Shear capacity of slab-column connections under gravity and biaxial cyclic lateral loading

Edward Anggadjaja

2007

Edward Anggadjaja. (2007). Shear capacity of slab-column connections under gravity and biaxial cyclic lateral loading. Doctoral thesis, Nanyang Technological University, Singapore.

<https://hdl.handle.net/10356/39358>

<https://doi.org/10.32657/10356/39358>

9529 589

SHEAR CAPACITY OF SLAB-COLUMN CONNECTIONS UNDER GRAVITY AND BIAXIAL CYCLIC LATERAL LOADING

EDWARD ANGGADJAJA

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING
NANYANG TECHNOLOGICAL UNIVERSITY

2007

**SHEAR CAPACITY OF
SLAB-COLUMN CONNECTIONS UNDER
GRAVITY AND BIAXIAL CYCLIC LATERAL LOADING**

A thesis submitted to the Nanyang Technological University
In partial fulfillment of the requirements for the
Degree of Doctor of Philosophy

Supervised by
ASSOC. PROF. SUSANTO TENG

Submitted by
EDWARD ANGGADJAJA

School of Civil and Environmental Engineering
Nanyang Technological University

2007

ABSTRACT

Reinforced concrete flat plate structures have been widely used for residential and office buildings around the world due to their economical and aesthetical advantages over the conventional beam and slab system. However, the transfer of shear and unbalanced moment at the slab-column connections may result in a sudden and brittle punching failure at these connections. Thus, the economy of the flat plate structure depends on the degree to which the ultimate strength of the slab-column connections can be predicted and utilized.

The purpose of this research program is to investigate the punching behavior and the strength of slab-column connections, considering the effects of column rectangularity, gravity shear ratio, biaxial unbalanced moment, and cyclic lateral loading simulating earthquake effects. For this purpose, a combined experimental and analytical research program was carried out to study these issues.

The experimental program included design, construction, laboratory testing, and analyses of five large-scale rectangular edge-column slab connections subjected to gravity and cyclic lateral loading. Two specimens were uniaxially laterally loaded, two more specimens were bi-axially laterally loaded, and the fifth specimen was tested under gravity load only. Three levels of gravity loading were incorporated in the experimental program.

High column rectangularity ratio was found to contribute to a significant flexibility about the weaker column axis, resulting in a high drift ratio due to column flexure about its minor axis. Column rectangularity also significantly enhances the ability of slab-column connections to transfer more unbalanced moment about the strong column axis. Limit on gravity loads imposed on biaxially laterally loaded rectangular-column slab connections is proposed to be $0.3 V_c$ to ensure 1.5 % drift ratio capacity in both directions. Biaxial cyclic lateral loading was also found to severely reduce the strength, stiffness, ductility and drift capacity of slab-column connections with a high column aspect ratio.

Abstract

A semi-empirical model is proposed to predict the shear strength of slabs based on a comprehensive and systematic study on the databank collected. Only one equation is needed for predicting the punching strength and it includes the influence of flexural reinforcement ratio and the strength degradation due to cyclic loading. The proposed simple equation had been verified against 238 slab data under symmetrical punching and was found to be more accurate than existing ACI code formulas.

A new method is presented for calculating the slab ultimate shear stress at the critical shear perimeter of interior and exterior slab-column connections due to combined gravity load and unbalanced moment transfer. The method is based on an interaction equation for shear and unbalanced moments, assuming that moment transfer capacity is limited solely by the available flexural reinforcement within an effective transfer width of $c_2 + 1.5h$ on each side of the column. Column rectangularity, connection geometry, influence of reinforcement ratio and gravity shear ratio are taken into consideration. The proposed equation has been verified for accuracy by predicting the shear strength of 217 slab-column connections including interior, edge, and corner connections, ranging from square-column slab connections transferring uniaxial unbalanced moment to rectangular-column slab connections transferring biaxial unbalanced moments due to cyclic lateral loading. The proposed formula gives a very good agreement between the predicted and experimental values, and it also simplifies the design and analysis process.

The allowable gravity shear ratio for slab-column connections under cyclic loading is investigated. A function to calculate the allowable gravity shear ratio is established based on the proposed unbalanced moment-and-shear interaction that includes effects of story drift. The equation shows that the gravity shear ratio is not only limited by the ultimate drift ratio capacity DR_u , but also depends on connection geometry d/b_o , column rectangularity β , and the moment transfer capacity M_f .

ACKNOWLEDGEMENT

The author would like to express his deepest gratitude to his supervisor, Dr Susanto Teng who has been giving his continuous guidance, patient supervision, valuable advice and encouragement during the past four years. Without these, the success of the research would be impossible.

The research program was also made possible by the funding from the Ministry of Education and the Building and Construction Authority of Singapore.

The author also wishes to acknowledge all his fellow colleagues and project officers who are under Dr Teng's supervision for their help and contribution in the experimental work. The assistance of the technicians in the Construction Lab and Heavy Structures Lab is greatly appreciated.

Finally, special gratitude for the author's mother and sister who have given so much love and support directly or indirectly during the period of this research.

TABLE OF CONTENTS

	<i>Page</i>
ABSTRACT	<i>i</i>
ACKNOWLEDGEMENT	<i>iii</i>
TABLE OF CONTENTS	<i>iv</i>
LIST OF TABLES	<i>x</i>
LIST OF FIGURES	<i>xiii</i>
LIST OF SYMBOLS	<i>xxii</i>
 CHAPTER 1 INTRODUCTION	
1.1 Background	1
1.2 Objectives	5
1.3 Scope of Research and Report Outline	6
 CHAPTER 2 LITERATURE REVIEW	
2.1 Overview	8
2.2 Connections under Symmetrical Punching	8
2.2.1 Introduction	8
2.2.2 Elastic Models	8
2.2.2.1 Model by Kinnunen and Nylander	8
2.2.2.2 Limiting Shear Stress on Critical Section Model	10
2.2.2.3 Two Phase Method	12
2.2.3 Plastic Models	13
2.2.3.1 Shear Plasticity Model	13
2.2.3.2 Shear Friction Model	15
2.2.3.3 Model by Menetrey	16
2.3 Connections under Unsymmetrical Punching	17
2.3.1 Introduction	17
2.3.2 Elastic Models	18
2.3.2.1 Linear Distribution of Shear Stress on Critical Section	18

Table of Contents

2.3.2.2 Moehle's Model	21
2.3.3 Plastic Models	23
2.3.3.1 Beam Analogy Model	23
2.3.3.2 Strut-and-Tie Model	29
2.4 Connections under Reversed Cyclic Loading	31
2.4.1 Introduction	31
2.4.2 Experimental Research	31
2.4.3 Lateral Drifts Requirements	35
2.4.4 Effects of Gravity Load on Drift Capacity	35
2.4.5 Ductility of Slab-Column Connection	37
2.4.6 Seismic Analysis of Flat Plate	37
2.5 Summary	39
CHAPTER 3 REVIEW ON PUNCHING SHEAR DESIGN CODES	
3.1 Overview	40
3.2 The Absence of Unbalanced Moment	41
3.2.1 American Concrete Institute ACI 318-05	42
3.2.2 Australian Code 3600-1988	43
3.2.3 British Standard BS 8110-97	43
3.2.4 Canadian Standard Association CSA A23.3-04	44
3.2.5 CEB/FIP Model Code 90	45
3.2.6 Eurocode EC 2-2003	46
3.2.7 Japanese Society of Civil Engineers JSCE 1986	47
3.3 The Presence of Unbalanced Moment	47
3.3.1 American Concrete Institute ACI 318-05	47
3.3.2 Australian Code 3600-1988	52
3.3.3 British Standard BS 8110-97	52
3.3.4 Canadian Standard Association CSA A23.3-04	53
3.3.5 CEB/FIP Model Code 90	54
3.3.6 Eurocode EC 2-2003	56
3.3.7 Japanese Society of Civil Engineers JSCE 1986	57
3.4 Summary	58

Table of Contents

CHAPTER 4 EXPERIMENTAL PROGRAM

4.1 Introduction	59
4.2 Definition of Test Specimens	60
4.3 Preliminary Analysis	63
4.4 Reinforcement Layout	68
4.5 Materials	70
4.5.1 Properties of Concrete	70
4.5.2 Properties of Reinforcement	71
4.6 Fabrication of Specimens	72
4.7 Experimental Setup	73
4.7.1 Column Support	75
4.7.2 Vertical Edge Link	75
4.7.3 Torsion Restraining Frame	77
4.8 Instrumentation	77
4.8.1 Load Cells	77
4.8.2 Transducers	78
4.8.3 Steel Strain Gauges	79
4.8.4 Data Logging Equipment	81
4.9 Loading Procedures	81
4.9.1 Column Axial Load	81
4.9.2 Slab Gravity Load	82
4.9.3 Column Cyclic Lateral Displacement	83

CHAPTER 5 EXPERIMENTAL RESULTS AND DISCUSSIONS

5.1 Introduction	86
5.2 Crack Formation and Modes of Failure	86
5.2.1 Stage I (Application of Gravity Loading)	86
5.2.2 Stage II (Application of Cyclic Lateral Displacement)	87
5.3 Hysteretic Response	90
5.3.1 Uniaxial Tests Response	94
5.3.2 Biaxial Tests Response	96
5.4 Strain in Flexural Reinforcements	101

Table of Contents

5.5 Slab Deflection	106
5.6 Connection Strength	110
5.6.1 Effect of Column Rectangularity	110
5.6.2 Effect of Biaxial Loading	112
5.6.3 Effect of Gravity Shear	115
5.7 Connection Stiffness	116
5.6.1 Effect of Column Rectangularity	118
5.6.2 Effect of Biaxial Loading	119
5.6.3 Effect of Gravity Shear	119
5.8 Connection Ductility	120
5.6.1 Effect of Column Rectangularity	122
5.6.2 Effect of Biaxial Loading	122
5.6.3 Effect of Gravity Shear	123
5.9 Connection Drift Capacity	124
5.6.1 Effect of Column Rectangularity	124
5.6.2 Effect of Biaxial Loading	125
5.6.3 Effect of Gravity Shear	126
5.10 Summary and Conclusions	129
CHAPTER 6 PROPOSED PROCEDURE FOR PREDICTING STRENGTH OF SLAB-COLUMN CONNECTIONS	
6.1 Introduction	130
6.2 Main Variables Affecting Punching Shear Strength	131
6.2.1 Concrete Strength	131
6.2.2 Reinforcement Ratio	133
6.3 Proposed Simple Equation	136
6.4 Applied Shear Stress Calculation For Rectangular Columns	138
6.5 Comparison with Experimental Data	139
6.6 Parametric Studies	142
6.6.1 Concrete Strength	145
6.6.2 Reinforcement Ratio	146
6.7 Influence of Size Effect	147

Table of Contents

6.7.1 Proposed Size Effect Factor	148
6.7.2 Predictions With and Without Size Effect Term	151
6.8 Summary	153
CHAPTER 7 PROPOSED PROCEDURE TO CALCULATE ULTIMATE SHEAR STRESS OF SLAB-COLUMN CONNECTIONS WITH UNBALANCED MOMENTS	
7.1 Introduction	154
7.2 Review of Experimental Data	155
7.3 Punching Shear Strength Calculations by ACI Code Method	158
7.4 Proposed Unbalanced Moment-and-Shear Interaction	165
7.5 The Parameter K	168
7.5.1 Effect of Material Properties	168
7.5.2 Effect of Gravity Shear	170
7.5.3 Effect of Connection Geometry	171
7.6 Punching Shear Strength Calculations by Proposed Method	174
7.6.1 Proposed Equation to Calculate Ultimate Shear Stress	174
7.6.2 Comparison with Experimental Data	175
7.7 Parametric Studies	178
7.8 Slab-Column Connections under Cyclic Loading	185
7.8.1 Proposed Shear Strength Degradation due to Cyclic Lateral Loading	185
7.9 Allowable Gravity Shear Ratio	188
7.10 Analysis of Rectangular Edge-Column Slab Connections under Gravity and Biaxial Cyclic Lateral Loadings	191
7.11 The Conservatism in the Moment Transfer Provisions	194
7.12 Complete General Expression Proposed for Design	197
7.13 Summary	198
CHAPTER 8 CONCLUSIONS	200
REFERENCES	204

Table of Contents

APPENDIX A	E-TABS MODEL	219
APPENDIX B	TEST SET-UPS AND SPECIMEN DETAILS	227
APPENDIX C	EXPERIMENTAL PHOTOGRAPHS	238
APPENDIX D	EXPERIMENTAL RESULTS	247
APPENDIX E	SLAB DETAILS AND PREDICTIONS OF 455 SLAB-COLUMN CONNECTIONS DATA	268

LIST OF TABLES

Table No.	Description	Page
Chapter 3		
Table 3.1	Values of K in CEB-FIP Model Code	55
Chapter 4		
Table 4.1	Test Variables of Experimental Program	63
Table 4.2	Concrete Properties	71
Table 4.3	Reinforcement Properties	71
Chapter 5		
Table 5.1	Experimental Peak Unbalanced Moments And Peak Drift Ratios	101
Table 5.2	Experimental Drift Ratio and Ductility Ratio μ and μ_{80}	120
Table 5.3	Test Specimen Data For Edge Connections With Cyclic Moment Transfer	127
Chapter 6		
Table 6.1	Range of Variables in The Data Collected For Symmetrical Punching Case of Slab-Column Connections	136
Table 6.2	Summary of Results For Slab-Column Connections Under Symmetrical Punching	140
Table 6.3(a)	Variation of v_u / v_c based on the concrete strength f'_c	143
Table 6.3(b)	Variation of v_u / v_c based on the flexural reinforcement ratio ρ	144
Table 6.4	Summary of Predictions with and without Size Effect Term For Slab-Column Connections Under Symmetrical Punching	150

List of Tables

Chapter 7

Table 7.1	Range of Variables in The Data Collected For Slab-Column Connections With Unbalanced Moment Transfer	162
Table 7.2	Summary of Results For Slab-Column Connections With Moment Transfer	176
Table 7.3(a)	Variation of v_u / v_c based on the concrete strength f'_c	179
Table 7.3(b)	Variation of v_u / v_c based on the flexural reinforcement ratio ρ	180
Table 7.3(c)	Variation of v_u / v_c based on the gravity shear ratio V_u/V_o	181
Table 7.3(d)	Variation of v_u / v_c based on the connection geometry d/b_o	182
Table 7.3(e)	Variation of v_u / v_c based on the slab effective depth d	183
Table 7.3(f)	Variation of v_u / v_c based on the critical shear perimeter b_o	184
Table 7.4	Experimental Data for Rectangular Edge-Column Slab Connections under Gravity and Cyclic Lateral Loadings	192
Table 7.5	Comparison of Experimental Results and Predicted Strengths for Rectangular Edge-Column Slab Connections under Gravity and Cyclic Lateral Loadings (Author's specimens)	192

Appendix B

Table B.1	Steel Tensile Test Results for T10 Reinforcement Bars	234
Table B.2	Steel Tensile Test Results for T13 Reinforcement Bars	235
Table B.3	Steel Tensile Test Results for T20 Reinforcement Bars	236

Appendix E

Table E.1	Slab Details and Predictions for Interior Connections with Square Columns under Symmetrical Punching	268
Table E.2	Slab Details and Predictions for Interior Connections with Rectangular Columns under Symmetrical Punching	274
Table E.3	Slab Details and Predictions for Interior Connections with Square Columns Transferring Unbalanced Moment	275
Table E.4	Slab Details and Predictions for Interior Connections with Rectangular Columns Transferring Unbalanced Moment	278

List of Tables

Table E.5	Slab Details and Predictions for Edge Connections with Square Columns	279
Table E.6	Slab Details and Predictions for Edge Connections with Rectangular Columns	281
Table E.7	Slab Details and Predictions for Corner Connections with Square Columns	282
Table E.8	Slab Details and Predictions for Corner Connections with Rectangular Columns	284
Table E.9	Slab Details and Predictions for Slab-Column Connections under Cyclic Loading	285
Table E.10	The Error due to Moment Transfer Provisions (Slab-Column Connections With Zero or Negligible Direct Shear Force V_u)	287

LIST OF FIGURES

Figure No.	Description	Page
Chapter 1		
Fig. 1.1	Flat Plate	1
Fig. 1.2	Punching Shear Failure	2
Chapter 2		
Fig. 2.1	Mechanical Model by Kinnunen/Nylander (1960)	9
Fig. 2.2	Plastic Model by Braestrup and Nielsen (1976)	14
Fig. 2.3	Menetrey's Model (1996)	16
Fig. 2.4	Typical Crack Patterns of Punching Failure	17
Fig. 2.5	Assumed Shear Stress Distribution by Di Stasio and Van Buren (1960)	19
Fig. 2.6	Moe's model (1961)	20
Fig. 2.7	Moehle's Moment-Shear Interaction Diagram (1988)	22
Fig. 2.8	Effective Transfer Width by Moehle	23
Fig. 2.9	Beam Analogy for Interior Slab-Column Connections	24
Fig. 2.10	Stamenkovic and Chapman's Moment-Shear Interaction Diagram (1969)	26
Fig. 2.11	Beam Analogy Model by Islam and Park (1976)	26
Fig. 2.12	Kanoh and Yoshizaki's Moment-Shear Interaction Diagram (1979)	27
Fig. 2.13	Beam Analogy Model by Alexander and Simmonds (1992)	29
Fig. 2.14	Truss Model Mechanism by Alexander (1987)	29
Fig. 2.15	Truss Model Interaction Diagram	30
Fig. 2.16	Cyclic Lateral Force Simulation	32
Fig. 2.17	Effects of Gravity Loads on Lateral Drift Capacity (Megally, 2000)	36

*List of Figures***Chapter 3**

Fig. 3.1	The Peripheral Length of Critical Section for Different Codes	41
Fig. 3.2	Location of Critical Shear Perimeters (ACI 318-05)	43
Fig. 3.3	Location of Shear Perimeter in BS8110: 97	44
Fig. 3.4	Consideration of Rectangular Columns in EC2	46
Fig. 3.5	Shear Stresses due to Shear and Moment Transfer at an Interior Connections	49
Fig. 3.6	Critical Sections and Shear Stress Distribution for Edge Connections	50
Fig. 3.7	Critical Sections and Shear Stress Distribution for Corner Connections	51
Fig. 3.8	Control Perimeter at Edge Columns in CEB-FIP Model Code	56
Fig. 3.9	Control Perimeter at Corner Columns in CEB-FIP Model Code	56

Chapter 4

Fig. 4.1	Geometry of Test Specimens	61
Fig. 4.2	Test Variables in terms of Gravity and Lateral Load Applications	62
Fig. 4.3	Plan View of Prototype Building	64
Fig. 4.4	Deflection Profile of Prototype and Model under Gravity Load	66
Fig. 4.5	Deflection Profile of Prototype and Model under Lateral Load	66
Fig. 4.6	Deflection Profile of Prototype and Model under Gravity and Lateral Load	67
Fig. 4.7	Flexural Reinforcement Layout of Slab Specimens	69
Fig. 4.8	Column Reinforcement Details of Test Specimens	69
Fig. 4.9	Anchorage Details of Top and Bottom Slab Steel	70
Fig. 4.10	Experimental Setup	73
Fig. 4.11	Diagram of Experimental Setup	74
Fig. 4.12	Details of Column Support	75
Fig. 4.13	Details of Vertical Edge Link	76
Fig. 4.14	Details of Torsion Restraining Frame	76
Fig. 4.15	Locations of Various Transducers on the Test Specimens	79
Fig. 4.16	Location of Strain Gauges in Top Reinforcement Bars	80

List of Figures

Fig. 4.17	Location of Strain Gauges in Bottom Reinforcement Bars	80
Fig. 4.18	Loading Points on The Top Column	81
Fig. 4.19	Steel Blocks on Slab Surface	82
Fig. 4.20	Target Drift Ratio for Uniaxial and Biaxial Specimens	84
Fig. 4.21	Biaxial Displacement Path for Biaxial Specimens	84
Fig. 4.22	Sign Convention for Lateral Loading	84
Fig. 4.23	Test Specimen E1H Subjected to Large Drift Before Failure	85

Chapter 5

Fig. 5.1a	Punching Shear Failure under Gravity and Cyclic Lateral Loading	89
Fig. 5.1b	Punching Shear Failure under Gravity Loading	90
Fig. 5.2	Hysteretic Loop for Specimen E1H	91
Fig. 5.3	Hysteretic Loop for Specimen E2H	92
Fig. 5.4	Biaxial Hysteretic Loops (North-South and East-West) for Specimen E12L	93
Fig. 5.5	Biaxial Hysteretic Loops (North-South and East-West) for Specimen E12H	94
Fig. 5.6	Load Displacement Sequences for Biaxially Loaded Specimens	98
Fig. 5.7	Yielded Bar Locations in Specimen E1H	103
Fig. 5.8	Yielded Bar Locations in Specimen E2H	103
Fig. 5.9	Yielded Bar Locations in Specimen E12H	104
Fig. 5.10	Yielded Bar Locations in Specimen E12L	104
Fig. 5.11	Yielded Bar Locations in Specimen E0U	105
Fig. 5.12	Strains in Top Flexural Reinforcement Bars Along N-S Directions	105
Fig. 5.13	Strains in Top Flexural Reinforcement Bars Along E-W Directions	106
Fig. 5.14	Column Vertical Displacement versus Lateral Loading	108
Fig. 5.15	Column Vertical Displacement versus Drift Ratio	108
Fig. 5.16	Slab Deflection Profiles along East-West Span Direction	109
Fig. 5.17	Slab Deflection Profiles along North-South Span Direction	109

List of Figures

Fig. 5.18	Effect of Column Rectangularity on Moment Transfer and Drift Capacity	111
Fig. 5.19	Effect of Biaxial Loading on Moment Transfer and Drift Capacity	112
Fig. 5.20	Interaction of Peak Lateral Loads for Specimen E12H	114
Fig. 5.21	Interaction of Peak Lateral Loads for Specimen E12L	115
Fig. 5.22	Gravity Load Effect for Rectangular Edge-Column Slab Connections	116
Fig. 5.23	Envelopes of Unbalanced Moment Versus Drift Ratio	117
Fig. 5.24	Definition of Stiffness Parameter S	117
Fig. 5.25	Stiffness Parameter S Versus Drift Ratio	118
Fig. 5.26	Definition of DR_y , DR_u and DR_{u80}	121
Fig. 5.27	Experimental Peak Drift Ratios versus Gravity Shear Ratios	125
Fig. 5.28	Gravity Load Limit for Rectangular-Column Slab Connections with Biaxial Loadings	128
 Chapter 6		
Fig. 6.1	Relationship Between Nominal Shear Stress and Concrete Strength	132
Fig. 6.2	Relationship Between Shear Strength Ratio v_u/v_c and Reinforcement Ratio	134
Fig. 6.3	Relationship Between Nominal Shear Stress and Reinforcement Ratio	135
Fig. 6.4.1(a)	Relationship Between Nominal Shear Stress and Concrete Strength Based on Sets of Experimental Data Tested by Elstner and Hognestad (1956)	138(a)
Fig. 6.4.1(b)	Relationship Between Nominal Shear Stress and Reinforcement Ratio Based on Sets of Experimental Data Tested by Elstner and Hognestad (1956)	138(a)
Fig. 6.4	Location of Critical Shear Perimeters (Proposed Method)	139
Fig. 6.5	Comparison Between the ACI method and the Proposed Method	141

List of Figures

Fig. 6.6a	v_u / v_c versus Concrete Compressive Strength	143
Fig. 6.6b	v_u / v_c versus Flexural Reinforcement Ratio	144
Fig. 6.7	Relationship Between Punching Shear Strength and Concrete Strength	145
Fig. 6.8	Relationship Between Punching Shear Strength and Reinforcement Ratio	146
Fig. 6.9	Relationship Between Nominal Shear Stress and Slab Effective Depth	147
Fig. 6.10	Size Effect Law for Blunt Fracture	148
Fig. 6.11	v_u / v_c versus Slab Effective Depth	150
Fig. 6.12	Predictions with and without Size Effect Term	151
Chapter 7		
Fig. 7.1	Shear Strength Ratio Based on ACI Method	160
Fig. 7.2	Unbalanced Moment-and-Shear Interaction Diagram Based on ACI Method	161
Fig. 7.3	Shear Strength Ratio v_u/v_c Calculated According to ACI 318-05	163
Fig. 7.4	Predicted Unbalanced Moment-and-Shear Interaction Based on ACI 318-05	164
Fig. 7.5	Comparison Between the ACI method and the Proposed Method	166
Fig. 7.6	Relationship Between The Parameter K and Concrete Cylinder Strength f'_c	169
Fig. 7.7	Relationship Between The Parameter K and Flexural Reinforcement Ratio ρ	169
Fig. 7.8	Relationship Between The Parameter K and Gravity Shear Ratio V_u/V_o	171
Fig. 7.9	Relationship Between The Parameter K and Connection Geometry d/b_o	172
Fig. 7.10	Predicted Unbalanced Moment-and-Shear Interaction Based on Proposed Method	173
Fig. 7.11	The Shear Strength Ratios v_u/v_c Calculated According to	

List of Figures

	Proposed Method	177
Fig. 7.12(a)	v_u / v_c versus Concrete Strength f'_c	179
Fig. 7.12(b)	v_u / v_c versus Flexural Reinforcement Ratio ρ	180
Fig. 7.12(c)	v_u / v_c versus Gravity Shear Ratio V_u/V_o	181
Fig. 7.12(d)	v_u / v_c versus Connection Geometry d/b_o	182
Fig. 7.12(e)	v_u / v_c versus Slab Effective Depth d	183
Fig. 7.12(f)	v_u / v_c versus Critical Shear Perimeter b_o	184
Fig. 7.13	Shear Strength Ratio v_u/v_c Calculated Using The Proposed Shear Strength Equation that Includes Shear Strength Degradation due to Cyclic Lateral Loading	186
Fig. 7.14	Comparison Between The Original ACI Equations and The ACI Equations that Includes The Proposed Shear Strength Degradation due to Cyclic Lateral Loading	187
Fig. 7.15	Gravity Shear Effect on Story Drift Ratio Capacity Based on ACI Equations	188
Fig. 7.16	Gravity Shear Effect on Story Drift Ratio Capacity Based on Proposed Equation	190
Appendix A		
Fig. A.1	Prototype Structure Modeled by ETABS	219
Fig. A.2	Deflection and Stress Contour of Prototype under Gravity Load	220
Fig. A.3	Deflection and Stress Contour of Prototype under Lateral Load (x-direction)	221
Fig. A.4	Deflection and Stress Contour of Prototype under Lateral Load (y-direction)	222
Fig. A.5	Test Specimen Modeled by ETABS	223
Fig. A.6	Deflection and Stress Contour of Model under Gravity Load	224
Fig. A.7	Deflection and Stress Contour of Model under Lateral Load (x-direction)	225
Fig. A.8	Deflection and Stress Contour of Model under Lateral Load (y-direction)	226

*List of Figures***Appendix B**

Fig. B.1	Schematic Diagram of Test Set-up	227
Fig. B.2	Test Set-up : Top Arms Assembly	228
Fig. B.3	Test Set-up : Bottom Arms Assembly	229
Fig. B.4	Test Set-up : Top Column Assembly	230
Fig. B.5	Test Set-up : Bottom Column Assembly	231
Fig. B.6	Test Set-up : Torsion-Restraining Frame Assembly	232
Fig. B.7	Test Set-up : Edge Links Assembly	234
Fig. B.8	Sample of Concrete Strength vs Curing Period Graph	237
Fig. B.9	Sample of Stress vs Strain Graph for T13 Bars	237

Appendix C

Fig. C.1	Formwork of Specimen Before Casting	238
Fig. C.2	Casting of Specimen	238
Fig. C.3	Experimental Set-up	239
Fig. C.4	Top Arm Connected to A-frame Stand	239
Fig. C.5	Frame with WLVDT Attached	240
Fig. C.6	Torsion-Restraining Frame Connected to Steel Links	240
Fig. C.7	Top Arm Pin-Pin Connection Attached to The A-frame	241
Fig. C.8	Bottom Arm Pin-Pin Connection Attached to The A-frame	241
Fig. C.9	WLVDT Connected to The Edge of Column	242
Fig. C.10	LVDTs Located Below The Slab	242
Fig. C.11	Steel Blocks Simulating Gravity Load	243
Fig. C.12	Sample of Cracks at Very Low Drift Ratio (0.3 %)	243
Fig. C.13	Specimen E1H Subjected to High Drift at Failure	244
Fig. C.14	Sample of Punching Shear Cracks	244
Fig. C.15	Top Slab Showing U-shaped Punching Failure	245
Fig. C.16	Bottom Slab Showing Flexural Cracks	245
Fig. C.17	Exposed Bottom Reinforcement Bar Besides Column at Failure	246
Fig. C.18	Typical Diagonal Shear Cracks	246

*List of Figures***Appendix D**

Fig. D.1	Deflection vs Location of LVDT 4,9,14,16(Positive drift)	247
Fig. D.2	Deflection vs Location of LVDT 4,9,14,16(Negative drift)	247
Fig. D.3	Deflection vs Location of LVDT 3,4,5,6,7 (Positive drift)	248
Fig. D.4	Deflection vs Location of LVDT 3,4,5,6,7 (Negative drift)	248
Fig. D.5	Deflection vs Location of LVDT 4,9,14,16(Positive drift)	249
Fig. D.6	Deflection vs Location of LVDT 4,9,14,16(Negative drift)	249
Fig. D.7	Deflection vs Location of LVDT 3,4,5,6,7 (Positive drift)	250
Fig. D.8	Deflection vs Location of LVDT 3,4,5,6,7 (Negative drift)	250
Fig. D.9	Deflection vs Location of LVDT 4,9,14,16(Positive drift)	251
Fig. D.10	Deflection vs Location of LVDT 4,9,14,16(Negative drift)	251
Fig. D.11	Deflection vs Location of LVDT 3,4,5,6,7 (Positive drift)	252
Fig. D.12	Deflection vs Location of LVDT 3,4,5,6,7 (Negative drift)	252
Fig. D.13	Deflection vs Location of LVDT 4,9,14,16(Positive drift)	253
Fig. D.14	Deflection vs Location of LVDT 4,9,14,16(Negative drift)	253
Fig. D.15	Deflection vs Location of LVDT 3,4,5,6,7 (Positive drift)	254
Fig. D.16	Deflection vs Location of LVDT 3,4,5,6,7 (Negative drift)	254
Fig. D.17	Deflection vs Location of LVDT 4,9,14,16	255
Fig. D.18	Deflection vs Location of LVDT 6,11,18	255
Fig. D.19	Deflection vs Location of LVDT 3,4,5,6,7	256
Fig. D.20	Location of Strain Gauges at The Slab Top Reinforcement	257
Fig. D.21	Location of Strain Gauges at The Slab Bottom Reinforcement	257
Fig. D.22	Strain vs Location of gauges – top row b (Positive Drift)	258
Fig. D.23	Strain vs Location of gauges – top row b (Negative Drift)	258
Fig. D.24	Strain vs Location of gauges – top column c (Positive Drift)	259
Fig. D.25	Strain vs Location of gauges – top column c (Negative Drift)	259
Fig. D.26	Strain vs Location of gauges – top row b (Positive Drift)	260
Fig. D.27	Strain vs Location of gauges – top row b (Negative Drift)	260
Fig. D.28	Strain vs Location of gauges – top column c (Positive Drift)	261
Fig. D.29	Strain vs Location of gauges – top column c (Negative Drift)	261
Fig. D.30	Strain vs Location of gauges – top row b (Positive Drift)	262
Fig. D.31	Strain vs Location of gauges – top row b (Negative Drift)	262

List of Figures

Fig. D.32	Strain vs Location of gauges – top column c (Positive Drift)	263
Fig. D.33	Strain vs Location of gauges – top column c (Negative Drift)	263
Fig. D.34	Strain vs Location of gauges – top row b (Positive Drift)	264
Fig. D.35	Strain vs Location of gauges – top row b (Negative Drift)	264
Fig. D.36	Strain vs Location of gauges – top column c (Positive Drift)	265
Fig. D.37	Strain vs Location of gauges – top column c (Negative Drift)	265
Fig. D.38	Strain vs Location of Strain gauges – top row a	266
Fig. D.39	Strain vs Location of Strain gauges – top row b	266
Fig. D.40	Strain vs Location of Strain gauges – top row d	267
Fig. D.41	Strain vs Location of Strain gauges – top column c	267

LIST OF SYMBOLS

A	= dimension of the critical shear perimeter measured parallel to direction of M_v^* defined in AS 3600-1988
A_{steel}	= cross sectional area of steel tension ties
d	= thickness of the structure in Bazant and Cao (1987)
B	= diameter of the column in Kinnunen/Nylander approach
b_o	= perimeter of shear critical section
b_1	= total width of the critical section measured perpendicular to the axis about which the moment acts
b_2	= total width of the critical section measured parallel to the axis about which the moment acts
c	= length of column side
c	= distance from the centroidal axis (Z-Z) of the shear perimeter in Fig. 3.5 to the point where the shear stresses are being calculated
c_1	= column dimension parallel to the eccentricity of the load defined in MC 90
c_2	= column dimension perpendicular to the eccentricity of the load defined in MC 90
c_x	= distance from the centroidal axis in x-direction of the shear perimeter to the point where the shear stresses are being calculated
c_y	= distance from the centroidal axis in y-direction of the shear perimeter to the point where the shear stresses are being calculated
d	= diameter of loaded area in Jiang and Shen (1986)
d	= effective depth of slab
d_1	= diameter at base of failure cone in Jiang and Shen (1986)
d_1, d_2	= effective depths of tension steel, face r_1 and face r_2 respectively defined by Hawkins and Corley (1971)
d_1'	= effective depth of compression steel
d_a	= maximum aggregate size

List of Symbols

DR_y	= drift ratio at first yield of slab reinforcement
DR_u	= drift ratio at peak lateral load
DR_{u80}	= drift ratio at which the lateral load has dropped to 80 percent of the peak load
E_c	= modulus of elasticity of concrete
E_s	= modulus of elasticity of reinforcing bars
e_x	= eccentricity in x-direction
e_y	= eccentricity in y-direction
$f(\alpha)$	= function of the inclination α of the conical shell in Kinnunen/Nylander approach
F_c	= concrete strength by integrating the vertical tensile stresses in concrete defined by Menetrey (1996)
f'_c	= characteristic cylinder strength of concrete
f_{ck}	= characteristic compressive strength of concrete in MPa defined in MC 90
f_{ck2}	= characteristic compressive strength of cracked concrete in MPa defined in MC 90
f_{cu}	= characteristic cube strength of concrete
F_{dow}	= resistance dowel force due to horizontal reinforcement defined by Menetrey (1996)
F_{prt}	= vertical projection of pre-stressing tendon strength defined by Menetrey (1996)
F_{pun}	= punching strength of a general slab defined by Menetrey (1996)
f_s	= stresses in the steel tension ties
F_{sd}	= concentrated force defined in MC 90
f_{SP}	= splitting tensile strength of concrete
F_{sti}	= vertical strength of shear reinforcement that are well-anchored defined by Menetrey (1996)
f'_t	= effective tensile strength of concrete
f_y	= yield stress of reinforcing bars
f_{D1}	= story drift effect on gravity shear ratio
f_{D2}	= story drift effect on unbalanced moment ratio

List of Symbols

h	= thickness of concrete slabs
J_c	= properties of critical section analogous to polar moment of inertia
k	= shear strength degradation factor
K	= parameter that relates shear and unbalanced moment ratio
l_1	= length of shorter side of critical section
l_2	= length of longer side of critical section
M_v^*	= bending moment transferred from the slab to the column defined in AS 3600-1988
M_t	= design moment defined in BS 8110
M_u	= ultimate unbalanced moment acting at the centroid of the column cross section
M_s	= ultimate unbalanced moment acting at the centroid of the slab critical section
M_u/M_f	= unbalanced moment ratio
M_o	= moment capacity in absence of shear force, respectively
M_f	= flexural capacity of slab reinforcement within the prescribed transfer width of $c_2 + 1.5h$ on each side of the column
P_{vf}	= predicted flexural capacity in Rankin and Long's model
P_{vs}	= predicted shear capacity in Rankin and Long's model
S	= peak-to-peak stiffness parameter
V_u	= ultimate shear force acting at the centroid of the column
V_o	= shear capacity in absence of unbalanced moment
V_u/V_o	= gravity shear ratio
v_c	= slab shear capacity
v_u	= slab ultimate shear stress
v_u/v_c	= shear strength ratio (ratio of the experimental to the predicted shear strength)
α_s	= factor to consider the column location in ACI 318-05
β_R	= ratio of long side to short side of the critical shear perimeter
β	= ratio of long side to short side of the column cross section

List of Symbols

γ_v	= fraction of unbalanced moment transferred by shear
γ_f	= fraction of unbalanced moment transferred by flexure
ρ	= slab flexural reinforcement ratio
μ	= displacement ductility calculated as DR_u / DR_y
μ_{80}	= displacement ductility calculated as DR_{u80} / DR_y
σ_l	= stress in the conical shell of Kinnunen and Nylander's model
ϕ_o	= ratio between the theoretical capacity of the connections based on shear and flexural strength

CHAPTER 1

INTRODUCTION

CHAPTER 1

INTRODUCTION

1.1 Background

One of the various types of two-way reinforced concrete slabs is the flat plate floor system. This floor system consists of a slab of uniform thickness supported directly on columns without the use of intermediary beams, as shown in Figure 1.1. The flat plate floor system has become widely adopted due to its economical and aesthetical advantages compared to the conventional beam and slab system. Because of the absence of beams, flat plates provide architectural flexibility, more clear space, less total building height, easier formwork, and consequently shorter construction time.

Figure 1.1 Flat Plate

However, a serious problem that may arise in the flat plate floor system is brittle punching failure due to transfer of shearing forces and unbalanced moments between slabs and columns. Figure 1.2 illustrates the location of diagonal shear cracks developed within the slab thickness, which leads to brittle punching failure at the slab-column connection. Up to or close to failure, the diagonal shear cracks are not visible on the slab surfaces. Once the shear strength is reached, the diagonal shear cracks cross the full slab thickness forming a cone that, with the supporting column, punches through the slab. This is also accompanied by a sudden drop in the load-carrying capacity of the slab. Therefore, the economy of the entire structure largely depends by the degree to which the ultimate strength of slab-column connections can be predicted and utilized.

Figure 1.2 Punching Shear Failure

There are mainly two areas related to the punching problems of slab-column connections, namely symmetrical punching, i.e. without unbalanced moments, and unsymmetrical punching which includes the condition with unbalanced moments. Typically, unbalanced moments are transferred between slabs and columns in the following cases:

1. At interior columns, under gravity loads that are different on adjacent spans,
2. At interior columns, with different span length of adjacent panels,
3. At interior columns, in exterior slab panels,
4. At exterior columns,
5. At interior and exterior columns, under gravity and lateral loads.

In the past, most research works were done on the strength of slab-column connections under symmetrical conditions. Many theoretical analyses and experimental studies have been carried out. Talbot (1913) introduced the first model of shear transfer for slab-column connections based on a limiting shear stress on critical section. This model was later developed by other researchers (Hognestad 1953; Moe 1961; Herzog 1970; Hawkins 1971), and was finally adopted by the ACI-ASCE Committee 326 (1962) due to its simplicity. Kinnunen/Nylander (1960) proposed a mechanical model for analyzing the symmetrical punching based on experiments on polar-symmetrical slabs supported on columns. Yitzhaki (1966), and Rankin and Long (1987) suggested a two-phase method which was based on

flexural and shear failure. Some other researchers tried to use a plasticity approach to analyze this problem (Braestrup and Nielsen 1976; Jiang and Shen 1986; Bortolotti 1990). In the model by Menetrey (1996), contribution of dowel effect, shear reinforcement, and prestressing tendons were taken into account to calculate the shear strength. Recently a shear friction model for beams previously developed by Loov (1978) was extended to concentric punching of slab-column connections by Dechka (2001).

There are also some mechanical models to treat the condition with unbalanced moments. Di Stasio and Van Buren (1960) presented a working stress method for the strength of slab-column connections under combined shear and unbalanced moment. The model was based on a linear interaction between shear and unbalanced moment, and was later modified by Moe (1961), Hanson and Hanson (1968), and the ACI-ASCE committee 326. The model by Di Stasio and Van Buren was later adopted in the 1971 ACI Code with some minor modifications. Moehle (1988) reviewed the experimental data on edge connections based on the ACI Code equations and found that there was no significant interaction between shear and moment strength. Moehle's proposal was the basis for a new recommendation introduced for the first time in the ACI 318-95.

Stamenkovic and Chapman (1969) used beam analogy to develop an interaction expression of shear-and-unbalanced moment for edge-column slab connections. They assumed a width for each beam equal to the width of the column face. Hawkins and Corley (1971) also used this concept to develop an interaction diagram for interior slab-column connections transferring shear and unbalanced moment. Islam and Park (1976) and Kanoh and Yoshizaki (1979) also proposed a design procedure for interior slab-column connections with bi-linear and linear interaction of shear and unbalanced moment transfer, respectively. Alexander and Simmonds (1987) proposed a strut-and-tie model for the analysis of slab-column connections representing the internal force flow within a slab-column connection. By re-arranging the strut-and-ties around the column, various combinations of vertical shear and unbalanced moment can be found, and the interaction diagrams for both interior and exterior slab-column connections can be established.

Design codes for shear strength of slabs are written in terms of nominal shear stresses calculated with reference to a control perimeter around the column. For the condition without moment transfer, code rules differ mainly in the definition of the control perimeter, calculation of the nominal shear stress and the choice of concrete strength parameter. For the condition with unbalanced moment transfer, there are two common groups of methods adopted by codes of practice. One group is based on elastic theory, while the other group is based on plasticity theory.

The elastic theory assumed that the shear stresses resulting from vertical shear force and unbalanced moment could be added together. So, if there are biaxial unbalanced moments, the shear stresses due to the two unbalanced moments can be added together as well. Failure is assumed to occur when the maximum sum of the shear stresses reaches a limiting value. For the plasticity theory, the effective shear force will be increased according to the magnitude of the unbalanced moments and the type of the connection. The shear stress is then calculated based on the effective shear force. The two unbalanced moments in the two perpendicular directions are considered independently and the worst case is the one that controls the design.

Although slab-column connections may not be accounted for as part of the lateral-force resisting system, they still should have the ability to go through the lateral deformations of the lateral-force resisting system without loss of ability to support the gravity loads during an earthquake. Hence, this indicates that the effect of load reversal on punching shear strength of concrete slabs has to be taken into account in the design of slab-column connections since the applied shear stresses must not exceed the shear resistance at the given level of deterioration. Present design recommendations for flat plates under seismic loads are based on experimental researches on slab-column connections subjected to cyclic loading. Although there have been some studies on slab-column connections under uniaxial cyclic lateral loads, there is a lack of experimental data on the biaxial cyclic lateral loads, particularly for edge connection. Besides, experiments on slab-column connections involving rectangular columns are also very limited. To further study important issues related to slab-column connections, an experimental program was conducted by the authors on rectangular edge-column slab connections subjected to gravity loads only, and uniaxial as well as biaxial cyclic lateral loadings.

As discussed above, there is a need to study further the strength of slab-column connections experimentally and analytically in order to obtain a more reliable and complete design method. The method should be capable of predicting the shear capacity as well as the ultimate shear stress of both interior and exterior slab-column connections under symmetrical punching, unsymmetrical punching, and also under cyclic loading conditions. In addition, the model should lead to a simple design procedure without significant loss of accuracy over a wide range of relevant parameters, such as connection geometry, flexural reinforcement, and concrete properties.

1.2 Objectives

The main objective of this research is to study the punching strength of slab-column connections with shear and unbalanced moment transfer. In this research, effects of column rectangularity, gravity shear ratio, biaxial unbalanced moment, and cyclic lateral loading simulating earthquake effects are studied. The investigation is based on a series of experimental and analytical studies.

The major contributions and originality of the research work include:

- An original experimental work including test set-ups and specimens design, construction and laboratory testing of five large-scale rectangular edge-column slab connections subjected to gravity and cyclic lateral loading
- Development of a new method and new equations that are able to describe a consistent relationship between shearing forces and unbalanced moment for all punching cases of interior and exterior slab-column connections including those with rectangular columns.

Other contributions include:

- A comprehensive review of the relevant literature, design codes and a databank of 455 previous experimental results collected for interior, edge and corner slab-column connections.

- Detailed evaluation and interpretation of the experimental results, which give valuable insights for design engineers and researchers dealing with column rectangularity and cyclic lateral loadings in flat plate structures.
- Development of a new semi empirical model for predicting the shear strength of slabs, taking into account the effect of flexural reinforcement ratio and the shear strength degradation for connections under cyclic loading.
- Development of a new equation to calculate the allowable gravity shear ratio for slab-column connections under cyclic lateral loading.

1.3 Scope of Research and Report Outline

This research focuses on the punching shear of slab-column connections without drop panels, shear capitals, or any forms of shear reinforcement. All the considered slabs are made of non-prestressed reinforced concrete.

Chapter 1 gives an introduction for this study, including the background of flat plate floor structures. A brief literature review on research studies of slab-column connections for flat plate structures is included. The objective of this research is discussed as well.

Chapter 2 continues the introductory material with a detailed discussion on past research works on punching strength of slab-column connections. The discussion includes both analytical and empirical models for connections under symmetrical and unsymmetrical punching problems, as well as connections under cyclic loading.

Chapter 3 summarizes different code methods currently used (ACI 318-05, BS8110-1997, CEB-FIP Model Code 1990 and other codes) to estimate the strength of slab-column connections from concentric condition without unbalanced moment to the condition with unbalanced moments. Method for analysis of slab-column connections under cyclic loading is also discussed.

Chapter 4 describes details of the experimental work of this research. A total of five large-scale reinforced concrete flat plate edge connections with rectangular columns were tested to failure under combined gravity and cyclic lateral loading.

Chapter 5 gives discussions on the experimental result for each of the tested specimens. The strength, stiffness, ductility, and drift capacity with respect to the experimental variables are studied. The variables include effect of column rectangularity, biaxial cyclic lateral loadings, and levels of gravity loadings. Lastly, conclusions and important findings of the experimental study are made.

Chapter 6 proposes a semi-empirical method for predicting the shear strength of slabs based on a comprehensive study on relatively complete databank collected from previous experimental works. Only one equation is needed for predicting the punching strength and it includes the influence of flexural reinforcement ratio. For applied shear stress calculation, a coefficient is introduced to the direct stress component considering the rectangularity of the columns.

Chapter 7 proposes new approach for calculating the slab ultimate shear stress at the critical shear perimeter of interior, edge and corner slab-column connections due to combined gravity load and unbalanced moment transfer. Column rectangularity, connection geometry, influence of reinforcement ratio and gravity shear ratio are taken into consideration. The proposed formula gives a very good agreement between the predicted and experimental values, and it also simplifies the design and analysis process. A simple expression is also proposed to consider shear strength degradation for connections under cyclic loading. The ACI Code provision on gravity shear ratio limit for seismic-resistant slab-column connections is reviewed. An equation to calculate the allowable gravity shear ratio is established based on the proposed moment-and-shear interaction that includes effect of story drift.

Conclusions and recommendations for future research works are given in Chapter 8.

CHAPTER 2

LITERATURE REVIEW

CHAPTER 2

LITERATURE REVIEW

2.1 Overview

Much theoretical and experimental research has been conducted for the analysis on punching strength of slab-column connections with and without unbalanced moment transfer. The research ranges from the investigation on the effect of slab properties to attempts to propose new theoretical models. This chapter provides a relatively complete, but not exhaustive, review on the previous research works from symmetrical to unsymmetrical punching problem, and also for connections under cyclic loading simulating earthquake effects. The critical codes review will be given in Chapter 3.

2.2 Connections under Symmetrical Punching

2.2.1 Introduction

In this section, models proposed to explain the behavior of slab-column connections subjected to symmetrical punching are presented. These can generally be divided into models based on elastic behavior, and models based on plastic behavior. Some of the models described below are specifically meant for symmetrical punching, while others have subsequently been expanded to include unbalanced moments, which will be discussed further in the next section.

2.2.2 Elastic Models

2.2.2.1 Model by Kinnunen and Nylander (1960)

Kinnunen and Nylander (1960) derived a mechanical model to explain the punching shear phenomenon and to predict the failure load of slab-column connections. This model is based on the equilibrium of forces acting on a polar-symmetrical slab supported on a column (see Fig 2.1). In the model, the slab portion

outside the shear crack is carried by a compressed conical shell, which develops from the column to the root of the shear crack. Failure is assumed to occur when the tangential concrete strain at the bottom surface of the slab reaches a critical value. The model was calibrated with measured tangential concrete strains from punching tests on slab with ring reinforcement. The equation of the vertical projection gives the ultimate load for two-way reinforced slabs without shear reinforcement:

$$P_u = 1.1\pi \frac{Bx}{d^2} \frac{(1+2x/B)}{(1+x/B)} f(\alpha) \sigma_t d^2 \quad (2.1)$$

where,

B = diameter of the column;

x = distance from the slab bottom surface to the root of the shear crack;

$f(\alpha)$ = a function of the inclination α of the conical shell;

σ_t = stress in the conical shell;

d = effective depth of the slab.

The factor 1.1 in Eq. (2.1) is applied to compensate for the dowel action of the reinforcement.

Figure 2.1 Mechanical Model by Kinnunen/Nylander (1960)

This method is able to predict the ultimate load irrespective of whether the type of failure is flexural failure or punching failure. It gives a continuous transition between these two types of failure. It also predicts the deformations of the slab at failure. Regan and Braestrup (1985) commented that this mechanical approach is a reasonable model for the punching failure of slabs.

2.2.2.2 Limiting Shear Stress on Critical Section Model

In this model, the shear stress is calculated on a pre-defined and fixed critical section. Various authors have assumed this critical section to be as small as the perimeter of the column (Hognestad, 1953) to as large as a perimeter that approaches the column no closer than twice the effective depth $2d$ (CEB-FIP Model Code, 1990). The depth of the critical section is usually assumed to be some fraction of the slab thickness that is effective in resisting shear stress. This was the first model of shear transfer applied to slab-column connections.

Talbot (1913) proposed an equation to calculate the shear stress in concrete, v , at failure, assuming a vertical critical section located at a distance d from the face of the column, giving a perimeter of $4(r+2d)$. The area of the critical section is the perimeter multiplied by the shear depth jd . The shear stress at failure is then simply the failure load V divided by the area of the critical section $4(r+2d)jd$.

Moe (1961) published the most comprehensive study at that time. He examined the results of 43 of his own specimens as well as 160 results reported by other researchers. He suggested that the shear stress of slab-column connections can be expressed as:

$$v_u = \frac{V_u}{bd} = \left[15 \left(1 - 0.075 \frac{r}{d} \right) - 5.25 \phi_o \right] \sqrt{f'_c} \quad (2.2)$$

where,

- V = column load at failure;
- b, d = column perimeter and slab effective depth, respectively;
- r = width of column face;
- f'_c = concrete compressive strength;
- ϕ_o = ratio between the theoretical capacity of the connections based on shear and flexural strength

In 1968, ACI-ASCE Committee 326 altered Moe's equation into a simpler model. The committee assumed that in normal design, the punching and flexural resistance will be designed to resist the same load, and it is reasonable to take the parameter ϕ_o or the ratio between the theoretical capacity of shear and flexural strength, equals to unity. Eq. (2.2) then reduces to:

$$v_u = \left(9.75 - 1.125 \frac{r}{d} \right) \sqrt{f'_c} \quad (2.3)$$

Herzog (1970) reviewed 133 reported tests from 14 investigations and presented empirical relations to predict the punching strength of slabs. This approach is somewhat similar to that taken by today's North American codes. Generally, the maximum stress at a critical section v_u is compared to the shear stress that can be resisted by the connection v_c . The failure is taken at a distance $d/2$ from the face of an equivalent square column of dimension r . The stress on the critical section can be calculated as:

$$v_u = \frac{V_u}{4(r+d)d} \left[1 - \left(\frac{r+2d}{L} \right)^2 \right] \quad (2.4)$$

where L is the span length and the term in the square brackets represents the portion of the load directly resisted by the failure surface. The shear that can be resisted by the concrete takes into account the beneficial effect of the flexural reinforcement ratio ρ , and is expressed as:

$$v_c = \left[2.64 + 0.0047 \rho f_y \right] \sqrt{f'_c} \leq 6.3 \sqrt{f'_c} \quad (\text{psi}) \quad (2.5)$$

Hawkins et. al (1971) published the results of a study that examined the effect of column rectangularity on shear capacity. The aspect ratio was varied from 1 to 4.3. They observed that as the aspect ratio increases, the peak load decreases. Hence, the shear stress resistance can be expressed as:

$$v_u = \frac{V_u}{b_o d \sqrt{f'_c}} = \left(2.5 + 3.0 \frac{s}{l} \right) \leq 4.0 \quad (2.6)$$

where; s/l is the short to long side ratio of the column, b_o is the perimeter of the critical section, taken as a distance $d/2$ from the face of the column. Eq. (2.6) indicates that a reduction in shear capacity applies when the column is rectangular.

2.2.2.3 Two-Phase Method

Yitzhaki (1966) published two independent equations to calculate the strength of slab-column connections. One was based on flexural failure while the other was based on shear failure. He noted that the main variables that influence the punching capacity of slabs are effective depth, amount of flexural reinforcement, concrete strength, and the ratio of the column size to effective depth. He also observed that flexural reinforcement ratio influenced both punching and flexural capacity. Thus, in his proposed equations, he utilized a parameter q as the flexural reinforcement index, defined by $q = \rho f_y / f'_c$. His proposed equations for flexural and punching resistance are given in Eq. (2.7) and Eq. (2.8) respectively.

$$P_{flex} = 8\rho f_y d^2 \left(1 - \frac{q}{2}\right) \quad (2.7)$$

$$P_{pun} = 8 \left(1 - \frac{q}{2}\right) d^2 (149.3 + 0.164\rho f_y) \left(1 + \frac{r}{2d}\right) \quad (2.8)$$

Rankin and Long (1987) proposed a method derived from rational concepts of various modes of failure. These modes can be classified broadly as either flexural or shear, depending on whether failure is initiated by the yielding of the reinforcement (flexure), crushing of the concrete (flexure) or by internal diagonal cracking (shear). The ultimate flexural capacity is related to the yield moment by an analytically based linear interpolative moment factor (Eq. 2.9). The ultimate shear capacity is based on a semi-empirical relationship for vertical shear stress on a critical section close to the column perimeter (Eq. 2.10). The punching strength is the lesser of the two capacities.

The importance of the two principal modes of failure is that for a slab of a given span/depth ratio, the flexural capacity can be increased effectively by increasing the amount of flexural reinforcement until the mode of failure becomes shear at which point the strength of the concrete becomes more important. However, this method was later viewed to be ambiguous with the nature of punching failure, because according to the two-phase method the calculated punching strength can be governed solely by the ultimate flexural capacity, and hence, it ignores the influence of the concrete strength on the punching capacity.

The flexural capacity of conventional slab-column specimens can be predicted from the expression:

$$P_{vf} = \left(k_{yl} - \left(k_{yl} - \frac{k_b}{r_f} \right) \frac{M_b}{M_{(bal)}} \right) M_b \quad (2.9)$$

where,

- P_{vf} = predicted flexural capacity;
- k_{yl} = moment factor for overall tangential yielding;
- k_b = ratio of applied load to internal bending moment;
- r_f = reduction coefficient to allow for column shape;
- M_b = bending moment of resistance;
- $M_{(bal)}$ = balanced moment of resistance.

The shear capacity of conventional slab-column specimens was determined on an empirical basis as:

$$P_{vs} = 1.66 \sqrt{f'_c} (c + d) d (100 \rho)^{0.25} \quad (2.10)$$

where,

- P_{vs} = predicted shear capacity;
- f'_c = cylinder compressive strength of concrete;
- c = length of column side;
- d = average effective depth to tensile reinforcement;
- ρ = reinforcement ratio.

2.2.3 Plastic Models

2.2.3.1 Shear Plasticity Model

The theory of plasticity essentially described an upper bound solution to punching of slabs subjected to a concentrated load. The method assumes that the concrete fails along axi-symmetric failure surface around the point of load. The failure surface is similar to a truncated cone with sides concave in the direction of applied load. During failure, the concrete within the cone translates through the slab while the remainder of the slab remains stationery. Plastic work is done during this

process at all points on the failure surface. This work can be determined utilizing a modified Coulomb failure criterion for the concrete. The total work done along the failure surface must equal to the work done by the external load as it displaces with the punched out cone. This allows calculation of the punching load.

This procedure is greatly complicated by the fact that it is necessary to find the shape of the failure surface that minimizes the internal energy. This approach was first introduced by Braestrup and Nielsen (1976). The plasticity solution for the punching shear strength is based on the failure mechanism shown in Fig. 2.2. Nevertheless, the expression for the punching strength is quite lengthy and not attractive to practicing engineers.

Figure 2.2 Plastic Model by Braestrup and Nielsen (1976)

Jiang and Shen (1986) presented a simpler solution than Braestrup-Nielsen (1976) for the punching strength of a concrete slab, in which a second-degree parabola is used for a Coulomb-Mohr yield envelope as the failure criterion for the concrete. In the Jiang and Shen (1986) solution, punching failure is possible for a smaller angle, but with a higher ultimate strength. At the same time, Jiang and Shen (1986) pointed out that punching shear of slab is not governed by the tensile strength of concrete only, as has been shown by Nielsen (1984). The failure is rather a three-dimensional one, mostly in the region of tension-compression-compression or compression-compression-compression, in which compression plays a more dominant role. The solution is as follows:

$$P = \pi f'_t \left(\frac{d_1^2 - d^2}{4} + \frac{2Kh^2}{\ln d_1 - \ln d} \right) \quad (2.11)$$

where, d = diameter of loaded area;

d_1 = diameter at base of failure cone;

f'_t = effective tensile strength of concrete;

h = thickness of concrete slab;

K = parameter depending upon the ratio between effective compressive strength to effective tensile strength of concrete.

Bortolotti (1990) took into account softening of concrete when converting the plastic solution of punching strength into practical use. The solution is very complex. And there is a limitation when using this method i.e. the slabs are rigid enough to prevent displacement of the border of the slab. If this does not happen and border displacement and rotations are allowed, the theoretical values disagree with experimental values.

2.2.3.2 Shear Friction Model

The method of shear friction has been developed by Loov (1978) to predict the shear capacity of beams by calculating the shear stress capacity, v on an inclined surface, as a function of the normal stress σ across the surface. The relationship between these two stresses is defined as follows:

$$v = k \sqrt{\sigma f'_c} \quad (2.12)$$

where k is a calibration constant.

The shear friction model was extended to concentric punching of slab-column connection by Dechka (2001). The failure plane of this model, in many aspects, is similar to the shear plasticity models discussed previously. The concentric curved failure surfaces of the shear plasticity models are replaced with a multi-planar concentric cone, while the failure criteria are replaced with the shear friction model. However, his model ignored the effects of column shape and slab effective depth, as they would cause more difficulties in finding the best failure plane.

The concentric shear capacity of a slab-column connection, V_{sf} is then expressed, by the simplified procedure as:

$$V_{sf, simple} = \frac{k^2 f'_c h}{4} \left(\frac{h}{x} + \frac{4r_t}{k^2} \right) (l_o + \pi x) \quad (2.13)$$

where: k = calibration coefficient;

h = slab thickness;

x = horizontal projection of the assumed shear crack;

r_t = tensile to compressive strength ratio of the concrete;

l_o = perimeter of the failure surface at the underside of the slab.

2.2.3.3 Model by Menetrey

Menetrey (1996) proposed a mechanical model that is able to compute the punching strength of reinforced concrete slabs. The model requires a finite element program as it involves long computations because the punching strength is calculated by integrating the vertical tensile stresses around the punching shear crack. The vertical tensile stress, σ_v , was assumed to be constant around the shear crack to form a truncated cone around the column as shown in Fig. 2.3.

Figure 2.3 Menetrey's Model (1996)

The results from the numerical simulation allow main parameters influencing the punching strength to be calibrated. These parameters are the tensile strength of concrete, percentage of steel reinforcement, slab thickness, and radius of punching crack initiation. The contributions of dowel effect, shear reinforcement, pre-stressing tendons to the punching strength were also taken into account.

However, it would be extremely difficult to extend the model to cater for the unsymmetrical punching case. The basic format of the punching strength computed by integrating the vertical tensile stresses in the concrete is as follows:

$$F_{pun} = F_c + F_{dow} + F_{sti} + F_{prt} \quad (2.14)$$

where: F_{pun} = punching strength of slab;

F_c = concrete strength;

F_{dow} = resistance dowel force due to horizontal reinforcement;

F_{sti} = vertical strength of shear reinforcement that are well-anchored;

F_{prt} = vertical projection of pre-stressing tendon strength.

2.3 Connections under Unsymmetrical Punching

2.3.1 Introduction

In this section, the most important models proposed to predict the strength of slab-column connections subjected to monotonically applied unbalanced moment are reviewed. It has long been realized that the shear strength of a slab-column connection is affected by the magnitude of the unbalanced moment acting on the connection. In general, the models can be divided into either elastic or plastic in derivation.

Regan and Braestrup (1985) provide a thorough review of many aspects of slab-column connection behavior. Typical crack patterns in the vicinity of the column, when viewed from above, are shown in Fig.2.4. In Fig.2.4, the punching failure is shown by a thickened, irregular line.

Figure 2.4 Typical Crack Patterns of Punching Failure (Regan and Braestrup, 1985)

The extent of the punching failure surface depends upon the amount of unbalanced moment being transferred from the slab to the column. Under concentric loading, the failure surface surrounds the column, as shown in Fig.2.4(a). The resulting shape has been described as a truncated cone or pyramid. With unbalanced loading, the punching surface is restricted to the region of slab near the more heavily loaded face of the column, as shown in Fig.2.4 (b) and (c).

2.3.2 Elastic Models

2.3.2.1 Linear Distribution of Shear Stress on Critical Section Model

Di Stasio and Van Buren (1960) presented a working stress method for the strength of slab-column connections under combined shear and unbalanced moment. The vertical shear stress on the critical sections is assumed to distribute as shown in Fig.2.5 for typical interior and exterior slab-column connections. The major criterion is the limitation of the vertical shear stress on two critical sections. The first, assumed at the column perimeter, is the section for maximum punching shear. The second, located at a distance $h - 1.5$ inches away from the column face, is a critical section for diagonal tension. The vertical shear stress, v , is calculated from:

$$v = \frac{8h}{7d} \left[\frac{V}{A_c} + \frac{(M - m_{AB} - m_{CD})C_1}{J_c} \right] K \quad (2.15)$$

where: V and M are resultant shear and unbalanced moment acting at the centroidal axis, o-o', of the critical section; m_{AB} and m_{CD} are flexural moments on sides AB and CD of the critical section; h is overall slab thickness; d is slab effective depth; A_c is area of concrete at assumed critical section; J_c is property of the assumed critical section analogous to polar moment of inertia; C_1 is distance from centroidal axis to the most remote part of critical section; K is a constant that modifies A_c and J_c to account for the dowel action of the reinforcement.

$$K = \frac{1}{1 + (n - 1)\rho} \quad (2.16)$$

where n = modular ratio, and ρ = reinforcement ratio. The factor $\frac{7}{8}$ was used in Eq. (2.15) to conform to the American design practice in 1960 for the calculation of shear stress by $\frac{V}{bjd}$, assuming j equal to $7/8$.

The allowable stress on the critical section for diagonal tension followed recommendations of the 1956 ACI Building Code. The vertical shear stress on this section was limited to $0.03 f'_c$ when at least 50 per cent of the reinforcement required for bending in the column strip passed through the critical section. When unbalanced moments in two directions at right angles occurred simultaneously, they recommended that the analysis for each direction be made independently and the resulting values added to give the final maximum shear.

Figure 2.5 Assumed Shear Stress Distribution by Di Stasio and Van Buren (1960)

Moe (1961) developed a method for the analysis of the strength of slab-column connections under combined shear and unbalanced moment loading. He assumed a critical section directly adjacent to the column face.

Figure 2.6 Moe's model (1961)

Fig. 2.6 shows a square column stub loaded with a vertical shear, P , and a moment, M , in one of the planes of symmetry parallel to the faces of the column. The forces and moments acting between the slab and the column contributing to the transfer of the external load are indicated in this figure. The external moment, M , is balanced by the following internal moments:

- (a) Torsional moments, M_t , on sides BC and AD, Fig. 2.6 (a).
- (b) The resultant flexural moments of slab sections AB and CD.
- (c) A shearing couple, Ve , in which V is the resultant of the vertical shearing stresses and e is the eccentricity of the resultant V .

The vertical shear stress is assumed to distribute as shown in Fig. 2.6 (b). The maximum vertical shearing stress, v , is calculated from:

$$v = \frac{V}{A_c} + \frac{KMc}{J_c} \quad (2.17)$$

In which, J_c is equal to $\int x^2 dA$ taken over the entire area of the critical section, and c is the distance from the centroidal axis to the most remote part of critical section. K is a moment reduction factor which accounts for the fact that part of the external unbalanced moment is resisted by bending moments and part by torsional moments.

Moe determined K experimentally. He made the following two more assumptions. First, the nominal vertical shearing stresses are uniformly distributed across the effective depth, d , of the slab. Second, the maximum vertical shear stress at failure is equal to the shear stress at failure if the slab were loaded concentrically. He then found that the ultimate shear strength of all his slabs could be predicted with a standard deviation of 0.103 when K was taken equal to $1/3$.

Based on tests of 17 reinforced concrete slab-column specimens, Hanson and Hanson (1968) evaluated four design methods for shear strength, presented by Di Stasio and Van Buren (1960), Moe (1961), ACI-ASCE committee 326, and the commentary on the 1963 ACI building code. They found that the working stress method recommended by Di Stasio and Van Buren (1960), modified to agree with the 1963 ACI Code, was found to have a variable factor of safety always greater than 2. The ultimate strength design method recommended by Moe (1961) was found to be simple in application and to give good results. Hanson and Hanson (1968) recommended a change in K value to 0.4 in the American Code. This recommendation was later adopted in the 1971 ACI Code.

Different researchers have assumed different locations of the critical section and fractions of the unbalanced moment carried by shear. Mast (1970) performed a linear finite element analysis and determined that the fraction of unbalanced moment resisted by shear is taken as 0.3. Hawkins (1974) suggested a value of 0.4 based on a review of available literature. Because of its ease of use, the limiting shear stress on a critical section model has been incorporated into most modern design codes. These codes will be reviewed in next chapter of this thesis.

2.3.2.2 Moehle's Model

Moehle (1988) reviewed experimental data on edge slab-column connections transferring unbalanced moment about an axis parallel to slab free edge. He concluded that unlike interior connections, there appears to be less pronounced interaction between shear and moment at edge connections. Moehle plotted the test data from Stamenkovic and Chapman and a subset of the test series done by Zaghlool in terms of M_u/M_o and V_u/V_o to show the degree of interaction between shear and moment at failure. M_u and V_u are the moment and shear force at

failure, respectively. M_o is the flexural capacity in the absence of applied shear force and V_o is the pure shear capacity in the absence of applied unbalanced moment. It was observed that there was no significant interaction between shear and moment strength. Either the connections fail at its pure shear strength or at its pure flexural strength, whichever is reached first. Thus, a simple bi-linear interaction diagram shown in Fig. 2.7 was proposed.

Figure 2.7 Moehle's Moment-Shear Interaction Diagram (1988)

Moehle also observed that the analysis model incorporated in the ACI Code (eccentric shear stress) is excessively conservative and does not correlate well with the test results. For design purpose, he suggested that the applied shear and moment should be taken as the value at the centroid of the slab critical section rather than at the centroid of the column cross section. He also proposed that if the applied shear force V_u is less than 0.75 the slab shear capacity V_o , no unbalanced moment is considered to be transferred by shear to the connections, and thus, the unbalanced moment is resisted entirely by flexure within a prescribed effective width. It implied that for a relatively low shearing force, there is no interaction between shear and unbalanced moment, and therefore, the moment transfer capacity can be calculated as the capacity of the flexural reinforcement within a specified width. The effective width was suggested to be $c_2 + 2c_1$ as shown in Fig. 2.8. This model was the basis for a new recommendation introduced for the first time in the ACI 318-95 Code.

Figure 2.8 Effective Transfer Width by Moehle (1988)

2.3.3 Plastic Models

2.3.3.1 Beam Analogy Model

The equivalent beam approach was first proposed by Anderson (1966). However, his model only applied to concentric loading. Later researchers have since extended the principle to include unbalanced moments. Much work has been done on this type of model and some very encouraging results have been obtained. Fig. 2.9 shows the basic concept of beam analogies for interior slab-column connections transferring shear and unbalanced moment. The slab-column junction is considered as four crossing beams with a width equals to $c+d$.

Hawkins and Corley (1971) suggested that the strength of a slab-column connection be based on the strength of the four beams framing into the column faces (Fig 2.9a). Each beam is assumed to develop its ultimate shear, bending or torsional moment capacities at its junction with the column. The portion of the applied shear or moment that exceeds the ultimate strength of one of the beams is assumed to be redistributed to adjacent beams. Failure is assumed to occur in an interior slab-column connection when at least three of the four beams shown in Fig 2.9a collapse. Different possible combinations of shear, bending and torsional moments may be developed at failure. These combinations can be grouped into two categories and defined as moment-torsion and shear-torsion failure modes, as shown in Fig 2.9b.

Figure 2.9 Beam Analogy by Hawkins and Corley (1971)

Application of beam analogy for design of interior slab-column connections is relatively complex as there are six possible failure conditions for interior connection. Regan and Braestrup (1985) discussed the difficulties associated with the use of beam analogy models.

Stamenkovic and Chapman (1969) used beam analogy to develop an interaction expression for edge-column slab connections. They assumed the width for each beam equal to the width of the column face. Moment resistance of the connections was provided by the number of faces parallel to the axis of unbalanced moment. For the case where the bending resistance was provided by one column face only, the capacity of that face M_{flex} as given by the ACI 318-63, is:

$$M_{flex} = 0.9 A_{sf} f_y d \left(1 - \frac{0.59 \rho f_y}{f'_c} \right) \quad (2.18)$$

where: A_{sf} is the area of the tensile reinforcement passing through the column face perpendicular to moment plane; ρ is the reinforcement ratio within the column width; d is slab effective depth; f_y and f'_c are the steel yield stress and concrete compressive stress, respectively.

For the case where the bending resistance was provided by two column faces, i.e. interior connection case and edge connection subjected to unbalanced moment about an axis perpendicular to slab free edge, they derived the following expression for the combined capacity of the two faces:

$$M_{flex} = K_c 0.85 f'_c c_2 \left(\frac{h}{2} \right)^2 + 0.1 f'_c \sum_o c_1 (d - d') + K_s 0.85 f'_c c_2 d c_1 \quad (2.19)$$

where: K_c and K_s are constants determined from experiments; $\sum_o$ is the sum of the perimeters of the top and bottom bars passing through the column.

For the beam section in torsion, they used the shear-friction concept to develop an expression for torsional resistance. Assuming the reinforcement yielded, the torsional resistance was expressed as:

$$M_t = 0.5 n k_t \frac{A_{st} f_y}{b d} d^2 \left(c_1 - \frac{d}{3} \right) \quad (2.20)$$

where: n is the number of interfaces at which torsional resistance is developed; k_t is a friction coefficient determined from the experiments.

The interaction diagram for edge slab-column connections transferring an unbalanced moment about an axis parallel to slab free edge was represented as a quarter circle with the center at the origin and a radius equal to unity as shown in Fig 2.10b, and the interaction formula is as follows:

$$\left(\frac{V}{V_o} \right)^2 + \left(\frac{M}{M_{flex} + M_o} \right)^2 = 1 \quad (2.21)$$

For edge connections transferring unbalanced moment about an axis perpendicular to slab free edge, the bending resistance is provided by two faces and the torsional resistance is provided by one face only. The interaction diagram obtained based on test results was represented by a straight line as shown in Fig.2.10a. This interaction is similar to the interaction obtained for interior slab-column connections, and is given by a linear equation as follows:

$$\left(\frac{V}{V_o} \right) + \left(\frac{M}{M_{flex} + M_o} \right) = 1 \quad (2.22)$$

a) Moment parallel to slab free edge b) Moment perpendicular to slab free edge

Figure 2.10 Stamenkovic and Chapman's Moment-Shear Interaction Diagram (1969)

Simplification of the beam analogy method was introduced by Islam and Park (1976). They assumed a full redistribution of internal actions such that the strength of beams in bending, torsion and shear are mobilized at failure. Hence, the strength of the connection is obtained by summing the flexural, torsional, and shear strength of all the beams. This implies sufficient ductility in bending, torsion, and shear at the critical sections to allow a simultaneous development of the ultimate capacities and enables easier application for interior connections. The resulting shear-moment interaction diagram is shown in Fig 2.11b.

a) Internal Actions at Critical Section b) Moment-Shear Interaction Diagram

Figure 2.11 Beam Analogy Model by Islam and Park (1976)

Kanoh and Yoshizaki (1979) suggested a linear equation between shear and unbalanced moment for interior slab-column connections based on beam analogy model, as shown in Fig 2.12. The pure shear capacity V_o is determined by multiplication of shear critical section area by the nominal shear stress given by Eq. (2.23). The pure moment transfer capacity M_o is determined by Eq. (2.24). The interaction model by Kanoh and Yoshizaki (1979) compared well with experimental data of interior slab-column connections without shear reinforcement.

$$v_n = 0.33\sqrt{f'_c} \text{ (MPa)} \quad (2.23)$$

$$M_o = M_{f-BC} + M_{f-AD} + V_{BC}(c + d) + 2M_t \quad (2.24)$$

where: M_f is the flexural capacity of the corresponding face of the critical section; V_{BC} is the shear capacity of face BC of critical section; c is the column dimension; and M_t is the pure torsional moment capacity of one of the critical section side faces (face AB or CD).

Figure 2.12 Kanoh and Yoshizaki's Moment-Shear Interaction Diagram (1979)

Alexander and Simmonds (1992) proposed the following beam analogy approach given in Fig. 2.13. As shown in Fig. 2.13(a), the slab is divided into radial strips and quadrants. In the quadrant, shear transfer is governed by beam action as shown in Fig. 2.13(b). The shear that was carried by the arch at the face of column must be dissipated in a direction perpendicular to the arch at some distance away from the column. The rate at which shear can be dissipated determines the curvature of the arch.

The capacity of a radial strip is limited by two independent quantities, namely the flexural capacity of the radial strip and the ability of the slab to generate bar force gradient by which the strip is loaded. Anything that reduces or limits either of these quantities will affect the capacity of the radial strip.

Since beam action requires a force gradient in the reinforcement, failure may occur if the force gradient exceeds its limitation. The term Bond Model is also used here because bond strength is the most important limitation on force gradient for those connections that failed prior to widespread yielding. Thus, this explains why extensive yielding of the reinforcement in the vicinity of the column lowers the punching strength.

(a) Layout of radial strips.

(b) Shear transfer in flat plate-column connections

Figure 2.13 Beam Analogy Model by Alexander and Simmonds (1992)

Shear capacity of a connection is assumed to be equal to the sum of the shear capacities of its radial strips. The ultimate capacity of the connection is governed by the lesser of the two independent capacities: the yield line mechanism load, P_y , and the shear capacity of the connection, P_v , as shown in Fig. 2.13(c). The most important distinction between the Bond model and most other models is that the Bond Model provides a lower bound estimate for the shear capacity of slabs.

2.3.3.2 Strut-and-Tie Model

The strut-and-tie model presented by Alexander and Simmonds (1987) is capable of predicting the punching strength of a slab-column connection subjected to both vertical shear and unbalanced moment. Strut-and-tie model consist of a concrete compression strut and a steel tension tie. The in-plane steel of the slab makes up the tension tie. This model is consistent with the observed cracking that exists in a slab prior to failure. Three types of struts are described in the model shown in Fig. 2.14 as follows:

1. Gravity Struts. The tension ties are the top steel bars perpendicular to the face of the column. They resist a downward movement of the slab.
2. Uplift Struts. The tension ties are the bottom steel bars perpendicular to the face of the column. They resist an upward movement of the slab.
3. Anchoring Struts. These struts lie in the plane of the slab at the level of the top steel. They are anchored by two orthogonal top bars. These bars resist in-plane translations/rotations of the top surface of the slab.

By combining these struts around the column, various combinations of vertical shear and unbalanced moment can be resisted provided there is adequate strength in the tie. Typical truss model interaction diagrams for interior and exterior slab-column connections are shown in Fig. 2.15. It was assumed that the strength of the compression strut would not be a limiting factor. This is supported by the fact that in almost all test series, the steel was observed to yield prior to failure.

Figure 2.15 Truss Model Interaction Diagram

2.4 Connections under Reversed Cyclic Loading

2.4.1 Introduction

Present design procedures for flat plate connections under seismic loads are based on experimental research of slab-column connections subjected to cyclic lateral loading that simulates earthquake effects. However, due to limited available experimental data on slab-column connections subjected to cyclic loading, most of the design codes have restrictions on flat plate structures located in seismically active regions. No theoretical models for the transfer of forces under cyclic lateral loading have yet been put forward. However, it is reasonable to assume that a correct model under monotonic conditions should also be applicable for seismic conditions provided that the capacity of some of the assumed load paths are reduced as a function of the loading. There exists in the literature design recommendations for slab-column connections under cyclic loading. Some relevant experimental studies were reviewed in this section. These studies have been utilized to formulate empirical expressions for the design procedures of slab-column connections under seismic loading.

2.4.2 Experimental Research

Most of the cyclic load experiments reported in the literature were conducted on interior slab-column connections. The connections were isolated from an idealized structure and subjected to cyclic lateral forces, simulating earthquake forces. Figure 2.16 shows two methods of cyclic lateral force simulations. In the first method, the vertical loads are transferred from the column to the slab and lateral forces are applied at the column ends as shown in Fig 2.16(a), while the slab is supported at its edges. In the second method, the specimen is supported on the columns and cyclic vertical loads are applied at the slab edges as shown in Fig 2.16(b). In the latter loading procedure, the vertical loads on the opposite slab edges may not be equal in order to transfer vertical loads to the column simulating gravity loads. In addition, the vertical gravity loads may be simulated by adding some steel weights on the top slab surface.

Figure 2.16 Cyclic Lateral Force Simulation

Except for Robertson and Durrani (1991 and 1992) and Hwang and Moehle (2000) who performed experiments on multi-panel specimens, all other researchers adopted one of the two loading procedures shown in Fig 2.16 for their experiments. It should be mentioned that the lateral forces at column ends (Fig. 2.16a) or at slab ends (Fig. 2.16b) were applied by means of a displacement control.

Hawkins, Mitchell, and Sheu (1974) conducted four experiments on interior slab-column connections with no drop panels or shear reinforcement. They found that cyclic loading reduces the connection shear capacity by 20 percent when compared with the shear capacity of a similar specimen subjected to monotonic loads. They also found that connection strength is dictated by the amount of slab flexural reinforcement passing through the column.

Symonds (1976) also conducted some experiments on interior slab-column connections without and with stirrups shear reinforcement. He found that slabs with no shear reinforcement have very limited ductility and drift capacity, where the drift capacity ratio is calculated as the summation of horizontal deflections of slab ends divided by the slab height. He also found that slabs with no shear reinforcement do not have adequate residual shear strength after punching failure to resist the applied gravity loads after earthquake occurrence.

Ghali et. al (1976) tested three interior slab-column connections loaded dynamically. They found that the rapid rate of loading increased the shear strength compared to statically loaded specimens. They also found that increasing slab flexural reinforcement has unfavorable effects on ductility and energy absorption capacity of slab-column connections. This can be considered a warning against

using heavily reinforced slabs in structures located in seismic zones unless other solutions to guarantee satisfactory seismic performance are proven.

Pan and Moehle (1989) conducted tests on four interior slab-column connections. Two specimens were subjected to uniaxial cyclic moment transfer and the other two under biaxial cyclic moment transfer. In their paper, Pan and Moehle also reviewed experiments conducted by earlier researchers. They gave particular attention to study the lateral drift capacity and displacement ductility of the tested connections, and they found that the magnitude of the gravity load carried by the slab is a primary variable affecting the lateral drift capacity and ductility. To achieve adequate drift capacity and ductility, the gravity shear ratio V_u/V_c acting on the slab critical section should not exceed 0.4. They also concluded that for a given level of gravity load V_u , biaxial lateral loading reduces strength, stiffness, and available ductility.

Four more experiments were conducted by Pan and Moehle (1992) to investigate the effect of gravity loading on interior slab-column connections transferring uniaxial as well as biaxial cyclic moment transfer. Their earlier conclusions (Pan and Moehle, 1989) were confirmed. In addition, they addressed the importance of providing enough bottom slab reinforcement through the column to prevent progressive collapse of slabs subsequent to punching failure. The ACI eccentric shear stress model is concluded to be conservative for both uniaxial and biaxial cases. The effective beam width model and the equivalent frame model can provide a reasonably close estimate of lateral stiffness for slab-column connections if stiffness reductions due to cracking are taken into account. They also recommend retrofitting with braces or shear wall to make up for the loss of strength and stiffness in the structure.

Robertson and Durrani (1991 and 1992) tested three subassemblies of two edge and one interior slab-column connections under combined vertical loads and uniaxial cyclic moment transfer. Each test specimen represented a two-bay flat plate subassembly bounded by the column contraflexure. The variable of this experimental program was the level of applied gravity loads on the slabs. Robertson and Durrani (1991) concluded that for exterior slab-column connections, the gravity load V_u should not exceed 0.5 the value given by the ACI equation to calculate pure

punching capacity V_c in order to achieve adequate drift capacity. The gravity load V_u should not exceed $0.35 V_c$ in the case for interior connections.

Dilger and Cao (1994) reported a series of 7 tests on isolated interior slab-column connections subjected to reversed cyclic loadings. The main test variables were the percentage of shear reinforcement and the gravity shear applied at the connection. The main conclusion related to the author's study were:

1. Reversed cyclic loading reduces the maximum unbalanced moment by 30% as compared to monotonic loading
2. The story drift ratio that can be achieved without failure is significantly increased as the ratio of the applied gravity shear to the theoretical shear capacity V_u/V_c is decreased.
3. The ductility factor is better determined by defining the yielding of the connection as the point at which all the top steel within a $c+3h$ column strip yields.
4. The shear stress resisted by the concrete should be taken as 0.8 the value given by the ACI Code when seismic loading is considered.
5. For connections with no shear reinforcement, the ratio V_u/V_c must be restricted to 0.4 in order to attain a story drift ratio >1.5 %. No such restriction is required when shear studs are used.

Durrani et. al. (1995) tested another four subassemblies specimens. The variables of the experimental program were the detailing of slab reinforcement, use of straight or bent-up bars, and the level of applied gravity loads. He concluded that if the gravity shear ratio does not exceed 0.375, the ultimate drift ratio can reach about 2 percent. They also found that a higher portion of moment is transferred through flexure for relatively low gravity loads. Furthermore, the shear strength of connections was found to decrease with increasing cyclic lateral drifts.

Megally and Ghali (1994) studied the results reported by 8 other researchers. In total, 31 specimens were considered, including 5 specimens of their own. All of the tests were conducted under cyclic loading. While some of the specimens were provided with shear reinforcement in the form of stirrups or headed studs, others were not. The main conclusions of this study were:

1. In seismic zones, flat plate structures should be designed with an independent lateral load resisting mechanism that limits the story drift ratio to less than 1.5 %.
2. For a slab-column connection to be able to safely attain a story drift ratio of 1.5 %, the ratio of the factored concentric load, V_u/V_c should be restricted to 0.4, unless shear reinforcement is provided.

2.4.3 Lateral Drift Requirements

A key factor that controls the vulnerability of a reinforced concrete flat plate structure to earthquake damage is its flexibility against lateral forces. Thus, the lateral deformations, or drifts must be minimized under earthquake actions. The story drift is defined as the lateral displacement of one level or floor relative to the level above or below. The story drift ratio is then calculated as the story displacement divided by the story height.

The National Building Code of Canada, NBCC85 requires that the maximum story drift ratio to be three times the recommended story drift ratio from the elastic analysis, which is 1.5 percent. This is to account for plastic deformation that occurs in the structural system. On the other hand, The Uniform Building Code, UBC91 requires that the calculated story drift ratio, including inelastic deformation, should not exceed 2.0 and 2.5 percent for structures having a fundamental period of 0.7 second or greater, and for structures having a fundamental period of less than 0.7 second, respectively.

A common limit of story drift ratio is 1.5 percent. This ratio has been recommended by several researchers, as reviewed earlier. Thus, it can be concluded that structural lateral force-resisting elements should be combined with flat plates in order to limit the inelastic story drift ratio to 1.5 percent. Consequently, it is required that slab-column connections withstand at least a 1.5 percent story drift ratio without punching shear failure.

2.4.4 Effects of Gravity Loads on Drift Capacity

Megally and Ghali (2000) summarized the variation of ultimate story drift ratio versus the gravity shear ratio ($V_u/\phi V_c$) in Fig. 2.17 for interior slab-column

connections supported on square column, and subjected to gravity and uniaxial cyclic moment transfer. V_u represents the maximum shear force at failure, while V_c is the punching shear capacity of the slab-column connection without shear reinforcement in absence of moment transfer. The strength reduction factor ϕ is taken equal to 1.0 since the strength of materials are known.

Figure 2.17 indicates that the capability of the slab-column connection to withstand the story drift without failure decreases with the increasing magnitude of the applied gravity loads. For interior connections with no shear reinforcement, V_u should not exceed $0.4\phi V_c$ in order to satisfy the required 1.5 % story drift ratio (ACI-ASCE Committee 421). However, unlike interior connections, data on exterior slab-column connections under cyclic loading are still limited, especially on biaxial cyclic moment transfer. Besides, experiments on slab-column connections involving rectangular columns are also very limited.

Figure 2.17 Effects of Gravity Loads on Lateral Drift Capacity (Megally and Ghali, 2000)

2.4.5 Ductility of Slab-Column Connection

Ductility of a structural element can be defined as the ability of the structural element to dissipate a significant amount of energy through inelastic behavior under large amplitude cyclic deformations without substantial strength reduction. There are several quantitative definitions of ductility such as displacement or curvature ductility. The displacement ductility parameter is more commonly used, since economy of design is achieved by allowing yielding to take place in some locations in the structure. The displacement ductility of slab-column connection is specified as the ratio between the drift ratio of connection at failure and the drift ratio at first yield of slab reinforcement. Pan and Moehle (1989) defined the ultimate and yield displacement by a bilinear idealization of the load-displacement graph, which will be discussed later in chapter 5.

The UBC91 code requires that the displacement ductility factor should range between 3 and 4. Also, Park and Paulay (1975) stated that the typical values of the overall displacement ductility factor of framed structures in seismic zones may range from 3 to 5. However, when shear walls are provided to resist the entire lateral forces and to limit the drift ratio to 1.5 percent as discussed earlier, Pan and Moehle (1989) suggested a minimum displacement ductility factor of 1.2 for slab-column connections.

2.4.6 Seismic Analysis of Flat Plate

Megally and Ghali (2000) provide design recommendations for flat plates subjected to cyclic loading associated with earthquake loading. The paper does not cover the design of these systems as lateral load resisting mechanism. It is intended to ensure that slab-column connections can undergo the story drift associated with the response of the primary lateral load resisting mechanism while continuing to safely carry the gravity load

The paper identified the condition for a slab-column connection to perform satisfactorily under seismic loading, without the provision of some form of shear reinforcement. It was proposed that there exists a relationship between the concentric gravity load on the connection V_u , and the ultimate story drift ratio DR_u . The proposed relationship is given in Eq. (2.25).

$$\frac{V_u}{\phi V_c} \leq 0.625 - 15DR_u \quad (2.25)$$

The term ϕV_c in Eq. (2.25) represents the factored shear capacity of the connections in absence of unbalanced moment. This relation indicates that provided the ratio $V_u / \phi V_c$ is less than 0.4, the connection can safely withstand an imposed story drift ratio of at least 1.5 %. Where the ratio of $V_u / \phi V_c$ is less than 0.25, the connection can safely withstand a story drift ratio of 2.5 %. When Eq. (2.25) cannot be satisfied, some form of shear reinforcement must be provided. Megally and Ghali (2002) also noted that although shear capitals can be used for this purpose, they do not provide a ductile mechanism. Hence, shear capitals are not recommended to be used for flat plate structures in seismic zones. The provision of vertical shear reinforcement, either in the form of stirrups or headed studs, is recommended.

Punching shear strength equations given in all the design codes were based on experimental results of slab-column connections subjected to shear and monotonic unbalanced moment transfer. Thus, it implies that the prevailing equations should be used regardless of the loading procedure, whether it is monotonic or cyclic. As cyclic loading reduces the shear strength, Durrani et al. (1995) suggested a reduction of the nominal punching shear stress with increased story drift ratio according to the following equation:

$$V_c = (0.33 - 0.021DR) \sqrt{f'_c} \quad (2.26)$$

where DR is the story drift ratio of the slab-column connection (in percent terms).

This equation indicates that the nominal punching shear stress is reduced due to cyclic loading by about 10 percent if the story drift ratio is limited to 1.5 percent. However, the equation is only valid within the range of parameter variation used in their test programs. Thus, further verifications and research works are needed to investigate the shear strength degradation due to cyclic loading.

2.5 Summary

Most of the models presented to predict the punching resistance incorporated the influence of flexural resistance of the slab. Despite the fact that it seems counter-intuitive to use flexural criteria to describe a shear type failure, these models have demonstrated that there is indeed a significant interaction between the shear and flexural strength. It was shown from the experimental studies, that prior to punching failure, the top flexural bars passing through the column yielded. This explains why models based on flexural parameters will give a reasonable prediction of the punching load. Hence, it also indicates that the correlation between flexural and shear capacity cannot be ignored, and thus, a plausible explanation is required.

Two general approaches have normally been taken, often in combination, to examine and ultimately to predict the punching strength of slab-column connections. The first method is to do an experimental study and formulate empirical equations. The second is to formulate mathematical models that describe the stress distribution within the connections. It was examined from previous research works that any successful model or equations that able to predict the punching strength accurately should account for the influence of the main variables affecting the punching resistance. Some of these are:

- The concrete strength
- The flexural reinforcement ratio
- The shape of the column
- The ratio of the column size to the effective depth of the slab
- The nature of loading, i.e. reversed cyclic or pseudo static
- The effect of in-plane membrane forces
- Scale effects
- The beneficial effects of any shear reinforcement

No equations, that the author aware of, have yet been developed, that account for all these factors. Besides completeness and accuracy, one also needs to account for the simplicity and the practicality of the model proposed.

CHAPTER 3

REVIEW ON PUNCHING SHEAR DESIGN CODES

CHAPTER 3

REVIEW ON PUNCHING SHEAR DESIGN CODES

3.1 Overview

The punching shear strength of slab-column connections for flat plate structures is defined in terms of nominal shear stress calculated with reference to a control perimeter around the column. Due to its simplicity, the control surface approach has been adopted by most of the building codes for punching shear design of slabs. Although different building codes share the same design concept of control surface approach, they differ in the definition of the nominal shear stress, the control parameter and the concrete strength.

The methods in these codes are largely empirical equations that have been chosen mainly for their simplicity in use and a wide range of conditions over which they produce acceptable results. Although most of the code methods had been sufficiently checked for reliability by numerous experimental data, the experimental data dealing with the strength of rectangular-column slab connections subjected to gravity and biaxial cyclic moment transfer are not available.

This chapter outlines the current design methods to calculate the punching shear capacity of slab-column connections, such as the American Code-ACI 318-05; Australian Code 3600-1988; British Code-BS8110: 1997; Canadian Code-CSA A23.3-04; CEB-FIP Model Code 1990; Eurocode EC 2; and Japanese Code-JSCE 1986. As shown in Figure 3.1, the control surface perimeter is either taken to conform with but at a certain specified distance from the loaded area, or to join all the points with a certain specified distance from the loaded area. The control surface method was first introduced by Talbot (1913) who tested square footings with concentrically loaded square columns.

3.2 The Absence of Unbalanced Moment

In many design codes, the punching shear capacity in the absence of unbalanced moment can be calculated from the shear strength given in the codes:

$$V_c = v_c \cdot u \cdot d \tag{3.1}$$

where, v_c is the nominal shear stress, u is the peripheral length of critical section, which varies for different codes (Figure 3.1), and d is the effective depth of the slab flexural reinforcement.

Since the geometry of control surface differs among different building codes, the corresponding values of nominal shear stress differ. The formula for the nominal shear stress may be modified to account for the effect of several parameters on the calculated strength such as concrete compressive or tensile strength, column shape and dimensions, slab depth and the amount and yield strength of slab flexural reinforcement. When shear reinforcement is provided, the nominal shear stress will increase and another control surface outside the shear-reinforced zone must be checked.

Figure 3.1 The Peripheral Length of Critical Section for Different Codes

3.2.1 American Concrete Institute ACI 318-05

According to the ACI 318-05, the punching shear capacity of slabs without shear reinforcement is determined as follows. The design is based on:

$$\phi V_n \geq V_u \quad (3.2)$$

where: ϕ is the strength reduction factor;

V_u is the factored shear force;

V_n is the nominal shear strength computed by:

$$V_n = V_c + V_s \quad (3.3)$$

where: V_s is the nominal shear strength provided by shear reinforcement;

V_c is the nominal shear strength provided by concrete, calculated as follows:

$$V_c = v_c \cdot b_o \cdot d \quad (3.4)$$

where, v_c is the nominal shear stress, b_o is the peripheral length of critical section located at a distance $d/2$ from the loaded area, and d is the effective depth of the slab flexural reinforcement.

For the research presented in thesis the term v_c , the nominal shear stress, is the more convenient term because of the presence of unbalanced moments, and for comparison with other codes. Since the research is in essence geared to improve the ACI code it is recommended that the ACI term nominal shear stress v_c is used throughout the thesis. According to the ACI 318-05, the lowest v_c will be the slab shear capacity:

$$\text{a) } v_c = \left(1 + \frac{2}{\beta}\right) \frac{\sqrt{f'_c}}{6} \quad (\text{MPa}) \quad (3.5)$$

$$\text{b) } v_c = \left(2 + \frac{\alpha_s d}{b_o}\right) \frac{\sqrt{f'_c}}{12} \quad (\text{MPa}) \quad (3.6)$$

$$\text{c) } v_c = \frac{1}{3} \sqrt{f'_c} \quad (\text{MPa}) \quad (3.7)$$

where: β = ratio of the long side to the short side of the column;

f'_c = characteristic cylinder strength of concrete;

α_s = 40 for interior column;

= 30 for edge column;

= 20 for corner column.

Figure 3.2 Location of Critical Shear Perimeters (ACI 318-05)

3.2.2 Australian Code 3600-1988

According to Australian Standard, the shear capacity of slabs without shear reinforcement is determined from:

$$v_c = (f_{cv} + 0.3 \sigma_{cp}) \quad (3.8)$$

where: $f_{cv} = 0.17 \left(1 + \frac{2}{\beta_h} \right) \sqrt{f'_c} \leq 0.34 \sqrt{f'_c} \text{ (MPa)}$

f'_c = characteristic cylinder strength of concrete in MPa;

β_h = ratio of l/b , note that $l \geq b$;

σ_{cp} = average intensity of effective prestress in concrete.

3.2.3 British Standard BS 8110-97

In the BS 8110-97, the design shear capacity of slabs without shear reinforcement is based on the shear capacity of beam which is given by

$$v_c = 0.79 \left(\frac{100 A_s}{bd} \right)^{1/3} \left(\frac{400}{d} \right)^{1/4} \left(\frac{f_{cu}}{25} \right)^{1/3} / \gamma_m \text{ (N/mm}^2\text{)} \quad (3.9)$$

where: f_{cu} = characteristic concrete cube strength (MPa) $\leq 40 \text{ N/mm}^2$;
 ρ = percentage of flexural reinforcement, $100 A_s/bd$ along the critical section, $\leq 3\%$;
 d = average slab effective depth, $\leq 400 \text{ mm}$;
 γ_m = material safety factor for concrete in shear.

Figure 3.3 Location of Shear Perimeter in BS8110: 97

The location of shear perimeter as specified in Fig. 3.16 of the BS 8110:97 are presented in Fig. 3.3. According to BS 8110, the ratio of flexural reinforcement is calculated for a width of slab equal to that of the loaded area plus $1.5d$ to either side of it. The range of slab depth over which a size effect that affects the punching shear capacity is between 100 to 400 mm. BS 8110 recommends an upper limit to the shear resistance expressed in terms of the shear stress at the periphery of the loaded area, which has to be lesser of $0.8 \sqrt{f_{cu}}$ or 5 N/mm^2 .

3.2.3 Canadian Standard Association CSA A23.3-04

The CSA A23.3-04 recommends the following expressions for the calculation of shear capacity without shear reinforcement. It is noted that the equations are rather similar with those given in the ACI Code. The nominal shear strength is given by the smallest of the following:

$$\text{a) } v_c = \left(1 + \frac{2}{\beta_c}\right) 0.19 \lambda \phi_c \sqrt{f'_c} \quad (\text{MPa}) \quad (3.10)$$

$$\text{b) } v_c = \left(0.19 + \frac{\alpha_s d}{u}\right) \lambda \phi_c \sqrt{f'_c} \quad (\text{MPa}) \quad (3.11)$$

Chapter 3: Review on Punching Shear Design Codes

$$c) \quad v_c = 0.38 \lambda \phi_c \sqrt{f'_c} \quad (\text{MPa}) \quad (3.12)$$

- where:
- d = slab average effective depth;
 - u = shear perimeter, calculated as $\pi(c+d)$ for interior circular columns and $b+4d$ for interior rectangular columns;
 - b = side dimension of rectangular column;
 - c = diameter of circular column;
 - f'_c = concrete cylinder strength, not greater than 64 MPa;
 - β_c = ratio of longer to shorter dimension of loaded area;
 - α_s = 4 for interior column;
= 3 for edge column;
= 2 for corner columns;
 - λ = 1.00 for normal density;
= 0.75 for low density;
 - ϕ_c = strength reduction factor for concrete
= 0.65 for CSA A23.3-04
= 0.60 for CSA A23.3-94

If the effective depth used in the two way shear calculation exceeds 300 mm, then v_c obtained from Eqs. (3.10) to (3.12) shall be multiplied by $1300/(1300+d)$.

3.2.4 CEB/FIP Model Code 90

In CEB/FIP model code, the shear capacity of a reinforced concrete slab is expressed as a shear stress on a control perimeter, and may be taken as:

$$\tau_{Rd} = 0.18 \xi (100 \rho f_{ck})^{1/3} \quad (\text{MPa}) \quad (3.13)$$

- where:
- ξ = $1 + (200/d)^{1/2}$, is the size effect coefficient;
 - ρ = $(\rho_x \rho_y)^{1/2}$, is the flexural reinforcement ratio;
 - f_{ck} = characteristic concrete cylinder strength, ≤ 80 MPa.

The characteristic punching load for slabs without shear reinforcement is obtained from the following:

Chapter 3: Review on Punching Shear Design Codes

$$F_{Rk} = \tau_{Rd} u_l d \leq 0.5 f_{ck2} u_o d \quad (\text{N}) \quad (3.14)$$

where u_l = control perimeter at a distance $2d$ from the column;
 f_{ck2} = $0.6 (1 - f_{ck}/250) f_{ck}$, is the characteristic compressive strength of cracked concrete in MPa;
 u_o = length of the periphery of the column.

3.2.5 Eurocode EC 2-2003

In EC2, the punching shear capacity of slabs without shear reinforcement is given as:

$$v_r = (0.18/\gamma_c) \xi (\rho f_{ck})^{1/3} + 0.1 \sigma_{cp} \geq (v_{min} + 0.1 \sigma_{cp}) \quad (\text{MPa}) \quad (3.15)$$

where: $\xi = 1 + (200/d)^{1/2} < 2.0$, is the size effect coefficient;
 $\rho = (\rho_x \rho_y)^{1/2} < 0.02$, is the flexural reinforcement ratio;
 f_{ck} = characteristic concrete cylinder strength;
 $\gamma_c = 1.5$, is the concrete partial safety factor;
 $\sigma_{cp} = (\sigma_{cx} + \sigma_{cy}/2)$, is the average precompression;
 σ_{cx}, σ_{cy} = normal concrete stresses at critical section.

EC2 has recommended a method to determine the shear capacity of flat plate with rectangular columns, which is shown in Fig. 3.4. The equations $a_1 \leq 5.6d - b_1$ and $b_1 \leq 2.8d$ limit the shear strength resistance to a lower value at the longest and shortest sides of the rectangular columns.

Figure 3.4 Consideration of Rectangular Columns in Eurocode EC2

3.2.6 Japanese Society of Civil Engineers JSCE 1986

The design shear capacity of slabs in JSCE is based on the following equation to determine shear capacity of beams:

$$v_c = 0.6 \beta_d \beta_p \beta_r (f'_c/10)^{1/2} \quad (\text{MPa}) \quad (3.16)$$

where:

$$\beta_d = (1000/d)^{1/4} \leq 1.5, \text{ is the size effect;}$$

$$\beta_p = \rho^{1/3} \leq 1.5, \text{ is the reinforcement ratio;}$$

$$\beta_r = 1 + 1/(1+0.25 u/d), \text{ is the effect of loaded area.}$$

3.3 The Presence of Unbalanced Moment

3.3.1 American Concrete Institute ACI 318-05

The ACI 318-05 presents a method known as the eccentric shear stress model to calculate the ultimate shear stresses when both shear force and unbalanced moment are transferred. It assumes that a fraction of unbalanced moment can be transferred to the slab critical section as eccentric shear stresses that can be added to the shear stresses on the same section resulting from the direct shear force. The shear stresses due to a fraction of unbalanced moment are distributed linearly on the critical section, as shown in Fig. 3.5. Punching shear failure occurs when the maximum sum of the shear stresses reaches the shear strength determined by the lowest of Eq. (3.5) to Eq. (3.7).

Slab flexural steel is required within a width of $c_2 + 1.5h$ on each side of the column to resist the portion of unbalanced moment not resisted by slab shear stresses, i.e. transferred by flexure. When the portion of unbalanced moment transferred by flexure, $\gamma_f M_u$ reached the limiting flexural strength M_f of slab reinforcement within the prescribed transfer width, flexural failure is deemed to occur at the connection.

Figure 3.5 shows the assumed critical section and the distribution of shear stresses for an interior connection subjected to uniaxial moment transfer. The maximum shear stress occurred in the slab at the face AB is given by:

Chapter 3: Review on Punching Shear Design Codes

$$v_{AB} = \frac{V_u}{b_o d} + \frac{\gamma_v M_u c_{AB}}{J_c} \quad (3.17)$$

where,

V_u , M_u = applied shear force and unbalanced moment, respectively;

c_{AB} = distance from face AB of the critical section to centroidal axis zz ;

J_c = critical section property analogous to the polar moment of inertia;

b_o = critical shear perimeter;

d = slab effective depth;

γ_v = fraction of unbalanced moment transferred by shear, calculated as:

$$\gamma_f = \frac{1}{1 + (2/3)\sqrt{b_1/b_2}} \quad (3.18)$$

γ_f = fraction of unbalanced moment transferred by flexure, given by:

$$\gamma_v = 1 - \gamma_f \quad (3.19)$$

b_1 , b_2 = widths of critical shear perimeter in the shape of a closed rectangle, with b_1 and b_2 measured parallel and perpendicular to the directions of the span for which moment is determined.

Chapter 3: Review on Punching Shear Design Codes

(a) Transfer of unbalanced moments to column.

(b) Shear stresses due to V_u .

(c) Shear due to unbalanced moments.

(d) Total shear stresses.

Figure 3.5 Shear Stresses due to Shear and Moment Transfer at an Interior Column (ACI 318-05)

Chapter 3: Review on Punching Shear Design Codes

When biaxial unbalanced moments are transferred, the maximum shear stress occurs at one point on critical section where the shear stresses due to the biaxial moments are additive. It can be expressed by:

$$v_u = \frac{V_u}{b_o d} + \frac{\gamma_{v1} M_{u1} c_{AB}}{J_{c1}} + \frac{\gamma_{v2} M_{u2} c_{BD}}{J_{c2}} \quad (3.20)$$

where: c_{AB} and c_{BD} are distances from the centroidal axis of critical section to face AB and BD of the critical section, respectively, and the subscripts 1 and 2 represent the two perpendicular directions about which the unbalanced moments are transferred.

Figure 3.6 shows the distribution of shear stresses in the assumed critical section for an edge connection when moment is transferred about an axis parallel to the edge of the slab. The unbalanced moment acting at the centroid of slab critical section, M_s is calculated as $(M_u - V_u g)$, in which g is the distance between the centroidal axis of the critical section, zz , and the centroidal axis of the column. The maximum shear stress in the slab occurred at either face AB or free edge CD (in an opposite direction), and is given by:

$$v_{AB} = \frac{V_u}{b_o d} + \frac{\gamma_v (M_u - V_u g) c_{AB}}{J_c} \quad (3.21)$$

$$v_{CD} = \frac{V_u}{b_o d} - \frac{\gamma_v (M_u - V_u g) c_{CD}}{J_c} \quad (3.22)$$

For an unbalanced moment transferred about an axis perpendicular to the edge of the slab, the ultimate shear stresses are calculated as in the case of interior connections.

Figure 3.6 Critical Sections and Shear Stress Distribution for Edge Connections (ACI 318-05)

Chapter 3: Review on Punching Shear Design Codes

Figure 3.7 shows the distribution of shear stresses in the assumed critical section when biaxial bending moments are transferred in a corner connection. The dimensions g_1 and g_2 are the distances between the centroidal axes of the critical section, z_1z_1 and z_2z_2 , and the centroidal axes of the column. The maximum shear stress in the slab occurred at either point B or point C (in an opposite direction), and is given by:

$$v_B = \frac{V_u}{b_o d} + \frac{\gamma_{v1}(M_{u1} - V_u g_1)c_{AB}}{J_{c1}} + \frac{\gamma_{v2}(M_{u2} - V_u g_2)c_{BD}}{J_{c2}} \quad (3.23)$$

$$v_C = \frac{V_u}{b_o d} - \frac{\gamma_{v1}(M_{u1} - V_u g_1)c_{AC}}{J_{c1}} - \frac{\gamma_{v2}(M_{u2} - V_u g_2)c_{CD}}{J_{c2}} \quad (3.24)$$

where: c_{AB} , c_{AC} , c_{BD} and c_{CD} are distances from the centroidal axis of critical section to face AB , AC , BD and CD of the critical section, respectively.

Figure 3.7 Critical Sections and Shear Stress Distribution for Corner Connections (ACI 318-05)

For unbalanced moment transfer at interior slab-column connections, and at edge connections about an axis perpendicular to the slab free edge, ACI Code permits the fraction of unbalanced moment transferred by shear to be reduced to 0.25 provided that the applied shearing force V_u does not exceed $0.4 V_c$. For unbalanced moment transfer about an axis parallel to the slab free edge at exterior supports, the value is allowed to be reduced to zero provided that V_u applied does not exceed $0.75 V_c$ and $0.5 V_c$ at edge and corner connections, respectively. This means that the unbalanced moment can be considered to produce no shear stresses. This recommendation was made based on the findings by Moehle (1988) on the interaction between shear and unbalanced moment transfer.

3.3.2 Australian Code 3600-1988

According to Australian Standard, the ultimate shear capacity of slabs without shear reinforcement is determined from:

$$V_u = \frac{V_{uo}}{1 + \frac{u.M_v}{8N_z.a.d}} \quad (\text{N}) \quad (3.25)$$

where: $V_{uo} = u.d (f_{cv} + 0.3 \sigma_{cp}) \quad (3.26)$

$$f_{cv} = 0.17 \left(1 + \frac{2}{\beta_h} \right) \sqrt{f'_c} \leq 0.34 \sqrt{f'_c} \quad (3.27)$$

which: f'_c = characteristic cylinder strength of concrete in MPa;
 β_h = ratio of l/b , note that $l \geq b$;
 σ_{cp} = average intensity of effective prestress in concrete;
 M_v = unbalanced slab moment at the connection;
 N_z = net axial force acting at the connection;
 a = dimension of the critical shear perimeter measured parallel to the direction of M_v .

3.3.3 British Standard BS 8110-97

The plasticity approach is adopted in the computation of effective shear stresses in BS 8110. The code could predict the effective punching shear stress at interior, edge and corner slab-column connections. However, it is noted that the shear and unbalanced moment should be applied independently for both x and y axes in the following equations, and only the maximum shear at one particular axes should be used in the shear design.

BS 8110 provides two methods to calculate the effect of combined shear and unbalanced moments of interior columns: a variation of the eccentric shear expression or simple shear force multipliers. The factored shear force at an interior slab-column connection can be calculated by:

$$V_{eff} = V_t \left(1 + \frac{1.5M_t}{V_t.x} \right) \quad (\text{N}) \quad (3.28)$$

Chapter 3: Review on Punching Shear Design Codes

For edge slab-column connections with bending about an axes perpendicular to the free edge:

$$V_{eff} = V_t \left(1.25 + \frac{1.5M_t}{V_t \cdot x} \right) \quad (N) \quad (3.29)$$

Alternatively, the nominal shear force can be increased by 15% to accommodate unbalanced moments at an interior column. For corner and edge slab-column connections with bending about an axes parallel to the free edge, a simple expression independent of the eccentricity of the load is provided:

$$V_{eff} = 1.25 V_t \quad (N) \quad (3.30)$$

where: V_{eff} = effective design shear including unbalanced moment transfer;
 V_t = design shear transfer to the column;
 x = length of the side of the perimeter, parallel to the axes of bending;
 M_t = design moment at the connection.

3.3.4 Canadian Standard Association CSA A23.3-04

The CSA Code implements a similar approach as in the ACI 318-05, taking into account for the shear stresses resulting from the factored shear force and also from the unbalanced moments in x and y direction, which vary linearly about the centroid of the critical shear area.

Similar to the ACI 318-05, the fraction of the unbalanced moment transferred by shear is given by:

$$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3} \sqrt{\frac{b_1}{b_2}}} \quad (3.31)$$

where b_1 = the length of the side of the critical perimeter parallel to the direction of moment transfer;
 b_2 = the length of the side of the critical perimeter perpendicular to the direction of moment transfer.

The factored shear stress shall be computed using the following equation:

$$v_f = \frac{V_f}{b_o \cdot d} + \left[\frac{\gamma_v \cdot M_f \cdot e}{J} \right]_{x_1} + \left[\frac{\gamma_v \cdot M_f \cdot e}{J} \right]_{x_2} \quad (3.32)$$

Chapter 3: Review on Punching Shear Design Codes

where:

- V_f = factored shear force at the centroid of critical section;
- M_f = unbalanced moment about the centroid of critical section;
- J = polar moment of inertia for the critical section;
- e = distance from the centroid of critical section to the point where the shear stress is being calculated;
- b_o = perimeter of the critical section.

CSA A23.3-94 required that corner column-slab connections be checked for one-way shear on the minimum length straight line located at $d/2$ across the corner. However, CSA A23.3-04 requires that one-way shear be checked along a critical section $d/2$ from the edge of the column or column capital according to:

$$v_c = \beta \lambda \phi_c \sqrt{f'_c} \quad (\text{MPa}) \quad (3.33)$$

$\beta = 0.21$ if the slab thickness is less than 350 mm. If the slab thickness exceeds 350 mm, $\beta = 230/(1000+d_v)$ where d_v is the greater of $0.9d$ and $0.72h$.

CSA A23.3-04 requires integrity steel on all faces of the connection such that:

$$\sum A_{sb} \geq 2 V_{se} / f_y \quad (3.34)$$

where $\sum A_{sb}$ is the total of the anchored bottom steel passing through the column and V_{se} is the shear transmitted to the column due to specified unfactored loads but not less than the shear corresponding to twice the self weight of the slab.

3.3.5 CEB/FIP Model Code 90

CEB-FIP Code implements the “section modulus” approach in the analysis of shear stresses distribution which is due to an eccentric load at the connection, as follows:

$$\tau_{sd} = \frac{F_{sd}}{u_1 d} + \frac{K M_{sd}}{W_1 d} \quad (\text{MPa}) \quad (3.35)$$

where: F_{sd} = concentrated force;
 M_{sd} = unbalanced moment due to an eccentric load;

- $W_1 = \int_0^{u_1} e \, dl$, is the parameter of the control perimeter;
- $= c_1^2/2 + c_1 c_2 + 4c_2 d + 16 d^2 + 2\pi d c_1$ (for rectangular column);
- K = a fraction of unbalanced moment transmitted by uneven shear, bending and torsion that depends on the column aspect ratio c_1/c_2 .

Table 3.1 Values of K in CEB-FIP Model Code

c_1/c_2	0.5	1.0	2.0	3.0
K	0.45	0.6	0.7	0.8

- c_1 = column dimensions parallel to the eccentricity of the load;
- c_2 = column dimensions perpendicular to the eccentricity of the load;
- dl = elementary length of the perimeter;
- e = distance of dl from the axes about which the moment M_{sd} acts;
- u_1 = length of control perimeter.

For edge connection, the shear distribution around the perimeter should be determined as according to Figure 3.8a to calculate τ_{sd} . However, provided the eccentricity of loading is toward the interior of the slab and there is no eccentricity parallel to the edge, τ_{sd} may be calculated on the assumption of uniform shear using the perimeter u_1^* , as shown in Figure 3.8b, using only the first term of the equation. On the other hand, when a moment M_{sd} is acting about an axis parallel to the free edge, an additional shear stress equal to $K M_{sd} / W_1 d$ (2nd term of the equation) will be produced.

For corner connection, the shear distribution around the perimeter should be determined as according to Figure 3.9a to calculate τ_{sd} , using the general equation above. However, τ_{sd} may be calculated on the assumption of uniform shear using the perimeter u_1^* , as shown in Figure 3.9b, provided the eccentricity of loading is toward the interior of the slab.

Figure 3.8 Control Perimeter at Edge Columns in CEB-FIP Model Code

Figure 3.9 Control Perimeter at Corner Columns in CEB-FIP Model Code

3.3.6 Eurocode EC 2-2003

In EC2, a simplified prediction is implemented for punching shear strength with moment transfer in slab-column connections. According to EC2, a factor β which takes into account the effect of eccentricity of loading needs to be added into the punching shear stress calculation, as follows:

$$v_f = \beta \frac{V_f}{u d} \quad (\text{N/mm}^2) \quad (3.36)$$

where: $\beta = 1 + K \frac{M_{fu} u}{V_f W}$;

$u = \pi (c + 4d)$ (shear perimeter of interior circular column);

$u = 2a + 2b + 4\pi d$ (shear perimeter of interior rectangular column);

Chapter 3: Review on Punching Shear Design Codes

$W = \int_0^{u1} e \, dl$, is the parameter of the control perimeter;

K = a fraction of unbalanced moment transmitted by uneven shear, bending and torsion that depends on the column aspect ratio c_1/c_2 (similar values with those in Table 3.1 by CEB-FIP Model Code).

For edge and corner connections, provided the eccentricity of the loading is towards the interior of the slab, the load factored shear stress can be calculated as being uniformly distributed, that is $\beta = 1$. For structures where lateral stability does not depend upon frame action between the slab and the columns, Eq. (3.36) is used with $\beta = 1.15$ for interior column connections, 1.4 for edge column connections, and 1.5 for corner column connections.

3.3.7 Japanese Society of Civil Engineers JSCE 1986

JSCE proposed a strength reduction factor, $1/\alpha$ to reduce the punching shear capacity under applied eccentric load on the slab-column connection according to the equations below:

For circular loaded area,
$$\alpha = \left(1 + \frac{2|e|}{d + d_o} \right) \quad (3.37)$$

For rectangular loaded area,
$$\alpha = \left(1 + 1.5 \frac{|e_x| + |e_y|}{\sqrt{b_x \cdot b_y}} \right) \quad (3.38)$$

3.4 Summary

From the above review on various available punching shear design codes, the following conclusions can be drawn:

- The nominal punching shear strength is mainly dependent on the concrete strength, and it can be predicted with a function of some other parameters, such as reinforcement ratio, column dimension, and slab thickness etc.
- In the absence of unbalanced moment, the ultimate shear stress v_u is assumed to be uniform on the critical shear perimeter. For a safe design, the ultimate shear stress v_u should be less than the nominal shear strength v_c .
- In the presence of unbalanced moment, there are two basic groups of code methods. One group assumes that the shear stresses resulting from the moment transfer can be added to the shear stresses due to direct shear force. The other group suggests that the design shear force should be increased to an effective value in order to include the effect of moment transfer.
- Only the eccentric shear expression (ACI 318 and CSA A23.3) can accommodate moment transfer for an edge connection with the moment vector parallel to the slab edge. The other codes ignore punching shear with the moment vector parallel to the slab edge.
- In the case of biaxial moment transfer, there are two basic methods considered by the codes. One method assumes that the resulted shear stresses are additive, other suggests that the shear stress is to be checked separately and the worst case controls the design.
- In the case of cyclic unbalanced moment transfer, all code methods are silent except the ACI Code that limit the applied gravity shear at the slab-column connections to be 0.4 of the shear capacity in absence of unbalanced moment. This limit was recommended in order to achieve adequate ductility and a lateral drift capacity of 1.5 percent as discussed in the previous chapter.

CHAPTER 4

EXPERIMENTAL PROGRAM

CHAPTER 4

EXPERIMENTAL PROGRAM

4.1 Introduction

Over the years, many flat plate structures have collapsed by punching shear in past earthquakes (AISI, 1964; Mitchell, Tinawi, and Redwood, 1990; AISI, 1991; Mitchell et al., 1995; Saatcioglu, 2001). In the 1985 Mexico City earthquake, 91 flat plate buildings collapsed and 44 were severely damaged due to punching failure. During an earthquake or a strong wind, the resulting lateral loads may cause significant shear and moment transfers at the slab-column connections, especially at exterior connections where unbalanced moment already exist due to unsymmetrical slab geometry. To assess the behavior of flat plate under realistic lateral loads, such as those imposed by wind and seismic forces, it is necessary to consider lateral loading from multiple directions. Unfortunately, no test data is available on edge connections subjected to biaxial cyclic lateral loadings.

Previous experimental studies on punching shear behavior of slab-column connections subjected to cyclic lateral loading were mostly done on interior connections. For interior slab-column connections subjected to uniaxial cyclic lateral loading, tests has been performed by Hawkins et. al (1974); Symmonds et. al (1976); Islam and Park (1976); Morrison and Sozen (1983); Zee and Moehle (1984); Dilger and Cao (1991); Wey and Durrani (1992); Mohamed et. al (1997); Robertson et. al (2002); Brown and Dilger (2003). For interior connections under biaxial cyclic lateral loadings, only two sets of experiments were found from the literature, which was tested by Tan and Teng (2005), and Pan and Moehle (1992).

Meanwhile, past research on exterior slab-column connections subjected to cyclic lateral loading are very limited. They had mostly focused on edge connections transferring uniaxial cyclic lateral loading in the perpendicular direction to the slab free edge. Such experiments were done by Robertson and Durrani (1992); Durrani et. al (1995); Megally and Ghali (2000); Ritchie and Ghali

(2004). However, there are no available test data on edge connections subjected biaxial cyclic lateral loadings. The experimental work reported in this chapter can fill this void.

Conclusions from the experimental researches on rectangular-column slab connections subjected gravity loads showed that column rectangularity significantly influenced the punching strength of the connections. However, research on laterally loaded slab-column connections simulating earthquake effects had mostly dealt with the use of square columns only. Hence, understanding on the behavior of rectangular-column slab connections subjected to cyclic lateral loadings has not been established. To address this issue, the use of rectangular columns is incorporated in this experimental program.

This chapter describes an experimental program designed to study the punching behavior of rectangular edge-column slab connections in flat plate structures, taking into account the effects of column rectangularity, biaxial lateral loading, and different levels of gravity loading.

This experimental program included the testing of five large-scale rectangular edge-column slab connections subjected to combination of gravity and lateral loadings. The tested specimens geometry represented a portion of a slab-column connection terminated at the mid-panel and at the mid-height of the column. The test parameters were the directions of cyclic lateral loading applications (strong versus weak axis of the rectangular column), uniaxial versus biaxial cyclic lateral loadings, and three levels of gravity loadings. Two specimens were uniaxially laterally loaded, two more specimens were biaxially laterally loaded, and the fifth specimen was tested under gravity load only. All specimens were identical and cast with normal density concrete of approximately 40 MPa cube compressive strength.

4.2 Definition of Test Specimens

All test specimens were identical and modeled after a typical exterior connection of a flat plate building. Due to limitation of the available crane capacity, the slab-column specimens were constructed at a scale of ninety percent of the prototype structure. The test specimens represented edge-column slab connections terminated at the mid-panel and at the mid-height of the columns, where moment

due to lateral load was practically zero. This resulted in specimens with 135 mm slab thickness (d_x , d_y and d_{ave} are 100.5 mm, 113.5 mm and 107 mm, respectively), 4 m x 2.9 m slab dimensions, 2.7 m column height, and 0.9 m x 0.18 m column size.

Figure 4.1 Geometry of Test Specimens

The geometry of the specimen is illustrated in Figure 4.1. A total of five rectangular edge-column slab connection specimens were tested by first simulating the desired level of gravity loading and then applying the target cyclic lateral displacements until failure. The test variables in this experimental program are summarized in Table 4.1 and Fig. 4.2. The specimens were identified with first letter, E, representing Edge connections; numeral 1 indicates specimen laterally loaded along North-South (stronger column direction); or 2 indicates specimen laterally loaded along East-West (weaker column direction); or 12 indicates a biaxially laterally loaded specimen. The last letter, H, represents High gravity loading (gravity load V_u equal to $0.31 V_c$) or L represents Low gravity loading (gravity load V_u equal to $0.16 V_c$). V_c is the total shear capacity without unbalanced moment, calculated according to ACI 318-05 (Eq. 3.5 to 3.7). Specimen E0U was tested under gravity load only, without any imposed cyclic lateral loading.

Figure 4.2 Test Variables in terms of Gravity and Lateral Load Applications

Table 4.1 Test Variables of Experimental Program

Specimen	E1H	E2H	E12H	E12L	E0U
Gravity Load, V_u/V_c	High, 0.31	High, 0.31	High, 0.31	Low, 0.16	Ultimate
Lateral Load	Uniaxial <i>N-S</i>	Uniaxial <i>E-W</i>	Biaxial	Biaxial	-

4.3 Preliminary Analysis

Prior to the fabrication of the test setup and the test specimens, a preliminary analysis was carried out in order to simulate the actual behavior of the slab under a combined gravity and cyclic lateral loadings. The simulations were done using the finite element design software, ETABS. Two models were generated using the ETABS software. The first model was the prototype flat plate structure, and the second model was the test specimen. A comparative study was carried out in order to determine a suitable boundary condition for the experimental test setups, in which the behavior of the connection in the prototype structure can be well reflected by the isolated test specimen. An acceptable boundary condition was assumed to have been obtained when the resulting stresses and deflection contours over the slab surface of the test specimen were similar to those in the prototype structure.

The prototype building system consisted of a flat plate floor, acting as a main vertical load carrying system, combined with shear walls or moment-resisting frames designed to resist the expected lateral forces. The prototype building was designed with a 150 mm slab thickness, 4 x 5 m span dimensions, 3 m story heights, and 1 x 0.2 m rectangular columns with the longer dimension extend in the transverse direction. Layout of the prototype building is shown in Figure 4.3.

Figure 4.3 Plan View of Prototype Building

As mentioned, the prototype structure was modeled using ETABS structural design software. A typical ten-story flat plate building was designed in combination with a primary lateral load-resisting system in a form of a lift core, as given in Appendix A. Service gravity loads of 1 kPa superimposed dead load, and 2 kPa design live loads were specified for residential type occupancies. However, during earthquakes only 30% of the live load was deemed present, resulting in a uniformly distributed design load of 1.6 kPa (unfactored). A lateral drift of 1.5 percent of the frame height was introduced at the floor joints, and the corresponding bending moments were determined. These values were then used to design the slab flexural reinforcement. The column of the specimen was over-reinforced to avoid a plastic hinge formation, and to prevent premature failure at the column.

In the modeled specimen, application of gravity loading was simulated by an upward force from the base of the column, and the resulted deflection profile was verified with those obtained from the prototype structure (Fig. 4.4). Additional imposed loads (in the experiment, they were represented in a form of steel blocks) were also placed on the slab so that the distribution of shears and moments in the slab near the column would closely resemble those in the prototype building.

Application of lateral loadings was simulated by horizontal forces acted on the top of the column. To model the simply supported boundary condition along the slab edges, ten vertical steel edge links were provided along the edges, as shown in Appendix A. It is to be noted that due to some limitations in the laboratory, namely the holes provided by the strong floor and limited available space, finally only 8 edge links were fabricated and used in the experiment. The result from the modeling in Appendix A can still be reliable since the change in the number of edge links is such that the difference is minimized. Each side of the boundary conditions still retained the same number of edge links, i.e. 3, 4, and 3 edge links along the simply supported boundary condition. The edge links behave like roller supports, allowing lateral movements with no vertical deflection to represent the midspan of the prototype floor. It was assumed that no deflection occurred at the midspan during cyclic lateral loading, but rotations at the slab midspan were allowed. In addition, pin type connections were also provided at the mid height of top and bottom columns. This boundary condition was designed to enable cyclic lateral loading application to the test specimens. Figure 4.5 illustrates the deflected shape of both the prototype structure and the modeled specimen subjected to uniaxial lateral loading.

These support conditions of the specimen would have resulted in a significant in-plane rigid body rotation when lateral loads are applied during testing. To limit this rigid body rotation, a specially designed “torsion-restraining frame”, discussed in more detail in subsequent section, was attached to one side of the slab specimen. Lastly, comparison on the final deflected shape between the prototype and the model due to combined gravity and lateral loads showed that the modeled boundary conditions appears to be satisfactory (Figure 4.6). Each of the boundary conditions used in the experimental test setups will be discussed in details in the following section.

Figure 4.4 (a) Deflection Profile of Prototype under Gravity Load
(b) Deflection Profile of Model under Gravity Load

Figure 4.5 (a) Deflection Profile of Prototype under Lateral Load
(b) Deflection Profile of Model under Lateral Load

Figure 4.6 (a) Deflection Profile of Prototype under Gravity and Lateral Load
(b) Deflection Profile of Model under Gravity and Lateral Load

4.4 Reinforcement Layout

All five specimens were identical and they consisted of a reinforced concrete slab and a rectangular column, which were monolithically cast at the connection. The slabs were reinforced in top and bottom layers. The top and bottom flexural reinforcements were of high strength deformed bars of diameter 13 mm (T-13) and 10 mm (T-10), respectively.

Top slab reinforcement was concentrated in the column strip with a reinforcement ratio of 1.2 percent each way. The bar arrangements were in such a way that the top bars in the N-S direction (y-axis) were placed at the outermost layer. Thus, the effective depths d_x , d_y and d_{ave} are 100.5 mm, 113.5 mm and 107 mm, respectively. The bottom reinforcement had a uniform reinforcement ratio of 0.33 percent. At least two continuous bottom bars passing through the column were provided to prevent total collapse of the specimen at the occurrence of punching shear failure.

The column was reinforced with T-20 bars with a spacing of 140 mm center to center, and T-10 bars were used as the reinforcement cage for the column with a spacing of 100 mm center to center. A reinforcement ratio of 2.5 percent was provided for the columns, and was also post-tensioned by two 40 mm post-tensioning bars to model the column load from upper stories. Details on the reinforcements for the slab, column, and the anchorage details are given in Figs 4.7, 4.8 and 4.9 respectively.

(a) Top flexural reinforcement

(b) Bottom flexural reinforcement

Figure 4.7 Flexural Reinforcement Layout of Slab Specimens

(a) Column Elevation View

(b) Column Cross Section A-A

Figure 4.8 Column Reinforcement Details of Test Specimens

Fig. 4.9 Anchorage Details of Top and Bottom Slab Steel

4.5 Materials

4.5.1 Properties of Concrete

The ready-mixed concrete used to construct the specimens had the following specifications:

- a. Cement Type : Ordinary Portland Cement
- b. Concrete cube characteristic strength, f_{cu} : 40 MPa
- c. Nominal Aggregate size : 20 mm
- d. Slump : 175 ± 25 mm

The actual compressive cylinder and cube strengths of the five specimens were obtained by testing at least three standard cylinders (150 x 150 x 300 mm) and cubes (150 x 150 x 150 mm). Each was tested using a compression machine each time at various days of curing, and the test results are tabulated in Appendix B. Table 4.2 lists the corresponding average value of concrete strengths for each of the slab specimens. The average concrete cylinder strength f'_c was 34 MPa, while the average cube compression strength f_{cu} was 39 MPa. The Young's Modulus of the concrete E_c was 26 GPa.

Table 4.2 Concrete Properties

Specimen	Strengths for cubes, f_{cu} (N/mm ²)	Strengths for cylinders, f'_c (N/mm ²)
E1H	40.3	33.0
E2H	39.1	32.5
E12H	38.7	34.4
E12L	39.5	35.4
E0U	39.2	33.3

4.5.2 Properties of Reinforcement

The high strength deformed steel bars (T bars) were used as reinforcements throughout the whole specimen. To verify the yield strength of the steel bars, tensile tests were conducted on three samples of each bar size of T10, T13 and T20. The properties of the reinforcing bars, including the average yield strength, ultimate strength and elastic modulus, were obtained by tensile tests conducted in a universal machine. The results were presented in the form of load-strain curves, and they were tabulated and given in Appendix B. The summary of the average reinforcement properties for T10 and T13 (for slabs) and T20 (for columns) are shown in Table 4.3.

Table 4.3 Reinforcement Properties

Bar reference	Diameter (mm)	Yield strength (N/mm ²)	Ultimate strength (N/mm ²)	Young's modulus $\times 10^3$ (N/mm ²)
T10	9.5	538.4	632.1	185.0
T13	12.9	549.3	629.5	189.0
T20	19.7	536.0	641.3	180.5

4.6 Fabrication of The Specimens

All the specimens were cast in two stages as to prevent excessive deformation at the lower portion of the column when the concrete was set. However, the connection was monolithically cast with full length of the column below the slab and one-half length of the column above the slab.

After the bottom column and slab formwork was oiled, aligned and leveled, the reinforcement cage was placed. First, the column reinforcements were assembled and put in place, aligned, leveled, and tied temporarily to the four corners of the slab formwork by means of wires. The clear cover to the stirrups of the column cage was 20 mm. Then, the slab bottom flexural reinforcement in the direction perpendicular with the slab free edge was laid first in the formwork on concrete chairs, leaving about 15mm clear concrete cover below the reinforcement mat. Bottom rebars in the parallel direction were then placed accordingly. Concrete chairs with 60mm heights were then used to support top flexural reinforcement bars in the parallel direction to the slab free edge. Hence, with the top reinforcement mat running in the perpendicular direction to slab free edge were placed on top, the clear concrete cover was approximately 15mm.

After all reinforcements were tied and checked properly, the slab together with bottom column and a portion of the top column were cast. Concrete was ordered in one batch to guarantee uniformity. In the second stage, 4 days after the first, a portion of the top column was cast while care was taken to ensure that the interface of the concrete at the column was rough and would provide good interlock. After casting, each slab was covered by plastic sheets and was kept wet for seven days of the curing period. Control cubes and cylinders were taken in both stages. Once the slabs had sufficiently hardened, each test specimen was demolded and painted in order to make concrete cracks more visible during testing.

4.7 Experimental Setup

Figure 4.10 shows the overall view of the experimental setup, which was originally designed and developed by the author for the purpose of this experimental test. The main components are the column support, vertical edge links along the simply supported slab edges, torsion-restraining frame, and horizontal N-S and E-W jack attached to the top of the column. A complete diagram outlining the various components used in the test is shown in Figure 4.11.

Figure 4.10 Experimental Setup

(a) Plan View of Test Setup

(b) Elevation View of Test Setup

Figure 4.11 Diagram of Experimental Setup

4.7.1 Column Support

A rocker-type support utilizing a universal ball bearing was provided at the bottom of the column, as shown in Fig. 4.12. The column bottom is held in its place using steel arms attached on two perpendicular sides. Pin-type connections are used on each end of steel arm allowing free end-rotations but restricting horizontal translation in any direction. A hydraulic jack was placed at the bottom of the column in order to apply the specified gravity load to the connection.

Figure 4.12 Details of Column Support

4.7.2 Vertical Edge Link

To model the simply supported boundary condition along the slab edges, eight vertical steel edge links were provided along the edges of the test specimens. The pin-type connections on each end would allow rotation as well as horizontal translation with no vertical deflection, as shown in Figure 4.13. This boundary condition was designed to enable the cyclic lateral loading application to the test specimens.

Figure 4.13 Details of Vertical Edge Link

Figure 4.14 Details of Torsion Restraining Frame

4.7.3 Torsion Restraining Frame

As discussed earlier in the preliminary analysis significant in-plane rigid body torsion can result when lateral loads are applied during testing. A torsion-restraining frame was attached to the slab to limit any rigid body torsion of the specimen when lateral loads were applied. While limiting the in-plane torsion of the specimen, the frame would still allow free horizontal translations in two horizontal directions. The idea of the torsion-restraining frame follows that of Pan & Moehle (1989). Details of the torsion-restraining frame are shown in Figure 4.14. Detailed design drawings of each component used in the test setup are attached in Appendix B.

4.8 Instrumentation

Instrumentations for this experimental investigation included load cells, a series of transducers, strain gauges, and a set of data logging equipment installed on each specimen prior to testing. They were used to record the applied lateral and vertical forces, reaction forces, slab displacements and rotations, as well as column deformations and displacements. Prior to casting of the specimens, selected slab reinforcing bars were fitted with strain gauges to obtain the strain profiles (64 gauges on slab reinforcement and 4 gauges on column reinforcement). The details of each type of instrumentation are described as follows:

4.8.1 Load Cells

In general, the load cell functions as a load-recording device. Five load cells were used in this experiment. Two sets of 50-tonnes compression load cell were placed at the top of the column to monitor the axial load in the column due to the post-tensioning, while another two sets of 50-tonnes tension-compression load cell were attached on both N-S and E-W steel arms to allow recording of the pushing or pulling of lateral loads applied. Lastly, one 100-tonnes load cell was installed at the bottom of the column in order to record the reaction force of the column support due to gravity loads.

4.8.2 Transducers

Two types of transducer used in the experiment. First type is the Linear Variable Differential Transformers or LVDT. Basically they measured the deflection of the slab specimens at 22 locations beneath the slab, as indicated in Figure 4.15(a). These LVDTs were initially set to half of their measured displacement capabilities (100mm), to cater for upward and downward deflections due to the nature of cyclic loading. Another 6 LVDTs were positioned horizontally at the column's longer dimension to measure any twisting of the column (Fig. 4.15b).

Second type of the transducer used was the cable or wire-type transducer, denoted as WLVDt in the experiment. Nine of these WLVDtS were used to assess any lateral displacements of the column (Fig. 4.15(b)). To accommodate this, a steel reaction frame was designed and placed near the column where the transducers were fastened. Another 3 WLVDtS were mounted on another steel reaction frame located along one side of the slab in order to measure any in-plane rigid body rotations of the slab specimens, shown in Fig 4.15(a).

Fig. 4.15(a) Location of LVDTs and WLVDtS on The Slab

Fig. 4.15(b) Location of LVDTs and WLVDTs on The Column

Figure 4.15 Locations of Various Transducers on the Test Specimens

4.8.3 Steel Strain Gauges

Electrical resistance strain gauges were attached to the slab flexural reinforcements to measure the strains induced in the reinforcing bars during the experiment. For each slab specimen, there were 46 strain gauges on the top reinforcement and 18 strain gauges on the bottom reinforcement. Layout of the strain gauges placed on the top and the bottom reinforcing bars for all specimens are shown in Figure 4.16 and Figure 4.17, respectively. Strain gauges were also attached to some parts of the bottom steel connectors, the torsion-restraining frame, and the vertical steel edge links to monitor any strain that may result from the loading.

Figure 4.16 Location of Strain Gauges in Top Reinforcement Bars

Figure 4.17 Location of Strain Gauges in Bottom Reinforcement Bars

4.8.4 Data Logging Equipment

Three sets of data logging equipment including two personal computers and two 100-channel switch boxes were used in the experiment. All measuring instruments were connected to the data logger by means of a switch box, unless the number of the measuring instruments are small. The personal computer recorded all the measured data such as the load, strain and deflection, continuously throughout the testing. A visual basic program called Laboratory Windows was used for this purpose.

4.9 Loading Procedures

4.9.1 Column Axial Load

Axial load on the column, replicating a constant service load of 15 % of the column axial capacity that needs to be transferred on each column in the prototype building, was provided by two 40 mm diameter post-tensioning bars through the middle of the column. Two hydraulic jacks were placed at the top of the column to give a post-tensioning force of 365 kN, and this load was kept constant during the test (Figure 4.18).

Figure 4.18 Loading Points on The Top Column

4.9.2 Slab Gravity Load

Gravity loading was applied using a vertical hydraulic jack located at the bottom column. Gravity load V_u and the resulting unbalanced moment M_u due to load eccentricity were applied simultaneously in a load control mode. The gravity loading was introduced in steps, up to 100 kN for specimens with high gravity shear ratio (Specimen E1H, E2H, and E12H), and 50 kN for specimens with low gravity shear ratio (Specimen E12L), respectively. Specimen E0U was loaded up to its ultimate capacity. A total of 48 steel blocks were also placed on the slab specimens (Fig 4.19), in addition to the slab self-weight to ensure that the distribution of shear forces and moments at the connection would closely resemble those in the prototype building.

During testing, the force in the vertical jack was maintained according to the specified gravity load. The gravity load applied at the end of this loading stage is denoted as V_u and it was sustained until failure of the specimen. The magnitude of V_u is one of the test variables studied in the experiment.

Figure 4.19 Steel Blocks on Slab Surface

4.9.3 Column Cyclic Lateral Displacement

The purpose of this experiment was to subject the test specimens to cyclic moment transfer simulating earthquake effects. Considering the end of the gravity load application as a new datum, cyclic lateral loading was then applied in displacement control mode provided by two horizontal jacks attached on the top of the column (Fig 4.18). Thus, the specimen was subjected to cyclic moment transfer, applied either uniaxially or biaxially, depending on the test parameter studied. Note that specimen E0U did not undergo this loading stage since it was subjected to ultimate shearing force V_u only.

The displacement history shown in Figure 4.20 was followed for both uniaxial and biaxial tests with target drifts expressed in percentage. For each drift amplitude, two complete cycles are performed. After 1.5 % target drift ratio was reached, an “unloading” of one cycle of 1 % drift ratio was performed after each subsequent target drift ratio was achieved. The purpose was to study the behavior of the connection at service level after loading at higher drift ratios. The target drift ratio shown in Figure 4.20 was followed until approximately 25 % of the moment transfer capacity is lost. The lateral drift ratio was calculated as the ratio of the imposed lateral displacement at the column top to the effective column height. In this case, effective column height is the distance from the point of lateral load application to the point where the universal bearing was provided at the bottom of the column.

The loading path in Figure 4.21 was followed for specimens subjected to biaxial loading. Figure 4.21 shows the biaxial loading path for each target drift level shown in Figure 4.20. Sequential biaxial loading involves an applied displacement in one direction that is maintained, while an equal displacement is applied in the orthogonal direction (Figure 4.21). The entire loading path for one cycle of a particular target drift level consists of 14 loading stages, designed to simulate all possible loadings history.

Figure 4.22 shows the sign convention adopted for the applied lateral loadings. Positive and negative values of lateral loading and displacement were set for the North and East direction (pushing of the horizontal jack) and for the South and West direction (pulling of the horizontal jack), respectively.

Figure 4.20 Target Drift Ratio for Uniaxial and Biaxial Specimens

Figure 4.21 Biaxial Displacement Path for Biaxial Tests

Figure 4.22 Sign Convention for Lateral Loading

Chapter 4: Experimental Program

In all stages, formation of new cracks and development of existing cracks were highlighted on the surface of the slab. Crack widths were measured and recorded, and the widening of the cracks was monitored, after which the experiment was continued. During the test, the horizontal load was plotted against the horizontal drift at the top of the column, resulting in a hysteretic loop for each cycle. Finally, the test was terminated after a significant loss of strength was observed, and the specimen was then completely unloaded for safety reasons. Figure 4.23 illustrates test specimen (specimen E1H) subjected to a very large drift before the occurrence of punching failure. Discussions on the experimental results for all the tested specimens are made in the next chapter. The strength, stiffness, ductility, and drift capacity with respect to the experimental variables are studied. All photographs taken during and after the experimental course are given in Appendix C.

Figure 4.23 Test Specimen E1H Subjected to Large Drift Before Failure

CHAPTER 5

EXPERIMENTAL RESULTS AND DISCUSSIONS

CHAPTER 5

EXPERIMENTAL RESULTS

5.1 Introduction

A total of five rectangular edge-column slab connection specimens were tested by first simulating the desired level of gravity loading and then applying the target cyclic lateral displacements until failure. Two specimens were uniaxially laterally loaded (Specimen E1H and E2H), two more specimens were biaxially laterally loaded (Specimen E12L and E12H), and the fifth specimen was tested under gravity load only (Specimen E0U). This chapter presents the experimental results and discussions for all specimens tested. The strength, stiffness, ductility, and drift capacity with respect to the experimental variables are studied. The variables include effect of column rectangularity, biaxial cyclic lateral loadings, and levels of gravity loadings. Lastly, conclusions and important findings of the experimental study are made. Detailed results on slab deflections and strain gauge reading of the slab flexural reinforcement are given in Appendix D.

5.2 Crack Formation and Modes of Failure

Cracks were marked at every 10 kN shearing force increments during the application of gravity loading (stage I) and at every 0.5 % drift ratio increments during the application of cyclic lateral displacement (stage II).

5.2.1 Stage I (Application of Gravity Loading):

In the initial application of gravity loading, there was no visible crack up to a load of 50 kN. Hence, for the specimen subjected to low gravity loads (Specimen E12L), no cracks were found at this stage. Additional gravity loads were applied up to 100 kN for specimens with high gravity shear (Specimen E1H, E2H, and E12H), and resulted in some formation of minor flexural cracks on the top slab surface. The

first flexural crack was observed near the short side of the column, extending radially from the face of the column at 70 kN shearing force. The first circumferential and torsional cracks were also observed at the same load. The torsional cracks were initiated at the column face, on both halves of the test specimen, propagated towards the slab free edge at approximately 45 degrees angle to the slab free edge. With further increase of gravity loads, which resulted in an increase in unbalanced moment due to load eccentricity, more radial cracks were observed on the slab surface while the existing ones propagated further towards the supported edges. At the end of stage I, crack widths in the range of 0.1 to 0.12 mm were measured from the prominent cracks in Specimen E1H, E2H and E12H.

- **Specimen E0U**

For Specimen E0U, application of gravity loading was continued until failure. Diagonal shear cracks became more prominent at a load of 130 kN, and propagated through the slab thickness as the load increases. No new cracks were developed, but widening of cracks was observed. Enormous crushing and spalling of concrete cover exposing the top flexural bar was observed. Eventually, the top flexural bars running perpendicular to the free edge were bent and pushed the concrete cover outwards at load exceeding 200 kN. After reaching the ultimate load of 245 kN coupled with an unbalanced moment transfer of 81 kNm due to load eccentricity, punching failure occurred simultaneously with a drastic drop in the load capacity (Figure 5.1).

5.2.2 Stage II (Application of Cyclic Lateral Displacement):

Application of lateral loading in the North direction (positive values of lateral loading and drift) caused tension in the top slab flexural reinforcement, and thus, the resulting unbalanced moment is additive with the initial unbalanced moment from stage I due to gravity loading. Hence, during the application of positive drift, the existing cracks due to the application of gravity loads widened and propagated throughout the top slab surface. While application of lateral loading in the South direction (negative values of lateral load and drift) caused tension in the bottom slab flexural reinforcement, and thus, initiating the occurrence of cracks

on the slab bottom surface. However, the bottom cracks only occurred at high negative drift values because the bottom flexural reinforcement was in compression due to the gravity load application at loading stage I. For specimens loaded along East-West direction (Specimen E2H), more new cracks were formed as a result of the cyclic unbalanced moment transfer along this direction, and only few of the existing cracks in stage I propagated across the slab.

In the biaxially loaded specimens (Specimen E12L and E12H), propagation of circumferential cracks around the connection region crossing the radial and torsional cracks was more prominent than the uniaxially loaded specimens (Specimen E1H and E2H). More flexural cracks and wider propagation of cracks were found on both top and bottom slab surfaces, especially for the specimen with high gravity load (Specimen E12H).

These cracks spread and opened as the test progressed until the final rupture surface surrounding the column due to punching developed. Ultimately, crack widths increased substantially once the peak lateral load was reached, and punching shear failure occurred along the rupture surface followed by a significant drop in the load carrying capacity. All the tested specimens failed by punching of the slab in the column vicinity. Figure 5.1 shows the punching shear failure that occurred on all the specimens with the typical radial and circumferential cracks shown on the top slab surface. Considerable spalling of concrete cover exposing the top flexural bar was observed at the occurrence of punching failure. The angle of cracking into the slab was approximately 30 degrees to the horizontal, shown by the diagonal shear crack formed on the side of the slab free edge at the column junction (Fig. 5.1a-(e)).

It can be seen from Figures 5.1a-(a) and (b) that the punching shear crack in Specimen E1H (uniaxially loaded along the stronger column direction) formed at a much further distance from the column face, approximately two times than that in specimen E2H (uniaxially loaded along the weaker column direction). Hence, this suggests that column rectangularity affects the punching shear behavior of laterally loaded flat plate connections. More detailed discussions on the influence of column rectangularity on the strength, stiffness, ductility and drift capacity of slab-column connections will be made later in this chapter.

Chapter 5: Experimental Results

Figure 5.1a Punching Shear Failure of Specimens E1H, E2H, E12L and E12H
(under Gravity and Cyclic Lateral Loading)

Figure 5.1b Punching Shear Failure of Specimens E0U (under Gravity Loading)

5.3 Hysteretic Response

Hysteretic response in terms of unbalanced moment versus drift ratio may show the performance of slab-column connections during an earthquake. Figure 5.2 to 5.5 summarize the hysteretic response for all the test specimens, except specimen E0U which was tested under ultimate gravity load only. The unbalanced moment at a stipulated drift was obtained by multiplying the applied lateral load by the effective column height, while the drift ratio was calculated as the ratio of the imposed lateral displacement at the top column to the effective column height. The experimental unbalanced moment-drift ratio responses shown in Fig. 5.2 to Fig 5.5 appeared to have different characteristics among each specimen, indicating different behavior resulted from different test variables.

The characteristics of the lateral response can generally be separated into two sets. The first sets are those results involving unbalanced moment transfer along N-S direction, or in the stronger column direction (Fig. 5.2, Fig. 5.4a, and Fig. 5.5a). Application of lateral loading in the North direction (positive values of lateral loading and displacement) caused tension in the top slab flexural reinforcement, and thus, the resulted unbalanced moment is additive with the initial unbalanced moment due to gravity loading. While, application of lateral load in the South direction (negative values of lateral loading and displacement) reduces the initial

unbalanced moment due to gravity loading because the top slab flexural reinforcement would be in compression.

The second sets of hysteretic response are those where the application of gravity loading contributed to practically zero unbalanced moment at the connection, or the unbalanced moment transfer along E-W direction (Fig. 5.3, Fig. 5.4b, and Fig. 5.5b). In the East-West direction, the resulting positive and negative unbalanced moment values due to cyclic lateral loading remain the same throughout, since there are two slab spans next to the column that are able to develop an equal and opposite resistances in the top and bottom slab flexural reinforcement, similar to interior connections

Figure 5.2 Hysteretic Loop for Specimen E1H

Figure 5.3 Hysteretic Loop for Specimen E2H

Figure 5.4a North-South Hysteretic Loop for Specimen E12L

Figure 5.4b East-West Hysteretic Loop for Specimen E12L

Figure 5.4 Biaxial Hysteretic Loops (North-South and East-West) for Specimen E12L

Figure 5.5a North-South Hysteretic Loop for Specimen E12H

Figure 5.5b East-West Hysteretic Loop for Specimen E12H

Figure 5.5 Biaxial Hysteretic Loops (North-South and East-West) for Specimen E12H

5.3.1 Uniaxial Tests Response

In the uniaxial cyclic lateral loading tests, there are only 2 lateral loading paths involved, push or positive displacement and pull or negative displacement. Thus, specimens subjected to uniaxial load resulted in only one hysteretic curve in the lateral direction considered, i.e. North-South (strong column direction) or East-West (weak column direction).

• Specimen E1H

Specimen E1H (subjected to uniaxial cyclic lateral loading in the strong column direction and with high gravity shear) failed in punching shear and its overall hysteretic response is given in Fig. 5.2. Peak unbalanced moment of 146.1 kNm was observed at the first cycle of a lateral drift ratio of 3.02 percent. A subsequent cycle at the same target drift ratio of 3 percent caused a load drop at about 10 percent. It was noted that a drop in the load resistance of the second displacement cycle was always observed beyond 1 percent drift ratio, and it became

more imminent near failure, as there were no new cracks formed. This deteriorating load capacity on the second cycle beyond 1 percent drift ratio indicated that existing cracks propagated and widened across the slab depth, causing more loss of stiffness throughout the course of experiment. Finally, brittle punching failure occurred, followed by a significant drop in the unbalanced moment transfer at about 70 percent of its capacity, observed at the first load cycle of 3.9 percent drift ratio. The test was stopped at this stage as the area surrounding the column was severely damaged due to punching (Fig. 5.1a). From the hysteretic loops in Figure 5.2, it can be seen that the specimen E1H failed in a very brittle manner with only three post-ultimate load cycles achieved, while the last load cycle caused a drastic drop in moment transfer to a value just slightly above its initial unbalanced moment.

- **Specimen E2H**

Specimen E2H (subjected to uniaxial cyclic lateral loading in the weak column direction and with high gravity shear) also failed in punching shear, and the hysteretic response is given in Fig. 5.3. The punching shear failure occurred after the peak unbalanced moment transfer of 101.5 kNm at 4.29 % drift ratio, and was indicated by a sudden load drop of approximately 25 percent of its capacity in the subsequent loading cycle. Further application of displacement cycles in this direction showed some form of ductility, indicated by a gradual strength reduction up to a very high drift ratio of 6 percent, after it had lost most of the moment transfer capacity. Figure 5.3 shows that much more cycles of higher drift ratio were needed in order to dissipate its moment transfer capacity, and the load drop was found to be more gradual than the one observed in Figure 5.2. Although the Specimen E2H (loaded in the weaker column direction) demonstrated a good ductility, its unbalanced moment capacity of 101 kNm as shown in Fig. 5.3 is lower than the unbalanced moment capacity of Specimen E1H (loaded in the strong column direction) by about one-third. The differences in the hysteretic responses obtained for both uniaxial specimens (Fig. 5.2 and 5.3) showed that column rectangularity significantly influence the punching behavior of slab-column connections. Brittle punching behavior was observed in the strong column direction

(Specimen E1H in Fig. 5.2), while a relatively more ductile behavior was observed in the weak column direction (Specimen E2H in Fig. 5.3).

5.3.2 Biaxial Tests Response

Biaxial cyclic lateral loading tests resulted in two hysteretic curves in two perpendicular lateral directions that are inter-dependent, involving 14 loading stages as shown earlier in Fig. 4.21. The hysteretic responses obtained from the biaxial tests can be explained by examining each of the 14 loading stages at a particular target drift ratio. Fig. 5.6a shows the typical load-displacement sequences in the North-South direction where the initial unbalanced moment due to gravity loading exists, while Fig. 5.6b shows the typical load-displacement sequences in the East-West direction where the initial unbalanced moment is practically zero. The numerals in Figs. 5.6a and 5.6b correspond to the loading paths described in Fig. 4.21.

In general, it can be observed in both Figs. 5.6a and 5.6b that while the displacement is maintained in one direction, the lateral resistance in that direction drops due to a displacement applied in the orthogonal direction. These load drops can be observed in both Fig. 5.6a (step 1 to 2, step 3 to 4, step 5 to 6, step 7 to 8, step 9 to 10, step 11 to 12, and step 13 to 14) and Fig. 5.6b (step 2 to 3, step 4 to 5, step 6 to 7, step 8 to 9, step 10 to 11, and step 12 to 13). Hence, this suggests that the application of biaxial loading reduces the lateral strength of slab-column connection, possibly as more cracks occurred due to biaxial actions.

(a) N-S Displacement Sequence

(b) N-S Load Displacement Sequence

Figure 5.6a Load Displacement Sequences in North-South (N-S) Direction for Biaxially Loaded Specimens

(a) E-W Displacement Sequence

(b) E-W Load Displacement Sequence

Figure 5.6b Load Displacement Sequences in East-West (E-W) Direction
for Biaxially Loaded Specimens

- **Specimen E12L**

The hysteretic responses for Specimen E12L (subjected to biaxial cyclic lateral loading and with low gravity shear) are presented in Fig. 5.4. Punching failure affected the behavior of the connection in both directions, and it occurred at about 1.9 percent drift ratio. After the peak unbalanced moment of 104.3 kNm was reached in the strong column direction (N-S) at a drift ratio of 1.9 percent, the load could not increase any further in the subsequent application of 2.5 percent drift ratio (Fig. 5.4a), and it even dropped in the subsequent loading path (Fig. 5.4b). Peak unbalanced moment of 104.3 kNm in the N-S direction was recorded at 1.90 % drift ratio (Fig. 5.4a), while peak unbalanced moment of 74 kNm in the E-W direction was recorded at 2.02 % drift ratio (Fig. 5.4b). It was observed that the occurrence of punching failure was simultaneous in both directions and was governed by the strength in the strong column direction. From the crack patterns, it could be seen that the critical direction for punching failure was along the strong column direction.

It was also observed that the punching failure occurred at loading stage 3 of the biaxial loading path shown in Fig. 4.21. An investigation into the hysteretic curve patterns in Fig. 5.6a and 5.6b showed that only at loading stage 3 the peak lateral load is at maximum in both directions. Hence, the applied shear stresses at loading stage 3 represent the maximum achievable shear stresses that may cause punching. Once these shear stresses exceeded the shear capacity of the slab, punching shear failure occurred, followed with a significant drop in the load resistance. Other loading stages that also caused biaxial unbalanced moment transfer, i.e. loading stage 6, 9 and 12 resulted in lower stresses compared to those stresses in loading stage 3. In fact, for loading stage 6 and 9 shown in Fig. 5.6a, negative unbalanced moment was transferred, reducing the positive initial unbalanced moment due to gravity loading.

- **Specimen E12H**

Similar characteristics of hysteretic response were also found on Specimen E12H (subjected to biaxial cyclic lateral loading and with high gravity shear) shown in Fig. 5.5. Peak unbalanced moment of 93.7 kNm was achieved in the strong column direction at 2.1 % drift ratio (Fig 5.5.a), while peak unbalanced moment of 60.2 kNm was recorded at 1.6 % drift ratio in the weaker column direction (Fig. 5.5b). Punching failure in Specimen E12H only occurred after 2.1 percent drift ratio was reached in both loading directions, and was governed by the strength in the strong column direction. In Fig. 5.5a, it can be seen that after the peak unbalanced moment in the strong column direction was reached at 2.1 % drift ratio, the load-displacement loop flattened significantly as the loading path continued.

The sudden drop in the overall hysteretic loops in Fig. 5.5a immediately after punching indicated a rapid loss of stiffness and a high-energy dissipation capacity. As observed in the experiment that most of the energy was dissipated by yielding of the slab reinforcement and opening of the flexural cracks. Some dissipation of energy also contributed due to slippage of the slab reinforcing bars, local crushing of concrete and friction along the major cracks. This phenomenon of rapid loss of stiffness suggests that the high gravity shear level applied in Specimen E12H caused more brittle punching behavior compared to that in Specimen E12L.

Fig. 5.4a and Fig. 5.5a showed that for a biaxially loaded rectangular-column slab connection, a more brittle behavior was found in the strong column direction. In addition, higher peak unbalanced moment value was always obtained for the strong column direction. This observation is in line with the behavior found in the uniaxial specimens discussed previously, where more brittle punching behavior with higher unbalanced moment transfer capacity was also observed in the strong column direction. Hence, this indicates that besides biaxial loading, column rectangularity also significantly influenced the modes of failure as well as the performance of slab-column connections under lateral loadings. Table 5.1 summarized the experimental peak unbalanced moments recorded for all five specimens with their corresponding peak drift ratios.

Table 5.1 Experimental Peak Unbalanced Moments With Its Corresponding Drift Ratios
And Peak Drift Ratios With Its Corresponding Unbalanced Moments

Specimen	V_u/V_c	Lateral Load Application	$M_{u,x}$ (kNm)	$M_{u,y}$ (kNm)	$DR_{u,y}$ (%)	$DR_{u,x}$ (%)
E1H	0.31	Uniaxial N-S	146.1* 79.8 -81.6* -43.7	-	3.02 3.96+ -3.50 -3.92+	-
E2H	0.31	Uniaxial E-W	50.1	101.5* 9.10 -98.3* -25.2	-	4.29 6.02+ -4.30 -5.97+
E12L	0.16	Biaxial	104.3* 80.1 -55.8* -9.51	74.0* 20.1 -72.4* -10.1	2.51 3.10+ -2.10 -3.20+	2.02 2.97+ -2.10 -3.20+
E12H	0.31	Biaxial	93.7* 60.0 -51.5* -16.3	60.2* 20.4 -60.0* -20.1	2.10 2.50+ -2.10 -2.60+	1.60 2.50+ -1.59 -2.50+
E0U	0.77	-	81.0*	-	-	-

* Peak Unbalanced Moment

+ Peak Drift Ratio

5.4 Strain in Flexural Reinforcements

Strains in the reinforcing bars were recorded using strain gauges, and their locations on the slab were described in the previous chapter. Strain measurements were taken to determine the occurrence of the yielding of reinforcements, the distribution of the applied lateral moment across the slab, and to compare strain values among the test specimens. Yielding of the slab flexural reinforcements can be considered as one of the indicators of ductility and the intensity of the unbalanced moment transfer. Figures 5.7 to 5.11 show locations of the yielded reinforcing bars at failure in top and bottom flexural reinforcements for all the specimens. Strain distributions along the shorter and longer sides of the rectangular column recorded at the peak unbalanced moment transfer are summarized in Fig 5.12 and 5.13, respectively. The complete strain profile curves at various lateral drift ratios for all specimens are shown in Appendix D.

The higher strain values for all bars running parallel in the strong column direction (Fig.5.12) than values occurring on bars running parallel in the weak column direction (Fig.5.13) suggests that more unbalanced moment was being transferred about the strong column axis. It is also noted in Fig 5.13 that although specimen E2H was uniaxially loaded in the weak column direction, none of the bars running parallel to the weak column direction yielded, indicating less unbalanced moment was transferred about the weak column axis. These observations imply that in rectangular-column slab connections, higher unbalanced moment transfer is expected about the stronger column axis.

The strain measurements plotted in Figs. 5.12 and 5.13 indicates that only those reinforcing bars located near the short side of the rectangular column are likely to yield. Fig. 5.12 shows that bars 4 to 7 yielded in all the specimens, except in E2H. For reinforcing bars parallel to the slab edge (Fig 5.13), bar 3 and 4 which were located near the column short side also yielded, except in E1H and E2H. This indicates that high shear stresses were concentrated around the short side of the rectangular column, and these stresses decreased as it moved away from the short side of the column. Hence, punching shear is critical in rectangular-column slab connections transferring unbalanced moment about the strong column axis since the shear capacity of the connections may be limited by the resulting shear stresses near the short side of the column

Figure 5.12 shows that the effective moment transfer width of $1.5 h$ away from the column face specified by the ACI Code to resist the unbalanced moment transferred by flexure is reasonable. However, this width is not suitable to define the effective moment transfer width on the longer side of rectangular column, as shown in Fig. 5.13. Figure 5.13 indicates that almost half of the longer side of the column is not effective in transferring unbalanced moment. Only those bars located near the short side of the rectangular column are likely to yield and hence, for rectangular columns, the real effective moment transfer width for the longer side of the column is much less than that specified by the ACI Code.

(a) Top Bars

(b) Bottom Bars

Figure 5.7 Yielded Bar Locations in Specimen E1H

(a) Top Bars

(b) Bottom Bars

Figure 5.8 Yielded Bar Locations in Specimen E2H

(a) Top Bars

(b) Bottom Bars

Figure 5.9 Yielded Bar Locations in Specimen E12H

(a) Top Bars

(b) Bottom Bars

Figure 5.10 Yielded Bar Locations in Specimen E12L

Chapter 5: Experimental Results

Figure 5.11 Yielded Bar Locations in Specimen E0U

Figure 5.12 Strains in Top Flexural Reinforcement Bars Along N-S Directions

Figure 5.13 Strains in Top Flexural Reinforcement Bars Along E-W Directions

5.5 Slab Deflection

Vertical slab deflections were measured by vertical LVDTs on the underside of the slab. Although V_u was sustained at a constant value until failure, applications of cyclic lateral loading (which was applied in displacement control) would deteriorate the connection strengths, and hence, increased the slab vertical deflection for a constant V_u . Figure 5.14 shows the variation of the column stub displacement during stage II with the applied lateral load up to failure for all specimens, taken after the application of the positive target drift ratio of the first cycle. Fig. 5.14a shows the displacement responses under North-South lateral loadings, while Fig. 5.14b shows the displacement responses under East-West

lateral loadings. Figure 5.14 indicates that higher gravity loading and biaxial load application increase the vertical deflection significantly. Specimen E0U which was subjected to ultimate gravity loading attained the highest column vertical displacement.

The variations of the column stub displacement during stage II with the applied lateral drift ratio up to failure for all specimens, taken after the application of the positive target drift ratio of the first cycle are plotted in Figure 5.15. It can be seen from Fig. 5.15 that the column vertical displacement increased with the increase in lateral drift ratios, due to successive shear deterioration experienced with increased lateral drifts. Figure 5.15 shows that more shear deterioration are observed for specimens with higher gravity loading and specimens with biaxial loading. Application of lateral loading in the strong column direction (N-S) is also more critical than loading in the weak column direction (E-W), as indicated by higher vertical displacement shown by Specimen E1H than E2H in Fig. 5.15.

Profiles of the slab deflections along the North-South and East-West span directions at the peak positive unbalanced moment transfer are plotted in Fig. 5.16 and Fig. 5.17, respectively. All deflection profiles plotted in Fig. 5.16 and 5.17 include those deflections recorded during stage I (gravity load application). The high value of slab vertical deflection signifies the damage concentration in the slab around the column. This can be attributed to the push-out effect of the top flexural reinforcement causing separation of top concrete cover around the column, indicating that a full diagonal shear crack has developed across the slab depth causing punching at the connection.

Fig. 5.16 and 5.17 confirm the observation discussed previously that higher gravity load specimens and biaxially loaded specimens would attain higher slab vertical displacement due to more shear deterioration. Fig. 5.16 and 5.17 also show that the application of lateral loading in the strong column direction (Specimen E1H) would cause higher vertical displacement than in the weak column direction (Specimen E2H). As shown in Fig 5.16 and 5.17 that Specimen E2H (uniaxially loaded in the weak column direction) attained the lowest value of vertical slab displacement, while Specimen E0U (subjected to ultimate gravity loading) attained the highest slab vertical displacement.

Figure 5.14 Column Vertical Displacement vs Lateral Loading

Figure 5.15 Column Vertical Displacement vs Drift Ratio

Figure 5.16 Slab Deflection Profiles along East-West Span Direction

Figure 5.17 Slab Deflection Profiles along North-South Span Direction

5.6 Connection Strength

The connection strengths in terms of peak unbalanced moment transfer for all the specimens are tabulated in Table 5.1. Sign convention follows the diagram shown in Fig. 4.22 where x-axis is the weak column direction (E-W) and y-axis is the strong column direction (N-S).

5.6.1 Effect of Column Rectangularity

Two uniaxial specimens with high gravity shear, namely Specimen E1H and Specimen E2H were laterally loaded along the y and x-axis respectively. From Table 5.1, it can be seen that the ability of specimen E1H and E2H in transferring unbalanced moment was substantially different. Specimen E1H was capable of transferring a higher value of unbalanced moment (146.1 kNm), which is about 44 percent more than specimen E2H (101.5 kNm), as shown from the unbalanced moment-drift ratio envelopes in Fig. 5.18a. Thus, for flat plate connections supported by rectangular columns, higher unbalanced moment transfer capacity about the strong column axis than its capacity about the weak column axis would be expected.

Similar observations are also made in the unbalanced moment-drift ratio envelopes for the biaxially loaded specimens, Specimen E12L and E12H, shown in Fig. 5.18b. The unbalanced moment transfer capacity about the strong column axis is higher than its capacity about the weak column axis. This enhancement effect on the capacity of moment transfer about the strong column axis due to column rectangularity was recorded to be about 56 percent for Specimen E12H, and 41 percent for Specimen E12L (Fig. 5.18b).

The ability of the rectangular-column slab connections in transferring more unbalanced moment about the strong column axis can also be seen from strain measurements plotted in Fig. 5.12 and Fig. 5.13. High strain values obtained for all reinforcing bars running parallel in the strong column direction (Fig.5.12) than values occurring on bars running parallel in the weak column direction (Fig.5.13) confirms that more unbalanced moment was being transferred about the strong column axis.

Figure 5.18 (a) Uniaxial Specimens: Specimen E1H and E2H

Figure 5.18 (b) Biaxial Specimens: Specimen E12L and E12H

Figure 5.18 Effect of Column Rectangularity on Moment Transfer and Drift Capacity

5.6.2 Effect of Biaxial Loading

The effects of biaxial loading on the connection strength are illustrated using the envelopes of unbalanced moment versus drift ratio given in Fig. 5.19. In the strong column direction (N-S), comparison between Specimen E1H and Specimen E12H are made, while in the weak column direction (E-W), Specimen E12H is compared with Specimen E2H. Fig. 5.19 shows that the biaxial load effect was found to reduce the unbalanced moment transfer capacity by 36 percent in the strong column direction (N-S), while a reduction of 40 percent was observed in the weak column direction (E-W). Hence, the above two observations suggest that biaxial load effect significantly reduces the connection strength in both directions.

Figure 5.19 Effect of Biaxial Loading on Moment Transfer and Drift Capacity

The effects of biaxial loading on connection strength can also be seen from the P_x vs P_y curves shown in Figure 5.20 and 5.21. Note that the curves shown in Fig.5.20 and 5.21 represent those from the first cycles only. The P_x vs P_y curves show the interaction between the applied lateral forces P_x and P_y as the target drift ratio increases. For each target drift ratio, the curve consists of four skewed quadrilaterals with the peak load values for each direction plotted. The initial

position when both P_x and P_y are not activated is indicated as loading stage 1. In load stage 1, where only gravity load was applied, an initial value of 16.8 kN was recorded for P_y due to load eccentricity of the edge connection. This initial value is different for high and low gravity shear specimens.

Example of biaxial load actions in Specimen E12H can be studied by examining the first quadrant (consist of loading stage 1, 2, 3, and 4) for drift ratio equals to 0.25 % in Fig. 5.20. At loading stage 2, where P_y is not activated, P_x will record a value of 6.1 kN. When P_y is being applied and increased to 17.7 kN at loading stage 3, P_x decreased to value of 4.8 kN due to biaxial actions. However, when P_x is being unloaded from 4.8 kN to 0 kN at loading stage 4, the value of P_y increased from 17.7 kN at loading stage 3 to 18.3 kN at loading stage 4. This observation suggests that at low drift ratios, where the cracking inside the slab are still limited, the connection can still regained back its uniaxial strength although it has been previously subjected to biaxial actions. Similar observations can also be found in other loading stages of 0.25 % drift, that the peak loads P_x and P_y at uniaxial loading stages (5, 7, 8, 10, 11, 13) are all higher than the peak loads P_x and P_y at biaxial loading stages (6, 9, and 12).

As the slab cracking progressed and opened up at more load cycles, followed by yielding in some of the top slab reinforcements, the connection strength plotted at 0.75 % drift ratio shows a different behavior as previously observed (Fig. 5.20). The interaction of peak lateral loads at 0.75 % drift ratio in Fig. 5.20 shows that after the biaxial loading stage (loading stage 3, 6, 9, and 12), the peak load at the uniaxial stage did not regain back its uniaxial strength. In fact, it can be seen that at 1 % drift ratio (Fig.5.20), the peak load P_x or P_y dropped after the connection was loaded biaxially in loading stages 3, 6, 9, and 12. The observed load drop in both directions after the biaxial loading stage indicates the severity of biaxial loading in affecting the strength of slab-column connection. Hence, application of biaxial loadings would cause punching failure to occur much earlier, as it reduces the connection strength capacity in both directions.

From Figs. 5.20 and 5.21, it can also be seen that the value of the peak lateral loads in both directions P_x and P_y at loading stage 3 are always higher than the loads in other biaxial loading stages. Hence, this confirms the earlier discussions that the loading stage 3 governed the punching strength of the slab-column connection.

The effect of column rectangularity can be deduced by the different values of P_x and P_y recorded at the critical loading path, i.e. loading stage 3. It can be seen from loading stage 3 at 1.5 % drift ratio in Fig.5.20, that the value of P_x is 17.9 kN, while P_y is 30.4 kN. The difference in values of the applied loadings P_x and P_y at the same loading point (at loading stage 3) proves that the strength of the connection in the stronger column direction or y-axis is higher than the strength in the weaker column direction or x-axis. The ratio of the peak lateral load in the stronger to weaker column directions (P_y/P_x) is approximately 1.7 for both specimen E12H and E12L.

Figure 5.20 Interaction of Peak Lateral Loads for Specimen E12H

Figure 5.21 Interaction of Peak Lateral Loads for Specimen E12L

5.6.3 Effect of Gravity Shear

The effect of gravity shear on connection strength can be observed from Fig. 5.22. It can be seen from Fig. 5.22 that the higher gravity shear specimen (E12H) transfers less unbalanced moment in both N-S and E-W directions than the lower gravity shear specimen (E12L). This suggests that higher gravity shear applied at the connections reduce the unbalanced moment transfer capacity of the connections. Hence, limiting the amount of gravity shear imposed on slab-column connections can be considered as one of the means to safeguard the unbalanced moment transfer capacity needed to withstand lateral actions arising from either wind or earthquake loadings.

Although the gravity shear in Specimen E12H is twice the value of gravity shear in Specimen E12L, the reduction in unbalanced moment transfer capacity is only 19 percent about the column weak axis and 10 percent about the column strong axis. Hence, the results indicate that among other effects considered in this experiment, the effect of high gravity shear on rectangular-column slab connections having a column aspect ratio of five is relatively low in reducing the connection strength.

Figure 5.22 Gravity Load Effect for Rectangular Edge-Column Slab Connections

5.7 Connection Stiffness

The hysteresis envelopes are plotted in Figure 5.23, and it can be seen that as the connections undergo inelastic lateral deformations, their stiffness suffers more deterioration due to cracking. The peak-to-peak stiffness parameter S adopted by several other researchers (Pan and Moehle, 1989; Robertson and Durrani, 1992; Durrani et. al, 1995; Megally and Ghali, 2000), as shown in Fig. 5.24, is used to calculate the connection stiffness for each displacement cycle. The resulting stiffness parameter S for all test specimens is plotted in Fig. 5.25 and it shows that for rectangular-column slab connections, stiffness in the weak column direction is expected to be lower than stiffness in the strong column direction. Hence, this indicates that the use of rectangular columns may affect the connection stiffness. Note that for the biaxial specimens, the stiffness plotted in Fig. 5.25 are those where the lateral loads in both directions were active. The following discussions are made based on the stiffness parameter at 1.5 percent drift ratio, as indicated by the dotted line in Fig. 5.25.

Figure 5.23 Envelopes of Unbalanced Moment vs Drift Ratio

Figure 5.24 Definition of Stiffness Parameter S

Figure 5.25 Stiffness Parameter S vs Drift Ratio

5.7.1 Effect of Column Rectangularity

The effect of column rectangularity on connection stiffness can be deduced by comparing the stiffness parameter S in both North-South and East-West directions. Fig. 5.25 shows that for rectangular-column slab connections, stiffness in the weak column direction (E-W) is expected to be lower than the stiffness in the strong column direction (N-S). For the uniaxial specimens, it was found that the stiffness in the weak column direction (Specimen E2H) is only 0.55 times that in the stronger column direction (Specimen E1H). Hence, the results indicate that high column aspect ratio would lower the available connection stiffness needed to resist lateral loading in the weak column direction, as the column would be flexible in this direction. The level of reduction of lateral stiffness in the weaker column direction would depend on the column rectangularity ratio.

The column rectangularity effect on lateral stiffness can also be found in the biaxial specimens, i.e. Specimen E12L and E12H. It was observed from Fig. 5.25 that, for the biaxial specimens, the stiffness in the weak column direction or E-W direction is also lower by approximately 0.8 times that in the strong column

direction for Specimen E12L and by 0.6 times for Specimen E12H. The difference in stiffness values in these two specimens signifies the gravity load effect. The much lower stiffness value observed in Specimen E12H indicates that high gravity shear further reduces the lateral stiffness in the weaker column direction. Thus, it can be concluded that for rectangular-column slab connections, stiffness in the weak column direction is expected to be lower than the stiffness in the strong column direction, and application of high gravity shear would further reduce the connection stiffness in the weaker column direction.

5.7.2 Effect of Biaxial Loading

The effect of biaxial loading on the connection stiffness can be explained by comparing the uniaxial specimens, E1H and E2H, with the biaxial specimens, E12H (N-S) and E12H (E-W), of the same gravity loading, respectively. Fig. 5.25 shows that in the strong column direction (N-S), the stiffness of the biaxial specimen E12H is only 75% of that in the uniaxial specimen E1H. Meanwhile, in the weak column direction (E-W), the stiffness of the biaxial specimen E12H is 80% of the uniaxial specimen E2H. These results confirmed that application of biaxial loading would decrease the lateral stiffness of slab-column connection. In addition, the result also suggests that the stiffness reduction due to biaxial loading effect is approximately the same in both sides of the rectangular column.

5.7.3 Effect of Gravity Shear

The effect of gravity loading on lateral stiffness can also be observed by comparing the specimen with low gravity shear (E12L) and the specimen with high gravity shear (E12H) in both N-S and E-W directions at 1.5 percent drift ratio (Fig. 5.25). It was observed that the connection stiffness in the high gravity shear specimen (E12H) is lower than the low gravity shear specimen (E12L) by 14% in the strong column direction (N-S) and 33% in the weaker column direction (E-W). These results confirmed that high gravity shear reduces the lateral stiffness of slab-column connections, especially in the weaker column direction.

5.8 Connection Ductility

Ductility of slab-column connection is an essential requirement for flat plate structures located in seismic zones as they are expected to undergo deformations into the inelastic range. The ductility of slab-column connection has been normally specified as the ratio between the drift ratio of the connection at failure to the drift ratio at first yield of slab reinforcement. However, the yield point in slabs is not well defined, and it requires some arbitrary procedures to define the yield drift DR_y . One such definition by Moehle (1989) is shown in Fig. 5.26, and is adopted in this paper. Few approaches have also been suggested by researchers in determining the point where the connection failed. It can be taken as the drift corresponding to the peak lateral load DR_u , or the drift at which the lateral load has dropped below some percentage of the peak load, i.e. 80 percent of the peak load (DR_{u80}). In order to have a clearer observation, both terms of displacement ductility factor are presented in Eqs. (5.1) and (5.2), and results are tabulated in Table 5.2.

Table 5.2 Experimental Drift Ratio and Ductility Ratio μ and μ_{80}

Specimen (1)	$DR_y(\%)$ (2)	$DR_u(\%)$ (3)	$DR_{u80}(\%)$ (4)	μ (5)	μ_{80} (6)
E1H	2.60 _(y)	3.02 _(y)	3.50 _(y)	1.15 _(y)	1.35 _(y)
E2H	2.50 _(x)	4.29 _(x)	4.60 _(x)	1.72 _(x)	1.84 _(x)
E12L	1.29 _(x) ;1.70 _(y)	2.02 _(x) ;2.51 _(y)	2.70 _(x) ;3.00 _(y)	1.57 _(x) ;1.47 _(y)	2.10 _(x) ;1.76 _(y)
E12H	1.26 _(x) ;1.68 _(y)	1.60 _(x) ;2.10 _(y)	2.40 _(x) ;2.80 _(y)	1.27 _(x) ;1.25 _(y)	1.90 _(x) ;1.67 _(y)
E0U	-	-	-	-	-

Peak displacement ductility factor μ is defined as:

$$\mu = \frac{DR_u}{DR_y} \tag{5.1}$$

Ultimate displacement ductility factor μ_{80} is defined as:

$$\mu_{80} = \frac{DR_{u80}}{DR_y} \tag{5.2}$$

(a) Strong axis (N-S)

(b) Weak axis (E-W)

Figure 5.26 Definition of DR_y , DR_u and DR_{u80}

5.8.1 Effect of Column Rectangularity

Using the calculated ductility in Table 5.2, comparison of ductility ratios for all specimens can be made. Results obtained from both uniaxial and biaxial specimens show that the ductility ratio in the strong column direction is always lower than in the weak column direction. It can be seen from column 5 of Table 5.2, that the ductility ratio for uniaxial specimen loaded in the strong column direction, i.e. Specimen E1H, is much lower ($\mu = 1.15$) than when it is loaded in the weaker column direction, i.e. Specimen E2H ($\mu = 1.72$). For biaxial specimen E12L, ductility ratio $\mu = 1.57$ was recorded in the weak column direction (x-axis), while a value of $\mu = 1.29$ is obtained in the stronger column direction (y-axis). Similar observation can also be found in Specimen E12H. It was noted that for biaxial specimens, the difference in the ductility ratios in both directions is relatively small. This may be due to biaxial actions, which limit the ductile behavior in the column weak direction to develop.

Hence, these observations suggest that column rectangularity is a significant factor in controlling the connection ductility, and for rectangular-column slab connections, low ductility ratio or brittle modes of failure can be expected to occur in the stronger column direction.

5.8.2 Effect of Biaxial Loading

From column 5 of Table 5.2, it was found that the ductility ratio for biaxial specimen E12H ($\mu = 1.25$ in N-S and $\mu = 1.27$ in E-W direction) is higher than the value for the uniaxial specimen E1H ($\mu = 1.15$ in N-S direction). One possibility is that the column rectangularity effect may be more severe than the biaxial load effect due to the high column aspect ratio of five used in this investigation.

From column 3 of Table 5.2, it was shown that the drift capacity ratio in the uniaxial specimen E1H is much higher ($DR_u = 3.02\%$) than the drift capacity ratio in biaxial specimen E12H ($DR_u = 1.60\%$ in x-direction and 2.10% in y-direction). Hence, based on this observation, the uniaxial specimen was found to be more ductile than the biaxial specimen due to its ability to sustain higher drift ratio. However, the ductility ratio for the biaxial specimen E12H shown in column 5 of Table 5.2 was found to be higher than the ductility ratio for the uniaxial specimen

E1H. This inconsistency in the calculated ductility ratio may indicate that the yield drift DR_y obtained using the procedure shown in Fig. 5.26 may not be suitable for rectangular connections. It was observed that for Specimen E1H, yielding of slab reinforcement in the column strip occurred at a lower drift ratio (at 1.5% drift ratio) than the one found using the procedure shown in Fig. 5.26. While for Specimen E2H, there was no yielding of top slab reinforcement along the direction of the applied lateral load until failure (Fig. 5.13), but a value of 2.5 % drift ratio was obtained using the procedure shown in Fig. 5.26 to assume the point where slab reinforcement yielded.

The above observation suggests that the arbitrary value chosen ($2/3 M_{max}$) to determine the point where the slab reinforcement yielded may not be applicable for rectangular connections, especially with a high column aspect ratio of five. It is noted that the procedure shown in Fig. 5.26 was based on observations of experimental results on square-column slab connections (Moehle, 1989). Since the yield point of the slab reinforcement is not well defined for the case of rectangular-column slab connections, ductility of the connection may be observed from the drift capacity ratio obtained. Ductile behavior is indicated by the ability of the connection to sustain high drift ratio in terms of either DR_u or DR_{u80} . It can be seen from columns 3 and 4 of Table 5.2 that the drift capacity ratios achieved in biaxial specimens are much lower than the drift capacity ratio obtained from uniaxial specimens, indicating low ductility for biaxial specimens. The same conclusion can also be found from the comparison of the envelopes of unbalanced moment-drift ratio plotted for both uniaxial and biaxial specimens (Fig. 5.19).

5.8.3 Effect of Gravity Shear

The effect of gravity shear on ductility can also be seen by comparing the ductility ratio obtained for Specimen E12L and E12H in Table 5.2. It was observed that the slab-column connection would attain higher ductility ratio if it carries lesser gravity shear at the connection. However, limiting the amount of gravity loading may be less effective for rectangular connections since high column aspect ratio would still lower the connection ductility in the stronger column direction causing brittle punching modes of failure to occur in this direction.

5.9 Connection Drift Capacity

5.9.1 Effect of Column Rectangularity

Normally, in the design of flat plate subjected to lateral loading, the columns are always assumed to be rigid and relatively much stiffer than the slab. Hence, the effect of column deformations is assumed to be relatively small compared to the resulted connection drift ratio. However, as design of flat plate floor system begins to incorporate rectangular columns, investigation into how the column aspect ratio would affect the connection drift capacity is needed. As demonstrated in these experimental results that a very high drift ratio can result in the weak column direction due to column flexure about its minor axis.

Fig. 5.18 showed that for a rectangular-column slab connection, high connection drift capacity ratio was found in the weaker column direction. For the same amount of moment transfer, higher drift ratio in the weaker column direction than that in the strong column direction is observed by an amount of 150 percent for uniaxial specimens (Fig. 5.18a), and 190 percent for biaxial specimens (Fig. 5.18b). This phenomenon suggests that for connections involving rectangular columns, high column aspect ratio contributes significantly to the flexibility in the weaker column direction. As a result, the slab-column connection finally behaves much more ductile and is accompanied by a relatively high drift ratio due to flexure about the weak column axis. This observation also suggests that the effect of column rectangularity in this case is significant due to high column rectangularity ratio of five.

Thus, in the design of flat plate subjected to lateral loading, it is important to be able to quantify the effect of column flexure due to column rectangularity on the drift capacity of slab-column connections. However, a complete investigation into the matter may not be feasible at the moment as experimental results on rectangular-column slab connections subjected to lateral loading are still rare, especially those with the variation in column aspect ratios.

5.9.2 Effect of Biaxial Loading

The experimental results in terms of the peak drift ratios (at peak unbalanced moments) were plotted against the gravity shear ratios as shown in Fig. 5.27. Note that for the biaxial specimens, the peak drift ratio shown in Fig. 5.27 is the smaller of the two peak drift ratios obtained in both directions. From Fig. 5.27, it can be seen that the biaxial load effect significantly reduces the connection drift capacity. The peak drift ratio of the specimen subjected to biaxial loading (specimen E12H) is only 0.5 times and 0.37 times the peak drift ratio of specimens under uniaxial loading (specimen E1H and E2H, respectively). This indicates that the effect of biaxial loading on drift ratio is more severe compared to that of uniaxial loading for connections with high column rectangularity ratio.

Effect of column rectangularity on connection drift capacity can also be assessed from Fig. 5.27. Higher peak drift ratio in Specimen E2H (weak column direction) by 1.43 times that in Specimen E1H (strong column direction) indicates that column rectangularity significantly increases the connection drift capacity in the weaker column direction.

Figure 5.27 Experimental Peak Drift Ratios vs Gravity Shear Ratios

5.9.3 Effect of Gravity Shear

The experimental results shown in Fig. 5.27 demonstrate that the amount of gravity load would affect the lateral behavior of slab-column connections. Observation can be made by comparing the drift capacity ratios obtained for Specimen E12L and E12H shown in Fig. 5.27. An increase in drift ratios of approximately 25 percent and 19 percent were observed in the column weak and strong directions, respectively when the gravity load was reduced. The result suggests that while the magnitude of gravity shear may not affect connection drift significantly as compared to the column rectangularity effect and the biaxial load effect, high gravity shear still decreases the ultimate drift ratio of the slab-column connection.

A total of 27 data for edge-column slab connections transferring cyclic unbalanced moment were collected from the literature, and resulting the peak drift ratios were plotted against the gravity shear ratios (Fig. 5.28). The summary of the slab details and failure loads are given in Table 5.3. For all test slabs included in this study, the outer faces of the columns were flush with the slab edge. All specimens had columns extended both above and below the slab. The specimens included both isolated and multi-column specimens. Only those connections without spandrel beam and without any types of shear reinforcement were used in this study. Specimens tested by the authors, and one of the specimens tested by Hawkins et. al. included rectangular columns. In addition, none of those connections were subjected to biaxial unbalanced moment, except those tested by the authors i.e. specimens E12L and E12H.

The limit on gravity load to ensure a minimum drift capacity ratio of 1.5 % was found to be $0.3 V_c$ to cater for biaxial lateral loadings as well as column rectangularity with an aspect ratio of five (Fig. 5.28). Although this limit is lower than the limit recommended by the ACI Committee 352 ($0.4 V_c$), this limit ensures the slab-column connections to achieve a minimum 1.5 % drift ratio in both directions when subjected to wind or earthquake loadings, even if the connections utilizes high column rectangularity ratios.

Table 5.3 Test Specimen Data For Edge Connections With Cyclic Moment Transfer

Researcher	Mark	h , mm	c_1 , mm	c_2 , mm	d_{ave} , mm	ρ_x , %	f'_c , MPa	f_y , GPa	V_u/V_c	DR_u
Authors	E1H	135	900	180	107	1.20	33.0	532	0.31	3.02
	E2H	135	900	180	107	1.20	32.5	528	0.31	4.29
	E12L	135	900	180	107	1.20	34.4	530	0.16	2.02
	E12H	135	900	180	107	1.20	35.4	531	0.31	1.60
	E0U	135	900	180	107	1.20	33.3	529	0.77	-
Megally and Ghali	MG-1	150	250	250	113	0.92	34.2	486	0.56	1.57
	MG-2A	150	250	250	113	1.82	31.6	519	0.58	0.63
	MG-7	150	250	250	113	1.82	31.2	519	0.29	1.89
	MG-8	150	250	250	113	1.82	34.3	478	0.42	1.70
	MG-9	150	250	250	113	1.82	82.7	456	0.36	1.78
Robertson and Durrani	A-w	115	250	250	92	0.83	33.0	500	0.32	3.50
	A-e	115	250	250	92	0.83	33.0	500	0.31	3.50
	B-w	115	250	250	92	0.83	30.7	524	0.40	1.50
	B-e	115	250	250	92	0.83	30.7	524	0.40	1.50
	C-w	115	250	250	92	0.83	32.2	524	0.46	1.00
	C-e	115	250	250	92	0.83	32.2	524	0.50	1.00
Durrani, Du and Luo	DNY_1W	115	250	250	97	0.59	35.2	372	0.20	3.10
	DNY_1E	115	250	250	97	0.59	35.2	372	0.20	3.10
	DNY_2W	115	250	250	97	0.59	25.7	372	0.30	2.00
	DNY_2E	115	250	250	97	0.59	25.7	372	0.30	2.00
	DNY_3W	115	250	250	97	0.59	24.6	372	0.24	2.00
	DNY_3E	115	250	250	97	0.59	24.6	372	0.24	2.00
	DNY_4W	115	250	250	97	0.59	19.1	372	0.28	2.60
	DNY_4E	115	250	250	97	0.59	19.1	372	0.28	2.60
Hawkins, Wong and Yang	E1	165	300	300	130	0.76	22.5	462	0.26	4.00
	E2	180	400	400	140	1.07	29.5	425	0.22	3.50
	E3	180	500	200	140	1.87	22.6	447	0.28	3.00

Note:

c_1 and c_2 are column dimensions parallel and perpendicular to the direction of moment transfer, respectively; h is the slab thickness; d_{ave} is the average slab effective depth; ρ_x and ρ_y are the slab flexural reinforcement ratio in the x and y direction, respectively; f'_c is the concrete compressive strength; f_y is the steel yield strength; V_u/V_c is the gravity shear ratio; DR_u is the peak drift ratio at peak unbalanced moment.

Figure 5.28 Gravity Load Limit for Rectangular-Column Slab Connections with Biaxial Loadings

5.10 Summary and Conclusions

Based on the experimental program, the following conclusions can be obtained:

1. The $0.4 V_c$ limit on gravity load recommended by the ACI Committee 352 for slab-column connections to achieve drift capacity of at least 1.5 % should be reduced to $0.3 V_c$ in order to cater for biaxially loaded rectangular-column slab connections with high column aspect ratio.
2. Column rectangularity significantly enhances the ability of slab-column connection to transfer more unbalanced moment about the strong column axis.
3. For rectangular-column slab connection, low ductility ratio or brittle modes of failure can be expected to occur in the strong column direction due to higher shear stresses developed along the column short side, and these shear stresses can be critical in limiting the shear capacity of the slab.
4. High column rectangularity ratio contributes to a significant flexibility about the column minor axis. As a result, a very high drift ratio can result in the weak column direction due to column flexure about its minor axis.
5. For rectangular-column slab connections, stiffness in the weak column direction is expected to be lower than the stiffness in the strong column direction, and application of high gravity shear would further reduce the connection stiffness in the weaker column direction.
6. Biaxial cyclic lateral loading was found to severely reduce the strength, stiffness, ductility and drift capacity of slab-column connections with high column rectangularity ratios.
7. Higher gravity shear reduces the connection strength, stiffness, ductility and drift capacity. However, relatively small reductions were observed even if the amount of gravity shear was doubled for rectangular-column slab connections with an aspect ratio of five.

CHAPTER 6

PROPOSED PROCEDURE FOR PREDICITING STRENGTH OF SLAB-COLUMN CONNECTIONS

CHAPTER 6

PROPOSED PROCEDURE FOR PREDICTING STRENGTH OF SLAB-COLUMN CONNECTIONS

6.1 Introduction

According to the American Concrete Institute Code, ACI 318-05, the critical shear perimeter for calculating the punching shear strength of slab-column connections is located at $0.5d$ away from the column face and is taken to conform with the column cross section or loaded area. To obtain the shear strength of slab-column connections, three equations are used (Eqs. (3.5) to (3.7)) and the lowest value determines the punching shear strength of slab-column connections. ACI 318 method treats column rectangularity by reducing the nominal shear strength, and the different types of connections (such as interior, edge and corner) are also taken into account based on different values for α_s . However, the ACI method ignores the influence of flexural reinforcement on the punching shear capacity.

In this chapter, the punching shear strength of slabs is reviewed in detail, and a general and simple formula to calculate the punching shear strength of slabs is proposed. Since the research is in essence geared to improve the ACI Code, the candidate adopts the control perimeters $d/2$ from the columns for convenience. The advantages of the proposed formula are its simplicity (only a single equation) and accuracy, while it still gives a safe design. The proposed formula considers the influence of flexural reinforcement ratio, and in the following chapter, a simple expression to estimate the shear strength degradation due to cyclic lateral loading is included in the formula. The applied shear stress calculation for slab-column connections under symmetrical punching is reviewed in this chapter, and a factor based on the column aspect ratio is introduced to consider column rectangularity.

Chapter 6: Proposed Punching Shear Strength Equation

The proposed formula is applicable to all cases of slab-column connections without shear reinforcement. In this chapter, comparisons between experimental results and predicted values from the proposed formula are made for the symmetrical punching case of slab-column connections. In the next chapter, verifications for the case with unbalanced moment transfer due to both monotonic and cyclic loadings are made, including the author's own specimens. It is shown that the proposed semi-empirical equation for predicting the shear strength of slabs is accurate and reliable.

6.2 Main Variables Affecting Punching Shear Strength

6.2.1 Concrete strength

It is well known that concrete strength plays an important role in the shear resistance of slab-column connections. Normally it is represented by the concrete tensile capacity, which is related to the standard cylinder or cube strength of concrete. In the truss model by Alexander and Simmonds (1987), it was assumed that the concrete tensile capacity is related to the square root of the concrete strength. Gonzalez-Vidoso et al (1988) used a nonlinear finite element analysis in which failure is governed by the tensile strength of the concrete, which was also assumed to be proportional to the square root of the concrete strength. Bazant and Cao (1987) used fracture mechanics to investigate size effect on punching shear strength, in which it was assumed that the shear strength was directly proportional to concrete strength. Gardner (1990) proposed an equation to calculate the punching strength, which is related to the cube root of the concrete strength.

For the design methods, normally there are two ways to consider the influence of concrete strength on punching shear capacity. The first one assumes that the punching shear strength is related to the square root of concrete strength, such as ACI Code, CSA Code, AS Code; the second one assumes that the punching shear strength is related to the cube root of concrete strength, such as BS Code and CEB-FIP Code (see Chapter 3 for details).

Fig. 6.1 shows the relationship between the nominal shear stress and the concrete strength in order to see the effect of concrete strength on punching shear strength of slabs. The nominal shear stress was calculated by dividing the

Chapter 6: Proposed Punching Shear Strength Equation

experimental shear force by the area of the critical section. Since the research is in essence geared to improve the ACI Code, the shear perimeter $d/2$ from the columns is adopted. The use of other assumed control perimeters is also possible as it only represents an assumed critical shear perimeter where the shear stresses are being calculated. Other control perimeters would change the conclusion: bigger control perimeter would mean lower v_c values while smaller control perimeter would mean higher v_c values. As the distance of the critical section from the column is increased, the length of the critical perimeter increases and for the same failure load, the shear stress is decreased.

The experimental data are collected from previous research works (Elstner and Hognestad, 1956; Kinnunen and Nylander, 1960; Moe, 1961; Taylor and Hayes, 1965; Corley and Hawkins, 1968; Roll et al, 1971; Vanderbilt et al, 1971; Hawkins, 1971; Regan, 1986; Rankin and Long, 1987; Tolf, 1988; Marzouk and Hussein, 1991; Chana and Desai, 1992a; Chana and Desai, 1992b; Regan et al., 1993; Lee and Teng, 1999; Kuang and Teng, 2001; and a collection of slab data from the *fib* bulletin No. 12 [CEB- *fib* 2001]). The details of all data collected are summarized in Appendix E. From Fig. 6.1, it can be seen clearly that the nominal shear stress increases with the increase in the concrete compressive strength. The curve in the figure is the trend line generated by regression.

Fig. 6.1 Relationship Between Nominal Shear Stress and Concrete Strength

6.2.2 Reinforcement Ratio

Some design codes already include the influence of flexural reinforcement ratio in the formula for calculating the punching shear strength of slab-column connections, such as BS8110 and CEB-FIP Code. Some researchers have also incorporated reinforcement ratio in the punching shear strength equation (Regan, 1981; Hawkins et. al., 1989; Gardner, 1990; Menetrey, 1996; Teng et. al.; 2004). In the following, some experimental results by previous researchers are shown to provide the supporting information which shows that flexural reinforcement ratio does affect the punching shear strength of slab-column connections, and therefore, it has to be taken into account in the punching shear strength equation.

Fig. 6.2(a) Relationship Between Ratio of v_u/v_c and Reinforcement Ratio in The Absence of Unbalanced Moment.

Fig. 6.2(b) Relationship Between Ratio of v_u/v_c and Reinforcement Ratio in The Presence of Unbalanced Moment.

Fig. 6.2 Relationship Between Shear Strength Ratio v_u/v_c and Reinforcement Ratio

In Fig. 6.2, the vertical axis represents the ratio of experimental shear strength (v_u) to predicted shear strength (v_c) at the critical shear perimeter by ACI method and the horizontal axis represents the flexural reinforcement ratio. The predicted shear strength (v_c) is calculated by using ACI Eqs. (3.5) to (3.7), and the experimental shear strength (v_u) is calculated by using experimental ultimate punching load divided by the area of critical section.

There are six series of curves in the graphs. The specimens selected for comparison shared the same properties, except the value of flexural reinforcement ratio used. The specimens shown in Fig. 6.2(a) are specimens under symmetrical punching cases (specimens 1A to 4A, 1B to 4B, 1C to 4C from Rankin (1987)). The specimens shown in Fig. 6.2(b) are specimens under the condition with unbalanced moment transfer (specimens 6AH, 9.6AH and 14AH; 6CH, 9.6CH and 14CH; 6DH, 14DH from Hawkins et. al. (1989)).

In Fig. 6.2, it is clearly shown that the ratio of v_u/v_c increases when the flexural reinforcement ratio increases. This suggests that when the reinforcement

Chapter 6: Proposed Punching Shear Strength Equation

ratio is high, the ACI Code method underestimates the punching strength of slab-column connections. If the code method includes the effect of flexural reinforcement ratio in the v_c equation, a higher punching strength would be predicted. Fig. 6.2 also shows that the influence of reinforcement ratio on the punching shear strength of slabs is higher for the symmetrical punching case than that when unbalanced moment is being transferred, indicated from the higher slope shown in Fig. 6.2(a) than that in Fig. 6.2(b). The presence of unbalanced moment would decrease the punching shear strength, and hence, reduces the increase of the punching shear strength due to an increase in reinforcement ratio.

Note that there are four specimens in Fig. 6.2(a) and two specimens in Fig. 6.2(b) that have unsafe predictions, i.e. the predicted strength v_c is higher than the experimental strength v_u . This may be due to the low reinforcement ratio used in those specimens and was not taken into account in the ACI equations, resulting in flexural failure prediction though the specimens was reported to fail by punching failure modes.

Fig. 6.3 shows the relationship between the nominal shear stress on $d/2$ critical shear perimeter and flexural reinforcement ratio. The same experimental data are used here as that in Fig. 6.1. It is seen clearly that the nominal shear stress also increases with an increase in the flexural reinforcement ratio. The curve in the figure is again the trend line generated by regression analysis.

Fig. 6.3 Relationship Between Nominal Shear Stress and Reinforcement Ratio

6.3 Proposed Simple Design Equation

Based on the above discussion, a simple design equation is proposed to calculate the punching shear strength of slab-column connections for flat plate structures, as shown in Eq. (6.1).

$$v_c = C(\rho)^A (f'_c)^B \tag{6.1}$$

- where:
- v_c = punching shear strength of slab-column connections;
 - ρ = average top flexural reinforcement ratio in percentage;
 - f'_c = concrete cylinder strength in MPa;
 - A, B, C = constants to fit the experimental data.

In order to find the suitable values for the three constants A , B , and C , 238 experimental data were collected from literature (Elstner and Hognestad, 1956; Kinnunen and Nylander, 1960; Moe, 1961; Taylor and Hayes, 1965; Corley and Hawkins, 1968; Roll et al, 1971; Vanderbilt et al, 1971; Hawkins, 1971; Regan, 1986; Rankin and Long, 1987; Tolf, 1988; Marzouk and Hussein, 1991; Chana and Desai, 1992a; Chana and Desai, 1992b; Regan et al., 1993; Lee and Teng, 1999; Kuang and Teng, 2001; and a collection of slab data from the *fib* bulletin No. 12 [CEB- *fib* 2001]). The details of the experimental data can be seen in Table E1 of the Appendix E. Table 6.1 lists the range of the variables in these experimental works.

Table 6.1 Range of Variables in The Data Collected For Symmetrical Punching Case of Slab-Column Connections

Variables	Min	Max
Effective depth d (mm)	35	240
Reinforcement ratio ρ (%)	0.31	3.70
Concrete cylinder strength f'_c (N/mm ²)	9.50	119.0
Column size c (mm)	50.8	500
Maximum aggregate size d_a (mm)	5.0	38.1

Chapter 6: Proposed Punching Shear Strength Equation

For research purposes, the best fit values for A and B are 0.345 and 0.402 respectively, as shown in Fig. 6.1 and 6.3. In order for Eq. (6.1) to give the average value v_u/v_c equals to 1.00, the value of C is 0.577 when A=0.345 and B=0.402, while the value of C is 0.739 when A and B are chosen to be equal to 1/3. Thus, the best fit equations that give the average value v_u/v_c equals to 1.00 can be written either in a form of Eq. (6.2) or Eq. (6.3) with its respective Coefficient of Variation (COV) as follows:

$$v_c = 0.577(\rho)^{0.345}(f'_c)^{0.402} \quad \text{with COV} = 0.129 \quad (6.2)$$

$$v_c = 0.739(\rho)^{1/3}(f'_c)^{1/3} \quad \text{with COV} = 0.135 \quad (6.3)$$

If Eq. (6.1) is set such that the average value of $v_u/v_c = 1.00$, half of the data will definitely fall below the mean (1.00) which makes the equation itself unsafe for predicting the shear capacity of slabs. It is to be noted that Eq. (6.1) was essentially proposed for design purpose that is to give the best prediction as possible without compromising the safety of the connection strength.

From regression analysis on the databank collected, it is found that when $A=1/3$ $B=1/3$ and $C=0.6$, the proposed semi-empirical equation can provide a good prediction for all 238 data, and most importantly a safe design. Thus, the final proposed simple design formula to calculate punching shear strength of slab-column connection can be written as follows:

$$v_c = 0.6(\rho)^{1/3}(f'_c)^{1/3} \quad \text{in MPa} \quad (6.4)$$

The form of Eq. (6.4) has been used by many researchers (Regan, 1981; Hawkins et. al., 1989; Gardner, 1990; Menetrey, 1996; Teng et. al., 2004) and in codes (CP 110-72, BS 8110-85, BS 8110-97, CEB-FIP MC90). Eq. (6.4) give $C = 0.6$ in order for 95% of the predictions v_u/v_c to fall above 1.00 which guarantee a safe prediction for design purposes (with 95% confidence level).

Figs. 6.4.1(a) and (b) show that the proposed power of 1/3 for both the concrete strength and the reinforcement ratio are within the acceptable range as investigated previously by Elstner and Hognestad (1956). Fig. 6.4.1(a) shows the relationship between the nominal shear stress and the concrete strength for various

reinforcement ratios based on sets of experimental data tested by Elstner and Hognestad (1956) for symmetrical punching with concrete strength as the only variable; while Fig. 6.4.1(b) shows the relationship between the nominal shear stress and the reinforcement ratio for various concrete strengths based on the same sets of experimental data tested by Elstner and Hognestad (1956) for symmetrical punching with reinforcement ratio as the only variable.

6.4 Applied Shear Stress Calculation for Rectangular Columns

The proposed Eq. (6.4) is a unified equation, which disregards the effect of rectangularity of the column on the shear stress. Therefore, it needs to be checked against the data for rectangular-column slab connections. For rectangular columns, the ultimate shear stress calculated by assuming a uniform stress distribution along the critical shear perimeter $v_u = V_u/b_o d$ may result in a lower shear stress compared to the actual shear stress that develops along the shorter side of the rectangular column. Normally, there are two ways to consider this effect. One is to reduce the nominal shear strength as done by the ACI Code. The other one is to increase the ultimate shear stress in order to take into account the stress concentration around the shorter side of the rectangular columns. Here, a factor β_R is introduced into the shear stress calculation to consider column rectangularity, as shown in Eq. (6.5).

$$v_u = \beta_R \frac{V_u}{b_o d} \quad (6.5)$$

where,

v_u = ultimate shear stress;

V_u = direct shear force;

d = slab effective depth;

b_o = length of critical shear perimeter taken at a distance of $0.5d$ away from the column face, and the shape of the critical shear perimeter has square corners regardless of the shape of the column (Fig. 6.4);

β_R = column rectangularity factor, calculated as follows:

Chapter 6: Proposed Punching Shear Strength Equation

Fig. 6.4.1(a) Relationship between nominal shear stress and concrete strength based on sets of experimental data tested by Elstner and Hognestad (1956)

Fig. 6.4.1(b) Relationship between nominal shear stress and reinforcement ratio based on sets of experimental data tested by Elstner and Hognestad (1956)

$$\beta_R = \left(\frac{b_2}{b_1} \right)^{1/4} \tag{6.6}$$

where b_2 is the length of the longer side of the critical shear perimeter, and b_1 is the length of the shorter side of the critical shear perimeter.

The proposed column rectangularity factor β_R has been verified with 15 experimental data collected from the literature (Hawkins, Fallsen and Hinojosa, 1971; Lee and Teng, 1998; Kuang and Teng, 2001). The complete information of these slab details are given in Table E2 of the Appendix E. The accuracy of the predictions calculated using Eqs. (6.4) to (6.6) in terms of the average v_u/v_c , standard deviation and coefficient of variation are shown in Table 6.2. Table 6.2 shows that the inclusion of the column rectangularity factor leads to a better prediction than that using the ACI method.

Fig. 6.4 Location of Critical Shear Perimeter (Proposed Method)

6.5 Comparison with Experimental Data

The accuracy of the proposed simple design formula in Eq. (6.4) for calculating the punching shear strength of slab-column connections has been verified with 238 experimental data collected from literature. Table 6.2 summarizes the performance of the proposed formula in predicting strength of square and rectangular-column slab connections under symmetrical punching case. Comparison on the predictions from the ACI equations for punching shear strength and the proposed single equation are given in Table 6.2. For square columns, the average shear strength ratio v_u/v_c (ratio of the experimental shear strength to the predicted shear strength) calculated using the ACI Code method is 1.410 with a coefficient of

Chapter 6: Proposed Punching Shear Strength Equation

variation (COV) of 0.197, while the proposed method gives an average shear strength ratio v_u/v_c of 1.232 with a COV of 0.137. For rectangular columns, a much improved prediction is also obtained with an average shear strength ratio v_u/v_c and a COV of 1.172 and 0.078, respectively.

It should be noted that for round columns, the critical shear perimeter is taken as having square corners (see Fig. 6.4), and hence, the cross section needs to be transferred from round to square based on the equivalent area principal. It is seen clearly from the results summarized in Table 6.2 that the proposed semi-empirical equation can provide a much better prediction than the ACI code equations.

Table 6.2 Summary of Results For Slab-Column Connections Under Symmetrical Punching

Column geometry	Equation	Min v_u/v_c	Max v_u/v_c	Average v_u/v_c	Standard deviation	Coefficient of variation
Square columns (223 specimens)	ACI 318-05	0.644	2.100	1.410	0.278	0.197
	<i>Eqs. (6.4 - 6.6)</i>	<i>0.858</i>	<i>1.691</i>	<i>1.232</i>	<i>0.169</i>	<i>0.137</i>
Rectangular columns (15 specimens)	ACI 318-05	1.033	1.570	1.267	0.171	0.135
	<i>Eqs. (6.4 - 6.6)</i>	<i>1.020</i>	<i>1.359</i>	<i>1.172</i>	<i>0.092</i>	<i>0.078</i>
Total: 238 specimens						

Fig. 6.5 shows the comparison of the predictions of the 238 connections data calculated using the ACI equations for punching shear strength and the proposed simple design equation. In Fig. 6.5(a), the horizontal axis is the punching shear strength calculated by using ACI Eqs. (3.5) to (3.7), while the proposed simple design formula shown in Eq. (6.4) is used in Fig. 6.5(b). The vertical axis is the experimental shear stress on critical shear perimeter. It can be seen from Fig. 6.5 that the punching shear strength of slab-column connection is much better predicted using the proposed method than that using the ACI method.

Fig. 6.5(a) Predicted Shear Strength by the ACI Method

Fig. 6.5(b) Predicted Shear Strength by the Proposed Method (Eqs. 6.4 to 6.6)

Fig. 6.5 Comparison Between the ACI Method and the Proposed Method

6.6 Parametric Studies

Basically only a set of data is used for the derivation of the proposed semi-empirical equation v_c , which consists of 223 square and 15 rectangular-column slab connections under symmetrical punching case. The same set of data is also used to verify the accuracy as well as the safety of the proposed design equation to ensure accurate and safe predictions. Since the same set of data is used for both the derivation and the verification of the proposed equation, parametric studies have been carried out to investigate the consistency of each parameter used in the proposed equation.

Figs 6.6(a) and (b) show that the proposed simple design equation (Eq. (6.4)) treats the effects of these parameters (f'_c and ρ) successfully since the trend lines in these two figures are nearly horizontal, i.e. consistent. Tables 6.3(a) and (b) give the variation of v_u/v_c based on each parameter used in the proposed equation, which have been divided into a small range, to examine the independence of the numbers used to develop the equation. The consistency of the resulting statistics for each parameter given in Table 6.3 indicate that the proposed semi-empirical equation is consistent throughout various range of the parameters used in the equation.

Chapter 6: Proposed Punching Shear Strength Equation

Fig. 6.6 (a) v_u / v_c versus Concrete Compressive Strength.

Table 6.3(a) Variation of v_u / v_c based on the concrete strength parameter f'_c

Concrete strength, f'_c (N/mm^2)	No. of specimens	Min v_u/v_c	Max v_u/v_c	Average v_u/v_c	Standard deviation	Coefficient of variation
$f'_c < 23$	34	0.894	1.507	1.197	0.135	0.113
$23 \leq f'_c \leq 26$	30	0.881	1.474	1.221	0.151	0.124
$26 < f'_c \leq 30$	42	0.858	1.673	1.235	0.165	0.134
$30 < f'_c < 35$	28	1.027	1.655	1.233	0.137	0.111
$35 \leq f'_c < 40$	44	0.961	1.691	1.240	0.162	0.131
$40 \leq f'_c < 70$	29	1.033	1.558	1.238	0.147	0.119
$70 \leq f'_c \leq 100$	25	0.991	1.543	1.237	0.181	0.146
$f'_c > 100$	6	1.015	1.419	1.228	0.202	0.164
Total	238					

Chapter 6: Proposed Punching Shear Strength Equation

Fig. 6.6 (b) v_u / v_c versus Flexural Reinforcement Ratio.

Table 6.3(b) Variation of v_u / v_c based on the flexural reinforcement ratio parameter ρ

Reinforcement ratio, ρ	No. of specimens	Min v_u/v_c	Max v_u/v_c	Average v_u/v_c	Standard deviation	Coefficient of variation
$\rho \leq 0.5$	14	0.858	1.289	1.181	0.136	0.115
$0.5 < \rho \leq 0.7$	27	1.015	1.476	1.227	0.120	0.098
$0.7 < \rho < 0.9$	36	0.999	1.556	1.246	0.148	0.119
$0.9 \leq \rho \leq 1.1$	36	0.991	1.673	1.235	0.172	0.140
$1.1 < \rho \leq 1.3$	41	0.961	1.691	1.234	0.165	0.133
$1.3 < \rho \leq 1.7$	33	1.020	1.487	1.232	0.140	0.113
$1.7 < \rho \leq 2.5$	34	0.894	1.543	1.238	0.196	0.158
$\rho > 2.5$	17	0.921	1.405	1.187	0.158	0.133
Total	238					

Chapter 6: Proposed Punching Shear Strength Equation

A general semi-empirical design equation to predict the punching shear strength of slab-column connections is proposed. In this section, a parametric study is conducted to show how the parameters included in the proposed equation, namely concrete strength and flexural reinforcement ratio, influence the punching strength equation. In general, the parametric studies described below are carried out by varying one parameter while all other parameters are kept constant.

6.6.1 Concrete Strength

Concrete strength is the main parameter to affect the punching strength of slab-column connections as discussed in Section 6.2. In the proposed semi-empirical equation, the power of the concrete cylinder strength is set to 1/3. The effect of setting the concrete strength to the power of 1/3 on the punching shear strength when the reinforcement ratio is fixed at 1.00 is shown in Fig. 6.7.

Fig. 6.7 shows that punching shear strength of slab-column connections increases with an increase in the concrete strength. The influence of concrete strength on the punching strength is very profound for concrete strength below 60 N/mm², and the effect becomes lesser as the concrete strength goes higher. If we change the concrete strength from 20 to 60 N/mm², the punching shear strength will increase by about 44.2%, however, if we change the concrete strength from 60 to 100 N/mm², the punching shear strength will only increase by about 18.6%.

Fig. 6.7 Relationship Between Punching Shear Strength and Concrete Strength

6.6.2 Flexural Reinforcement Ratio

The flexural reinforcement ratio is included in the proposed semi-empirical design equation for calculating the punching shear strength of slab-column connections as shown in Eq. (6.4) Fig. 6.8 shows the relationship between the predicted shear strength calculated according to the proposed Eq. (6.4) and the flexural reinforcement ratio when the concrete compressive strength f'_c is fixed at 40 N/mm².

It can be seen from Fig. 6.8 that punching shear strength of slab-column connections increases with an increase in the flexural reinforcement ratio. The rate of increase becomes slower when the reinforcement ratio is high, since the slope of the curve becomes smaller. If we change the reinforcement ratio from 2.0 to 3.0 percent, the punching shear strength will increase by about 14.5%, however, if we change the reinforcement ratio from 1.0 to 2.0 percent, the punching shear strength will increase by about 26.0%. These suggest that punching shear strength of slabs can be increased by increasing the flexural reinforcement ratio, but it is limited to a point where the strength cannot be improved much, that is if the flexural reinforcement ratio is already high.

Fig. 6.8 Relationship Between Punching Shear Strength and Reinforcement Ratio

6.7 Influence of Size Effect

The influence of specimen size or effective depth d on the punching shear strength of slabs has been studied by Kinnunen, Nylander, and Tolf (1978), Bazant and Cao (1987), and Hallgreen (1997). However, it is still unclear if size effect d has any strong effect on punching shear strength of slabs, especially for slabs of approximately 200 mm thick (Teng et al., 2004). The typical and simple way to consider size effect in slabs is through a size effect factor that is introduced into the shear equation. In this section, a size effect factor is introduced into the proposed simple design equation given earlier in Eq. 6.4, and the resulting predictions from the cases with and without the size effect term are presented.

Normally some factor is introduced into the punching shear strength equation of slab-column connections to consider the size effect. Broms (1990) introduced a factor $(1000/d)^{0.333}$ to consider the size effect. BS 8110 –1997 provides a factor equal to $(400/d)^{0.25}$ to consider the size effect, while CEB-FIP Model Code gives a factor equal to $\xi = 1 + (200/d)^{1/2}$ (see Chapter 3 for details).

The relationship between the nominal shear stress on critical shear perimeter and the effective depth of slabs is shown in Fig. 6.9. The same experimental data are used here as that in Fig. 6.1. Fig. 6.9 shows a high degree of scatter of the relationship between effective depth of slabs and punching strength of slab-column connections, as given by the regression equation shown in Fig. 6.9.

Fig. 6.9 Relationship Between Nominal Shear Stress and Slab Effective Depth

6.7.1 Proposed Size Effect Factor

Consider geometrically similar concrete structures of different characteristic sizes d , made of the same concrete. If the failure is purely brittle, i.e., if it is caused by concrete cracking, and if plastic energy dissipation during failure is negligible, then the nominal stress at failure approximately obeys the following size effect law.

$$\sigma_N = Bf_t' \phi(\lambda), \quad \phi(\lambda) = \left(1 + \frac{\lambda}{\lambda_0}\right)^{-\frac{1}{2}} \quad (6.7)$$

where,

$\sigma_N = P / bd$; P = load at failure; b = thickness of the structure;

f_t' = direct tensile strength of concrete;

$\lambda = d / d_a$ = relative structure size; d_a = maximum aggregate size;

B, λ_0 = empirical parameters characterizing the fracture energy of the material and the shape of the structures.

Eq. 6.7, which was derived by dimensional analysis and similitude arguments (Bazant, 1984), represents a gradual transition from the failure criterion of limit analysis to the linear elastic fracture mechanics (Fig. 6.10).

Fig. 6.10 Size Effect Law for Blunt Fracture

Chapter 6: Proposed Punching Shear Strength Equation

In order to consider the size effect in the punching strength of slab-column connections, a size-effect factor is proposed, following Bazant's Size-Effect Law:

$$K(d) = \left(1 + \frac{d}{B}\right)^{-0.5} \quad (6.8)$$

where, $K(d)$ = size-effect coefficient in the formula for calculating the punching shear strength of slab-column connection;

d = effective depth of slabs (mm);

B = empirical constant.

Hence, taking into account Bazant's size effect factor into the proposed simple equation given earlier in Eq. (6.4), the complete equation then becomes:

$$v_c = A(\rho)^{1/3}(f'_c)^{1/3} \left(1 + \frac{d}{B}\right)^{-1/2} \quad \text{in MPa} \quad (6.9)$$

where: v_c = punching shear strength of slab-column connections;

ρ = average flexural reinforcement ratio in percentage;

f'_c = concrete cylinder strength in MPa;

d = slab effective depth in mm;

A, B = constants to fit the experimental data.

From regression analysis on the databank collected (the same 238 experimental data collected earlier from literature in Section 6.3), it is found that when $A=0.6$ and $B=1000$, inclusion of the size effect term into the proposed equation as given in Eq. (6.10) can provide a better prediction than the one without the size effect term (Table 6.4). Thus, the complete proposed semi-empirical formula to calculate punching shear strength of slabs, which includes the size effect factor, is as follows:

$$v_c = 0.6(\rho)^{1/3}(f'_c)^{1/3} \left(1 + \frac{d}{1000}\right)^{-1/2} \quad \text{in MPa} \quad (6.10)$$

The definition of the notations is the same as before. The consistency of the proposed complete equation that include the size effect factor (Eq. 6.10) is verified

Chapter 6: Proposed Punching Shear Strength Equation

by plotting the ratios of the experimental results to the predicted values v_u/v_c versus the slab effective depth parameter, as shown in Fig. 6.11. Fig. 6.11 shows that the size effect factor that was introduced into Eq. 6.10 treats the effect of slab effective depth successfully since the trend lines in the figure is nearly horizontal.

Fig. 6.11 v_u/v_c versus Slab Effective Depth

Table 6.4 Summary of Predictions with and without Size Effect Term For Slab-Column Connections Under Symmetrical Punching

Column geometry	Equation	Min v_u/v_c	Max v_u/v_c	Average v_u/v_c	Standard deviation	Coefficient of variation
Square columns (223 specimens)	ACI 318-05	0.644	2.100	1.410	0.278	0.197
	<i>Eq. (6.10) (with size effect)</i>	0.854	1.660	1.215	0.164	0.135
	<i>Eq. (6.4) (w/o size effect)</i>	0.858	1.691	1.232	0.169	0.137
Rectangular columns (15 specimens)	ACI 318-05	1.033	1.570	1.267	0.171	0.135
	<i>Eq. (6.10) (with size effect)</i>	1.007	1.340	1.163	0.092	0.079
	<i>Eq. (6.4) (w/o size effect)</i>	1.020	1.359	1.172	0.092	0.078
Total: 238 specimens						

6.7.2 Predictions With and Without Size Effect Term

Fig. 6.12 shows the comparison of the predictions of the 238 connections data calculated with and without the size effect factor using Eqs. (6.10) and (6.4), respectively. It can be seen from Fig. 6.12 that the difference between the two cases are minimal. For simplicity, the size effect factor can be removed and the accuracy of prediction can still be satisfactory and safe, as shown in Fig. 6.12b.

Fig. 6.12(a) Predicted Shear Strength by the Complete Equation (Eq. 6.10)

Fig. 6.12(b) Predicted Shear Strength by the Simple Equation (Eq. 6.4)

Fig. 6.12 Comparison Between the Simple Equation, without Size Effect Term (Eq. 6.4) and the Complete Equation, with Size Effect Term (Eq. 6.10)

Chapter 6: Proposed Punching Shear Strength Equation

The two cases, with and without the size effect term $\left(1 + \frac{d}{1000}\right)^{-0.5}$, calculated using Eqs. (6.10) and (6.4), respectively to predict the punching shear strength of slabs are presented in Table 6.4. From Table 6.4, it is suggested that for accurate prediction of punching shear strength, the size effect factor should be included in the equation for calculating the punching shear strength of slab-column connections. However, for simplicity, the size effect factor can be removed and the accuracy of prediction can still be satisfactory. Although the prediction becomes stronger if the size effect is removed, it is still in the safe side.

For the research presented in the thesis, the final proposed simple design equation need not include the size effect factor due to its simplicity and good accuracy. Thus, the final proposed equation to calculate the punching shear strength can be written simply as given in Eq. (6.4):

$$v_c = 0.6(\rho)^{1/3}(f'_c)^{1/3} \quad \text{in MPa} \quad (6.4)$$

It is to be noted that, the nominal shear strength proposed in Eq. (6.4) has to be multiplied for design by the strength reduction factor ϕ . Hence, for a safe design, the complete proposed equations for concentric shear are as follows:

$$\phi v_c \geq v_u \quad (6.11)$$

$$0.6\phi(\rho)^{1/3}(f'_c)^{1/3} \geq \left(\frac{b_2}{b_1}\right)^{1/4} \frac{V_u}{b_o d} \quad (6.12)$$

The definition of the notations used is the same as before.

6.8 Summary

A simple semi-empirical design formula is proposed to predict the punching shear strength of slab-column connections based on a comprehensive systematic study on the databank collected. The effect of flexural reinforcement ratio is included in the proposed formula. For slab-column connections with rectangular columns, a factor is introduced into the shear stress component from the direct shear force to consider the rectangularity of the columns.

The comparison between predicted results calculated using the proposed method and the ACI Code method shows that the proposed semi-empirical method represents an improvement over the ACI Code method for the symmetrical punching case of slab-column connections, including those with rectangular columns.

The proposed simple design equation given in Eq. (6.4) gives a general and unified method to predict the punching shear strength of slab-column connections. The proposed formula has the following advantages:

- The punching shear strength is calculated by just one equation that consists of two parameters, i.e. concrete strength and flexural reinforcement ratio that significantly influence the shear strength of slab-column connections. Whereas in the ACI Code method, three equations need to be calculated and checked for the smallest value to be used to determine the strength of the connections;
- A simple factor is introduced in the semi-empirical method to consider the rectangularity of the columns;
- Besides its simplicity, the semi-empirical method improves the punching shear strength prediction of slab-column connections considerably.

It is suggested that for accurate prediction of punching shear strength, the size effect factor proposed in Eq. (6.10) should be included in the equation for calculating the punching shear strength of slab-column connections. However, for simplicity, the size effect factor can be removed and the accuracy of prediction can still be satisfactory and safe.

CHAPTER 7

PROPOSED PROCEDURE TO CALCULATE ULTIMATE SHEAR STRESS OF SLAB-COLUMN CONNECTIONS WITH UNBALANCED MOMENTS

CHAPTER 7

PROPOSED PROCEDURE TO CALCULATE ULTIMATE SHEAR STRESS OF SLAB-COLUMN CONNECTIONS WITH UNBALANCED MOMENTS

7.1 Introduction

The shear strength of slabs v_c in the ACI 318-05 is expressed by three equations. However, these equations do not consider the influence of slab flexural reinforcement, and hence, underestimate the shear strength when the slab flexural reinforcement ratio is high. Possible reduction in shear strength due to cyclic loading had also not been taken into account. Although these equations can give reasonable prediction, it can be improved further. The use of three equations for v_c can be simplified by using only one equation that also improves the strength prediction considerably, as proposed in the previous chapter. This chapter extends the applicability of the proposed simple equation (Eq. (6.4)) to predict the shear strength of slab-column connections with unbalanced moment transfer.

The ACI 318-05 presents a method known as the eccentric shear stress model to calculate the ultimate shear stress v_u when both shear force and unbalanced moments are transferred. The eccentric shear stress model involves the calculation of the property of the critical section analogous to the polar moment of inertia J_c , in which the value is different for interior, edge, and corner connections. Ghali (1999) also pointed out that J_c is difficult to calculate for circular columns. The relatively long computations involving J_c and the assumed fraction of unbalanced moment transferred by shear γ_v are neither verified nor justified by the high inaccuracies and the scattered plot of its predictions. Based on the above considerations, a new method

and a new equation is derived and verified by experiments to simplify calculations for the ultimate shear stress v_u .

A new method is presented for calculating the slab ultimate shear stress at the critical shear perimeter of interior and exterior slab-column connections due to combined gravity load and unbalanced moment transfer. The method is based on an interaction equation for shear and unbalanced moments, assuming that moment transfer capacity is limited solely by the available flexural reinforcement within an effective transfer width of $c_2 + 1.5h$ on each side of the column. Column rectangularity, connection geometry, influence of reinforcement ratio and gravity shear ratio are taken into consideration. The proposed equation has been verified for accuracy by predicting the shear strength of 217 slab-column connections including interior, edge, and corner connections, ranging from square-column slab connections transferring uniaxial unbalanced moment to rectangular-column slab connections transferring biaxial unbalanced moments due to cyclic lateral loading. The proposed formula gives a very good agreement between the experimental and predicted values, and it also simplifies the design and analysis process.

This chapter begins with a summary of data obtained from numerous experiments on interior, edge and corner slab-column connections transferring shear and unbalanced moments. The strength analysis model embodied in the current ACI Code is described and compared with experimental data. Discussions on the interaction between shear and unbalanced moment are emphasized, and a new method is proposed for the interaction between the unbalanced moment and shear in slab-column connections. Finally, a new equation is derived and verified by experiments to calculate the slab ultimate shear stress at the critical shear perimeter.

7.2 Review of Experimental Data

Numerous experimental data on slab-column connections were collected from the literature. Included in this study are 14 slab-column connections tested by the author and colleagues in Nanyang Technological University, and they were subjected to a combination of gravity and monotonic unbalanced moment transfer (Suwita, Irawan and Teng, 2001), and gravity and cyclic unbalanced moment transfer (Tan and Teng, 2005; Anggadajaja and Teng, 2006; Widjaja and Teng, 2006).

Chapter 7: Proposed Ultimate Shear Stress Equation

A total of 217 slab-column connections subjected to a combination of gravity load and unbalanced moment transfer, tested by more than 30 research centers around the world, were collected and reviewed. Only those specimens that failed by punching shear failures were included here in order to study the shear strength of slab-column connections. All connections were without closed ties, stirrups or any form of shear reinforcement. Of the 217 slab-column connection specimens, 112 are interior connections, 67 are edge connections and 38 are corner connections. Also, included are 63 interior and exterior slab-column connections subjected to cyclic moment transfer simulating earthquake effects.

The slab details of 112 interior slab-column connections with unbalanced moment transfer are listed in Tables E3 and E4 of the Appendix E for square and rectangular columns, respectively. The test specimens included isolated single-column slab specimens (Suwita, Irawan and Teng, 2001; Tan and Teng, 2005; Moe, 1961; Stamenkovic and Chapman, 1974; Hawkins, Mitchell and Sheu, 1974; Symmonds et. al., 1976; Elgabry and Ghali, 1987; Hawkins, Bao and Yamazaki, 1989; Hanson and Hanson, 1968; Ghali, Elmasri and Dilger, 1976; Islam and Park, 1976; Zee and Moehle, 1984; Pan and Moehle, 1992; Pan and Moehle, 1989; Farhey et. al., 1993; Luo and Durrani, 1995; Robertson et. al., 2002; Dilger and Cao, 1991; Dilger and Brown, 1994; Wey and Durrani, 1992; Emam et. al., 1997) and multi-column slab specimens (Robertson and Durrani, 1990; Robertson and Durrani, 1992; Durrani et. al., 1995; Sherif and Dilger, 2000). The loading applied included both static and cyclic loading. Most of the specimens tested by Stamenkovic and Chapman (1974), Hawkins, Bao and Yamazaki (1989), and Hanson and Hanson (1968) had square columns, while all the specimens tested by Suwita, Irawan and Teng (2001), Tan and Teng (2005) and Farhey et. al. (1993) had rectangular columns. All of the collected interior specimens were subjected to gravity and uniaxial unbalanced moment, except the specimens tested by Suwita, Irawan and Teng (2001), Tan and Teng (2005), and Pan and Moehle (1989, 1992) which were subjected to biaxial unbalanced moments about the two perpendicular axes.

Chapter 7: Proposed Ultimate Shear Stress Equation

Details of 67 edge connection data are summarized in Tables E5 and E6 of the Appendix E. Only the connections without spandrel beam were used in this study. Both square and rectangular-column slab connections have been tested by researchers, and the test specimen was either isolated single-column slab specimens (Anggadajaja and Teng, 2006; Stamenkovic and Chapman, 1974; Hanson and Hanson, 1968; Zaghlool, 1971; Hawkins, Wong and Yang, 1978; Kane, 1978; Mortin and Ghali, 1991; Lim and Rangan, 1995; Gardner and Shao, 1996; El-Salakawy et. al., 1999; Gilbert and Glass, 1987; Megally and Ghali, 2000; Ritchie and Ghali, 2005) or multi-column slab specimens (Sherif and Dilger, 2000; Scavuzzo, 1978; Regan, 1981; Hall and Rangan, 1983; Rangan, 1990; Robertson and Durrani, 1991; Falamaki and Loo, 1992). The collected data included application of cyclic lateral loading to simulate earthquake effects (Anggadajaja and Teng, 2006; Hawkins, Wong and Yang, 1978; Megally and Ghali, 2000; Ritchie and Ghali, 2005; Robertson and Durrani, 1991). Only the specimens tested by Anggadajaja and Teng (2006) were subjected to biaxial unbalanced moments about the two perpendicular axes, while all other specimens were subjected only to uniaxial unbalanced moment about an axis parallel to the slab free edge.

A total of 38 data were collected from the literature for corner connections, and the slab details are listed in Tables E7 and E8 of the Appendix E for square and rectangular columns, respectively. All the collected corner specimens had square columns, except the specimens tested by Widjaja and Teng (2006) which had rectangular columns. The specimens included isolated single column specimens (Widjaja and Teng, 2006; Zaghlool et. al., 1973; Ingvarsson, 1974; Hammill and Ghali, 1994; Desayi and Seshadri, 1997) and multi column specimens in a form of a corner bay of flat plate floors supported on four corner columns (Falamaki and Loo, 1992; Zaghlool et. al., 1970; Walker and Regan, 1987). The biaxial unbalanced moments are transferred from the slab to the column due to either the application of gravity load only (Falamaki and Loo, 1992; Zaghlool et. al., 1970; Ingvarsson, 1974; Desayi and Seshadri, 1997; Zaghlool et. al., 1970; Walker and Regan, 1987), or gravity and monotonic lateral loads (Hammill and Ghali, 1994), or gravity with cyclic lateral loads applied at the top of the column (Widjaja and Teng, 2006).

7.3 Punching Shear Strength Calculations by ACI Code Method

Strengths for various combinations of unbalanced moment and shear at failure for slab-column connections can be represented in a form of an interaction diagram of (M_u/M_o) versus (V_u/V_o) , known as the unbalanced moment-and-shear interaction diagram. Essentially, the resisting unbalanced moment is normalized to the moment strength measured in the absence of shear, and the resisting shear is normalized to the shear strength measured in the absence of unbalanced moment. According to the ACI 318-05, the slab ultimate shear stress v_u is given by Eq. (3.17) while the slab shear capacity v_c is the lowest value given from Eqs. (3.5) to (3.7):

$$v_u = \frac{V_u}{b_o d} + \frac{\gamma_v M_u c}{J_c} \quad (3.17)$$

$$v_c = \left(1 + \frac{2}{\beta}\right) \frac{\sqrt{f'_c}}{6} \quad (\text{MPa}) \quad (3.5)$$

$$v_c = \left(2 + \frac{\alpha_s d}{b_o}\right) \frac{\sqrt{f'_c}}{12} \quad (\text{MPa}) \quad (3.6)$$

$$v_c = \frac{1}{3} \sqrt{f'_c} \quad (\text{MPa}) \quad (3.7)$$

Dividing Eq. (3.17) by v_c gives:

$$\frac{v_u}{v_c} = \frac{V_u}{v_c b_o d} + \frac{\gamma_v M_u c}{v_c J_c} \quad (7.1)$$

Simplifying Eq. (7.1) into Eq. (7.2), where V_o (shear force capacity in absence of unbalanced moment) and M_o (unbalanced moment capacity in absence of shear force) are defined according to Eqs. (7.3) and (7.4), respectively gives the basic form of the ACI unbalanced moment-and-shear interaction equation:

$$\frac{v_u}{v_c} = \frac{V_u}{V_o} + \frac{M_u}{M_o} \quad (7.2)$$

$$\text{where: } V_o = v_c b_o d \quad (7.3)$$

$$M_o = \frac{v_c J_c}{\gamma_v c} \quad (7.4)$$

The definition of the notations is the same as defined in Chapter 3 for ACI Code.

Chapter 7: Proposed Ultimate Shear Stress Equation

-
- When the slab ultimate shear stress reaches the slab shear capacity $v_u = v_c$ or a straight line of $\frac{v_u}{v_c} = 1$ in Fig. 7.1a, then Eq. (7.2) can be written as Eq. (7.5) which represents a sloping interaction line in the ACI unbalanced moment-and-shear interaction diagram shown in Fig. 7.2a:

$$\frac{V_u}{V_o} + \frac{M_u}{M_o} = 1 \quad (7.5)$$

- When the slab ultimate shear stress exceeds the slab shear capacity $v_u > v_c$ or the area of $\frac{v_u}{v_c} > 1$ in Fig. 7.1b, then Eq. (7.2) can be written as Eq. (7.6) which represents the area above the sloping line in the ACI unbalanced moment-and-shear interaction diagram shown in Fig. 7.2b:

$$\frac{V_u}{V_o} + \frac{M_u}{M_o} > 1 \quad (7.6)$$

- When the slab ultimate shear stress does not exceed the slab shear capacity $v_u < v_c$ or the area of $\frac{v_u}{v_c} < 1$ in Fig. 7.1c, then Eq. (7.2) can be written as Eq. (7.7), which represents the area below the sloping line in the ACI unbalanced moment-and-shear interaction diagram shown in Fig. 7.2c:

$$\frac{V_u}{V_o} + \frac{M_u}{M_o} < 1 \quad (7.7)$$

Chapter 7: Proposed Ultimate Shear Stress Equation

Fig. 7.1a ACI Shear Strength Ratio Diagram ($v_u = v_c$)

Fig. 7.1b ACI Shear Strength Ratio Diagram ($v_u > v_c$)

Fig. 7.1c ACI Shear Strength Ratio Diagram ($v_u < v_c$)

Fig. 7.1 Shear Strength Ratio Based on ACI Method

Fig. 7.2a ACI Unbalanced Moment-and-Shear Interaction Diagram ($v_u = v_c$)

Fig. 7.2b ACI Unbalanced Moment-and-Shear Interaction Diagram ($v_u > v_c$)

Fig. 7.2c ACI Unbalanced Moment-and-Shear Interaction Diagram ($v_u < v_c$)

Fig. 7.2 Unbalanced Moment-and-Shear Interaction Diagram Based on ACI Method

Chapter 7: Proposed Ultimate Shear Stress Equation

A total of 112 interior, 67 edge, and 38 corner slab-column connections subjected to shear and unbalanced moment transfer were collected from the literature. Table 7.1 lists the range of the variables in these experimental works, including the normal-strength concrete and high-strength concrete. Figure 7.3 gives the shear strength ratio v_u/v_c calculated according to ACI equations (Eqs. 3.5-3.7 and Eqs. 3.17-3.24, respectively), while Figure 7.4 essentially shows the interaction between unbalanced moment and shear at failure for interior, edge and corner connections calculated based on the same ACI equations given for v_u and v_c (Eqs. 7.2-7.4).

As shown in Fig. 7.3, the ultimate shear stress predictions v_u by the ACI method are mostly conservative while some predictions for the interior connections are not safe. Similarly, the scattered plots shown in Figs. 7.4 essentially indicate that the ACI unbalanced moment-and-shear interaction needs further refinement to increase their accuracy while some predictions for the interior connections are not safe (shown by experimental results that falls below the solid lines). The relatively long computations involving J_c in which the value is different for interior, edge, and corner connections and the assumed fraction of unbalanced moment transferred by shear γ_v are neither verified nor justified by the high inaccuracies and the scattered plot of its predictions shown in Figs. 7.3 and 7.4. This indicates that an improvement to the ACI equations is in order. Based on the above considerations, a new method is derived and verified by experiments to give an accurate prediction and to simplify calculations for the ultimate shear stress with unbalanced moment transfer v_u .

Table 7.1 Range of Variables in The Data Collected For Slab-Column Connections, With Unbalanced Moment Transfer

Variables	Min	Max
Effective depth d (mm)	38.0	140.0
Reinforcement ratio ρ (%)	0.28	2.30
Concrete cylinder strength f'_c (N/mm ²)	15.0	84.1
Column size c (mm)	68	900
Critical shear perimeter b_o (mm)	500	2648

Chapter 7: Proposed Ultimate Shear Stress Equation

Figure 7.3 Shear Strength Ratio v_u/v_c Calculated According to ACI 318-05

Chapter 7: Proposed Ultimate Shear Stress Equation

Figure 7.4 Predicted Unbalanced Moment-and-Shear Interaction Based on ACI 318-05

7.4 Proposed Unbalanced Moment-and-Shear Interaction

The ACI equation for moment transfer strength or moment capacity in absence of shear force M_o given in Eq. (7.4) essentially defines the moment transfer strength as a function of connection geometry J_c and c , shear strength v_c , and the fraction of unbalanced moment transferred by shear γ_v . Note that the ACI equation for moment transfer strength M_o , used to normalize the unbalanced moment M_u , ignores the influence of flexural reinforcement. Moehle (1988) defines the moment transfer strength to be used in the unbalanced moment-and-shear interaction as the flexural capacity of slab reinforcement within a prescribed transfer width.

A linear interaction of shear and unbalanced moment is also used initially by the author, assuming that the moment transfer strength (denoted as M_f in the proposed method) is limited solely by the available flexural reinforcement within an effective transfer width of $c_2 + 1.5h$ on each side of the column. This effective transfer width of $c_2 + 1.5h$ is adopted since it has been used by the ACI Code to define the extent of yielding of slab flexural reinforcement to resist the unbalanced moment transferred by flexure. Thus, it is proposed that the resisting unbalanced moment M_u is normalized to its moment strength M_f measured in the absence of shear, and the resisting shear V_u is normalized to its shear strength V_o measured in the absence of unbalanced moment. The shear strength V_o is to be calculated by using the proposed simple equation for v_c given in Eq. (6.4).

Therefore, the basic format of the proposed unbalanced moment-and-shear interaction, applicable for both interior and exterior slab-column connections is given as follows:

$$\frac{V_u}{V_o} + \left(\frac{M_u}{M_f} \right)^n = 1 \quad (7.8)$$

where, V_u and M_u are the ultimate shear force and unbalanced moment at failure, respectively; V_o is the shear capacity calculated based on Eq. (6.4); M_f is the moment transfer capacity calculated as the flexural capacity of slab reinforcement within the prescribed transfer width of $c_2 + 1.5h$ on each side of the column.

Based on a statistical analysis involving 217 slab-column connections data with unbalanced moment transfer collected from the literature, the unbalanced moment-and-shear interaction in Eq. (7.8) can be further improved significantly by raising the unbalanced moment ratio M_u/M_f to the power of 1/3, and the results are given in Fig. 7.5(b). Fig. 7.5 shows the comparison of the unbalanced moment-and-shear interaction calculated based on the ACI method and the proposed method.

Fig. 7.5(a) Unbalanced Moment-and-Shear Interaction by the ACI Method

Fig. 7.5(b) Unbalanced Moment-and-Shear Interaction by the Proposed Method (Eq. 6.4 and Eq. 7.8)

Fig. 7.5 Comparison Between the ACI method and the Proposed Method

Chapter 7: Proposed Ultimate Shear Stress Equation

It can be seen from Fig. 7.5 that the proposed method gives a better degree of interaction between the unbalanced moment and shear than that using the ACI method. The slope of the interaction diagram in Fig. 7.5(b) is more profound than that in Fig. 7.5(a). The sloping trend of the plotted data in the proposed interaction diagram shown in Fig. 7.5(b) signifies the degree of interaction between the gravity shear ratio and the unbalanced moment ratio. This degree of interaction between the unbalanced moment and shear ratios, or the slope in the interaction diagram basically depends on various factors, such as material properties, type of connections (interior, edge or corner), size and geometry of the slab-column connections, applied gravity shear, nature of loading, etc. Hence, in order to take into account for these influencing factors, a dimensionless parameter K is then introduced to relate these factors to the unbalanced moment-and-shear ratios. Therefore, the proposed unbalanced moment-and-shear interaction becomes:

$$\frac{V_u}{V_o} + K \left(\frac{M_u}{M_f} \right)^{1/3} = 1 \quad (7.9)$$

where:

K = a dimensionless parameter that relates shear and unbalanced moment ratio, or the slope in the proposed unbalanced moment-and-shear interaction diagram;

V_u/V_o = gravity shear ratio; that is, the ratio of the ultimate shear force to the shear capacity based on Eq. (6.4);

M_u/M_f = unbalanced moment ratio; that is, the ratio of the ultimate unbalanced moment to the moment transfer capacity calculated as the flexural capacity of slab reinforcement within the prescribed transfer width of $c_2 + 1.5h$ on each side of the column.

7.5 The Parameter K

The parameter K that relates shear and unbalanced moment ratio is very much dependent on the material properties and geometry of the connections. These factors include concrete cylinder strength f'_c , flexural reinforcement ratio in relevant direction ρ , slab effective depth d , and slab critical shear perimeter b_o .

7.5.1 Effect of Material Properties

The influences of material properties such as concrete cylinder strength and flexural reinforcement ratio on the parameter K are shown in Fig. 7.6 and 7.7, respectively. The parameter K is calculated by using the proposed unbalanced moment-and-shear interaction equation given in Eq. (7.9). The specimens selected for comparison shared the same properties for each series, except the concrete cylinder strength for specimens in Fig. 7.6 and the slab flexural reinforcement ratio for specimens in Fig. 7.7.

The interior slab-column connection specimens (specimens HHHC0.5, NHHC0.5, HHHC1.0 and NHHC1.0) tested by Mohammed et. al. (1997) and the edge connection specimens (specimens MG-2A and MG-9) tested by Megally and Ghali (2000) are plotted in Fig. 7.6 to show the effect of concrete cylinder strength on the parameter K . Fig. 7.6 shows that increasing the concrete strength would increase the parameter K , and hence, increases the degree of interaction between the unbalanced moment and shear in the proposed interaction diagram.

Fig. 7.7 shows the influence of flexural reinforcement ratio on the parameter K . For this purpose, three sets of specimens with different value of reinforcement ratio were collected, and they include interior connections (specimens SM0.5, SM1.0 and SM1.5 from Ghali et. al. (1976)), edge connections (specimens MG-1 and MG-2A from Megally and Ghali (2000)), and corner connections (specimens SC1, SC2 and SC3 from Walker and Regan (1987)). It can be seen from Fig. 7.7 that increasing the slab flexural reinforcement would increase the parameter K , or the slope in the unbalanced moment-and-shear interaction diagram. It is noted that the effect of flexural reinforcement ratio on the parameter K is higher than that of the concrete cylinder strength, indicated by higher slope in Fig. 7.7 than the slope shown in Fig. 7.6.

Chapter 7: Proposed Ultimate Shear Stress Equation

Fig. 7.6 Relationship Between The Parameter K and Concrete Cylinder Strength f_c

Fig. 7.7 Relationship Between The Parameter K and Flexural Reinforcement Ratio ρ

7.5.2 Effect of Gravity Shear

Further investigation on sets of specimens from the literature with different levels of gravity loads showed that the parameter K also depends on the gravity shear ratio V_u/V_o . Some experimental studies have been done by researchers (Tan and Teng, 2005; Anggadajaja and Teng, 2006; Widjaja and Teng, 2006; Pan and Moehle, 1989; Pan and Moehle, 1992; Megally and Ghali, 2000; Robertson and Durrani, 1992; Moe, 1961; Stamenkovic and Chapman, 1974; Hawkins, Mitchell and Sheu, 1974; Hanson and Hanson, 1968) to investigate the effect of gravity shear on the punching shear strength of slab-column connections. These experimental studies were reviewed and used to establish the relationship between the gravity shear ratio and the degree of interaction K . In the following, some experimental results by previous researchers are shown to provide the supporting information which shows that gravity shear ratio V_u/V_o does affect the degree of interaction between shear and unbalanced moment K , and therefore, it has to be taken into account in the equation for calculating the ultimate shear stress.

In Fig. 7.8, the horizontal axis represents the gravity shear ratio V_u/V_o and the vertical axis represents the parameter K . The shear capacity V_o is calculated by using the proposed simple equation Eq. (6.4), and the parameter K is calculated by using the proposed unbalanced moment-and-shear interaction equation Eq. (7.9). There are three sets of specimens in the graphs. The specimens selected for comparison shared the same properties for each series, except the value of gravity shear ratio specified. The specimens shown in Fig. 7.8 are isolated interior specimens (specimens M6, M7, and M8 from Moe (1961)), isolated exterior specimens (specimens E12L, E12H and E0U tested by the author and reported in this thesis), and multi-panel specimens (specimens A, B and C from Robertson and Durrani, 1992).

Fig. 7.8 shows that an increase in the gravity shear ratio would reduce the degree of interaction between the unbalanced moment and shear, or the parameter K . This suggests that for slab-column connections transferring high gravity shear, the unbalanced moment ratio or the ratio of the ultimate unbalanced moment to the moment transfer capacity will be less. Note that the influence of gravity shear ratio V_u/V_o on K has indirectly included the influences of f'_c and ρ in the equation for V_o since f'_c and ρ are used to calculate the shear strength v_c in Eq.(6.4).

Fig. 7.8 Relationship Between The Parameter K and Gravity Shear Ratio V_u/V_o

7.5.3 Effect of Connection Geometry

The proposed unbalanced moment-and-shear interaction shown in Fig. 7.5(b) and given by Eq. (7.9) essentially ignores the types of slab-column connections used (interior, edge and corner connections). A better prediction in terms of accuracy and consistency for all types of connections can be achieved if the relationship between the effect of connection geometry on the parameter K is established. One of the ways to consider the effect of connection geometry is by taking into account the ratio of the slab effective depth over the slab critical shear perimeter d/b_o . It was observed that for a constant slab effective depth, the ratio of d/b_o increases from interior to exterior connections with the highest value attained for corner connections, since the critical shear perimeter is lesser in exterior than that in interior connections.

Fig. 7.9 shows the relationship between the parameter K and the connection geometry d/b_o . A series of test specimens (specimens SC1-SC7) done by Walker and Regan (1987) is used to illustrate the relationship, as shown in Fig. 7.9. The curve in the figure is the trend line generated by regression. It can be seen from Fig. 7.9 that as the ratio of d/b_o is increased, the parameter K also increases. In other words, the degree of interaction between the unbalanced moment and shear is higher when either the slab effective depth increases or the critical shear perimeter reduces.

Chapter 7: Proposed Ultimate Shear Stress Equation

Fig. 7.9 Relationship Between The Parameter K and Connection Geometry d/b_o

Based on a statistical analysis involving 217 slab-column connections with unbalanced moment transfer, a semi-empirical equation for K that relates shear and unbalanced moment ratio is proposed here. The effects of gravity shear ratio and connection geometry are included as follows:

$$K = \left(1 - 0.7 \frac{V_u}{V_o} \right) \left(\frac{d}{b_o} \right)^{0.1} \tag{7.10}$$

where:

- V_u/V_o = gravity shear ratio;
- d = slab effective depth;
- b_o = slab critical shear perimeter.

7.6 Punching Shear Strength Calculations by Proposed Method

7.6.1 Proposed Equation to Calculate Ultimate Shear Stress

The proposed unbalanced moment-and-shear interaction equation as shown in Eq. (7.9) can also be written in a form of an equation to calculate the ultimate shear stress at the slab critical shear perimeter. Derivation of the proposed equation for ultimate shear stress is as follows:

$$1 = \frac{V_u}{V_o} + K \left(\frac{M_u}{M_f} \right)^{1/3} \quad (7.9)$$

Assuming that shear failure occurs when the ultimate shear stress reaches the nominal shear strength, $v_u = v_c$, then Eq. (7.9) can be written as:

$$\frac{v_u}{v_c} = \frac{V_u}{V_o} + K \left(\frac{M_u}{M_f} \right)^{1/3} \quad (7.11)$$

Multiplying Eq. (7.11) by v_c , resulted in an equation to calculate the slab ultimate shear stress based on the proposed unbalanced moment-and-shear interaction as shown below:

$$v_u = \frac{V_u}{b_o d} + K \left(\frac{M_u}{M_f} \right)^{1/3} v_c \quad (7.12)$$

As discussed earlier in Chapter 6, a factor β_R needs to be introduced into the shear stress calculation to consider column rectangularity; and thus, the complete equation to calculate the ultimate shear stress of slab-column connections becomes:

$$v_u = \beta_R \frac{V_u}{b_o d} + K \left(\frac{M_u}{M_f} \right)^{1/3} v_c \quad (7.13)$$

where: V_u , M_u = ultimate shear force and unbalanced moment at failure,
respectively;

M_f = the flexural capacity of slab reinforcement within the prescribed transfer width of $c_2 + 1.5h$ on each side of the column.

b_o , d = slab critical shear perimeter and effective depth, respectively;

v_c = shear strength of slabs, calculated using Eq.(6.4);

K = a dimensionless parameter that relates shear and unbalanced moment ratio, calculated according to Eq. (7.10);

$$K = \left(1 - 0.7 \frac{V_u}{V_o}\right) \left(\frac{d}{b_o}\right)^{0.1} \quad (7.10)$$

β_R = column rectangularity factor, calculated as:

$$\beta_R = \left(\frac{b_2}{b_1}\right)^{1/4} \quad (6.6)$$

where b_2 is the length of the longer side of the critical shear perimeter, and b_1 is the length of the shorter side of the critical shear perimeter.

The proposed equation for calculating the ultimate shear stress shown in Eq.(7.13) is a general equation applicable for interior, edge and corner slab-column connections. For connections with biaxial moment transfer, Eq. (7.13) should be applied independently for the shear and moment about x and y axes of the column, and the worst case (highest v_{ux} or v_{uy}) controls the design.

7.6.2 Comparison with experimental data

The proposed equation for K shown in Eq. (7.10) has been verified by using 217 experimental connections with unbalanced moment transfer, including interior, edge, and corner slab-column connections. The data include various ranges of material properties, geometry, as well as gravity shear ratio, as given in Tables A3 to A8 of the Appendix E. It can be seen from Fig. 7.10 that a much improved interactions between unbalanced moment-and-shear ratios were obtained for all interior, edge and corner slab-column connections.

The accuracy of the proposed formulas in Eq. (6.4) for predicting the punching shear strength and Eq. (7.13) for calculating the ultimate shear stress involving unbalanced moments had been verified against 217 slab-column connections data with moment transfer collected from the literature. Table 7.2 and Fig.7.11 summarize the performance of the proposed formulas in predicting the strength of interior, edge, and corner slab-column connections. Comparisons with the current ACI Code equations are also shown in Table 7.2. It can be seen from Table 7.2 that the mean values of the measured over predicted strengths v_u/v_c , standard deviations and coefficients of variations from the proposed formulas are significantly better than those calculated based on the ACI equations.

Chapter 7: Proposed Ultimate Shear Stress Equation

Table 7.2 Summary of Results For Slab-Column Connections with Moment Transfer

Connections	Equation	Min v_u/v_c	Max v_u/v_c	Average v_u/v_c	Standard deviation	Coefficient of variation
Interior – sq column (92 specimens)	ACI 318-05	0.724	2.011	1.202	0.257	0.214
	<i>Proposed Eqs. (6.4), (7.13)</i>	<i>0.935</i>	<i>1.269</i>	<i>1.090</i>	<i>0.067</i>	<i>0.062</i>
Interior – rec column (20 specimens)	ACI 318-05	1.040	2.544	1.665	0.471	0.283
	<i>Proposed Eqs. (6.4), (7.13)</i>	<i>0.949</i>	<i>1.494</i>	<i>1.181</i>	<i>0.119</i>	<i>0.101</i>
Edge – sq column (46 specimens)	ACI 318-05	0.807	2.527	1.413	0.382	0.271
	<i>Proposed Eqs. (6.4), (7.13)</i>	<i>0.907</i>	<i>1.393</i>	<i>1.116</i>	<i>0.107</i>	<i>0.096</i>
Edge – rec column (21 specimens)	ACI 318-05	1.235	2.149	1.740	0.291	0.167
	<i>Proposed Eqs. (6.4), (7.13)</i>	<i>0.915</i>	<i>1.381</i>	<i>1.196</i>	<i>0.123</i>	<i>0.103</i>
Corner – sq column (35 specimens)	ACI 318-05	1.061	6.490	2.198	1.053	0.479
	<i>Proposed Eqs. (6.4), (7.13)</i>	<i>1.041</i>	<i>1.491</i>	<i>1.202</i>	<i>0.109</i>	<i>0.091</i>
Corner – rec column (3 specimens)	ACI 318-05	2.023	2.292	2.173	0.138	0.063
	<i>Proposed Eqs. (6.4), (7.13)</i>	<i>1.291</i>	<i>1.411</i>	<i>1.332</i>	<i>0.068</i>	<i>0.051</i>
Total: 217 specimens						

Chapter 7: Proposed Ultimate Shear Stress Equation

Fig. 7.10 Predicted Unbalanced Moment-and-Shear Interaction Based on Proposed Method (Eqs. 6.4 and 7.13)

Chapter 7: Proposed Ultimate Shear Stress Equation

Figure 7.11 The Shear Strength Ratios v_u/v_c Calculated According to Proposed Method (Eqs. 6.4 and 7.13)

7.7 Parametric Studies

A general equation for calculating the ultimate shear stress of slab-column connections at the critical shear perimeter is proposed. In this section, a parametric study is conducted to show the consistency of each parameter considered in the proposed equation (Eq. 7.13). In general, the parametric studies described below are carried out by plotting the ratios of the experimental results to the predicted values v_u/v_c versus the parameters considered in the equation, i.e. gravity shear ratio V_u/V_o , connection geometry d/b_o , concrete strength f'_c and flexural reinforcement ratio ρ , as shown in Fig. 7.12. Figs 7.12(a) to (f) essentially show that the proposed general equation (Eq. 7.13) treats the effects of these parameters successfully since the trend lines in all the figures are nearly horizontal.

Tables 7.3(a) to (f) give the variation of v_u/v_c based on each parameter used in the proposed equation, which have been divided into a small range, to examine the independence of the numbers used to develop the equation. The consistency of the resulting statistics for each parameter given in Tables 7.3 (a) to (f) indicate that the proposed general equation (Eq. 7.13) is consistent throughout various range of the parameters used in the equation.

Fig. 7.12(a) v_u/v_c versus Concrete Strength f_c

Table 7.3(a) Variation of v_u/v_c based on the concrete strength parameter f_c

Concrete strength, f_c (N/mm^2)	No. of specimens	Min v_u/v_c	Max v_u/v_c	Average v_u/v_c	Standard deviation	Coefficient of variation
$f_c < 20$	7	0.996	1.292	1.141	0.122	0.107
$20 \leq f_c \leq 25$	25	0.965	1.470	1.134	0.138	0.122
$25 < f_c \leq 28$	28	1.006	1.320	1.141	0.086	0.076
$28 < f_c < 31$	31	1.011	1.381	1.129	0.105	0.093
$31 \leq f_c < 33$	31	0.972	1.393	1.141	0.111	0.097
$33 \leq f_c < 35$	33	0.976	1.329	1.163	0.082	0.071
$35 \leq f_c \leq 40$	30	1.012	1.340	1.181	0.103	0.087
$40 < f_c < 60$	28	1.026	1.411	1.139	0.088	0.077
$f_c > 70$	3	1.002	1.269	1.153	0.137	0.119
Total	217

Fig. 7.12(b) v_u / v_c versus Flexural Reinforcement Ratio ρ

Table 7.3(b) Variation of v_u / v_c based on the flexural reinforcement ratio parameter ρ

Reinforcement ratio, ρ	No. of specimens	Min v_u/v_c	Max v_u/v_c	Average v_u/v_c	Standard deviation	Coefficient of variation
$\rho \leq 0.5$	14	1.186	1.400	1.240	0.056	0.045
$0.5 < \rho \leq 0.7$	35	0.965	1.381	1.157	0.097	0.084
$0.7 < \rho < 0.9$	33	1.000	1.339	1.124	0.083	0.074
$0.9 \leq \rho \leq 1.1$	44	1.006	1.444	1.146	0.108	0.094
$1.1 < \rho \leq 1.3$	38	1.010	1.470	1.155	0.112	0.097
$1.3 < \rho \leq 1.5$	28	0.978	1.340	1.109	0.108	0.097
$1.5 < \rho \leq 2.5$	25	0.972	1.330	1.143	0.101	0.088
Total	217					

Chapter 7: Proposed Ultimate Shear Stress Equation

Fig. 7.12(c) v_u / v_c versus Gravity Shear Ratio V_u/V_o

Table 7.3(c) Variation of v_u / v_c based on the gravity shear ratio parameter V_u/V_o

Gravity shear ratio, V_u/V_o	No. of specimens	Min v_u/v_c	Max v_u/v_c	Average v_u/v_c	Standard deviation	Coefficient of variation
$V_u/V_o < 0.2$	16	0.978	1.330	1.114	0.121	0.108
$0.2 \leq V_u/V_o < 0.3$	26	1.000	1.411	1.116	0.107	0.096
$0.3 \leq V_u/V_o < 0.4$	17	1.010	1.295	1.149	0.088	0.077
$0.4 \leq V_u/V_o < 0.5$	28	0.976	1.270	1.090	0.071	0.065
$0.5 \leq V_u/V_o < 0.6$	26	0.972	1.269	1.107	0.075	0.068
$0.6 \leq V_u/V_o \leq 0.7$	25	0.996	1.243	1.121	0.073	0.065
$0.7 < V_u/V_o < 0.8$	25	1.022	1.381	1.140	0.092	0.081
$0.8 \leq V_u/V_o < 0.9$	22	0.965	1.347	1.175	0.080	0.068
$0.9 \leq V_u/V_o < 1.0$	14	1.146	1.329	1.234	0.050	0.041
$V_u/V_o \geq 1.0$	18	1.200	1.470	1.308	0.078	0.059
Total	217

Fig. 7.12(d) v_u / v_c versus Connection Geometry d/b_o

Table 7.3(d) Variation of v_u / v_c based on the connection geometry parameter d/b_o

Connection geometry, d/b_o	No. of specimens	Min v_u/v_c	Max v_u/v_c	Average v_u/v_c	Standard deviation	Coefficient of variation
$0.04 \leq d/b_o \leq 0.06$	33	1.010	1.411	1.154	0.109	0.095
$0.06 < d/b_o \leq 0.067$	30	1.026	1.381	1.159	0.077	0.067
$0.067 < d/b_o < 0.07$	40	0.978	1.270	1.084	0.074	0.068
$0.07 \leq d/b_o \leq 0.075$	41	1.000	1.340	1.150	0.092	0.080
$0.075 < d/b_o < 0.08$	33	1.006	1.393	1.147	0.090	0.079
$0.08 \leq d/b_o < 0.10$	28	0.965	1.470	1.168	0.129	0.110
$0.10 \leq d/b_o \leq 0.12$	12	1.014	1.444	1.247	0.127	0.102
Total	217					

Chapter 7: Proposed Ultimate Shear Stress Equation

Fig. 7.12(e) v_u / v_c versus Slab Effective Depth d

Table 7.3(e) Variation of v_u / v_c based on the slab effective depth parameter d

Slab effective depth, d	No. of specimens	Min v_u/v_c	Max v_u/v_c	Average v_u/v_c	Standard deviation	Coefficient of variation
$d < 60$	33	0.978	1.470	1.200	0.128	0.107
$60 \leq d \leq 90$	31	1.012	1.444	1.170	0.120	0.102
$90 \leq d \leq 100$	43	1.011	1.345	1.141	0.088	0.077
$100 \leq d \leq 110$	20	1.010	1.280	1.127	0.071	0.063
$110 \leq d \leq 115$	29	0.996	1.270	1.133	0.086	0.076
$115 \leq d \leq 120$	27	0.972	1.269	1.117	0.073	0.065
$d \geq 120$	34	0.965	1.411	1.128	0.111	0.098
Total	217

Fig. 7.12(f) v_u/v_c versus Critical Shear Perimeter b_o

Table 7.3(f) Variation of v_u/v_c based on the critical shear perimeter parameter b_o

Critical shear perimeter, b_o	No. of specimens	Min v_u/v_c	Max v_u/v_c	Average v_u/v_c	Standard deviation	Coefficient of variation
$0.50 \leq b_o \leq 0.75$	27	1.006	1.470	1.246	0.122	0.098
$0.75 < b_o \leq 1.0$	11	0.978	1.381	1.166	0.137	0.118
$1.0 < b_o \leq 1.25$	26	1.012	1.345	1.178	0.102	0.087
$1.25 < b_o \leq 1.4$	30	1.012	1.309	1.151	0.066	0.057
$1.4 < b_o \leq 1.5$	40	0.965	1.269	1.096	0.074	0.068
$1.5 < b_o \leq 1.6$	28	1.012	1.320	1.113	0.081	0.073
$1.6 < b_o \leq 2.0$	40	0.996	1.340	1.110	0.080	0.072
$b_o \geq 2.0$	15	1.010	1.411	1.186	0.114	0.096
Total	217					

7.8 Slab-Column Connections under Cyclic Loading

Although slab-column connections may not be accounted for as part of the lateral-force resisting system, they still should have the ability to go through the lateral deformations of the lateral-force resisting system without loss of ability to support the gravity loads during an earthquake (Megally and Ghali, 2000). This indicates that the effect of load reversal on punching shear strength of concrete slabs has to be taken into account in the design of slab-column connections.

7.8.1 Proposed Shear Strength Degradation due to Cyclic Lateral Loading

It is known that the punching capacity of slab is reduced by repeated cycles of lateral loading. Thus, it is important to be able to predict the shear strength degradation since the applied shear stresses must not exceed the shear resistance at the given level of deterioration. Based on the observed response of 63 slab-column connections data under cyclic lateral loading collected from literature and given in Table E9 of the Appendix E, the following expression to estimate the shear strength degradation is proposed:

$$k = (1 - DR_u^{1/3}) \quad (7.14)$$

where:

k = strength reduction factor;

DR_u = ultimate drift ratio corresponding to the drift ratio at the peak lateral load, and expressed in a decimal form.

Taking into account the shear strength degradation into the proposed simple design equation for calculating the shear strength of slabs given in Eq. (6.4), resulted in a general equation for punching shear strength as shown in Eq.(7.15).

$$v_c = 0.6\rho^{1/3}(f'_c)^{1/3}(1 - DR_u^{1/3}) \quad (\text{MPa}) \quad (7.15)$$

Chapter 7: Proposed Ultimate Shear Stress Equation

The proposed general equations shown in Eq. (7.13) and Eq. (7.15) were used to predict the shear strengths of 63 slab-column connections under cyclic lateral loading, and the results in terms of shear strength ratio v_u/v_c are plotted in Fig. 7.13. It can be seen from Fig. 7.13 that inclusion of the strength reduction factor k as a function of story drift ratio ensures a safe design for slab-column connections subjected to cyclic lateral loading, i.e. punching failure will only occur when the ultimate shear stress reached the nominal shear strength, $v_u/v_c \geq 1$. Similar conclusion is also obtained in Fig. 7.14 when the proposed shear strength degradation is included in the ACI 318-05 Eqs. (3.5) to (3.7) for the shear strength of slabs.

Fig. 7.13 Shear Strength Ratio v_u/v_c Calculated Using The Proposed Shear Strength Equation that Includes Shear Strength Degradation due to Cyclic Lateral Loading (Eq. 7.15)

Chapter 7: Proposed Ultimate Shear Stress Equation

(a) Using ACI 318-05 (without strength degradation factor)

(b) Using modified ACI 318-05 (with strength degradation factor, Eq. 7.14)

Fig. 7.14 Comparison Between The Original ACI Equations and The ACI Equations That Includes The Proposed Shear Strength Degradation due to Cyclic Lateral Loading

7.9 Allowable Gravity Shear Ratio

Another possible way to consider the effect of story drift on the design of slab-column connections is by limiting the applied gravity shear. ACI 318-05 limits the gravity shear ratio V_u/V_o to 0.4 to ensure the ability of slab-column connections to undergo a minimum drift ratio capacity of 1.5 %. Fig.7.14 shows the effect of gravity shear ratio on drift ratio capacity of 63 slab-column connections collected from literature, and calculated using ACI Eqs. (3.5) to (3.7). It can be seen from Fig. 7.15 that there are some test specimens with gravity shear ratio greater than 0.4, and can still achieve a drift ratio capacity greater or equals to 1.5 %. On the other hand, connections with high column aspect ratio (specimens tested by Tan and Teng, Anggadajaja and Teng, and Widjaja and Teng) can only achieve a drift ratio capacity greater or equals to 1.5 %, if the gravity shear ratio V_u/V_o is limited to approximately 0.3. The scatter of data shown in Fig. 7.15 also indicates that the ACI 318-05 limit on gravity shear ratio may be too simplistic. Using the proposed unbalanced moment-and-shear interaction equation discussed earlier, the allowable gravity shear ratio for slab-column connections is investigated.

Fig. 7.15 Gravity Shear Effect on Story Drift Ratio Capacity Based on ACI Equations

Chapter 7: Proposed Ultimate Shear Stress Equation

The effect of story drift on the proposed unbalanced moment-and-shear interaction equation can be found by examining the effect on each of the respective gravity shear and unbalanced moment ratios, and they can be written in a general form as shown in Eq. (7.16).

$$1 = \beta_R \frac{V_u}{V_o} f_{D1} + K \left(\frac{M_u}{M_f} \right)^{1/3} f_{D2} \quad (7.16)$$

where: f_{D1} = story drift effect on gravity shear ratio;

f_{D2} = story drift effect on unbalanced moment ratio.

Based on a statistical analysis involving 63 experimental slab data with cyclic lateral loading collected from the literature, an empirical function to correct the story drift effect on gravity shear and unbalanced moment ratios are expressed in Eqs. (7.17) and (7.18).

$$f_{D1} = (1 + 10DR_u) \quad (7.17)$$

$$f_{D2} = (1 - 10DR_u) \quad (7.18)$$

Substituting Eqs. (7.17) and (7.18) into Eq.(7.16), the proposed unbalanced moment-and-shear interaction equation that includes the story drift effect then becomes:

$$1 = \beta_R \frac{V_u}{V_o} f_{D1} + K \left(\frac{M_u}{M_f} \right)^{1/3} f_{D2} \quad (7.19)$$

Re-arranging Eq. (7.19), a function to express the allowable gravity shear ratio can be written as follows:

$$\frac{V_u}{V_o} = \frac{1 - \left[K \left(\frac{M_u}{M_f} \right)^{1/3} f_{D2} \right]}{\beta_R f_{D1}} \quad (7.20)$$

Chapter 7: Proposed Ultimate Shear Stress Equation

The proposed formula of Eq. (7.20) shows that the allowable gravity shear ratio V_u/V_o is not solely limited by the ultimate drift capacity. V_u/V_o also depends on connection geometry d/b_o expressed in the parameter K , column rectangularity β_R , and the moment transfer capacity M_f expressed in terms of the unbalanced moment ratio M_u/M_f . Hence, given the material and geometrical properties of a slab-column connection, the underlying conditions for the limit of both gravity shear ratio and moment transfer capacity (in order to achieve a minimum drift ratio capacity of, say, 1.5 %) can be found.

According to the proposed Eq. (7.20), a typical square column-slab connections with $d/b_o \geq 0.08$ is able to achieve a drift capacity of 1.5 % with gravity shear ratio of 0.5, if the unbalanced moment ratio M_u/M_f was limited to 1. Eq. (7.20) also shows that a high column aspect ratio β_R would lower the allowable gravity shear ratio V_u/V_o , as observed previously in the experiments on rectangular-column slab connections by Tan and Teng (2005), Anggadajaja and Teng (2006), and Widjaja and Teng (2006). The relationship of the allowable gravity shear ratio on drift ratio capacity calculated using the proposed equation (Eq. 7.20) is given in Fig. 7.16.

Fig. 7.16 Gravity Shear Effect on Story Drift Ratio Capacity Based on Proposed Eq. 7.20

7.10 Analysis of Rectangular Edge-Column Slab Connections under Gravity and Cyclic Lateral Loadings

In Chapter 4 and 5, an experimental program of five large-scale rectangular edge-column slab connections subjected to gravity and cyclic lateral loading was discussed in detail. In Chapter 6 and 7, new methods or new equations are proposed to predict the shear strength of slabs and to calculate the slab ultimate shear stress at the critical shear perimeter of interior and exterior slab-column connections due to combined gravity load and unbalanced moment transfer. A simple expression is also proposed to consider the shear strength degradation for connections under cyclic loading. In this section, the punching shear capacities of all five specimens tested by the author are analyzed by using the ACI Code method and the proposed method. Some discussions are provided based on the comparisons between the ACI Code method and the proposed method. Note that only the ACI Code method is included in the comparative analysis in this chapter. The reason is that the ACI method is capable of considering all the various factors that are considered in this research.

The required data for analysis of all five slab-column connection specimens tested by the author are shown in Table 7.4. More detailed information on these connections can be seen in Chapter 4 and 5. The comparisons between the ACI Code method and proposed method are shown in Table 7.5 in terms of the measured over predicted strength ratios.

In the ACI Code method, the punching shear capacities v_c were predicted from Eqs. (3.5) to (3.7), while the ultimate shear stresses v_u were calculated using Eqs. (3.21) and (3.22) for connections transferring uniaxial unbalanced moment and Eq. (3.20) for connections transferring biaxial unbalanced moment. The property of the critical section analogous to the polar moment of inertia J_c in which the values are different for interior, edge and corner connections and the assumed fraction of unbalanced moment transferred by shear γ_v were also calculated. Meanwhile, in the proposed method, only two simple equations were used. The punching shear capacities were predicted from Eq. (7.15) while the ultimate shear stresses were calculated using Eq. (7.13).

Chapter 7: Proposed Ultimate Shear Stress Equation

Table 7.4 Experimental Data for Rectangular Edge-Column Slab Connections under Gravity and Cyclic Lateral Loadings

Slab ID	h , mm	c_1 , mm	c_2 , mm	d , mm	ρ_x , %	ρ_y , %	f'_c , MPa	f_y , GPa	DR_{ux} , %	DR_{uy} , %	V_u , kN	M_{ux} , kN-m	M_{uy} , kN-m
E1H	135	900	180	107	1.2	1.2	33.0	532	3.02	-	100.0	146.1	-
E2H	135	900	180	107	1.2	1.2	32.5	528	-	4.29	100.0	50.1	101.5
E12L	135	900	180	107	1.2	1.2	34.4	530	2.02	2.51	50.0	104.3	74.0
E12H	135	900	180	107	1.2	1.2	35.4	531	1.60	2.10	100.0	93.7	60.0
E0U	135	900	180	107	1.2	1.2	33.3	529	-	-	245.0	81.0	-

Table 7.5 Comparison of Experimental Results and Predicted Strengths for Rectangular Edge-Column Slab Connections under Gravity and Cyclic Lateral Loadings (Author’s specimens)

Slab ID	ACI 318-05 Method			Proposed Method (Eqs. 7.13, 7.15)		
	v_u (N/mm ²)	v_c (N/mm ²)	v_u / v_c	v_u (N/mm ²)	v_c (N/mm ²)	v_u / v_c
E1H	1.862	1.340	1.389	1.827	1.427	1.280
E2H	1.735	1.330	1.304	1.353	1.340	1.010
E12L	1.898	1.369	1.387	1.591	1.487	1.070
E12H	1.832	1.388	1.320	1.737	1.537	1.130
E0U	1.663	1.346	1.235	2.571	2.090	1.230
Average v_u / v_c			1.327			1.144
Min v_u / v_c			1.235			1.010
Max v_u / v_c			1.389			1.280
Standard Deviation			0.064			0.111
Coefficient of Variation			0.004			0.012

Chapter 7: Proposed Ultimate Shear Stress Equation

It can be seen from Table 7.5 that using the ACI Code method, the average ratio of experimental to predicted shear strength is 1.327, which means that the ACI Code method underestimates more than 30 percent of the actual shear strength. Compared to the ACI Code method, the proposed method improves the accuracy considerably. Based on the proposed method, the average actual shear strengths are underestimated by only 15 percent of the predicted values, shown by an average ratio of experimental to predicted shear strength of 1.144 in Table 7.5. In general, the proposed method can provide an improvement over the ACI Code method given its simplicity and accuracy to analyze the punching shear capacities of slab-column connections under gravity and cyclic lateral loadings. The influence of flexural reinforcement ratio and the shear strength degradation due to cyclic loading, not considered in the ACI Code method, has been taken into account in the proposed shear strength formula, while column rectangularity and gravity shear effect has been considered in the proposed equation to calculate the slab ultimate shear stress.

7.11 The Conservatism in The Moment Transfer Provisions

The ACI Code provisions for punching shear with moment transfer are extensions of the no-moment or concentric shear provisions. Any conservatism in the concentric shear provisions are reflected in the punching shear predictions for moment transfer. Thus, the error due to concentric shear provisions that are carried through in the moment transfer provisions has to be identified in order to assess the error arising from the moment transfer provision only.

The conservatism in the concentric shear provisions

In Table 6.2, the candidate showed that the mean ACI v_u/v_c for concentric shear is 1.410 for square column-slab connections and 1.267 for rectangular column-slab connections. These errors are resulted from the ACI v_u and v_c equations for concentric shear shown below:

$$v_u = \frac{V_u}{b_o d} \quad (7.21)$$

$$v_c = \left(1 + \frac{2}{\beta}\right) \frac{\sqrt{f'_c}}{6} \quad (\text{MPa}) \quad (3.5)$$

$$v_c = \left(2 + \frac{\alpha_s d}{b_o}\right) \frac{\sqrt{f'_c}}{12} \quad (\text{MPa}) \quad (3.6)$$

$$v_c = \frac{1}{3} \sqrt{f'_c} \quad (\text{MPa}) \quad (3.7)$$

The conservatism in the moment transfer provisions

Table 7.2 summarizes the results for slab-column connections with moment transfer and they were calculated using the ACI Code provisions for moment transfer as follows:

$$v_u = \underbrace{\frac{V_u}{b_o d}}_{\text{Direct shear term}} + \underbrace{\frac{\gamma_v M_u c}{J_c}}_{\text{Moment transfer term}} \quad (3.17)$$

Chapter 7: Proposed Ultimate Shear Stress Equation

$$v_c = \left(1 + \frac{2}{\beta}\right) \frac{\sqrt{f'_c}}{6} \quad (\text{MPa}) \quad (3.5)$$

$$v_c = \left(2 + \frac{\alpha_s d}{b_o}\right) \frac{\sqrt{f'_c}}{12} \quad (\text{MPa}) \quad (3.6)$$

$$v_c = \frac{1}{3} \sqrt{f'_c} \quad (\text{MPa}) \quad (3.7)$$

It can be seen from Eq. (3.17) that the v_u equation for moment transfer essentially is an extension of Eq. (7.21) used in the concentric shear provision, and it comprises of two terms, namely the direct shear term and the moment transfer term. In addition, the same v_c equations used in the concentric shear provision are used to predict the slab shear capacity for connections with moment transfer. Thus, the mean ACI v_u/v_c shown in Table 7.2 for connections with moment transfer essentially represents the error that arises due to the moment transfer provision $\frac{\gamma_v M_u c}{J_c}$ as well as the error that is carried through from the concentric shear provisions $\frac{V_u}{b_o d}$ and v_c . In order to check the interaction equation, the error associated with the moment transfer provision only ($\frac{\gamma_v M_u c}{J_c}$) has to be quantified, and therefore, the following procedures are proposed:

- Normalize the v_c equations such that the mean ACI v_u/v_c for concentric shear equals to 1.0 to eliminate the errors due to v_c that are carried through in the moment transfer provisions.
- Only those data with zero or negligible direct shear force are included in order to eliminate the errors due to the direct shear term ($V_u/b_o d$) in the v_u equation.

From Table 6.2, the mean ACI v_u/v_c for concentric shear is 1.410 for square column-slab connections and 1.267 for rectangular column-slab connections. Thus, the normalized v_c or v_c^* for square and rectangular column-slab connections are as follows:

Chapter 7: Proposed Ultimate Shear Stress Equation

For square columns:

$$\frac{v_u}{v_c} = 1.410$$
$$\frac{v_u}{1.410 v_c} = 1.000 \quad \times \frac{1}{1.410}$$

$$v_c^* = 1.410 v_c$$

For rectangular columns:

$$\frac{v_u}{v_c} = 1.267$$
$$\frac{v_u}{1.267 v_c} = 1.000 \quad \times \frac{1}{1.267}$$

$$v_c^* = 1.267 v_c$$

This v_c^* is then applied to those data with zero or negligible direct shear force. From 217 data collected from literature for connections with unbalanced moment transfer, there are 8 slab-column connections data with zero or negligible direct shear force (Hanson and Hanson, 1968; Zaghlool, 1971; Stamenkovic and Chapman, 1974) and the results are given in Table E10. Basically from these procedures, the mean ACI v_u/v_c^* shown in Table E10 gives the error resulted from the moment transfer provision only ($\frac{\gamma_v M_u c}{J_c}$) since the direct shear term $\frac{V_u}{b_o d}$ is zero due to zero shear force and the error due to concentric shear provision v_c has been eliminated using the normalized v_c^* .

Table E10 shows that the mean ACI v_u/v_c^* for moment transfer provision is 1.271. This indicates that the ACI concentric shear provision is much more conservative (1.410) than the ACI moment transfer provision (1.271). Hence, the ACI punching shear prediction for slabs with moment transfer would give a highly conservative results since the conservatism in the moment transfer provision is amplified with the highly conservative results due to ACI concentric shear provisions.

Similar procedures are also applied for the candidate's proposed formula with a difference in that the v_u equation was affected by the normalized v_c^* due to the gravity shear ratio term V_u/V_o in the v_u equation (thus, becomes v_u^*). The results given in Table E10 show that the mean v_u^*/v_c^* for moment transfer provision calculated from the proposed formula is 1.015. This indicates that the unbalanced moment-and-shear interaction equation proposed by the candidate can give reasonably accurate predictions for slab-column connections with moment transfer.

7.12 Complete General Expression Proposed for Design

The complete general expression of the author's proposed formulas for flat plate design of punching shear with unbalanced moment expressed in a form of unbalanced moment-and-shear interaction is summarized.

For a safe design, the nominal shear stress v_c multiplied by the strength reduction factor ϕ should be greater or equal to the ultimate shear stress v_u as follows:

$$\phi v_c \geq v_u \quad (7.22)$$

Eq. (7.22) can also be written as:

$$\frac{v_u}{v_c} < \phi \quad (7.23)$$

Substituting Eq. (7.23) with the basic format of the proposed unbalanced moment-and-shear interaction equation Eq. (7.11) including column rectangularity factor gives:

$$\beta_R \frac{V_u}{V_o} + K \left(\frac{M_u}{M_f} \right)^{1/3} < \phi \quad (7.24)$$

Substituting Eqs. (6.6) and (7.9) for β_R and K respectively, the complete general expression in a form of unbalanced moment-and-shear interaction proposed for design is:

$$\left(\frac{b_2}{b_1} \right)^{1/4} \frac{V_u}{V_o} + \left(1 - 0.7 \frac{V_u}{V_o} \right) \left(\frac{d}{b_o} \right)^{0.1} \left(\frac{M_u}{M_f} \right)^{1/3} < \phi \quad (7.25)$$

where: $V_o = 0.6 \rho^{1/3} (f'_c)^{1/3} (1 - DR_u^{1/3}) b_o d \quad (7.26)$

The notations are defined as follows:

- b_2 = the length of the longer side of the critical shear perimeter;
- b_1 = the length of the shorter side of the critical shear perimeter;
- V_u = ultimate shear force at failure;
- V_o = shear force capacity calculated from the unified Eq. (7.26);
- ρ = average flexural reinforcement ratio in percentage;
- f'_c = concrete cylinder strength in MPa;

-
- DR_u = ultimate drift ratio corresponding to the drift ratio at the peak lateral load, and expressed in a decimal form;
 b_o = slab critical shear perimeter taken at a distance $d/2$ from the column;
 d = slab effective depth;
 M_u = ultimate unbalanced moment at failure;
 M_f = the flexural capacity of slab reinforcement within the prescribed transfer width of $c_2 + 1.5h$ on each side of the column;
 ϕ = strength reduction factor.

7.13 Summary

A new method is presented for calculating the slab ultimate shear stress at the critical shear perimeter of interior and exterior slab-column connections due to combined gravity load and unbalanced moment transfer. The proposed model provides valuable insights into the interaction between the unbalanced moment and shear transferred at slab-column connections under various loading conditions. It also gives a consistent way to calculate the slab ultimate shear stress of slab-column connections under different loading cases, i.e. balanced loading and unbalanced loading, and for all types of connections, i.e. interior, edge and corner connections.

The method is based on an interaction equation for shear and unbalanced moments, assuming that moment transfer capacity is limited solely by the available flexural reinforcement within an effective transfer width of $c_2 + 1.5h$ on each side of the column. Column rectangularity, connection geometry, influence of reinforcement ratio and gravity shear ratio are taken into consideration. The proposed equation has been verified for accuracy by predicting the shear strength of 217 slab-column connections including interior, edge, and corner connections, ranging from square-column slab connections transferring uniaxial unbalanced moment to rectangular-column slab connections transferring biaxial unbalanced moments due to cyclic lateral loading. The proposed formula gives a very good agreement between the predicted

Chapter 7: Proposed Ultimate Shear Stress Equation

and experimental values, and it also simplifies the design and analysis process. Comparisons with the current ACI Code equations shows that the mean values of the measured over predicted strengths v_u/v_c , standard deviations and coefficients of variations from the proposed formulas are significantly better than those calculated based on the ACI equations.

A simple expression is also proposed to consider shear strength degradation for connections under cyclic loading. The inclusion of strength reduction as a function of story drift has been proven to give a better estimate of the shear resistance at the given level of deterioration to ensure a safe design. The ACI Code provision on gravity shear ratio limit for seismic-resistant slab-column connections is reviewed. An equation to calculate the allowable gravity shear ratio is established based on the proposed unbalanced moment-and-shear interaction that includes effect of story drift. The equation shows that the gravity shear ratio is not only limited by the ultimate drift ratio capacity DR_u , but also depends on connection geometry d/b_o , column rectangularity β , and the moment transfer capacity M_f .

CHAPTER 8

CONCLUSIONS

CHAPTER 8

CONCLUSIONS

The purpose of this research work was to investigate the ultimate punching shear capacity of slabs supported on rectangular columns, under the combined effect of shear force and cyclic unbalanced moment transfer. In this research, these effects of gravity shear ratio and biaxial unbalanced moments were also studied. For this purpose, a combined experimental and analytical research program was carried out to study these issues.

The experimental program included design, construction, laboratory testing, and analyses of five large-scale rectangular edge-column slab connections subjected to gravity and cyclic lateral loading. Two specimens were uniaxially laterally loaded, two more specimens were bi-axially laterally loaded, and the fifth specimen was tested under gravity load only. Three levels of gravity loading were incorporated in the experimental program.

Conclusions related to the experimental studies are listed below:

1. The $0.4 V_c$ limit on gravity load recommended by the ACI Committee 352 for slab-column connections to achieve drift capacity of at least 1.5 % should be reduced to $0.3 V_c$ in order to cater for biaxially loaded rectangular-column slab connections with high column aspect ratio.
2. As expected, column rectangularity significantly enhances the ability of slab-column connection to transfer more unbalanced moment about the strong column axis.

3. For rectangular-column slab connection, low ductility ratio or brittle modes of failure can be expected to occur in the strong column direction due to higher shear stresses developed along the column short side, and these shear stresses can be critical in limiting the shear capacity of the slab.
4. High column rectangularity ratio contributes to a significant flexibility about the column minor axis as expected. Therefore, very high drift ratio can result in the weak column direction due to column flexure about its minor axis.
5. For rectangular-column slab connections, stiffness in the weak column direction is expected to be lower than the stiffness in the strong column direction, and application of high gravity shear would further reduce the connection stiffness in the weaker column direction.
6. Biaxial cyclic lateral loading was found to severely reduce the strength, stiffness, ductility and drift capacity of slab-column connections with high column rectangularity ratios.
7. Higher gravity shear reduces the connection strength, stiffness, ductility and drift capacity. However, relatively low reductions were observed even if the amount of gravity shear was doubled for rectangular-column slab connections with an aspect ratio of five.

Two sets of new procedures are proposed to analyze the strength of slab-column connections. These proposed methods had been verified for accuracy by predicting the shear strength of 455 slab-column connections collected from literature including interior, edge, and corner connections, ranging from symmetrical punching to connections having rectangular columns transferring unbalanced moments due to cyclic lateral loading.

Conclusions related to the analytical research works are listed as follows:

1. A semi-empirical model is proposed for predicting the shear strength of slabs v_c based on a comprehensive and systematic study on the databank collected. Only one equation is needed for predicting the punching strength and it includes the influence of flexural reinforcement ratio and the strength degradation due to cyclic loading.
2. The proposed simple equation for v_c has been verified against 238 slab data under symmetrical punching and was found to be more accurate than existing ACI code formulas. For slab-column connections with rectangular columns, a factor is introduced into the shear stress component from the direct shear force to consider the rectangularity of the columns. Besides its simplicity, the proposed simple equation for v_c improves the punching shear strength prediction of slab-column connections considerably.
3. A new method is proposed to describe a consistent relationship between shearing forces and unbalanced moments for all punching cases of interior, edge, and corner slab-column connections. The method is based on an interaction equation for shear and unbalanced moment, assuming that moment transfer capacity is limited solely by the available flexural reinforcement within an effective transfer width of $c_2 + 1.5h$ on each side of the column. The influence of concrete strength, flexural reinforcement ratio, connection geometry, and gravity shear ratio are included in the proposed unbalanced moment-and-shear interaction.
4. A new equation is presented for calculating the slab ultimate shear stress v_u at the critical shear perimeter of interior, edge, and corner slab-column connections due to combined gravity load and unbalanced moment transfer. The proposed equation is derived based on the proposed unbalanced moment-and-shear interaction. The proposed general equation has been verified for accuracy and consistency by predicting the shear strength of 217

slab-column connections with unbalanced moment transfer including those with rectangular columns and subjected to biaxial unbalanced moments. The proposed formula gives a very good agreement between the predicted and experimental values, and it also simplifies the design and analysis process. Comparisons with the current ACI Code equations shows that the mean values of the measured over predicted strengths v_u/v_c , standard deviations and coefficients of variations from the proposed formula are significantly better than those calculated based on the ACI equations.

5. A simple expression is also proposed to consider shear strength degradation for connections under cyclic loading. The inclusion of strength reduction as a function of story drift has been shown to give a better estimate of the shear resistance at the given level of deterioration to ensure a safe design. The ACI Code provision on gravity shear ratio limit for seismic-resistant slab-column connections may be too simplistic. An equation to calculate the allowable gravity shear ratio is established based on the proposed unbalanced moment-and-shear interaction that includes the effect of story drift. The equation shows that the gravity shear ratio is not only limited by the ultimate drift ratio capacity DR_u , but also depends on connection geometry d/b_o , column rectangularity β , and the moment transfer capacity M_f .
6. The good agreement between the predicted and experimental values obtained for all cases of 455 slab-column connections data collected from literature indicates that the proposed equations for predicting the shear strength of slabs v_c and for calculating the slab ultimate shear stress v_u are accurate and reliable, and they can be included in design procedures.

REFERENCES

REFERENCES

1. ACI Committee 318, "Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary," American Concrete Institute, Farmington Hills, Mich., 2005, 443pp.
2. ACI-ASCE Committee 326, "Shear and Diagonal Tension," *ACI Journal*, Proceedings, Vol. 59, January to March 1962, pp 1-30, 277-330, and 353-395.
3. ACI-ASCE Committee 352, "Recommendations for Design of Slab-Column Connections in Monolithic Reinforced Concrete Structures," (*ACI 352.1R-89*), (Reapproved 1997) American Concrete Institute, Farmington Hills, Mich., 22 pp.
4. Alexander, S. D. B.; and Simmonds, S. H., "Ultimate Strength of Slab-Column Connections," *ACI Structural Journal*, Vol. 84, No. 3, May-June 1987, pp255-261.
5. Alexander, S. D. B.; and Simmonds, S. H., "Bond model for Concentric Punching Shear," *ACI Structural Journal*, Vol. 89, No. 3, May-June 1992, pp325-334.
6. Alexander, S. D. B.; and Simmonds, S. H., "Moment Transfer at Interior Slab-Column Connections," *ACI Structural Journal*, Vol. 100, No. 2, March 2003, pp. 197-202.
7. Anggadajaja, E., and Teng, S., "Behaviour of Rectangular Edge Column-Slab Connections Subjected to Gravity and Biaxial Cyclic Lateral Loading," *Final Report*, A BCA-NTU Joint Research on Flat Plate Structures, Phase IIIA, 2006, 250pp.

References

-
8. AS 3600, "Australian Standard for Concrete Structures," *Standard Association of Australia*, North Sydney, Australia, 1994, 155 pp.
 9. AISI, "Anchorage and the Alaska Earthquake," *American Iron and Steel Institute*, Washington DC, 1964, pp. 23-38.
 10. AISI, "Performance of Steel Buildings in Past Earthquakes: The Mexico Earthquake of 1985," *American Iron and Steel Institute*, Washington DC, 1991, pp. 45-64.
 11. Bazant, Z. P., "Size Effect in Blunt Fracture: Concrete, Rock, Metal," *Journal of Engineering Mechanics*, ASCE, V.110, No. 4, Apr. 1984, pp.518-535.
 12. Bazant, Z. P., and Cao Z., "Size Effect in Punching Shear Failure of Slabs," *ACI Structural Journal*, Vol. 84, No. 1, Jan-Feb. 1987, pp. 44-53.
 13. Braestrup, M. W., and Nielsen, M. P., "Axisymmetric Punching of Plain and Reinforced Concrete," *Report*, No. R-75, Structural Research Lab, Tech. Univ. of Denmark, Copenhagen, Denmark, 33pp.
 14. British Standards Institution, "BS 8110 Structural Use of Concrete: Part I – Code of Practice for Design and Construction," London, 1997, 120pp.
 15. Broms, C. E., "Punching of Flat Plates – A Question of Concrete Properties in Biaxial Compression and Size Effect," *ACI Structural Journal*, Vol. 87, No.3, May-June 1990, pp. 292-304.
 16. Brotolotti, L., "Punching Shear Strength in Concrete Slabs," *ACI Structural Journal*, Vol. 87, No. 2, Mar-Apr. 1990, pp. 208-219.
 17. Brown, S. J., 2003, Seismic Response of Slab-Column Connections," *Ph.D. Thesis*, Civil Engineering, University of Calgary, Canada.
-

References

-
18. Canadian Standards Association, "Design of Concrete Structures for Buildings," CAN3, A23.3-M94, Rexdale, Toronto, Ontario, Canada, 1994, 199pp.
 19. Cao, H., "Seismic Design of Slab-Column Connections," *M.Sc. Thesis*, 1993, Civil Engineering, University of Calgary, Canada.
 20. CEB-FIP, "Punching of Structural Concrete Slabs," Bulletin 12, *CEB-fib Task Group*, Utilization of Concrete Tension in Design, 2001, 307pp.
 21. Chana, P. S., and Desai, S. B., "Design of Shear Reinforcement Against Punching," *The Structural Engineer*, Vol. 70, No. 5, May 1992, pp. 159-164.
 22. Chana, P. S., and Desai, S. B., "Membrane Action and Design Against Punching Shear," *The Structural Engineer*, Vol. 70, No. 19, Oct. 1992, pp. 339-343.
 23. Corley, W. G., and Hawkins, N. M., "Shearhead Reinforcement for Slabs," *ACI Journal*, Proceedings, Vol. 65, Oct. 1968, pp. 811-823.
 24. Desayi, P., and Seshadri, H. K., "Punching Shear Strength of Flat Slab Corner Column Connections," *Proceedings*, Institution of Civil Engineers, Part I, V.122, Feb.1997, pp10-20.
 25. Dilger, W., and Cao, H., "Behaviour of Slab-Column Connections under Reversed Cyclic Loading," *Proceedings*, 2nd International Conference of High Rise Buildings, China, 1991.
 26. Dilger, W. H., and Brown, S. J., "Earthquake Resistance of Slab Column Connections," *Proceedings of Canadian Society of Civil Engineering*, Vol. 2, Winnipeg, Canada, 1994.
-

References

-
27. Di Stasio, J., and Van Buren, M. P., "Transfer of Bending Moments between Flat Plate Floors and Columns," *ACI Journal, Proceedings*, Vol.57, No.3, Sep. 1960, pp. 299-314.
 28. Durrani, A.J.; Du, Y.; Luo, Y.H., "Seismic Resistance of Nonductile Slab-Column Connections in Existing Flat-Slab Buildings," *ACI Structural Journal*, V. 92, No. 4, August 1995, pp.479-487.
 29. Elgabry, A.A.; and Ghali, A., "Tests on Concrete Slab-Column Connections with Stud-Shear Reinforcement Subjected to Shear-Moment Transfer," *ACI Structural Journal*, Vol. 84, No.5, Oct. 1987, pp. 433-442.
 30. Elgabry, A.A.; and Ghali, A., "Transfer of Moments between Columns and Slabs: Proposed Code Revisions," *ACI Structural Journal*, Vol. 93, No.1, Jan-Feb. 1996, pp. 56-61.
 31. Elstner, R. C., and Hognestad, E., "Shearing Strength of Reinforced Concrete Slabs," *ACI Journal, Proceedings*, Vol. 53, No. 1, Oct. 1956, pp. 29-58.
 32. El-Salakawy, E.F.; Polak, M.A.; Soliman, M.H., "Reinforced Concrete Slab-Column Edge Connections with Openings," *ACI Structural Journal*, Vol. 96, No. 1, Feb. 1999, pp.79-88.
 33. Emam, M., Marzouk, H., Hilal, M. S., "Seismic Response of Slab-Column Connections Constructed with High-Strength Concrete," *ACI Structural Journal*, V. 94, No. 2, Mar. 1997, pp.197-205.
 34. Eurocode 2: Design of Concrete Structures – Part I: General Rules for Buildings, *European Prestandard ENV 1992-1-1:1991*, 253pp.
-

References

35. Falamaki, M. and Loo, Y.C., "Punching Shear Tests of Half-Scale R. C. Flat-Plate Models with Spandrel Beams," *ACI Structural Journal*, Vol. 89, No. 3, June 1992, pp.263-271.
36. Farhey, D.N.; Adin, M.A.; Yankelevsky, D.Z., "RC Flat Slab-Column Subassemblages under Lateral Loading," *Journal of Structural Engineering*, ASCE, V.119, No.6, June 1993, pp.1903-1916.
37. Gardner, N. J., "Relationship of the Punching Shear Capacity of Reinforced Concrete Slabs with Concrete Strength," *ACI Structural Journal*, Vol. 87, No.1, Jan-Feb. 1990, pp. 66-71.
38. Gardner, N.J. and Shao, X.Y., "Punching Shear of Continuous Flat Reinforced Concrete Slabs," *ACI Structural Journal*, Vol. 93, No. 2, April 1996, pp.218-228.
39. Ghali, A.; Elmasri, M.Z.; Dilger, W., "Punching of Flat Plates under Static and Dynamic Horizontal Forces," *ACI Journal, Proceedings*, Vol.73, No.10, Oct.1976, pp. 566-572.
40. Ghali, A., and Megally, S. "Design for Punching Shear Strength with ACI 318-95," *ACI Structural Journal*, Vol. 96, No. 4, July 1999, pp. 539-549.
41. Gilbert, S. G., and Glass, C., "Punching Failure of Reinforced Concrete Flat Slabs at Edge Columns," *The Structural Engineer*, Vol. 65B, No.1, March 1987, pp. 16-21.
42. Hall, A.S. and Rangan, B.V., "Forces in the Vicinity of Edge Columns in Flat-Slab Floors," *Magazine of Concrete Research*, Vol. 35, No. 122, Mar. 1983, pp.19-26.

References

-
43. Hammill, N. and Ghali, A., "Punching Shear Resistance of Corner Slab-Column Connections," *ACI Structural Journal*, Vol.91, No.6, Dec. 1994, pp.697-707.
 44. Hanson, N.W. and Hanson, J.M., "Shear and Moment Transfer between Concrete Slabs and Columns," *Journal of the Research and Development Laboratories*, Portland Cement Association, Vol.10, No.1, Jan. 1968, pp. 2-16.
 45. Hawkins, N. M.; Fallsen, H.; Hinojosa, R. C., "Influence of Column Rectangularity on the Behaviour of Flat Plate Structures," *Cracking, Deflection, and Ultimate Load of Concrete Slab Systems*, SP-30, American Concrete Institute, 1971, pp. 127-146.
 46. Hawkins, N. M.; Bao, A.; Yamazaki, J., "Moment Transfer from Concrete Slabs to Columns," *ACI Structural Journal*, Vol. 86, No.6, Dec. 1989, pp.705-716.
 47. Hawkins, N. M.; Mitchell, D.; Sheu, M. S., "Cyclic Behavior of Six Reinforced Concrete Slab-Column Specimens Transferring Moment and Shear," *Progress Report on NSF Project GI-38717*, Department of Civil Engineering, University of Washington, Seattle, 1974.
 48. Hawkins, N. M., and Mitchell, D., "Progressive Collapse of Flat Plate Structures," *ACI Journal*, Proceedings, Vol. 76, No.7, July 1979, pp.775-808.
 49. Hawkins, N. M.; Wong, C. F.; Yang, C.H., "Slab-Edge Column Connections Transferring High Intensity Reversing Moments Normal to the Edge of the Slab," *Structures and Mechanics Report No. SM78-1*, Department of Civil Engineering, University of Washington, Seattle, May 1978.
-

References

50. Hueste, M.B.D., and Wight, J.K., "Nonlinear Punching Shear Failure Model for Interior Slab-Column Connections," *Journal of Structural Engineering*, ASCE, V. 125, No. 9, September 1999, pp. 997-1008.
51. Hwang, S., and Moehle, J. P., "Vertical and Lateral Load Tests of Nine-Panel Flat Plate Frame" *ACI Structural Journal*, Vol. 97, No.1, Jan. 2000, pp.193-203.
52. Ingvarsson, H., "Experimentellt Studium Av Betongplattor Understodda Av Hornpelare, (Experimental Study of Concrete Slabs Supported by Corner Columns)" *Meddelande* Nr.111, Institutionen For Byggnadsstatik, Kungliga Tekniska Hogskolon, Stockholm, 1974.
53. Islam, S.; and Park, R., "Test on Slab-Column Connections with Shear and Unbalanced Flexure," *Journal of the Structural Division*, ASCE, V. 102, No. 3, Mar. 1976, pp.549-568.
54. Joint ACI-ASCE Committee 426, "The Shear Strength of Reinforced Concrete Members," *Proceedings*, ASCE, V. 100, No. ST8, Aug. 1974, pp. 1543-1591.
55. Kane, K.A., "Some Model Tests on the Punching Action of Reinforced Concrete Slabs t Edge Columns," *Honors Project*, The Queen's University of Belfast, 1978.
56. Kinnunen, S., and Nylander, H., "Punching of Concrete Slabs without Shear Reinforcement," *Transaction No. 158*, Royal Institute of Technology, Stockholm, 1960.
57. Kuang, J. S., and Morley, C. T., "A Plasticity Model for Punching Shear of Laterally Restrained Slabs with Compressive Membrane Action," *Int. J. Mech.*, V.35, No.5, 1993, pp. 371-385.

References

-
58. Kuang, K. L., and Teng, S., "Punching Shear Strength of Slabs with Openings and Supported on Rectangular Columns," *Final Report*, A BCA-NTU Joint Research on Flat Plate Structures, Phase-1A, 2001, 298pp.
 59. Lee, S. C., and Teng, S., "Punching Shear Strength of Slabs with Rectangular Columns," *Interim Research Report*, School of Civil and Environmental Engineering, Nanyang Technological University, Singapore, 1999, 93pp.
 60. Lim, F.K. and Rangan, B.V., "Studies on Concrete Slabs with SSR in Vicinity of Edge and Corner Columns," *ACI Structural Journal*, V. 92, No. 5, Oct. 1995, pp.515-525.
 61. Long, A. E., "A Two-Phase Approach to the Prediction of Punching Strength of Slabs," *ACI Journal*, Vol. 72, No.2, Feb. 1975, pp. 37-45.
 62. Luo, Y.H. and Durrani, A.J., "Equivalent Beam Model for Flat-Slab Buildings – Part I: Interior Connections," *ACI Structural Journal*, V. 92, No. 1, Feb. 1995, pp.115-124.
 63. Macgregor, J. G., "Reinforced Concrete – Mechanics and Design," Prentice Hall International, Inc. 1997, 939pp.
 64. Marzouk, H., and Hussein, A., "Punching Shear Analysis of Reinforced High-Strength Concrete Slabs," *Canadian Journal of Civil Engineering*, Vol. 18, No. 4, 1991, pp. 954-963.
 65. Marzouk, H., Emam, M., and Hilal, M. S., "Effect of High-Strength Concrete Slab on the Behavior of Slab-Column Connections," *ACI Structural Journal*, V. 95, No. 3, May. 1998, pp.227-237.

References

66. Megally, S., 1998, "Punching Shear Resistance of Concrete Slabs to Gravity and Earthquake Forces," *Ph.D. Thesis*, Department of Civil Engineering, University of Calgary, Calgary, Alberta, Canada, 468 pp.
67. Megally, S., and Ghali, A., "Seismic Behavior of Edge Column-Slab Connections with Stud Shear Reinforcement," *ACI Structural Journal*, Vol. 97, No. 1, Feb. 2000, pp. 53-60.
68. Megally, S., and Ghali, A., "Punching of Concrete Slabs Due to Column's Moment Transfer," *ASCE Journal of Structural Division*, V. 126, No. 2, Feb. 2000, pp. 180-189.
69. Megally, S., and Ghali, A., "Cautionary Note on Shear Capitals," *Concrete International* V. 24, No. 3, Mar. 2002, pp. 75-82.
70. Menetrey, P., "Analytical Computation of the Punching Strength of Reinforced Concrete," *ACI Structural Journal*, Vol. 93, No.5, Sep-Oct. 1996, pp. 503-511.
71. Mitchell, D.; Tinawi, R.; and Redwood, R. G., "Damage to Buildings Due to the 1989 Loma Prieta Earthquake – A Canadian Code Perspective," *Canadian Journal of Civil Engineering*, V. 17, No. 10, Oct. 1990, pp. 813-834.
72. Mitchell, D.; DeVall, R. H.; Saatcioglu, M.; Simpson, R.; Tinawi, R.; and Tremblay, R., "Damage to Concrete Structures Due to the 1994 Northridge Earthquake," *Canadian Journal of Civil Engineering*, V. 22, No. 4, Apr. 1995, pp. 361-377.
73. Moe, J., "Shearing Strength of Reinforced Concrete Slabs and Footings Under Concentrated Loads," *Development Department Bulletin D47*, Portland Cement Association, Skokie, Ill., April 1961, 130 pp.

References

74. Moehle, J.P., "Strength of Slab-Column Edge Connections," *ACI Structural Journal*, Vol. 85, No. 1, Feb. 1988, pp. 89-98.
75. Moehle, J.P., Kreger, M.E., and León R., 1988, "Background of Recommendations for Design of Reinforced Concrete Slab-Column Connections," *ACI Structural Journal*, V. 85, No. 6, Nov.-Dec., pp. 636-644.
76. Morrison, D. G., Hirasawa, I., and Sozen, M. A., "Lateral Load Tests of RC Slab-Column Connections," *Journal of Structural Engineering*, Vol. 109, No. 11, Nov. 1983, pp. 2698-2714.
77. Mortin, J.D. and Ghali, A., "Connection of Flat Plates to Edge Columns," *ACI Structural Journal*, Vol. 88, No.2, 1991, pp.191-198.
78. Mowrer, R. D., and Vanderbilt, M. D., "Shear Strength of Lightweight Aggregate Reinforced Concrete Flat Plate," *Proceedings of ACI Journal*, Vol. 64, No.11, 1967, pp. 722-729.
79. Nielsen, M. P., "Limit Analysis and Concrete Plasticity," Prentice-Hall Inc., Eaglewood Cliffs, N. J., 320pp.
80. Pan, A.D. and Moehle, J.P., "An Experimental Study of Slab-Column Connections," *ACI Structural Journal*, V. 89, No. 6, Dec. 1992, pp.626-638.
81. Pan, A.D. and Moehle, J.P., "Lateral Displacement Ductility of Reinforced Concrete Flat Plates," *ACI Structural Journal*, V. 86, No. 3, May. 1989, pp.250-258.
82. Polak, M. A., "Modeling Punching Shear of Reinforced Concrete Slabs Using Layered Finite Elements," *ACI Structural Journal*, Vol. 95, No.1, Jan-Feb. 1998, pp. 71-80.

References

83. Rangan, B.V., "Tests on Slabs in the Vicinity of Edge Columns," *ACI Structural Journal*, Vol. 87, No.6, Dec. 1990, pp.623-629.
84. Rangan, B. V., and Hall, A. S., "Moment and Shear Transfer Between Slab and Edge Columns," *ACI Structural Journal*, Vol. 80, No.3, May-June 1983, pp. 183-191.
85. Rankin, G. I. B., and Long, A. E., "Predicting the Punching Strength of Conventional Slab-Column Specimens," *Proceedings, Institution of Civil Engineers*, Part I, Vol. 82, Dec. 1987, pp. 1165-1186.
86. Regan, P. E., "Symmetrical Punching of Reinforced Concrete Slabs," *Magazine of Concrete Research*, Vol. 38, No. 136, Sept. 1986, pp. 115-128.
87. Regan, P.E., "Behavior of Reinforced Concrete Flat Slabs," *CIRIA Report* No.89, Construction Industry Research and Information Association, London, 1981.
88. Regan, P. E., and Braestrup, M. W., "Punching Shear in Reinforced Concrete – *A State of Art Report*," Jan. 1985, 232pp.
89. Regan, P. E.; Al-Hussaini, A.; Ramdane, K. E.; Xue, H. Y., "Behaviour of High Strength Concrete and Its Effect on the Shear Strength of Longitudinally Reinforced Concrete Members," *Utilization of High Strength Concrete, Proceedings of the Third International Symposium*, Vol.1, Norwegian Concrete Association, Oslo, June 1993, pp. 269-276.
90. Ritchie, M., and Ghali, A., "Seismic-Resistant Connections of Edge Columns with Prestressed Slabs," *ACI Structural Journal*, Vol. 102, No. 32, March 2005, pp. 256-275.

References

91. Robertson, I.N.; Kawai, T.; Lee, J.; Enomoto, B., "Cyclic Testing of Slab-Column Connections with Shear Reinforcement," *ACI Structural Journal*, Vol. 99, No.5, Oct. 2002, pp.605-613.
92. Robertson, I.N. and Durrani, A.J., "Gravity Load Effect on Seismic Behavior of Interior Slab-Column Connections," *ACI Structural Journal*, V. 89, No. 3, Feb. 1992, pp.37-45.
93. Robertson, I.N. and Durrani, A.J., "Seismic Response of Connections in Indeterminate Flat-Slab Subassemblies," Structural Research at Rice, *Report* No. 41, Department of Civil Engineering, Rice University, Houston, July 1990, 266pp.
94. Robertson, I.N. and Durrani, A.J., "Gravity Load Effect on Seismic Behavior of Exterior Slab-Column Connections," *ACI Structural Journal*, Vol. 88, No. 3, June 1991, pp.255-267.
95. Roll, F.; Saidi, S. T. H.; Sabnis, G.; Chuang, K., "Shear Resistance of Perforated Reinforced Concrete Slabs," *Crack, Deflection and Ultimate Load of Concrete Slab System*, SP-30, American Concrete Institute, pp. 77-102.
96. Saatcioglu, M., "Cracking in Concrete Structures During the August 17, 1999 Earthquake in Turkey," *ACI Special Publication*, SP 204-13, 2001, pp. 127-142.
97. Scavuzzo, L., "Shear Reinforcement at Slab-Column Connections in a Reinforced Concrete Flat Plate Structure," The Royal Military College, Kingston, 1978.
98. Shehata, I. A. E. M., and Regan, P. E., "Punching in R.C. Slabs," *Journal of Structural Engineering*, Vol. 115, No.7, July 1989, pp. 1726-1740.

References

-
99. Sherif, A.G. and Dilger, W.H., "Tests of Full-Scale Continuous Reinforced Concrete Flat Slabs," *ACI Structural Journal*, Vol. 97, No. 3, June 2000, pp.455-467.
100. Simmonds, S. H., and Alexander, S. D. B., "Truss Model for Edge-Column Slab Connections," *ACI Structural Journal*, Vol. 84, No.4, July-Aug. 1987, pp. 296-303.
101. Sozen, M.A., 1980, "Review of Earthquake Response of Reinforced Concrete Buildings With a View to Drift Control, State-of-the-Art in Earthquake Engineering," *7th World Conference on Earthquake Engineering*, Istanbul, pp. 119-174.
102. Stamenkovic, A. and Chapman, J.C., "Local Strength at Column Heads in Flat Slabs Subjected to A Combined Vertical and Horizontal Loading," *Proceedings, Institution of Civil Engineers* (London), Part 2, V. 57, June 1974, pp. 205-232.
103. Suwita, M.G.; Irawan, P.; Teng, S., "Strength of Slab-Column Connections with Biaxial Unbalanced Moments," *Final Report, A BCA-NTU Joint Research on Flat-Plate Structures, Phase-1B*, January 2001, 158pp.
104. Symmonds, D.W.; Mitchell, D.; Hawkins, N.M., "Slab-Column Connections Subjected to High Intensity Shear and Transferring Reversed Moments," *Progress Report on NSF Project GI-38717*, Department of Civil Engineering, University of Washington, Seattle, 1976.
105. Talbot, A. N., "Reinforced Concrete Wall Footings and Column Footings," Bulletin No. 67, University of Illinois Engineering Experiment Station, March, 1913, 114 pp.
-

References

106. Tan, Y., and Teng, S., "Interior Slab-Rectangular Column Connections Under Biaxial Lateral Loading," *ACI Special Publication*, SP-232-9, Oct 2005, pp.147-174.
107. Taylor, R., and Hayes, B., "Some Tests on the Effect of Edge Restraint on Punching Shear in Reinforced Concrete Slab," *Magazine of Concrete Research*, Vol. 17, No. 50, Mar 1965, pp. 39-44.
108. Teng, S.; Cheong, H.K.; Kuang, K.L.; Geng J.Z., "Punching shear strength of Slabs with openings and supported on rectangular columns," *ACI Structural Journal*, V. 101, No.67, Oct. 2004, pp. 678-687.
109. Van Dusen, M. H., "Unbalanced Moments and Shear Transfer at Slab-Column Connections: A Literature Review," *M.Eng. Thesis*, University of Toronto, Toronto, Ontario, 125pp.
110. Vanderbilt, M. D., "Shear Strength of Continuous Plates," *Journal of Structural Division*, ASCE, Vol. 98, No. ST5, May 1972, pp. 961-973.
111. Walker, P.R. and Regan, P.E., "Corner Column-Slab Connections in Concrete Flat Plates," *Journal of Structural Engineering*, Vol. 113, No. 4, April 1987, pp.704-720.
112. Wey, E., and Durrani, A., "Seismic Response of Interior Slab-Column Connections with Shear Capitals," *ACI Structural Journal*, Vol. 89, No. 6, Dec. 1992, pp. 682-691.
113. Widjaja, S., and Teng, S., "Corner Slab-Rectangular Column Connections Under Biaxial Cyclic Lateral Loading," *Final Report*, A BCA-NTU Joint Research on Flat Plate Structures, Phase IIIB, 2006, 250pp.

References

114. Zaghlool, E.R.F., "Strength and Behavior of Corner and Edge Column-Slab Connections in Reinforced Concrete Flat Plates," *PhD Thesis*, Department of Civil Engineering, The University of Calgary, Calgary, 1971, 366pp.
115. Zaghlool, E.R.F.; Paiva, H.A.R.; Glockner, P.G., "Tests of Flat-Plate Corner-Slab Connections," *Journal of the Structural Division*, Proceedings of the American Society of Civil Engineers, Vol. 99, No. ST3, Mar. 1973 (2), pp.551-572.
116. Zaghlool, E.R.F.; Paiva, H. A. R.; Glockner, P.G., "Tests of Reinforced Concrete Flat Plate Floors," *Journal of the Structural Division*, Proceedings of the American Society of Civil Engineers, Vol. 96, No. ST3, Mar 1970, pp.487-507.
117. Zee, H.L. and Moehle, J.P., "Behavior of Interior and Exterior Flat Plate Connections subjected to Inelastic Load Reversals," *Report No. UVC/EERC-84/07*, College of Engineering, University of California at Berkeley, Aug. 1984, 130pp.