are the concrete mix and strength, the type of aggregate used, curing, ambient relative humidity, and the magnitude and duration of sustained

loading. As per IS 456, the ultimate creep strain ϵ_{cp} is to be calculated from the creep coefficient C_t (θ in IS nomenclature) given in Clause 6.2.5.1. Calculation of long-term deflection due to creep is provided in Section 12.4.1 of Chapter 12.

More information on creep, shrinkage, and temperature effects may be obtained from the work of Bamforth, et al. (2008).

1.8.10 Non-destructive Testing

Non-destructive tests are used to find the strength of existing concrete elements. They are classified as follows:

1. *Half-cell electrical potential method* to detect the corrosion potential of reinforcing bars in concrete
2. *Schmidt/Rebound hammer test* (IS 13311-Part 2:1992) to evaluate the surface hardness of concrete
3. *Carbonation depth measurement test* to determine whether moisture has reached the depth of the reinforcing bars, thereby leading to corrosion
4. *Permeability test* to measure the flow of water through the concrete
5. *Penetration resistance* or *Windsor probe test* to measure the surface hardness and hence the strength of the surface and near-surface layers of the concrete
6. *Covermeter test* to measure the distance of steel reinforcing bars beneath the surface of the concrete and the diameter of the reinforcing bars
7. *Radiographic test* to detect voids in the concrete and the position of prestressing ducts
8. *Ultrasonic pulse velocity test* (IS 13311-Part 1:1992) mainly to measure the time of travel of ultrasonic pulse passing through the concrete and hence concrete quality
9. *Sonic methods*, which use an instrumented hammer providing both sonic echo and transmission methods, to predict the integrity of piles and bridge decks
10. *Tomographic modelling*, which uses the data from ultrasonic transmission tests in two or more directions, to detect voids in concrete
11. *Impact echo testing* to detect voids, delamination, and other anomalies in concrete
12. *Ground penetrating radar* or *impulse radar testing* to detect the position of reinforcing bars or stressing ducts
13. *Infrared thermography* to detect voids, delamination, and other anomalies in concrete and also to detect water entry points in buildings

The details of these tests may be found in ACI 228.1R-03 manual and the work of Malhotra and Carino (2003).

1.9 DURABILITY OF CONCRETE

Although several unreinforced concrete structures, built 2000 years ago, such as the Pantheon in Rome and several

aqueducts in Europe, are still in excellent condition, many RC structures built in the twentieth century have deteriorated within 10–20 years. In several countries like the USA, about 40–50 per cent of the expenditure in the construction industry is spent on repair, maintenance, and rehabilitation of existing structures. These deteriorating concrete structures not only affect the productivity of the society but also have a great impact on our resources, environment, and human safety. It has been realized that the deterioration of concrete structures is due to the main emphasis given to mechanical properties and the structural capacity and the neglect of construction quality and life cycle management (ACI 201.2R-08). Strength and durability are two separate aspects of concrete; neither will guarantee the other. Hence, clauses on durability were included for the first time in the fourth revision of IS 456, published in 2000 (see Clause 8 of the code).

As per Clause 8.1 of IS 456, a *durable concrete* is one that performs satisfactorily in the working environment of anticipated exposure conditions during its service life. The following factors affect the durability of concrete: (a) Environment, (b) concrete cover to the embedded steel, (c) quality and type of constituent materials, (d) cement content and w/c ratio of concrete, (e) degree of compaction and curing of concrete, and (f) shape and size of member. The prescriptive requirements given in IS 456 are discussed in Section 4.4.5 of Chapter 4. The requirement of concrete exposed to sulphate attack is provided in Clause 8.2.2.4 and Table 4 of IS 456. Guidance to prevent alkali–aggregate reaction is given in Clause 8.2.5.4 of IS 456.

EXAMPLES

EXAMPLE 1.1 (Mix proportioning for M25 concrete): Calculate the mix proportioning for M25 concrete if the following are the stipulations for proportioning:

1. Grade designation: M25
2. Type of cement: OPC 43 grade conforming to IS 8112
3. Maximum nominal size of aggregate: 20 mm
4. Exposure condition: Moderate
5. Minimum cement content (Table 5 of IS 456): 300 kg/m³
6. Workability: Slump 75 mm
7. Method of concrete placing: Pumping
8. Degree of supervision: Good
9. Type of aggregate: Crushed angular aggregate
10. Maximum cement content: 450 kg/m³
11. Chemical admixture type: Superplasticizer

The test data for materials is as follows:

1. Cement used: OPC 43 grade conforming to IS 8112
2. Specific gravity of cement: 3.15
3. Chemical admixture: Superplasticizer conforming to IS 9103

4. Specific gravity of materials is as follows:
 - (a) Coarse aggregate: 2.68
 - (b) Fine aggregate: 2.65
 - (c) Chemical admixture: 1.145
5. Water absorption is as follows:
 - (a) Coarse aggregate: 0.6 per cent
 - (b) Fine aggregate: 1.0 per cent
6. Free (surface) moisture data is as follows:
 - (a) Coarse aggregate: Nil (absorbed moisture also nil)
 - (b) Fine aggregate: Nil
7. Sieve analysis data is as follows:
 - (a) Coarse aggregate: Conforming to Table 2 of IS 383:1970
 - (b) Fine aggregate: Conforming to grading zone 1 of Table 4 of IS 383:1970

SOLUTION:

Step 1 Calculate the target strength for mix proportioning. From Eq. (1.1)

$$f'_{ck} = f_{ck} + 1.65 \times s$$

From Table 8 of IS 456 (see Table 1.13), standard deviation for M25, $s = 4 \text{ N/mm}^2$

Therefore, target strength = $25 + 1.65 \times 4 = 31.6 \text{ N/mm}^2$

Step 2 Select the w/c ratio. From Table 5 of IS 456 (Table 4.5), maximum water cement ratio for moderate exposure is 0.50. Adopt w/c ratio as $0.45 < 0.50$.

Step 3 Select water content. From Table 2 of IS 10262, Maximum water content = 186 kg (for 25–50 mm slump and for 20 mm aggregate)

Estimated water content for 75 mm slump = $186 + 3/100 \times 186 = 191.58 \text{ kg}$

As superplasticizer is used, the water content can be reduced to more than 20 per cent. Based on trials with superplasticizer, water content reduction of 20 per cent has been achieved. Hence, the assumed water content = $191.58 \times 0.80 = 153.2 \text{ kg}$.

Step 4 Calculate the cement content.

$$\text{w/c ratio} = 0.45$$

Cement content = $153.2/0.45 = 340.4 \text{ kg/m}^3$

From Table 5 of IS 456 (Table 4.5), minimum cement content for moderate exposure condition = 300 kg/m^3 . Since $340.4 \text{ kg/m}^3 > 300 \text{ kg/m}^3$, it is acceptable.

Step 5 Determine the proportion of volume of coarse aggregate and fine aggregate content. From Table 3 of IS 10262 (Table 1.15), volume of coarse aggregate corresponding to 20 mm size aggregate and fine aggregate (Zone 1) for w/c ratio of 0.50 is 0.60. We now have w/c ratio as 0.45. Therefore, the volume of coarse aggregate has to be increased to decrease the fine aggregate content. As the w/c ratio is lower by 0.05,

the proportion of volume of coarse aggregate is increased by 0.01 (at the rate of $-/+0.01$ for every $+0.05$ change in the w/c ratio). Therefore, corrected proportion of volume of coarse aggregate for the w/c ratio of 0.45 is 0.61.

Note: Even if the selected coarse aggregate is not angular, the volume of coarse aggregate has to be increased suitably, based on experience.

For pumpable concrete, these values should be reduced by 10 per cent.

Therefore, volume of coarse aggregate = $0.61 \times 0.09 = 0.55$

Volume of fine aggregate content = $1 - 0.55 = 0.45$

Step 6 Perform the mix calculations. The mix calculations per unit volume of concrete are as follows:

1. Volume of concrete = 1 m^3
2. Volume of cement = Mass of cement/Specific gravity of cement $\times 1/1000$

$$a = 340.4/3.15 \times 1/1000 = 0.108 \text{ m}^3$$

3. Volume of water = Mass of water/Specific gravity of water $\times 1/1000$

$$b = 153.2/1 \times 1/1000 = 0.153 \text{ m}^3$$

4. Volume of chemical admixture (superplasticizer) (at 1.0 per cent by mass of cementitious material)

$$\begin{aligned} c &= \text{Mass of chemical admixtures/Specific gravity of admixture} \times 1/1000 \\ &= 3.4/1.145 \times 1/1000 = 0.00297 \text{ m}^3 \end{aligned}$$

5. Total volume of aggregate (coarse + fine)

$$\begin{aligned} d &= [1 - (a + b + c)] = 1 - (0.108 + 0.153 + 0.00297) \\ &= 0.736 \text{ m}^3 \end{aligned}$$

6. Mass of coarse aggregate = $d \times \text{volume of coarse aggregate} \times \text{specific gravity of coarse aggregate} \times 1000 = 0.736 \times 0.55 \times 2.68 \times 1000 = 1084.86 \text{ kg}$
7. Mass of fine aggregate = $d \times \text{volume of fine aggregate} \times \text{specific gravity of fine aggregate} \times 1000 = 0.736 \times 0.45 \times 2.65 \times 1000 = 877.68 \text{ kg}$

Step 7 Determine the mix proportions for trial number 1.

$$\text{Cement} = 340.40 \text{ kg/m}^3$$

$$\text{Water} = 153.2 \text{ kg/m}^3$$

$$\text{Fine aggregate} = 878 \text{ kg/m}^3$$

$$\text{Coarse aggregate} = 1085 \text{ kg/m}^3$$

$$\text{Chemical admixture} = 3.4 \text{ kg/m}^3$$

$$\text{w/c ratio} = 0.45$$

The following are the adjustments for moisture in aggregates and water absorption of aggregates and the correction for aggregates:

Free (surface) moisture is nil in both fine and coarse aggregates.

$$\text{Corrected water content} = 153.2 + 878 (0.01) + 1085 (0.006) = 168.49 \text{ kg}$$

The estimated batch masses (after corrections) are as follows:

$$\text{Cement} = 340.4 \text{ kg/m}^3$$

$$\text{Water} = 168.5 \text{ kg/m}^3$$

$$\text{Fine aggregate} = 878.0 \text{ kg/m}^3$$

$$\text{Coarse aggregate} = 1085 \text{ kg/m}^3$$

$$\text{Superplasticizer} = 3.4 \text{ kg/m}^3$$

Two more trial mixes with variation of ± 10 per cent of w/c ratio should be carried out to achieve the required slump and dosage of admixtures. A graph between the three w/c ratios and their corresponding strengths should be plotted to correctly determine the mix proportions for the given target strength.

EXAMPLE 1.2 (Mix proportioning for M25 concrete, using fly ash as part replacement of OPC):

Calculate the mix proportioning for M25 concrete with the same stipulations for proportioning and the same test data for materials as given in Example 1.1, except that fly ash is used as part replacement of OPC.

SOLUTION:

Considering the same data as in Example 1.1 for M25 concrete, the mix proportioning steps from 1 to 3 will remain the same.

The procedure of using fly ash as a partial replacement to OPC has been explained in step 4.

Step 4 Calculate the cement content.

$$\text{From Example 1.1, cement content} = 340.4 \text{ kg/m}^3$$

Now, to proportion a mix containing fly ash, the following steps are suggested:

1. Decide percentage of fly ash to be used based on project requirement and quality of materials.
2. In certain situations, increase in cementitious material content may be warranted.

The decision to increase cementitious material content and its percentage may be based on experience and trial. Let us consider an increase of 10 per cent in the cementitious material content.

$$\text{Cementitious material content} = 340.4 \times 1.1 = 374.4 \text{ kg/m}^3$$

$$\text{Water content} = 153.2 \text{ kg/m}^3 \text{ (from Example 1.1)}$$

$$\text{Hence, w/c ratio} = 153.2/374.4 = 0.41$$

$$\text{Fly ash at 35 per cent of total cementitious material content} = 374.4 \times 35\% = 131 \text{ kg/m}^3$$

$$\text{Cement (OPC) content} = 374.4 - 131 = 243.4 \text{ kg/m}^3$$

$$\text{Saving of cement while using fly ash} = 374.4 - 243.4 = 97 \text{ kg/m}^3$$

$$\text{Fly ash being utilized} = 131 \text{ kg/m}^3$$

Step 5 Determine the proportion of volume of coarse aggregate and fine aggregate content. From Table 3 of IS 10262 (Table 1.15), the volume of coarse aggregate corresponding to 20 mm size aggregate and fine aggregate (Zone I) for w/c ratio

of 0.50 is 0.60. In this example, w/c ratio is 0.41. Therefore, the volume of coarse aggregate is required to be increased to decrease the fine aggregate content. As the w/c ratio is lower by approximately 0.10, the proportion of volume of coarse aggregate is increased by 0.02 (at the rate of $-/+0.01$ for every $+0.05$ change in the w/c ratio). Therefore, the corrected proportion of volume of coarse aggregate for the w/c ratio of 0.41 is 0.62.

Note: Even if the selected coarse aggregate is not angular, the volume of coarse aggregate has to be increased suitably, based on experience.

For pumpable concrete, these values should be reduced by 10 per cent.

Therefore, volume of coarse aggregate = $0.62 \times 0.09 = 0.56$
Volume of fine aggregate content = $1 - 0.56 = 0.44$

Step 6 Perform the mix calculations. The mix calculations per unit volume of concrete shall be as follows:

1. Volume of concrete = 1 m^3
2. Volume of cement = Mass of cement/Specific gravity of cement $\times 1/1000$

$$a = 243.4/3.15 \times 1/1000 = 0.0773 \text{ m}^3$$

3. Volume of fly ash = Mass of fly ash/Specific gravity of fly ash $\times 1/1000$

$$b = 131/2.0 \times 1/1000 = 0.0655 \text{ m}^3$$

4. Volume of water = Mass of water/Specific gravity of water $\times 1/1000$

$$c = 153.2/1 \times 1/1000 = 0.153 \text{ m}^3$$

5. Volume of chemical admixture (superplasticizer) (at 0.8 per cent by mass of cementitious material)

$$d = \text{Mass of chemical admixture/Specific gravity of admixture} \times 1/1000 \\ = 3/1.145 \times 1/1000 = 0.0026 \text{ m}^3$$

6. Total volume of aggregate (coarse + fine)

$$e = [1 - (a + b + c + d)] \\ = 1 - (0.0773 + 0.0655 + 0.153 + 0.0026) = 0.7016 \text{ m}^3$$

7. Mass of coarse aggregate

$$= e \times \text{volume of coarse aggregate} \times \text{Specific gravity of coarse aggregate} \times 1000 = 0.7016 \times 0.56 \times 2.68 \times 1000 = 1053 \text{ kg}$$

8. Mass of fine aggregate

$$= e \times \text{volume of fine aggregate} \times \text{specific gravity of fine aggregate} \times 1000 = 0.7016 \times 0.44 \times 2.65 \times 1000 = 818 \text{ kg}$$

Step 7 Determine the mix proportions for trial number 1.

$$\text{Cement} = 243.4 \text{ kg/m}^3$$

$$\text{Fly ash} = 131 \text{ kg/m}^3$$

$$\text{Water} = 153 \text{ kg/m}^3$$

$$\text{Fine aggregate} = 818 \text{ kg/m}^3$$

$$\text{Coarse aggregate} = 1053 \text{ kg/m}^3$$

$$\text{Chemical admixture} = 3 \text{ kg/m}^3$$

$$\text{w/c ratio} = 0.41$$

Note: The aggregate should be used in saturated surface dry condition. As mentioned in Example 1.1, three trial mixes with slightly varying w/cm ratio has to be made to determine experimentally the exact mix proportions that will result in the required workability, strength, and durability.

SUMMARY

Concrete technology has advanced considerably since the discovery of the material by the Romans more than 2000 years ago. A brief history of developments that resulted in the current day RC is provided. The advantages and drawbacks of concrete as a construction material are listed. Cement is the most important ingredient of concrete as it binds all the other ingredients such as fine and coarse aggregates. The cements that are in use today include OPC, rapid hardening Portland cement, low heat Portland cement, sulphate-resisting Portland cement, PSC, PPC, and ternary blended cement. The making and properties of these various types of cements are briefly discussed. The three grades of cement and their properties are also provided. The fine and coarse aggregates occupy about 60–75 per cent of the concrete volume (70–85% by mass) and hence strongly influence the properties of fresh as well as hardened concrete, its mixture proportions, and the economy. Mixing water plays an important role in the workability, strength, and durability of concrete. Hence, their properties and use in concrete are briefly discussed.

As we now use a variety of chemical and mineral admixtures to improve properties of concrete, a brief introduction to them is also provided. It is important to realize the chemical interaction of these

admixtures with the ingredients of cement, as they may ultimately affect the performance of concrete. Proportioning of concrete mixes, as per the latest IS 10262:2009, is described. Hydration of cement and heat of hydration are also described. In addition to the ordinary concrete, we now have a host of different types of concretes, such as RMC, HPC, SCC, SLWC, AAC, FRC, DFRC (which include ECC, UHPC, SIFCON and SIMCON), polymer concrete, and ferrocement. They are used in some situations to achieve strength and durability.

When reinforcing steel (often called *rebar*) is placed inside a concrete mass (they are often placed in the tension zone, as concrete is weak in tension), the solidified mass is called RC. Though traditionally mild steel was used as rebar, a number of different types of rebars are now available and include hot rolled HYSD, hard drawn wire fabric, TMT bars, and TMT CRS bars. The mechanical properties of these steel bars are also provided. A brief description of the corrosion of steel bars, which is mainly responsible for the deterioration of RCC structures all over the world, is also included. Corrosion may be mitigated by the use of fusion-bonded epoxy-coated rebars, galvanized rebars, FRP bars, basalt bars, or TMT CRS bars.

In order to get quality concrete, careful mixing, placing, compacting, and curing of concrete is necessary at site. Forms should be removed only after concrete has gained sufficient strength to carry at least twice the stresses it may be subjected to at the time of removal of forms. Important properties of concrete such as workability of concrete (usually measured by slump test), compressive strength (measured by conducting tests on carefully made and cured cubes or cylinders on the 28th day), stress–strain characteristics, tensile and bearing strength, modulus of elasticity, and Poisson’s ratio are

discussed. Expressions for finding compression strength at any day, modulus of elasticity, and tensile, shear, bond, and bearing strengths, are provided as per Indian codes and compared with the provisions of the US code. Discussions on strength under combined stresses and shrinkage, temperature, and creep effects are also included. Various non-destructive tests performed on concrete to assess the strength of existing structures are also listed. Two examples are provided to explain the mix proportioning of concrete.

REVIEW QUESTIONS

- Write a short history of concrete, beginning with the Roman concrete.
- What are the advantages and drawbacks of concrete?
- Compare the major properties of steel, concrete, and wood.
- What are the processes by which modern cement is made? Explain the dry process of cement manufacture.
- List five different cements that are in use today.
- What are the three different grades of cements used in India? How is the grade of cement fixed?
- How does the fineness of cement affect the concrete?
- What are the four major compounds used in cement? How do they affect the different properties of concrete?
- How is PPC manufactured? What are its advantages?
- How is PSC manufactured? What are its advantages?
- Name any three tests that are conducted on cement.
- What are the different classifications of aggregates? List five factors of aggregates that may affect the properties of concrete.
- The specific gravity of gravel is _____.
(a) 2.80 (c) 2.67
(b) 2.85 (d) 3.10
- The maximum size of coarse aggregate used in concrete is the lesser of _____.
(a) one-fourth the size of member, 5 mm less than max. clear distance between bars, and min. cover
(b) one-fourth the size of member and 20 mm
(c) one-fourth the size of member, 5 mm less than max. clear distance between bars, and 10 mm less than min. cover
- Can sea water be used for mixing or curing of concrete? State the reason.
- Name any three chemical admixtures used in concrete.
- Name any two compounds used as superplasticizers in India.
- Name any three mineral admixtures used in concrete.
- Write short notes on the following:
(a) Fly ash
(b) Silica fume
(c) GGBS
- What are the main objectives of concrete mix proportioning?
- How is target mean compressive strength fixed for mix proportioning?
- How is initial w/c ratio assumed in mix proportioning?
- What is meant by hydration of cement? What is heat of hydration?
- Name any three types of concretes.
- Why is it better to use RMC than site-mixed concrete?
- As per IS 456, which of the following is considered standard concrete (NSC)?
(a) M25–M60, (c) M50–M75,
(b) M30–M50, (d) M20–M40
- As per IS 456, which of the following is considered HSC?
(a) M50–M80, (c) M60–M90,
(b) M65–M100, (d) M50–M90
- How does HPC differ from HSC?
- Write short notes on the following:
(a) HPC (e) DFRCC
(b) SCC (f) SIFCON and SIMCON
(c) FRC (g) Ferrocement
(d) SLWC
- How are TMT bars manufactured? How do they differ from cold twisted deformed bars?
- Draw the stress–strain curve for mild steel bars and HYSD bars.
- As per IS 13920, which of the following should not be used in earthquake zones?
(a) Bars of grade Fe 500 and above
(b) Bars of grade Fe 550 and above
(c) Bars of grade Fe 600 and above
(d) All of these
- When does corrosion of rebars take place? What are the different methods adopted to mitigate corrosion?
- Name three types of rebars that are used in corrosive environments.
- State the three methods by which concrete is compacted.
- What is workability of concrete? Name and describe the test to measure workability.
- How is compressive strength of concrete determined?
- Name any three factors that may affect the compressive strength of concrete.
- How does the stress–strain curve of HSC differ from NSC?
- Write the expressions of modulus of elasticity, tensile, shear, and bearing strength of concrete as per IS 456.
- Write short notes on shrinkage, temperature, and creep effects of concrete.
- Name any three non-destructive tests performed on concrete.

EXERCISES

- Determine the mix proportioning for M30 concrete for the data given in Example 1.1.
- Determine the mix proportioning for M30 concrete for the data given in Example 1.1, with fly ash as part replacement of OPC.

REFERENCES

- ACI 201.2R-08 2008, *Guide to Durable Concrete*, American Concrete Institute, Farmington Hills, p. 49.
- ACI 209R-92 1992, *Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures*, American Concrete Institute, Farmington Hills, p. 47.
- ACI 211.1-91 1991, *Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete*, American Concrete Institute, Farmington Hills, p. 38.
- ACI 211.4R-93 1993, *Guide for Selecting Proportions for High-strength Concrete with Portland Cement and Fly Ash*, American Concrete Institute, Farmington Hills, p. 13.
- ACI 213R-03 2003, 'Guide for Structural Lightweight Aggregate Concrete', in *ACI Manual of Concrete Practice, Part 1: Materials and General Properties of Concrete*, American Concrete Institute, Farmington Hills, p. 38.
- ACI 214R-02 2002, *Evaluation of Strength Test Results of Concrete*, American Concrete Institute, Farmington Hills, p. 20.
- ACI 222R-01 2001, *Protection of Metals in Concrete against Corrosion*, American Concrete Institute, Farmington Hills, p. 41.
- ACI 228.1R-03 2003, *In-Place Methods to Estimate Concrete Strength*, American Concrete Institute, Farmington Hills, p. 44.
- ACI 301M:10 2010, *Specifications for Structural Concrete*, American Concrete Institute, Farmington Hills, p. 75.
- ACI 305R:10 2010, *Guide to Hot Weather Concreting*, American Concrete Institute, Farmington Hills, p. 23.
- ACI 309R:05 2005, *Guide for Consolidation of Concrete*, American Concrete Institute, Farmington Hills, p. 39.
- ACI 347-04, *Guide to Formwork for Concrete*, American Concrete Institute, Farmington Hills, p. 32.
- ACI 363R-10 2010, *State-of-the-art Report on High-strength Concrete*, American Concrete Institute, Farmington Hills, p. 65.
- ACI 440R-07 2007, *Report on Fiber-reinforced Polymer (FRP) Reinforcement in Concrete Structures*, American Concrete Institute, Farmington Hills, p. 100.
- ACI 544.1R-96 1996, *Report on Fiber-reinforced Concrete*, American Concrete Institute, Farmington Hills, p. 64.
- ACI 549.1R-93 1993 (reapproved 1999), *Guide for the Design, Construction, and Repair of Ferrocement*, American Concrete Institute, Farmington Hills, p. 30.
- ACI Committee E-701 2007, *Aggregates for Concrete*, American Concrete Institute, Farmington Hills, p. 29.
- Aïtcin, P.-C. 1998, *High Performance Concrete*, CRC Press, Boca Raton, FL, p. 624.
- Aïtcin, P.-C., C. Jolicouer, and J.G. MacGregor 1994, 'Superplasticizers: How They Work and Why They Occasionally Don't', *Concrete International, ACI*, Vol. 16, No. 5, pp. 45–52.
- Aïtcin, P.-C. and A. Neville 1993, 'High-performance Concrete Demystified', *Concrete International, ACI*, Vol. 15, No. 1, pp. 21–6.
- Alimchandani, C.R. 2007, 'The Indian Experience of Ready Mixed Concrete', *32nd Conference on Our World in Concrete & Structures*, 28–29 August 2007, Singapore, (also see http://www.cipremier.com/e107_files/downloads/Papers/100/32/100032002.pdf, last accessed on 12 October 2012).
- Ambuja Technical Booklets: No. 5, *Aggregates for Mortar and Concrete*, May 1996, p. 11, No. 125, *FAQs on Aggregates*, May 2007, p. 20, Ambuja Cements Ltd, Mumbai.
- Bamforth, P., D. Chisholm, J. Gibbs, and T. Harrison 2008, *Properties of Concrete for Use in Eurocode 2*, The Concrete Centre, Surrey, p. 53.
- Bapat, J.D. 2012, *Mineral Admixtures in Cement and Concrete*, CRC Press, Boca Raton, p. 310.
- Basu, P.C., P. Shylamoni, and A.D. Roshan 2004, 'Characterisation of Steel Reinforcement for RC Structures: An Overview and Related Issues', *The Indian Concrete Journal*, Vol. 78, No. 1, pp. 19–30.
- Bentur, A. and Mindess, S. 2007, *Fibre Reinforced Cementitious Composites*, 2nd edition, Taylor and Francis, Oxon, p. 624.
- Bentz, D.P. and P.-C. Aïtcin 2008, 'Meaning of Water–Cement ratio: Distance between Cement Particles is Fundamental', *Concrete International, ACI*, Vol. 30, No. 5, pp. 51–4.
- Bentz, D.P., P. Lura, and J.W. Roberts 2005, 'Mixture Proportioning for Internal Curing', *Concrete International, ACI*, Vol. 27, No. 2, pp. 35–40.
- Blais, P.Y. and M. Couture 1999, 'Precast, Prestressed Pedestrian Bridge: World's First Reactive Powder Concrete Structure', *PCI Journal*, pp. 60–71.
- Chandra, S. and L. Berntsson 2002, *Lightweight Aggregate Concrete: Science, Technology, and Applications*, William Andrew Publishing, Norwich, p. 430.
- Clarke, J.L. (ed.) 1993, *Structural Lightweight Aggregate Concrete*, Blackie Academic and Professional, London, p. 148.
- de Larrard, F. 1999, *Concrete Mixture Proportioning: A Scientific Approach*, CRC Press, Boca Raton, p. 448.
- Dhir, R.K. and K.A. Paine 2010, 'Value Added Sustainable Use of Recycled and Secondary Aggregates in Concrete', *The Indian Concrete Journal*, Vol. 84, No. 3, pp. 7–26.
- Dogan, E. and N. Krstulovic-Opara 2003, 'Seismic Retrofit with Continuous Slurry-infiltrated Mat Concrete Jackets', *ACI Structural Journal*, Vol. 100, No.6, pp.713–22.
- EFNARC-2005, *The European Guidelines for Self-Compacting Concrete: Specification, Production and Use*, (also see <http://www.efnarc.org/pdf/SCCGuidelinesMay2005.pdf>, last accessed on 11 October 2012).
- Fehling, E., M. Schmidt, and S. Stürwald, (eds) 2008, *Proceedings of the Second International Symposium on Ultra High Performance Concrete*, University of Kassel, Germany, 5–7 March 2008, Kassel University Press, p. 920, (also see <http://www.uni-kassel.de/upress/online/frei/978-3-89958-376-2.volltext.frei.pdf>, last accessed on 12 January 2013).
- fib 2007, *Fire Design of Concrete Structures: Materials, Structures and Modeling*, State-of-the-art Report, Bulletin 38, fédération internationale du béton (fib), Lausanne, p. 98.
- fib 2008, *Fire Design of Concrete Structure: Structural Behaviour and Assessment*, State-of-the-art Report, Bulletin 46, fédération internationale du béton (fib), Lausanne, p. 209.
- Fischer, G. and V.C. Li 2002, 'Influence of Matrix Ductility on the Tension-stiffening Behavior of Steel Reinforced ECC', *ACI Structural Journal*, Vol. 99, No. 1, pp. 104–11.

- Gambhir, M.L. 2004, *Concrete Technology*, Tata McGraw-Hill Education, New Delhi, p. 658.
- GangaRao, H.V.S., N. Taly, and P.N. Vijay 2007, *Reinforced Concrete Design with FRP Composites*, CRC Press, Boca Raton, p. 382.
- Graybeal, B. and M. Davis 2008, 'Cylinder or Cube: Strength Testing of 80 to 200MPa (11.6 to 29 ksi) Ultra-high-performance Fiber-reinforced Concrete', *ACI Materials Journal*, Vol. 105, No. 6, pp. 603–9.
- Hackman, L.E., M.B. Farrell, and O.O. Dunham 1992, 'Slurry Infiltrated Mat Concrete (SIMCON)', *Concrete International, ACI*, Vol. 14, No. 12, pp. 53–6.
- Holt E.E. 2001, *Early Age Autogenous Shrinkage of Concrete*, Technical Research Center of Finland, ESPOO, VTT Publication No. 446, p. 197, (also see <http://vtt.fi/inf/pdf/publications/2001/P446.pdf>, last accessed on 12 September 2012).
- Hurd, M.K. 2005, *Formwork for Concrete*, SP-4, 7th edition, American Concrete Institute, Farmington Hills, p. 500.
- IS 269:2013, *Specifications for 33 Grade Portland cement*, Bureau of Indian Standards, New Delhi.
- IS 383:1970 (reaffirmed 1997), *Specifications for Coarse and Fine Aggregates from Natural Sources for Concrete*, Bureau of Indian Standards, New Delhi.
- IS 432 (Part 1):1982 (reaffirmed 1995), *Specification for Mild Steel and Medium Tensile Steel Bars and Hard-drawn Steel Wire for Concrete Reinforcement, Part 1 Mild Steel and Medium Tensile Steel Bars*, 3rd revision, Bureau of Indian Standards, New Delhi.
- IS 455:1989 (reaffirmed 2000), *Specification for Portland Slag Cement*, Bureau of Indian Standards, New Delhi.
- IS 516:1959 (reaffirmed 1999), *Methods of Tests for Strength of Concrete*, Bureau of Indian Standards, New Delhi.
- IS: 1489 (Part-1):1991(reaffirmed 2000), *Specification for Portland Pozzolana Cement, Part 1: Fly Ash Based*, Bureau of Indian Standards, New Delhi.
- IS: 1489 (Part-2):1991(reaffirmed 2000), *Specification for Portland Pozzolana Cement, Part 2: Calcined Clay Based*, Bureau of Indian Standards, New Delhi.
- IS 1786:2008, *Specification for High Strength Deformed Steel Bars and Wires for Concrete Reinforcement*, 4th revision, Bureau of Indian Standards, New Delhi.
- IS: 3812:2003, *Specification for Pulverized Fuel Ash, Part 1: For Use as Pozzolana in Cement, Cement Mortar and Concrete, Part 2: For Use as Admixture in Cement Mortar and Concrete*, Bureau of Indian Standards, New Delhi.
- IS 4031 (Parts 1–15):1988 (reconfirmed 2000) *Methods of Physical Tests for Hydraulic Cement, Part 6: Determination of Compressive Strength of Hydraulic Cement other than Masonry Cement*, Bureau of Indian standards, New Delhi, p. 3.
- IS 5816:1999, *Methods of Tests for Splitting Tensile Strength of Concrete Cylinders*, 1st revision, Bureau of Indian Standards, New Delhi.
- IS: 8112:2013, *Specifications for 43 Grade Portland Cement*, 1st revision, Bureau of Indian Standards, New Delhi.
- IS 9103:1999, *Specifications for Admixtures for Concrete*, 1st revision, Bureau of Indian Standards, New Delhi.
- IS 10262:2009, *Concrete Mix Proportioning: Guidelines*, Bureau of Indian Standards, New Delhi.
- IS 12269:2013, *Specification for 53 Grade Ordinary Portland Cement*, Bureau of Indian Standards, New Delhi.
- IS 12089:1987 (reconfirmed 1999), *Specification for Granulated Slag for Manufacture of Portland Slag Cement*, Bureau of Indian Standards, New Delhi.
- IS 12594:1988 (reaffirmed 2011), *Hot Dip Coatings on Structural Steel Bars for Concrete Reinforcement—Specifications*, Bureau of Indian Standards, New Delhi.
- IS 13620:1993 (reaffirmed 2009), *Fusion Bonded Epoxy Coated Reinforcing Bars—Specification*, Bureau of Indian Standards, New Delhi.
- IS 15388:2003, *Silica Fume-Specification*, Bureau of Indian Standards, New Delhi.
- Indian Concrete Journal (ICJ), Special Issues on Self-compacting Concrete*, July 2004, June 2009, and August 2009.
- Jayasree, C., M. Santhanam, and R. Gettu 2011, 'Cement–Superplasticiser Compatibility Issues and Challenges', *The Indian Concrete Journal*, Vol. 85, No.7, pp. 48–60.
- Johansen, V.C., W.A. Klemm, and P.C. Taylor 2002, 'Why Chemistry Matters in Concrete?', *Concrete International, ACI*, Vol. 24, No. 3, pp. 84–9.
- Kerkhoff, B. 2007, *Effects of Substances on Concrete and Guide to Protective Treatments*, Portland Cement Association, p. 36.
- Kosmatka, S.H., B. Kerkhoff, and W.C. Panarese 2011, *Design and Control of Concrete Mixtures*, 15th edition, Portland Concrete Association, Skokie, p. 444.
- Krishnamurthy, S. (ed.) 1997, *ICI Monograph on Concrete Admixtures*, Indian Concrete Institute, Chennai, p. 35.
- Krishna Raju, N. 2002, *Design of Concrete Mixes*, 4th edition, CBS Publishers and Distributors Pvt. Ltd, New Delhi, p. 316.
- Kupfer, H., H.K. Hilsdorf, and H. Rüschi 1969, 'Behavior of Concrete under Biaxial Stresses', *Journal of ACI*, Vol. 66, No. 8, pp. 656–66.
- Kulkarni, V.R. 2012, *E-conference on Durability of Concrete*, 27 February to 11 March, 2012, (also see <http://www.sefindia.org/forum/viewtopic.php?t=11706&view=next>, last accessed on 12 January 2013).
- Kumar P., and S.K. Kaushik 2003, 'Some Trends in the Use of Concrete: Indian Scenario', *The Indian Concrete Journal*, Vol. 77, No. 12, pp. 1503–8.
- Kumar, S. and M. Santhanam 2004, 'Use of Particle Packing Model to Produce HPC at Optimum Cement Content', *The Indian Concrete Journal*, Vol. 78, No. 12, pp. 22–8.
- Lea, F.M., 1971 *The Chemistry of Cement and Concrete*, 3rd edition, Chemical Publishing Co., Inc., New York, p. 740.
- Hwang, S-D, K.H. Khayat, and O. Bonneau 2006, 'Performance-based Specifications of Self-consolidating Concrete used in Structural Applications', *ACI Materials Journal*, Vol. 103, No. 2, pp. 121–9.
- Lafarge-Ductal®, *Executive Summary: Completed North American Ductal® Bridges*, http://www.fhwa.dot.gov/hfl/innovations/uhpc_bridges_in_north_america.cfm, last accessed on 12 January 2013.
- Lankard, D.R. 1984, 'Properties and Applications of Slurry Infiltrated Fiber Concrete (SIFCON)', *Concrete International, ACI*, Vol. 6, No. 12, pp. 44–7.
- Li, V.C. 2003, 'On Engineered Cementitious Composites (ECC): A Review of the Material and its Applications', *Journal of Advanced Concrete Technology*, Vol. 1, No. 3, pp. 215–30.
- Li, V.C., D.K. Mishra, A.E. Naaman, J.K. Wight, J.M. LaFave, H.C. Wu, and Y. Inada 1994, 'On the Shear Behavior of ECC', *Journal of Advanced Cement Based Materials*, Vol. 1, No. 3, pp. 142–9.

- Malhotra, V.M. and N.J. Carino 2003, *Handbook on Nondestructive Testing of Concrete*, 2nd edition, CRC Press, Boca Raton, p. 384.
- Matsumoto, T. and H. Mihashi 2003, 'JCI-DFRCC Committee Report: DFRCC Terminology and Application Concepts', *Journal of Advanced Concrete Technology*, Vol. 1, No. 3, pp. 335–40.
- Mehta, P.K. 1987, 'Natural Pozzolans: Supplementary Cementing Materials in Concrete', *CANMET Special Publication*, Vol. 86, pp. 1–33.
- Mehta, P.K. and P.J.M. Monteiro 2006, *Concrete: Microstructure, Properties and Materials*, 3rd edition, McGraw-Hill, New York, p. 659 (also contains a CD-ROM with numerous PowerPoint slides, videos, and additional material).
- Mindess, S., J.F. Young, and D. Darwin 2003, *Concrete*, 2nd edition, Prentice Hall and Pearson Education, Harlow, p. 644.
- Moir, G. 2003, *Cements in Advanced Concrete Technology*, edited by J. Newman and B.S. Choo, Elsevier Butterworth-Heinemann, Amsterdam, p. 1–45.
- Mullick, A.K. 2005, 'High Performance Concrete in India: Development, Practices and Standardisation', *ICI Journal*, Vol. 5, No. 2, pp. 7–14.
- Mullick, A.K. 2007, 'Performance of Concrete with Binary and Ternary Cement Blends', *Indian Concrete Journal*, Vol. 81, No. 1, pp. 15–22.
- Mullick, A.K. 2008, 'Cement–Superplasticiser Compatibility and Method of Evaluation', *The Indian Concrete Journal*, Vol. 82, No. 6, pp. 8–15.
- Mullick, A.K. 2012, 'Green Options for Binder System and Aggregate in Sustainable Concrete', *Indian Concrete Journal*, Vol. 86, No. 6, pp. 9–17.
- Muthukumar, D. and N. Subramanian 1999, 'Characteristics of High Strength Concrete with Respect to Ageing', *Civil Engineering & Construction Review (CE & CR)*, Vol. 12, No. 2, pp. 14–7.
- Naaman, A.E. 2000, *Ferrocement and Laminated Cementitious Composites*, Techno Press 3000, Ann Arbor, p. 372.
- Naaman, A.E., D. Otter, and H. Najm 1992, 'Elastic Modulus of SIFCON in Tension and Compression', *ACI Materials Journal*, Vol. 88, No. 6, pp. 603–13.
- Nagendra, R. 2010, 'Concrete Mix Proportioning as per New Revised IS 10262:2009', *The Indian Concrete Journal*, Vol. 84, No. 9, pp. 9–21.
- Nataraja, M.C. and L. Das 2010, 'Concrete Mix Proportioning as per Draft IS 10262:2009: Comparison with IS 10262:2009 and ACI 211.1-91', *The Indian Concrete Journal*, Vol. 84, No. 9, pp. 64–70.
- Nataraja, M.C., Nagaraj, T.S. and Ashok Reddy 2001, 'Proportioning Concrete Mixes with Quarry Waste', *International Journal of Cement, Concrete and Aggregates*, ASTM, Vol. 23, No. 2, pp. 1–7.
- Nataraja, M.C. and B.M.R. Reddy 2007, 'Mix proportioning of plain and rice husk ash concrete as per draft IS 10262', *The Indian Concrete Journal*, Vol. 81, No. 3, pp. 36–42, and Discussions by M.P. Rajamane, Vol. 81, No. 5, pp. 44–6, 55–56.
- Nematollahi, B., M.R.R. Saifulnaz, M.S. Jaafar, and Y.L. Voo 2012, 'A Review on Ultra High Performance "ductile" Concrete (UHPdC) Technology', *International Journal of Civil and Structural Engineering*, Vol. 2, No. 3, pp. 1003–18.
- Neville, A.M. 2012, *Properties of Concrete*, 5th edition, Prentice Hall Harlow, p. 872.
- Neville, A.M. and J.J. Brooks 2010, *Concrete Technology*, 2nd edition, Prentice Hall & Pearson Education, Harlow, p. 442.
- Niyogi, S.M. 1974, 'Concrete Bearing Strength: Support, Mix, Size Effect', *Journal of the Structural Division*, ASCE, Vol. 100, No. 8, pp. 1685–1702.
- Noguchi, T., F. Tomosawa, K.M. Nemati, B.M. Chiaia, and A.P. Fantilli 2009, 'A Practical Equation for Elastic Modulus of Concrete', *ACI Structural Journal*, Vol. 106, No. 5, pp. 690–6.
- Okamura, H. and M. Ouchi 2003, 'Self-compacting Concrete', *Journal of Advanced Concrete Technology*, Vol. 1, No. 1, pp. 5–15.
- Ouchi, M., S.A. Nakamura, T. Osterberg, S-E. Hallberg, M. Lwin 2003, *Applications of Self-compacting Concrete in Japan, Europe, and the United States*, ISHPC, p. 20, (also see <http://www.fhwa.dot.gov/bridge/scc.pdf>, last accessed on 5 October 2012).
- Panarese, W.C. 1987, *Transporting and Handling Concrete*, IS178, Portland Cement Association.
- Parameswaran, V.S. 1996, *Behaviour and Applications of Slurry Infiltrated Fibrous Concrete (SIFCON)*, Technical Manual: ICFRC-TM 2, International Centre for Fibre Reinforced Concrete Composites, Madras, p. 13.
- Parameswaran, V.S. and K. Balasubramanian 1993, *Fibre Reinforced Concrete (Theory, Properties and Applications)*, Technical Manual: ICFRC-TM1, International Centre for Fibre Reinforced Concrete Composites, Madras, p. 45.
- Parameswaran, V.S., T.S. Krishnamoorthy, and K. Balasubramanian 1990, 'Behaviour of High Volume Fibre Cement Mortar in Flexure', *Cement and Concrete Composites*, Vol. 12, No. 4, pp. 293–301.
- Persson, B. and G. Fagerlund, (eds) 2002, 'Self-desiccation and its Importance in Concrete Technology', *Proceedings of the Third International Research Seminar in Lund*, 14–15 June 2002, (also see <http://lup.lub.lu.se/luur/download?func=downloadFile&recordId=633777&fileId=634557>, last accessed on 12 September 2012).
- Persson, B., D. Bentz, and L.-O. Nilsson 2005, 'Self-desiccation and its Importance in Concrete Technology', *Proceedings of the Fourth International Research Seminar*, Gaithersburg, (also see <http://ciks.cbt.nist.gov/~bentz/Lund2005/TVBM-3126hp1.pdf>, last accessed on 12 September 2012).
- Pillai, S.U. and Menon, D. 2003, *Reinforced Concrete Design*, 2nd edition, Tata McGraw Hill Publishing Company Ltd, New Delhi, p. 875.
- Popovics, S. 1998, *Strength and Related Properties of Concrete*, John Wiley, New York, p. 552.
- Powers, T.C. 1961, *Some Physical Aspects of the Hydration of Portland Cement*, Research Department Bulletin RX125, Portland Cement Association.
- Rajamane, N.P. 2004, 'A Procedure for Concrete Mix Design Using Provisions of IS 456:2000', *New Building Materials and Construction World*, Vol. 10, No. 12, pp. 74–89.
- Rajamane, N.P. 2005, 'A General Formula to Replace Concrete BIS Mix Design Curves of SP:23 (S&T):1981', *New Building Materials and Construction World*, Vol. 10, No. 11, pp. 124–9.
- Ramachandran, V.S. (ed.) 1995, *Concrete Admixtures Handbook: Properties, Science and Technology*, 2nd edition, Noyes Publications, Park Ridge, Indian edition by Standard Publishers, New Delhi, 2002, p. 1183.
- Rao, M.C., S.K. Bhattacharyya, and S.V. Barai 2011, 'Recycled Course Aggregate and Its Use in Concrete', *ICI Journal*, Vol. 11, No. 4, pp. 27–40.
- Reed, P., K. Schoonees, and J. Salmond 2008, *Historic Concrete Structures in New Zealand: Overview, Maintenance and Management*, Science & Technical Publishing, Wellington, p. 90, (also see <http://www.doc.govt.nz/upload/documents/science-and-technical/sap248.pdf>, last accessed on 12 October 2012).

- Rixom, R. and N.P. Mailvaganam 1999, 'Chemical Admixtures for Concrete', 3rd edition, CRC Press, Boca Raton, p. 456.
- Rossi, P. 2001, 'Ultra-high-performance Fiber-reinforced Concretes', *Concrete International, ACI*, Vol. 23, No. 12, pp. 46–52.
- SP 23:1982, *Handbook on Concrete Mixes*, Bureau of Indian Standards, New Delhi, p. 122.
- Santhakumar, A.R. 2006, *Concrete Technology*, Oxford University Press, New Delhi, p. 784.
- Sashidhar, C., H. Sudarsana Rao, N.V. Ramana and K. Ganeswar 2010, 'Compression and Tension Behaviour of SIFCON Produced with Low Tensile Strength Steel Fibre', *Indian Concrete Journal*, Vol. 84, No. 10, pp. 31–36, and 'Flexural Behaviour of SIFCON Produced with Low Tensile Strength Steel Fibre', Vol. 85, No. 10, October 2011.
- Schmidt, M., E. Fehling, C. Geisenhanslüke (eds) 2004, 'Ultra High Performance Concrete (UHPC)', *Proceedings of the International Symposium on UHPC*, University of Kassel, Germany, 13–15 September 2004, Kassel University Press, p. 868, (also see <http://www.uni-kassel.de/upress/online/frei/978-3-89958-086-0.volltext.frei.pdf>, last accessed on 12 January 2013).
- Schmidt, M., E. Fehling, C. Glotzbach, S. Fröhlich, and S. Piotrowski (eds) 2012, *Proceedings of the 3rd International Symposium on UHPC and Nanotechnology for High Performance Construction Materials*, University of Kassel, Germany, 7–9 March 2012, Kassel University Press, p. 1058, (also see <http://www.uni-kassel.de/upress/online/frei/978-3-86219-264-9.volltext.frei.pdf>, last accessed on 12 January 2013).
- Shaeffer, R.E. 1992, *Reinforced Concrete: Preliminary Design for Architects and Builders*, McGraw-Hill, p. 196.
- Shetty, M.S. 2005, *Concrete Technology: Theory and Practice*, 6th revised edition, S. Chand and Company Ltd, New Delhi, p. 624.
- Smith, J.L. and Y.P. Virmani 1996, *Performance of Epoxy-coated Rebars in Bridge Decks*, Report No. FHWA-RD-96-092, Federal Highway Administration.
- SP 34 (S & T):1987, *Handbook on Concrete Reinforcement and Detailing*, Bureau of Indian Standards, New Delhi 1987.
- Spanos, A., R.S. Ravindrarajah, and R.N. Swamy 2012, 'Material and Structural Implications of Using Ferrocement as Permanent Formwork', *The Indian Concrete Journal*, Vol. 86, No. 8, pp. 9–23.
- Subramanian, N. 1976a, 'Ferrocement: A Review', *Science & Engineering*, Vol. XXIX, No. 9, pp. 121–5.
- Subramanian, N. 1976b, 'Fibre Reinforced Concrete', *Science & Engineering*, Vol. XIX, No. 6, pp. 82–7.
- Subramanian, N. 1999, 'Reactive Powder Concrete: A Material of the Future?', *Civil Engineering & Construction Review*, Vol. 12, No. 2, pp. 33–6.
- Subramanian, N. 2007, *Space Structures: Principles and Practice*, 2 volumes, Multi-Science Publishing Co., Essex, p. 820.
- Subramanian, N. 2010, 'Sustainability of RCC Structures using Basalt Composite Rebars', *The Master Builder*, Vol. 12, No. 9, pp. 156–64.
- Subramanian, N. 2012. 'The Principles of Sustainable Building Design', in G.M. Sabnis (ed.), *Green Building with Concrete: Sustainable Design and Construction*, CRC Press, Boca Raton, p. 333.
- Subramanian, N. and T. Gnana Sambanthan 1979, 'Polymers to Improve the Properties of Concrete', *Proceedings, All India Seminar on Prestressed Concrete Structures*, Thiagarajar College of Engineering, Madurai, 2–3 May, pp. 7–11.
- Suprenant, B.A. 2001, 'Free Fall of Concrete', *Concrete International, ACI*, Vol. 23, No. 6, pp. 44–5.
- Taylor, H.F.W. 1997, *Cement Chemistry*, Thomas Telford Publishing, London, p. 477.
- Tennis, P.D. and H.M. Jennings 2000, 'A Model for Two Types of Calcium Silicate Hydrate in the Microstructure of Portland Cement Pastes', *Cement and Concrete Research*, Vol. 30, No.6, pp. 855–63.
- Venugopal, M.S. and Subramanian, N. 1977, 'Hot Weather Concreting', *Indian Concrete Journal*, Vol. 51, No. 11, pp. 348–9.
- Viswanatha, C.S., L.N. Prasad, Radhakrishna, and H.S. Nagaraj 2004, 'Sub-standard Rebars in the Indian Market: An Insight', *The Indian Concrete Journal*, Vol. 78, No. 1, pp. 52–5, (also see http://www.mmf.com/wp-content/uploads/2012/06/MMF_Product_Information2011.pdf, last accessed on 12 January 2013).
- Zia, P., M.L. Leming, and S.H. Ahmad 1991, *High-performance Concretes: A State-of-the-art Report*, SHRP-C/FR-91-103 and FHWA-RD-97-030, Strategic Highway Research Program, National Research Council, Washington, DC, p. 236, (also see <http://www.fhwa.dot.gov/publications/research/infrastructure/structures/hpc/97030/index.cfm#toc>, last accessed on 5 October 2012).
- Zia, P., M.L. Leming, S.H. Ahmad, J.J. Schemmel, R.P. Elliott, and A.E. Naaman 1993. *Mechanical Behavior of High-Performance Concretes, Volume 1: Summary Report*. SHRP-C-361, Strategic Highway Research Program, National Research Council, Washington, DC, p. 98, (also see <http://onlinepubs.trb.org/onlinepubs/shrp/SHRP-C-361.pdf>, last accessed 5 October 2012).