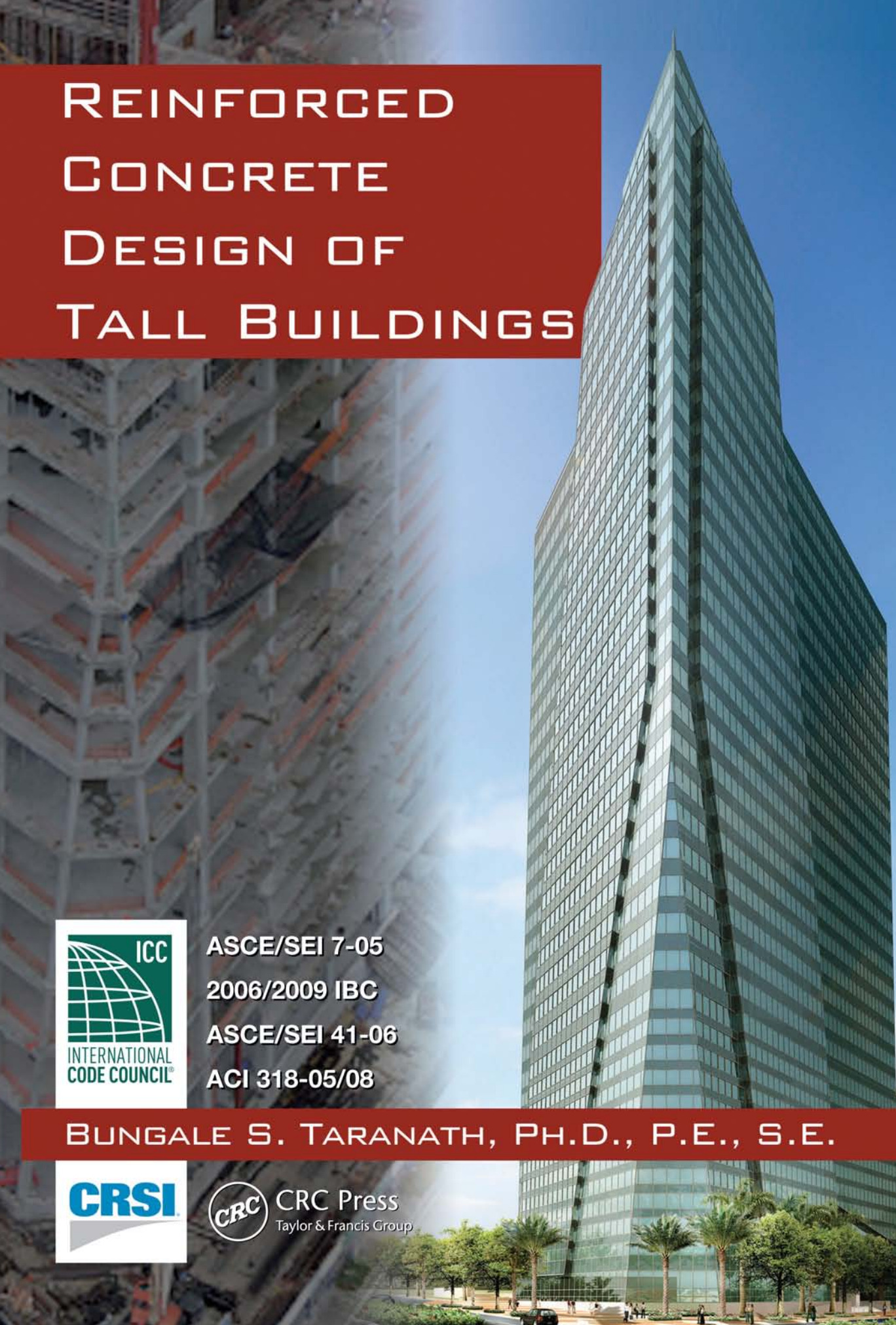
Reinforced Concrete Design of Tall Buildings



ASCE/SEI 7-05

2006/2009 IBC

ASCE/SEI 41-06

ACI 318-05/08

Bungale S. Taranath, Ph.D., P.E., S.E.

CRC Press
Taylor & Francis Group

Reinforced Concrete Design of Tall Buildings

Reinforced Concrete Design of Tall Buildings

Bungale S. Taranath, Ph.D., P.E., S.E.

CRC Press
Taylor & Francis Group
Boca Raton London New York

CRC Press is an imprint of the
Taylor & Francis Group, an **informa** business

CRC Press
Taylor & Francis Group
6000 Broken Sound Parkway NW, Suite 300
Boca Raton, FL 33487-2742

Printed in the United States of America on acid-free paper
10 9 8 7 6 5 4 3 2 1

International Standard Book Number: 978-1-4398-0480-3 (Hardback)

Library of Congress Cataloging-in-Publication Data

Taranath, Bungale S.
Reinforced concrete design of tall buildings / by Bungale S. Taranath.
p. cm.
Includes bibliographical references and index.
ISBN 978-1-4398-0480-3 (alk. paper)
1. Reinforced concrete construction. 2. Tall buildings--Design and construction. 3. Tall buildings--Design and construction--Case studies. I. Title.

TH1501.T37 2010
691'.3--dc22 2009024350

Visit the Taylor & Francis Web site at
http://www.taylorandfrancis.com

and the CRC Press Web site at
http://www.crcpress.com

This book is dedicated to my wife

SAROJA

Without whose patience and devotion, this book would not be.

Contents

List of Figures xxi
List of Tables xlvii
Foreword li
ICC Foreword lv
Preface lvii
Acknowledgments lxi
A Special Acknowledgment lxiii
Author lxv

Chapter 1 Design Concept 1

1.1 Characteristics of Reinforced Concrete 1
1.1.1 Confined Concrete 1
1.1.2 Ductility 4
1.1.3 Hysteresis 5
1.1.4 Redundancy 6
1.1.5 Detailing 6
1.2 Behavior of Reinforced Concrete Elements 7
1.2.1 Tension 7
1.2.2 Compression 7
1.2.3 Bending 8
1.2.3.1 Thumb Rules for Beam Design 8
1.2.4 Shear 14
1.2.5 Sliding Shear (Shear Friction) 18
1.2.6 Punching Shear 21
1.2.7 Torsion 22
1.2.7.1 Elemental Torsion 22
1.2.7.2 Overall Building Torsion 25
1.3 External Loads 26
1.3.1 Earthquakes Loads 26
1.3.2 Wind Loads 27
1.3.2.1 Extreme Wind Conditions 29
1.3.3 Explosion Effects 31
1.3.4 Floods 32
1.3.5 Vehicle Impact Loads 32
1.4 Lateral Load-Resisting Systems 32
1.4.1 Shear Walls 33
1.4.2 Coupled Shear Walls 36
1.4.3 Moment-Resistant Frames 37
1.4.4 Dual Systems 38
1.4.5 Diaphragm 38
1.4.6 Strength and Serviceability 39
1.4.7 Self-Straining Forces 40
1.4.8 Abnormal Loads 40

1.5 Collapse Patterns 40
1.5.1 Earthquake Collapse Patterns 41
1.5.1.1 Unintended Addition of Stiffness 41
1.5.1.2 Inadequate Beam–Column Joint Strength 42
1.5.1.3 Tension/Compression Failures 42
1.5.1.4 Wall-to-Roof Interconnection Failure 43
1.5.1.5 Local Column Failure 43
1.5.1.6 Heavy Floor Collapse 44
1.5.1.7 Torsion Effects 44
1.5.1.8 Soft First-Story Collapse 45
1.5.1.9 Midstory Collapse 45
1.5.1.10 Pounding 45
1.5.1.11 P-Δ Effect 45
1.5.2 Collapse due to Wind Storms 47
1.5.3 Explosion Effects 47
1.5.4 Progressive Collapse 47
1.5.4.1 Design Alternatives for Reducing Progressive Collapse 49
1.5.4.2 Guidelines for Achieving Structural Integrity 49
1.5.5 Blast Protection of Buildings: The New SEI Standard 50
1.6 Buckling of a Tall Building under Its Own Weight 50
1.6.1 Circular Building 51
1.6.1.1 Building Characteristics 52
1.6.2 Rectangular Building 53
1.6.2.1 Building Characteristics 53
1.6.3 Comments on Stability Analysis 53

Chapter 2 Gravity Systems 55

2.1 Formwork Considerations 55
2.1.1 Design Repetition 58
2.1.2 Dimensional Standards 58
2.1.3 Dimensional Consistency 59
2.1.4 Horizontal Design Techniques 60
2.1.5 Vertical Design Strategy 63
2.2 Floor Systems 65
2.2.1 Flat Plates 65
2.2.2 Flat Slabs 65
2.2.2.1 Column Capitals and Drop Panels 66
2.2.2.2 Comments on Two-Way Slab Systems 67
2.2.3 Waffle Systems 67
2.2.4 One-Way Concrete Ribbed Slabs 67
2.2.5 Skip Joist System 67
2.2.6 Band Beam System 68
2.2.7 Haunch Girder and Joist System 70
2.2.8 Beam and Slab System 73
2.3 Design Methods 73
2.3.1 One-Way and Two-Way Slab Subassemblies 73
2.3.2 Direct Design Method for Two-Way Systems 74
2.3.3 Equivalent Frame Method 75

2.3.4 Yield-Line Method 77
2.3.4.1 Design Example: One-Way Simply Supported Slab 78
2.3.4.2 Yield-Line Analysis of a Simply Supported Square Slab 81
2.3.4.3 Skewed Yield Lines 82
2.3.4.4 Limitations of Yield-Line Method 83
2.3.5 Deep Beams 83
2.3.6 Strut-and-Tie Method 85
2.4 One-Way Slab, T-Beams, and Two-Way Slabs: Hand Calculations 92
2.4.1 One-Way Slab; Analysis by ACI 318-05 Provisions 92
2.4.2 T-Beam Design 97
2.4.2.1 Design for Flexure 97
2.4.2.2 Design for Shear 100
2.4.3 Two-Way Slabs 103
2.4.3.1 Two-Way Slab Design Example 106
2.5 Prestressed Concrete Systems 108
2.5.1 Prestressing Methods 111
2.5.2 Materials 111
2.5.2.1 Posttensioning Steel 111
2.5.2.2 Concrete 112
2.5.3 PT Design 113
2.5.3.1 Gravity Systems 113
2.5.3.2 Design Thumb Rules 115
2.5.3.3 Building Examples 118
2.5.4 Cracking Problems in Posttensioned Floors 120
2.5.5 Cutting of Prestressed Tendons 121
2.5.6 Concept of Secondary Moments 123
2.5.6.1 Secondary Moment Design Examples 124
2.5.7 Strength Design for Flexure 133
2.5.7.1 Strength Design Examples 134
2.5.8 Economics of Posttensioning 142
2.5.9 Posttensioned Floor Systems in High-Rise Buildings 143
2.5.9.1 Transfer Girder Example 144
2.5.10 Preliminary Design of PT Floor Systems; Hand Calculations 146
2.5.10.1 Preview 146
2.5.10.2 Simple Span Beam 149
2.5.10.3 Continuous Spans 152
2.5.11 Typical Posttensioning Details 172
2.6 Foundations 172
2.6.1 Pile Foundations 178
2.6.2 Mat Foundations 179
2.6.2.1 General Considerations 179
2.6.2.2 Analysis 182
2.6.2.3 Mat for a 25-Story Building 183
2.6.2.4 Mat for an 85-Story Building 185
2.7 Guidelines for Thinking on Your Feet 187
2.8 Unit Quantities 187
2.8.1 Unit Quantity of Reinforcement in Columns 188
2.8.2 Unit Quantity of Reinforcement and Concrete in Floor Framing Systems 197

Chapter 3 Lateral Load-Resisting Systems 199

3.1 Flat Slab-Frame System 201
3.2 Flat Slab-Frame with Shear Walls 203
3.3 Coupled Shear Walls 204
3.4 Rigid Frame 205
3.4.1 Deflection Characteristics 207
3.4.1.1 Cantilever Bending Component 207
3.4.1.2 Shear Racking Component 207
3.5 Tube System with Widely Spaced Columns 210
3.6 Rigid Frame with Haunch Girders 210
3.7 Core-Supported Structures 212
3.8 Shear Wall–Frame Interaction 212
3.8.1 Behavior 217
3.8.2 Building Examples 218
3.9 Frame Tube System 224
3.9.1 Behavior 225
3.9.2 Shear Lag 225
3.9.3 Irregular Tube 229
3.10 Exterior Diagonal Tube 230
3.10.1 Example of Exterior Diagonal Tube: Onterie Center, Chicago 231
3.11 Bundled Tube 232
3.11.1 Example of Bundled Tube: One Magnificent Mile, Chicago 232
3.12 Spinal Wall Systems 234
3.13 Outrigger and Belt Wall System 234
3.13.1 Deflection Calculations 238
3.13.1.1 Case 1: Outrigger Wall at the Top 238
3.13.1.2 Case 2: Outrigger Wall at Quarter Height from the Top 239
3.13.1.3 Case 3: Outrigger Wall at Midheight 241
3.13.1.4 Case 4: Outrigger Wall at Quarter Height from the Bottom 241
3.13.2 Optimum Location of a Single Outrigger Wall 242
3.13.3 Optimum Locations of Two Outrigger Walls 247
3.13.4 Recommendations for Optimum Locations 250
3.14 Miscellaneous Systems 251

Chapter 4 Wind Loads 253

4.1 Design Considerations 253
4.2 Natural Wind 255
4.2.1 Types of Wind 256
4.3 Characteristics of Wind 256
4.3.1 Variation of Wind Velocity with Height (Velocity Profile) 257
4.3.2 Wind Turbulence 258
4.3.3 Probabilistic Approach 260
4.3.4 Vortex Shedding 261
4.3.5 Dynamic Nature of Wind 264
4.3.6 Pressures and Suctions on Exterior Surfaces 264
4.3.6.1 Scaling 264
4.3.6.2 Internal Pressures and Differential Pressures 265
4.3.6.3 Distribution of Pressures and Suctions 265
4.3.6.4 Local Cladding Loads and Overall Design Loads 266

4.4 ASCE 7-05: Wind Load Provisions 267
4.4.1 Analytical Procedure—Method 2, Overview 273
4.4.2 Method 2: Step-by-Step Procedure 274
4.4.2.1 Wind Speedup over Hills and Escarpments: K_{zt} Factor 280
4.4.2.2 Gust Effect Factor 281
4.4.2.3 Determination of Design Wind Pressures Using Graphs 289
4.4.2.4 Along-Wind Response 292
4.4.2.5 Worksheet for Calculation of Gust Effect Factor, G_f, Along-Wind Displacement and Acceleration 296
4.4.2.6 Comparison of Gust Effect Factor and Along-Wind Response 299
4.4.2.7 One More Example: Design Wind Pressures for Enclosed Building, Method 2 301
4.5 National Building Code of Canada (NBCC 2005): Wind Load Provisions 304
4.5.1 Static Procedure 304
4.5.1.1 Specified Wind Load 304
4.5.1.2 Exposure Factor, C_c 305
4.5.1.3 Gust Factors, C_g and C_{gi} 305
4.5.1.4 Pressure Coefficient, C_p 306
4.5.2 Dynamic Procedure 306
4.5.2.1 Gust Effect Factor, C_g (Dynamic Procedure) 307
4.5.2.2 Design Example: Calculations for Gust Effect Factor, C_g 309
4.5.2.3 Wind-Induced Building Motion 311
4.5.2.4 Design Example 312
4.5.2.5 Comparison of Along-Wind and Across-Wind Accelerations 314
4.5.3 Wind Load Comparison among International Codes and Standards 315
4.6 Wind-Tunnels 315
4.6.1 Types of Wind-Tunnel Tests 320
4.6.1.1 Rigid Pressure Model 321
4.6.1.2 High-Frequency Base Balance and High-Frequency Force Balance (HFBB/HFFB Model) Model 322
4.6.1.3 Aeroelastic Model 324
4.6.1.4 Multidegree-of-Freedom Aeroelastic Model 330
4.6.1.5 Option for Wind-Tunnel Testing 331
4.6.1.6 Lower Limit on Wind-Tunnel Test Results 331
4.6.2 Prediction of Acceleration and Human Comfort 331
4.6.3 Load Combination Factors 332
4.6.4 Pedestrian Wind Studies 332
4.6.5 Motion Perception: Human Response to Building Motions 335
4.6.6 Structural Properties Required for Wind-Tunnel Data Analysis 335
4.6.6.1 Natural Frequencies 336
4.6.6.2 Mode Shapes 336
4.6.6.3 Mass Distribution 337
4.6.6.4 Damping Ratio 337
4.6.6.5 Miscellaneous Information 338
4.6.6.6 Example 338
4.6.7 Period Determination and Damping Values for Wind Design 341

Chapter 5 Seismic Design 347

5.1 Building Behavior 349
5.1.1 Influence of Soil 349
5.1.2 Damping 350
5.1.3 Building Motions and Deflections 352
5.1.4 Building Drift and Separation 352
5.2 Seismic Design Concept 353
5.2.1 Structural Response 353
5.2.2 Load Path 353
5.2.3 Response of Elements Attached to Buildings 354
5.2.4 Adjacent Buildings 354
5.2.5 Irregular Buildings 355
5.2.6 Lateral Force–Resisting Systems 356
5.2.7 Diaphragms 357
5.2.8 Ductility 358
5.2.9 Damage Control Features 360
5.2.10 Continuous Load Path 361
5.2.11 Redundancy 361
5.2.12 Configuration 362
5.2.13 Dynamic Analysis 364
5.2.13.1 Response-Spectrum Method 367
5.2.13.2 Response-Spectrum Concept 371
5.2.13.3 Deformation Response Spectrum 372
5.2.13.4 Pseudo-Velocity Response Spectrum 373
5.2.13.5 Pseudo-Acceleration Response Spectrum 374
5.2.13.6 Tripartite Response Spectrum: Combined Displacement–Velocity–Acceleration (*DVA*) Spectrum ... 374
5.2.13.7 Characteristics of Response Spectrum 379
5.3 An Overview of 2006 IBC 381
5.3.1 Occupancy Category 381
5.3.2 Overturning, Uplifting, and Sliding 383
5.3.3 Seismic Detailing 383
5.3.4 Live-Load Reduction in Garages 384
5.3.5 Torsional Forces 384
5.3.6 Partition Loads 384
5.4 ASCE 7-05 Seismic Provisions: An Overview 384
5.5 An Overview of Chapter 11 of ASCE 7-05, Seismic Design Criteria 386
5.5.1 Seismic Ground-Motion Values 386
5.5.1.1 Site Coefficients, F_a and F_v 388
5.5.1.2 Site Class 389
5.5.1.3 Design Response Spectrum 389
5.5.2 Equivalent Lateral Force Procedure 390
5.5.2.1 Parameters S_s and S_1e 396
5.5.2.2 Site-Specific Ground Motion Analysis 397
5.5.3 Importance Factor and Occupancy Category 398
5.5.3.1 Importance Factor, I_E 398
5.5.3.2 Occupancy Categories 399
5.5.4 Seismic Design Category 400
5.5.5 Design Requirements for SDC A Buildings 401
5.5.6 Geologic Hazards and Geotechnical Investigation 404

5.5.7 Base Shear for Preliminary Design 405
5.5.8 Design Response Spectrum for Selected Cities in the U.S.A. 414
5.6 An Overview of Chapter 12 of ASCE 7-05, Seismic Design Requirements for Building Structures 427
5.6.1 Seismic Design Basis 427
5.6.2 Structural System Selection 427
5.6.3 Diaphragms 429
5.6.3.1 Irregularities 430
5.6.4 Seismic Load Effects and Combinations 430
5.6.5 Direction of Loading 431
5.6.6 Analysis Procedure 432
5.6.7 Modeling Criteria 432
5.6.8 Modal Analysis 433
5.6.9 Diaphragms, Chords, and Collectors 433
5.6.10 Structural Walls and Their Anchorage 434
5.6.11 Drift and Deformation 435
5.6.12 Foundation Design 436
5.6.12.1 Foundation Requirements for Structures Assigned to Seismic Design Category C 437
5.6.12.2 Foundation Requirements for Structures Assigned to Seismic Design Categories D, E, or F 437
5.7 ASCE 7-05, Seismic Design: An In-Depth Discussion 438
5.7.1 Seismic Design Basis 439
5.7.2 Structural System Selection 440
5.7.2.1 Bearing Wall System 440
5.7.2.2 Building Frame System 441
5.7.2.3 Moment Frame System 441
5.7.2.4 Dual System 441
5.7.3 Special Reinforced Concrete Shear Wall 442
5.7.4 Detailing Requirements 442
5.7.5 Building Irregularities 443
5.7.5.1 Plan or Horizontal Irregularity 446
5.7.5.2 Vertical Irregularity 448
5.7.6 Redundancy 448
5.7.7 Seismic Load Combinations 449
5.7.7.1 Seismic Load Effect 450
5.7.7.2 Seismic Load Effect with Overstrength 451
5.7.7.3 Elements Supporting Discontinuous Walls or Frames 451
5.7.8 Direction of Loading 451
5.7.9 Analysis Procedures 452
5.7.9.1 Equivalent Lateral-Force Procedure 455
5.7.9.2 Modal Response Spectrum Analysis 463
5.7.10 Diaphragms, Chords, and Collectors 464
5.7.10.1 Diaphragms for SDC A 465
5.7.10.2 Diaphragms for SDCs B through F 465
5.7.10.3 General Procedure for Diaphragm Design 465
5.7.11 Catalog of Seismic Design Requirements 473
5.7.11.1 Buildings in SDC A 473
5.7.11.2 Buildings in SDC B 474

5.7.11.3 Buildings in SDC C 475
5.7.11.4 Buildings in SDC D 476
5.7.11.5 Buildings in SDC E 478
5.7.11.6 Buildings in SDC F 478
5.8 Seismic Design Example: Dynamic Analysis Procedure (Response Spectrum Analysis) Using Hand Calculations 478
5.9 Anatomy of Computer Response Spectrum Analyses (In Other Words, What Goes on in the Black Box) 487
5.10 Dynamic Response Concept 497
5.10.1 Difference between Static and Dynamic Analyses 500
5.10.2 Dynamic Effects due to Wind Loads 503
5.10.3 Seismic Periods 504
5.11 Dynamic Analysis Theory 505
5.11.1 Single-Degree-of-Freedom Systems 505
5.11.2 Multi-Degree-of-Freedom Systems 508
5.11.3 Modal Superposition Method 511
5.11.4 Normal Coordinates 511
5.11.5 Orthogonality 512
5.12 Summary 518

Chapter 6 Seismic Design Examples and Details 523

6.1 Seismic Design Recap 523
6.2 Design Techniques to Promote Ductile Behavior 526
6.3 Integrity Reinforcement 529
6.4 Review of Strength Design 530
6.4.1 Load Combinations 532
6.4.2 Earthquake Load *E* 532
6.4.2.1 Load Combination for Verifying Building Drift 534
6.4.3 Capacity Reduction Factors, ϕ 534
6.5 Intermediate Moment-Resisting Frames 535
6.5.1 General Requirements: Frame Beams 535
6.5.2 Flexural and Transverse Reinforcement: Frame Beams 535
6.5.3 Transverse Reinforcement: Frame Columns 537
6.5.4 Detailing Requirements for Two-Way Slab Systems without Beams 538
6.6 Special Moment-Resisting Frames 539
6.6.1 General Requirements: Frame Beams 539
6.6.2 Flexural Reinforcement: Frame Beams 540
6.6.3 Transverse Reinforcement: Frame Beams 541
6.6.4 General Requirements: Frame Columns 541
6.6.5 Flexural Reinforcement: Frame Columns 541
6.6.6 Transverse Reinforcement: Frame Columns 544
6.6.7 Transverse Reinforcement: Joints 546
6.6.8 Shear Strength of Joint 546
6.6.9 Development of Bars in Tension 548
6.7 Shear Walls 548
6.7.1 Minimum Web Reinforcement: Design for Shear 548
6.7.2 Boundary Elements 549
6.7.3 Coupling Beams 550
6.8 Frame Members Not Designed to Resist Earthquake Forces 551

6.9 Diaphragms 552
6.9.1 Minimum Thickness and Reinforcement 552
6.9.2 Shear Strength 552
6.9.3 Boundary Elements 553
6.10 Foundations 553
6.10.1 Footings, Mats, and Piles 553
6.10.2 Grade Beams and Slabs-on-Grade 554
6.10.3 Piles, Piers, and Caissons 554
6.11 Design Examples 554
6.11.1 Frame Beam Example: Ordinary Reinforced Concrete Moment Frame 555
6.11.2 Frame Column Example: Ordinary Reinforced Concrete Moment Frame 557
6.11.3 Frame Beam Example: Intermediate Reinforced Concrete Moment Frame 559
6.11.4 Frame Column Example: Intermediate Reinforced Concrete Moment Frame 561
6.11.5 Shear Wall Example: Seismic Design Category A, B, or C 563
6.11.6 Frame Beam Example: Special Reinforced Concrete Moment Frame 565
6.11.7 Frame Column Example: Special Reinforced Concrete Moment Frame 570
6.11.8 Beam–Column Joint Example: Special Reinforced Concrete Frame 574
6.11.9 Special Reinforced Concrete Shear Wall 577
6.11.9.1 Preliminary Size Determination 579
6.11.9.2 Shear Design 579
6.11.9.3 Shear Friction (Sliding Shear) 580
6.11.9.4 Longitudinal Reinforcement 581
6.11.9.5 Web Reinforcement 581
6.11.9.6 Boundary Elements 583
6.11.10 Special Reinforced Concrete Coupled Shear Walls 587
6.11.10.1 Coupling Beams 588
6.11.10.2 Wall Piers 593
6.12 Typical Details 599
6.13 ACI 318-08 Update 600
6.13.1 Outline of Major Changes 600
6.13.2 Summary of Chapter 21, ACI 318-08 605
6.13.3 Analysis and Proportioning of Structural Members 605
6.13.4 Reinforcement in Special Moment Frames and Special Structural Walls 605
6.13.5 Mechanical Splices in Special Moment Frames and Special Structural Walls 606
6.13.6 Welded Splices in Special Moment Frames and Special Structural Walls 606
6.13.7 Ordinary Moment Frames, SDC B 606
6.13.8 Intermediate Moment Frames 606
6.13.9 Two-Way Slabs without Beams 607
6.13.10 Flexural Members (Beams) of Special Moment Frames 607
6.13.11 Transverse Reinforcement 608
6.13.12 Shear Strength Requirements 609

6.13.13 Special Moment Frame Members Subjected to Bending and Axial Loads 609
6.13.14 Shear Strength Requirements for Columns 611
6.13.15 Joints of Special Moment Frames 611
6.13.16 Special Structural Walls and Coupling Beams 611
6.13.17 Shear Wall Design for Flexure and Axial Loads 612
6.13.18 Boundary Elements of Special Structural Walls 613
6.13.19 Coupling Beams 613

Chapter 7 Seismic Rehabilitation of Existing Buildings 617

7.1 Code-Sponsored Design 619
7.2 Alternate Design Philosophy 619
7.3 Code Provisions for Seismic Upgrade 621
7.4 Building Deformations 622
7.5 Common Deficiencies and Upgrade Methods 623
7.5.1 Diaphragms 624
7.5.1.1 Cast-in-Place Concrete Diaphragms 624
7.5.1.2 Precast Concrete Diaphragms 627
7.5.2 Shear Walls 627
7.5.2.1 Increasing Wall Thickness 627
7.5.2.2 Increasing Shear Strength of Wall 628
7.5.2.3 Infilling between Columns 628
7.5.2.4 Addition of Boundary Elements 628
7.5.2.5 Addition of Confinement Jackets 629
7.5.2.6 Repair of Cracked Coupling Beams 629
7.5.2.7 Adding New Walls 629
7.5.2.8 Precast Concrete Shear Walls 629
7.5.3 Infilling of Moment Frames 629
7.5.4 Reinforced Concrete Moment Frames 630
7.5.5 Open Storefront 631
7.5.6 Clerestory 631
7.5.7 Shallow Foundations 632
7.5.8 Rehabilitation Measures for Deep Foundations 632
7.5.9 Nonstructural Elements 633
7.5.9.1 Nonload-Bearing Walls 633
7.5.9.2 Precast Concrete Cladding 633
7.5.9.3 Stone or Masonry Veneers 634
7.5.9.4 Building Ornamentation 634
7.5.9.5 Acoustical Ceiling 634
7.6 Seismic Rehabilitation of Existing Buildings, ASCE/SEI 41-06 634
7.6.1 Overview of Performance Levels 641
7.6.2 Permitted Design Methods 642
7.6.3 Systematic Rehabilitation 643
7.6.3.1 Determination of Seismic Ground Motions 644
7.6.3.2 Determination of As-Built Conditions 644
7.6.3.3 Primary and Secondary Components 645
7.6.3.4 Setting Up Analytical Model and Determination of Design Forces 645
7.6.3.5 Ultimate Load Combinations: Combined Gravity and Seismic Demand 647

7.6.3.6 Component Capacity Calculations, Q_{CE} and Q_{CL} 648
7.6.3.7 Capacity versus Demand Comparisons 649
7.6.3.8 Development of Seismic Strengthening Strategies 651
7.6.4 ASCE/SEI 41-06: Design Example 661
7.6.4.1 Dual System: Moment Frames and Shear Walls 661
7.6.5 Summary of ASCE/SEI 41-06 666
7.7 Fiber-Reinforced Polymer Systems for Strengthening of Concrete Buildings 667
7.7.1 Mechanical Properties and Behavior 667
7.7.2 Design Philosophy 668
7.7.3 Flexural Design 668
7.8 Seismic Strengthening Details 668
7.8.1 Common Strategies for Seismic Strengthening 669

Chapter 8 Tall Buildings 685

8.1 Historical Background 688
8.2 Review of High-Rise Architecture 692
8.3 Functional Requirements 694
8.4 Definition of Tall Buildings 695
8.5 Lateral Load Design Philosophy 695
8.6 Concept of Premium for Height 696
8.7 Relative Structural Cost 697
8.8 Factors for Reduction in the Weight of Structural Frame 697
8.9 Development of High-Rise Architecture 699
8.9.1 Architect–Engineer Collaboration 704
8.9.2 Sky Scraper Pluralism 704
8.9.3 Structural Size 705
8.10 Structural Scheme Options 705
8.10.1 Space Efficiency of High-Rise Building Columns 716
8.10.2 Structural Cost and Plan Density Comparison 717
8.11 Summary of Building Technology 718
8.12 Structural Concepts 719
8.13 Bending and Shear Rigidity Index 720
8.14 Case Studies 724
8.14.1 Empire State Building, New York, City, New York 724
8.14.2 South Walker Tower, Chicago, Illinois 724
8.14.3 Miglin-Beitler Tower, Chicago, Illinois 726
8.14.4 Trump Tower, Chicago, Illinois 730
8.14.4.1 Vital Statistics 731
8.14.5 Jin Mao Tower, Shanghai, China 731
8.14.6 Petronas Towers, Malaysia 734
8.14.7 Central Plaza, Hong Kong 736
8.14.8 Singapore Treasury Building 739
8.14.9 City Spire, New York City 740
8.14.10 NCNB Tower, North Carolina 740
8.14.11 Museum Tower, Los Angeles, California 743
8.14.12 MGM City Center, Vdara Tower, Las Vegas, Nevada 744
8.14.13 Citybank Plaza, Hong Kong 746
8.14.14 Trump Tower, New York 746

8.14.15 Two Prudential Plaza, Chicago, Illinois 747
8.14.16 Cent Trust Tower, Miami, Florida 749
8.14.17 Metropolitan Tower, New York City 751
8.14.18 Carnegie Hall Tower, New York City 752
8.14.19 Hopewell Center, Hong Kong 753
8.14.20 Cobalt Condominiums, Minneapolis, Minnesota 754
8.14.21 The Cosmopolitan Resort & Casino, Las Vegas, Nevada 757
8.14.22 Elysian Hotel and Private Residences, Chicago, Illinois 759
8.14.22.1 Foundations 759
8.14.22.2 Floor Systems 760
8.14.22.3 Gravity System 761
8.14.22.4 Lateral System 761
8.14.22.5 Tuned Liquid Damper 761
8.14.23 Shangri-La New York (610 Lexington Avenue), New York 762
8.14.24 Millennium Tower, 301 Mission Street, San Francisco, California 768
8.14.25 Al Bateen Towers, Dubai, UAE 773
8.14.25.1 Wind Loads 777
8.14.25.2 Seismic Loads 778
8.14.26 SRZ Tower, Dubai, UAE 778
8.14.26.1 Wind Loads 779
8.14.26.2 Seismic Loads 782
8.14.26.3 Computer Model 782
8.14.26.4 Building Behavior 783
8.14.26.5 Wind 783
8.14.27 The Four Seasons Hotel and Tower, Miami, Florida 783
8.14.28 Burj Dubai 786
8.15 Future of Tall Buildings 791

Chapter 9 Special Topics 793

9.1 Damping Devices for Reducing Motion Perception 793
9.1.1 Passive Viscoelastic Dampers 793
9.1.2 Tuned Mass Damper 795
9.1.2.1 Citicorp Tower, New York 796
9.1.2.2 John Hancock Tower, Boston, Massachusetts 798
9.1.2.3 Design Considerations for TMD 799
9.1.3 Sloshing Water Damper 799
9.1.4 Tuned Liquid Column Damper 799
9.1.4.1 Wall Center, Vancouver, British Columbia 800
9.1.4.2 Highcliff Apartment Building, Hong Kong 800
9.1.5 Simple Pendulum Damper 800
9.1.5.1 Taipei Financial Center 802
9.1.6 Nested Pendulum Damper 803
9.2 Seismic Isolation 804
9.2.1 Salient Features 806
9.2.2 Mechanical Properties of Seismic Isolation Systems 808
9.2.3 Elastomeric Isolators 808
9.2.4 Sliding Isolators 810
9.2.5 Seismically Isolated Structures: ASCE 7-05 Design Provisions 810

9.2.5.1 Equivalent Lateral Force Procedure 813
9.2.5.2 Lateral Displacements 813
9.2.5.3 Minimum Lateral Forces for the Design of Isolation System and Structural Elements at or below Isolation System 816
9.2.5.4 Minimum Lateral Forces for the Design of Structural Elements above Isolation System 816
9.2.5.5 Drift Limits 817
9.2.5.6 Illustrative Example: Static Procedure 817
9.3 Passive Energy Dissipation 829
9.4 Preliminary Analysis Techniques 830
9.4.1 Portal Method 833
9.4.2 Cantilever Method 834
9.4.3 Lateral Stiffness of Frames 837
9.4.4 Framed Tube Structures 845
9.5 Torsion 846
9.5.1 Preview 846
9.5.2 Concept of Warping Behavior 857
9.5.3 Sectorial Coordinate ω' 861
9.5.4 Shear Center 863
9.5.4.1 Evaluation of Product Integrals 865
9.5.5 Principal Sectorial Coordinate ω_s Diagram 865
9.5.5.1 Sectorial Moment of Inertia I_ω 865
9.5.6 Torsion Constant J 865
9.5.7 Calculation of Sectorial Properties: Worked Example 866
9.5.8 General Theory of Warping Torsion 867
9.5.8.1 Warping Torsion Equations for Shear Wall Structures 870
9.5.9 Torsion Analysis of Shear Wall Building: Worked Example 871
9.5.10 Warping Torsion Constants for Open Sections 881
9.5.11 Stiffness Method Using Warping-Column Model 883
9.6 Performance-Based Design 885
9.6.1 Design Ideology 885
9.6.2 Performance-Based Engineering 886
9.6.3 Linear Response History Procedure 887
9.6.4 Nonlinear Response History Procedure 887
9.6.5 Member Strength 888
9.6.6 Design Review 888
9.6.7 New Building Forms 889
9.7 Wind Deflections 890
9.8 2009 International Building Code (2009 IBC) Updates 892
9.8.1 An Overview of Structural Revisions 892
9.8.1.1 Earthquake Loads 892
9.8.1.2 Wind Loads 892
9.8.1.3 Structural Integrity 893
9.8.1.4 Other Updates in Chapter 16 893
9.8.1.5 Chapter 18: Soils and Foundations 893
9.8.1.6 Chapter 19: Concrete 893
9.8.2 Detail Discussion of Structural Revisions 893
9.8.2.1 Section 1604.8.2: Walls 893
9.8.2.2 Section 1604.8.3: Decks 894
9.8.2.3 Section 1605.1.1: Stability 894

9.8.2.4 Sections 1607.3 and 1607.4: Uniformly Distributed Live Loads and Concentrated Live Loads 894
9.8.2.5 Section 1607.7.3: Vehicle Barrier Systems 894
9.8.2.6 Section 1607.9.1.1: One-Way Slabs 894
9.8.2.7 Section 1609.1.1.2: Wind Tunnel Test Limitations..... 894
9.8.2.8 Section 1613.7: ASCE 7-05, Section 11.7.5: Anchorage of Walls..... 897
9.8.2.9 Section 1607.11.2.2: Special Purpose Roofs 897
9.8.2.10 Section 1613: Earthquake Loads 897
9.8.2.11 Minimum Distance for Building Separation..... 898
9.8.2.12 Section 1613.6.7: Minimum Distance for Building Separation 899
9.8.2.13 Section 1614: Structural Integrity 899
9.8.3 Chapter 17: Structural Tests and Special Inspections..... 900
9.8.3.1 Section 1704.1: General 900
9.8.3.2 Section 1704.4: Concrete Construction..... 900
9.8.3.3 Section 1704.10: Helical Pile Foundations 900
9.8.3.4 Section 1706: Special Inspections for Wind Requirements 900
9.8.4 Chapter 18: Soils and Foundations..... 900
9.8.4.1 Section 1803: Geotechnical Investigations 900
9.8.4.2 Section 1807.2.3: Safety Factor..... 900
9.8.4.3 Section 1808.3.1: Seismic Overturning 901
9.8.4.4 Sections 1810.3.1.5 and 1810.3.5.3.3: Helical Piles..... 901
9.8.5 Chapter 19: Concrete 901
9.8.5.1 Section 1908.1: General..... 901
9.8.5.2 Section 1908.1.9: ACI 318, Section D.3.3 901
9.8.5.3 Sections 1909.6.1 and 1909.6.3: Basement Walls and Openings in Walls 901
9.8.6 Anticipated Revisions in 2012 IBC 901

References..... 903

Index..... 907

List of Figures

FIGURE 1.1 Confinement of column concrete by transverse reinforcement: (a) confinement by spiral or circular hoops, (b) confinement by a rectangular hoop, and (c) confinement by hoops and cross ties. 2

FIGURE 1.2 Confinement of circular column: (a) column with circular ties, (b) radial forces, and (c) free-body of upper portion of tie. 2

FIGURE 1.3 Column ties and seismic hooks: (a) overlapping 90° hooks at corners cannot confine a concrete core after concrete cover spalls, (b) 135° hooks, required in high seismic areas, provide the necessary confinement for the core while simultaneously resisting buckling of column vertical bars, and (c) seismic hook. ... 3

FIGURE 1.4 Frame-beam subjected to cyclic loads: (a) cracks due to $-M_u$ and (b) cracks due to $+M_u$. 4

FIGURE 1.5 Ductility model. The ability of the structure to provide resistance in the inelastic domain of response is termed "ductility". Δ_u is the limit to ductility corresponding to a specified limit of strength degradation. 5

FIGURE 1.6 Hysteresis loops. (a) Idealized elastoplastic loop. (b) Well-detailed beam plastic hinge loop. 6

FIGURE 1.7 Tension member in a transfer truss. *Note*: Column C not required for truss action but recommended. 8

FIGURE 1.8 Transfer girder: schematic elevation with concrete column. 9

FIGURE 1.9 Transfer girder: schematic elevation with encased composite column. 10

FIGURE 1.10 Transfer girder: schematic elevation with filled steel column. 11

FIGURE 1.11 Tension member in a strut-and-tie model. *Note*: Capacity reduction factor, $\phi = 0.75$. 12

FIGURE 1.12 Development of cracks in a flexural member. Vertical cracks may develop near midspan, stable, hairline cracks are normal, but widening cracks indicate impending failure. 12

FIGURE 1.13 Two-way slab systems: (a) flat plate, (b) flat plate with column capitols, (c) flat plate with drop panels, (d) band beams, (e) one-way beam and slab, (f) skip joist system, (g) waffle slab, and (h) standard joist system. 14

FIGURE 1.14 Unit shearing stresses acting at right angles to each other. 14

FIGURE 1.15 Shear stress distribution in a rectangular beam. 15

FIGURE 1.16 Complimentary shear stresses. (a) Rectangular beam subject to vertical shear force V_u, (b) beam element between two parallel sections, and (c) shear stresses on perpendicular faces of a beam element. 16

FIGURE 1.17 Diagonal tension. 17

FIGURE 1.18 Shear-resisting forces along a diagonal crack: $V_{ext} = V_{cy} + V_{iy} + V_d + V_s$. 17

FIGURE 1.19 Frictional resistance generated by shear-friction reinforcement. (a) Applied shear, (b) enlarged crack surface, and (c) free body diagram of concrete above crack. 19

FIGURE 1.20 Potential locations of sliding shear. (a–c) Squat wall, (d) tall wall, and (e) plastic hinge region in a frame beam. 20

FIGURE 1.21 Punching shear failure in a two-way slab system. It is the tendency of the slab to drop as a unit around the column. 21

FIGURE 1.22 Transfer of slab load into columns. 22

FIGURE 1.23 Strut-and-tie model for punching shear. 22

FIGURE 1.24 Torsion in spandrel beams. 23

FIGURE 1.25 Equilibrium torsion. 23

FIGURE 1.26 Compatibility torsion. 24

FIGURE 1.27 Earthquake loading: Dynamic action of earthquakes can be simplified as a group of horizontal forces that are applied to the structure in proportion to its mass, and to the height of the mass above the ground. 27

FIGURE 1.28 Circulation of world's winds. 28

FIGURE 1.29 Wind load distribution: positive pressure on windward wall, and negative pressure (suction) on leeward wall and roof. For a hermetically sealed building, internal pressures cancel out, hence no effect on overall building loads. 29

FIGURE 1.30 Shear deformations in a shear wall. 33

FIGURE 1.31 Diaphragm concept. *Note*: Seismic elements in the E–W direction not shown for clarity. 34

FIGURE 1.32 Floor plan: building with shear walls and perimeter frame. 35

FIGURE 1.33 Building with perimeter shear walls. 35

FIGURE 1.34 Shear walls with openings. 36

FIGURE 1.35 Lateral load-resistance of single and coupled shear walls. 36

FIGURE 1.36 Coupling beam resistance. 37

FIGURE 1.37 Moment-resisting frame: The lateral resistance is provided by keeping the frame from changing into a parallelogram. The interconnection of columns and beams is rigid. 37

FIGURE 1.38 Shear wall–frame interaction. 38

FIGURE 1.39 Diaphragms with openings. 39

FIGURE 1.40 Distress in columns due to unintended stiffness addition. 42

FIGURE 1.41 Column joint failure. Concrete in columns is not well enough confined by rebar ties, resulting in failure as concrete splits off and rebar buckles. 42

FIGURE 1.42 Collapse patterns. (a) Inadequate shear strength. (b) Inadequate beam/column strength. (c) Tension/compression failure due to overturning. 43

FIGURE 1.43 More collapse patterns. 43

FIGURE 1.44 Heavy floor collapse: Major force is in inertia of floors and is concentrated at each level. If columns crack and fail, heavy floors collapse. 44

FIGURE 1.45 Torsion effect: property-line walls placed on one side of a frame structure, create eccentric condition which can lead to collapse. 44

FIGURE 1.46 Soft story collapse: Lower story that is weakened by too many openings becomes racked (rectangles become parallelograms). This may result in failure of first story columns due to mismatch of demand versus strength. 45

FIGURE 1.47 P-Δ effect; simple cantilever model. $M_u = V_u H + W_g \Delta$. 46

FIGURE 1.48 P-Δ effect: Shear wall-frame system. 46

FIGURE 1.49 Collapse due to P-Δ effect. 47

FIGURE 1.50 Exterior explosion. 48

FIGURE 1.51 Damage due to exterior explosion. (a) Exterior windows, columns, and walls. (b) Roof and floor slabs. (c) Building sway due to ground shock. 48

FIGURE 1.52 Interior explosion: when explosions occur within structures, pressures can build up within confined spaces, causing lightly attached wall, floor, and roof surfaces to be blown away. 49

FIGURE 1.53 Buckling of tall building under its own weight: (a) equivalent cantilever, (b) circular building, (c) rectangular building, and (d) equivalent channels to account for shear lag effects. 51

FIGURE 2.1 Common types of floor systems: (a) Two-way flat plate, (b) Two-way waffle, (c) Two-way flat slab with drops, (d) One-way beam and slab, (e) Skip joist wide module, (f) Two-way beam and slab, (g) One-way joist slab, and (h) One-way flat slab. 56

FIGURE 2.2 Relative form cost, live load = 125 psf. 57

FIGURE 2.3 Relative form cost, live load = 60 psf. 57

FIGURE 2.4 Influence of lumber dimension on site-cast concrete. Designing for nominal lumber dimensions results in economy. 58

FIGURE 2.5 Pilasters and wall columns. 59

FIGURE 2.6 Pour strip as a simple span supported by cantilevers. 60

FIGURE 2.7 Drop panel dimensions: (a) plan view and (b) section view. Dimensions d, x, and y should remain constant throughout for maximum economy. 61

FIGURE 2.8 Soffit at the same horizontal level. 61

FIGURE 2.9 Band and narrow beams: Band beam, Section A, is more economical than narrow deep beam, Section B. 61

FIGURE 2.10 Suggested formwork dimensions for deep beams. 62

FIGURE 2.11 Beam haunches: (a) plan and (b) section. 62

FIGURE 2.12 Formwork for spandrel beams. Narrow and deep spandrels framing into columns result in more expensive formwork. 63

FIGURE 2.13 Suggestions for economical framing: (a) use locally available modulus, (b) maintain soffit at same elevation, and (c) step down wall thickness as shown in the figure. 64

FIGURE 2.14 Flat plate system. 66
FIGURE 2.15 Flat slab system. 66
FIGURE 2.16 Waffle system. 67
FIGURE 2.17 One-way joist system: (a) building plan and (b) Section A. 68
FIGURE 2.18 Skip joist system: (a) building plan and (b) Section A. 69
FIGURE 2.19 Skip joist system, another example: (a) plan view, (b) Section A, and (c) Section B. 69
FIGURE 2.20 Band beam system: (a) floor plan and (b) section. 70
FIGURE 2.21 Haunch girder-framing system. 71
FIGURE 2.22 Tapered haunch girder. 71
FIGURE 2.23 Hammerhead haunch girder. 71
FIGURE 2.24 10 m × 20 m (32.81 ft × 65.62 ft) gird; haunch girder with transverse beams: Plan. 72
FIGURE 2.25 Section AA: haunch girder elevation. 73
FIGURE 2.26 Typical frame design strip. 75
FIGURE 2.27 Equivalent frame concept, N–S direction. 75
FIGURE 2.28 Moment transfer through torsion. 76
FIGURE 2.29 Formation of yield lines. Yielding starts in regions of high moment and spreads to areas that are still elastic. 77
FIGURE 2.30 Yield-line design example. (a) Yield-line at center span. (b) Yield-line rotations. 78
FIGURE 2.31 Yield-line design example. (a) Yield line at a varter span. (b) Yield-line rotations. 79
FIGURE 2.32 Yield-line patterns. (a) Simply supported slab with uniformly distributed load. (b) Same as (a) but with built-in edges. (c) Equilateral, triangular simply supported slab with uniformly distributed load. (d) same as (b) but with built-in edges. 81
FIGURE 2.33 Yield-line analysis of simply supported square slab: (a) yield lines and (b) yield-line rotations. 81
FIGURE 2.34 Stress distribution in deep beams: (a) $l/h = 4$, (b) $l/h = 2$, (c) $l/h = 1$, and (d) $l/h < 1.0$. 84
FIGURE 2.35 Example of Saint-Venant's principle: pinched region is similar to the discontinuity, D, region (ACI 318-08, Appendix A). Regions away from D are bending, B, regions. 86
FIGURE 2.36 Strut-and-tie terminology: (a) strut-and-tie model (b) C-C-C node resisting three compressive forces (c) and (d) C-C-T nodes resisting two compressive, and one tensile force. 86
FIGURE 2.37 Strut-and-tie model: Node and element identification. 88

FIGURE 2.38 Strut-and-tie model: (a) C-C-C node and (b) C-C-T node. 88

FIGURE 2.39 Member stress limits and effective widths. 89

FIGURE 2.40 Factored forces and design strengths. 89

FIGURE 2.41 Stress ratios. 89

FIGURE 2.42 Design Summary. *Note:* Struts E4, E5, and E6 are transfer column and supporting columns. Nodes N4, N5, and N6 are at the face of transfer column and supporting columns. 90

FIGURE 2.43 Transfer girder-schematic reinforcement. 91

FIGURE 2.44 One-way slab example: (a) typical 1 ft strip; (b) slab modeled as a continuous beam; and (c) design moments. 92

FIGURE 2.45 ACI coefficients for positive moments: (a) interior span, (b) exterior span, discontinuous end integral with supports, and (c) exterior span, discontinuous end unrestrained. 93

FIGURE 2.46 ACI coefficients for negative moments: (a) at interior supports; (b) at exterior face of first interior support, more than two spans; and (c) at exterior face of first interior support, two spans. 94

FIGURE 2.47 Design example, one-way slab: (a) partial floor plan and (b) section. See Figure 2.44 for dimensions and loading. 95

FIGURE 2.48 T-beam design example. 98

FIGURE 2.49 Shear reinforcement. 102

FIGURE 2.50 Interior span moments. 104

FIGURE 2.51 Exterior span moments. 105

FIGURE 2.52 Design example: Two-way slab. 106

FIGURE 2.53 Two-way posttensioned flat plate system. 119

FIGURE 2.54 Band-beam system. 119

FIGURE 2.55 Cracking in PT slab caused by restraint of perimeter walls. 120

FIGURE 2.56 Recommended distance between pour strips. 1. 150 ft ± typical. 2. 200 ft ± if restraint is minimal. 3. 300 ft ± maximum length PT slab irrespective of the number of pour strips. 121

FIGURE 2.57 Method of minimizing restraining forces: (a) temporary sliding joint, (b) sleeves around rebar dowels, (i) section and (ii) plan, and (c) temporary pour strips: (i) at perimeter of building; (ii) at interior of slab. 121

FIGURE 2.58 Concept of secondary moments: (a) two-span continuous beam, (b) vertical upward displacement due to PT, (c) primary moment, (d) reactions due to PT, (e) secondary moment, and (f) final moments. 124

FIGURE 2.59 Secondary moment example 2A: (a) Two-span continuous prestressed beam, (b) equivalent loads due to prestress, consisting of upward load, horizontal compression due to prestress W_p and downward loads at A, B, and C, (c) shear force diagram, statically indeterminate beam, (d) moment diagram, statically indeterminate beam, (e) primary shear force diagram, (f) primary moment diagram, and (h) secondary moments. 126

FIGURE 2.60 Secondary moment: Compatibility method: Example 2B. (a) two-span continuous beam, (b) equivalent loads, (c) upward deflection Δ_1 due to W_p, (d) downward deflection Δ_2 due to a load of 66.15 kip at center span, (e) load P_s corresponding to Δ_1–Δ_2, and (f) secondary moments. 129

FIGURE 2.61 Secondary moment: Example 2C: (a) two-span continuous beam, (b) equivalent moment $M = Pe$ at center of span, (c) equivalent loads and moments, (d) upward deflection Δ_1 due to W_p, (e) downward deflection Δ_2 due to a load of 66.15 at center of span, (f) downward deflection Δ_3 due to moments $M = Pe$ at center of span, (g) load P_s corresponding to Δ_1–Δ_2–Δ_3, and (h) secondary moments. 131

FIGURE 2.62 Idealized stress–strain curve (Adapted from *Posttensioning Concrete Institute Design Handbook*, 5th Edn.) Typical stress-strain cure with seven-wine low-relaxation prestressing strand. These curves can be approximated by the following equations:

250 ksi	270 ksi
$\varepsilon_{ps} \le 0.0076$: $f_{ps} = 28{,}500\ \varepsilon_{ps}$ (ksi)	$\varepsilon_{ps} \le 0.0086$: $f_{ps} = 28{,}500\ \varepsilon_{ps}$ (ksi)
$\varepsilon_{ps} > 0.0076$: $f_{ps} = 250 - \dfrac{0.04}{\varepsilon_{ps} - 0.0064}$ (ksi)	ε_{ps} 0.0086: $f_{ps} = 270 - \dfrac{0.04}{\varepsilon_{ps} - 0.007}$ (ksi)

134

FIGURE 2.63 Strength design example 1: beam section. 135

FIGURE 2.64 Example 1: strain diagram, first trial. 135

FIGURE 2.65 Example 1: strain diagram, second trial. 136

FIGURE 2.66 Example 1: force diagram. 137

FIGURE 2.67 Example 2: strain diagram, first trial. 138

FIGURE 2.68 Example 2: strain diagram, second trial. 139

FIGURE 2.69 Example 3: prestressed T-beams. 140

FIGURE 2.70 Example 3: strain diagram, first trial. 141

FIGURE 2.71 Example 3: strain diagram, second trial. 141

FIGURE 2.72 Posttensioned transfer girder: (a) elevation and (b) section. 145

FIGURE 2.73 Load balancing concept; (a) beam with parabolic tendon; and (b) free-body diagram. 148

FIGURE 2.74 Equivalent loads and moments due to sloped tendon: (a) upward uniform load due to parabolic tendon; (b) constant moment due to straight tendon; (c) upward uniform load and end moments due to parabolic tendon not passing through the centroid at the ends; and (d) vertical point load due to sloped tendon. 148

FIGURE 2.75 Preliminary design: simple span beam. 150

FIGURE 2.76 Tendon profile in continuous beam: (a) simple parabolic profile and (b) reverse curvature in tendon profile. 152

FIGURE 2.77 Tendon profile: (a) typical exterior span and (b) typical interior span. 153

FIGURE 2.78 Equivalent loads due to prestress. 154
FIGURE 2.79 Example 1: one-way posttensioned slab. 154
FIGURE 2.80 Example 1: one-way moment diagram. 155
FIGURE 2.81 Example 1: one-way tendon profile; interior bay. 155
FIGURE 2.82 Characteristics of tendon profile. 156
FIGURE 2.83 Example 2: posttensioned continuous beam dimensions and loading. 161
FIGURE 2.84 Example 2: posttensioned continuous beam: service load moments. 162
FIGURE 2.85 Example 3; flat plate design: (a) span and loading and (b) elastic moments due to dead load. 164
FIGURE 2.86 End bay tendon profiles. 165
FIGURE 2.87 Anchorage of added tendons. 168
FIGURE 2.88 Example problem 3: flat plate, tendon profiles: (a) interior span; (b) exterior span, reverse curvature at right support; and (c) exterior span, reverse curvature at both supports. 169
FIGURE 2.89 Anchor device at distributed tendons: (a) dead end and (b) stressing end. 173
FIGURE 2.90 Anchor device at banded tendons; (a) dead end and (b) stressing end. 174
FIGURE 2.91 Construction joint with intermediate stressing. 174
FIGURE 2.92 Typical reinforcement of tendon band at slab edge or at intermediate stressing. 175
FIGURE 2.93 Placement of added tendon. 175
FIGURE 2.94 Flaring of banded tendons at slab edge. 176
FIGURE 2.95 Typical column-slab section. 176
FIGURE 2.96 Typical interior column. 177
FIGURE 2.97 Typical column section. 177
FIGURE 2.98 Typical drop panel section. 178
FIGURE 2.99 Foundation system for a corner column of an exterior X-braced tube building: (a) plan and (b) schematic section. 180
FIGURE 2.100 Foundation mat for a 25-story building: finite element idealization and column ultimate loads. 184
FIGURE 2.101 Vertical deflection of mat along section x–x. 185
FIGURE 2.102 Bending moment variation along section x–x. 185
FIGURE 2.103 Foundation mat for a proposed 85-story office building: (a) finite element idealization and (b) cross section x–x. 186
FIGURE 2.104 Contact pressure contour, ksf: (a) dead plus live loads (b) dead plus live plus wind. *Note:* $K = 100\,\text{lb/in.}^3$. 186
FIGURE 2.105 Flat slab with drop panels. 187
FIGURE 2.106 Beam and slab system 26 ft × 26 ft bays. 188

FIGURE 2.107 Beam and slab system 30 ft × 40 ft bays. 189
FIGURE 2.108 One-way joist (par joist) system 30 ft × 30 ft bays. 189
FIGURE 2.109 One-way joist system 30 ft × 40 ft bays. 190
FIGURE 2.110 One-way joist with constant depth girders 30 ft × 40 ft bays. 190
FIGURE 2.111 One-way skip joist with haunch girders 30 ft × 40 ft bays. 191
FIGURE 2.112 Waffle slab (two-way joist) system 30 ft × 30 ft bays. 191
FIGURE 2.113 Waffle slab (two-way joist) system 30 ft × 40 ft bays. 192
FIGURE 2.114 Posttensioned system for parking structure 20 ft × 60 ft bays. 192
FIGURE 2.115 Posttensioned flat plato 26 ft × 26 ft bays. 193
FIGURE 2.116 One-way solid slabs, unit quantities: (a) reinforcement and (b) concrete. 193
FIGURE 2.117 One-way pan joists, unit quantities: (a) reinforcement and (b) concrete. 194
FIGURE 2.118 Two-way slabs, unit quantities: (a) reinforcement and (b) concrete. 194
FIGURE 2.119 Waffle slabs, unit quantities: (a) reinforcement and (b) concrete. 195
FIGURE 2.120 Flat plates, unit quantities: (a) reinforcement and (b) concrete. 195
FIGURE 2.121 Flat slabs, unit quantities: (a) reinforcement and (b) concrete. 196
FIGURE 2.122 Unit quantity of reinforcement in columns. 197
FIGURE 2.123 Preliminary design and material quantities for floor systems. 198
FIGURE 3.1 Structural systems categories. 200
FIGURE 3.2 Response of flat slab-frames to lateral loads: Displacement compatibility between slab and walls. 202
FIGURE 3.3 Typical floor systems for flat slab-frames: (a) flat plate; (b) flat slab with drop panels; (c) two-way waffle system. 202
FIGURE 3.4 Flat slab-frame with shear walls. 204
FIGURE 3.5 Coupled shear walls. 204
FIGURE 3.6 Representation of coupled shear wall by continuum model: (a) Wall with openings. (b) Analytical model for close-form solution. 205
FIGURE 3.7 Rigid frame: Forces and deformations. 206
FIGURE 3.8 Shear wall–frame interaction. 206
FIGURE 3.9 Bending deformation of rigid frame: (a) Moment resisted by axial loads in columns. (b) Cantilever bending of shear wall. 207
FIGURE 3.10 Shear deflection analogy: The lateral deflections of a story-high rigid frame due to beam and column rotations may be considered analogous to the shear deflections of a story-high segment of a shear wall. 208
FIGURE 3.11 Story mechanism: Strong-column-weak beam requirement aims at preventing story mechanism. 209
FIGURE 3.12 Tube building with widely spaced perimeter columns. 210
FIGURE 3.13 Typical floor framing plan: Haunch girder scheme. 211
FIGURE 3.14 Haunch girder elevation and reinforcement. 211

FIGURE 3.15 Haunch girder section. 212

FIGURE 3.16 The Huntington. (Architects, Talbot Wilson & Associates; structural engineers, Walter P. Moore and Associates; contractor, W. S. Bellows Construction Corp.). 212

FIGURE 3.17 A 28-story haunch girder building, Houston, Texas. (a) Typical floor plan and (b) photograph. (Structural engineers, Walter P. Moore and Associates.) ... 213

FIGURE 3.18 Examples of core-supported buildings. (a) cast-in-place shear walls with precast surround; (b) shear walls with posttensioned flat plate, and (c) shear walls with one-way joist system. 214

FIGURE 3.19 Concrete core with steel surround. 214

FIGURE 3.20 Shear walls with perimeter frames. 215

FIGURE 3.21 Shear walls with interior frames. 216

FIGURE 3.22 Shear walls with outrigger girders. 216

FIGURE 3.23 Full depth interior shear walls acting as gaint K-brace. (a) Plan and (b) schematic section. 216

FIGURE 3.24 Example of shear wall–frame interaction: Typical floor plan. 218

FIGURE 3.25 Simplified analytical model of bygone era. With the availability of computer software, simplified methods are no longer in use. 219

FIGURE 3.26 Shear force distribution. 220

FIGURE 3.27 Example of shear wall–frame interaction: 50-Plus-story haunch girder—shear wall building. 220

FIGURE 3.28 Framing plans. (a) Levels 2 through 14 floor plan, (b) levels 15 through 26 floor plan, (c) levels 27 through 39 floor plan, (d) levels 40 through 47 floor plan, (e) levels 48 and 49 floor plan, and (f) levels 50 and 51 floor plan. 221

FIGURE 3.29 Distribution of shear forces. 224

FIGURE 3.30 Frame tube building. (a) Schematic plan and (b) isometric view. 225

FIGURE 3.31 Shear lag effects in a hollow tube structure: (a) cantilever tube subjected to lateral loads, (b) shear stress distribution, and (c) distortion of flange element caused by shear stresses. 226

FIGURE 3.32 Axial stress distribution in a square hollow tube with and without shear lag. 226

FIGURE 3.33 Free-form tubular configurations. 227

FIGURE 3.34 Shear lag in framed tube. 227

FIGURE 3.35 Cantilever box beam with two end channels. 228

FIGURE 3.36 Shear lag effects in T-beams flanges: (a) Cross-section of T beam. (b) Horizontal shear stresses between beam web and flange. (c) Nonuniform distribution of compressive stresses in flange. 229

FIGURE 3.37 Secondary frame action in an irregular tube; schematic axial forces in perimeter columns. 230

FIGURE 3.38 Exterior diagonal braces in a tall steel building. 231
FIGURE 3.39 Example of exterior diagonal tube: Onterie Center, Chicago, IL. 232
FIGURE 3.40 Bundled tube: schematic plan. 232
FIGURE 3.41 Schematics of bundled tubes. 233
FIGURE 3.42 One Magnificent Mile, Chicago, IL; structural system. 233
FIGURE 3.43 Burj Dubai, schematic plan. 234
FIGURE 3.44 Outrigger and belt wall system with centrally located core. 235
FIGURE 3.45 Outrigger and belt wall system with an offset core. 235
FIGURE 3.46 Vierendeel frames acting as outrigger and belt wall system. 236
FIGURE 3.47 Haunch girders as outriggers. 236
FIGURE 3.48 Cap wall system: (a) Plan. (b) Schematic section. 237
FIGURE 3.49 Outrigger located at top, $z = L$. 238
FIGURE 3.50 Outrigger at quarter-height from top, $z = 0.75L$. 240
FIGURE 3.51 Outrigger at midheight, $z = 0.5L$. 241
FIGURE 3.52 Outrigger at quarter-height from bottom, $z = 0.25L$. 242
FIGURE 3.53 Outrigger at distance x from top. 243
FIGURE 3.54 Schematic plan of a steel building with outriggers and belt trusses at a single level. 245
FIGURE 3.55 Schematic plan of a concrete building with outriggers and belt walls at a single level. 246
FIGURE 3.56 Schematic section showing outriggers and belt walls at a single level. 246
FIGURE 3.57 Deflection index verses outrigger and belt wall location. *Note:* DI = Deflection without outrigger/Deflection with outrigger. 247
FIGURE 3.58 Structural schematics; building with outrigger and belt walls at two locations. 248
FIGURE 3.59 Analytical model of a building with outriggers and belt walls at two locations. 248
FIGURE 3.60 Method of analysis for two outrigger system: (a) Two-outrigger structure, (b) external moment diagram, (c) M_1 diagram, (d) M_2 diagram, and (e) core resultant moment diagram. 249
FIGURE 3.61 Deflection index verses belt wall and outrigger locations. 250
FIGURE 3.62 Optimum location of outriggers, (a) single outrigger, (b) two outriggers, (c) three outriggers, and (d) four outriggers. 251
FIGURE 3.63 Cellular tube with interior vierendeel frames. 251
FIGURE 3.64 Structural concept for supertall buildings. 252
FIGURE 4.1 Wind flow around buildings. 254
FIGURE 4.2 Wind velocity profiles as defined in the ASCE 7-05. Velocity profiles are determined by fitting curves to observed wind speeds. 257

FIGURE 4.3 Schematic record of wind speed measured by an anemometer. 258

FIGURE 4.4 Critical components of wind in aeronautical engineering. 261

FIGURE 4.5 Simplified wind flow consisting of along-wind and across-wind. 262

FIGURE 4.6 Vortex shedding: periodic shedding of vertices generates building vibrations in the transverse direction. 262

FIGURE 4.7 High pressures and suctions around building corners. 266

FIGURE 4.8 Pressure contours as measured in a wind tunnel test: (a) building elevation showing suction (negative pressures); (b) building elevation showing positive pressures. 268

FIGURE 4.9 Wind pressure diagram for cladding design: (a) block diagram relating measured pressures, psf, to building grid system; (b) pressures measured in wind tunnel, psf. 269

FIGURE 4.10 Wind speed map for United States and Alaska. (a) Map of the United States, (b) western Gulf of Mexico hurricane coastline (enlarged), (c) eastern Gulf of Mexico and southeastern United States hurricane coastline (enlarged), (d) mid- and north-Atlantic hurricane coastline (enlarged). (Adapted from ASCE 7-05.). 271

FIGURE 4.11 Topographic factor K_{zt} (From ASCE 7-05 Figure 6.4). 275

FIGURE 4.12 Topographic factor K_{zt} based on equations. 276

FIGURE 4.13 External pressure coefficient C_p with respect to plan aspect ratio L/B: (a) $0 \le L/B \le 1$; (b) $L/B = 2$; (c) $L/B > 4$. Linear variation permitted for leeward suction (see Figure 4.15). 277

FIGURE 4.14 Building elevation showing variation of C_p: (a) $0 \le L/B \le 1$, (b) $L/B = 2$, and (c) $L/B > 4$. 277

FIGURE 4.15 Leeward suction C_p versus plan aspect ratio, W/D. 278

FIGURE 4.16 Combined velocity pressure and exposure coefficients, K_h and K_z. 278

FIGURE 4.17 K_z values for buildings up to 1500 ft tall. 279

FIGURE 4.18 Variation of positive velocity pressure, q_z, versus wind speed and exposure categories. *Note:* Gust factor, G_f, varies from 1.05 to 1.25. 281

FIGURE 4.19 Variation of velocity pressure, q_z, versus wind speed. 282

FIGURE 4.20 Comparison of topographic effects, Exposure C, wind on narrow face. 283

FIGURE 4.21 Comparison of topographic effect, Exposure C, wind on broad face. 283

FIGURE 4.22 Variation of gust factor, G_f. 289

FIGURE 4.23 Building period versus height. 302

FIGURE 4.24 Design wind pressure perpendicular to 200 ft wall. *Note:* $G_f = 1.24$, $I_w = 1.15$. 303

FIGURE 4.25 Design wind pressure perpendicular to 120 ft wall. *Note:* $G_f = 1.24$, $I_w = 1.15$. 304

FIGURE 4.26 External wind pressure coefficient C_p. (Adapted from NBCC 2005.) 306

FIGURE 4.27 Exposure factor C_{eH}. (From NBCC 2005.) 308

FIGURE 4.28 Background turbulence factor, B, as a function of height and width of building. (From NBCC 2005.) 308

FIGURE 4.29 Size reduction factor, s, as a function of width, height, and reduced frequency of the building. (From NBCC 2005.) 309

FIGURE 4.30 Gust energy ratio, F, as a function of wave number. (From NBCC 2005.) 309

FIGURE 4.31 Peak factor, g_p, as a function of average fluctuation rate. (From NBCC 2005.) 311

FIGURE 4.32 Wind induced peak accelerations; NBCC 2005 procedure: (a) 30-story building, wind on narrow face; (b) 30-story building, wind on broad face; (c) 60-story building with equal plan dimensions. 314

FIGURE 4.33 Schematics of 33 m-long wind tunnel; overall size 33 m × 2.4 m × 2.15 m. 315

FIGURE 4.34 Schematics of 64 m-long wind tunnel; over all size 64 m × 15 m × 6 m. 316

FIGURE 4.35 Schematics of two additional wind tunnels. 316

FIGURE 4.36 Photographs of rigid model in wind tunnel. Chifley Plaza, Sydney. (Courtesy of CPP Wind Engineering & Air Quality Consultants.) 316

FIGURE 4.37 Model in wind tunnel: DIFC Dubai. (Courtesy of CPP Wind Engineering & Air Quality Consultants.) 317

FIGURE 4.38 Close-up of model in wind tunnel: Pentominium, Dubai. (Courtesy of CPP Wind Engineering & Air Quality Consultants.) 317

FIGURE 4.39 Mode in wind tunnel: Shams, Dubai. 318

FIGURE 4.40 (a) Rigid model in wind tunnel. (b) Close-up of a rigid model. (Photos courtesy of Rowan, Williams, Davis, and Irwin, RWDI.) 318

FIGURE 4.41 Simple stick aeroelastic model. 323

FIGURE 4.42 Aeroelastic model. 323

FIGURE 4.43 Rigid aeroelastic model mounted as a flexible steel bar. 324

FIGURE 4.44 High-frequency force balance model. 324

FIGURE 4.45 Detail view of high-frequency force balance model: (a,b) Close-up view of instrumentation and (c) model. 325

FIGURE 4.46 Schematic of five-component force balance model. 325

FIGURE 4.47 Aeroelastic model: cutaway view. 326

FIGURE 4.48 Aeroelastic model with provisions for simulating torsion. 326

FIGURE 4.49 Aeroelastic model with rotation simulators. 327

FIGURE 4.50 Aeroelastic model; schematic section. 327

FIGURE 4.51 Aeroelastic model: (a) A proposed tower in Chicago and (b) close-up view of instrumentation. 328

FIGURE 4.52 Simple stick aeroelastic model. 328

FIGURE 4.53 Aeroelastic model. 329

FIGURE 4.54 Pedestrian reactions (a–d). 333

FIGURE 4.55 Measured peak accelerations for the Allied Bank Tower during hurricane Alicia. 336

FIGURE 4.56 Mode shapes using different units. (a) Mode dominated by translation, (b) mode dominated by twist. 337

FIGURE 4.57 Building typical floor plan. 338

FIGURE 4.58 Tabulation of building properties. 339

FIGURE 4.59 Modes shapes: (a) Modes 1, 2, and 3, (b) modes 4, 5, and 6. 341

FIGURE 4.60 Tabulation of dynamic properties. 343

FIGURE 5.1 Building behavior during earthquakes. 348

FIGURE 5.2 Schematic representation of seismic forces. 349

FIGURE 5.3 Concept of 100%g (1g). 351

FIGURE 5.4 Linear viscous damper. 351

FIGURE 5.5 Bilinear force–displacement hysteresis loop. 352

FIGURE 5.6 Plan irregularities: (a) geometric irregularities, (b) irregularity due to mass-resistance eccentricity, and (c) irregularity due to discontinuity in diaphragm stiffness. 356

FIGURE 5.7 Elevation irregularities: (a) abrupt change in geometry, (b) large difference in floor masses, and (c) large difference in story stiffnesses. 357

FIGURE 5.8 Diaphragm drag and chord reinforcement for north–south seismic loads. 358

FIGURE 5.9 Diaphragm web failure due to large opening. 359

FIGURE 5.10 Hysteric behavior: (a) curve representing large energy dissipation and (b) curve representing limited energy dissipation. 360

FIGURE 5.11 Examples of nonuniform ductility in structural systems due to vertical discontinuities. (Adapted from SEAOC Blue Book, 1999 Edition.). 361

FIGURE 5.12 Reentrant corners in L-, T-, and H-shaped buildings. (As a solution, add collector elements and/or stiffen end walls.) 364

FIGURE 5.13 Idealized SDOF system. 365

FIGURE 5.14 Undamped free vibrations of SDOF system. 365

FIGURE 5.15 Damped free vibration of SDOF system. 365

FIGURE 5.16 Representation of a multi-mass system by a single-mass system: (a) fundamental mode of a multi-mass system and (b) equivalent single-mass system. 366

FIGURE 5.17 Graphical description of response spectrum. 367

FIGURE 5.18 Concept of response spectrum (a, b) buildings of varying heights and (c, d) pendulums of varying lengths. 368

FIGURE 5.19 Acceleration spectrum: El Centro earthquake. 369

FIGURE 5.20 Examples of SDOF systems: (a) elevated water tank. (b) Restaurant atop tall concrete core. Note from Figure 5.19, the acceleration = 26.25 ft/s^2 for $T = 0.5$ s and $\beta = 0.05$ (water tank), and the acceleration = 11.25 ft/s^2 for $T = 1.00$ s and $\beta = 0.10$ (restaurant). 369

FIGURE 5.21 Recorded ground acceleration: El Centro earthquake. 370

FIGURE 5.22 (a) Ground acceleration; (b) deformation response of three SDOF systems with $\beta = 2\%$ and $T_n = 0.5$, 1, and 2 s; and (c) deformation response spectrum for $\beta = 2\%$. 372

FIGURE 5.23 Response spectra ($\beta = 2\%$) for El Centro ground motion: (a) deformation response spectrum, (b) pseudo-velocity response spectrum, and (c) pseudo-acceleration response spectrum. 373

FIGURE 5.24 Pseudo-acceleration response of SDOF systems to El Centro ground motion. 375

FIGURE 5.25 Combined *DVA* response for El Centro ground motion; $\beta = 2\%$. 376

FIGURE 5.26 Tripartite site-specific response spectra: (a) earthquake A, (b) earthquake B, (c) earthquake C, and (d) earthquake D. 377

FIGURE 5.27 Velocity, displacement, and acceleration readout from response spectra. 379

FIGURE 5.28 Idealized response spectrum for El Centro ground motion. 380

FIGURE 5.29 Schematic response of rigid and flexible systems. (a) Rigid system, acceleration at top is nearly equal to the ground acceleration; (b) flexible system, structural response is most directly related to ground displacement. 380

FIGURE 5.30 MCE ground motion for the United States, 0.2 s. Spectral response acceleration, S_S, as a percent of gravity, site class B with 5% critical damping. 387

FIGURE 5.31 MCE ground motion for the United States, 1.0 s. Spectral response acceleration, S_1, as a percent of gravity, site class B with 5% critical damping. 387

FIGURE 5.32 Design response spectrum. 389

FIGURE 5.33 Locations of cities cited in Tables 5.20, 5.21 and Figure 5.34. 414

FIGURE 5.34 Design response spectrum for selected cities in the United States. 415

FIGURE 5.35 Different systems used along two orthogonal axes; use appropriate value of *R* for each system. 428

FIGURE 5.36 Different systems used over the height of a structure. The response modification coefficient, *R*, for any story above, shall not exceed the lowest value, in the direction under consideration. 428

FIGURE 5.37 (a) Highly redundant structure and (b) not-so-redundant structure. 447

FIGURE 5.38 Permitted analysis procedures for seismic design. (Developed from ASCE 7-05, Table 12.6-1.) 453

FIGURE 5.39 Tributary weights for seismic dead-load calculation. 456

FIGURE 5.40 Column deformation for use in compatibility considerations. Deformation of column = building deflection Δ_B + diaphragm deflection Δ_D. (Adapted from SEAOC Blue Book, 1999 edition.) 462

FIGURE 5.41 Deformation compatibility consideration of foundation flexibility. (Adapted from SEAOC Blue Book, 1999 edition.) 463

FIGURE 5.42 Eccentric collector: F_c is the axial force concentric to wall and F_e is the axial force eccentric to wall. 466

FIGURE 5.43 Diaphragm design example: (a) floor plan, (b) equivalent loads due to primary diaphragm action, (c) equivalent loads due to torsional effects, (d) final equivalent loads (= (b) + (c)), (e) shear diagram, and (f) bending moment diagram. 469

FIGURE 5.44 Summary of unit shears. 472

FIGURE 5.45 Two-story example: dynamic analysis hand calculations. 479

FIGURE 5.46 Vibration modes, two-story example: (a) first mode and (b) second mode. 482

FIGURE 5.47 Distribution of modal shears: (a) first mode and (b) second mode. 487

FIGURE 5.48 Three-story building example: dynamic analysis. 488

FIGURE 5.49 Three-story building: response spectrum. 489

FIGURE 5.50 Three-story building: modal analysis to determine base shears. 490

FIGURE 5.51 Three-story building: modal analysis to determine story forces, accelerations, and displacements. 491

FIGURE 5.52 Three-story building: comparison of modal story shears and the SRSS story shears. 491

FIGURE 5.53 Seven-story building example: dynamic analysis. 492

FIGURE 5.54 Response spectrum for seven-story building example: (a) acceleration spectrum, (b) tripartite diagram, and (c) response spectra numerical representation. 493

FIGURE 5.55 Seven-story building: modal analysis to determine base shears. 494

FIGURE 5.56 Seven-story building: first-mode forces and displacements. 495

FIGURE 5.57 Seven-story building: second-mode forces and displacements. 496

FIGURE 5.58 Seven-story building: third-mode forces and displacements. 497

FIGURE 5.59 Seven-story building, modal analysis summary: (a) modal story forces, kip; (b) modal story shears, kip; (c) modal story overturning moments, kip-ft; (d) modal story accelerations, g; and (e) modal lateral displacement, inches. 498

FIGURE 5.60 Time-load functions. (a) Rectangular pulse (b) triangular pulse and (c) constant force with finite rise time. *Note:* F_t = Load function, t_d = time function. 499

FIGURE 5.61 Dynamic response of a cantilever. *Note:* Dynamic Load Factor, DLF, is equal to 2.0 for a load applied instantaneously. 499

FIGURE 5.62 Dynamic load factor, DLF, for common time-load functions. t_d = time duration of pulse, T = fundamental period of the system to which load is applied. 500

FIGURE 5.63 Cantiveler column with weight at top. 501

FIGURE 5.64 Acceleration response spectrum. 502

FIGURE 5.65 Single-bay single-story portal frame. 505

FIGURE 5.66 Analytical models for SDOF system: (a) model in horizontal position and (b) model in vertical position. 506

FIGURE 5.67 Damped oscillator: (a) analytical model and (b) forces in equilibrium. 506

FIGURE 5.68 MDOF: (a) multistory analytical model with lumped masses. 509

FIGURE 5.69 Generalized displacement of a simply supported beam: (a) loading, (b) full-sine curve, (c) half-sine curve, (d) one-third-sine curve, and (e) one-fourth-sine curve. 512

FIGURE 5.70 Two-story lumped-mass system illustrating Betti's reciprocal theorem: (a) lumped model, (b) forces during first mode of vibration, and (c) forces acting during second mode of vibration. 512

FIGURE 5.71 Two-story shear building, free vibrations: (a) building with masses, (b) mathematical model, and (c) free-body diagram with masses. 514

FIGURE 5.72 Major earthquake faults in California. 520

FIGURE 6.1 Shear strength of joints. 527

FIGURE 6.2 Structural integrity reinforcement in flat slabs without beams. 530

FIGURE 6.3 Structural integrity reinforcement in joists. 530

FIGURE 6.4 Integrity reinforcement in perimeter beams: (a) perimeter beam elevation and (b) Section 1. 531

FIGURE 6.5 Integrity reinforcement in beams other than perimeter beams. 531

FIGURE 6.6 IMRF: flexural reinforcement requirements for frame beams. 536

FIGURE 6.7 IMRF: transverse reinforcement requirements for frame beams. 536

FIGURE 6.8 IMRF: transverse reinforcement requirements for frame columns. 537

FIGURE 6.9 Seismic detailing requirements for two-way slabs in areas of moderate seismic risk; flat slab-beams not permitted in UBC zones 3 and 4, or for buildings assigned to SDC C, D, E, or F. 538

FIGURE 6.10 Seismic detailing requirements for two-way slabs in areas of moderate seismic risk: column strip. 539

FIGURE 6.11 Seismic detailing requirements for two-way slabs in areas of moderate seismic risk: middle strip. 539

FIGURE 6.12 Frame beam: general requirements, special moment frame. 540

FIGURE 6.13 Frame beam: transverse reinforcement requirements, special moment frame. 542

FIGURE 6.14 Arrangement of hoops and crossties: frame beams; special moment frame. 543

FIGURE 6.15 Frame column: detailing requirements, special moment frame. 544

FIGURE 6.16 Examples of minimum transverse reinforcement in frame columns of SMRF. *Note:* $f'_c = 5$ ksi, $f_y = 60$ ksi. Vertical spacing of ties = 4 in. Ties #5 for 24 in. × 24 in. and 30 in. × 30 in. columns. #4 for 38 in. × 38 in. and 44 in. × 44 in. columns. 545

FIGURE 6.17 Ductile frame, SMRF: schematic reinforcement detail. 547

FIGURE 6.18 Frame beam and column example; OMF: (a) plan and (b) elevation. 555

FIGURE 6.19 Design example, frame beam; OMF. For this example problem, although by calculations no shear reinforcement is required in the midsection of the beam, it is good practice to provide #3 four-legged stirrups at 15 in. spacing. 557

FIGURE 6.20 Design example, frame column; OMF. 559

FIGURE 6.21 Frame beam and column example; IMF. 559

FIGURE 6.22 Design example, frame beam; IMF. 561

FIGURE 6.23 Design example, frame column; OMF. *Note:* ℓ_o is the same as for columns of SMRF. There is no requirement to splice column bars at mid-height. 563

FIGURE 6.24 Shear wall: low-to-moderate seismic zones (SDC A, B, or C). *Note:* Vertical reinforcement of #7 @ 9 in. at each end is enclosed by lateral ties, since the reinforcement area of eight #7 vertical bars equal to $8 \times 0.6 = 4.8\,\text{in.}^2$ is greater than 0.01 times the area of concrete = $12 \times 30 = 360\,\text{in.}^2$ (see ACI 318-05 Section 14.3.6). 564

FIGURE 6.25 Design example, frame beam; special moment frame. 570

FIGURE 6.26 Design example, frame column; special moment frame. 571

FIGURE 6.27 Column panel shear forces. 575

FIGURE 6.28 Beam–column joint analysis: (a) forces and moments, case 1; (b) forces and moments, case 2; (c) resolved forces, case 1; and (d) resolved forces, case 2. 576

FIGURE 6.29 Beam–column joint; special moment frame. Transverse reinforcing in the joint is the same as for the frame column. A 50% reduction is allowed if the joint is confined on all the four faces. Maximum spacing of transverse reinforcement is equal to 6 in. 577

FIGURE 6.30 Design example; partial shear wall elevation and plan. 578

FIGURE 6.31 (a) Shear wall load/moment interaction diagram and (b) cross section of wall. 582

FIGURE 6.32 Shear wall example; schematic reinforcement. 585

FIGURE 6.33 Wall elevation showing schematic placement of reinforcement. 587

FIGURE 6.34 Coupled shear walls: (a) partial elevation and (b) plan. 588

FIGURE 6.35 Geometry for calculating α, the angle between the diagonal reinforcement and the longitudinal axis of the coupling beam. *Note*: $\tan\alpha = h/2 - x/\cos\alpha/\ell_n/2$ (solve for α by trial and error). 590

FIGURE 6.36 Parameters for calculating diagonal beam reinforcement. 592

FIGURE 6.37 Coupling beam with diagonal reinforcement. Each diagonal reinforcement must consist of at least four bars with closely spaced ties. Use wider closed ties or crossties at central intersection. Use crossties to confine development length ℓ_d. 593

FIGURE 6.38 Section 1.1. Schematic section through coupling beam. The purpose of this sketch is to ensure that the wall is thick enough for the proper placement of wall and diagonal beam reinforcement and concrete. 594

FIGURE 6.39 (a) Wall pier W1, load/moment interaction diagram and (b) cross section of wall pier W1. 596

FIGURE 6.40 Schematic reinforcement layout for the wall pier, example 2. 599

FIGURE 6.41 Exterior joint detailing; schematics: (a) plan and (b) section. 599

FIGURE 6.42 Interior joint detailing; schematics: (a) plan and (b) section. 600

FIGURE 6.43 Beam bar placement. 601

FIGURE 6.44 SDC D, E, or F: Frame columns. 602

FIGURE 6.45 SDC D, E, or F: Gravity columns in which induced moments and shears due to deformation compatibility, combined with factored gravity moments and shears do not exceed design moments and shears. 602

FIGURE 6.46 Typical caisson detail. 603

FIGURE 6.47 SDC C: Frame columns and, SDC D, E, or F: Gravity columns, that is, columns not designed as part of a lateral system in which: (1) Induced moment or shear due to deformation compatibility exceeds design moment or shear. (2) Induced moments and shears to deformation compatibility are not calculated. 604

FIGURE 6.48 Design shears for IMFs: (a) moment frame, (b) loads on frame beams, (c) beam shear, (d) loads on frame columns, and (e) column shear. 607

FIGURE 6.49 Two-way slabs: Effective width concept placement for reinforcement: (a) corner column and (b) interior column. 608

FIGURE 6.50 Reinforcement placement in two-way slabs without beams. (Applies to both top and bottom reinforcement.) *Note*: h = slab thickness. 608

FIGURE 6.51 Two-way slabs: Arrangement of reinforcement in (a) column and (b) middle strips. 609

FIGURE 6.52 Definition of effective width of wide beams for placement of transverse reinforcement: (a) plan and (b) Section A-A. 610

FIGURE 6.53 Design shears for special moment frames: (a) moment frame, (b) loads on frame beams (same as for IMFs), (c) beam shear, (d) loads on frame columns, and (e) column shear. 612

FIGURE 6.54 Diagonal reinforcement in coupling beams. Detail 1: ACI 318-05 required confinement of individual diagonals. Detail 2: As an option, ACI 318-08 allows full-depth confinement of diagonals. 614

FIGURE 6.55 Coupling beam diagonal reinforcement. Detail 1: Confinement of individual diagonals: (a) elevation and (b) section. 615

FIGURE 6.56 Coupling beam diagonal reinforcement. Detail 2: Full-depth confinement of diagonals: (a) elevation and (b) section. 616

FIGURE 7.1 Idealized earthquake force–displacement relationships. 620

FIGURE 7.2 Superimposed diaphragm slab at an existing concrete wall. 625

FIGURE 7.3 Diaphragm chord for existing concrete slab. 625

FIGURE 7.4 Strengthening of openings in a superimposed diaphragm; (a) Section, (b) Plan. 626

FIGURE 7.5 New chords at reentrant corners. 626

FIGURE 7.6 Common methods for upgrading buildings with open storefronts. 631

FIGURE 7.7 (N) openings in an (E) 3-story concrete shear wall building. The seismic upgrade consisted of providing concrete overlay to restore shear capacity of walls and adding boundary elements around (N) openings: (a) wall elevation, (b) concrete overlay with (N) beam below (E) slab; (c) concrete overlay with (N) beam above (E) slab, (d) plan detail at (N) boundary element. *Note:* (E), existing, (N), new. 670

FIGURE 7.8 Seismic upgrade of a concrete hospital building with an external concrete moment frame. Modifications were restricted to the periphery of the building to keep the building operational. (a) Plan showing (N) foundations, (N) concrete overlay in the transverse direction, and (N) moment frames in the longitudinal direction. (b) Enlarged plan at (N) coupling beam and shear wall overlay. (c) Section through longitudinal frame. (d) Section through transverse wall. (e) Connection between (N) and (E) frame. 672

FIGURE 7.9 Fiber wrap of a transfer girder: (a) elevation and (b) section. 675

FIGURE 7.10 Strengthening of existing connecting beams in reinforced concrete walls. 676

FIGURE 7.11 Upgrading of an existing pile foundation. Add additional piles or piers, remove, replace, or enlarge existing pile caps. *Note*: Existing framing to be temporarily shored to permit removal of existing pile cap and column base plate. Drive new piles; weld new base plate and moment connection to column; pour new pile cap; and drypack under base plate. 677

FIGURE 7.12 Strengthening of an existing concrete frame building by adding (N) a reinforced concrete shear wall: (a) section a–a; (b) section b–b, and (c) elevation. 678

FIGURE 7.13 New concrete shear wall at existing slab. 679

FIGURE 7.14 Strengthening of existing reinforced concrete wall or piers. 679

FIGURE 7.15 Strengthening of existing reinforced concrete walls by filling in openings. 680

FIGURE 7.16 Jacketing of circular column. 680

FIGURE 7.17 Braced structural steel buttresses to strengthen an existing reinforced concrete building. 681

FIGURE 7.18 (a) Building plan showing locations of (N) steel props. (b) Section A; elevation of (N) steel prop. 682

FIGURE 7.19 Upgrading an existing building with external frames. 683

FIGURE 8.1 Empire State Building, New York City. 686

FIGURE 8.2 World Trade Center Twin Towers, New York City. 686

FIGURE 8.3 Petronas Towers, Kuala Lumpur. 687

FIGURE 8.4 Burj Dubai. 687
FIGURE 8.5 Monadock Building, Chicago. 689
FIGURE 8.6 Woolworth Building, New York City. 690
FIGURE 8.7 Chrysler Building, New York City. 690
FIGURE 8.8 Sears Tower, Chicago. 691
FIGURE 8.9 Rockefeller Center, New York City. 693
FIGURE 8.10 John Hancock Center, Chicago. 694
FIGURE 8.11 Chicago skyline. 695
FIGURE 8.12 Concept of premium for height. 697
FIGURE 8.13 Tubular system: Closely spaced exterior columns interconnected with deep spandrels. 701
FIGURE 8.14 (a) Transamerica Tower, San Francisco, California. (b) Transamerica Tower, schematic. 702
FIGURE 8.15 City Corp Building, New York City. 703
FIGURE 8.16 Structural system study: 62-story building. (a) Plan and (b) elevation. 706
FIGURE 8.17 Exterior braced tube: plan. 706
FIGURE 8.18 Isometric of exterior braced tube. 707
FIGURE 8.19 Isometric view of four-story high braced tube. 707
FIGURE 8.20 Interior cross bracing system: plan. 708
FIGURE 8.21 Isometric of interior cross bracing system. 708
FIGURE 8.22 Interacting framed tube and braced frame. 709
FIGURE 8.23 Isometric of framed tube and brace frame. 709
FIGURE 8.24 Framed tube with 10 and 15 ft column spacing. 710
FIGURE 8.25 Framed tube with 10 ft column spacing. 710
FIGURE 8.26 Twin-tube system with 10 ft column spacing. 711
FIGURE 8.27 Framed tube with 15 ft column spacing. 711
FIGURE 8.28 Twin-tube system with 15 ft column spacing. 712
FIGURE 8.29 Framed tube with 20 ft column spacing. 712
FIGURE 8.30 Twin-tube system with 20 ft column spacing. 712
FIGURE 8.31 Shear wall frame interacting system. 713
FIGURE 8.32 Moment frame and braced core system. 713
FIGURE 8.33 Outrigger and belt truss system: schematic plan. 714
FIGURE 8.34 Jumbo column scheme. 715
FIGURE 8.35 Structural concept of tall building. 719
FIGURE 8.36 Building shear resistance: (a) building must not break and (b) building must not have excessive shear deflection. 719

FIGURE 8.37 Bending resistance of building: (a) building must not overturn, (b) columns must not fail in tension or compression, and (c) bending deflection must not be excessive. 720

FIGURE 8.38 Building plan forms: (a) uniform distribution of columns and (b) columns concentrated at the edges. 721

FIGURE 8.39 Column layout and BRI: (a) square building with corner columns, BRI = 100; (b) traditional building of the 1930s, (c) modern tube building, BRI = 33; (d) Sears Towers, BRI = 33; (e) City Corp Tower, BRI = 31; (f) building with corner and core columns, BRI = 56; and (g) Bank of Southwest Tower, BRI = 63. 722

FIGURE 8.40 Tall building shear systems (a) shear wall system; (b) diagonal web system; (c) web system with diagonals and horizontals; (d–g) rigid frames. 723

FIGURE 8.41 Empire State Building, New York City. 725

FIGURE 8.42 South Walker Tower, Chicago, Illinois. 725

FIGURE 8.43 Miglin-Beitler Tower, Chicago: (a) elevation and (b) plan; (c) typical floor framing plan, and (d) lateral loads; (i) building elevation (ii) effective static wind pressure (iii) shear force and (iv) overturning moment. 726

FIGURE 8.44 Trump Tower, Chicago: (a) architectural rendering, (b) photograph, (c) 20th floor framing plan, (d) schematic elevation, (e) schematic section, (f) schematic 3-D. 728

FIGURE 8.45 Jin Mao Tower, Shanghai, China: (a) typical office floor framing plan; (b) structural system elevation. 732

FIGURE 8.46 Petronas Towers, Malaysia. (a) Elevation and (b) structural system plan. (c) Height comparison: (1) Petronas Towers and (2) Sears Tower, Chicago. 735

FIGURE 8.47 Central Plaza, Hong Kong: (a) elevation; (b, c) floor plans. 737

FIGURE 8.48 Singapore Treasury Building, Singapore: (a) schematic section and (b) typical floor framing plan. 739

FIGURE 8.49 City Spire, New York City. 741

FIGURE 8.50 NCNB Tower, North Carolina. 742

FIGURE 8.51 Museum Tower, Los Angeles: (a) building elevation; (b) lateral bracing system; (c) typical floor framing plan. 743

FIGURE 8.52 Vdara tower; MGM Block B, Las Vegas, Nevada: (a) floor plan and (b–d) construction photos. 745

FIGURE 8.53 City Bank Plaza, Hong Kong. 746

FIGURE 8.54 Trump Tower, New York City. 748

FIGURE 8.55 Two Prudential Plaza, Chicago. 749

FIGURE 8.56 Cent Trust Tower, Miami, Florida. (a) Schematic plan, (b) shear wall layout, and (c) column transfer. 750

FIGURE 8.57 Metropolitan Tower, New York City. (a, b) Framing plans. 752

FIGURE 8.58 Carnegie Hall Tower, New York City. (a, b) Framing plans. 753

FIGURE 8.59 Hopewell Center, Hong Kong. Schematic Plan. 754

FIGURE 8.60 Cobalt Condominiums, Minneapolis, Minnesota: (a) photograph of completed building, (b) erection of prestressed precast truss, (c) eighth floor framing plan, (d) interior truss, and (e) truss reinforcement details. 755

FIGURE 8.61 The Cosmopolitan Resort & Casino, Las Vegas, Nevada. (a) East tower, (b) West tower, (c–f) project photographs. 757

FIGURE 8.62 Elysian Hotel and Private Residences, Chicago. (a) Schematic foundation plan, (b) foundation bracing system, (c) mat foundation, (d) levels 5–10 post-tensioning plan, and (e) levels 30–34 post-tensioning plan. 762

FIGURE 8.63 Shangri La New York City (610 Lexington Avenue), New York City. (a) Schematic elevation, (b) structural system consists of (1) 10 in. thick conventionally reinforced concrete slab, (2) posttensioned shear walls, (3) concrete belt and hat walls, (c) typical floor framing plan, (d) core framing details, (e) high-rise tower connection to 10-story podium (f) schematics of folded pendulum tuned mass damper, TMD; two 375 t TMDs will be located on either side of the core, below the roof parapet. 765

FIGURE 8.64 Millennium Tower, 310 Mission Street, San Francisco, California: (a) Architectural rendering, (b) building plan, (c) isometric of shear walls and outriggers, (d) construction photograph, (e) outrigger reinforcement, (f) building section, (g) sloping columns, (h) test specimens (i) conventional hoops and ties, (j) welded reinforcement grid, (k) photograph of link beams, and (l) details of outrigger reinforcement. 769

FIGURE 8.65 Al Bateen Towers, Dubai, UAE: (a) upper residential and hotel tower framing plans, (b) mid-residential and hotel room framing plans, (c) lower-residential and hotel room framing plans, (d) garage section. (e) residential deflected shape (wind on broad face), (f) residential tower effective applied wind pressure versus height, (g) hotel tower effective applied wind pressure versus height, and (h) residential tower, 10-year inter-story drift. 774

FIGURE 8.66 SRZ Tower, Dubai, UAE: (a) 1st to 30th floor plan, (b) E-line elevation, (c) maximum stress in reinforced concrete core, (d) maximum stress in outrigger columns, (e) story shear (transverse direction), (f) overturning moment (transverse direction), (g) story shear (longitudinal direction), and (h) overturning moment (longitudinal direction). 779

FIGURE 8.67 The Four Seasons hotel and tower, Miami, Florida: (a) lower level framing plan; (b) Upper level framing plan, and (c) photograph. 784

FIGURE 8.68 Burj Dubai: (a) Boundary-layer wind-tunnel facility, (b) wind tunnel high frequency force balance model, (c) wind tunnel aeroelastic model, (d) wind tunnel cladding model, (e) pedestrian level wind study, (f) photograph, (g), schematic floor plan showing six major wind directions, and (h) view from top of Burj Dubai. 786

FIGURE 8.69 Comparative heights of several of the world's tallest buildings. 792

FIGURE 9.1 Schematics of viscoelastic damper. 794

FIGURE 9.2 Viscoelastic dampers in World Trade Center Towers. 794

FIGURE 9.3 Viscoelastic dampers, schematics. 795

FIGURE 9.4 Tuned mass damper (TMD) for City Corp, New York: (a) building elevation, (b) first-mode response, (c) TMD atop the building, and (d) plan. 797

FIGURE 9.5 Schematics of TMD, City Corp Center, New York. 797

FIGURE 9.6 Dual TMD: John Hancock Tower, Boston, MA. 798

FIGURE 9.7 Sloshing water damper, schematics. 798

FIGURE 9.8 Tuned liquid column dampers, TLCD, Wall Center, Vancouver, BC. 800

FIGURE 9.9 TLCD, Wall Center, Vancouver, BC. 801

FIGURE 9.10 Highcliff apartments, Hong Kong. 801

FIGURE 9.11 (a) Simple pendulum damper. (b) Hydraulic dampers attached to mass block. 802

FIGURE 9.12 Spherical tuned mass pendulum damper, TMPD, Taipei Financial Center. 802

FIGURE 9.13 Taipei 101, tuned mass pendulum damper: (a) TMPD, (b) TMPD principle, and (c) schematics. 803

FIGURE 9.14 (a) Simple pendulum damper. (b) Nested pendulum damper. 804

FIGURE 9.15 Design concept for base-isolated buildings. 805

FIGURE 9.16 Comparison of response of fixed-base and base-isolated building: (a) fixed-base and (b) base-isolated. 806

FIGURE 9.17 Moat around base-isolated building. 807

FIGURE 9.18 Moat detail at ground level. 808

FIGURE 9.19 Elastomeric isolators: (a) high-damping rubber bearing made by bonding rubber sheets to steel plates and (b) high-damping rubber bearing. 809

FIGURE 9.20 (a) Lead-rubber bearing under interior columns. (b) Lead-rubber bearing for an interior column for a five-story steel frame building: approximate dimensions. (c) Natural rubber bearing with press-fit lead core. 809

FIGURE 9.21 Single stage friction pendulum. 811

FIGURE 9.22 Sliding bearing: friction pendulum system base-isolation. 811

FIGURE 9.23 Friction pendulum system, details. 811

FIGURE 9.24 Installation details, FPS under existing interior columns. 812

FIGURE 9.25 Base isolator operating in concert with viscous damper. 812

FIGURE 9.26 (a) Base isolation: ASCE 7-05 nomenclature. (b) Isolator displacements: ASCE 7-05 definitions. 814

FIGURE 9.27 Isolator displacement terminology. 814

FIGURE 9.28 Idealized force–displacement relationship for base-isolation systems: (a) hysteretic system and (b) viscous system. 821

FIGURE 9.29 Fluid viscous damper consisting of a piston in a damping housing filled with a compound of silicone or similar type of fluid. 830

FIGURE 9.30 Example frame; dimensions and properties. 834

FIGURE 9.31 Portal method. (a) Moments and forces at roof level. (b) Moments and forces at level 15. (c) Moments and forces at level 2. 835

FIGURE 9.32 Cantilever method: (a) moments and forces at roof level, (b) moments and forces at level 29, (c) moments and forces at level 15, and (d) Moments and forces at level 2. 836

FIGURE 9.33 Moments and forces at (a) roof level, (b) level 29, (c) level 15, and (d) level 2. 838

FIGURE 9.34 Portal frame shear deflections: (a) Frame subjected to lateral loads and (b) typical story segment. 839

FIGURE 9.35 Lateral deflection due to bending of columns. 839

FIGURE 9.36 (a) Lateral deflection due to girder rotations and (b) deflection comparison (30-story frame). 840

FIGURE 9.37 Example portal frame for deflection calculations. 842

FIGURE 9.38 Equivalent cantilever. 843

FIGURE 9.39 Shear deformation of a cantilever of unit height *h*. 843

FIGURE 9.40 Deflection comparison. 844

FIGURE 9.41 Infinitely rigid panel zones. 845

FIGURE 9.42 Framed tube: (a) axial stress distribution with shear lag and (b) axial stresses distribution in equivalent channels without shear lag. 846

FIGURE 9.43 Axial forces in tube columns assuming two equivalent curvilinear channels. 847

FIGURE 9.44 Axial forces in tube columns from three dimensional analysis of framed tube. 847

FIGURE 9.45 (a) Singly symmetrical shear wall. (b) Shear center *S* as it relates to centroid *C*. (c) Shear forces in flanges. (d) Shear center in a T-section shear wall. (e) Shear center concept; shear walls bending about a common neutral axis. 848

FIGURE 9.46 (a,b) Twisting of circular shaft. (c) Torsion failure of a brittle material by tension cracking along a 45° helical surface. 852

FIGURE 9.47 Variation of torsional shear stresses in circular shaft. 852

FIGURE 9.48 Shear flow in a rectangular section. 852

FIGURE 9.49 Shear flow in thin-walled sections: load at shear center. 853

FIGURE 9.50 Shear center in C-section. 853

FIGURE 9.51 Shear stresses in hollow section: load at shear center. 854

FIGURE 9.52 Shear stresses in hollow rectangular section. 855

FIGURE 9.53 Shear flow in cellular sections: (a) load at shear center, (b) section rendered open with two cuts, (c) shear flows required for compatibility, and (d) final shear flow = (b) + (c). 855

FIGURE 9.54 (a) Warping of solid beams. (b) Thin rectangular beam: bending moment due to warping restraint. 856

FIGURE 9.55 Bimoment in wide flange column. 857

FIGURE 9.56 I-section core. 857

FIGURE 9.57 Core properties. 858

FIGURE 9.58 (a) Bending of flanges due to torque. (b) Shear forces due to warping torsion. 859

FIGURE 9.59 (a) Section profile, (b) sectorial coordinate ω_s diagram, (c) singly symmetric curve, (d) ω_s diagram, (e) *y*-coordinate diagram, (f) principal sectional coordinates, and (g) sectorial coordinates for common profiles. 862

FIGURE 9.60 Calculation of sectorial properties: (a) cross section, (b) ω_p diagram, (c) *x*-coordinate diagram, and (d) principal sectorial coordinate $\omega_{\underline{s}}$ diagram. 866

FIGURE 9.61 Cantilever column of solid section: (a) vertical load at corner, (b) symmetrical axial loading, (c) bending about *x*-axis, (d) bending about *y*-axis, and (e) self-equilibrating loading producing bimoment. 868

FIGURE 9.62 I-shaped cantilever beam: (a) vertical load at a corner, (b) symmetrical axial loading, (c) bending about *x*-axis, (d) bending about *y*-axis, and (e) self-equilibrating loading producing bimoment. 869

FIGURE 9.63 Plan section of I-shaped column: (a) displacement of flanges due to bimoment load and (b) rotation due to geometric compatibility between flanges and web. 869

FIGURE 9.64 (a) Twin-core example. (b) Core properties. (c) ω_s diagram (sectorial coordinates). (d) Comparison of stresses: (i) bending stress σ_b and (ii) warping stress σ_ω. 871

FIGURE 9.65 Torsion example; randomly distributed shear walls. (a) Plan, (b) Warping coordinates. (c) Axial stresses due to torsion. (d) Rotation comparison. 876

FIGURE 9.66 (a) 12×12 stiffness matrix for prismatic three-dimensional element, (b) coordinate axes, and (c) positive sign convention. 884

FIGURE 9.67 (a) 14×14 stiffness matrix for thin walled open section, (b) coordinate axes, and (c) positive sign convention. 885

List of Tables

TABLE 1.1 Comparison of Flexural Reinforcement 13

TABLE 2.1 Minimum Thickness of Beams or One-Way Slabs Unless Deflections Are Computed 95

TABLE 2.2 Percentage of Positive Moment to Column Strip, Interior Span 104

TABLE 2.3 Percentage of Negative Moment to Column Strip at an Interior Support 104

TABLE 2.4 Percentage of Negative Moment to Column Strip at an Exterior Support 105

TABLE 2.5 Minimum Thickness of Slabs Without Interior Beams 107

TABLE 2.6 Approximate Span Depth Ratios for Posttensioned Systems 107

TABLE 2.7 Sample Unit Quantities for Hotels 114

TABLE 2.8 Unit Quantity of Reinforcement in Mat Foundations 196

TABLE 2.9 Unit Quantity of Reinforcement in Columns 197

TABLE 4.1 Probability of Exceeding Design Wind Speed during Design Life of Building 261

TABLE 4.2 Terrain Exposure Constants 280

TABLE 4.3 Design Wind Pressures, Graphical Procedure Using ASCE 7-05 291

TABLE 4.4 Buildings' Characteristics and Wind Environment 299

TABLE 4.5 Design Parameters for Example Buildings 300

TABLE 4.6 Comparison of Dynamic Response of Example Buildings 300

TABLE 5.1 Horizontal Structural Irregularities 362

TABLE 5.2 Vertical Structural Irregularities 363

TABLE 5.3 Occupancy Category of Buildings and Importance Factors 382

TABLE 5.4 Site Coefficient, F_a 388

TABLE 5.5 Site Coefficient, F_v 388

TABLE 5.6 Importance Factors 391

TABLE 5.7 Values of Approximate Period Parameters C_t and x 393

TABLE 5.8 Coefficient for Upper Limit on Calculated Period 393

TABLE 5.9 Design Coefficients and Factors for Seismic Force–Resisting Systems: ASCE 7-05 394

TABLE 5.10 Ordinary Reinforced Concrete Moment Frames: Seismic Factors 395

TABLE 5.11 Intermediate Reinforced Concrete Moment Frames: Seismic Factors 395

TABLE 5.12 Special Reinforced Concrete Moment Frames: Seismic Factors 395

TABLE 5.13 Ordinary Reinforced Concrete Shear Walls: Seismic Factors 396

TABLE 5.14 Special Reinforced Concrete Shear Walls: Seismic Factors 396
TABLE 5.15 Permitted Building Systems for Different SDC 397
TABLE 5.16 SDC Based on Short-Period Response Acceleration Parameter 399
TABLE 5.17 SDC Based on 1 s-Period Response Acceleration Parameter 399
TABLE 5.18 SDC Based on S_{D1} and S_1 401
TABLE 5.19 SDC Based on S_{DS} and S_1 402
TABLE 5.20 Approximate Base Shear as a Percent of Building Weight: Site Class D 406
TABLE 5.21 Approximate Base Shear as a Percent of Building Weight: Site Class C 410
TABLE 5.22 Allowable Story Drift, $\Delta_a^{a,b}$ 435
TABLE 5.23 Horizontal Irregularities 444
TABLE 5.24 Vertical Irregularities 445
TABLE 5.25 Requirements for Each Story Resisting More Than 35% of the Base Shear 450
TABLE 5.26 Permitted Analytical Procedures 452
TABLE 6.1 Design Criteria Comparison, ACI 318-05, Moment Frames 525
TABLE 6.2 Design Criteria Comparison, ACI 318-05, Shear Walls 526
TABLE 6.3 Design Bending Moments and Shear Forces for Frame Beam B3: OMF 556
TABLE 6.4 Design Axial Forces, Bending Moments, and Shear Forces for Frame Column C3: OMF 557
TABLE 6.5 Design Bending Moments and Shear Forces for B2: Special Moment Frame 558
TABLE 7.1 Rehabilitation Objectives 637
TABLE 7.2 Structural Performance Levels and Damage[a,b,c]—Vertical Elements 639
TABLE 7.3 Structural Performance Levels and Damage[a,b]—Horizontal Elements 640
TABLE 7.4 Effective Stiffness Values[a] 644
TABLE 7.5 Default Lower-Bound Tensile and Yield Properties of Reinforcing for Various ASTM Specifications and Periods[a] 650
TABLE 7.6 Default Lower-Bound Compressive Strength of Structural Concrete (psi) 651
TABLE 7.7 Factors to Translate Lower-Bound Material Properties to Expected Strength Material Properties 651
TABLE 7.8 Numerical Acceptance Criteria for Linear Procedures—Reinforced Concrete Beams 652
TABLE 7.9 Numerical Acceptance Criteria for Linear Procedures—Reinforced Concrete Columns 653
TABLE 7.10 Numerical Acceptance Criteria for Linear Procedures—Reinforced Concrete Beam—Column Joints 654
TABLE 7.11 Numerical Acceptance Criteria for Linear Procedures—Two-Way Slabs and Slab–Column Connections 655

TABLE 7.12 Numerical Acceptance Criteria for Linear Procedures—Reinforced Concrete Shear Walls and Associated Components Controlled by Flexure 656

TABLE 7.13 Numerical Acceptance Criteria for Linear Procedures—Reinforced Concrete Shear Walls and Associated Components Controlled by Shear 657

TABLE 7.14 Numerical Acceptance Criteria for Linear Procedures—Reinforced Concrete Infilled Frames 658

TABLE 8.1 Structural Cost and Column Density Comparison 717

TABLE 9.1 Lower-Bound Limits on Dynamic Procedures Specified in Relation to ELF Procedure Requirements 818

TABLE 9.2 Lateral Loads for 30-Story Building Shown in Figure 9.30 835

TABLE 9.3 Torsion Terminology 851

TABLE 9.4 Analogy between Bending and Warping Torsion 851

TABLE 9.5 Product Integral Tables 864

TABLE 9.6 Calculations for Integral $\omega_\rho \, dA$ 867

TABLE 9.7 Calculations for Sectorial Moment of Inertia 868

TABLE 9.8 Torsion Constants for Open Sections 881

Foreword

In 1980, an ACI convention was convened in San Francisco. During the presentations, the qualities of concrete with respect to strength and durability were discussed, among other properties. It was clear from the reaction of the local audience that certain sections of the structural engineering profession believed and expressed their beliefs that concrete was not a viable product for seismic areas and loads.

However, during this convention, it became apparent that many representatives from other parts of the world were in fact determined to present this material as modern and capable of being a strong, durable, and flexible building product. The facts for such a large demonstration for concrete came from the Caribbean, South Africa, the Middle East, and most of Africa—all areas where structural steel cannot be easily procured.

Since then, we have witnessed the science of enhanced concrete properties. We have developed high-strength concrete from 10 to 14 ksi. We have learned that improved aggregates can create a variable modulus of elasticity. Tests have also shown that steel ratios are important in improving durability.

Having said this, we owe a great debt of gratitude to Dr. Taranath. He has created a state-of-the-art book on modern concrete systems, and environmental responses.

His introduction is worth the price of this book. He doesn't confine his expertise to concrete buildings; he expresses his concern about the state of the industry concerning our loss of judgment, and our sixth sense as engineers. His description of being able to smell a reasonable solution is really the same as what the great engineer Pier Luigi Nervi meant when he said there is no substitute for intuition. Intuition, that is, smell, is not an emotional response but rather that wonderful ability we have to accumulate experiences, and to bring them together at the proper time to create a solution.

I think one of the charming attributes of this book is that it reflects the writer in an easy, humorous way. Dr. Taranath expresses his concern for technology versus judgment and is undoubtedly a teacher with natural ability. This book reads like a novel. It first tackles simple issues and then works its way up to some esoteric topics. It covers performance-based designs as well as the most advanced science of seismic engineering and retrofitting.

The chapters in this book are rationally organized. Each chapter builds on the other. The beginning of each chapter discusses in detail the phenomenon the chapter deals with, for example, seismic systems, etc., and then it applies this phenomenon to concrete and concludes with several diagrams and details.

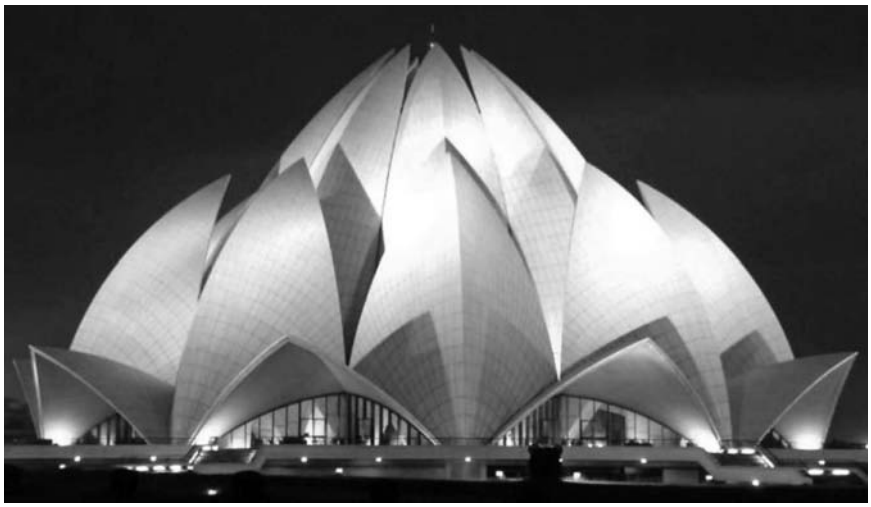
Dr. Taranath has turned concrete into a modern building material with modern systems. This book is complete and spectacular. There is no doubt that reinforced concrete is the most widely used building material in the world. Concrete can be as simple as local stone and cement, or as complex as aggregate, silica fume, and cement and admixtures. Concrete is used for block masonry in emerging nations and for super-high-rise buildings such as the wonderful Burj Dubai, which towers at 2,684 ft.

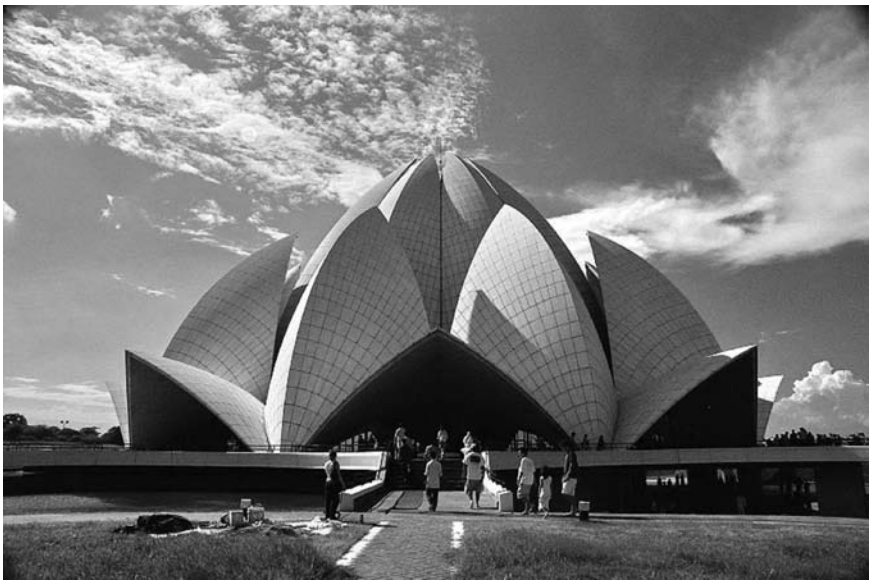
As a building product, it can be molded into any form that is required. Its production is safe, and there is no danger of molten fluids. Concrete, in the words of I.M. Pei, is a material with a soul. People are often puzzled by this statement. How can concrete have a soul? Concrete can be formed and shaped; it can have its color varied and its surface textured; it can be delivered by placement or pumping; and of all the modern building materials, it comes the closest to being able to replicate the Hagia Sophia or the Cathedral at Chartres. Thus, it has a soul.

One of the greatest concrete buildings is the Baha'i House of Worship in New Delhi, India. This temple was cast on wooden forms. The concrete was placed on the forms to resemble a giant lotus blossom—the icon for the sect. The Baha'i temple is approximately 131 ft high and comprises of 26 acres of land including surrounding gardens. The structure is absolutely amazing. The complexity of the building belies the nature of its construction; thousands of workers worked tirelessly taking filled concrete pans atop steep scaffoldings. It is easy to see how concrete can have a soul when you see this building.

Concrete is a common denominator throughout the world. It is found everywhere. When I designed the Jianguo Hotel in Beijing in 1978, we did not even have an ambassador to that country. The preferred material was reinforced concrete. There was no steel construction close enough to cater to this first American/Communist Chinese hotel in China. These were early times in the development of our nations, yet concrete was the vernacular that I and the number one construction company in Peking chose.

The Jianguo went up as a simple reinforced concrete–bearing wall structure. The forms were wood, the concrete locally produced, and the workforce from the surrounding city; thus a safe seismic-resistant egg crate was developed.





Lotus Temple—Bahá'í House of Worship, New Delhi, India. Built in 1986 using reinforced concrete, the Bahá'í House of Worship comprises of 27 freestanding concrete forms that cluster together to resemble a lotus flower. Architect: Fariborz Sahba. Structural Engineer: Flint & Neil Partnership.

The Northrup Avations Peace Hawk V program in Saudi Arabia demonstrated the versatility of concrete. The F-5 fighter program was supported by ground avionics. The facilities consisted of hangars, offices, aviation departments, and warehouses. The material and product used were concrete. This concrete, however, was made into concrete blocks with the use of a very primitive U.S. aid machine, which literally stamped the shape of an ASTM C-90 Block. These blocks were placed up to 20 ft high, reinforced in their cells, and grouted. I made a discovery at that time. Due to the variable nature of the ingredients in cements then (1976), we experienced unusual amounts of drying shrinkage. In order to avoid demolishing and removing these pavements and jet plane runways, it was decided to put a 6 in. sand dike around the concrete area and flood the area with water. The concrete was submerged for six days under water, resulting in a dense and translucent concrete that looked like marble. All the cracks were autogenously healed. The water was reintroduced into the hydration cycle of the concrete, mixed with unhydrated cement, and quickly refilled until the cracks could no longer be located other than thin marble-like veins, which, as in the human body, can be self-healing. Just as a human skeleton is strongest at a previous fracture, so too is concrete.

Because concrete is easily formed and shaped, its compression strength makes it the ideal material for shells of any nature. The work of Felix Candela in developing plates and shells wherein he spans 100 m with a 5 cm concrete shell truly opens the possibility for the free form Catia-driven shapes of Gehry Buildings in concrete.

Concrete is perfect for membrane stresses. The potential was realized in my design of the 120 ft diameter roof of the Lebanon Senior High School. This roof was flat and was spanned using a 20 in. thick hollow slab. This span and thickness of concrete were made possible by the fact that as a circular shell deflects, it creates compression in membrane stresses, thus minimizing the deflection. Dr. Timoshenko, the father of concrete plates and shells, shows that the span of a circular slab is represented by the radius of the circle rather than its diameter.

The grand dome of the Meridian Hotel in Chennai, India, was constructed using concrete. This dome was created by hundreds of workers who climbed scaffoldings with pans full of concrete on their heads. The dome was constructed in concentric rings. There were no doubt cold joints in the concrete, but the dome was in compression and the issue was minimized.

A dome was simultaneously being built in the Bahamas at Paradise Island's Atlantis Resort. This dome was built with two men instead of hundreds using balloon technology. A balloon was inflated with a 140 ft diameter and the balloon took on the form of the dome. This is an example of the same material being used with two totally different technologies, which clearly shows the versatility and ability of concrete to be adjusted to different levels of technology. No other building material has this quality.

There is no doubt that concrete has life and form, and will probably be the material of the future. Advances in exotic reinforcing and chemical admixtures can now produce concrete batches with up to 24 ksi strength in the laboratory.

This book is the key to understanding not only the technology of concrete structures but also the organic nature of this basic material, which is used in every corner of the world.

Vincent J. DeSimone
DeSimone Consulting Engineers
Miami, Florida

ICC Foreword

The field of structural engineering and concrete design of buildings has gone through enormous changes since the time I was in college in the mid-1970s. At that time, one of the best books on the subject of concrete was *Reinforced Concrete Fundamentals* by Phil M. Ferguson. Computerized design of concrete buildings was in its infancy, and I took my stack of 100 "punch cards" for even the simplest design of a reinforced concrete column to a computer center to be processed by a room-sized central campus computer.

Advances in technology, state-of-the-art research, globalization in the immediate transfer of information, and many other trends have transformed design by leaps and bounds. While the basics of flexure member design, compression, torsion, and related concepts have remained fundamentally the same, today's structural engineering students and practicing engineers have access to multiple computer design programs through the most powerful computers on their laptops. Hence, it is much more difficult to assist today's engineers in identifying the best and most appropriate resources among hundreds of textbooks, articles, research papers, and online information. Tall and super-tall concrete buildings are very common now, not just in developed countries, but in most parts of the world, including highly populated countries such as China and India.

Reinforced Concrete Design of Tall Buildings by Dr. Taranath leads readers through an exploration of the intricacies of today's concrete design in a skillful manner keeping real-life issues in mind. The most complicated issues of design are presented in an easy-to-understand language, supplemented by numerous illustrations to further enhance the understanding of the subject. This book is packed with design examples, with Chapter 6 dedicated entirely to seismic design examples.

The most recent findings of building damage or failures caused by seismic or high-wind events have resulted in extensive changes in the areas of seismic and wind designs and detailing. Advances in research and technology necessitate that the International Building Code (IBC) and ACI 318 be published every three years to keep up with innovations and new technologies and research. Both seismic and wind designs based on today's building codes seem to be more complicated than ever before. Accordingly, Dr. Taranath has included updates to ACI 318–08 and the recently released 2009 IBC, as well as the new wind-design provisions of the National Building Code of Canada. To facilitate easy application and use, complete chapters have been dedicated to seismic and wind designs.

Failure patterns, considerations for explosions, progressive collapse, and alternative designs for the reduction of the potential for progressive collapse are other important and current design issues that are covered in this book. Finally, the seismic rehabilitation of existing buildings, which is seldom found in a reinforced concrete design book, is extensively addressed in the last chapter.

In addition to overseeing most of the technical support publications of the International Code Council (ICC), many of which are in the field of structural engineering, I also review an extensive number of books for the ICC's joint efforts and partnerships on a regular basis. This book is truly one of the most interesting and well-laid-out publications that I have reviewed, which is why it was an easy decision for the ICC to be a partner in its co-branding. Structural engineers comprise the most important core of building safety and sustainability professionals by developing responsible, efficient, effective, safe, and economical designs. This book is a significant contribution to that effort.

Hamid A. Naderi
International Code Council

Preface

As I reflect on my career as a practicing engineer, I am struck by the profound conceptual and methodological changes that computer-enhanced design has brought to our field. Today, and especially in the last decade or so of computer use and software engineering, we can develop numerical solutions to an astonishing number of decimals with a degree of precision that was previously unfathomable. On account of liability issues, engineering innovations these days must also be analytically proven and strenuously tested to an extent unknown in the past. In spite of these concerns, the art of being able to smell or feel a reasonable solution must necessarily continue to exist. Without such intuition and creativity, we might tend to rely on computer applications as engineering itself, instead of as a necessary tool.

As structural engineers, our primary task is to take someone else's vision of a project, convert it into analytical and numerical models, and then produce a set of buildable documents. However, the current trend in engineering education seems to focus more on the behavior of computer-based mathematical models while seldom acknowledging their fallibilities. Given this scenario, one may wonder if the era of engineers who endorsed structural attitudes based on their qualitative knowledge of the behavior of the structures is gone.

There is no doubt that navigating complicated software is certainly a critical and necessary part of a designer's vocabulary. My sense, however, is that such skills would be more powerful, accurate, and useful if built upon a solid foundation of engineering principles and conceptual knowledge. I am not alone in voicing these ideas; a plethora of recently published journal articles, opinion pieces, and conference presentations address this ever-increasing gap between the conceptual approach and the scientific illusion created by computer solutions.

These thoughts occur to me in my day-to-day engineering and more specifically as I was preparing this manuscript. Therefore, the challenge I set for myself in this book was to bridge these two approaches: one that was based on intuitive skill and experience, and the other that relied on computer skills. Imagine then the design possibilities when experiential intuition marries unfathomable precision and numerical accuracy.

Engineers are generally characterized as imaginative in their design approach as supported by historical evidence, which includes the creation of ancient structures, medieval cathedrals, and the skyscrapers of today. None of these structures, except for those built in the last decade, were developed using intense calculations as we know them today, but were more products of inventive imagery.

Even with the availability of immense analytical backup, imaginative thinking can and must be effectively used to apply basic concepts to complex problems. Therefore, the stimulus for writing this book was to develop imaginative approaches by examples, and, where appropriate, relate these specific examples to building codes that are essential and mandatory tools of the trade.

The motivation that propelled me into writing this book addresses the question frequently proposed to the designer by the architects: "Can we do this?" And, in the flash-track world that we live in, the time frame allowed for coming up with an answer is measured in days, and, sometimes, even in hours. Such a time constraint does not allow for extensive research or for time-consuming analytical procedures. What is needed is the proverbial back-of-the-envelope analysis that serves as a quick means of evaluating the efficacy of a concept that would then also serve as a check of computer solutions.

Typically, when we prepare a back-of-the-envelope design, the purpose is to make sure we get into the ballpark; once you are in, it is easy enough to find the right row in the analysis phase, and, eventually, to find the right seat.

Finding the ballpark is thus an essential part of the conceptual design. As a designer you will soon learn that once a building program is set it cannot be changed, and the only real option is to mitigate mistakes in concept. On the other hand, if the first step is in the right direction with allowances for potential contingencies, the design will flow smoothly so long as the design has some breathing room.

Chapter 1 discusses selected fundamental concepts. The objective is to develop a "feeling" for overall structural behavior and to provide the designer with the basic insight necessary to the effective development of a design. The subsequent chapters provide detailed discussions of the basic concepts.

Chapter 2 deals with the behavior of gravity components. In addition to common types of framings such as one-way and two-way slabs, novel systems, such as haunch girder systems, are also discussed. An in-depth discussion of prestressed concrete design is presented along with approximate methods to assist engineers in "doing schematics in a meeting."

The focus of Chapter 3 is the design of lateral load–resisting systems. The objective is to control the building behavior through a bracing program that is effective from both the perspectives of cost and behavior. The design concept must be less expensive and better than its alternative if it is to be accepted or adapted. Thus, it is incumbent on the designer to create a cost-effective design in order for it to be realized. This chapter discusses flat slab-frames, coupled shear walls, core-supported structures, tube buildings, and spine-wall structures.

Chapter 4 deals with the determination of design wind loads using the provisions of ASCE 7–05. Wind-tunnel procedures using rigid, high-frequency base and aeroelastic models are discussed, including analytical methods for determining wind response and motion perception. Guidelines are presented for evaluating the acceptability of wind-induced motions of tall buildings.

Chapter 5 covers seismic designs. It develops a design methodology for each component and shows how seismically induced demands may force members to deform well beyond their elastic limits. Detailing considerations for such nonelastic excursions are discussed, and, where appropriate, codification concepts are reduced to a level of analytical simplicity appropriate for the design. The goals are to reduce component design to as simple a process as possible and to highlight design objectives often concealed in the codification procedure. Also discussed in this chapter is the design approach prior to IBC 2002, in which the magnitude of seismic force and level of detailing were strictly a function of the structure's location. This is compared with relatively recent provisions, in which these are not only a function of the structure's location but also of its use and occupancy, and of the type of soil it rests upon. This comparison will be particularly useful for engineers practicing in seismically low- and moderate-risk areas of the United States, who previously did not have to deal with aspects of seismic design. This chapter concludes with an in-depth review of structural dynamic theory.

Chapter 6 provides examples of seismic designs and detailing requirements of concrete buildings. Detailing provisions prescribed in ACI 318–05/08 (Chapters 1 through 20 for buildings assigned to SDC A or B, and in Chapter 21 for those in SDC C and higher) are discussed. Also presented are the designs of special moment frames, shear walls, floor diaphragm-chords, and collectors. Recent revisions to ACI 318 are discussed in the final section.

Chapter 7 is devoted to the structural rehabilitation of seismically vulnerable buildings. Design differences between a code-sponsored approach and the concept of ductility trade-off for strength are discussed, including seismic deficiencies and common upgrade methods.

Chapter 8 is dedicated to the design of tall buildings. It begins with a discussion on the evolution of their structural forms. Case studies of structural systems that range from run-of-the-mill bracing techniques to unique systems—including megaframes and spine-wall structures—are examined.

Finally, Chapter 9 covers a wide range of topics. It begins with a discussion on damping devices that are used to reduce the perception of building motions, including passive viscoleatic dampers, tuned mass dampers, slashing water dampers, tuned liquid column dampers, and simple and nested pendulum dampers. It then deals with seismic isolation and energy dissipation techniques. This is

followed by a discussion on preliminary analysis techniques such as portal and cantilever methods and an in-depth discourse on torsion analysis of open section shear walls with a particular emphasis on their warping behavior. The final section of this chapter covers performance-based designs (PBDs) for the structural design of new buildings. This approach, used for the seismic design of very tall buildings constructed in the western United States within the last few years, has set in motion new ways of doing things. A discussion on the more challenging design issues that may defy codified doctrines, such as height limits, the selection of response modification factors, and peer-review requirements, is presented to introduce engineers to this emerging technology.

Before concluding the preface, it is worth remembering that reinforced concrete as a building material provides a medium that inspires architectural freedom. The design is not peculiar to the material and must satisfy the same basic fundamental laws of equilibrium, compatibility, and compliance with the appropriate stress–strain relationship. The choice of concrete does not pose constraints on the architectural expressionism of structure nor on the free form of today's architectural styles.

This book is a modest attempt to explore the world of concrete as it applies to the construction of buildings while simultaneously striving to seek answers to the challenges I set for myself. It is directed toward consulting engineers, and, within the academy, the book may be helpful to educators and students alike, particularly as a teaching tool in courses for students who have completed an introductory course in structural engineering and seek a deeper understanding of structural design principles and practices. It is my hope that this book serves as a comprehensive reference for the structural design of reinforced concrete buildings, particularly those that are tall.

Bungale S. Taranath
DeSimone Consulting Engineers
Las Vegas, Nevada

Acknowledgments

I wish to express my sincere appreciation and thanks to Vincent J. DeSimone, chairman, DeSimone Consulting Engineers, for his support and encouragement during the preparation of this book and for setting a positive tone for the entire book by writing an eloquent foreword.

My sincere gratitude to Farro Tofighi, managing principal, DeSimone Consulting Engineers, Las Vegas, Nevada, for reviewing the entire manuscript and offering valuable suggestions. I am indebted to him for his confidence in me and will always remain true to his friendship.

I would like to acknowledge the following individuals for their help in the preparation of this book: Samantha Roy, Nichole Hern, Ken Martin, Daniel Schepp, Dr. Ali Shirazi, Alan Dyck, Carol McCullough, David Sze, Elie Elriachi, Felix Madrigal, Filbert Apanay, Frank Reppi, Heinz Kuo, Hui Li, Jason Crabtree, Rani Alhelou, Robert Fortney, Roozbeh Tabatabaii, Sandy Peltyn, Tarek Bannoura, Vijayarajan Krishnaswamy, William Schaffer, and Kyle Fisher, with special thanks to Michael Stavropoulos for organizing tables and graphs in the book.

We are also grateful to Brian Breukelman, consultant, CPP Wind Engineering & Air Quality Consultants, Fort Collins, Colorado, for reviewing the wind chapter and offering constructive suggestions.

My sincere appreciation and thanks to Mark A. Johnson, senior vice president of business and product development, International Code Council (ICC), and Mike C. Mota, manager, Atlantic Region, Concrete Reinforcing Steel Institute (CRSI), for their help in co-branding this book.

Thanks are also due to Jennifer Ahringer, production coordinator, editorial product development, and Joseph Clements, acquisitions editor, CRC press, Taylor & Francis, for their patience and cooperation in the production of this book.

Thanks in no small measure are due to my friend of many years, M.V. Ravindra, president and CEO, LeMessuirier Consultants, Cambridge, Massachusetts, for valuable advice during the preparation of this book.

Everlasting thanks to my daughter, Dr. Anupama Taranath; son-in-law, Dr. Rajesh Rao; son, Abhi Taranath; daughter-in-law, Kristin Taranath; grandsons, Vijay and Kavi; and granddaughters, Anika and Maya, for their love throughout the writing of this book that stole valuable time from my family life.

A Special Acknowledgment

This book, like my other ones, would not have been written had it not been for my wife Saroja Taranath. Her wisdom, generosity, and power of reassurance are awe-inspiring. I can only wonder at my good fortune at having met her in this life, and say from the bottom of my heart, thank you for all you have given me.

My source of inspiration, she helped shepherd the manuscript through its many stages with good humor and grace. Her patience and unyielding support, unconditional love, and congenial manner, always present over the many years of my book-writing career, have often left me astounded. Without her absolute commitment, this modest contribution to structural engineering would not have been possible.

Author

Dr. Bungale S. Taranath, PhD, SE, is a corporate consultant to DeSimone Consulting Engineers, a consulting firm, with offices in New York, Miami, San Francisco, New Haven, Las Vegas, Hong Kong, and Abu Dhabi. He has extensive experience in the design of concrete, steel, and composite tall buildings and has served as principal-in-charge for many notable high-rise buildings. He has held positions as a senior project engineer in Chicago, Illinois, and as vice president and principal-in-charge with two consulting firms in Houston, Texas; he has also served as a senior project manager with a consulting firm in Los Angeles, California. Dr. Taranath is a member of the American Society of Civil Engineers and the American Concrete Institute, and a registered structural and professional engineer in several states. He has conducted research into the behavior of tall buildings and shear wall structures and is the author of a number of published papers on torsion analysis and multistory construction projects. He has previously published three books: *Structural Analysis and Design of Tall Buildings*; *Steel, Concrete, and Composite Design of Tall Buildings*; and *Wind and Earthquake Resistant Buildings, Structural Analysis and Design*. Two of his books were translated into Chinese and Korean and were widely accepted in Asia. Dr. Taranath has conducted seminars on tall-building design in the United States, China, Hong Kong, Singapore, Mexico, India, and England. He was awarded a bronze medal in recognition of a paper presented in London, when he was a fellow of the Institution of Structural Engineers, London, England. Taranath's passion for tall buildings has never slackened. Today his greatest joy is sharing that enthusiasm with owners, architects, and fellow structural engineers to develop imaginative solutions for seemingly impossible structures.

1 Design Concept

Design concept is an impressive term that we use to describe the intrinsic essentials of design. The concept encompasses reasons for our choice of design loads, analytical techniques, design procedures, preference for particular structural systems, and of course, our desire for economic optimization of the structure. To assist engineers in tackling the design challenge, this introductory chapter is devoted to developing a "feeling" for behavior of structural systems.

It is this "feeling" for the nature of loads and their effect on structural systems that paves the way for our understanding of structural behavior and allows the designer to match structural systems to specific types of loading. For example, designers of tall buildings, recognizing the cost premium for carrying lateral loads by frame action alone, select a more appropriate system such as a belt and outrigger wall or a tubular system instead. And engineers designing for intense earthquakes knowing that building structures must sustain gravity loads at large deformations, select moment frames and/or shear walls with ductile connections to provide for the deformation capacity.

As with other materials, the strength and deformation characteristics of reinforced concrete members are important in the design of buildings. In particular, buildings designed to resist seismic forces must have well-detailed members and joints such that the building can sustain large lateral deformations without losing its vertical load-carrying capacity.

In reinforced concrete structures, the reinforcing bars and concrete are almost always subject to axial stress in tension or compression resulting from various load applications. However, they are usually stressed in a manner quite different from that in a simple axial compression or tensile test.

1.1 CHARACTERISTICS OF REINFORCED CONCRETE

1.1.1 Confined Concrete

The term "confined concrete" generally applies to a condition in which concrete is confined in all directions. A reinforced concrete member with closely spaced spiral reinforcement or hoops is one such example. The confining reinforcement restrains the lateral strain in the concrete by increasing both its strength and ductility, as compared to unconfined concrete.

Examples of confined concrete elements are circular columns provided with transverse reinforcement in the form of continuous helical reinforcement, often referred to as spiral reinforcement or circular hoops. Rectangular columns enclosed by rectangular hoops, more common in building construction, are another example. Note that the transverse reinforcement is not stressed until an axial load is reached at which point the unconfined concrete tends to develop appreciable lateral strains. This generally occurs at about 85% of the unconfined strength. Beyond this point, the concrete tends to push against the transverse reinforcement, thereby creating a confining reaction as indicated schematically in Figure 1.1.

The shape of stress–strain curve for reinforced concrete member, among other variables, is a function of spacing and diameter of the transverse reinforcement.

Transverse reinforcement in a rectangular column acts merely as ties between the vertical bars and bow outward rather than effectively confining the concrete between the vertical bars. The larger the diameter of the tie, greater is its bending stiffness, resulting in better confinement. In the case of circular tie, this bending stiffness has no significance. This is because given its shape, the spiral will be in axial tension and will apply a uniform radial pressure to the concrete.

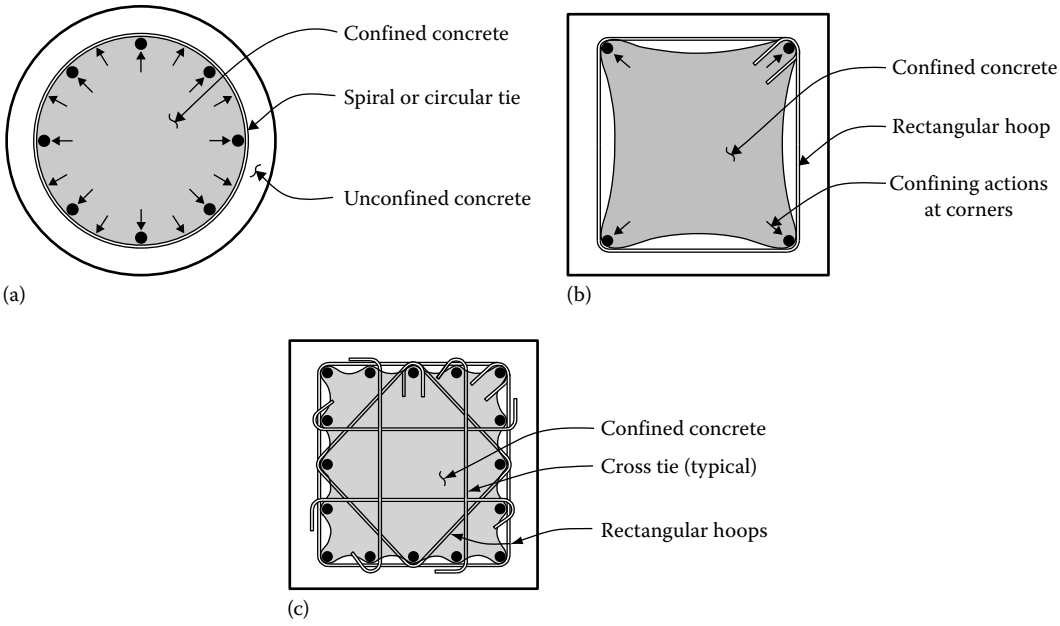


FIGURE 1.1 Confinement of column concrete by transverse reinforcement: (a) confinement by spiral or circular hoops, (b) confinement by a rectangular hoop, and (c) confinement by hoops and cross ties.

The bursting force in a circular column due to lateral expansion of concrete may be considered equivalent to a system of uniformly distributed radial forces acting along the circumference of the transverse tie. The radial forces produce a uniform enlargement of the tie resulting in a tensile force T_u. To determine T_u, let us imagine that the tie is cut at the horizontal diametral section (Figure 1.2) and consider the upper portion of the tie as a free body. If q denotes the uniform radial load per unit

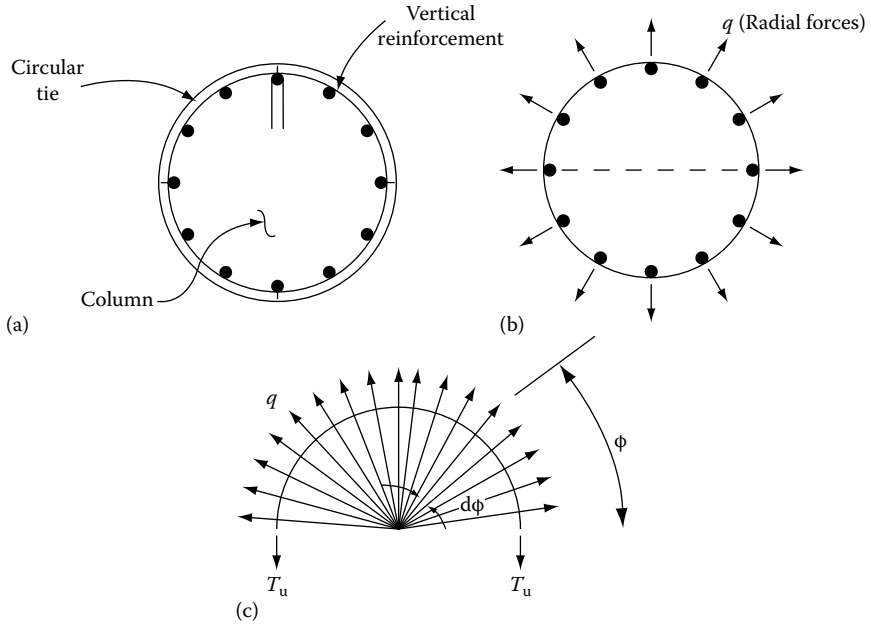


FIGURE 1.2 Confinement of circular column: (a) column with circular ties, (b) radial forces, and (c) free-body of upper portion of tie.

length of the ring, and r_c is the radius of the ring, the force acting on an element of the ring cut by two radial cross sections will be $q_c\,d\phi$, where $d\phi$ is the angle corresponding to the element. Taking the sum of the vertical components of all the forces acting on half the ring, the following equation of equilibrium will be obtained

$$2T_u = 2\int_0^{\pi/2} qr_c \sin\phi\, d\phi$$

from which

$$T_u = qr_c$$

This tension, T_u, is often referred to as hoop tension.

In a spiral column, the lateral expansion of the concrete inside the spiral stresses the spiral in tension and this, in turn, causes a confining pressure on the core concrete, leading to an increase in the strength and ductility of the core. This is the reason why the ACI 318 in the seismic design Chapter 21, requires that beams, columns, and ends of shear walls have hoops in regions where the reinforcement is expected to yield in compression. Hoops are closely spaced closed ties or continuously wound ties or spirals, the ends of which have 135° hooks commonly referred to as seismic hooks, with 6 bar diameter (but no less than 3 in.) extensions (see Figure 1.3). The hoops must enclose

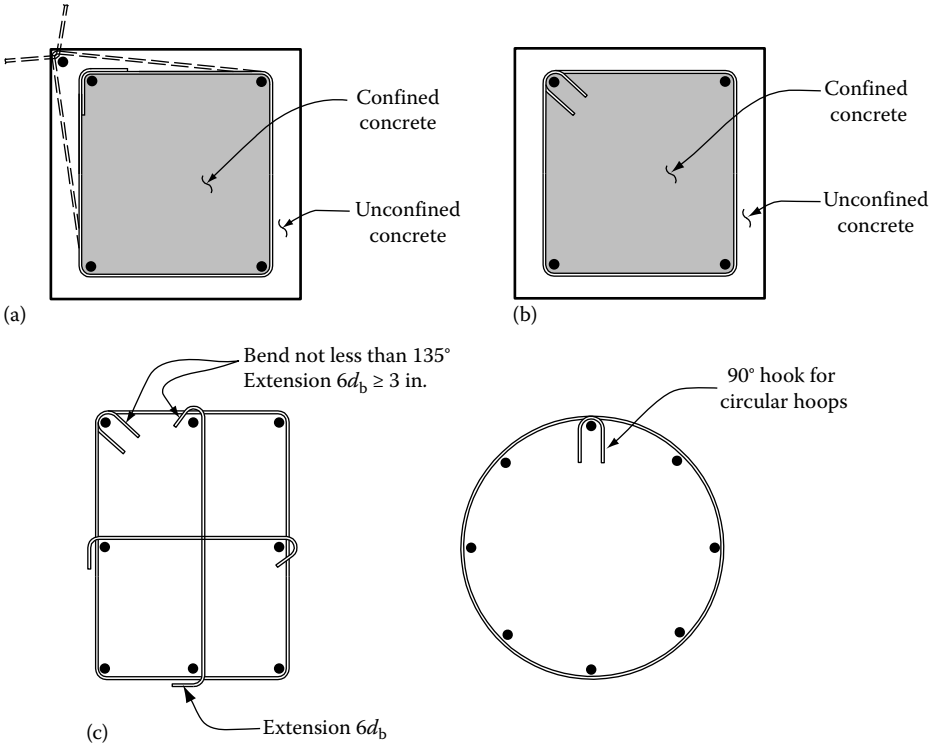


FIGURE 1.3 Column ties and seismic hooks: (a) overlapping 90° hooks at corners cannot confine a concrete core after concrete cover spalls, (b) 135° hooks, required in high seismic areas, provide the necessary confinement for the core while simultaneously resisting buckling of column vertical bars, and (c) seismic hook.

the longitudinal reinforcement and give lateral support to those bars in the manner required for column ties. Although hoops can be circular, they most often are rectangular since most beams and columns have rectangular cross sections. In addition to confining the core concrete, the hoops restrain the buckling of the longitudinal bars and act as shear reinforcement as well. Reinforced concrete frames with confining hoops that are in compliance with the ductile detailing requirements of the ACI 318-05/08 Chapter 21 can achieve deflection ductilities in excess of 5 and shear walls about 4, compared to 1 to 2 for conventionally reinforced nonductile concrete frames.

1.1.2 Ductility

An excellent example for discussing "ductility" of reinforced concrete members is a frame-beam shown in Figure 1.4. The term "frame-beam" applies to a beam that is designed as part of a lateral system. Otherwise it is simply referred to as a gravity beam.

When subjected to seismic ground motions, the frame sways back and forth resulting in flexural and shear cracks in the beam. These cracks close and open alternately due to load reversal and following several cycles of loading, the beam will resemble Figure 1.4. As a result of the back and forth lateral deflections, the two ends of the beam are divided into a series of blocks of concrete held together by the reinforced cage.

If the beam cracks through, shear is transferred across the crack by the dowel action of the longitudinal reinforcement and shear friction along the crack. After the concrete outside the reinforcement crushes, the longitudinal bar will buckle unless restrained by closely spaced stirrups or hoops. The hoop also provides confinement of the core concrete increasing it ductility.

Ductility is the general term that describes the ability of the structure or its components to provide resistance in the inelastic domain of response. It includes the ability to sustain large deformations and a capacity to absorb energy by hysteretic behavior, the characteristics that are vital to a building's survival during and after a large earthquake. This capability of sustaining a high proportion of their strength that ensures survival of buildings when a major earthquake imposes large deformation is the single most important property sought by the designer of buildings located in regions of significant seismicity.

The limit to ductility, such as the displacement of Δ_u, typically corresponds to a specified limit to strength degradation. Even after attaining this limit, sometimes termed "failure," significant additional inelastic deformations may still be possible without structural collapse. Brittle failure, on the other hand, implies near-complete loss of resistance, often complete disintegration without adequate warning. For these reasons, brittle failure, which is the overwhelming cause for collapse of buildings in earthquakes, and the consequent loss of lives, must be avoided.

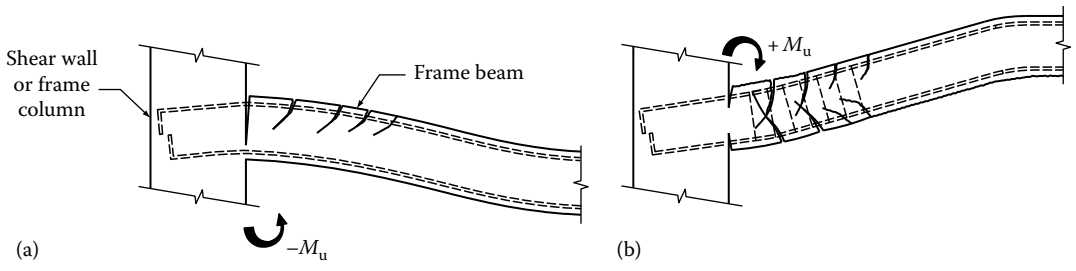


FIGURE 1.4 Frame-beam subjected to cyclic loads: (a) cracks due to $-M_u$ and (b) cracks due to $+M_u$.

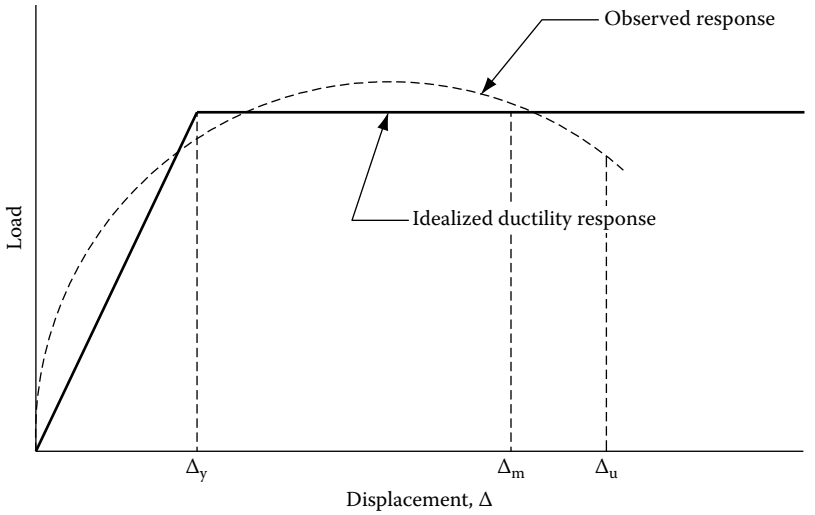


FIGURE 1.5 Ductility model. The ability of the structure to provide resistance in the inelastic domain of response is termed "ductility." Δ_u is the limit to ductility corresponding to a specified limit of strength degradation.

Ductility is defined by the ratio of the total imposed displacements Δ at any instant to that at the onset of yield Δ_y. From Figure 1.5, we have

$$\mu = \Delta/\Delta_y > 1 \tag{1.1}$$

Ductility may also be defined in terms of strain, curvature, rotation, or deflection. An important consideration in the determination of the required seismic resistance will be that the estimated maximum ductility demand during shaking, $\mu_m = \Delta_m/\Delta_y$ does not exceed the ductility potential μ_u.

In structural engineering, the roles of both stiffness and strength of members, as well as their quantification is well understood. However, quantification and utilization of the concept of ductility as a design tool are generally less well understood. For this reason, many aspects of ductile structural response and its application in seismic design are examined in considerable detail in subsequent chapters of this book.

Ductility in structural members can be developed only if the constituent material itself is ductile. Concrete is an inherently brittle material. Although its tensile strength cannot be relied upon as a primary source of resistance, it is eminently suited to carry compression stresses. However, the maximum strains developed in compression are rather limited to about 0.003, unless special precautions are taken. Therefore, the primary aim of seismic detailing of concrete structures is to combine mild steel reinforcement and concrete in such way as to produce ductile members that are capable of meeting the inelastic deformation demands imposed by severe earthquakes.

1.1.3 Hysteresis

When the structure is able to respond inelastically without significant strength degradation to a design-level earthquake, typically defined as an event with a recurrence interval of once in 2500 years, it is said to possess ductility. Observe that ductility must be provided for the full duration of the earthquake, possibly implying many inelastic excursions in each direction.

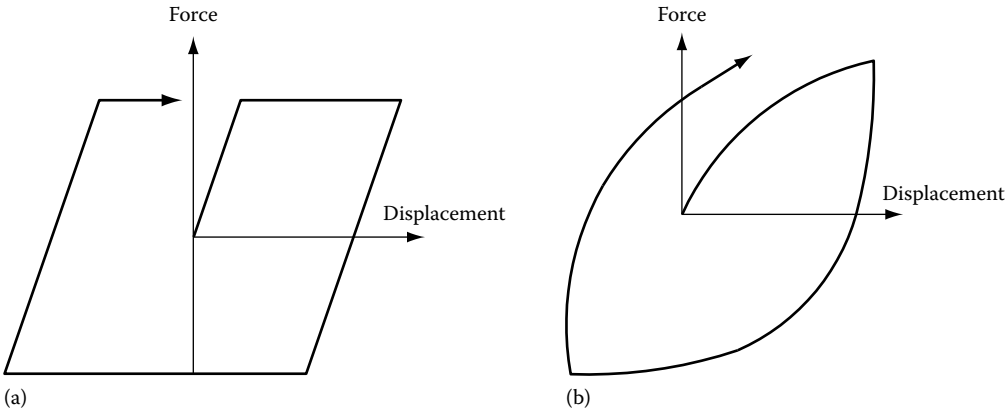


FIGURE 1.6 Hysteresis loops. (a) Idealized elastoplastic loop. (b) Well-detailed beam plastic hinge loop.

Perfect ductility is defined by the ideal elastic/perfectly plastic (often also called elastoplastic) model shown in Figure 1.6, which describes typical response in terms of force versus displacement at the center of mass. Diagrams of this form are termed "hysteresis loops."

The structural response indicated in Figure 1.6a is a structural ideal, seldom if ever achieved in the real world. Hysteresis loops more typical of reinforced concrete are shown in Figure 1.6b. In reinforced concrete frames, it is desirable to concentrate the inelastic deformation in plastic hinges occurring in the beams, adjacent to column faces. Under ideal conditions, hysteresis loops of the form shown in Figure 1.6b typically provide energy absorption of perhaps 70%–80% of the absorption of an equivalent elastoplastic loop. When energy is dissipated in plastic hinges located in columns, the loops diverge further from the ideal elastoplastic shape indicating less energy absorption.

Properly detailed reinforced concrete structural elements exhibit dependable ductile behavior although with hysteresis loops different from the elastoplastic loops. However, all the loops represent essentially ductile behavior, in that they do not indicate excessive strength degradation with increasing displacement or with successive cycling to the same deflection. It should be noted that the area inside the loop is a measure of the energy that can be dissipated by the plastic hinge.

The fullness of a hysteresis loop has always been considered as a positive attribute. How full must it be to have the desired control over building response? At this time (2009), explicit answers are nonexistent, but it is believed that considerable improvement in building response will result, provided that reasonable levels of both strength and energy dissipation are provided.

1.1.4 Redundancy

Especially for buildings in seismic design category (SDC) C and above, it is important for the lateral load system to possess some degree of redundancy.

The SDCs discussed at length in Chapter 5, establish among other requirements, the energy-absorbing capacity of various structural systems. Six SDCs (A through F) are assigned to buildings in ASCE 7-05, A being the least and F the most severe.

Redundancy in a structure means that there is more than one path of resistance for lateral forces. As an example, redundancy can be achieved by having a moment-resistant frame with many columns and beams, all with ductile connections or by having a dual system, such as shear walls plus a moment-resistant frame.

1.1.5 Detailing

Detailing incorporates a design process by which the designer ensures that each part of the structure can perform safely under service load conditions and also when specially selected critical regions

are to accommodate large inelastic deformations. Thus detailing based on an understanding of a feeling for structural behavior, with due regard to the limitations of construction practices, is what makes structural design both a science and an art.

1.2 BEHAVIOR OF REINFORCED CONCRETE ELEMENTS

In the previous section, we discussed certain aspects of seismic design such as confinement, ductility, and hysteresis behavior. However, it should be remembered that a great majority of buildings built in the United States are assigned to lower SDC, that is, A or B, that do not require as much detailing for ductility. The design provisions given in Chapters 1 through 20 and 22 of ACI 318-05/08 are deemed adequate for the design of such buildings. With this in mind, we will review in general terms, the basic behavior of reinforced concrete elements subjected to external loadings such as seismic load.

- Tension (discussed in Section 1.2.1)
- Compression (discussed in Section 1.2.2)
- Bending (discussed in Section 1.2.3)
- Shear (discussed in Section 1.2.4)
- Sliding shear (shear friction) (discussed in Section 1.2.5)
- Punching shear (discussed in Section 1.2.6)
- Torsion (discussed in Section 1.2.7)

1.2.1 Tension

Tension forces stretch members. Concrete has no reliable tension strength. It is essentially cast rock that is strong in compression but weak in tension and shear. Mild steel reinforcement cast into concrete provides for the longitudinal tension while the enclosing ties and stirrups provide for confinement and shear resistance. Sufficient reinforcement can be added to provide adequate toughness for seismic resistance, enabling reinforced concrete to exhibit ductile properties.

Direct tension in reinforced concrete members is not as rare as one may think. Consider, for example, the transfer system shown in Figure 1.7 proposed for a high density resort and casino development in Las Vegas, Nevada. The system is intended for transferring gravity loads from typical interior columns of a multistory residential tower. The tower uses a typical posttensioned flat plate construction with columns spaced at 30 ft centers in the longitudinal direction. Extending the tower interior columns all the way to the foundation would have disrupted a considerable amount of convention, meeting, retail, and casino space in the podium. The solution was to create a reinforced concrete transfer truss as shown in Figures 1.8 through 1.10 with three options for columns below the transfer girder.

Another example of reinforced concrete members subjected to direct tension is a deep beam analyzed using the strut-and-tie model (see Figure 1.11). Note that the mild steel reinforcement for the tie is calculated using a strength reduction factor $\phi = 0.75$, as opposed to $\phi = 0.9$ typically used for tension-controlled design.

1.2.2 Compression

Compression forces push on members and can lead to crushing of materials when the members are short and relatively fat. At bearing surfaces between concrete beams and columns, crushing can also occur. The crushing failures tend to give warning in form of local splitting of concrete and other visible changes. When long, slender members are loaded in compression, they can fail suddenly by buckling or bowing. This type of sudden failure is to be avoided.

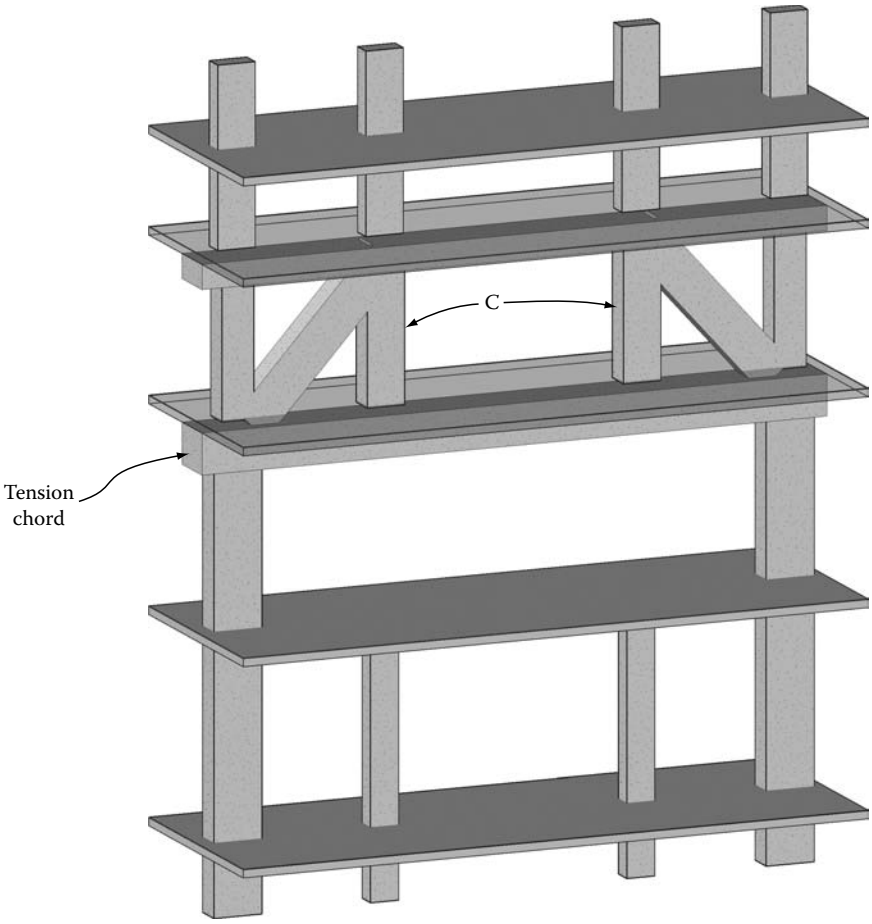


FIGURE 1.7 Tension member in a transfer truss. *Note*: Column C not required for truss action but recommended.

1.2.3 Bending

Bending forces occur mostly as a result of vertical loads applied to floor slabs and beams. Bending causes the bottoms of simple beams to become stretched in tension and the tops of beams to be pushed together in compression. Continuous beams and cantilever beams have tension forces at the top and a compression at the bottom near their supports. At midspan, the forces are in the same locations as for simple beams and slabs. Vertical cracks develop near the midspan of concrete, since the tension force causes the concrete to crack (see Figure 1.12). The reinforcing steel provided in the tension zones is assumed to resist the entire tension force. This tension cracking can be observed in damaged structures and may be used to monitor and determine the potential for collapse. Stable, hairline cracks are normal, but widening cracks indicate impending failure. As stated previously, beams in reinforced concrete moment-resistant frames may experience tension and compression stresses alternately due to stress reversals during earthquakes.

1.2.3.1 Thumb Rules for Beam Design

Thumb rules have been in existence ever since humans started building structures. With passage of time, the rules have been refined based on construction experience. One thumb rule used quite extensively by engineers at present for determining beam depths in concrete buildings is the following: For every foot of beam span, use ¾ in. as the depth. Thus, a 30 ft span beam would have a depth of $0.75 \times 30 = 22.5$, say 22 in., and a 40 ft span, a depth of $0.75 \times 40 = 30.0$ in.

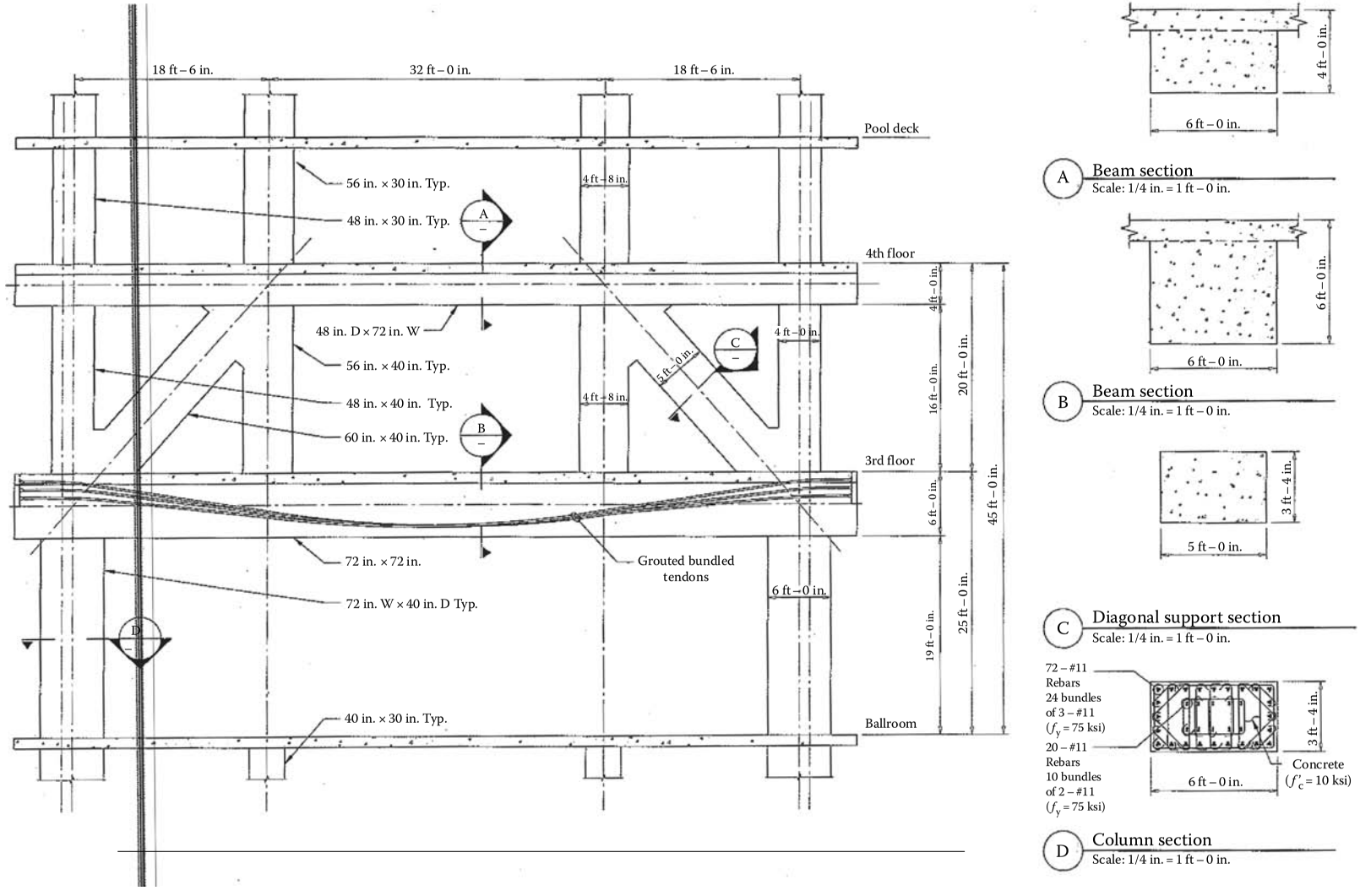


FIGURE 1.8 Transfer girder: schematic elevation with concrete column.

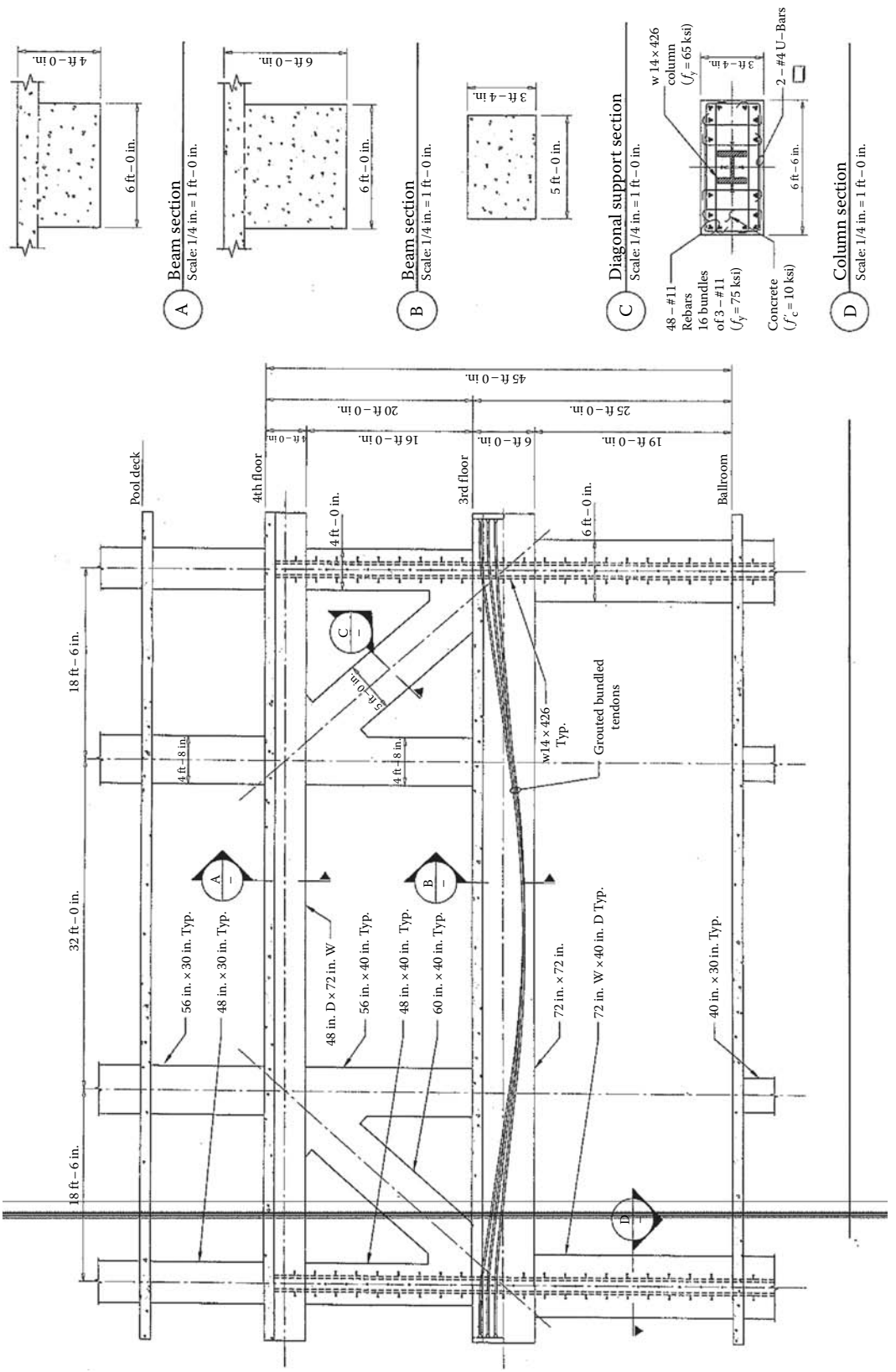


FIGURE 1.9 Transfer girder: schematic elevation with encased composite column.

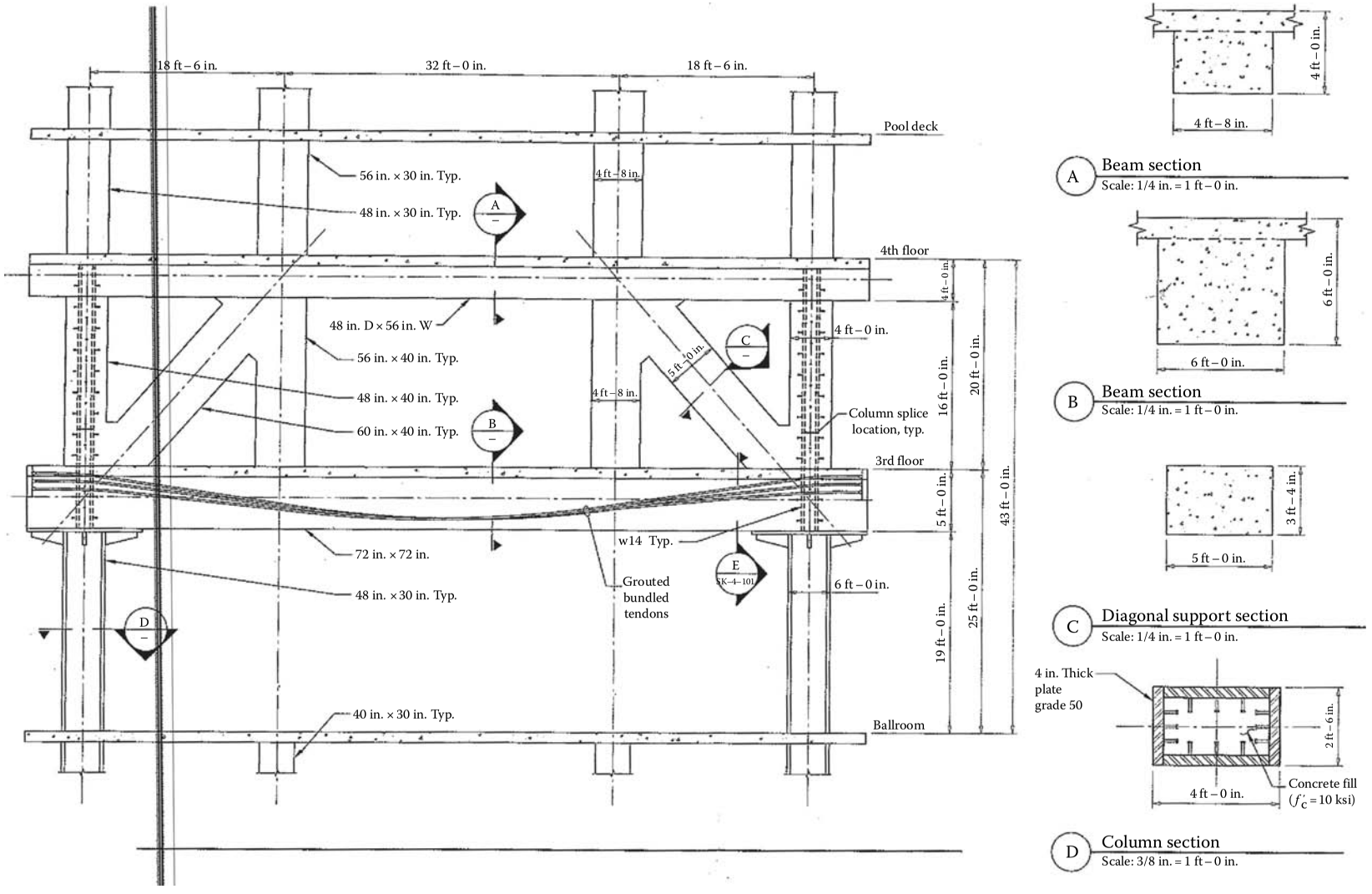


FIGURE 1.10 Transfer girder: schematic elevation with filled steel column.

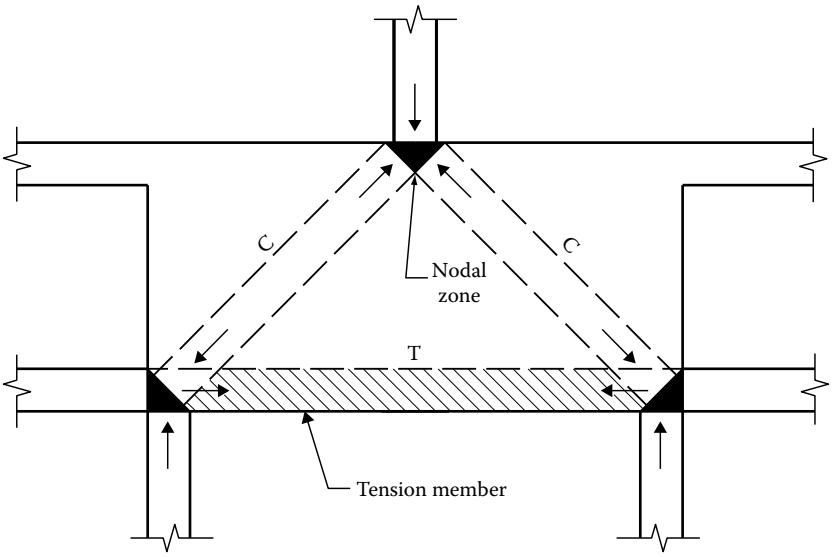


FIGURE 1.11 Tension member in a strut-and-tie model. *Note*: Capacity reduction factor, $\phi = 0.75$.

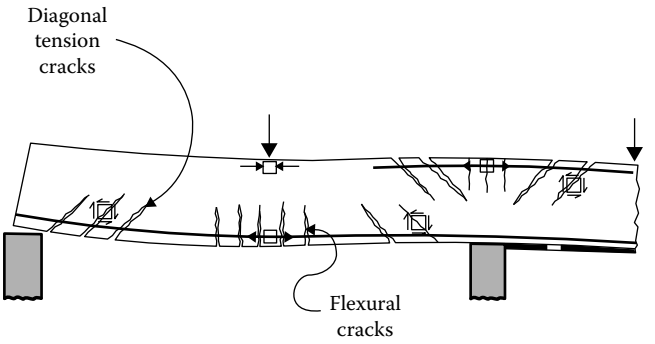


FIGURE 1.12 Development of cracks in a flexural member. Vertical cracks may develop near midspan, stable, hairline cracks are normal, but widening cracks indicate impending failure.

There also exists another easy to remember rule for determining the area of flexural reinforcement in beams:

$$A_s = \frac{M_u}{4d} \tag{1.2}$$

where

M_u is the ultimate design moment (kip-ft)

A_s is the area of flexural reinforcement (in.^{2})

d is the effective depth of flexural reinforcement from the compression face (in.)

This thumb rule works quite well for members with reinforcement ratios of 1%–1.5%. Noting that minimum flexural reinforcement is about ¼% with a maximum at around 2%–2.5%, beam designs using the thumb rule given in Equation 1.2 generally turn out to be quite economical.

Example 1.1

Given
Span = 36 ft
M_u = 500 kip-ft

Required
Area of flexural reinforcement

Solution
Beam depth = $0.75 \times 36 = 27$ in.
Depth, $d = 27 - 2.5 = 24.5$ in.

$$A_s = \frac{500}{4 \times 24.5} = 5.1 \text{ in.}^2$$

To demonstrate the accuracy of this quick method, a comparison is shown in Table 1.1. It is seen that the approximate method is applicable for a wide range of M_u values.

For purposes of resisting gravity loads, horizontal concrete systems such as beams and slabs can also be reinforced by adding high-strength cables or tendons that are pretensioned prior to their being loaded by the structure's weight and superimposed loads. One type commonly referred as precast, prestressed systems may be manufactured in a factory using cables that are stretched in a form, and then bonded to the concrete when it is cast. Another method is to place cables that are enclosed in plastic sleeves in the forms at a job site, cast the concrete, and then stretch and anchor the cables after the concrete has cured and achieved sufficient strength. Using this method, two types of construction are possible. In the first type, the cables are left unbonded to the concrete, but only anchored at the edges of the structures. In the second type, the entire length of tendons is bonded to the concrete. These two methods are commonly referred to unbonded and bonded systems, respectively.

Concrete shrinks, cracks, and creeps under normal circumstances, and this behavior needs to be differentiated from the cracking and spalling that indicates failure. As stated previously, properly reinforced concrete can provide seismically resistant construction if the reinforcing is proportioned such that the configuring ties, hoops, and stirrups are sufficient to resist the shear that can be generated by the overall structural configuration.

As compared to flexural design of beams, rational analysis of two-way slab has always lagged behind design and construction practices. In fact, more than 100 flat slab buildings had already been built when the first rational analysis was published in 1914.

TABLE 1.1
Comparison of Flexural Reinforcement

M_u (kip-ft)	Effective Depth, d (in.)	A_s (in.^{2})	
		Quick Method	Computer Method
500	25.5	4.9	5.08
650	28.5	5.7	5.53
800	30.5	6.56	6.35
1000	35	7.14	7.0
1200	39.5	7.59	7.62
2000	46	10.87	10.16

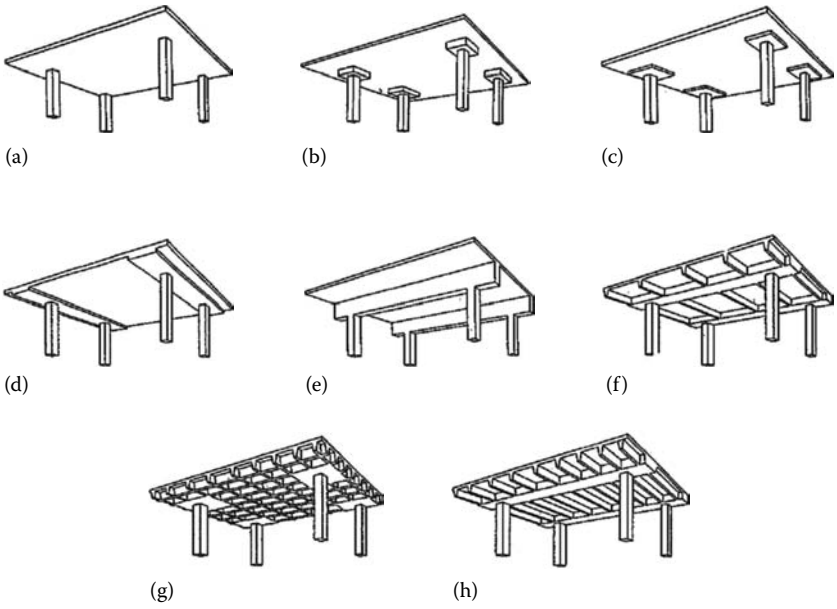


FIGURE 1.13 Two-way slab systems: (a) flat plate, (b) flat plate with column capitols, (c) flat plate with drop panels, (d) band beams, (e) one-way beam and slab, (f) skip joist system, (g) waffle slab, and (h) standard joist system.

The major types of two-way slabs are shown in Figure 1.13. The choice of slab type depends largely on the ease of formwork, superimposed loads, and span. The flat plate system is suitable for lighter loads and moderate spans, the characteristics that make it a popular system for residential constructions such as apartments and hotels. Slabs with beams or drop panels are more suitable for office and institutional buildings with heavier loads.

1.2.4 Shear

We begin with a discussion of horizontal shear stresses that occur in a beam as a result of vertical shear stresses.

To explain this concept, let *abcd* (Figure 1.14) represent an infinitely small prismatical element of unit thickness cut out of a beam subjected to shear stress. If there is a shear stress, V_{uv}, acting on

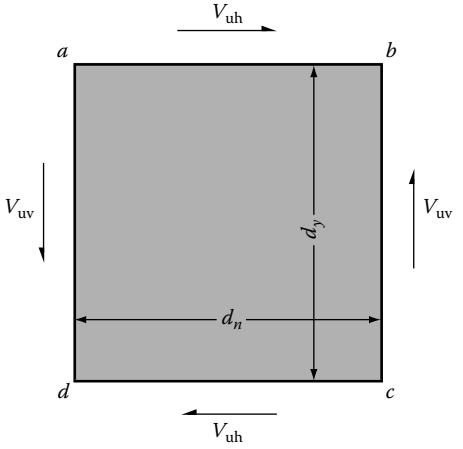


FIGURE 1.14 Unit shearing stresses acting at right angles to each other.

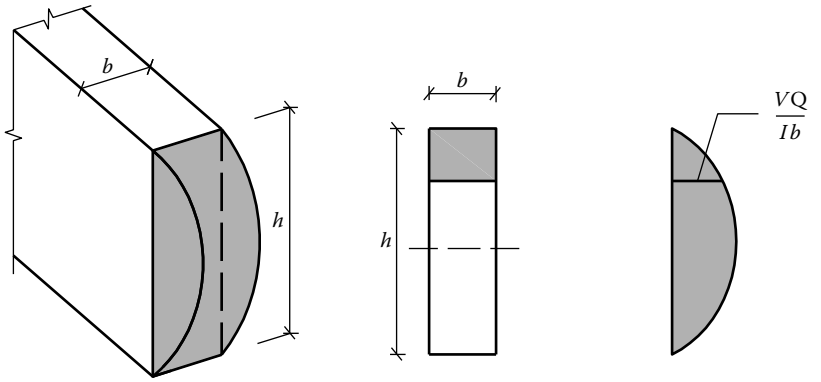


FIGURE 1.15 Shear stress distribution in a rectangular beam.

the right-hand face, the shear force acting on this face is $V_{uv} \times d_y \times 1$ and there must be an equal and opposite force on the left-hand face in order that the sum of the y-component be zero for equilibrium. These two forces, however, constitute a couple and to prevent rotation, there must be another couple made up of horizontal shear forces $V_{uh} \times d_x \times 1$ acting on the top and bottom faces. These two couples must be numerically equal and must act in opposite directions. The moment arm of the first couple is d_x and that of the second couple is d_y. Thus, $V_{uv} \times d_y \times d_x = V_{uh} \times d_x \times d_y$ and accordingly

$$V_{uv} = V_{uh}$$

Thus, the unit shearing stresses acting at a point and lying in planes that are at right angles to each other are numerically equal.

Let us consider the simplest case, with a beam of rectangular cross section having width b and height h (Figures 1.14 through 1.16). We can reasonably assume that (1) the shear stress V_u acts parallel to vertical sides of the cross section and (2) the distribution of shear is uniform across the width of the beam. These two assumptions will enable us to determine completely the distribution of shear stresses acting on the cross section.

Consider a small element of the beam cut out between two planes that are parallel to the neutral surface, as shown in Figure 1.15. Considering the foregoing assumptions, the vertical shear stresses V_u are uniformly distributed on the vertical faces of the element. Recall that shear stresses acting one side of an element are accompanied by shear stresses of equal magnitude acting on perpendicular faces of the element (Figure 1.16).

Thus there will be horizontal shear stresses between horizontal layers of the beam as well as transverse shear stresses on the vertical cross sections. At any point within the beam, the vertical and the complementary horizontal shear stresses are equal in magnitude.

The foregoing discussion leads to the well-known shear stress formula,

$$V_u = v_u Q / Ib \tag{1.3}$$

where

V_u is the total ultimate shear at a given section
v_u is the shear stress at the cross section
Q is the statical moment about neutral axis of that portion of cross section lying between a line through point in question parallel to neutral axis and nearest face (upper and lower) of beam
I is the moment of inertia of cross section about the neutral axis
b is the width of beam at given point

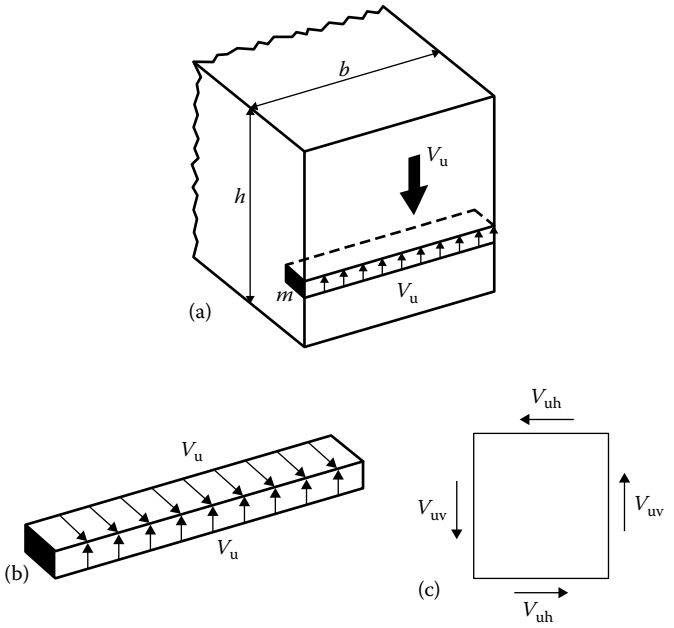


FIGURE 1.16 Complimentary shear stresses. (a) Rectangular beam subject to vertical shear force V_u, (b) beam element between two parallel sections, and (c) shear stresses on perpendicular faces of a beam element.

The intensity of shear along a vertical cross section in a rectangular beam varies as the ordinates of a parabola, the intensity being zero at the outer fibers of the beam and maximum at the neutral axis. The maximum is $3/2V/bd$, since at the neutral axis, $Q = bd^2/8$ and $I = bd^3/12$ (see Figures 1.14 through 1.16).

Shear forces occurring in beams are greatest adjacent to supports in gravity beams. Shear stress can be described as the tendency to tear apart the vertical surfaces of the beams. In concrete beams, these shear stresses do not produce significant vertical cracks, but develop diagonal tension cracks, since concrete is weak in tension.

The behavior of reinforced concrete members subjected to an external shear force is difficult to predict. This is because the shear stress formula (Equation 1.3) is applicable for homogeneous beams working in the elastic range. Reinforced concrete is neither homogeneous nor its behavior elastic. Therefore, it is important to realize that shear analysis and reinforcement design of reinforced concrete members is not really concerned with the determination of shear stresses as such. The real concern is with diagonal tension stress, resulting from the combination of shear stress and longitudinal bending stresses. Since the tension capacity of concrete is low, it is necessary to carefully consider the tension stress resulting from diagonal tension.

The action of vertical and horizontal shear stresses, combined with the flexural stresses, gives rise to a pair of inclined compressive stresses and a pair of inclined tension stresses, which are at right angles to each other. These stresses are known as principal stresses. Of particular concern in reinforced concrete design are the tension stresses that occur as a result of the principal stresses (see Figure 1.17). These stresses commonly referred to as diagonal tension stresses must be carefully considered in design by providing shear reinforcement, most frequently consisting of vertical stirrups. It is important, particularly in seismic design, to ensure that flexural failure would occur prior to shear failure should the member be overloaded.

Shown in Figure 1.18 are the resisting forces at diagonal crack in a beam that has been reinforced with vertical stirrups. Consider the part of the beam to the left of the diagonal crack subjected to an external shear force V_u. No tension force perpendicular to the crack can be transmitted across it

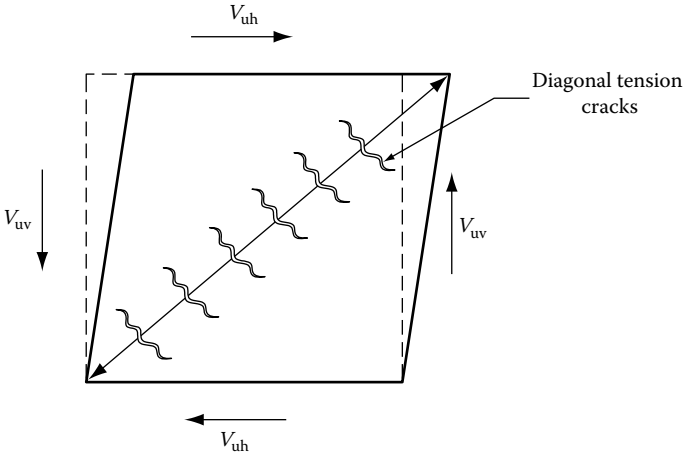


FIGURE 1.17 Diagonal tension.

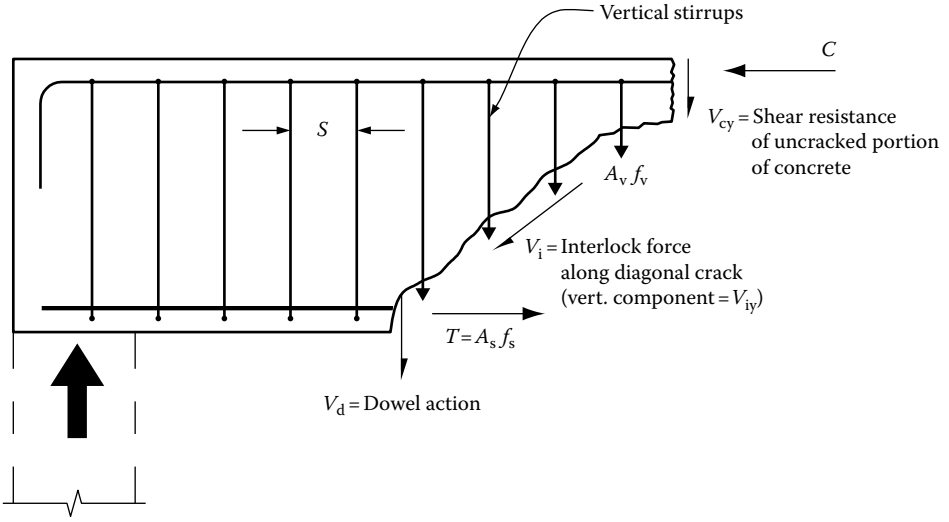


FIGURE 1.18 Shear-resisting forces along a diagonal crack: $V_{ext} = V_{cy} + V_{iy} + V_d + V_s$.

once the crack is formed. However, a sizable force, denoted as V_i in Figure 1.18, can still be transmitted along the crack through interlocking of the surface roughness. The vertical component of this force V_i, denoted as V_{iy} may be considered to provide part of resistance to external shear force V_u. Two other components of vertical resistance also manifest in resisting the shear force. These are the shear force in the uncracked portion of the concrete denoted by V_{cy} and the dowel action of the bottom reinforcement, V_d. The fourth and the final component of the internal resistance is the force $V_s = A_v f_y$ exerted by the stirrup traversing the crack.

Equilibrium of external shear force V_u and the internal resistance gives

$$V_u = V_{cy} + V_d + V_{iy} + V_s \tag{1.4}$$

Of the four components of internal shear resistance shown above, only the shear resistance provided by the stirrups is known with any certainty. The other three do not lend themselves for analytical

quantification. However, for design convenience, the contribution of the three resistances V_{cy}, V_d, and V_{iy} are conservatively lumped into a single term, V_c, somewhat loosely referred to as the shear strength contribution of the concrete to the total shear resistance.

Thus

$$V_c = V_{cy} + V_d + V_{iy} \tag{1.5}$$

The ACI 318-05/08 gives several equations for calculating the shear strength V_c, the most simple one being

$$V_c = 2\sqrt{f'_c}\, bd \tag{1.6}$$

1.2.5 Sliding Shear (Shear Friction)

The preceding section dealt with shear in the context of flexural loading of beams in which shear is used merely as a convenient measure of diagonal tension. In contrast, there are circumstances such that direct shear may cause failure of reinforced concrete members. Potential failure planes can be postulated for such cases along which direct shear stresses are high, and failure to provide adequate reinforcement across such planes may produce unwelcome results.

The necessary reinforcement may be determined on the basis of the shear-friction method of design. The basic approach is to assume that slip occurs along a predetermined plane of weakness. Reinforcement must be provided crossing the potential plane to prevent direct shear failure.

The shear-friction theory is very simple. A shear resistance V_n acts parallel to the crack, and the resulting tendency for the upper block to slip relative to the lower is resisted largely by friction along the concrete interface at the crack. Since the crack surface is typically rough and irregular, the effective coefficient of friction is quite high. In addition, the irregular surface will cause the two blocks of concrete to separate slightly, as shown in Figure 1.19.

If reinforcement is present perpendicular to the crack, then slippage and subsequent separation of the concrete will stress the steel in tension. The resulting tensile force sets up an equal and opposite clamping pressure between the concrete faces on either side of the crack. The maximum value of this interface pressure is $A_{vf}f_y$, where A_{vf} is the total area of steel crossing the crack, and f_y is its yield strength, limited to 60,000 psi.

The relative movement of the concrete on opposite sides of the crack also subjects the individual reinforcing bars to shearing action, and the dowel resistance of the bars to this shearing action contributes to shear resistance. However, it is customary to neglect the dowel effect for simplicity in design, and to compensate for this by using a relatively high value of the friction coefficient.

The required area of shear reinforcement is computed using

$$A_{vf} = V_u/\phi f_y \mu \tag{1.7}$$

where

A_{vf} is the area of shear-friction reinforcement (in.^{2})
V_u is the factored shear force (lbs)
ϕ is the strength reduction factor equal to 0.75
f_y is the yield strength of reinforcement (psi)
μ is the coefficient of friction, equal to 0.6 for normal weight concrete placed against existing concrete not intensionally roughened

The nominal shear strength $V_n = V_u/\phi$ for normal weight monolithic concrete or concrete placed against intentionally roughened surface shall not exceed the smallest of

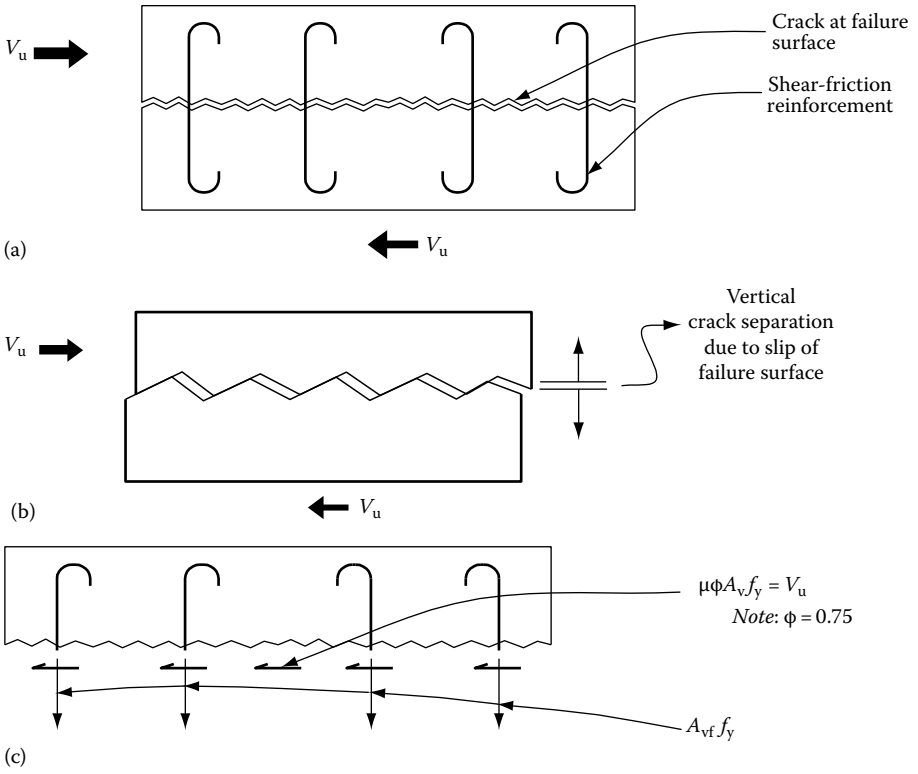


FIGURE 1.19 Frictional resistance generated by shear-friction reinforcement. (a) Applied shear, (b) enlarged crack surface, and (c) free body diagram of concrete above crack.

$$V_n = V_u/\phi \le 0.2 f_c' A_c \tag{1.8}$$

$$\le (480 \pm 0.88 f_c') A_c \tag{1.9}$$

$$\le 1800 A_c \tag{1.10}$$

For all other cases,

$$V_n = V_u/\phi \le f_c' A_c \tag{1.11}$$

$$\le 800 A_c \tag{1.12}$$

The effect of shear-friction reinforcement, A_{vf}, may be considered in a conceptual sense, similar to a gravity load that increases the sliding resistance along the shear plane. For example, a #9 bar of $A_{vf} = 1\,\text{in.}^2$, fully developed on either side of an unroughened sliding plane is conceptually equivalent to vertical load of 60 kip, resulting in a horizontal resistance of 27 kip (with a coefficient of friction equal to 0.45). If the surface is intentionally roughened of the #9 rebar increases to 45 kip, a pretty impressive resistance indeed in either of the two cases.

The yield strength of the reinforcement is not to exceed 60,000 psi. Direct tension across the shear plane, if present, is to be provided for by additional reinforcement, and permanent net compression

across the shear plane may be taken as additive to the force in the shear-friction reinforcement $A_{vf}f_y$ when calculating the required A_{vf}.

When shear is transferred between newly placed concrete against existing hardened concrete, the surface roughness is an important variable. For purposes of design, an intentionally roughened surface is defined as one having a full amplitude of approximately ¼ in.

Certain precautions should be observed in applying the shear-friction method of design. Reinforcement, of whatever type, should be well anchored to develop the yield strength of the steel, by the full development length or by hooks or bends, or by proper heads and welding.

The failure by sliding shear is a possibility in structures subjected to earthquakes. Construction joints across members, particularly when poorly prepared, present special hazards. Flexural cracks, interconnected during reversed cyclic loading, may also become potential sliding planes. Such possible locations shown in Figure 1.20 include

1. *Sliding shear in walls and diaphragms*: Shear transfer across potential sliding planes across walls and diaphragms, where construction joints occur or where wide flexural cracks originating from each of the two edges.
2. *Sliding shear in beams*: Sliding displacements along interconnected flexural and diagonal cracks in regions of plastic hinges can significantly reduce energy dissipation in beams. With reversed cyclic high-intensity shear load, eventually a sliding shear failure may develop.
3. *Sliding shear in columns*: For purposes of verifying sliding shear, particularly in potential plastic hinge regions, it is advisable to consider columns as beams. However, when the vertical reinforcement is evenly distributed around the periphery of the column section as is typical, more reliance may be placed on the dowel resistance against sliding of the vertical

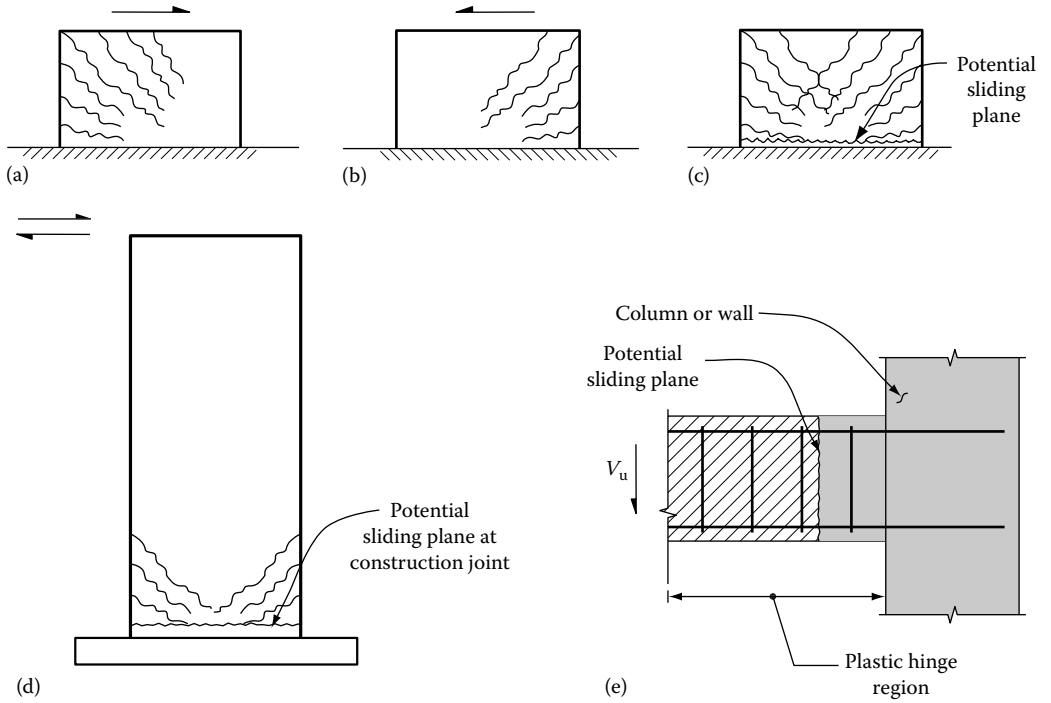


FIGURE 1.20 Potential locations of sliding shear. (a–c) Squat wall, (d) tall wall, and (e) plastic hinge region in a frame beam.

bars. Any axial compression on the column that may be present will greatly increase resistance against sliding shear. Therefore, no consideration need be given to sliding shear in columns constructed with a relatively even distribution of vertical reinforcement.

1.2.6 Punching Shear

Punching shear occurs where a two-way concrete flat slab or plate is connected to a column without beams. It is the tendency of the slab to drop as a unit around the column, as shown schematically in Figure 1.21. The column appears to "punch" through the slab, hence the term "punching shear." The cracking due to the overstress, which may lead to this type of collapse, is most visible on the top surface of the slab.

For expediency in design, the ACI 318 allows the punching shear check through a hypothetical stress distribution such as shown in Figure 1.22. In this design the column is visualized to punch through an assumed critical section. A set of hypothetical stresses over the critical section is computed to be in equilibrium with the associated applied moment and shear. The design procedure aims to keep the computed hypothetical stresses below stipulated, allowable values. These values are $6\sqrt{f_c}$ and $4\sqrt{f_c'}$ for slabs with and without shear reinforcement, respectively. The computed values are not meant to represent true stresses, such as would be obtained from an elastic analysis. In reality, the cracking of concrete and the role of reinforcement over the support dictate a different load path, similar to the simplified strut-and-tie model illustrated in Figure 1.23, in which the top reinforcement becomes central for the load-carrying ability of the joint. The safety of the joint is tied with the tension capacity of reinforcement over the columns. The dearth of reports on punching shear failure for structures built over the last several decades suggests that the minimum top reinforcement requirement over the columns included in the design for flexure in the ACI 318 05/08 is adequate to supplement the punching shear demand. Therefore, although not codified at this time (2009), it appears that wherever an irregularity at a column/slab joint prevents direct application of the hypothetical stress computation formula, the solution to a safe design is best sought by providing an adequate strut-and-tie model. Refined elastic formulations aimed at revealing maximum stresses in the irregular geometry may not be warranted.

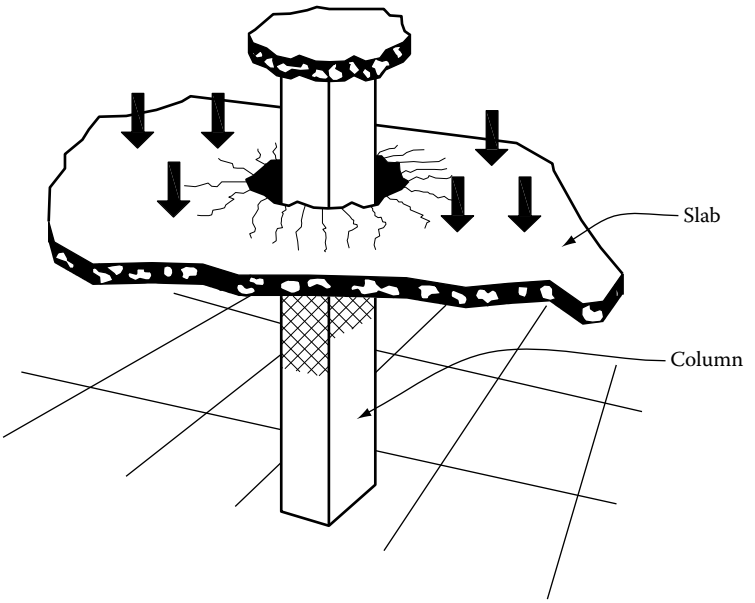


FIGURE 1.21 Punching shear failure in a two-way slab system. It is the tendency of the slab to drop as a unit around the column.

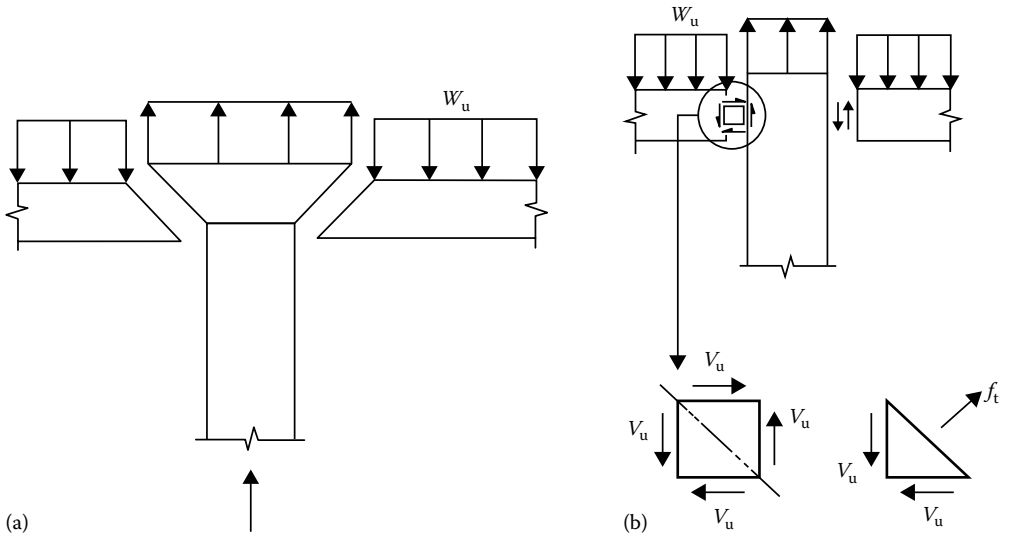


FIGURE 1.22 Transfer of slab load into columns.

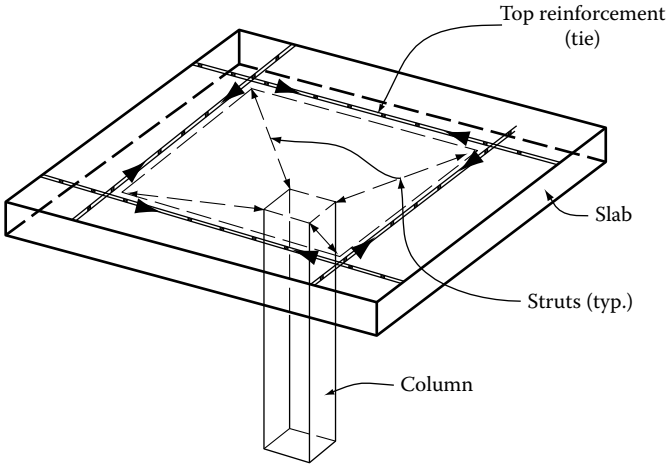


FIGURE 1.23 Strut-and-tie model for punching shear.

1.2.7 Torsion

1.2.7.1 Elemental Torsion

The distribution of gravity loads produces bending and shear forces in just about all the resisting elements. The load path travels from the slab—to floor beams—to girders and finally to the vertical support system. Because of the monolithic nature of reinforced concrete construction, rotational forces including torsion are also transmitted along the load path. Shown in Figure 1.24 is an example of a perimeter spandrel beam subject to torsion.

In designing reinforced concrete members for torsion, the loadings can be separated in two categories.

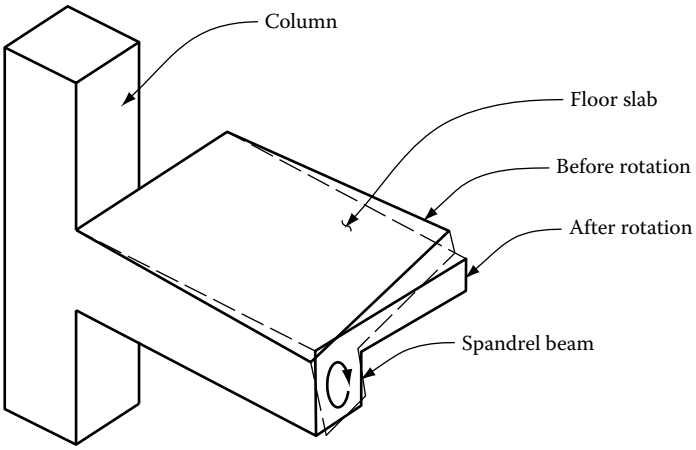


FIGURE 1.24 Torsion in spandrel beams.

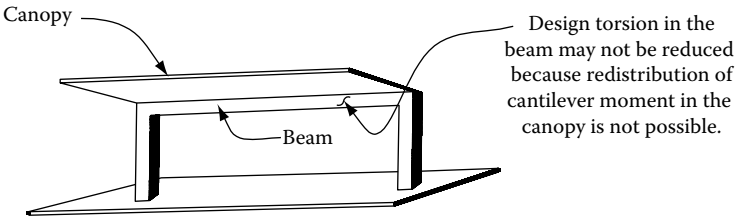


FIGURE 1.25 Equilibrium torsion.

1. Equilibrium torsion
2. Compatibility torsion

When design is for the equilibrium torsion, the entire calculated torsion needs to be accounted for in the analysis.

However, where the torsional moment results from the compatibility of deformations between members meeting at a joint, the designer is permitted to reduce the calculated torsion. This is based on the well-known concept of redistribution of moments in reinforced concrete members (see Figures 1.25 and 1.26).

1.2.7.1.1 ACI Design Method for Torsion

The design procedure for combined moment, shear, and torsion involves designing for the moment first, ignoring torsion and shear, and then calculating the stirrups and longitudinal reinforcement to provide the required resistance to the calculated vertical and torsional shear. Conceptual design steps are as follows:

1. For a given set of $+M_u$, $-M_u$, and V_u, determine the flexural reinforcement $+A_s$ and $-A_s$ and the shear reinforcement, A_v.
2. Determine if torsion should be considered in the analysis by comparing the calculated torsion, T_u to the threshold torsion value, given by

$$T_o = \phi 4\lambda\sqrt{f_c'}\,(A_{cp}^2/P_{cp}) \tag{1.13}$$

If $T_u > T_o$, torsion cannot be ignored.

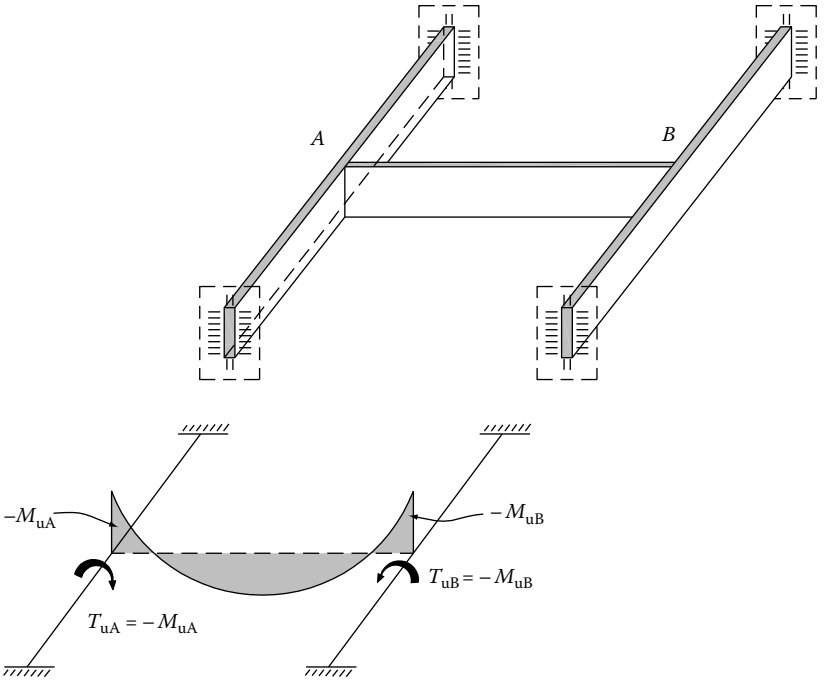


FIGURE 1.26 Compatibility torsion.

3. From an inspection of the given problem, determine if the design torsion, T_u, is due to equilibrium torsion or due to compatibility torsion. If it is due to equilibrium, design of member for the entire calculated torsion. If T_u is due to compatibility requirements, as in statically indeterminate structures, it is permissible to reduce T_u to a maximum value, T_o given by

$$T_u(\max) = \phi 4\sqrt{f_c'}\,(A_{cp}^2/P_{cp}) = T_o \tag{1.14}$$

4. Determine the area of stirrups, A_t, required for resisting torsion T_u by the equation:

$$A_t = \frac{T_n S}{2A_o f_y \cot\theta} \tag{1.15}$$

where

A_t is the area of one leg of a close stirrup resisting torsion within a spacing of S in.^{2}
A_o is the area enclosed by centerline of the outermost closed transverse torsional reinforcement, in.^{2}
θ is the 45° for nonprestressed beams
f_y is the specified yeild strength of reinforcement, psi
T_n nominal torsion moment strength, in.-lb

5. Add the stirrup requirements for torsion and shear using the equation:

$$\text{Total}\left(\frac{A_{v+t}}{S}\right) = \frac{A_v}{S} + \frac{2A_t}{S} \tag{1.16}$$

Note that even when the shear stirrup, A_v, has multiple legs, only the stirrups adjacent to the beam sides are permitted to be included in the summation. This is because inner legs would be ineffective in resisting torsional moments.

6. Check minimum stirrups:

$$\left(A_v + 2A_t\right) = 0.75\sqrt{f'_c}\left(\frac{b_w S}{f_{yt}}\right) \quad \text{and} \quad \frac{50 b_w S}{f_{yt}} \tag{1.17}$$

7. Determine the additional area of longitudinal reinforcement to resist torsion, A_l. Use ACI 318-05, Equation 11.24.

Refer to ACI 318-05/08, Chapter 11 for a complete description of various terms used in the equations above.

1.2.7.2 Overall Building Torsion

To avoid excessive lateral displacements torsional effects should be minimized by reducing the distance between the center of mass (CM), where horizontal seismic floor forces are applied, and the center of rigidity (CR) of the vertical elements resisting the lateral loads. A conceptual explanation of CM and CR follows.

1.2.7.2.1 Center of Mass

During earthquakes, acceleration-induced inertia forces will be generated at each floor level, where the mass of an entire story may be assumed to be concentrated. Hence the location of seismic force at a particular level will be determined by the center of the accelerated mass at that level. In regular buildings, the positions of the centers of floor masses will differ very little from level to level. However, irregular mass distribution over the height of a building may result in variations in centers of masses, which will need to be evaluated.

1.2.7.2.2 Center of Rigidity

This point, defined as the center of rigidity or center of stiffness, locates the position of a story shear force V_j, which will cause only relative floor translations and no torsion. Displacements due to story twist, when combined with those resulting from floor translations, can result in total interstory displacements that may be difficult to accommodate. For this reason, the distance between the CR and the CM should be minimized, but may not be possible due to building geometry. Invariably, effects of torsion are present in at all buildings although analysis may show that in some buildings torsional effects are negligible. This is because torsion occurs as a result of variations in material properties, section geometry, and also due to the effects of torsional component of ground motion. Thus, torsion arises also in theoretically perfectly symmetrical buildings. Hence the seismic requirement that allowance be made in all buildings for so-called accidental torsion. For nonflexible diaphragm buildings, the ASCE 7-05/08 requires that in addition to the calculated torsion, an accidental torsion caused by an assumed displacement of center of mass by a distance equal to 5% of the dimension of the building perpendicular to the direction of force be included. However, the specified accidental torsion need not be applied simultaneously in two directions.

Regarding the simultaneous application of loads in two mutually perpendicular directions, it is worth noting that for buildings in SDC B, the earthquake loads are assumed to act independently along the two orthogonal axes of the buildings. For SDC C buildings having nonparallel lateral load–resisting systems, and for all buildings in SDC D and higher, 100% of the forces for one direction are added to 30% of the forces in the perpendicular direction, the directions chosen to give the worst effect for the member being designed.

If a building is subject to twist, as all are, the torsional stiffness of the core, in a “core-only” structure can be a significant part of the total torsional resistance of the building. The torsional behavior of cores is a topic that is relatively unfamiliar to many engineers. The proportions of the height, length, and thickness of the core walls of a typical building obligate us to analytically treat the core as a thin-walled beam. Consequently, when the core twists originally plane sections of the core warp. Because the core is restrained from warping by the foundation, and to a smaller extent

by the floor slabs, warping stresses somewhat similar to axial stresses are induced throughout the height of core walls. In buildings that are predominantly dependent on a core for torsional and lateral resistance, it is imperative that consideration be given to warping effects. Further explanation of this phenomenon is given in Chapter 9.

1.3 EXTERNAL LOADS

In this section, we will review loads typically considered in building design. These are

- Earthquake loads (Section 1.3.1)
- Wind loads (Section 1.3.2)
- Explosion effects (Section 1.3.3)
- Floods (Section 1.3.4)
- Vehicle impact loads (Section 1.3.5)

1.3.1 Earthquakes Loads

Earthquakes are catastrophic events that occur mostly at the boundaries of portions of the earth's crust called tectonic plates. When movement occurs in these regions, along faults, waves are generated at the earth's surface that can produce very destructive effects.

Aftershocks are smaller quakes that occur after all large earthquakes. They are usually most intense in size and number within the first week of the original quake. They can cause very significant re-shaking of damaged structures, which makes earthquake-induced disasters more hazardous. A number of moderate quakes (6+ magnitude on the Richter scale) have had aftershocks that were very similar in size to the original quake. Aftershocks diminish in intensity and number with time. They generally follow a pattern of being at least 1 large (within magnitude 1 on the Richter scale) aftershock, at least 10 lesser (within magnitude 2 on the Richter scale) aftershocks, 100 within magnitude 3 on the Richter scale, and so on. The Loma Prieta earthquake had many aftershocks, but the largest was only magnitude 5.0, with the original quake being magnitude 7.1.

Some of the most destructive effects caused by shaking as a result of the earthquake are those that produce lateral loads in a structure. The input shaking causes the foundation of a building to oscillate back and forth in a more or less horizontal plane. The building mass has inertia and wants to remain where it is and therefore, lateral forces are exerted on the mass in order to bring it along with the foundation. For analysis purposes, this dynamic action is simplified as a group of horizontal forces that are applied to the structure in proportion to its mass and to the height of the mass above the ground. In multistory buildings with floors of equal weight, the loading is further simplified as a group of loads, each being applied at a floor line, and each being greater than the one below in a triangular distribution (see Figure 1.27). Seismically resistant structures are designed to resist these lateral forces through inelastic action and must, therefore, be detailed accordingly. These loads are often expressed in terms of a percent of gravity weight of the building and can vary from a few percent to near 50% of gravity weight. There are also vertical loads generated in a structure by earthquake shaking, but these forces rarely overload the vertical load-resisting system. However, earthquake-induced vertical forces have caused damage to structures with high dead load compared to design live load. These vertical forces also increase the chance of collapse due to either increased or decreased compression forces in the columns. Increased compression may exceed the axial compressive capacity of columns while decreased compression may reduce the bending strength of columns.

In earthquake engineering, we deal with random variables and therefore the design must be treated differently from the orthodox design. The orthodox viewpoint maintains that the objective of design is to prevent failure; it idealizes variables as deterministic. This simple approach is still valid and applied to design under only mild uncertainty. But when confronted with the effects of earthquakes, this orthodox viewpoint seems so overtrustful as to be worthless. In dealing with

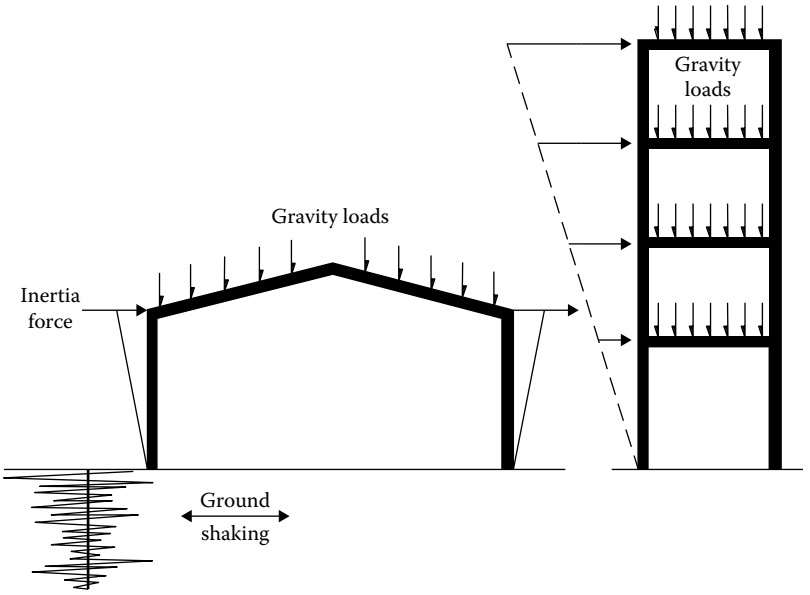


FIGURE 1.27 Earthquake loading: Dynamic action of earthquakes can be simplified as a group of horizontal forces that are applied to the structure in proportion to its mass, and to the height of the mass above the ground.

earthquakes, we must contend with appreciable probabilities that failure will occur in the near future. Otherwise, all the wealth of this world would prove insufficient to fill our needs: the most modest structures would be fortresses. We must also face uncertainty on a large scale while designing engineering systems—whose pertinent properties are still debated to resist future earthquakes—about whose characteristics we know even less.

Although over the years, experience and research have diminished our uncertainties and concerns regarding the characteristics of earthquake motions and manifestations, it is unlikely, though, that there will be such a change in the nature of knowledge to relieve us of the necessity of dealing openly with random variables. In a way, earthquake engineering is a parody of other branches of engineering. Earthquake effects on structures systematically bring out the mistakes made in design and construction, even the minutest mistakes. Add to this the undeniable dynamic nature of disturbances, the importance of soil structure interaction, and the extremely random nature of it all; it could be said that earthquake engineering is to the rest of the engineering disciplines what psychiatry is to other branches of medicine. This aspect of earthquake engineering makes it challenging and fascinating, and gives it an educational value beyond its immediate objectives. If structural engineers are to acquire fruitful experience in a brief span of time, expose them to the concepts of earthquake engineering, even if their interest in earthquake-resistant design is indirect. Sooner or later, they will learn that the difficulties encountered in seismic design are technically intriguing and begin to exercise that nebulous trait called engineering judgment to make allowance for these unknown factors.

1.3.2 Wind Loads

Wind is a term used to describe horizontal motion of air. Motion in a vertical direction is called a current. Winds are produced by differences in atmospheric pressure that are primarily attributable to differences in temperature. These differences are caused largely by unequal distribution of heat from the sun, and the difference in thermal properties of land and ocean surfaces. When temperatures of adjacent regions become unequal, the warmer, lighter air rises and flows over the colder, heavier air. Winds initiated in this way are modified by rotation of the earth.

In describing global circulation of wind, modern meteorology relies on wind phrases used by early long-distance sailors. For example, terms like trade winds and westerlies were used by sailors who recognized the occurrence of steady winds blowing for long periods of time in the same direction.

Near the equator, the lower atmosphere is warmed by the sun's heat. The warm air rises, depositing much precipitation and creating a uniform low-pressure area. Into this low-pressure area, air is drawn from the relatively cold high-pressure regions from northern and southern hemispheres, giving rise to trade winds between the latitudes of 30° from the equator. The air going aloft flows counter to the trade winds to descend into these latitudes, creating a region of high pressure. Flowing northward and southward from these latitudes in the northern and southern hemispheres, respectively, are the prevailing westerlies, which meet the cold dense air flowing away from the poles in a low-pressure region characterized by stormy variable winds. It is this interface between cold, dense air and warm, moist air which is of main interest to the television meteorologists of northern Europe and North America.

As the air above hot earth expands and rises, air from cooler areas such as the oceans floats in to take its place. The process produces two types of wind circulation:

1. General global circulation extending around the earth
2. Smaller secondary circulations producing local wind conditions

Figure 1.28 shows a model of circulation of prevailing winds that result from the general movement of air around the earth. Observe that there are no prevailing winds within the equatorial belt, which lies roughly between latitudes 10° S and 10° N. Therefore, near the equator and up to about 700 miles (1127 km) on either side of it, there exists a region of relative calm called the doldrums. In both hemispheres, some of the air that has risen at the equator returns to the earth's surface at about 30° latitude, producing little or no wind. These high-pressure areas are called horse latitudes, possibly

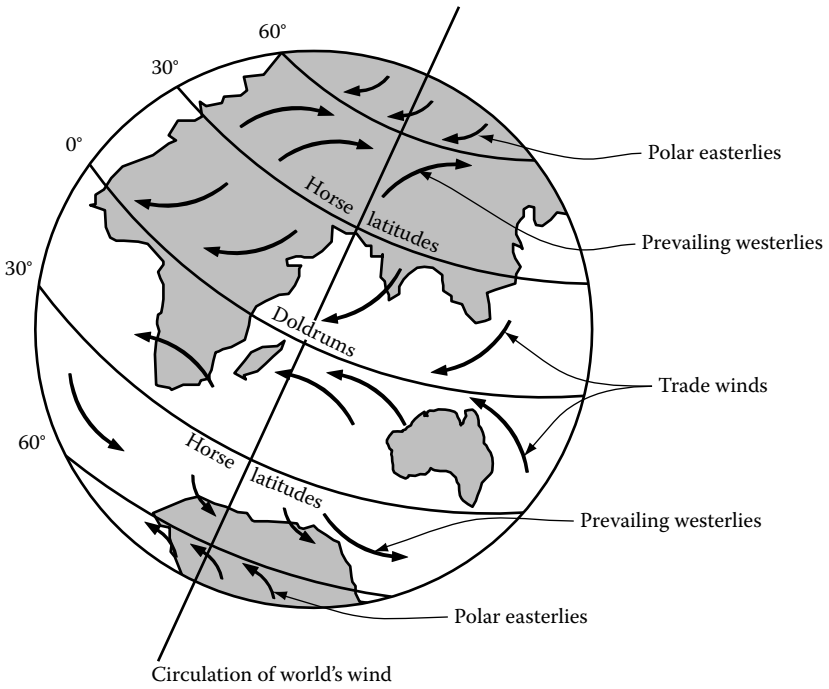


FIGURE 1.28 Circulation of world's winds.

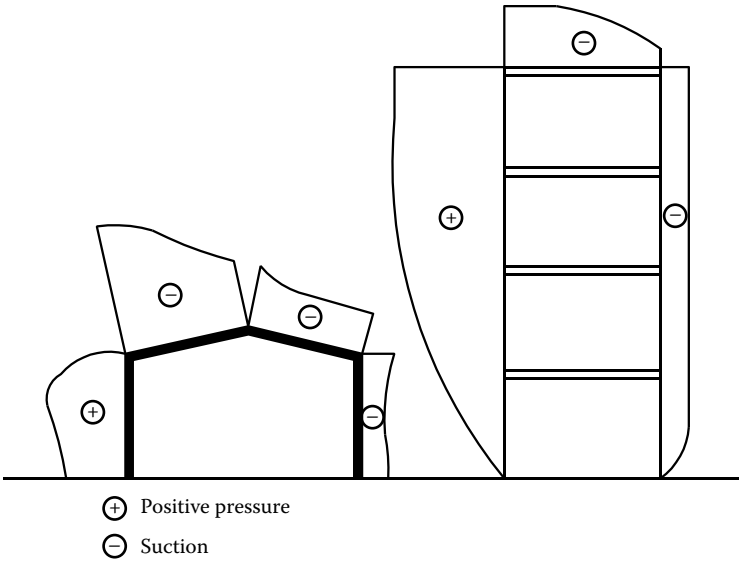


FIGURE 1.29 Wind load distribution: positive pressure on windward wall, and negative pressure (suction) on leeward wall and roof. For a hermetically sealed building, internal pressures cancel out, hence no effect on overall building loads.

because many horses died on the sailing ships that got stalled because of lack of wind. The winds that blow between the horse latitudes and the doldrums are called trade winds because sailors relied on them for sailing ships. The direction of trade winds is greatly modified by the rotation of the earth as they blow from east to west. Two other kinds of winds that result from the general circulation of the atmosphere are called the prevailing winds and the polar easterlies. The prevailing winds blow into the belts bounded by the horse latitudes and 60° north and south of the equator. Thus the moving surface air produces six belts of winds around the earth as shown in Figure 1.28.

Forces due to wind are generated on the exterior of the building based on its height, local ground surface roughness (hills, trees, and other buildings) and the square of the wind velocity. The weight of the building, unlike in earthquake design, has little effect on wind forces, but is helpful in resisting uplift forces. Unless the structure has large openings, all the wind forces are applied to the exterior surfaces of the building. This is in contrast to earthquake forces where both exterior and interior walls are loaded proportionally to their weight. Wind pressures act inward on the windward side of a building and outward on most other sides and most roof surfaces (see Figure 1.29). Special concentrations of outward force, due to aerodynamic lift, occur at building corners and roof edges, particularly so at overhangs. The overall structure is designed for the sum of all lateral and uplift pressures and the individual parts to resist the outward and inward pressure concentrations. They must be connected to supporting members to form a continuous load path. Forces are also generated on structures by airborne missiles such as those caused by dislodging of roofing gravel from neighboring buildings.

1.3.2.1 Extreme Wind Conditions

Extreme winds, such as thunderstorms, hurricanes, tornadoes, and typhoons, impose loads on structures that are many times more than those assumed in their design. Some standards, such as those published by the American National Standards Institute, provide for hurricane wind speeds for a specified probability of occurrence but do not consider directly the effect of other types of extreme wind conditions. A brief description of the characteristics of extreme winds and their effect on structures follows.

1.3.2.1.1 Thunderstorms

Thunderstorms are one of the most familiar features of temperate summer weather, characterized by long hot spells punctuated by release of torrential rain. The essential conditions for the development of thunderstorms are warm, moist air in the lower atmosphere and cold, dense air at higher altitudes. Under these conditions, warm air at ground level rises, and once it has started rising, it continues to rise faster and faster, building storm clouds in the upper atmosphere. Thunder and lightning accompany downpours, creating gusty winds that sometimes blow violently at great speeds. Wind speeds of 20–70 mph (9–31 m/s) are typically reached in a thunderstorm and are often accompanied with swirling wind action exerting high suction forces on roofing and cladding elements.

1.3.2.1.2 Hurricanes

Hurricanes are severe atmospheric disturbances that originate in the tropical regions of the Atlantic Ocean or Caribbean Sea. They travel north, northwest, or northeast from their point of origin and usually cause heavy rains. They originate in the doldrums and consist of high-velocity winds blowing circularly around a low-pressure center known as the eye of the storm. The low-pressure center develops when the warm saturated air prevalent in the doldrums interacts with the cooler air. From the edge of the storm toward its center, the atmospheric pressure drops sharply and the wind velocity rises. In a fully developed hurricane, winds reach speeds up to 70–80 mph (31–36 m/s), and in severe hurricanes, it can attain velocities as high as 200 mph (90 m/s). Within the eye of the storm, the winds cease abruptly, the storm clouds lift, and the seas become exceptionally violent.

The maximum basic wind velocity (3 s gust) for any area of the United States specified in ASCE 7-05 is 150 mph (67 m/s), which is less than the highest wind speeds in hurricanes. Except in rare instances, such as defense installations, a structure is not normally designed for full hurricane wind speeds.

Hurricanes are one of the most spectacular forms of terrestrial disturbances and produce the heaviest rains known on earth. They have two basic components, warmth and moisture, and consequently they develop only in the tropics. Almost invariably they move in a westerly direction at first and then swing away from the equator, either striking land with devastating results or moving out over the oceans until they encounter cool surface water and die out naturally. The region of greatest storm frequency is the northwestern Pacific, where the storms are called typhoons, a name of Chinese origin meaning "wind which strikes." The storms that occur in the Bay of Bengal and the seas of north Australia are called cyclones. Although there are some general characteristics common to all hurricanes, no two are exactly alike. However, a typical hurricane can be considered to have a 375 mile (600 km) diameter, with its circulating winds spiraling in toward the center at speeds up to 112 mph (50 m/s). The size of the eye can vary in diameter from as little as 3.7–25 miles (6–40 km). However, the typhoon that roared past the island of Guam in 1979 had a very large diameter of 1400 miles (2252 km) with the highest wind reaching 190 mph (85 m/s). Storms of such violence have been known to drive a plank of wood right through the trunk of a tree and blow straws end-on through a metal deck. Fortunately, storms of such magnitude are not common.

1.3.2.1.3 Tornadoes

Tornadoes develop within severe thunderstorms and occasionally in hurricanes. They consist of a rotating column of air usually accompanied by a funnel-shaped downward extension of a dense cloud having a vortex of several hundred feet, typically 200–800 ft (61–244 m) in diameter whirling destructively at speeds up to 300 mph (134 m/s). A tornado contains the most destructive of all wind forces, usually destroying everything along its path of approximately 10 miles (16 km) long and directed predominately toward the northeast. Tornadoes form when a cold storm front runs over warm, moist surface air. The warm air rises through the overlaying cold storm clouds and is intercepted by the high-altitude winds, which are even colder and are rapidly moving above the clouds. Warm air collides with the cooler air and begins to whirl. The pressure at the center of the spinning column of air is reduced because of the centrifugal force. This reduction in pressure causes more warm air to be sucked into it, creating a violent outlet for the warm air trapped under the storm.

As the velocity increases, more warm air is drawn in to the low-pressure area created in the center of the vortex. As the vortex gains strength, the funnel begins to extend toward the ground, eventually touching it. Funnels usually form close to the leading edge of the storm. Larger tornadoes may have several vortices within a single funnel. If the bottom of the funnel can be seen, it usually means that the tornado has touched down and begun to pick up visible debris from the ground. A typical tornado travels 20–30 mph (9–14 m/s), touches ground for 5–6 miles (8–10 km), and has a funnel 300–500 ft (92–152 m) wide. Distance from the ground to the cloud averages about 2000 ft (610 m). Although it is impractical to design buildings to sustain a direct hit from a tornado, it behooves the engineer to pay extra attention to anchorage of roof decks and curtain walls for buildings in areas of high tornado frequency. Rolling plains and flat country make a natural home for tornadoes. Statistically, flat plains get more tornadoes than other parts of the country. In North America, communities in Kansas, Nebraska, and Texas experience many tornadoes and are classified as "tornado belt" areas. No accurate measurement of the inner speed of a tornado has been made because tornadoes destroy standard measuring instruments. However, photographs of tornadoes suggest the wind speeds are of the order of 167–224 mph (75–100 m/s). Although there definitely are tornado seasons, tornadoes can occur at any time. Like a hurricane, the tornado consists of a mass of unstable air rotating furiously and rising rapidly around the center of an area with low atmospheric pressure. The similarity ends here, because whereas the hurricane is generally of the order of 300–400 miles (483–644 km) in diameter, a large tornado is unlikely to be more than 1500 feet (458 m) across. However, in terms of destructive violence, no other atmospheric disturbance comes even close to that caused by tornadoes.

Although the probability of any one particular building being hit by a tornado is very small (less than 10 per year), tornadoes account for the greatest incidence of death and serious injury of building occupants due to structural failure and cause considerable economic loss. With some exceptions, such as nuclear power plants, it is generally not economical to design buildings for tornadoes. It is, however, important to provide key construction details for the safety of building occupants. Investigations of tornado-damaged areas have shown that the buildings in which well over 90% of the occupants were killed or seriously injured by tornadoes did not satisfy the following two key details of building construction:

- The anchorage of house floors into the foundation or ground (the floor takes off with the occupant in it)
- The anchorage of roofs to concrete block walls (the roof takes off and the unsupported block wall collapses onto the occupants)

Deficiency of the second construction detail is especially serious for open assembly occupancies because there is nothing inside, such as stored goods, to protect the occupants from wall collapse. For such buildings in tornado-prone areas, it is recommended that the block walls contain vertical reinforcing linking the roof to the foundation.

For tornado protection, key details such as those indicated above should be designed on the basis of a factored uplift wind suction of 50 psf on the roof, a factored lateral wind pressure of 25 psf on the windward wall, and suction of 50 psf on the leeward wall, as recommended by the National Building Code of Canada, NBC 2005.

1.3.3 Explosion Effects

Explosions occur when a solid or concentrated gas is transformed into a large volume of hot gases in a fraction of a second. In the case of high explosives, detonation conversion of energy occurs at a very high rate (as high as 4 miles/s), while low explosives such as gunpowder undergo rapid burning at the rate of about 900 ft/s. The resulting rapid release of energy consists of sound (bang), heat and light (fireball), and a shock wave that propagates radically outward from the source at subsonic

speeds for most low explosives and supersonic speeds for high explosives. It is the shock wave consisting of highly compressed particles of air that causes most of the damage to structures. When natural gas explosions occur within structures, gas pressures can build up within confined spaces, causing extensive damage. In all explosions, large, weak, and/or lightly attached wall, floor and roof surfaces may be blown away. The columns and beams may survive a blast, but their stability may be compromised by the removal of their bracing elements such as floor diaphragms. In large explosions, concrete slabs, walls, and even columns may be blown away, leading to conditions that will produce progressive collapse. In 1967, a progressive collapse started when a natural gas explosion caused the collapse of an exterior wall on the 18th floor of a 22-story building. The force of falling debris from floors 19 to the roof then caused the remaining floors to collapse in that section of the building. In the case of an exterior explosion such as from a bomb, the shock wave is initially reflected and amplified by the building face and then penetrates through openings, subjecting floor and wall surfaces to great pressure. Diffraction occurs as the shock propagates around corners, creating areas of amplification and reduction in pressure. Finally the entire building is engulfed by the shock wave, subjecting all building surfaces to the overpressure. A secondary effect of an air blast is a very high velocity wind that propels the debris, which becomes deadly missiles. In very large explosions at close proximity to building surfaces, the effect can be so severe that the structure is locally disintegrated and separated away from the main structure.

1.3.4 Floods

Forces are generated on buildings due to hydrostatic lateral and lifting pressure, hydrodynamic forces, and debris impacts. Hydrostatic pressures can overload foundation and basement walls and lift up structures, when water level is not equalized between exterior and interior spaces. River and ocean currents may load frontal and side walls that are submerged and ocean waves can produce pressures as high as 1000 psf. Debris varying in sizes from floating wood pieces to floating structures can impact a building, causing anything from broken windows to a total collapse.

1.3.5 Vehicle Impact Loads

Structures have been severely damaged and set on fire by vehicle impacts. The most hazardous configurations include soft (high, open) first stories and open-front buildings typical of retail one and two-story structures.

1.4 LATERAL LOAD-RESISTING SYSTEMS

It can be said that there are as many types of lateral systems as there are engineers. However, most of the systems can be grouped into three basic types: (1) shear wall system, (2) frame system, and (3) combination of the two, the shear wall–frame system (dual system). Perhaps the most common of the three for design of buildings taller than, say 40 stories, is the dual system. However, in recent years (2009), there is a trend to push the height limits of the "core-only" type of shear wall systems without moment frames even for buildings assigned to high SDCs. These aspects are discussed in more detail in Chapter 9.

The selection of a structural system for buildings is influenced primarily by the intended function, architectural considerations, internal traffic flow, height, and aspect ratio, and to a lesser extent, the intensity of loading. The selection of a building's configuration often dictated by architectural considerations is perhaps one of the most important aspects of the overall design. It may impose severe limitations on the structure in its role to provide seismic protection. Some structural forms of construction such as a flat slab system supported by columns are considered unsuitable on its own to provide satisfactory performance under seismic actions although its use is quite prevalent in areas of low seismicity. The seismic issue arises because of excessive lateral displacements and the difficulty of providing for adequate and dependable shear transfer between columns and slabs.

Structural systems considered in this section are as follows:

- Shear walls (Section 1.4.1)
- Coupled shear walls (Section 1.4.2)
- Moment-resistant frames (Section 1.4.3)
- Dual systems (Section 1.4.4)

1.4.1 Shear Walls

Buildings engineered with structural walls are almost always stiffer than framed structures, reducing the possibility of excessive deformations and hence damage. The necessary strength to avoid structural damage under earthquakes can be achieved by providing a properly detailed longitudinal and transverse reinforcement. By adopting special detailing measures, dependable ductile response can be achieved under major earthquakes.

Lateral forces, that is, the forces applied horizontally to a structure derived from winds or earthquakes cause shear and overturning moments in walls. The shear forces tend to tear the wall just as if you had a piece of paper attached to a frame and changed the frame's shape from a rectangle to a parallelogram (see Figure 1.30). The changing of shape from a rectangle to a parallelogram is generally referred to as racking. At the ends of shear walls, there is a tendency for the wall to be lifted up at the end where the lateral force is applied, and a tendency for the wall to be pushed down at the end away from the force. This action provides resistance to overturning moments.

Because of a large fraction of, if not the entire, lateral shear force is often assigned to structural walls, they are usually referred to as shear walls. The name is in appropriate, for it presupposes that shear controls their behavior. This need not be so. And in earthquakes design, this must not be so: every attempt should therefore be made to inhibit inelastic shear modes of deformations. This is achieved readily in practice because shear walls provide nearly optimum means of providing stiffness, strengths, and ductility.

To receive gravity loads from the floors and roof, while simultaneously providing resistance to wind and seismic loads, the walls are typically connected by floor and roof planes often referred to as diaphragms. These planes act like giant beams as stresses in tension and compression are

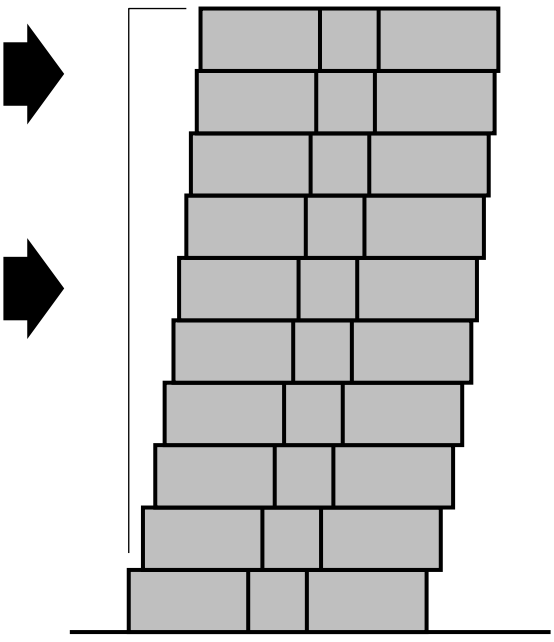


FIGURE 1.30 Shear deformations in a shear wall.

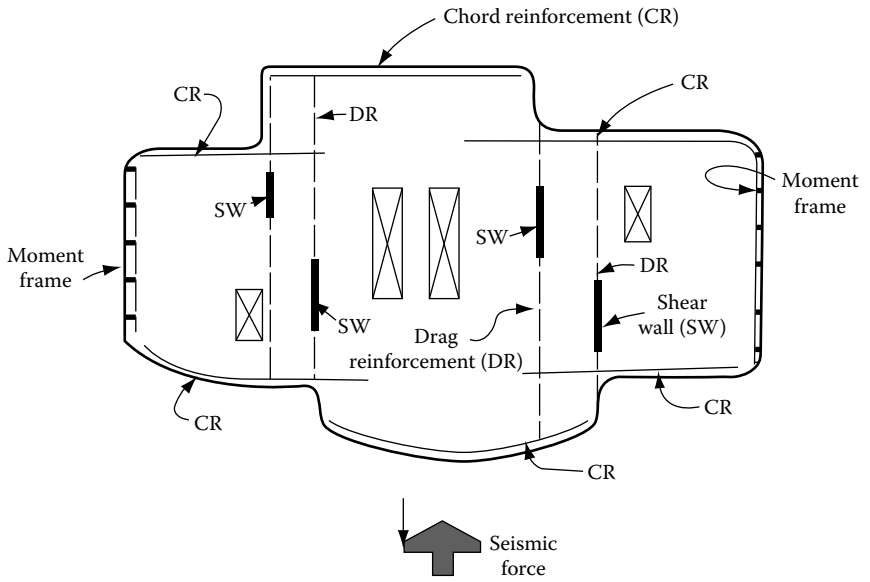


FIGURE 1.31 Diaphragm concept. *Note*: Seismic elements in the E–W direction not shown for clarity.

generated at the edges while shear stresses are distributed throughout the plane. The diaphragm spans horizontally between vertical elements resisting lateral loads as illustrated in Figure 1.31.

The first task of the designers will be to select a structural system most conducive to satisfactory wind and seismic performance within the constraints dictated by architectural requirements. They should discuss alternative structural configurations at the earliest stage of concept development to ensure that undesirable geometry is not locked-in to the system before structural design begins. Irregularities, often unavoidable, contribute to the complexity of structural behavior. When not recognized, they may result in unexpected damage and even collapse. Drastic changes in geometry, interruptions in load paths, discontinuities in both strength and stiffness, disruptions in critical regions by openings, unusual proportions of members, reentrant corners, lack of redundancy, and interference with structural deformations are only a few of the possible structural irregularities. The recognition of many of these irregularities and of conceptions for remedial measures for the avoidance or mitigation of their undesired effects relies on sound understanding of structural behavior. Awareness to search for undesired structural features and design experience in mitigating their adverse effects are invaluable attributes.

When functional requirements permit it, resistance to lateral forces may be assigned entirely to structural walls. Usually, there are also other elements within such a building, which are assigned to carry only gravity loads. Their contribution to lateral force resistance, if any, is often neglected. However, for buildings assigned to SDC D or higher, deformation compatibility of these elements must be verified using magnified elastic displacement. This aspect is discussed in Chapter 4 in more detail.

For buildings up to 20 stories, the use of structural walls is often a matter of choice. For buildings over 30 stories, structural walls may become imperative from the point of view of economy and control of lateral deflection.

Individual walls may be subjected to axial, translational, and torsional displacements. The extent to which a wall will contribute to the resistance of overturning moments, story shear forces, and story torsion depends on its geometric configuration, orientation, and location within the building. While it is relatively easy to accommodate any kind of wall arrangements to resist wind forces, it is much more difficult to ensure satisfactory overall building response to large earthquakes when wall locations deviate considerably. This is because, in the case of wind, a fully elastic response is expected, while during large earthquake demands, inelastic deformations will arise.

The major structural considerations for individual structural walls will be aspects of symmetry in stiffness torsional stability, and available overturning capacity of the foundations. The key in the

strategy of planning for structural walls is the desire that the inelastic deformations be distributed reasonably uniformly over the whole plan of the building rather than being allowed to concentrate in only a few walls. The latter case leads to the underutilization of some walls, while others might be subjected to excessive ductility demands.

Elevator shafts and stairwells lend themselves to the formation of reinforced concrete core. Traditionally, these have been used to provide the major component of lateral force resistance in multistory office and residential buildings. Additional resistance may be derived, if necessary, from perimeter frames shown in Figure 1.32. Such a centrally positioned large core may also provide sufficient torsional resistance without requiring additional perimeter framing.

In choosing suitable locations for structural walls, the following additional aspects should be considered:

1. For the best torsional resistance, locate as many of the walls as possible at the periphery of the building. Such an example is shown in Figure 1.33. The walls on each side may be single or they may be coupled to each other.
2. Route as much gravity load to the foundations as possible via a structural wall. This will lower the demand for flexural reinforcement in that wall. And more importantly uplift forces will be reduced in the foundations.

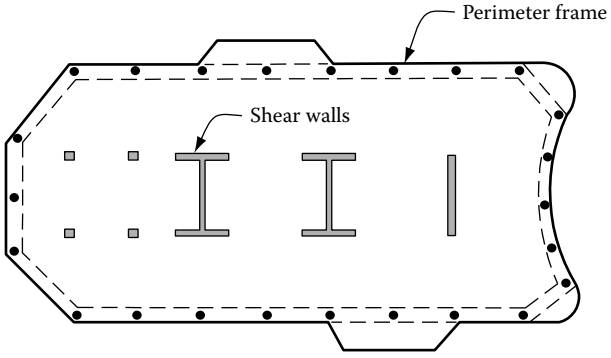


FIGURE 1.32 Floor plan: building with shear walls and perimeter frame.

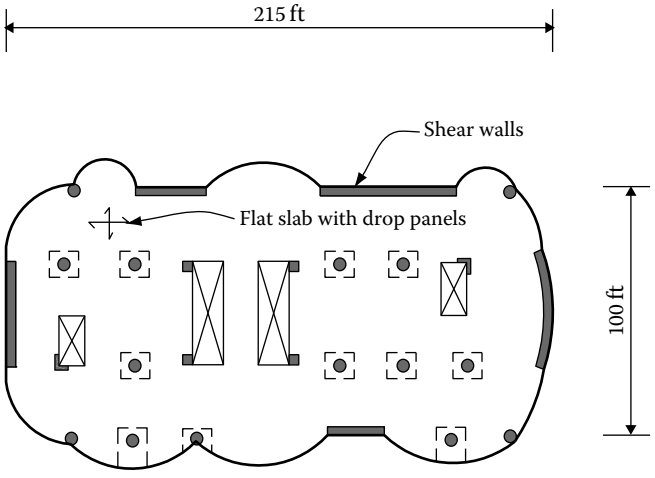


FIGURE 1.33 Building with perimeter shear walls.

1.4.2 Coupled Shear Walls

In many shear wall buildings, a regular pattern of openings will be required to accommodate windows or doors or both. Highly efficient structural systems, particularly suited for ductile response with good energy dissipation characteristics, can be conceived when openings are arranged in a regular pattern. Examples are shown in Figure 1.34 where a number of walls are interconnected or coupled to each other by beams. These walls are generally referred to as coupled shear walls.

The load-resisting mechanisms in a coupled shear wall are shown qualitatively in Figure 1.35b and c. It is seen that the total overturning moment, M, in the wall without openings shown in Figure 1.35a, is resisted at the base entirely by flexural stresses. On the other hand, in the coupled walls shown in Figure 1.35b and c, axial forces as well as moments occur at the base to resist the overturning moment, M, resulting in the following equilibrium statement:

$$M = M_1 + M_2 + Td \tag{1.18}$$

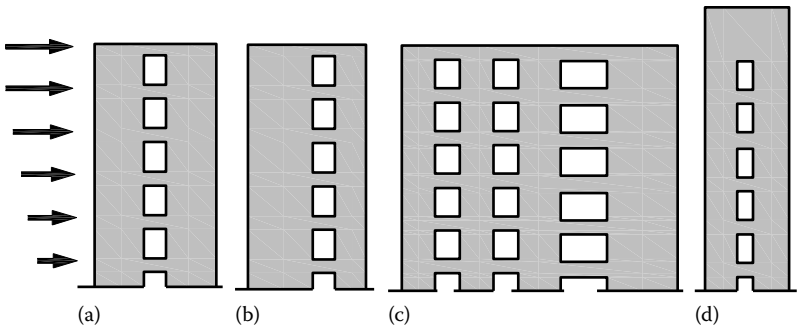


FIGURE 1.34 Shear walls with openings.

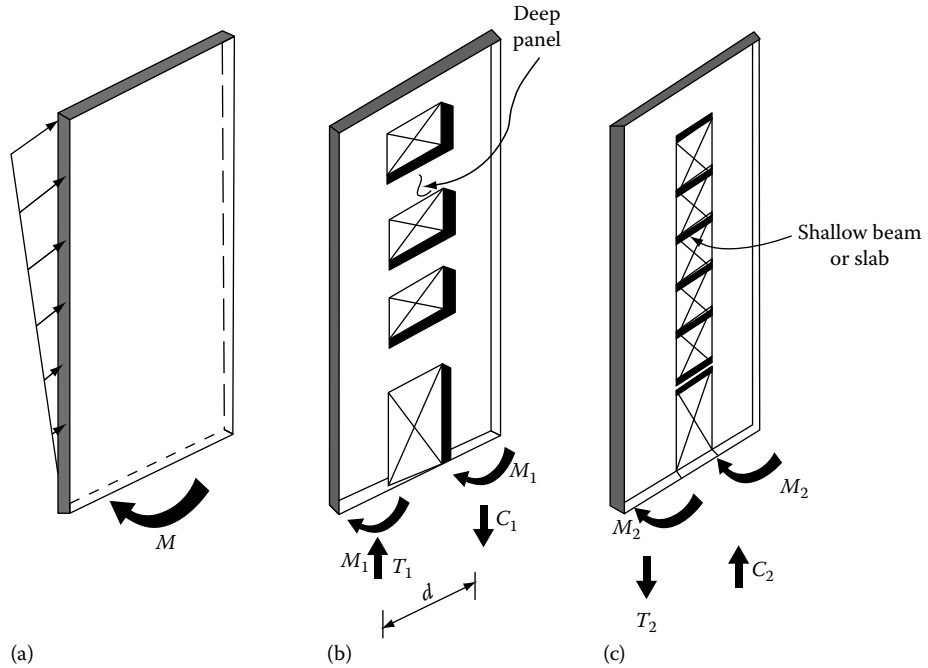


FIGURE 1.35 Lateral load-resistance of single and coupled shear walls.