UNIVERSITY OF CINCINNATI

Date: _____

I, _____,
hereby submit this work as part of the requirements for the degree of:

in:

It is entitled:

This work and its defense approved by:

Chair: _____

THE NEXT GENERATION OF COUPLING BEAMS

A Dissertation Submitted to the
Division of Research and Advanced Studies
of the University of Cincinnati

in Partial Fulfillment of the
Requirements for the Degree of
DOCTOR OF PHILOSOPHY

In the Department of Civil and Environmental Engineering
of the College of Engineering

2005

By

Patrick J. Fortney

B.S.C.E. University of Cincinnati, 2002

Dissertation Committee

Dr. Bahram M. Shahrooz, Chair

Dr. Gian A. Rassati

Dr. Kent A. Harries

Dr. T. Michael Baseheart

Mr. Paul Brien

ACKNOWLEDGMENTS

First, I would like to thank my advisor Dr. Bahram M. Shahrooz whose advise and constant support during my graduate tenure at the University of Cincinnati was invaluable. His seemingly endless knowledge, expertise, and creativity made this work possible and meaningful. Heartfelt thanks are given to Dr. Gian A. Rassati who selflessly committed his time and analytical expertise to this project. His unwavering encouragement will not be forgotten. Special thanks are also extended to Dr. Kent A. Harries for his expert input and genuine interest in the topics of this research in addition to demonstrating a commitment to producing meaningful work. In addition, sincere thanks and appreciation is extended to Mr. Paul Brien whose practical engineering expertise and participation in this research was both necessary and appreciated.

Genuine appreciation and thanks are given to Dr. James A. Swanson and Dr. Michael T. Baseheart for their continued advice and support and for those intangibles too many to be enumerated. Special thanks are extended to late Dr. Frank E. Weisgerber for the endless hours of discussions pertaining to engineering and research in general. He will not be forgotten.

I would like to express my gratitude to the numerous corporations and individuals whose donations of materials and assistance made the experimental phase of the research project possible. Special recognition is extended to Baker Construction, Inc., MMFX Steel Corp., and Goettle Construction Co. for their generous contributions in material and labor. Additionally, my heartfelt thanks and appreciation is given to Mr. Hugh

Shoemaker for the many months of work, assistance, and support selflessly contributed to the experimental phase of this research project.

Finally, I would like to express my thanks and appreciation to my wife Karen for her faith in me and the support provided over these past years. Relying on Karen's understanding, strength, and never-ending will power made the completion of my graduate studies and research work possible.

THE NEXT GENERATION OF COUPLING BEAMS

Patrick J. Fortney

ABSTRACT

A collection of experimental, parametric, and analytical studies was carried out to further the understanding of the behavior and constructability of coupled core wall (CCW) systems. Issues related to the design of wall piers in CCW systems and the beams used to couple the wall piers, structural response of the overall system, and construction difficulties arising from reinforcing steel congestion were explored. The research presented in this document focuses on a design approach of coupled core wall systems which concentrates on minimizing steel congestion while maintaining satisfactory structural behavior.

Several different configurations of coupled core wall systems were examined. The experimental portion of this research entailed reverse cyclic testing of representative subassemblies consisting of a coupling beam connecting two wall piers. The subassemblies are considered to be approximately $\frac{1}{2}$ scale of a typical coupled core wall structure and the four different types of beams tested are: (1) a diagonally-reinforced concrete coupling beam (DBCB), (2) an unencased steel beam (SCB), (3) an unencased steel beam with a steel fuse link located at mid span of the beam (FCB1 and FCB2) and, (4) a composite steel/concrete beam utilizing a vertical web shear plate with headed shear studs (SPCB). Specimens (1) and (2) are types of coupling beams common to CCW structures. Specimens (3) and (4) are new proposed types of coupling beams.

Testing was performed by installing the test specimen within two load frames provided with actuators used to apply load to the specimen. One wall pier was “clamped” to a reinforced concrete pedestal while the coupling beam and other wall pier was cantilevered beneath the load frames’ actuators. Constant axial forces were imparted to the wall piers by virtue of pre-tensioned rods. These forces resisted the overturning forces during testing. The cantilevered wall pier was then loaded/displaced in cycles of increasing loads and/or displacements to impart shear force to the coupling beam. For these tests, effects of horizontal shear in the wall piers was neglected.

The SCB test was performed as a control by which the other specimens would be measured. The main test goals for the DBCB specimen were (a) measuring behavior of a diagonally-reinforced concrete coupling beam with a practical length-to-depth ratio (in this case equal to 2.5) and (b) effect of lack of confinement steel around the diagonal bar groups (a minimum number of stirrups were provided as needed to hold the bars in place during construction). The main test goals of the FCB1 and FCB2 specimens were to investigate the potential for developing localized damage in the fuse to minimize post-damage repair and/or replacement of the coupling beam. The main goal for the SPCB was to evaluate the proposed design methodology for the new section, and its potential as a viable coupling beam alternative.

The steel coupling beam (SCB) demonstrated superior stiffness and energy dissipation characteristics over all the other coupling beams. The confined core of the DBCB remained intact through 3% rotation, suggesting that at rotational demands of 3% or less it is not necessary to provide ACI-compliant confining steel to the diagonal bar groups. The fuse coupling beam demonstrated stable hysteretic behavior similar to that of

the SCB specimen, and yielding of the fuse web was achieved while the main section of the beam remained elastic. The SPCB test failed to achieve sufficient composite action between the steel web plate and surrounding concrete. Further work is necessary to better engage the steel plate.

Parametric analytical studies were also conducted to supplement the experimental studies. A parametric study pertaining to practical designs of diagonally-reinforced concrete coupling beams shows that achieving a practical diagonally-reinforced coupling beam is difficult, if not impossible. For most combinations of realistic span-to-depth ratios and service shear stress levels, the required area of steel needed for the diagonal bar groups either creates significant construction difficulties or is simply impossible to achieve.

A parametric study pertaining to the design of wall piers shows that the current methods for determining the need for special boundary elements permitted by ACI 318-02 are not applicable to wall piers in coupled core wall systems. An alternative method for determining the need for special boundary elements in wall piers in coupled core wall systems is proposed. The proposed method recommends that the need for special boundary elements be determined based on maximum concrete compressive strains expected at design loads. It is demonstrated that for a family of coupled core wall configurations special boundary elements are not required.

To investigate the overall structural behavior of coupled core wall systems with wall piers and coupling beams designed based on the recommendations of this research, nonlinear time-history analyses were performed on two prototype structures. Both structures had the same plan dimensions, number of stories, slab thickness, wall

thickness, and floor-to-floor heights. The first structure was designed using a steel coupling beam, and the second was designed using a diagonally-reinforced concrete coupling beam. The need for special boundary elements in the wall piers was based on maximum concrete compressive strains at design loads. Properties of the coupling beams used in the analyses of the two structures were based on measured behavior obtained during the experimental phase of the research. The analyses show that the steel coupling beam is superior to the diagonally reinforced coupling beam in terms of energy dissipation and drift control. However, the DBCB building satisfies code requirements. It should be noted that analyses did not consider the effects of shear on the axial and flexural load-carrying capacities of wall piers with no special boundary elements. Additional studies are, therefore, needed before the presented conclusions can be generalized.

TABLE OF CONTENTS

LIST OF TABLES	IV
LIST OF FIGURES.....	VI
NOTATIONS	X
CHAPTER 1: INTRODUCTION AND PREVIOUS RELEVANT RESEARCH	1
1.1 INTRODUCTION.....	1
1.2 STRUCTURAL BEHAVIOR OF COUPLED CORE WALL SYSTEMS	1
1.3 PREVIOUS RELEVANT RESEARCH	2
1.3.1 Steel Coupling Beams.....	3
1.3.2 Concrete-Steel Composite Coupling Beams.....	5
1.3.3 Diagonally Reinforced Concrete Coupling Beams.....	6
1.3.4 Web Shear Plate Coupling Beam.....	7
1.3.5 Wall Pier Reinforcement	8
1.4 OBJECTIVES AND SCOPE.....	8
1.4.1 Objectives	8
1.4.2 Scope	9
CHAPTER 2: COUPLING BEAM DESIGN PROCEDURES	18
2.1 COUPLING BEAMS	18
2.1.1 COUPLING BEAM PARAMETERS	18
2.2 COMMON COUPLING BEAM.....	18
2.2.1 Design Procedure for the Unencased Steel Coupling Beam.....	18
2.2.2 Design Procedure for the Diagonally Reinforced Concrete Beam	23
2.3 NEW PROPOSED COUPLING BEAMS	25
2.3.1 Design Procedure for the Fuse Steel Coupling Beam.....	27
2.3.2 Design Procedure for the Shear Plate Coupling Beam	30
2.3.2.1 Derivation of Design Equation.....	30
2.3.2.2 Detailed Design Procedure.....	33
CHAPTER 3: EXPERIMENTAL TESTING.....	45
3.1 TEST SETUP DESCRIPTION.....	45
3.1.1 Design Procedure for the Test Specimen Wall Piers	47
3.2 INSTRUMENTATION AND DATA ACQUISITION	50
3.2.1 Control and External Instrumentation.....	50
3.2.2 Regional and local Instrumentation	51
3.3 LOADING SEQUENCES	51
3.4 TEST RESULTS.....	51
3.4.1 Measured Material Properties.....	51
3.4.2 Steel Coupling Beam (SCB) Test Results	52
3.4.3 Diagonally Reinforced Concrete Coupling Beam (DBCB) Test Results	53
3.4.3.1 Overstrength of Diagonally Reinforced Coupling Beam	54
3.4.4 Shear Plate Coupling Beam (SPCB) Test Results	56
3.4.5 Fuse Steel Coupling Beam (FCB) Test Results	58
3.5 COMPARISON OF TEST RESULTS.....	60
CHAPTER 4: DESIGN OF PRACTICAL DIAGONALLY REINFORCED COUPLING BEAMS	108
4.1 INTRODUCTION.....	108
4.2 RESEARCH OBJECTIVE	109
4.3 ACI 318 PROVISIONS FOR THE DESIGN OF DBCBS	109
4.4 DEPTH OF DIAGONAL ELEMENT, H_x	111
4.5 ANGLE OF INCLINATION, A	111

4.6	WIDTH OF DIAGONAL ELEMENT, B_x	112
4.7	DCB SHEAR STRESS	113
4.8	DESIGN OF CCWS	113
4.9	EXAMPLES OF DBCB DESIGN	114
4.10	RECOMMENDATIONS	117
4.11	IMPLICATIONS FOR PERFORMANCE BASED DESIGN.....	119
4.12	CONCLUSIONS.....	120
4.13	EPILOGUE.....	120
CHAPTER 5: WALL BOUNDARY ELEMENTS STUDY		127
5.1	INTRODUCTION.....	128
5.2	OBJECTIVE OF PARAMETRIC STUDY	130
5.3	PARAMETERS OF STUDY.....	130
5.4	STRUCTURAL ANALYSIS AND CALCULATED DESIGN LOADS	131
5.4.1	General Analysis and Design Procedures.....	131
5.4.2	Analysis of Structures.....	133
5.4.3	Design of the Structures.....	135
5.4.3.1	Coupling Beam Design	136
5.4.3.2	Design of Wall Pier Longitudinal Steel	136
5.5	DETERMINING WHETHER SPECIAL BOUNDARY ELEMENTS ARE REQUIRED	137
5.5.1	Stress Calculations Based on ACI 318-02 §21.7.6.3	137
5.5.2	Compression Block Depth Calculations Based on ACI 318-02 §21.7.6.2	139
5.5.3	Discussion.....	140
5.6	COMPARISON OF WALL BOUNDARY DETAILING	142
5.6	CONCLUSIONS	145
CHAPTER 6: NONLINEAR RESPONSE-HISTORY ANALYSIS.....		157
6.1	STRUCTURE PARAMETERS.....	158
6.2	ELASTIC ANALYSIS AND RESULTS	158
6.2.1	Coupling Beam Design.....	159
6.2.2	Wall Pier Design.....	160
6.2.2.1	Wall Overstrength Factor	161
6.3	NONLINEAR MODELING AND PARAMETERS	162
6.3.1	Determining Beam Hysteretic Rules and Parameters	162
6.3.1.1	Steel Coupling Beam Hysteresis	162
6.3.1.2	Diagonally Reinforced Coupling Beam Hysteresis.....	164
6.3.1.3	Wall Pier Properties	165
6.3.2	Nonlinear Modeling.....	169
6.4	NONLINEAR ANALYSIS.....	171
6.5	ANALYSES RESULTS.....	171
6.5.1	Base Shears.....	172
6.5.2	Coupling Beam Shear Demand.....	173
6.5.3	Wall Pier Demands	175
6.5.4	Interstory/Roof Drift and Beam Rotation Demands	178
6.6	DISCUSSION	180
CHAPTER 7: CONCLUSIONS AND RECOMMENDATIONS		242
7.1	SUMMARY	242
7.2	CONCLUSIONS	247
7.3	RECOMMENDATIONS	249
REFERENCES.....		251
APPENDIX A: COUPLING BEAM DAMAGE RECORDS AND MISCELLANEOUS TEST MEASUREMENTS		256
APPENDIX B: EXAMPLE CALCULATIONS USING THE ELF PROCEDURE.....		283

APPENDIX C: WALL BOUNDARY ELEMENT STUDY RESULTS.....	299
APPENDIX D: WALL BOUNDARY DETAILING EXAMPLE PROBLEM	340

LIST OF TABLES

TABLE 3.1. INSTRUMENTATION FOR TEST SCB. REFER TO FIGURES 3.1(A) AND 3.1(B).....	64
TABLE 3.2. INSTRUMENTATION FOR TEST DBCB. REFER TO FIGURES 3.2(A) AND 3.2(B).	65
TABLE 3.3. INSTRUMENTATION FOR TEST SPCB. REFER TO FIGURES 3.3(A) AND 3.3(B).....	66
TABLE 3.4. INSTRUMENTATION FOR TEST FCB-2. REFER TO FIGURES 3.4(A) AND 3.4(B).	67
TABLE 3.5. INSTRUMENTATION FOR TEST FCB-1. REFER TO FIGURES 3.5(A) AND 3.5(B).	68
TABLE 3.6. LOAD HISTORY DATA FOR SCB TEST. REFER TO FIGURES 3.6(A) AND 3.6(B).	69
TABLE 3.7. LOAD HISTORY DATA FOR DBCB TEST. REFER TO FIGURES 3.7(A) AND 3.7(B).....	70
TABLE 3.8. LOAD HISTORY DATA FOR SPCB TEST. REFER TO FIGURES 3.8(A) AND 3.8(B).....	71
TABLE 3.9. LOAD HISTORY DATA FOR FCB-2 TEST. REFER TO FIGURES 3.9(A) AND 3.9(B).....	72
TABLE 3.10. LOAD HISTORY DATA FOR FCB-1 TEST. REFER TO FIGURES 3.10(A) AND 3.10(B).....	73
TABLE 3.11. MEASURED STEEL MATERIAL PROPERTIES.	73
TABLE 3.11. MEASURED STEEL MATERIAL PROPERTIES.	74
TABLE 3.12. CONCRETE CYLINDER TEST DATA.	74
TABLE 3.13(A). STRAIN DATA AT FIRST YIELD IN WEB OF SCB (SEE FIGURE 3.3 FOR GAGE NUMBERS).	75
TABLE 3.13(B). STRAIN DATA AT FINAL PEAK FORCE AT LAST CYCLE OF SCB TEST (SEE FIGURE 3.3 FOR GAGE NUMBERS).....	75
TABLE 3.14(A). STRAIN DATA AT FIRST YIELD IN DIAGONAL BARS OF DBCB TEST (SEE FIGURE 3.4 FOR GAGE NUMBERS).....	76
TABLE 3.14(B). STRAIN DATA AT FIRST YIELD IN LONGITUDINAL BARS OF DBCB TEST (SEE FIGURE 3.4 FOR GAGE NUMBERS).	76
TABLE 3.14(C). STRAIN DATA AT FINAL PEAK FORCE DURING LAST CYCLE OF DBCB TEST (SEE FIGURE 3.3 FOR GAGE NUMBERS).	77
TABLE 3.15(A). STRAIN DATA AT END OF 1.5% SHEAR ANGLE CYCLES FOR SPCB TEST (POINT OF LOSS OF COMPOSITE ACTION) (SEE FIGURE 3.5 FOR GAGE NUMBERS).	77
TABLE 3.15(B). STRAIN DATA AT FINAL PEAK FORCE OF LAST CYCLE TESTED FOR SPCB TEST (SEE FIGURE 3.5 FOR GAGE NUMBERS).	77
TABLE 3.15(B). STRAIN DATA AT FINAL PEAK FORCE OF LAST CYCLE TESTED FOR SPCB TEST (SEE FIGURE 3.5 FOR GAGE NUMBERS).	78
TABLE 3.16(A). STRAIN DATA FIRST YIELD IN WEB OF FUSE OF THE 50% FUSE (FCB-2 TEST) (SEE FIGURE 3.6 FOR GAGE NUMBERS).	78
TABLE 3.16(B). STRAIN DATA AT THE END OF THE 50% FUSE (FCB-2 TEST) (SEE FIGURE 3.6 FOR GAGE NUMBERS).....	79
TABLE 3.17(A). STRAIN DATA FIRST YIELD IN THE WEB OF THE 70% FUSE (FCB-1 TEST) (SEE FIGURE 3.7 FOR GAGE NUMBERS).	79
TABLE 3.17(B). STRAIN DATA AT THE FINAL PEAK FORCE IN THE LAST CYCLE OF TEST FCB-1 (70% FUSE) (SEE FIGURE 3.7 FOR GAGE NUMBERS).....	80
TABLE 3.18. CALCULATED VERSUS MEASURED SHEAR CAPACITIES.	80
TABLE 4.1. CAPACITY OF DBCBs REPORTED IN LITERATURE.	122
TABLE 4.2. ASSUMPTIONS AND PARAMETERS USED TO DESIGN DBCBs (SEE FIGURE 4.1).	122
TABLE 5.1. SUPERIMPOSED GRAVITY LOADS.....	147
TABLE 5.2. PLAN DIMENSIONS OF STRUCTURES (REFER TO FIGURE 5.1).....	147
TABLE 5.3. SEISMIC RESPONSE COEFFICIENTS, WEIGHTS AND BASE SHEARS.	147
TABLE 5.4. ELF CALCULATED LATERAL FORCES AND STORY SHEARS.....	148
TABLE 5.5. REDUNDANCY AND WALL OVERSTRENGTH FACTORS.	148
TABLE 5.6. COUPLING BEAM DESIGN SHEAR DEMANDS AND FACTORED CAPACITIES.....	149
TABLE 5.7. WALL PIER DESIGN FORCES FOR STRUCTURE_01.	149
TABLE 5.8. BEAM DIMENSIONS.	150
TABLE 5.9. WALL PIER LONGITUDINAL STEEL REINFORCEMENT.	150
TABLE 5.10. EXTREME FIBER CONCRETE COMPRESSIVE STRESSES AND STRAINS.....	151
TABLE 6.1. SUPERIMPOSED GRAVITY LOADS.....	186
TABLE 6.2. SEISMIC RESPONSE COEFFICIENTS, DYNAMIC WEIGHTS, AND BASE SHEARS (BOTH BUILDINGS).	186
TABLE 6.3. LATERAL FORCES FROM ELF PROCEDURE (BOTH BUILDINGS).....	187
TABLE 6.4. REDUNDANCY AND WALL OVERSTRENGTH FACTORS (BOTH BUILDINGS).	187

TABLE 6.5. BEAM DESIGN SHEARS AND FACTORED CAPACITIES (BOTH BUILDINGS).	187
TABLE 6.6. EFFECTIVE MOMENT OF INERTIA OF DBCBs USED IN ELASTIC ANALYSIS.	188
TABLE 6.7. WALL PIER DESIGN FORCES FROM ELASTIC ANALYSIS.	189
TABLE 6.8. WALL LONGITUDINAL REINFORCEMENT BASED ON ELASTIC DESIGN (BOTH BUILDINGS).	190
TABLE 6.9. ELASTIC AND INELASTIC EI SLOPES FOR ALL WALL PIERS OF THE SCB BUILDING. SEE M- ϕ PLOTS IN FIGURES 6.14-6.16.	190
TABLE 6.10. ELASTIC AND INELASTIC EI SLOPES FOR ALL WALL PIERS OF THE DBCB BUILDING. SEE M- ϕ PLOTS IN FIGURES 6.17-6.19.	191
TABLE 6.11. MAXIMUM BASE SHEARS IMPARTED TO BUILDINGS DURING GROUND MOTIONS.	192
TABLE 6.12. MAXIMUM INTERSTORY DRIFTS, ROOF DRIFTS, AND BEAM ROTATIONS IMPARTED TO BUILDINGS DURING GROUND MOTIONS.	192
TABLE 6.13. COMPARISON OF WALL PIER CONCRETE COMPRESSIVE STRAINS (ELASTIC ANALYSIS VS. NONLINEAR ANALYSIS).	193

LIST OF FIGURES

FIGURE 1.1. GENERAL FLOOR PLAN OF A COUPLED CORE WALL SYSTEM.	12
FIGURE 1.2. ELEVATION A-A SHOWN IN FIGURE 1.1.	12
FIGURE 1.3. REACTION MECHANISMS TO LATERAL LOADS OF (A) A COUPLED WALL PIER SYSTEM AND (B) AN UNCOUPLED WALL PIER SYSTEM.	13
FIGURE 1.4. TYPICAL UNENCASED STEEL COUPLING BEAM.	13
FIGURE 1.5. PLACEMENT OF AUXILIARY TRANSFER BARS AND FACE BEARING PLATES.	14
FIGURE 1.6. TYPICAL COMPOSITE CONCRETE/STEEL COUPLING BEAM.	15
FIGURE 1.7. TYPICAL DIAGONALLY-REINFORCED CONCRETE COUPLING BEAM.	16
FIGURE 1.8. PLATE-REINFORCED CONCRETE BEAM TESTED BY SUBEDI.	17
FIGURE 2.1. AS-BUILT DETAILS OF SPECIMEN SCB.	37
FIGURE 2.2. (A) SHEAR RESISTING MECHANISM OF A DIAGONALLY-REINFORCED CONCRETE COUPLING BEAM. (B) ACI MINIMUM SIZES OF DIAGONAL BAR GROUPS.	38
FIGURE 2.3. AS-BUILT DETAILS OF SPECIMEN DBCB.	39
FIGURE 2.4. SCHEMATIC USED FOR DETERMINING THE REQUIRED MOMENT OF INERTIA OF THE SPLICE SECTION IN THE FUSE COUPLING BEAM SPECIMENS.	40
FIGURE 2.5. TRIAL SPLICE CONNECTION FOR DESIGN OF FCB-1 AND FCB-2.	40
FIGURE 2.6. AS-BUILT DETAILS OF SPECIMENS FCB-1 AND FCB-2.	41
FIGURE 2.7. FBD OF A CONCRETE COUPLING BEAM WITH A CENTRALLY-LOCATED VERTICAL STEEL PLATE.	42
FIGURE 2.8. MOHR'S CIRCLE USED TO RELATE F_{yp} , τ , AND F_1 IN DERIVING DESIGN EQUATION FOR A REINFORCED CONCRETE BEAM WITH A CENTRALLY-LOCATED VERTICAL STEEL PLATE.	42
FIGURE 2.9. SHEAR AND MOMENT DISTRIBUTION OVER THE LENGTH A BEAM IN DOUBLE CURVATURE. THE MINIMUM EMBEDMENT LENGTH OF THE STEEL PLATE FOR SPECIMEN SPCB IS $L/2$	43
FIGURE 2.10. AS-BUILT DETAILS OF SPECIMEN SPCB.	44
FIGURE 3.1. SUMMARY OF COUPLING BEAM CROSS SECTIONS TESTED.	81
FIGURE 3.2(A). SOUTH ELEVATION OF TEST SETUP.	82
FIGURE 3.2(B). SOUTH ELEVATION OF TEST SETUP WITH A SPECIMEN INSTALLED.	83
FIGURE 3.3(A). STRAIN GAGE INSTRUMENTATION FOR SCB TEST ($1IN=25.4MM$).	84
FIGURE 3.3(B). FORCE, DISPLACEMENT, AND ROTATION TRANSDUCERS FOR SCB TEST ($1IN=25.4MM$; $1KIP=4.448KN$).	84
FIGURE 3.4(A). STRAIN GAGE INSTRUMENTATION FOR DBCB TEST ($1IN=25.4MM$).	85
FIGURE 3.4(B). FORCE, DISPLACEMENT, AND ROTATION TRANSDUCERS FOR DBCB TEST ($1IN=25.4MM$; $1KIP=4.448KN$).	86
FIGURE 3.5(A). STRAIN GAGE INSTRUMENTATION FOR SPCB TEST ($1IN=25.4MM$).	86
FIGURE 3.5(B). FORCE, DISPLACEMENT, AND ROTATION TRANSDUCERS FOR SPCB TEST ($1IN=25.4MM$; $1KIP=4.448KN$).	87
FIGURE 3.6(A). STRAIN GAGE INSTRUMENTATION FOR 50% FUSE (FCB-2) ($1IN=25.4MM$).	88
FIGURE 3.6(B). FORCE, DISPLACEMENT, AND ROTATION TRANSDUCERS FOR FCB-2 TEST (50% FUSE) ($1IN=25.4MM$; $1KIP=4.448KN$).	88
FIGURE 3.7(A). STRAIN GAGE INSTRUMENTATION FOR 70% FUSE (FCB-1) ($1IN=25.4MM$).	89
FIGURE 3.7(B). FORCE, DISPLACEMENT, AND ROTATION TRANSDUCERS FOR FCB-1 TEST (70% FUSE) ($1IN=25.4MM$; $1KIP=4.448KN$).	89
FIGURE 3.8(A). FIRST PHASE OF LOAD HISTORY IN FORCE CONTROL (UP TO $\pm 71KN$) FOR TEST SCB.	90
FIGURE 3.8(B). SECOND PHASE OF LOAD HISTORY IN DISPLACEMENT CONTROL (CYCLIC TO $\pm 36.6MM$, MONOTONIC TO $-100.6MM$) FOR TEST SCB.	90
FIGURE 3.9(A). FIRST PHASE OF LOAD HISTORY IN FORCE CONTROL (UP TO $\pm 89KN$) FOR TEST DBCB.	91
FIGURE 3.9(B). SECOND PHASE OF LOAD HISTORY IN DISPLACEMENT CONTROL (UP TO $\pm 54.9MM$) FOR TEST DBCB.	91
FIGURE 3.10(A). FIRST PHASE OF LOAD HISTORY IN FORCE CONTROL (UP TO $\pm 89KN$) FOR TEST SPCB.	92
FIGURE 3.10(B). SECOND PHASE OF LOAD HISTORY IN DISPLACEMENT CONTROL (UP TO $\pm 54.9MM$) FOR TEST SPCB.	92
FIGURE 3.11(A). FIRST PHASE OF LOAD HISTORY IN FORCE CONTROL (UP TO $\pm 89KN$) FOR TEST FCB-2.	93
FIGURE 3.11(B). SECOND PHASE OF LOAD HISTORY IN DISPLACEMENT CONTROL (UP TO $\pm 18.3MM$) FOR TEST FCB-2.	93

FIGURE 3.12(A). FIRST PHASE OF LOAD HISTORY IN FORCE CONTROL (UP TO $\pm 89kN$) FOR TEST FCB-1.....	94
FIGURE 3.12(B). SECOND PHASE OF LOAD HISTORY IN DISPLACEMENT CONTROL (UP TO $\pm 100.6mm$) FOR TEST FCB-1.	94
FIGURE 3.13. CONCRETE CYLINDER COMPRESSIVE STRENGTH V. TIME.....	95
FIGURE 3.14. NORMALIZED SHEAR VS. SHEAR ANGLE FOR SCB TEST.	95
FIGURE 3.15. FORCE-DEFLECTION FOR SCB TEST.	96
FIGURE 3.16. NORMALIZED SHEAR VS. SHEAR ANGLE FOR DBCB TEST.	96
FIGURE 3.17. PHOTOGRAPH OF CONDITION OF DBCB AT END OF 3% ROTATION CYCLES.....	97
FIGURE 3.18. FORCE-DEFLECTION FOR DBCB TEST.	97
FIGURE 3.19. PHOTOGRAPH OF BUCKLED DIAGONAL BARS DURING 5% ROTATION CYCLES.	98
FIGURE 3.20. FREE BODY DIAGRAM OF A DBCB SHOWING DIAGONAL BAR STEEL, COMPRESSION STRUT, AND STIRRUP FORCES.	98
FIGURE 3.21. PHOTOS TAKEN OF SPCB TEST AT THE END OF THE 5% SHEAR ANGLE CYCLES. EVIDENCE OF THE STEEL PLATE HAS BEEN SLIDING RELATIVE TO THE CONCRETE.....	99
FIGURE 3.22. FORCE-DEFLECTION OF SPCB TEST.	99
FIGURE 3.23. SHEAR VS. SHEAR STRAIN PLOTTED UP TO THE END OF THE 1% SHEAR ANGLE CYCLES.....	100
FIGURE 3.24. SHEAR VS. SHEAR STRAIN PLOTTED UP TO THE END OF THE 1.5% SHEAR ANGLE CYCLES.....	100
FIGURE 3.25. SHEAR VS. SHEAR STRAIN PLOTTED UP TO END OF 3% SHEAR ANGLE CYCLES.	101
FIGURE 3.26. PRINCIPAL STRAINS IN WEB OF SPCB TEST.	101
FIGURE 3.27. NORMALIZED SHEAR VS. SHEAR ANGLE FOR FCB-2 TEST.....	102
FIGURE 3.28. PRINCIPAL STRAINS IN WEBS OF FUSE AND MAIN SECTIONS FOR FCB-2 (50% FUSE).....	102
FIGURE 3.29. FORCE-DEFLECTION FOR FCB-2 (50% FUSE).	103
FIGURE 3.30. NORMALIZED SHEAR VS. SHEAR ANGLE FOR FCB-1 TEST (70% FUSE).....	103
FIGURE 3.31. FORCE-DEFLECTION FOR FCB-1 TEST (70% FUSE).....	104
FIGURE 3.32. WEB SPLICE CONNECTION AT END OF 4% ROTATION CYCLES OF THE FCB-1 TEST (70% FUSE). EVIDENCE OF THE WEB SPLICE CONNECTION SLIPPING IS OBSERVED.....	104
FIGURE 3.33. PHOTO OF FCB-1 (70% FUSE) AT THE END OF THE 5% ROTATION CYCLES. WEB/FLANGE WELDS ARE FRACTURED IN THE END OF THE MAIN BEAM NEAR THE SPLICE CONNECTION.....	105
FIGURE 3.34. PEAK-TO-PEAK STIFFNESS OF STEEL BEAMS UP TO 4% ROTATION.....	105
FIGURE 3.35. PLASTIC ENERGY ACCUMULATION OF STEEL BEAMS UP TO 4% ROTATION.	106
FIGURE 3.36. TOTAL ENERGY DISSIPATED BY ALL SPECIMENS UP TO 4% ROTATION.	106
FIGURE 3.37. PEAK-TO-PEAK STIFFNESS OF CONCRETE BEAMS UP TO 4% ROTATION.....	107
FIGURE 3.38. PLASTIC ENERGY DISSIPATION OF CONCRETE BEAMS UP TO 4% ROTATION.	107
FIGURE 4.1. DIAGONAL COUPLING BEAM DETAILS.	123
FIGURE 4.2. DIAGONAL COUPLING BEAM DETAILS SATISFYING ACI 318-02 HAVING $F_c' = 5000psi (34.5MPa)$	124
FIGURE 4.3. DIAGONAL COUPLING BEAM DETAILS SATISFYING ACI 318-02 HAVING $F_c' = 8000psi (55.2MPa)$	125
FIGURE 4.4. EXAMPLE OF DBCB HAVING BUNDLED #11 BARS AND $d_v/h = 3$	126
151	
TABLE 5.11. C/LW RATIOS (RESULTS OF FIBER ANALYSES). LIMIT IS 0.24.....	152
FIGURE 5.1. (A) ONE-CELL TYPE CCW (B) TWO-CELL TYPE CCW. (DIMENSIONS GIVEN IN TABLE 1).	153
FIGURE 5.2. LOCATIONS OF INTEREST FOR STRESS CALCULATIONS AND REPORTED STRESSES AND STRAINS FOR (A) TENSION WALL, (B) COMPRESSION WALL, (C) I-SHAPED WALL, AND NOMENCLATURE USED IN EQUATIONS.	153
FIGURE 5.3. EXAMPLE OF ETABS MODEL. (A) FLOOR PLAN. (B) 3-D ELEVATION. STRUCTURE_02 IS SHOWN.	154
FIGURE 5.4(A). EXTREME FIBER CONCRETE STRESSES FOR STRUCTURE_01, LC1.	155
FIGURE 5.4(B). EXTREME FIBER CONCRETE STRAINS FOR STRUCTURE_05, LC1.	155
FIGURE 5.5. WALL BOUNDARY DETAILING OF THE BASE TENSION WALL PIER IN STRUCTURE_01 WHERE SPECIAL BOUNDARY ELEMENTS ARE ASSUMED TO BE (A) REQUIRED AND (B) NOT REQUIRED.	156
FIGURE 6.1. BUILDING PLAN DIMENSIONS.	194
FIGURE 6.2. FINAL DBCB DESIGNS.....	194
FIGURE 6.3. BEAM DEMAND AND CAPACITY DISTRIBUTIONS.....	195
FIGURE 6.4. BEAM HYSTERETIC RESPONSE COMPARISON OF (A) RAMBERG-OSGOOD AND (B) AL-BERMANI HYSTERESIS RULES.....	195

FIGURE 6.5. AL-BERMANI HYSTERESIS RULE. PARAMETERS SHOWN ARE FOR THE EXPERIMENTAL STEEL COUPLING BEAM.	196
FIGURE 6.6. HYSTERESIS CORRELATION FOR THE EXPERIMENTAL STEEL COUPLING BEAM.....	197
FIGURE 6.7. MODIFIED TAKEDA HYSTERESIS RULE. PARAMETERS SHOWN ARE FOR THE EXPERIMENTAL DIAGONALLY-REINFORCED COUPLING BEAM.	198
FIGURE 6.8. STRENGTH DEGRADATION FOR THE DIAGONALLY-REINFORCED COUPLING BEAM.....	199
FIGURE 6.9. HYSTERESIS CORRELATION FOR THE EXPERIMENTAL DIAGONALLY-REINFORCED CONCRETE BEAM.	199
FIGURE 6.10. P-M INTERACTION FOR WALL PIERS OF FLOORS 1-4 FOR SCB AND DBCB BUILDINGS.	200
FIGURE 6.11. P-M INTERACTIONS FOR WALL PIERS OF FLOORS 5-8 FOR SCB AND DBCB BUILDINGS.....	201
FIGURE 6.12. P-M INTERACTIONS FOR WALL PIERS OF FLOORS 9-20 FOR SCB AND DBCB BUILDINGS.....	202
FIGURE 6.13. QUADRATIC BEAM-COLUMN YIELD INTERACTION SURFACE AND CRITICAL POINT VALUES FOR ALL WALL PIERS FOR OF THE SCB AND DBCB BUILDINGS.	203
FIGURE 6.14. MOMENT-CURVATURE FOR TW AND CW OF SCB BUILDING AT FLOORS 1-4.....	204
FIGURE 6.15. MOMENT-CURVATURE FOR TW AND CW OF SCB BUILDING AT FLOORS 5-8.....	205
FIGURE 6.16. MOMENT-CURVATURE FOR TW AND CW OF SCB BUILDING AT FLOORS 9-20.....	206
FIGURE 6.17. MOMENT-CURVATURE FOR TW AND CW OF DBCB BUILDING AT FLOORS 1-4.	207
FIGURE 6.18. MOMENT-CURVATURE FOR TW AND CW OF DBCB BUILDING AT FLOORS 5-8.	208
FIGURE 6.19. MOMENT-CURVATURE FOR TW AND CW OF DBCB BUILDING AT FLOORS 9-20.	209
FIGURE 6.20. EQUIVALENT FRAME USED IN NONLINEAR MODEL.	210
FIGURE 6.21. STRENGTH DEGRADATION ASSIGNED TO WALL PIERS IN NONLINEAR MODEL.	211
FIGURE 6.22. RUAUMOKO SPRING MEMBER.....	211
FIGURE 6.23. ELEVATION OF NONLINEAR MODEL AND SPRING PARAMETERS (L1-4).	212
FIGURE 6.24. ELEVATION OF NONLINEAR MODEL AND SPRING PARAMETERS (L5-8).	213
FIGURE 6.25. ELEVATION OF NONLINEAR MODEL AND SPRING PARAMETERS (L9-12).	214
FIGURE 6.26. ELEVATION OF NONLINEAR MODEL AND SPRING PARAMETERS (L13-16).	215
FIGURE 6.27. ELEVATION OF NONLINEAR MODEL AND SPRING PARAMETERS (L17-20).	216
FIGURE 6.28. EL CENTRO GROUND MOTION RECORD.	217
FIGURE 6.29. NORTHRIDGE GROUND MOTION RECORD.....	217
FIGURE 6.30. ASCE7-02 10% IN 50 YEARS ARTIFICIAL GROUND MOTION RECORD.....	218
FIGURE 6.31. ASCE7-02 2% IN 50 YEARS ARTIFICIAL GROUND MOTION RECORD.....	218
FIGURE 6.32. SPECTRA FOR ALL GROUND MOTIONS.	219
FIGURE 6.33. BASE SHEARS FOR SCB BUILDING (ALL RECORDS).	220
FIGURE 6.34. BASE SHEARS FOR DBCB BUILDING (ALL RECORDS).	221
FIGURE 6.35. BEAM SHEAR DISTRIBUTION AT TIME OF MAXIMUM BASE SHEAR (SCB BUILDING - ALL RECORDS).	222
FIGURE 6.36. BEAM SHEAR DISTRIBUTION AT TIME OF MAXIMUM BASE SHEAR (DBCB BUILDING - ALL RECORDS).	223
FIGURE 6.37. BEAM SHEAR DISTRIBUTION VS. TIME (SCB BUILDING - EC RECORD).	224
FIGURE 6.38. BEAM SHEAR DISTRIBUTION VS. TIME (DBCB BUILDING - EC RECORD).	224
FIGURE 6.39. BEAM SHEAR DISTRIBUTION VS. TIME (SCB BUILDING - NR RECORD).	225
FIGURE 6.40. BEAM SHEAR DISTRIBUTION VS. TIME (DBCB BUILDING - NR RECORD).....	225
FIGURE 6.41. BEAM SHEAR DISTRIBUTION VS. TIME (SCB BUILDING - 10% IN 50 YEARS RECORD).	226
FIGURE 6.42. BEAM SHEAR DISTRIBUTION VS. TIME (DBCB BUILDING - 10% IN 50 YEARS RECORD).....	226
FIGURE 6.43. BEAM SHEAR DISTRIBUTION VS. TIME (SCB BUILDING - 2% IN 50 YEARS RECORD).	227
FIGURE 6.44. BEAM SHEAR DISTRIBUTION VS. TIME (DBCB BUILDING - 2% IN 50 YEARS RECORD).....	227
FIGURE 6.45. WALL PIER HYSTERETIC RESPONSES AT LEVELS 1, 5, AND 9 (SCB BUILDING - EC RECORD).	228
FIGURE 6.46. WALL PIER HYSTERETIC RESPONSES AT LEVELS 1, 5, AND 9 (DBCB BUILDING - EC RECORD).	229
FIGURE 6.47. WALL PIER HYSTERETIC RESPONSES AT LEVELS 1, 5, AND 9 (SCB BUILDING - NR RECORD).	230
FIGURE 6.48. WALL PIER HYSTERETIC RESPONSES AT LEVELS 1, 5, AND 9 (DBCB BUILDING - NR RECORD).	231
FIGURE 6.49. WALL PIER HYSTERETIC RESPONSES AT LEVELS 1, 5, AND 9 (SCB BUILDING - 10% IN 50 YEARS RECORD).	232
FIGURE 6.50. WALL PIER HYSTERETIC RESPONSES AT LEVELS 1, 5, AND 9 (DBCB BUILDING - 10% IN 50 YEARS RECORD).....	233

FIGURE 6.51. WALL PIER HYSTERETIC RESPONSES AT LEVELS 1, 5, AND 9 (SCB BUILDING – 2% IN 50 YEARS RECORD).	234
FIGURE 6.52. WALL PIER HYSTERETIC RESPONSES AT LEVELS 1, 5, AND 9 (DBCB BUILDING – 2% IN 50 YEARS RECORD).	235
FIGURE 6.53. INTERSTORY AND ROOF DRIFTS (SCB BUILDING).	236
FIGURE 6.54. INTERSTORY AND ROOF DRIFTS (DBCB BUILDING).	237
FIGURE 6.55. SCHEMATIC OF RELATIONSHIP BETWEEN ROOF DRIFT AND BEAM ROTATION.	238
FIGURE 6.56. DAMAGE TO DIAGONALLY-REINFORCED BEAM AT 1.5 % ROTATION.	238
FIGURE 6.57. DAMAGE TO DIAGONALLY REINFORCED BEAM AT 3% ROTATION.	238
FIGURE 6.58. DAMAGE TO DIAGONALLY REINFORCED BEAM AT 4% ROTATION.	239
FIGURE 6.59. COMPARISON OF WALL PIER DESIGN AXIAL FORCES AND MOMENTS AT WHICH CONCRETE COMPRESSIVE STRAINS ARE DETERMINED FROM ELASTIC ANALYSIS TO WALL PIER AXIAL FORCES AND MOMENTS GENERATED BY 10% IN 50 YEARS RECORD (SCB BUILDING).	240
FIGURE 6.60. COMPARISON OF WALL PIER DESIGN AXIAL FORCES AND MOMENTS AT WHICH CONCRETE COMPRESSIVE STRAINS ARE DETERMINED FROM ELASTIC ANALYSIS TO WALL PIER AXIAL FORCES AND MOMENTS GENERATED BY NORTHRIDGE RECORD (SCB BUILDING).	241
FIGURE A.1. LOAD WALL ROTATION DURING SCB TEST.	280
FIGURE A.2. LOAD WALL ROTATION DURING DBCB TEST.	280
FIGURE A.3. LOAD WALL ROTATION DURING SPCB TEST.	281
FIGURE A.4. LOAD WALL ROTATION DURING FCB TEST (50% FUSE).	281
FIGURE A.5. LOAD WALL ROTATION DURING FCB TEST (70% FUSE).	282

NOTATIONS

Chapter 2

A_s = Cross-sectional area of longitudinal reinforcing steel.

A_{sc} = Cross-sectional area of a shear stud connector.

A_v = Cross-sectional area of transverse reinforcement.

A_{vd} = Cross-sectional area of steel in one diagonal bar group in a diagonally reinforced concrete coupling beam.

C_c = Clear cover of concrete.

D = Height of the steel plate in a shear plate coupling beam.

E_c = Modulus of elasticity of concrete.

E_s = Steel modulus of elasticity.

L_b = Clear span length of a beam.

F_u = Ultimate tensile strength of a shear stud connector.

F_{yf} = Tensile yield strength of flange in steel beam.

F_{yw} = Tensile yield strength of web in steel beam.

M_p = Plastic flexural capacity of a steel beam.

M_{plate} = Flexural strength contribution of the steel plate in a shear plate coupling beam.

M_s = Flexural strength contribution of the longitudinal reinforcing steel in a shear plate coupling beam.

M_{sp} = Moment calculated at splice connection.

M_T = Total flexural strength of shear plate coupling beam.

M_u = Factored flexural capacity.

N_s = Number of slip planes in a slip-critical connection.

P_{act} = Force applied by a load actuator.

P_{fsp} = Longitudinal force in flange splice plate.

Q_n = Shear strength of one shear stud connector.

R = Radius of a Mohr's circle.

T_b = Specified pretension force for bolt installation in a slip-critical connection.

V_{cr} = Shear capacity of concrete used in calculating embedment length of a steel coupling beam into a wall pier.

V_p = Plastic shear capacity of steel beam.

V_{plate} = Shear strength contribution of steel plate in a shear plate coupling beam.

V_s = Shear strength contribution of transverse reinforcement in a shear plate coupling beam.

V_{steel} = Shear capacity of a steel section used in calculating embedment length of a steel coupling beam into a wall pier.

V_t = Total shear strength of a shear plate coupling beam.

V_u = Factored shear strength capacity.

Z = Plastic section modulus.

$a = L_b/2$.

b_f = Flange width of steel beam.

b_{fsp} = Width of steel flange splice plate in a slip-critical connection.

b_w = Width of a concrete beam.

d = Overall height of a beam.

d = effective depth of a reinforced concrete beam.

d_b = Reinforcement bar diameter.

d_h = Depth of a diagonal bar group measured outside-to-outside of transverse reinforcement.

d_w = Width of a diagonal bar group measured outside-to-outside of transverse reinforcement.

f_l = Normal stress in the steel plate of a shear plate coupling beam due to bending.

f_b = Bearing stress over embedded length of a steel coupling beam.

f'_c = Specified concrete strength.

f_y = Tensile yield strength of longitudinal reinforcing steel in a shear plate coupling beam.

f_{yr} = Reinforcement bar tensile yield strength.

f_{yd} = Yield strength of diagonal reinforcing bars in a diagonally reinforced concrete coupling beam.

h = Height of web in steel beam.

h = Overall height of a concrete beam.

h' = Center-to-center distance between top and bottom longitudinal reinforcing steel.

h_{wsp} = Width of steel web splice plate in a slip-critical connection.

$I_{main\ section}$ = Moment of inertia of the main section of a fuse steel coupling beam.

I_{sp} = Moment of inertia of section taken at splice connection.

l_e = Embedment length of a steel coupling beam into a wall pier.

r_{st} = nominal shear strength of one bolt.

s = Center-to-center spacing of transverse reinforcement.

s_{max} = Maximum c/c spacing of web stiffeners in a steel beam.

t = Thickness of the steel plate in a shear plate coupling beam.

t_f = Flange thickness of steel beam.

t_{fsp} = Thickness of steel flange splice plate in a slip-critical connection.

t_w = Web thickness of steel beam.

t_{wsp} = Thickness of steel web splice plate in a slip-critical connection.

t_{wall} = Wall thickness.

α = angle of inclination of a diagonal bar group measured from horizontal.

α_{SP} = Moment/shear overstrength factor used in the design of a shear plate coupling beam.

β_l = Concrete compressive strength factor as defined by ACI 318 Section 10.2.7.3.

ϕ_b = Flexural strength reduction factor.

ϕ_f = Structural bolt shear strength reduction factor.

ϕ_v = Shear strength reduction factor.

μ = mean slip coefficient.

τ = Shearing stress in the steel plate of a shear plate coupling beam.

τ_{max} = Maximum shearing stress in the steel plate of a shear plate coupling beam.

Chapter 3

A_{cd} = Cross-sectional area of concrete compression strut in a diagonally reinforced concrete coupling beam.

A_g = Horizontal cross-sectional area of test specimen wall pier.

A_s = Cross-sectional area of longitudinal reinforcing steel.

A_{vd} = Cross-sectional area of steel in one diagonal bar group in a diagonally reinforced concrete coupling beam.

$P_{n,max}$ = Maximum nominal axial compression strength of concrete compression member (test specimen wall pier).

$V_{calculated}$ = Calculated shear capacity using known material properties.

V_{Cap} = Calculated shear capacity of a diagonally reinforced concrete coupling beam considering contributions from the diagonal bar steel, compression strut, stirrups, and steel strain hardening.

V_n = Nominal shear capacity of a diagonally reinforced concrete beam as defined by ACI 318.

V_{yield} = Experimentally measured shear at which first yield occurs.

d = effective depth of a reinforced concrete beam.

f'_c = Specified concrete strength.

f_y = Tensile yield strength of longitudinal reinforcing steel.

f_{yv} = Tensile yield strength of transverse reinforcement.

α = angle of inclination of a diagonal bar group measured from horizontal.

ϕ_a = Wall compression strength reduction factor.

ρ = Ratio of steel reinforcement area to concrete cross-sectional area.

Chapter 4

A_{vd} = Cross-sectional area of steel in one diagonal bar group in a diagonally reinforced concrete coupling beam.

V_n = Nominal shear capacity of a diagonally reinforced concrete beam as defined by ACI 318.

b_w = Width of a concrete beam.

$b_x = d_w$ = (Chapter 2) = Width of a diagonal bar group measured outside-to-outside of transverse reinforcement.

$c = C_c$ (Chapter 2) = Clear cover of concrete.

d_b = Reinforcement bar diameter.

f'_c = Specified concrete strength.

$f_y = f_{yd}$ (Chapter 2) = Yield strength of diagonal reinforcing bars in a diagonally reinforced concrete coupling beam.

h = Overall height of a concrete beam.

$h_x = d_h$ (Chapter 2) = Depth of a diagonal bar group measured outside-to-outside of transverse reinforcement.

l_d = development length of reinforcing steel.

$l_n = L_b$ (Chapter 2) = Clear span length of a beam.

v_n = Nominal shear stress of a diagonally reinforced concrete beam as defined by ACI 318.

α = angle of inclination of a diagonal bar group measured from horizontal.

β_l = Concrete compressive strength factor as defined by ACI 318 Section 10.2.7.3.

Chapter 5

A = Cross-sectional area on which P is acting.

A_{cv} = Gross area of concrete section bounded by web thickness and length of section in the direction of shear force considered.

A_{sh} = Total cross-sectional area of transverse reinforcement (including crossties) within spacing, s , and perpendicular to dimension h_c .

C_d = Deflection amplification factor.

C_s = Seismic response coefficient.

C_{sx} = Story shear at level x .

D = Dead Load.

$E_{c,wall}$ = Modulus of elasticity of concrete wall pier.

F_x = Lateral force at level x .

I = Importance factor.

I_{xx} = Moment of inertia about the x -axis of a cross section.

I_{yy} = Moment of inertia about the y -axis of a cross section.

L = Live load.

M_{xx} = Moment about the x -axis of a cross section.

M_{yy} = Moment about the y -axis of a cross section.

P = Axial force assigned to wall pier.

R = Response modification factor.

S_{DS} = Design short period ground acceleration parameter.

S_{DI} = Design one-period ground acceleration parameter.

S_s = Short period ground acceleration parameter.

S_I = One-second period ground acceleration parameter.

V_n = Nominal beam shear capacity.

V = Base shear determined from ELF procedure.

W = Dynamic weight of structure.

c = Distance measured from extreme compression fiber to neutral axis.

f'_c = Specified concrete strength.

f_{yh} = Tensile yield strength of transverse reinforcing.

h_c = Cross-sectional dimension of column core measured center-to-center of confining reinforcement.

h_w = Height of the entire wall pier or segment of wall being considered

h_x = Maximum horizontal spacing of hoop or crosstie legs on all faces of the column.

l_0 = Minimum length, measured from joint face along axis of structural member, over which transverse reinforcement must be provided.

l_w = Length of wall pier or segment of wall in the direction of shear being considered.

s = Spacing of transverse reinforcement measured along the longitudinal axis of the structural member.

s_x = Longitudinal spacing of transverse reinforcing within the length l_0 .

y_B = Distance measured from neutral axis to bottom extreme fiber of a cross section.

y_T = Distance measured from neutral axis to top extreme fiber of a cross section.

x_L = Distance measured from neutral axis to left extreme fiber of a cross section.

x_R = Distance measured from neutral axis to right extreme fiber of a cross section.

δ_u = Design displacement.

σ_{cc} = Concrete normal compressive stress.

γ_{wall} = wall overstrength factor equal to the ratio of the sum of the nominal coupling beam shear capacities to the sum of the design beam shear demands (neglecting torsion).

ρ = Redundancy factor.

Chapter 6

E_c = Modulus of elasticity of concrete.

$EI_g = E_c I$ calculated using transformed section properties.

EI_{SH} = Slope of inelastic portion of bilinear moment-curvature curve.

EI_y = Slope of elastic portion of bilinear moment-curvature curve.

$EI_{y,AVG}$ = Average of the slope of elastic portion of tension wall and compression wall bilinear moment-curvature curves.

$F_y^{(+)}$ = Positive force corresponding to shear yielding.

$F_y^{(-)}$ = Negative force corresponding to shear yielding.

I_e = Effective moment of inertia.

I_g = Gross moment of inertia (Equation 6.1)

L = Clear span length of beam.

V_n = Nominal shear capacity.

$V_{n,conc}$ = Nominal shear strength of a concrete beam.

$V_{n,steel}$ = Nominal shear strength of a steel beam.

$V_{u,conc}$ = Factored shear strength of a concrete beam.

$V_{u,steel}$ = Factored shear strength of a steel beam.

h = Overall height of beam.

k_0 = Initial Stiffness.

α = angle of inclination of a diagonal bar group measured from horizontal.

$\phi_{v,conc}$ = Shear strength reduction factor for a concrete beam.

$\phi_{v,steel}$ = Shear strength reduction factor for a steel beam.

CHAPTER 1: Introduction and Previous Relevant Research

1.1 Introduction

This chapter first presents a summary the basic behavior of coupled core wall systems followed by summaries of some of the previous research relevant to this subject. Finally, a description of the objectives and scope of this research project is presented.

1.2 Structural Behavior of Coupled Core Wall Systems

Coupled core wall (CCW) systems are lateral-force-resisting-systems (LFRS) that have been shown to be very efficient in resisting seismic loads. The CCW system is a core generally located at the center of mid-to-high rise building structures (10 to 20 stories). Figure 1.1 shows the floor plan of a typical coupled core wall structure.

The beam over the openings can be used to couple the individual wall piers, in effect, creating a coupled wall system. This system exhibits significant lateral stiffness and strength if the openings in the core wall are placed in a regular pattern over the height of the building, and the coupling beams are designed and proportioned properly.. A typical elevation of the coupled wall system, called out in Figure 1.1 as A-A, is shown in Figure 1.2. Without the coupling of the individual wall piers, the overturning moment of the structure is resisted by flexural in the wall piers, and the energy introduced to the structure through a seismic event is dissipated by the flexural deformations of the wall piers. When the walls are coupled (via the coupling beams), the wall piers experience compression/tension axial forces creating a couple that acts to resist a large percentage of the total overturning moment rather than the individual wall piers resisting the overturning moment through flexural behavior. The wall axial forces are the sum of the

shear forces in the coupling beams above the floor being considered. Figure 1.3 illustrates the different mechanisms for resisting overturning moments for uncoupled wall piers and the coupled system. Coupled and uncoupled systems act similarly in terms of resisting gravity loads; as illustrated in Figure 1.3, the difference between the two structures is realized only when resistance of lateral loads is considered.

When the coupling beams over the height of the core wall system are proportioned appropriately to attain the desired behavior of the coupled core wall system, the coupling beams will form plastic hinges, in shear, simultaneously while going through very similar beam end rotations (Aktan and Bertero 1981, Shiu et al 1981, Aristizabal-Ochoa 1982). This behavior results in a desirable distribution of energy dissipation (in the coupling beams) over the height of the building as opposed to the energy dissipation being concentrated at the base of the flexural wall piers (Park and Paulay 1975, Fintel and Ghosh 1982, Aktan and Bertero 1984). As discussed previously, in order to achieve the desired behavior of the coupled core wall system, the coupling beams must be designed and proportioned for adequate stiffness and strength. However, the coupling beams must also yield prior to the wall piers and demonstrate stable hysteretic response and good energy-absorbing characteristics.

1.3 Previous Relevant Research

Several types of coupling beams have been investigated in the past, both experimentally and analytically. Steel, steel-concrete composite, and diagonally-braced reinforced concrete coupling beams are the types of coupling beams predominantly investigated. These types of coupling beams are the most common types found in buildings that utilize the coupled core wall system as the lateral-force-resisting-system.

These systems have been shown to demonstrate adequate strength, stiffness, and energy dissipation characteristics (Harries et al 2000).

Another type of composite beams, although not considered as a coupling beam, has also been investigated to a lesser degree than those mentioned previously. Some research (Subedi, 1989) has investigated the feasibility of a composite beam using conventional reinforcement to resist bending moment and an embedded vertical steel plate, in lieu of and/or in addition to conventional transverse reinforcement, to resist the shearing forces. In subsequent discussion, this type of coupling beam will be referred to as a shear plate beam.

1.3.1 Steel Coupling Beams

This type of beam is designed as a steel link beam in an eccentrically-braced frame (EBF). Properly designed steel links have been shown to exhibit excellent energy-absorbing characteristics (Roeder and Popov 1978, Malley and Popov 1983, Kasai and Popov 1986). The link beam in an EBF is the primary element for absorbing energy, and is similar to the behavior of a coupling beam in a coupled core wall system. The design and detailing requirements are provided in the Seismic Provisions for Structural Steel Buildings, 2002 (a supplement to the AISC Manual of Steel Construction).

The general design philosophy of the steel coupling beam is to ensure that the flanges of the beam remain elastic while the web yields in shear, and that appropriate slenderness ratios of the flanges and web are used in order to preclude local flange or web buckling. Full depth stiffeners are usually provided to delay web crippling or web buckling. A typical cross section and elevation of a steel coupling beam coupling reinforced concrete walls is shown in Figure 1.4.

Research (Harries et al 1993, and Harries et al 1995) conducted in the early-to-mid 1990's found that steel coupling beams designed as steel link beams in EBF's exhibited significant ductility and energy dissipation characteristics when subjected to cyclic loading. The experimental phase consisted of steel coupling beams embedded in reinforced concrete walls with no wall boundary elements. However, the vertical bars that were placed in the walls along the embedment length of the beam were increased beyond that required for the flexural capacity of the wall to provide confinement of the concrete along the embedment region. In this research, it was reported that the embedment region of the beam (into the reinforced concrete walls) must be designed to develop the full moment and shear capacity of the clear span portion of the steel coupling beam. It was also found that a reduction in the embedment length must be considered due to the effect of concrete spalling at the face of the wall around the beam/wall connection interface.

Other research into the behavior of steel coupling beams was performed in the early 1990's (Shahrooz et al 1992, Shahrooz et al 1993). Similarly, this research showed that steel coupling beams exhibit excellent ductility and energy dissipation characteristics when subjected to cyclic loads. The experimental phase consisted of steel coupling beams embedded in reinforced concrete walls with wall boundary elements. The embedment region of the coupling beam was detailed to interface with the wall boundary elements (vertical bars and transverse ties). The actual fixed point of the beam was found to be inside the wall boundary element rather than at the face of the wall (approximately 1/3 of the embedment length from the face of the wall). As a result, the calculated stiffness of the beam becomes smaller in comparison to the case in which the fixed point is taken at

the face of the wall. The addition of vertical bars (attached to the flanges of the beams along the embedded region) to the beam flanges was found to significantly reduce the loss of stiffness. These vertical flange bars, referred to as auxiliary transfer bars, allow for a more symmetric distribution of strength and stiffness during load reversals. Additionally, web stiffener plates, located at the wall pier/beam interface and referred to as face bearing plates, were shown to delay the spalling and concrete degradation at the wall pier/beam interface. Auxiliary transfer bars and face bearing plates are shown in Figure 5.

1.3.2 Concrete-Steel Composite Coupling Beams

The most typical concrete-steel composite coupling beam is a steel coupling beam encased in a confined concrete core. Limited research has investigated the behavior of this type of coupling beam. A typical elevation and cross-section of a concrete-steel composite coupling beam coupling reinforced concrete walls is shown in Figure 1.6.

In 2001, research and analytical studies (Gong and Shahrooz 2001a, Gong and Shahrooz 2001b) investigating the behavior of steel coupling beams encased in reinforced concrete cores were performed. The experimental phase of this research project consisted of cantilevered, one-half length steel coupling beams, embedded into reinforced concrete walls. The inflection points of the beams were considered to be at the beam's mid-span. The end of the coupling beam (the assumed mid-length point of an actual coupling beam) was subjected to cyclic shear forces. The main objectives of the experimental phase were (1) to study the effects of concrete encasement around steel coupling beams, (2) to examine whether nominally reinforced encasement can be used in lieu of web stiffeners for preventing lateral instability at large deformations and, (3) to

understand the contribution of concrete encasement to strength and stiffness of the coupling beam. The investigators concluded that nominal concrete encasement around the steel coupling beam increased the stiffness of the beam and prevented web buckling and flange instability precluding the need for web stiffeners. It was also concluded that unless design calculations consider the contribution of the concrete encasement toward shear capacity, a significant portion of the energy would be dissipated by inelastic deformations in the connection region, which is not desirable. Thus, the embedment length of the beam must be calculated properly in order to develop the expected shear capacity of the composite coupling beam.

1.3.3 Diagonally Reinforced Concrete Coupling Beams

A diagonally reinforced concrete coupling beam (DBCB) is a specially-detailed reinforced concrete beam which consists of a conventionally confined concrete beam with two intersecting groups of diagonal bars placed symmetrically about the midspan of the coupling beam (see Figure 1.7). The inclined bar groups must, at a minimum, consist of four corner bars with transverse reinforcement to confine the inclined concrete core. In effect, the detailing of the DBCB produces a frame action between the two walls piers. The frame action relies on compression/tension behavior of the diagonal bar groups that act as “bracing” members.

Research, as far back as the 1970's (Paulay and Binney 1974, Barney et al 1978), has demonstrated excellent energy dissipation and ductility characteristics of DBCB in comparison to conventionally reinforced concrete coupling beams. These studies have also shown that the sliding shear phenomenon, associated with reinforced concrete coupling beams at the face of the walls, is eliminated by the diagonal cores.

Variations on the configuration of the diagonal cores have been investigated (Galano and Vignoli 2000). A rhombic layout of the diagonal bars, in which the bars were bent to the horizontal over the last portion of the embedment length, was tested under cyclic loads. In this research, it was reported that the rhombic layout exhibited greater ductility and energy dissipation characteristics than that of the straight diagonally-placed bars. The investigators also concluded that the rhombic layout was less difficult to place than that of the straight diagonally-placed layout.

1.3.4 Web Shear Plate Coupling Beam

In the late 1980's, research was performed to investigate an alternative to the use of transverse reinforcement of a conventionally reinforced concrete beam (Subedi, 1989). The elevation and cross-section of a one of the test specimens using a shear plate is shown in Figure 1.8. In this research, Subedi investigated the feasibility of embedding a vertical steel plate in a concrete beam to resist shear while using conventional longitudinal reinforcement for resistance to bending moment. Six concrete beams with vertical steel plates were monotonically tested. The beam sections were comprised of the steel vertical plate and ordinary longitudinal reinforcement with nominal transverse ties. To ensure composite action of the section, the plates were profiled with semicircular cut-outs along the edges of the plate. Results of the test showed that the web shear plate section exhibited good strength, ductility, and stable hysteretic response. An average shear stress of $0.75\sqrt{f'_c}$ (MPa) was achieved.

The alternative design was based on the concept described above. This beam is referred to as a plate-reinforced or shear plate coupling beam.

1.3.5 Wall Pier Reinforcement

In the mid-1990's, the methods for establishing the proportioning and detailing requirements of structural walls were scrutinized (Wallace 1994). The research focused on a displacement-based design approach for structural walls in areas of high seismic activity where global building deformation demands are estimated based on characterized earthquake ground motions. A relationship between global and local deformations is established and compared to an estimated local deformation capacity of the wall cross section. Where the estimated maximum concrete compressive strains in the wall piers are found to be less than a critical strain (in this work, a strain of 0.004 is considered as the critical strain), special boundary elements are assumed not to be required.

Although the study considered only cantilevered shear walls, it was shown that there are a class of reinforced concrete structures in which heavily-reinforced boundary elements are not required. Wall attributes such as wall reinforcement ratios, wall axial loads, and material properties were found to have a direct impact on the maximum concrete compressive strains on the walls. Due to the relatively large axial forces generated in wall piers in coupled core wall systems, further research into the strain demands on coupled wall piers is required.

1.4 Objectives and Scope

1.4.1 Objectives

The primary objective of this research is to develop viable alternate design approaches of the components commonly used in coupled core wall systems where construction difficulties are minimized while maintaining sufficient structural integrity. Another objective is to develop new viable alternative coupling beam sections which

satisfy the primary goal of this research. The primary deliverables of this study are: (1) an evaluation of the performance of the coupling beams tested in this research project, (2) an evaluation of the application of current ACI 318 provisions pertaining to the design of diagonally-reinforced concrete beams, (3) an evaluation of the application of current ACI 318 provisions pertaining to the boundary detailing of wall piers in CCW systems, and (4) a comparison of the overall structural performance of a CCW system using a steel coupling beam to a CCW system using a diagonally-reinforced coupling beam with non ACI-compliant transverse reinforcement around the diagonal bar groups.

1.4.2 Scope

Five major tasks were performed in order to meet the objectives of this research: (1) literature review of current design procedures and relevant previous research, (2) development of two proposed new coupling beam sections, (3) experimental testing of four specimens each utilizing a different type of coupling beam, (4) parametric numerical studies to evaluate current ACI provisions for the design of the diagonally-reinforced coupling beams and wall piers in coupled core wall systems and, (5) nonlinear dynamic analyses to compare overall structural behavior of buildings with different types of coupling beams.

The background information and relevant previous research found during literature review and summarized in this chapter is by no means exhaustive but is deemed sufficient for meeting the scope and satisfying the objectives of this research project.

Two new coupling beam sections are proposed in this research project. The first new section is an unencased steel coupling beam with a “fuse” section located at mid-span of the beam. It is intended that the fuse will experience the majority of damage

during an earthquake while the main section remains undamaged. The other new section is a steel/concrete composite coupling beam consisting of a vertical steel web plate encased in concrete. Headed shear studs installed on the web plate are used for shear transfer between the concrete and steel plate. Detailed design methodologies and procedures for these two proposed sections are presented in Chapter 2. Additionally, the design procedures for the unencased steel and diagonally-reinforced concrete beams are presented.

The experimental phase of the research consisted of reverse cyclic testing of four test subassemblies. The subassemblies were $\frac{1}{2}$ scale models and each subassembly consisted of a coupling beam between two wall piers. Each subassembly was provided with a different type of coupling beam: (1) a diagonally-reinforced concrete coupling beam (DBCBC), (2) an unencased steel beam (SCB), (3) an uncased steel beam with a steel fuse link located at mid span of the beam (FCB1 and FCB2) and, (4) a composite steel/concrete beam utilizing a vertical web shear plate with headed shear studs (SPCB). Specimens (1) and (2) are types of coupling beams common to CCW structures. Specimens (3) and (4) are new proposed types of coupling beams. Details of the test specimens, experimental program, and test results are presented in Chapter 2.

To evaluate the application of current ACI 318 provisions to the design diagonally-reinforced concrete coupling beams, a parametric study is performed where different combinations span-to-depth ratios and shear stress levels are considered in the design of the diagonally-reinforced coupling beams. For each combination, attempts at designing the beam to meet the ACI provision are made. Conclusions will be drawn as to whether or not the design of the beam can possibly be designed to meet the ACI

requirements, and as to whether or not the beams are practically constructible. The results of this study are presented in Chapter 4.

To evaluate the application of current ACI 318 provisions pertaining to the detailing of the boundaries of wall piers in coupled core wall systems, elastic analyses of eight buildings with different core wall configurations were performed using the equivalent lateral force procedure. All eight buildings had the same story dimensions, plan dimensions, wall thickness, and were assumed to be located on the same site. Steel coupling beams were used in all eight buildings. Using these eight buildings, the ACI 318 provisions for establishing the need for special boundary elements were evaluated. Moreover, an alternative method for establishing need for special boundary elements is proposed. The study is presented in Chapter 5.

Nonlinear dynamic analyses were performed to compare the overall structural behavior of a coupled core wall system using the steel coupling beam section to that of a structure using the diagonally-reinforced coupling beams. The properties needed to model the walls and coupling beams were arrived at based system identifications of the measured properties of the coupling beams and prior experience in modeling and section analyses. The general layout and design loads were the same for the two structures. Design of the two buildings, their modeling, and the analytical results are provided in Chapter 6.

Finally, conclusions based on the parametric, experimental, and analytical studies are drawn in Chapter 7. Recommendations for further research are also presented in this chapter.

Figure 1.1. General floor plan of a coupled core wall system.

Figure 1.2. Elevation A-A shown in Figure 1.1.

Figure 1.3. Reaction mechanisms to lateral loads of (a) a coupled wall pier system and (b) an uncoupled wall pier system.

Figure 1.4. Typical unencased steel coupling beam.

Figure 1.5. Placement of auxiliary transfer bars and face bearing plates.

(a) ELEVATION

(b) SECTION A-A

Figure 1.6. Typical composite concrete/steel coupling beam.

Figure 1.7. Typical diagonally-reinforced concrete coupling beam.

(a) ELEVATION

(b) SECTION A-A

Figure 1.8. Plate-reinforced concrete beam tested by Subedi.

CHAPTER 2: Coupling Beam Design Procedures

This chapter first describes the design procedures used for the four different types of coupling beams examined in this research. Details of the resulting test subassemblies used in the experimental phase are also presented.

2.1 Coupling Beams

2.1.1 Coupling Beam Parameters

All four types of coupling beams are 356mm deep and 914mm long and couple 1,575mm high x 1,689mm long x 245mm thick wall piers. The dimensions of the beams are based on what the author feels to be ½ scale dimensions of coupling beams in a typical coupled core wall building.

The target capacities for all experimental beams are taken as 534kN. The target capacity was arrived at based on calculated capacities of the testing apparatus used. Buckling and elongation of the frame columns, actuator capacity, cross beam deflection, and strong floor tie-down capacity were considered.

All concrete was assumed to be 35MPa and all steel was assumed to be A36 steel. As will be seen in the following sections, steel yield strengths were increased by 25% to account for probable material strength.

2.2 Common Coupling Beams

2.2.1 Design Procedure for the Unencased Steel Coupling Beam

The target capacity of the section is $V_u=534kN$ and $M_u=244kNm$. The first step in the design of the steel coupling beam is to ensure that web and flange local buckling

requirements are satisfied. The following relationships for flange local buckling (FLB) and web local buckling (WLB), respectively, must be satisfied.

$$\frac{b_f}{t_f} \leq 0.31 \sqrt{\frac{E_s}{F_{yf}}} \quad (2.1)$$

$$\frac{h}{t_w} \leq 3.05 \sqrt{\frac{E_s}{F_{yw}}} \quad (2.2)$$

To account for material overstrength, the nominal yield strength of the flange and web steel is increased by 25%. Therefore, $F_{yf}=F_{yw}=(1.25)(252MPa)=315MPa$. $E_s=199,938MPa$. Substituting these values in to Equations 2.1 and 2.2, $b_f/t_f \leq 7.8$ and $h/t_w \leq 77$.

To establish a trial design, the plastic shear capacity of the beam is set equal to the design shear force. Therefore, the following relationship is established.

$$V_p = \phi_v 0.6 F_{yw} (h - 2t_f) t_w = V_u = 534kN \quad (2.3)$$

The overall depth of the section is known and is equal to $356mm$. Arbitrarily assuming that $t_f=25.4mm$, knowing that the design shear is $534kN$, and $\phi_v=0.9$, the minimum value for t_w can be directly computed. Therefore, using Equation 2.2, $t_w \geq 10.5mm$.

To ensure adequate flexural strength, the following relationship is defined.

$$\phi_b M_p = \phi_b F_{yf} Z_{required} = M_u \Rightarrow Z_{required} = \frac{M_u}{\phi_b F_{yf}} \quad (2.4)$$

Substituting $M_u=244kNm$, $\phi_b=0.9$, and $F_{yf}=315MPa$ into Equation 2.4, $Z_{required}$ can be directly computed. Therefore, $Z_{required}=860,670mm^3$.

To ensure that the coupling beam is shear critical, the length of the beam (considered as a link in an eccentrically-braced frame) must be less than or equal to $1.6M_p/V_p$. The following relationship is defined.

$$L_b = \frac{1.6M_p}{V_p} = \frac{1.6F_{yf}Z_{required}}{\frac{V_u}{\phi_v}} \Rightarrow Z_{required} = \frac{V_u L_b}{1.6F_{yf}\phi_v} \quad (2.5)$$

Knowing that the length of the beam is $914mm$ and substituting $V_u=534kN$, $F_{yf}=315MPa$, and $\phi_v=0.9$ into Equation 2.5, $Z_{required}$ can be directly calculated. Therefore, $Z_{required}=1,076,005mm^3$. The larger value of $Z_{required}=1,076,005mm^3$ governs the design of the section.

The plastic section modulus, Z , for a wide-flange section is calculated by the following equation:

$$Z = 2 \left(b_f t_f \left(\frac{h}{2} \right) + \frac{h^2 t_w}{8} \right) \quad (2.6)$$

To ensure that the flange of the coupling beam will clear the steel reinforcement in the wall piers, b_f is taken as $127mm$ and t_f has already been assumed to be $25.4mm$. Since $Z_{required}=1,076,005mm^3$, the minimum value of t_w , based on flexure, can be directly calculated. Therefore, using Equation 2.2, $t_w \geq 3.9mm$ which is less than the value of $t_w=10.5mm$ needed for shear capacity.

Since $h = d - 2t_f = 356mm - 2(25.4mm) = 305mm$ and $b_f = 127mm$ is now known, the local buckling requirements (WLB and FLB) can now be checked. For WLB,

$$t_w \geq \frac{h}{77} = \frac{305mm}{77} = 3.9mm < 10.5mm \text{ and for FLB, } \frac{b_f}{t_f} = \frac{127mm}{25.4mm} = 5 < 7.8. \text{ Therefore,}$$

local buckling requirements are satisfied.

The following section dimensions are selected for the steel coupling beam, and design checks are then made.

$$\begin{aligned}d &= 356mm \\b_f &= 127mm \\t_f &= 25.4mm \\t_w &= 12mm \\h &= d - 2t_f = 305mm\end{aligned}$$

$$\phi_v V_p = (0.9)(0.6)(315MPa)(305mm)(12mm) = 623kN \geq V_u = 534kN \quad \text{OK}$$

$$Z = 2 \left(\frac{(127mm)(25.4mm)(305mm)}{2} + \frac{(305mm)^2(12mm)}{8} \right) = 1,262,944mm^3 > 1,076,005mm^3$$

$$\phi_b M_p = \frac{(0.9)(315MPa)(1,262,944mm^3)}{(1,000mm/m)(1,000N/kN)} = 358kN-m > 244kN-m \quad \text{OK}$$

$$L_b = 914mm \leq \frac{1.6M_p}{V_p} = \frac{(1.6) \left(\frac{358kN-m}{0.9} \right) (1,000mm/m)}{\frac{623kN}{0.9}} = 919mm \quad \text{OK}$$

$$\frac{h}{t_w} = \frac{305mm}{12mm} = 25 \leq 77 \quad \text{OK}$$

$$\frac{b_f}{t_f} = \frac{127mm}{25.4mm} = 5 \leq 7.8 \quad \text{OK}$$

The next step is to determine the thickness and spacing for the web stiffeners. Since the beam is less than 610mm (24in) deep, the minimum required thickness for the web stiffeners is the larger of $t_w=12mm$ or 9.5mm (3/8in). However, stiffeners will be provided on both sides of the web; therefore, the minimum thickness is the larger of $0.75t_w=(0.75)(12mm)=9mm$. The required spacing of the web stiffeners is dependent on the rotation demand on the beam. Based on previous studies (Gong and Shahrooz, 2001a,

2001b), a maximum rotation angle equal to 0.04 radians was selected. For a rotation angle equal to 0.08 radians, stiffener spacing shall be less than or equal to $30t_w - \frac{d}{5}$. For a rotation angle equal to 0.02 radians, stiffener spacing shall be less than or equal to $52t_w - \frac{d}{5}$. AISC allows linear interpolation for intermediate rotation angles. The required spacing of the stiffeners for 0.08 and 0.02 radians is

$$(30)(12mm) - \frac{356mm}{5} = 289mm$$

$$(52)(12mm) - \frac{356mm}{5} = 553mm$$

Interpolating linearly between these two points, the maximum spacing of the stiffeners, for the assumed rotation angle equal to 0.04 radians, is $465mm$. Since $L_b/2 = 914mm/2 = 457mm < s_{max} = 465mm$, one $10mm$ thick stiffener will be provided on each side of the web at midspan of the beam.

The next step is to calculate the length of the embedment of the beam into the wall pier. Based on previous research (Gong and Shahrooz, 2001a, 2001b) the beam shear used to calculate the embedment length is taken as $1.56(V_{steel} + V_{cr})$. For $V_n = 623kN/0.9 = 692kN$, $f'_c = 35MPa$, $\beta_1 = 0.80$, $a = L_b/2 = 914mm$, $t_{wall} = 254mm$, $b_f = 127mm$ and taking $V_{cr} = 0$,

$$V_u = (1.56)(692kN) = 1,080kN$$

$$V_u = f_b \beta_1 b_f l_e \left[\frac{0.58 - 0.22 \beta_1}{0.88 + \frac{a}{l_e}} \right] = 1,080kN \quad (2.7)$$

$$\text{where } f_b = 54 \left(\frac{\sqrt{f'_c}}{12} \right) \left(\frac{t_{wall}}{b_f} \right)^{0.66} \quad (2.8)$$

Substituting Equation 2.8 into Equation 2.7 and applying the known values, the required embedment length, l_e , can be calculated. Therefore, $l_e=775mm$.

The final step is to determine the face bearing plates and auxiliary transfer bars. The face bearing plates are located at the wall pier/beam interfaces and will be the same size as the provided web stiffeners. An additional bearing plate is provided at mid-length of the embedded area. Two #6 ($d_b=19mm$) auxiliary transfer bars are provided for the top and bottom flanges at both of the embedded ends. Lenton weldable half-couplers are provided to the flanges to facilitate installation of the transfer bars. See Figure 2.1 for details of the SCB specimen (the as-built details are shown in this figure).

The aforementioned procedures were used to design the steel coupling beams considered in the parametric and analytical studies presented in Chapters 5 and 6.

2.2.2 Design Procedure for the Diagonally Reinforced Concrete Beam

The target nominal design shear for the diagonally-reinforced concrete beam is $345kN$. This target value was reduced as result of the impossibility of satisfying the steel area requirements for a target shear equal to $354kN$ (Refer to Chapter 4 for a detailed discussion on the design of practical diagonally reinforced coupling beams). $f'_c=35MPa$ and #2 stirrups ($d_b=6.4mm$) stirrups are to be used for transverse reinforcement. To account for material overstrength, the yield strength of the reinforcing bars will be taken

as $1.25f_{yr}$. Therefore, $f_{yr}=(1.25)(413,685MPa)=517,108MPa$. The depth of the beam is $356mm$, $L_b=914mm$, and $b_w=245mm$.

The first step is to calculate the required area of steel. The required area of steel is calculated using Equations 2.9 and 2.10. Refer to Figure 2.2 for a diagram illustrating the symbols used in these equations.

$$V_u = \phi_v 2A_{vd}f_{yd} \sin \alpha \leq 0.83\sqrt{f'_c} \quad (\text{MPa units}) \quad (2.9)$$

$$\text{where, } \alpha = \tan^{-1} \left(\frac{h - d_h - 2C_c}{L_b} \right) \quad (2.10)$$

$$\text{where } d_h \geq \frac{h}{5}$$

Since the value of d_h is dependent on the bar diameter used, the required area of steel is dependent on the value of α , and α is dependent on the value of d_h , iterations must be performed to satisfy Equation 2.9. After performing iterations, (4)-#8 (25.4mm) bars are used in each diagonal group. The final values for d_h and d_w are 108mm and 127mm, respectively. Final strength checks are made at this point to ensure adequate design.

$$d_w = 127mm \geq \frac{b_w}{2} = \frac{254mm}{2} = 127mm \quad \text{OK}$$

$$d_h = 108mm \geq \frac{h}{5} = \frac{356mm}{5} = 71mm \quad \text{OK}$$

$$\alpha = \tan^{-1} \left(\frac{356mm - 108mm - (2)(19mm)}{914mm} \right) = 13^\circ$$

$$A_{vd} = (4)(510mm^2) = 2,039mm^2$$

$$V_u = (0.75)(2)(2,039mm^2)(0.517kN/mm^2) \sin(13) = 356kN > 334kN \quad \text{OK}$$

$$V_u = 356kN \leq \left(\frac{0.83\sqrt{35MP}(356mm)(254mm)}{1000N/kN} \right) = 444kN \quad \text{OK}$$

The next step would be to determine the required spacing of the transverse reinforcement for the diagonal bar groups. Since one of the objectives of this test is to

investigate the behavior of the beam when the diagonal bar groups are not confined per ACI, this step is neglected.

The final step is to determine the required spacing of the transverse reinforcement for the main core of the beam. Four #6 ($d_b=19mm$) corner longitudinal bars are used for which #2 ($d_b=6.4mm$) transverse reinforcement is provided. Seismic design requires that transverse reinforcement be provided at no less than $h/4=356mm/4=89mm$. Therefore, #2 ($d_b=6.4mm$) hoops at an interval equal to $89mm$ will be provided. To facilitate construction of the specimen, the hoops will consist of (1)-U-stirrup and (1)-crosstie. The crosstie will have a standard 90° hook at one end and a 135° hook at the other. The 135° hooks of adjacent crossties are alternated such that a 135° hook engages the same longitudinal bar at every other stirrup.

Figure 2.3 provides details of the final beam design (the as-built details of the specimen are shown in the figure). Similar procedures were used for the design of all other diagonally reinforced concrete beams considered in the numerical studies presented in Chapters 4 and 6.

2.3 New Proposed Coupling Beams

Consistent with the objectives of this research project, two new types of coupling beams are proposed. The first new section is a variation of the unencased steel coupling beam and is referred to as a fuse steel coupling beam. This type of beam was developed with the intent of minimizing post-earthquake repair and/or replacement. The general concept of the beam is to provide a small length of an I-shape section at the mid-span of the beam in where all of the beam damage would occur. The beam consists of three separate lengths of I-shape sections: two like “main” sections which are embedded in

either wall pier and connected by the third section (installed between the two main sections) which is designed to act as a fuse. Ensuring that the majority of the damage is incurred in the fuse is achieved by proportioning the main and fuse sections in a manner that would allow the main sections to remain elastic beyond the stage of loading at which the web of the fuse has yielded. Assuming that the main sections are not damaged, during post-earthquake repairs, the damaged fuse section would simply be replaced with a new fuse.

The other new proposed section is a steel/concrete composite section consisting of a vertical web shear steel plate encased in a conventionally reinforced concrete beam with a moderate level of confinement. This new section is referred to as a shear plate coupling beam (SPCB). The general objective in the development of this beam is to provide an alternative solution to the diagonally-reinforced concrete beam and its inherent design and construction difficulties. Headed shear studs are installed on both sides of the plate for shear transfer between the concrete and the steel plate. Conventional transverse reinforcement and longitudinal top and bottom bars are provided and considered to contribute to shear and flexural resistance, respectively.

The design methodology adopted for this section considers shear to be resisted by both the vertical steel plate and transverse reinforcement; shear resistance of the concrete is taken as zero. Flexure is considered to be resisted by the longitudinal bars as well as the vertical steel plate. Since the plate is considered to contribute both to shear and flexural resistance, flexure-shear interaction must be considered. Composite action is assumed to take place through a shear transfer mechanism acting via the headed shear studs installed on the plate. To ensure that the beam is shear critical, the concept of

considering the beam as a link in an eccentrically-braced frame is adopted and incorporated into the design equation governing the proportioning of the beam.

The design procedure for this beam is detailed in Section 2.2.2. However, before design of the beam is presented, a derivation of the equation used to proportion the beam is presented in Section 2.2.2.1.

2.3.1 Design Procedure for the Fuse Steel Coupling Beam

The main sections of this beam and their embedment length into the wall piers are designed like that of the steel coupling beam discussed in Section 2.1.1. For this specimen, the main sections will be the same as the section designed for the steel coupling beam and the embedment length into the wall piers is also the same. The main-fuse connection is designed to be slip-critical and the web and flange splice connections must be designed to transfer the shear and moment calculated at the location of the connection. Design of the fuse section and slip-critical main-fuse connections are described in this section.

The percentage of shear capacity that the fuse will have relative to the main section is arbitrarily chosen. The testing protocol chosen for this test is to design two different fuse sections: one will be designed to have 70% of the main section's shear capacity and the other will be designed to have 50% of the main section's capacity. Refer to Chapter 3 for the specifics pertaining to the testing protocol for this specimen.

The design procedure for the fuse section is relatively straightforward. The top and bottom flanges were sized to be the same size as that of the main sections. Thus, the flanges of both fuses had $b_f=127mm$ and $t_f=25.4mm$. The overall height of the beam was the same ($h=256mm$). The only section dimension needed is the value of t_w . Since

$h=256mm$ and $t_f=25.4mm$ are the same for all the sections, the web thickness of the fuse, t_w , is simply proportional to the percentage of the design shear strength of the main section. Hence, $t_w=(0.5)(12mm)=6mm$ for the 50% fuse and $t_w=(0.7)(12mm)=8.4mm$ for the 70% fuse. For the remainder of all following discussions, the 70% fuse will be referred to as FCB-1 and the 50% fuse will be referred to as FCB-2. To complete the design of these specimens, the slip-critical main/fuse connection is designed.

The first step is to obtain a trial size for the flange-web splice plates. To simplify the design, the flange splice plates are assumed be the same size as the flanges of the main section. Hence, $t_{fsp}=25.4mm$ and $b_{fsp}=127mm$. To ensure that the splice connection can transfer at least a shear force equal to the shear capacity of the main section ($V_n=692kN$, refer to Section 2.1.1), the web splice plates (one on each side of the web) were assumed to have the same thickness as the main section web thickness, and the height of the web splice plates was sized to ensure clearance of the weld root. Clearance between the top and bottom edges of the web splice plate and the flanges of the beam is taken as $25mm$. Hence, $t_{wsp}=12mm$ and $h_{wsp}=d-25mm=305mm-50mm=255mm$. A check is now performed to ensure that $I_{sp} \geq I_{main \text{ section}}$. The trial splice connection is shown in Figure 2.5. Referring to Figure 2.5, the moment of inertia of the splice connection, $I_{sp}=269,176,863mm^4 > I_{main \text{ section}}=206,159,425mm^4$. Therefore, the splice plate sizes are adequate.

The next step is to determine the required number of bolts for the flange and web splice connections. The design shear for the connection is $534kN$ and each bolt will be in double shear. $1in$ ($25.4mm$) diameter A490 bolts are assumed. In order to minimize the

number of required bolts, a mean slip coefficient equal 0.33 is assumed. The shear capacity of one bolt is:

$$\phi_f r_{st} = 1.13 \mu T_b N_s = (1.0)(1.13)(0.33)(64k)(2) = 47.7kips (212.3kN)$$

Therefore, the number of bolts required is $\frac{534kN}{212.3kN/bolt} = 2.5$ bolts. Provide (3)-1in (25.4mm) diameter bolts on each side of the web splice connection. The design moment at the end of the beam is 244kNm (refer to Section 2.1.1). The same size bolts used for the web splice plates was used transfer the total flange force developed at the location of the flange splice plates. Assuming that two rows of bolts are required and that the length of the fuse must be at least 356mm long, the splice was located 191mm from the point of zero moment (at midspan of the beam). Therefore, the moment to be resisted, M_{SP} , at the connection is

$$M_{SP} = \frac{(191mm)(244kN - m)}{\frac{914mm}{2}} = 102kN - m \text{ and the total flange force that the bolts must}$$

$$\text{transfer is, } P_{fsp} = \frac{M_{SP} \left(\frac{h}{2} \right)}{I_{main section}} b_f t_f = \frac{\left(\frac{102kN - m}{1m/1,000mm} \right) \left(\frac{356mm}{2} \right) (127mm)(25.4mm)}{206,159,425mm^4} = 284kN .$$

The flange splice bolts are in single shear and therefore have a capacity equal to half of that calculated for the web splice plate bolts (the same size bolts are assumed). The

capacity of one bolt in single shear is $\phi_r_{st} = \frac{212.3kN/bolt}{2} = 106.2kN/bolt$. The number

of bolts needed to transfer the force in the flange is $\frac{284kN}{106.2kN/bolt} = 2.7$ bolts. In the

interest of having a symmetric bolt pattern, (4)-1in diameter (25.4mm) A490 bolts were provided on each side of the flange splice plates.

The spacing of the bolts was determined based on minimum distances required for entering and tightening. Bolt bearing and block shear was not considered. This decision is considered to be reasonable considering that for bolt bearing or block shear to occur, the connection must slip. If slipping occurs, the intended behavior of the beam will not be achieved.

Additionally, allowances were made for shim plates at the web splices between the web of the fuse and the web splice plate to make up for the difference in the web thicknesses of the main and fuse sections. The same connections were used for both of the fuse beam tests. However, due to the difference in the web thicknesses between the 50% and 70% fuses, the thickness of the shim plates are different. See Figure 2.6 for details of the final beam designs.

2.3.2 Design Procedure for the Shear Plate Coupling Beam

2.3.2.1 Derivation of Design Equation

Refer to Figure 2.7 for the following derivation. The steel plate and the transverse reinforcement are considered to contribute to the shear resistance. The concrete contribution to shear resistance is ignored as is typical in seismic design. Additionally, the concrete contribution to flexural strength is ignored in order to simplify design. The shear resistance contributed by the steel plate is

$$V_{plate} = \tau Dt \quad (2.11)$$

The shear resistance contributed by the transverse reinforcement is

$$V_s = \frac{A_v f_{yr} d}{s} \quad (2.12)$$

Thus, the total shear resistance of the beam is

$$V_T = V_{plate} + V_s = \tau Dt + \frac{A_s f_{yr} d}{s} \quad (2.13)$$

The flexural resistance of the section is provided by the longitudinal steel and the steel plate. The flexural resistance provided by the longitudinal steel is

$$M_s = A_s f_y h' \quad (2.14)$$

The flexural resistance provided by the steel plate, assuming full plasticity of the plate, is

$$M_{plate} = \frac{D^2 t f_1}{4} \quad (2.15)$$

The total flexural resistance, M_T , provided by the section is

$$M_T = M_s + M_{plate} = A_s f_y h' + \frac{D^2 t f_1}{4} \quad (2.16)$$

The relationship between V_T and M_T is

$$M_T = \frac{V_T L_b}{2} \quad (2.17)$$

Substituting Equation 2.16 into Equation 2.17

$$\frac{V_T L}{2} = A_s f_y h' + \frac{D^2 t f_1}{4} \quad (2.18)$$

To account for shear-flexure interaction, a relationship between the yield strength of the plate, F_{yp} , the shear stress in the plate, τ , and the bending stress in the plate, f_1 , must be defined. Refer to Figure 2.8.

$$R = \sqrt{\left(\frac{f_1}{2}\right)^2 + \tau^2} \quad (2.19)$$

The maximum shear stress, $\tau_{max} = R$. Thus,

$$R = \tau_{max} \quad (2.20)$$

Setting Equation 2.19 and Equation 2.20 equal to each other and taking the maximum shear stress, $\tau_{\max}$, equal to one-half of the tensile yield strength,

$$\frac{F_{yp}}{2} = \sqrt{\left(\frac{f_1}{2}\right)^2 + \tau^2} \quad (2.21)$$

In taking the maximum shear stress as $\frac{1}{2}$ of the tensile yield strength of the steel, Tresca's yield theory is assumed. For most steel applications, Von Mises's yield theory is generally applied. However, these cases usually involve I-shaped section where the shear and flexural resistance is considered to be uncoupled i.e., the flanges of the beam carry all of the normal stresses due to bending and the web carries all of shearing stresses due to shear. As shear and flexure are assumed to be coupled in this application, Tresca's yield theory is a more reasonable assumption.

For the test setup (refer to Chapter 3), the total force applied to the beam from the actuators, P_{act} , is equal to the total shear imparted the beam. Therefore,

$$P_{act} = V_T = \tau Dt + \frac{A_v f_{yr} d}{s} \quad (2.22)$$

To ensure that the section is shear critical, the flexural capacity of the beam will be increased by a factor α_{sp} . Accordingly, Equation 2.17 is modified.

$$\alpha_{sp} M_T = \frac{V_T L_b}{2} \quad (2.23)$$

After rearranging equations and making proper substitutions, the following quadratic equation for P_{act} is derived.

$$P_{act}^2 (4\alpha_{sp}^2 L_b^2 + 4D^2) - P_{act} (16\alpha_{sp} L_b R + 8V_s D^2) + 16R^2 + V_s^2 D^2 - f_{yp}^2 D^4 t^2 = 0 \quad (2.24)$$

$$R = A_s f_y h'$$

where,

$$V_s = \frac{A_v f_{yv} d}{s}$$

The following section details the procedure of the design for the shear plate coupling beam.

2.3.2.2 Detailed Design Procedure

Most of the variables needed for the design of this beam are known; the overall beam dimensions, the design shear force, and the probable yield strengths of the material. From these known values, other variables can be assumed. Equation 2.24 can easily be used by making appropriate assumptions about the values of all of the unknowns, except for the thickness of the steel plate, t . The resulting equation is quadratic in t with all other variables known. The plate thickness, t , can be directly calculated.

The required area of longitudinal steel, using Gr. 60 steel, was large enough that the required amount of longitudinal steel would not practically fit in the section. Therefore, it was decided to use high strength MMFX steel for the longitudinal bars in order to reduce the required area of steel needed for the longitudinal bars and #8 (25.4mm) MMFX steel reinforcing bars were provided. To ensure proper development of the MMFX steel, 180° hooks were provided at the ends of the bars. The design is based on #2 ($d_b=6.4mm$) stirrups spaced at 76mm o.c. (which satisfies the $s \leq d/4$ requirement). The nominal yield strengths are increased by 25% to allow for probable material overstrengths. The following values are assumed.

Beam Properties:

$$\begin{aligned}
b &= 254mm \\
d &= 318mm \\
L_b &= 914mm \\
h' &= 279mm
\end{aligned}$$

Material Properties

$$\begin{aligned}
f_{yr} &= 827MPa \\
f_{yv} &= 517MPa \\
F_{yp} &= 310MPa \\
A_s &= 565mm^2 \\
A_v &= 142mm^2 \\
D &= 279mm \\
s &= 76mm
\end{aligned}$$

The value for α_{sp} is taken as 1.6 and the design shear force (equal to P_{act}) for the beam is $534kips$. Substituting these values into Equation 2.24, the required plate thickness is calculated as $6.35mm$. Thus, the proportioning of the beam consists of the beam and material properties listed above and the plate thickness is equal to $6.35mm$.

The next step in the design is determining the required size and spacing of the headed shear studs to be installed on the plate in order to transfer shear between the steel plate and the concrete. For a trial section, $2in$ long x $\frac{1}{4}in$ diameter ($51mm$ x $6.35mm$) headed shear studs are assumed. The spacing requirement for the studs is calculated using the AISC (Manual of Steel Construction, 3rd Edition) method for stud spacing based on the shear strength of one stud connector embedded in solid concrete. The stud strength equation is written as

$$Q_n = 0.5A_{sc}\sqrt{f'_cE_c} \leq A_{sc}F_u \quad (2.25)$$

$$Q_n = \frac{(0.5)(31.65mm^2)\sqrt{(35MPa)(27,789MPa)}}{1,000N / kN} = 15.6kN$$

$$A_{sc}F_u = \frac{(31.65mm^2)(413MPa)}{1,000N / kN} = 13.1kN$$

$$\therefore Q_n = 13.1kN$$

The studs are provided to transfer the design shear force over each region of the beam's moment diagram from the point of zero moment to the point of maximum moment. In the case of a coupling beam in double curvature, two of these regions exist (refer to Figure 2.9) since the moment gradient is linear over the length of the beam with equal maximum moments at both ends of the beam and zero moment at midspan. In each of these two regions, the total required number of shear studs is

$$\text{Stud Quantity} = \frac{534kN}{13.1kN/stud} = 40.8 \text{ studs} .$$

Rounding this number to an even quantity of 40 studs, (20)-2in long x 1/4in diameter (51mm x 6.35mm) headed shear studs will be provided on both sides of the plate and in each half-spans of the beam. The studs will be provided in two horizontal rows (consisting of (10)-studs) of studs on each side of the plate. The top and bottom horizontal rows are placed 64mm from the upper and bottom edges of the plate respectively.

The last step in the design of the beam is to determine the embedment length of the plate into the wall piers. It is proposed that the embedment length should be at least equal to or greater than $L_b/2$ such that the number of studs provided in each embedded area is equal to the number of studs provided in each half-span. This method requires that the plate be embedded into the wall pier a minimum distance of $914mm/2=457mm$. Since not much is known about the stress distributions in the embedded regions for this type of

connection, the embedment length is arbitrarily set to be equal to the embedded length determined for the unencased steel coupling beam ($775mm$). Thus, the total number of studs required for the plate is $(40 \text{ studs/region})(4 \text{ regions})=160 \text{ studs}$.

See Figure 2.10 for the details of the final beam design for this test specimen. Note that the figure shows the as-built details of the specimen, and that the number of studs provided are not the same as the number of studs calculated. This is due to construction errors not discovered until after testing. Details pertaining to the construction errors are discussed in Chapter 3.

(a) ELEVATION

(b) SECTION A-A

Figure 2.1. As-built details of specimen SCB.

Figure 2.2. (a) Shear resisting mechanism of a diagonally-reinforced concrete coupling beam. (b) ACI minimum sizes of diagonal bar groups.

(a) ELEVATION

(b) SECTION A-A

Figure 2.3. As-built details of specimen DBCB.

$$I_{xx, \text{splice}} > I_{xx, \text{main beam}}$$

Figure 2.4. Schematic used for determining the required moment of inertia of the splice section in the fuse coupling beam specimens.

Figure 2.5. Trial splice connection for design of FCB-1 and FCB-2.

Figure 2.6. As-built details of specimens FCB-1 and FCB-2.

Figure 2.7. FBD of a concrete coupling beam with a centrally-located vertical steel plate.

Figure 2.8. Mohr's circle used to relate f_{yp} , τ , and f_l in deriving design equation for a reinforced concrete beam with a centrally-located vertical steel plate.

Figure 2.9. Shear and moment distribution over the length a beam in double curvature.
The minimum embedment length of the steel plate for specimen SPCB is $L/2$.

(a) ELEVATION

(b) SECTION A-A

NOTE: The headed shear stud quantity and spacing shown in drawings is the spacing actually provided and not the quantity and spacing calculated and specified.

Figure 2.10. As-built details of specimen SPCB.

CHAPTER 3: Experimental Testing

In this chapter, the test setup and testing protocol are described. Summaries of each of the four tests are provided. Finally, the results of each test are presented.

3.1 Test Setup Description

The specimens (see Figure 3.1) were tested in order to:

- Compare calculated beam shear capacities to experimentally measured beam shear capacities.
- Measure hysteretic behavior which will be used to calibrate hysteretic models for use in nonlinear dynamic time-history analyses.
- Relate beams' chord rotations to the level of damage incurred in the beam.
- Compare initial stiffness and stiffness degradation of the four different beams.
- Compare energy dissipation characteristics of the four beams.

The test setup consisted of two frames, a reinforced concrete pedestal, two actuators, and a load beam. Refer to Figure 3.2(a) and Figure 3.2(b) for details of the set up. The reinforced concrete pedestal and the columns of the frames were installed on the strong floor with pretensioning rods that were stressed sufficiently to provide the necessary clamping forces required during testing.

The two frames (load frame and stabilizer frame) consisted of two steel I-shape columns connected by a steel I-shape cross beam. The load frame was outfitted with I-

shape diagonal bracing to resist out-of-plane forces. On the load frame, a main $1,779kN$ load actuator was mounted and was used to apply shear to the specimen's coupling beam. On the stabilizer frame, a $267kN$ load actuator was mounted and served to apply the necessary force required to keep the load beam horizontally-oriented during loading. The total shear imparted to the beam is the sum of the two actuator forces at any given instant. The columns of both frames were bolted to floor beams that had been fixed to the strong floor via pre-tensioned threaded rods.

The fixed wall of the specimen was mounted atop reinforced concrete pedestal while the coupling beam and the cantilevered wall were underneath the frames' cross beams. Attached to the top of the cantilevered wall pier was the load beam. The actuators were mounted directly between the top of the load beam and the bottom of the load frames' cross beams.

The reactions of the fixed wall (i.e., the wall pier clamped on the reinforced concrete pedestal) to the applied loads were provided by clamping forces developed in pre-tensioned threaded rods used to attach the fixed wall to the reinforced concrete pedestal. The load beam reactions to the applied loads were provided by clamping forces developed in pre-tensioned DYWIDAG bars which were installed so as to clamp the load beam to the cantilevered wall. To prevent lateral movement of the cantilever wall, half A-frame braces were pretensioned to the strong floor on both sides of the cantilever wall. Adjustable I-shapes, attached to the half A-frames, were used to prevent lateral movement. A sufficient amount of grease was applied to the face of the wall pier to reduce the friction between the brace and the wall. These half-A shape braces can be seen in Figure 3.2(b).

To ensure a uniform distribution of stresses at the load beam/cantilevered wall interface and the fixed wall/pedestal interface, a 19mm thick layer of hydro-stone was provided. A bed of hydro-stone was also provided at all locations where bearing plates were installed between clamping tubes and the wall surfaces. Section 3.1.2 presents the calculations for establishing the axial stresses in the wall piers resulting from the clamping forces provided to resist overturning forces during testing.

3.1.1 Design Procedure for the Test Specimen Wall Piers

The wall piers for the test specimens are designed to resist only the axial loads that will be imparted to the walls resulting from the clamping forces generated by the pre-tensioned threaded rods and DYWIDAG bars (refer to Figure 3.2(a) for details of the test setup). The total clamping forces applied to the walls provide the required forces for resisting the applied forces during testing. Originally, the pre-tensioning forces were determined such that the total axial force on the walls would approximate the axial forces the walls of a real structure would see due to gravity loads. Overturning forces due to applied loads were also considered. However, very early on in the testing of the first specimen (SCB), the load beam began to separate from the cantilever wall and the fixed wall was began to rotate relative to the concrete pedestal in which it was clamped. Therefore, the pre-tensioning forces were increased to meet the demand of the testing loads. The following calculations pertaining to the design of the wall piers consider the increased pre-tensioning forces required to resist the overturning moments from the applied loads.

The total clamping force provided on the cantilevered wall is 5,071kN and is the sum of the clamping forces provided at each of the three DWYIDAG bar locations. Three

sets of 203mmx203mmx13mm (8x8x1/2) concrete-filled HSS tubes and DYWIDAG bars were provided on the cantilever wall (see Figure 3.2(a)). For each set, there were two DYWIDAG bars. Each bar was pre-tensioned to 845kN. Therefore, the total clamping force on the cantilever wall is (6)(845kN)=5,070kN. The cross-sectional area of the wall is (254mm)(1,689mm)=429,006mm². The compressive strength of the concrete is 35MPa. Therefore,

$$f'_c A_g = \left(\frac{35N}{mm^2} \right) \left(\frac{1kN}{1,000N} \right) (429,006mm^2) = 15,015kN.$$

Thus, the percentage of $f'_c A_g$ due to the axial force from the pre-tensioned rods is (5,070/15,015)(100%)=34% and causes a stress of 11.8MPa.

The total clamping force on the fixed wall is provided by five sets of HSS tubes and threaded rods (see Figure 3.2(a)). Set A and Set C consist of a 10x6x1/2 (254mmx152mmx13mm) HSS tube and four 1in (25.4mm) diameter threaded rods and each rod is pre-tensioned to 245kN. Set B consists of a 4x4x1/2 (102mmx102mmx13mm) HSS tube and four 1in (25.4mm) diameter threaded rods and each rod is re-tensioned to 67kN. The Set D consists of a 10x6x1/2 (254mmx152mmx13mm) HSS tube and two 1in (25.4mm) diameter threaded rods and each rod is re-tensioned to 245kN. Set E consists of a 10x6x1/2 (254mmx152mmx13mm) HSS tube and four 1in (25.4mm) diameter threaded rods and each rod is pre-tensioned to 289kN. The total clamping force on the fixed wall is (10)(245kN) + (2)(67kN) + (4)(289kN) = 3,740kN. Since the fixed wall has the same cross-sectional area as the cantilever wall, $f'_c A_g=15,015kN$. Hence, the percentage $f'_c A_g$ due to the clamping force is (3,740/15,015)(100%)=24.9% and causes a stress of 8.7MPa.

The force delivered to the walls from pretensioning is transferred through 254mm square steel bearing plates that are 19mm thick. Thus the bearing stress at these locations

must be checked. The critical force applied at any one location is at the bottom of the cantilever wall and has an applied force from the rods of $(2)(845kN)=1,690kN$. This force is distributed over $(254mm)(254mm)=64,516mm^2$. The bearing stress at this location is $(1,690,000N/64,516mm^2)=26.2MPa$. The allowable bearing stress is $0.85f'_c$ which is $(0.85)(35MPa)=29.8MPa$ which is less than the applied stress of $26.2MPa$. Therefore, the bearing stresses at the locations of the threaded rods and DYWIDAG bars are acceptable. Note that a ϕ factor was not used in these bearing stress calculations.

The wall pier reinforcement consists of two curtains of #4 (12.7mm) bars at 203mm intervals. This corresponds to a reinforcement ratio of

$$\rho = \frac{(18bars)(129mm^2)}{(254mm)(1,689mm)} = 0.0054 \text{ and}$$

$$\phi_a P_{n,max} = 0.80\phi_a [0.85f'_c(A_g - A_s) + f_y A_s] \quad (2.26)$$

$$\phi_a = 0.65$$

$$f'_c = 35MPa$$

$$A_g = (254mm)(1,575mm)$$

$$A_s = (18bars)(129mm^2 / bar) = 2,322mm^2$$

$$f_y = 414MPa$$

$$\phi_a P_{n,max} = \frac{(0.8)(0.65)[(0.85)(35)(429,006 - 2,322) + (414)(2,322)]}{1,000N / kN} = 7,101kN .$$

The largest axial load is on the cantilever wall and is equal to $5,070kN$ which is less than the allowable axial load of $7,101kN$. Therefore, two curtains of #4 (12.7mm) bars at 203mm o.c. is adequate. The wall reinforcement is the same for all the test specimens. Refer to Figures 2.1, 2.3, 2.6, and 2.10 for details of the final specimen designs.

3.2 Instrumentation and Data Acquisition

3.2.1 Control and External Instrumentation

Actuator control was achieved with the use of a three-channel MTS Test Star IIM digital controller, servo valves, actuator load cells, and wire potentiometers. The stabilizer actuator was slaved to the displacement of the load actuator to ensure that the load beam remained horizontal through all phases of loading. The measured responses shown in Appendix A indicate the rotation in the cantilever wall was nominal relative to the imparted vertical displacements. Wire potentiometers, tilt meter, and DCDT's were used to measure critical displacements, deformations, and rotations. The external instrumentation used for each of the five tests is provided in Figures 3.3-3.7 for the SCB, DBCB, SPCB, FCB-2, and FCB-1 tests, respectively. Tables 3.1-3.5 tabulate the gage factors and descriptions of all devices shown in the figures.

A combination of DCDTs and wire potentiometers were installed where needed to measure:

- Rigid body displacements and rotations in the reinforced concrete pedestal.
- Rotation in the load beam.
- Rotation of the load wall.
- Relative deformation between the fixed wall and the reinforced concrete pedestal.
- Vertical displacement of load beam and coupling beam relative to the floor.

The measured local and global deformations, rotations, and displacements of the individual test setup components were taken into account in the reduction of the test data to more accurately measure the actual applied loads and displacements. The vertical displacement of the coupling beam was measured relative to the strong floor rather than relying on the actuators' displacement transducers to ensure that the reaction frames' deformations would not affect the measurements.

3.2.2 Regional and local Instrumentation

A combination of strain gages, Displacement transducers, and rosettes were utilized to capture the internal responses of the specimens. The sensors for these measurements are provided in Figures 3.3-3.7 and Tables 3.1-3.5 for all test specimens.

3.3 Loading Sequences

To impart shear to the beam of the test specimen, the actuators applied load to the load beam pretensioned atop the cantilevered wall. Note that the effects of shear in the wall piers were not simulated. Loads were applied in cycles of increasing force/displacement. To measure stiffness degradation, “drop-back” cycles were run before moving to cycles of increased amplitude. The load histories are shown in Figures 3.8-3.12 for the SCB, DBCB, SPCB, FCB-2, and FCB-1 tests, respectively. Tables 3.6-3.10 summarize the load histories and load rates for the five tests.

3.4 Test Results

3.4.1 Measured Material Properties

Material samples of the critical components in each specimen were tested to determine the actual material properties. Steel coupons were fabricated from (a) the materials that were used for flanges and webs in the steel beams, (b) plate material from the shear plate coupling beam, and (c) reinforcement material used in the concrete beams. Concrete cylinders were also prepared during the construction of the specimens to determine the compressive strength of the concrete at the time of testing. Table 3.11 provides a summary of the steel material properties and Table 3.13 provides a summary of the concrete cylinder tests and measured concrete compressive strengths. Figure 3.13 provides a plot of concrete compressive strength versus time.

3.4.2 Steel Coupling Beam (SCB) Test Results

The steel coupling beam went through 11% rotation with no real appreciable damage. Cyclic loading was applied only up to 4% due to concerns of load frame stability before the addition of lateral braces. The test was finished with monotonic loading up to 11% rotation (see Figure 3.14). The calculated shear capacity of the beam, using measured material properties, is $578kN$. Yielding of the web was determined by calculating, and comparing the principal strains, using the measured rosette strains, against the yield strains obtained from coupon tests. The web of the SCB had first yield during the 0.75% shear angle cycles at which point the strains in the top and bottom flanges were well below yield. The load and displacement at the first yielding of the web were $565kN$ and $6.8mm$, respectively. The elastic stiffness is $83kN/mm$. The calculated shear capacity of $578kN$, which corresponds to the first yielding of the web, is close to the measured value of the load at the first yield ($565kN$). The strength of the beam continued to increase with increased loading beyond first yield and reached a maximum shear of $934kN$ at approximately 3% rotation, giving a maximum ratio of applied shear to calculated shear capacity of 1.6. The force-displacement curve shown in Figure 3.15 illustrates the stable hysteretic response measured during the test.

Damage to the walls around the connection regions was moderate throughout the duration of the test. Minor hairline cracks were observed at 1% rotation. However, no appreciable cracks were observed at rotations up to 3%. Beyond 3% rotation, cracks approximately $3mm$ wide were observed on the face of the west wall. The exceptional performance is partially attributed to the large level of compressive stress in the wall piers. A summary of the damage incurred during testing is provided in Appendix A.

At the final peak force of the last cycle at the end of testing, the web and flanges were well above yield. See Table 3.13 for tabulated values of strains at (a) first yield in the web and (a) strains at the final peak force during the last cycle of testing.

3.4.3 Diagonally Reinforced Concrete Coupling Beam (DBCB) Test Results

The diagonally-reinforced concrete beam was taken up to 6% rotation at which point the damage to the beam was considered to the failure limit state and the test was stopped. See Figure 3.16 for a plot of the normalized shear to shear angle. The beam behaved favorably up to 3% rotation at which point a significant amount of the cover concrete had spalled off exposing the confined portion of the beam which was still in good condition (see Figure 3.17).

The calculated nominal shear capacity of the beam, using measured material properties, is $396kN$ and first yielding occurred in the diagonal bar groups at a shear of $387kN$ and a displacement of $7.8mm$ corresponding to an elastic stiffness equal to $50kN/mm$. The calculated capacity, which per ACI 318 corresponds to yielding of the diagonal bars, is very close to the measured local yielding. The beam continued to carry more load up to 3% rotation reaching a maximum shear of $623kN$ (see Figure 3.18) resulting in a maximum ratio of applied shear to calculated shear capacity of 1.6. The hysteretic response of the specimen is reasonably stable up to 3% rotation, but exhibits significant strength degradation beyond this level of rotation.

Beyond 3% rotation, significant degradation of the confined area was observed. Once the confined area began to degrade, the diagonal bars began to buckle when in compression which apparently amplified the degradation of the confined area (see Figure

3.19). A detailed summary of the damage incurred during testing is provided in Appendix A.

At first yielding in the diagonal bar groups, the stirrup strains and longitudinal bar strains were approximately 80% of yield indicating that the shear was resisted through a combination of the “bracing” action of the diagonal bar groups and flexural action. At the final peak force of the last loading, stirrup, longitudinal bar, and diagonal bar strains were well beyond yield. See Table 3.14 for a tabulated summary of the measured strains at (a) first yielding in the diagonal bar groups, (b) first yielding in the longitudinal bars, and (c) strains at the final peak force of the last cycle of testing.

From a design perspective, the maximum shear force-carrying capacity demonstrated by the beam is better than what current ACI codes suggest. ACI limits the nominal shear capacity of a diagonally-reinforced coupling beam to $0.83\sqrt{f'_c}$, where f'_c is in *MPa*. The peak shear resisted during the test was 623kN which corresponds to approximately $1.2\sqrt{f'_c}$, suggesting that the ACI limit on nominal shear capacity is conservative.

3.4.3.1 Overstrength of Diagonally Reinforced Coupling Beam

The calculated nominal shear capacity of the diagonally reinforced concrete coupling beam (DBCB), using the ACI 318 provisions is 396kN. Recall that ACI considers only the shear strength of the diagonal bar steel when calculating the shear strength of a DBCB (refer to Equation 2.9). However, the peak shear carried by the experimental beam was 623kN. This overstrength may be directly accounted for by considering that ACI 318 does not include the contribution of the compression strut that may develop in the beam. Additionally, the contribution of the stirrups to shear strength

is not considered in the ACI equation (Equation 2.9). Refer to Figure 3.20 for a free body diagram of the DBCB provided to support the following discussion.

In order for the compression strut to develop, some constraint to axial growth of the beam would have to be provided. In most coupled core wall systems, this axial constraint may be provided the concrete slab. However, no constraint to axial growth was provided in the test setup used in the testing of the reported experimental beam. Thus, it would not be reasonable to consider any additional strength provided by a compression strut in evaluating the shear strength of the experimental beam. However, the measured strains in the diagonal bars and stirrups indicate that the transverse reinforcement around the main core of the beam contributed to the shear strength. As will be discussed in this chapter, the strain in the stirrups was approximately 80% of their yield strength at the time when the first yield in the diagonal bars occurred. Furthermore, at the time when the peak shear force occurred, the strains in the diagonal bars and stirrups were well beyond yield. Thus, it is reasonable to assume that both the diagonal bars and the stirrups were strain hardening at the time the peak shear force occurred.

As can be seen in Figure 3.20, the shear strength of a diagonally reinforced concrete beam, considering the contribution to shear strength from a developed compression strut, transverse reinforcement around the main core, and strain hardening, is calculated as

$$V_{cap} = 2A_{vd}1.25f_{yd} \sin \alpha + f'_c A_{cd} + \frac{A_v 1.25 f_{yv} d}{s} \quad (3.2)$$

The first term in Equation 3.2 represents the contribution of the diagonal bar steel. The second and third terms represent the contribution of the contribution of the concrete compression strut and stirrups, respectively. As discussed previously, it is no reasonable

to assume that the concrete compression strut developed in the experimental beam. Thus, taking the second term of Equation 3.2 equal to zero, the calculated shear strength of the experimental beam is

$$V_{cap} = (2)(2,039)(1.25)(0.431)\sin(13) + \frac{(2)(31.6)(1.25)(0.414)(318)}{76}$$

$$V_{cap} = 494kN + 137kN$$

$$V_{cap} = 631kN$$

The ratio of V_n/V_{cap} is $623/631=0.99$ indicating that considering the contribution of the stirrups to the shear strength of the beam would be appropriate. Hence, the ACI equation for determining the shear strength of a diagonally reinforced coupling beam should be modified to include this additional strength. Furthermore, the contribution of a developed compression strut to the shear strength of the beam can reasonably be assumed in a real structure where the slab is present to constrain axial growth of the beam. However, further research is required to assess the potential of a developed compression strut.

3.4.4 Shear Plate Coupling Beam (SPCB) Test Results

The shear plate beam was taken to 6% rotation at which point, the observed correspond to the expected level of damage at failure limit state and the test was stopped. At 5% rotation, a significant amount of the cover concrete had spalled off at which point it was observed that the steel plate was sliding relative to the concrete (see Figure 3.21).

The calculated shear capacity of the beam, using measured material properties, is $418kN$. The peak measure shear force is $394kN$. The beam did not reach this load as can be seen in the force-deflection plot shown in Figures 3.22. Adequate composite action was not achieved as evident by the plate slipping shown in the photo in Figure 3.21.

Further evidence is presented in the following discussion to attempt to reconcile the behavior of the beam.

First, subsequent to testing, it was discovered that the specified total number of studs had not been installed on the plate. Per design calculations (see Section 2.3.2.2), 160 studs were required, but only 40 studs (20 studs on each side of the plate) were actually provided. Of the 40 studs provided, only 16 studs were provided within the clear span of the beam. Additionally, post-test forensic investigations revealed that all but one of the headed shear studs installed on the plate in the clear span region had sheared off during testing. There are indications of improper installation of the studs as no “weld” material was found on the plate in the areas where the headed shear studs were located. As a result of having less than required studs combined with improper installation, the headed shear studs failed very early on during testing and the expected composite action between the plate and the concrete was not achieved. However, analyses of the measured behavior indicates that some composite action was achieved.

Figure 3.23 represents the shear strain measured up to the end of the 1% shear angle cycles. The majority of shear is resisted by the concrete. However, the plate has gone through some degree of shear deformation indicating that the plate was engaged at least partially. A plot of normalized shear vs. shear strain taken up to the end of the 1.5% shear angle cycles, shown in Figure 3.24, indicates that at this degree of shear angle, the concrete strain significantly increases while the shear strain in the plate does not. At this point, all composite action between the plate and the concrete is considered to be lost.

Refer to Figure 3.25 which shows a plot of normalized shear vs. shear strain up to the end of the 3% shear angle cycles. In later cycles of testing, the displacements become

large enough that the embedded portion of the plate becomes engaged by the walls at peak amplitudes of the cycles. At the peaks of these larger cycles, the plate is deformed and dissipates some energy. It should be noted that the participation of the shear plate at this stage is primarily due to rigid body deformations between the wall piers. In other words, the wall piers engage the embedded portions of the plate and deform the plate.

A plot of the principal web strains supports the behavior seen in shear vs. strain plots. A plot of principal web strain shown in Figure 3.26, at cycles beyond 1.5% shear angles, shows spikes in the measured shear strain in the plate. Although this behavior is not considered to be due to composite action over the length of the beam, it shows that the plate contributes, albeit a small value, to the strength of the beam.

Table 3.15 summarizes the measured strains at (a) the end of the 1.5% shear angle cycles when composite action is assumed to be lost, and (b) at the peak force of the last cycle of the test. A summary of the damage incurred during testing is provided in Appendix A.

3.4.5 Fuse Steel Coupling Beam (FCB) Test Results

The testing sequence for this beam consisted of testing the 50% fuse first and taking the loading up to the point where the web of the fuse had yielded in shear while the main section remained in the elastic range. At this point, the 50% fuse was removed and replaced with the 70% fuse. Testing was restarted, and the specimen was loaded until failure.

The 50% fuse was tested up to 2% rotation at which point the fuse had reached approximately two times its calculated shear capacity. The main section had reached approximately 96% of its calculated shear capacity (see Figure 3.27 for the normalized

shear vs. shear angle plot). However, analysis of the principal strains in the web of the main section indicates that the web of the main section was approximately two times the web's yield strength during the last cycles of the test (see Figure 3.28 for a plot of the web strains). As can be seen in Figure 3.29, the peak shear force was $554kN$ at a displacement equal to $18.3mm$. The calculated shear capacities of the main and 50% fuse sections, using measured material properties, are $578kN$ and $282kN$, respectively (recall that the fuse for this specimen was intended to have half the shear capacity of the main section).

The first yield in the web of the fuse occurred at approximately $329kN$ at a displacement of $4.1mm$ resulting in an elastic stiffness of $80kN/mm$. At this point, the strains in the flanges of the fuse and main sections, as well as the web of the main section, were below yield strain which is the intended behavior of the beam. A summary of the strains in the webs and flanges of the fuse and main sections at first yield in the web of the fuse is provided in Table 3.16. At the end of the 2% shear angle cycles, the 50% fuse was removed and replaced with the 70% fuse. A summary of the residual strains in the main sections of the beam at the end of test FCB-2 (and at the beginning of test FCB-1) is provided in Table 3.16(b).

With the 70% fuse in place, test FCB-1 was started and continued up to 11% rotation (see Figure 3.30) at which point the beam was considered to have failed completely and the testing was stopped. The calculated shear capacities of the main and 70% fuse sections, using measured material properties are $578kN$ and $461kN$, respectively. First yield in the fuse of the 70% fuse occurred at a shear of approximately $485kN$ and at a displacement of $7.9mm$ giving and elastic stiffness of $61kN/mm$. This

specimen reached a maximum peak shear of $816kN$ (see Figure 3.31). The decrease in stiffness, relative to FCB-2, is in part an effect of the residual strains in the web of the main section as a result of yielding during the FCB-2 test. A summary of the strains in the flanges and webs of the fuse and main sections, at first yield in the web of the fuse, is provided in Table 3.17.

The beam behaved as intended up to 4% rotation at which point, slippage of the web splice connections was observed. Figure 3.32 shows a photograph of the evidence of the slip in the connection at this point in the test. As loading continued, damage to the welds in the main sections began to develop and continually increased in severity for the remainder of the test. Figure 3.33 shows a photograph of the weld fractures observed at the end of the 5% rotation cycles.

As discussed in Chapter 2, the main motivation for using the fuse beam is to focus damage in the fuse while keeping the main sections undamaged in order to minimize the costs of post-earthquake repair/replacement. Although the main sections of the beam had significant damage at conclusion of testing, the beam performed very well at rotation levels below 4%, which is considered to be on the high end of the expected rotations in practical coupled core wall systems using steel coupling beams.

The damage to the wall of this specimen is very similar to the damage observed during the SCB test. A detailed summary of the damage incurred during testing is provided in Appendix A.

3.5 Comparison of Test Results

The steel coupling beam specimen demonstrates superior stiffness and energy dissipation over the specimens with reinforced concrete coupling beams as demonstrated

by the more stable robust force-deflection curve for the SCB test. The steel coupling beam (SCB) has higher stiffness than that of the fuse steel beam (FCB) as can be seen in Figure 3.34 which compares the peak-to-peak stiffness of the two different steel beams through 4% rotation. The figure compares stiffness only to 4% due to the monotonic loading beyond 4% in the SCB test (all other tests were cyclic beyond 4%). Additionally, the stiffness shown for the FCB test is inclusive of the FCB-1 and FCB-2 tests. The lower stiffness in the FCB specimen, in comparison to the SCB, can be attributed to the smaller web thickness of the fuse which resulted in yielding at a lower shear than that for the SCB. This difference also accounts for the early dissipation of plastic energy exhibited by the FCB in comparison to the SCB.

Figure 3.35 shows the plastic energy dissipation of the two steel coupling beam specimens through 4% rotation. As can be seen in the graph, the plastic energy dissipation of the SCB specimen dramatically increases approaching 4% rotation at which point it dissipates more plastic energy than the FCB (the dissipation shown in the figure for the FCB consists of the combined FCB1 and FCB-2 tests). The total energy dissipated by the SCB is greater for all other beams tested (refer to Figure 3.36).

Of the two concrete coupling beams, the diagonally-reinforced beam (DBCB) exhibited superior stiffness and energy dissipation than that of the shear plate coupling beam (SPCB) as is evident by the fatter hysteretic behavior of the DBCB in comparison the SPCB. This can be directly attributed to the lack of composite action achieved in the SPCB. Figures 3.37 and 3.38 show the peak-to-peak stiffness and plastic energy dissipation, respectively, of the two concrete beams. The initial stiffness of the concrete beams is superior to the steel coupling beams. However, the stiffness degradation at very

low beam chord rotations (e.g., up to 0.5%) in the concrete beams is significantly greater than the steel coupling beams.

The SPCB shows plastic energy dissipation at early stages of loading which is not desirable. The plastic energy dissipation exhibited by the DBCB begins to increase after yielding and, as can be seen in Figure 3.38, begins to dissipate more plastic energy than the SPCB beyond 3.5% rotation. As can be seen in the accumulated plastic energy plots, the SPCB began to dissipate plastic energy at smaller shear angle than did the DBCB and up to approximately 3% rotation, dissipates more plastic energy. However, at 3% rotation, the DBCB demonstrates a higher energy dissipation than that of the SPCB. The better performance of the DBCB over the SPCB may be attributed to the fact that the required number of shear studs were not provided.

The calculated capacities of the steel coupling beam and the two fuse steel coupling beams correlate very well with the measured shears at which first yielding occurred. The ratios of $V_{yield}/V_{calculated}$ are very close to unity. The ratio for the 50% fuse specimen is 1.17 which is considered acceptable. However, the SPCB failed to reach the calculated nominal shear capacity due to reasons discussed previously. Refer to Table 3.18 for a summary of the calculated versus measured shear capacities.

Also tabulated in Table 3.18 is the ratio of $V_{max}/V_{calculated}$ which shows that the SCB and DBCB reached a maximum shear approximately 60% greater than the calculated capacity. This overstrength can be attributed to strain hardening. The fuse steel coupling beams reached a maximum shear of almost two times the calculated capacity of the fuse. This overstrength can be attributed to the greater shear capacity of the main sections in comparison to the fuse section as well as strain hardening. Overstrength in the

SPCB specimen cannot be discussed since the calculated capacity of the beam was not achieved.

Table 3.1. Instrumentation for test SCB. Refer to Figures 3.1(a) and 3.1(b).

Type	Channel	Description	Cable Number	Gage Factor / Serial #
voltage	2-0	400-kip actuator load cell	1361	n/a
voltage	2-1	400-kip actuator w-pot.	15	61.527mV/V/in / H1003995
voltage	2-2	60-kip actuator load cell	2312	n/a
voltage	2-3	60-kip actuator w-pot.	44	61.227 mV/V/in / H1003986
voltage	2-4	#2-DCDT (FW-LB)	B3	0.0819 in/V
voltage	2-5	#1-DCDT (FW-LT)	5	0.1271 in/V
voltage	2-6	#18-DCDT (FW-RB)	10	0.1177 in/V
voltage	2-7	#13-DCDT (FW-RT)	K12	0.1237 in/V
voltage	2-8	#3-DCDT (Web-Up)	M5	0.1217 in/V
voltage	2-9	#15-DCDT (Web-Down)	F8	0.1203
voltage	2-10	15" w-pot. (LW-L)	9	61.213 mV/V/in / I0961968
voltage	2-11	15" w-pot. (LW-R)	A1	62.614 mV/V/in / H1003994
voltage	2-12	Tilt Meter (LW)	n/a	16.27
voltage	2-13	10" w-pot. (Beam Mid-Span)	G15	92.660 mV/V/in / J0962401
Strain	5-0	SG11	J	2.13
Strain	5-1	SG12	14	2.13
Strain	5-2	SG10	R	2.13
Strain	5-3	SG9	U	2.13
Strain	5-4	SG3	J4	2.13
Strain	5-5	SG4	J7	2.13
Strain	5-6	SG1	A3	2.13
Strain	5-7	SG2	11	2.13
Strain	6-0	SG13	4	2.13
Strain	6-1	SG14	J0	2.13
Strain	6-2	SG17	A10	2.13
Strain	6-3	SG18	E12	2.13
Strain	6-4	SG15A	W12	1.98
Strain	6-5	SG15B	T10	1.98
Strain	6-6	SG15C	N	1.98
Strain	6-7	SG16A	T	1.98
Strain	7-0	SG16B	S	1.98
Strain	7-1	SG16C	ZZ	1.98
Strain	7-2	SG7A	T11	1.98
Strain	7-3	SG7B	TU	1.98
Strain	7-4	SG7C	O	1.98
Strain	7-5	SG8A	13	1.98
Strain	7-6	SG8B	Y	1.98
Strain	7-7	SG8C	K13	1.98
Strain	8-0	SG5A	15	1.98
Strain	8-1	SG5B	J5	1.98
Strain	8-2	SG5C	D12	1.98
Strain	8-3	SG6A	J3	1.98
Strain	8-4	SG6B	12	1.98
Strain	8-5	SG6C	V	1.98

Table 3.2. Instrumentation for test DBCB. Refer to Figures 3.2(a) and 3.2(b).

Type	Channel	Description	Cable Number	Gage Factor / Serial #
voltage	2-0	400-kip actuator load cell	1361	n/a
voltage	2-1	400-kip actuator w-pot.	15	61.527mV/V/in/(1.5) H1003995
voltage	2-2	60-kip actuator load cell	2312	n/a
voltage	2-3	60-kip actuator w-pot.	44	61.227 mV/V/in/ H1003986
voltage	2-4	#2-DCDT (FW-LB)	B3	0.0819 in/V
voltage	2-5	#1-DCDT (FW-LT)	5	0.1271 in/V
voltage	2-6	#18-DCDT (FW-RB)	10	0.1177 in/V
voltage	2-7	#13-DCDT (FW-RT)	7	0.1237 in/V
voltage	2-8	#3-DCDT (Beam-Down)	M5	0.1217 in/V
voltage	2-9	#15-DCDT (Beam-Up)	F8	0.1203
voltage	2-10	15" w-pot. (LW-L)	9	61.213 mV/V/in / I0961968
voltage	2-11	15" w-pot. (LW-R)	A1	62.647 mV/V/in / H1003994
voltage	2-12	Tilt Meter (LW)	n/a	need to get
voltage	2-13	Control w-pot	3A	1.5
Strain	5-0	SG1	J2	2.13
Strain	5-1	SG2	14	2.13
Strain	5-2	SG3	R	2.13
Strain	5-3	SG4	U	2.13
Strain	5-4	SG5	J4	2.13
Strain	5-5	SG6	J7	2.13
Strain	5-6	SG7	A3	2.13
Strain	5-7	SG8	11	2.13
Strain	6-0	SG9	4	2.13
Strain	6-1	SG10	J0	2.13
Strain	6-2	SG11	A10	2.13
Strain	6-3	SG12	E12	2.13
Strain	6-4	SG13	W12	2.13
Strain	6-5	SG14	T10	2.13
Strain	6-6	SG15	N	2.13
Strain	6-7	SG16	T	2.13
Strain	7-0	SG17	S	2.13
Strain	7-1	SG18	ZZ	2.13

Table 3.3. Instrumentation for test SPCB. Refer to Figures 3.3(a) and 3.3(b).

Type	Channel	Description	Cable Number	Gage Factor / Serial #
voltage	2-0	400-kip actuator load cell	1361	n/a
voltage	2-1	400-kip actuator w-pot.	15	61.527mV/V/in/(1.5) H1003995
voltage	2-2	60-kip actuator load cell	2312	n/a
voltage	2-3	60-kip actuator w-pot.	44	61.227 mV/V/in/ H1003986
voltage	2-4	#2-DCDT (FW-LB)	B3	0.0819 in/V
voltage	2-5	#1-DCDT (FW-LT)	5	0.1271 in/V
voltage	2-6	#18-DCDT (FW-RB)	10	0.1177 in/V
voltage	2-7	#13-DCDT (FW-RT)	7	0.1237 in/V
voltage	2-8	#3-DCDT (Beam-Down)	M5	0.1217 in/V
voltage	2-9	#15-DCDT (Beam-Up)	F8	0.1203
voltage	2-10	15" w-pot. (LW-L)	9	61.213 mV/V/in / I0961968
voltage	2-11	15" w-pot. (LW-R)	A1	62.647 mV/V/in / H1003994
voltage	2-12	Tilt Meter (LW)	n/a	16.27
voltage	2-13	Control w-pot	3A	1.5
voltage	2-14	#19 DCDT (Plate-Up)	X	0.1244 in/V
voltage	2-15	#7 DCDT (Plate-Down)	BC	0.1218 in/V
Strain	5-0	SG1A	J2	1.98
Strain	5-1	SG1B	14	1.98
Strain	5-2	SG1C (BAD SG)	R	1.98
Strain	5-3	SG2A	U	1.98
Strain	5-4	SG2B	J4	1.98
Strain	5-5	SG2C	J7	1.98
Strain	5-6	SG3A	A3	1.98
Strain	5-7	SG3B	11	1.98
Strain	6-0	SG3C	4	1.98
Strain	6-1	SG4A	J0	1.98
Strain	6-2	SG4B	A10	1.98
Strain	6-3	SG4C	E12	1.98
Strain	6-4	SG5A	W12	1.98
Strain	6-5	SG5B	T10	1.98
Strain	6-6	SG5C	N	1.98
Strain	6-7	SG6A	T	1.98
Strain	7-0	SG6B	S	1.98
Strain	7-1	SG6C	ZZ	1.98
Strain	7-2	SG7A	T11	1.98
Strain	7-3	SG7B	TU	1.98
Strain	7-4	SG7C (BAD SG)	O	1.98
Strain	7-5	SG8	13	2.13
Strain	7-6	SG9	Y	2.13
Strain	7-7	SG10	K13	2.13
Strain	8-0	SG11	15	2.13
Strain	8-1	SG12	J5	2.13
Strain	8-2	SG13	D12	2.13
Strain	8-3	SG14 (BAD SG)	J3	2.13
Strain	8-4	SG15	12	2.13
Strain	8-5	SG16	V	2.13
Strain	8-6	SG17 (BAD SG)	Z12	2.13

Table 3.4. Instrumentation for test FCB-2. Refer to Figures 3.4(a) and 3.4(b).

Type	Channel	Description	Cable Number	Gage Factor / Serial #
voltage	2-0	400-kip actuator load cell	1361	n/a
voltage	2-1	400-kip actuator w-pot.	15	61.527mV/V/in/(1.5) H1003995
voltage	2-2	60-kip actuator load cell	2312	n/a
voltage	2-3	60-kip actuator w-pot.	44	61.227 mV/V/in/ H1003986
voltage	2-4	#2-DCDT (FW-LB)	B3	0.0819 in/V
voltage	2-5	#1-DCDT (FW-LT)	5	0.1271 in/V
voltage	2-6	#18-DCDT (FW-RB)	10	0.1177 in/V
voltage	2-7	#13-DCDT (FW-RT)	7	0.1237 in/V
voltage	2-8	#3-DCDT (Beam-Down)	M5	0.1217 in/V
voltage	2-9	#15-DCDT (Beam-Up)	F8	0.1203
voltage	2-10	15" w-pot. (LW-L)	9	61.213 mV/V/in / I0961968
voltage	2-11	15" w-pot. (LW-R)	A1	62.647 mV/V/in / H1003994
voltage	2-12	Tilt Meter (LW)	n/a	16.27
voltage	2-13	Control w-pot	3A	1.5
Strain	5-0	SG6A	J2	1.98
Strain	5-1	SG6B	14	1.98
Strain	5-2	SG6C	R	1.98
Strain	5-3	SG7A	U	1.98
Strain	5-4	SG7B	J4	1.98
Strain	5-5	SG7C	J7	1.98
Strain	5-6	SG8A	A3	1.98
Strain	5-7	SG8B	11	1.98
Strain	6-0	SG8C	4	1.98
Strain	6-1	SG1	J0	2.13
Strain	6-2	SG2	A10	2.13
Strain	6-3	SG3	E12	2.13
Strain	6-4	SG4	W12	2.13
Strain	6-5	SG9	T10	2.13
Strain	6-6	SG10	N	2.13
Strain	6-7	SG11	T	2.13
Strain	7-0	SG12	S	2.13
Strain	7-1	SG13A	ZZ	2.01
Strain	7-2	SG13B	T11	2.01
Strain	7-3	SG13C	TU	2.01
Strain	7-4	SG14A	O	1.89
Strain	7-5	SG14B	13	1.89
Strain	7-6	SG14C	Y	1.89
Strain	7-7	SG15A	K13	2.01
Strain	8-0	SG15B	15	2.01
Strain	8-1	SG15B	J5	2.01
Strain	8-2	SG16	D12	2.13
Strain	8-3	SG17	J3	2.13
Strain	8-4	SG18	12	2.13
Strain	8-5	SG19	V	2.13
Strain	8-6	SG20	Z12	2.13
Strain	8-7	SG21	WW	2.13

Table 3.5. Instrumentation for test FCB-1. Refer to Figures 3.5(a) and 3.5(b).

Type	Channel	Description	Cable Number	Gage Factor / Serial #
voltage	2-0	400-kip actuator load cell	1361	n/a
voltage	2-1	400-kip actuator w-pot.	15	61.527mV/V/in/(1.5) H1003995
voltage	2-2	60-kip actuator load cell	2312	n/a
voltage	2-3	60-kip actuator w-pot.	44	61.227 mV/V/in/ H1003986
voltage	2-4	#2-DCDT (FW-LB)	B3	0.0819 in/V
voltage	2-5	#1-DCDT (FW-LT)	5	0.1271 in/V
voltage	2-6	#18-DCDT (FW-RB)	10	0.1177 in/V
voltage	2-7	#13-DCDT (FW-RT)	7	0.1237 in/V
voltage	2-8	#3-DCDT (Beam-Down)	M5	0.1217 in/V
voltage	2-9	#15-DCDT (Beam-Up)	F8	0.1203
voltage	2-10	15" w-pot. (LW-L)	9	61.213 mV/V/in / I0961968
voltage	2-11	15" w-pot. (LW-R)	A1	62.647 mV/V/in / H1003994
voltage	2-12	Tilt Meter (LW)	n/a	16.27
voltage	2-13	Control w-pot	3A	1.5
Strain	5-0	SG6A	J2	1.98
Strain	5-1	SG6B	14	1.98
Strain	5-2	SG6C	R	1.98
Strain	5-3	SG7A	U	1.98
Strain	5-4	SG7B	J4	1.98
Strain	5-5	SG7C	J7	1.98
Strain	5-6	SG8A	A3	1.98
Strain	5-7	SG8B	11	1.98
Strain	6-0	SG8C	4	1.98
Strain	6-1	SG1	J0	2.13
Strain	6-2	SG2	A10	2.13
Strain	6-3	SG3	E12	2.13
Strain	6-4	SG4	W12	2.13
Strain	6-5	SG9	T10	2.13
Strain	6-6	SG10	N	2.13
Strain	6-7	SG11	T	2.13
Strain	7-0	SG12	S	2.13
Strain	7-1	SG13A	ZZ	2.01
Strain	7-2	SG13B	T11	2.01
Strain	7-3	SG13C	TU	2.01
Strain	7-4	SG14A	O	1.89
Strain	7-5	SG14B	13	1.89
Strain	7-6	SG14C	Y	1.89
Strain	7-7	SG15A	K13	2.01
Strain	8-0	SG15B	15	2.01
Strain	8-1	SG15B	J5	2.01
Strain	8-2	SG16	D12	2.13
Strain	8-3	SG17	J3	2.13
Strain	8-4	SG18	12	2.13
Strain	8-5	SG19	V	2.13
Strain	8-6	SG20	Z12	2.13
Strain	8-7	SG21	WW	2.13

Table 3.6. Load history data for SCB test. Refer to Figures 3.6(a) and 3.6(b).

Load History SCB									
Test Control					Experiment Control				
Cycle #	Load (kN)	Time (min)	Rate (kN/min)	Time (min)	Time (min)	Time (min)	Time (min)	Time (min)	Time (min)
Cycle #	Load (kN)	Time (min)	Rate (kN/min)	Time (min)	Time (min)	Time (min)	Time (min)	Time (min)	Time (min)
0	0	0	0	0	0	0	0	0	0
01a	17.79	1.00	17.79	1.00	1.83	1.00	1.83	1.00	1.83
01a	17.79	2.00	17.79	3.00	1.83	2.00	1.83	2.00	1.83
01a	0.00	1.00	17.79	4.00	0.00	1.00	1.83	6.00	1.83
01b	17.79	1.00	17.79	5.00	1.83	1.00	1.83	6.00	1.83
01b	17.79	2.00	17.79	7.00	1.83	2.00	1.83	6.00	1.83
01b	0.00	1.00	17.79	8.00	0.00	1.00	1.83	6.00	1.83
01c	17.79	1.00	17.79	9.00	1.83	1.00	1.83	6.00	1.83
01c	17.79	2.00	17.79	11.00	1.83	2.00	1.83	6.00	1.83
01c	0.00	1.00	17.79	12.00	0.00	1.00	1.83	6.00	1.83
02a	35.58	1.00	35.58	13.00	1.83	1.00	1.83	6.00	1.83
02a	35.58	2.00	35.58	15.00	1.83	2.00	1.83	6.00	1.83
02a	0.00	1.00	35.58	16.00	0.00	1.00	1.83	6.00	1.83
02b	35.58	1.00	35.58	17.00	1.83	1.25	1.83	7.25	1.83
02b	35.58	2.00	35.58	19.00	1.83	2.25	1.83	7.25	1.83
02b	0.00	1.00	35.58	20.00	0.00	1.25	1.83	7.25	1.83
02c	35.58	1.00	35.58	21.00	1.83	1.25	1.83	7.25	1.83
02c	35.58	2.00	35.58	23.00	1.83	2.25	1.83	7.25	1.83
02c	0.00	1.00	35.58	24.00	0.00	1.25	1.83	7.25	1.83
03a	53.38	1.00	53.38	25.00	2.29	1.25	1.83	8.25	1.83
03a	53.38	2.00	53.38	27.00	2.29	2.25	1.83	8.25	1.83
03a	0.00	1.00	53.38	28.00	0.00	1.25	1.83	8.25	1.83
03b	53.38	1.00	53.38	29.00	1.83	1.00	1.83	8.25	1.83
03b	53.38	2.00	53.38	31.00	1.83	2.00	1.83	8.25	1.83
03b	0.00	1.00	53.38	32.00	0.00	1.00	1.83	8.25	1.83
03c	53.38	1.00	53.38	33.00	4.57	2.50	1.83	9.50	1.83
03c	53.38	2.00	53.38	35.00	4.57	3.50	1.83	9.50	1.83
03c	0.00	1.00	53.38	36.00	0.00	2.50	1.83	10.00	1.83
03d	53.38	1.00	53.38	37.00	4.57	2.50	1.83	10.50	1.83
03d	53.38	2.00	53.38	39.00	4.57	3.50	1.83	10.50	1.83
03d	0.00	1.00	53.38	40.00	0.00	2.50	1.83	11.00	1.83
04a	71.17	1.00	71.17	41.00	4.57	2.50	1.83	11.50	1.83
04a	71.17	2.00	71.17	43.00	4.57	3.50	1.83	11.50	1.83
04a	0.00	1.00	71.17	44.00	0.00	2.50	1.83	12.00	1.83
04b	71.17	1.00	71.17	45.00	2.29	1.25	1.83	12.25	1.83
04b	71.17	2.00	71.17	47.00	2.29	2.25	1.83	12.25	1.83
04b	0.00	1.00	71.17	48.00	0.00	1.25	1.83	12.25	1.83
04c	71.17	1.00	71.17	49.00	6.86	3.75	1.83	12.75	1.83
04c	71.17	2.00	71.17	51.00	6.86	4.75	1.83	12.75	1.83
04c	0.00	1.00	71.17	52.00	0.00	3.75	1.83	13.00	1.83
04d	55.58	1.00	55.58	53.00	6.86	3.75	1.83	13.25	1.83
04d	55.58	2.00	55.58	55.00	6.86	4.75	1.83	13.25	1.83
04d	0.00	1.00	55.58	56.00	0.00	3.75	1.83	13.25	1.83

Table 3.7. Load history data for DBCB test. Refer to Figures 3.7(a) and 3.7(b).

Load History for Test DRCB													
Force Control						Displacement Control							
Cycle #	Load (kN)	Time (min)	Rate (kN/min)	Time Acc. (min)	Cycle #	Displacement (mm)	Time (min)	Rate (mm/min)	Time Acc. (min)	Cycle #	Displacement (mm)	Time (min)	Rate (mm/min)
01a	0.00	0.00	0.00	0	01a		0.00	0.00	0.00	139		0.00	0.00
	17.79	1.00	17.79	1			1.83	1.00	1.83	55		6.86	3.75
	-17.79	2.00	17.79	3			-1.83	2.00	1.83	57	09b	-6.86	7.50
01b	0.00	1.00	17.79	4	01b		0.00	1.00	1.83	58		0.00	3.75
	17.79	1.00	17.79	5			1.83	1.00	1.83	59		6.86	3.75
	-17.79	2.00	17.79	7		06b	-1.83	2.00	1.83	61	09c	-6.86	7.50
02a	0.00	1.00	17.79	8	02a		0.00	1.00	1.83	62		0.00	3.75
	35.58	1.00	35.58	9			1.83	1.00	1.83	63		4.57	2.50
	-35.58	2.00	35.58	11		06c	-1.83	2.00	1.83	65	09d	-4.57	5.00
02b	0.00	1.00	35.58	12	02b		0.00	1.00	1.83	66		0.00	2.50
	35.58	1.00	35.58	13			1.52	1.00	1.52	67		9.14	2.50
	-35.58	2.00	35.58	15		06d	-1.52	2.00	1.52	69	10a	-9.14	5.00
03a	0.00	1.00	35.58	16	03a		0.00	1.00	1.52	70		0.00	2.50
	53.38	1.00	53.38	17			2.29	1.25	1.83	71		9.14	2.50
	-53.38	2.00	53.38	19		07a	-2.29	2.50	1.83	74	10b	-9.14	5.00
03b	0.00	1.00	53.38	20	03b		0.00	1.25	1.83	75		0.00	2.50
	53.38	1.00	53.38	21			2.29	1.25	1.83	76		9.14	2.50
	-53.38	2.00	53.38	23		07b	-2.29	2.50	1.83	79	10c	-9.14	5.00
03d	0.00	1.00	35.58	24	03d		0.00	1.25	1.83	80		0.00	2.50
	35.58	1.00	35.58	25			2.29	1.25	1.83	81		6.86	3.75
	-35.58	2.00	35.58	27		07c	-2.29	2.50	1.83	84	10d	-6.86	7.50
04a	0.00	1.00	35.58	28	04a		0.00	1.25	1.83	85		0.00	3.75
	71.17	1.00	71.17	29			1.83	1.00	1.83	86		13.72	3.75
	-71.17	2.00	71.17	31		07d	-1.83	2.00	1.83	88	11a	-13.72	7.50
04b	0.00	1.00	71.17	32	04b		0.00	1.00	1.83	89		0.00	3.75
	71.17	1.00	71.17	33			4.57	2.50	1.83	92		13.72	3.75
	-71.17	2.00	71.17	35		08a	-4.57	5.00	1.83	97	11b	-13.72	7.50
04d	0.00	1.00	71.17	36	04d		0.00	2.50	1.83	99		0.00	3.75
	53.38	1.00	53.38	37			4.57	2.50	1.83	102		13.72	3.75
	-53.38	2.00	53.38	39		08b	-4.57	5.00	1.83	107	11c	-13.72	7.50
05a	0.00	1.00	53.38	40	05a		0.00	2.50	1.83	109		0.00	3.75
	88.96	1.25	71.17	41			4.57	2.50	1.83	113		9.14	2.50
	-88.96	2.50	71.17	44		08c	-4.57	5.00	1.83	117	11d	-9.14	5.00
05b	0.00	1.25	71.17	45	05b		0.00	2.50	1.83	119		0.00	2.50
	88.96	1.25	71.17	46			2.29	1.25	1.83	120		18.29	2.50
	-88.96	2.50	71.17	48		08d	-2.29	2.50	1.83	123	12a	-18.29	5.00
05d	0.00	1.25	71.17	50	05d		0.00	1.25	1.83	124		0.00	2.50
	71.17	1.00	71.17	51			6.86	3.75	1.83	128		18.29	2.50
	-71.17	2.00	71.17	53		09a	-6.86	7.50	1.83	135	12b	-18.29	5.00
	0.00	1.00	71.17	54		0.00	3.75	1.83	139		0.00	2.50	

Table 3.8. Load history data for SPCB test. Refer to Figures 3.8(a) and 3.8(b).

Load History for Test SPCB																			
Force Control					Displacement Control														
Cycle #	Load (kN)	Time (min)	Rate (kN/min)	Time Acc. (min)	Cycle #	Displacement (mm)	Time (min)	Rate (mm/min)	Time Acc. (min)	Cycle #	Displacement (mm)	Time (min)	Rate (mm/min)	Time Acc. (min)	Cycle #	Displacement (mm)	Time (min)	Rate (mm/min)	Time Acc. (min)
	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
01a	17.79	1.00	17.79	1.00	06a	1.83	1.00	1.83	55.00	09b	-6.86	3.75	1.83	142.75	12c	18.29	2.50	7.32	301.50
	-17.79	2.00	17.79	3.00		-1.83	2.00	1.83	57.00		-6.86	7.50	1.83	150.25		-18.29	5.00	7.32	306.50
	0.00	1.00	17.79	4.00		0.00	1.00	1.83	58.00		0.00	3.75	1.83	154.00		0.00	2.50	7.32	309.00
01b	17.79	1.00	17.79	5.00	06b	1.83	1.00	1.83	59.00	09c	6.86	3.75	1.83	157.75	12d	13.72	3.75	3.66	312.75
	-17.79	2.00	17.79	7.00		-1.83	2.00	1.83	61.00		-6.86	7.50	1.83	165.25		-13.72	7.50	3.66	320.25
	0.00	1.00	17.79	8.00		0.00	1.00	1.83	62.00		0.00	3.75	1.83	169.00		0.00	3.75	3.66	324.00
02a	35.58	1.00	35.58	9.00	06c	1.83	1.00	1.83	63.00	09d	4.57	2.50	1.83	171.50	13a	27.43	3.75	7.32	327.75
	-35.58	2.00	35.58	11.00		-1.83	2.00	1.83	65.00		-4.57	5.00	1.83	176.50		-27.43	7.50	7.32	335.25
	0.00	1.00	35.58	12.00		0.00	1.00	1.83	66.00		0.00	2.50	1.83	179.00		0.00	3.75	7.32	339.00
02b	35.58	1.00	35.58	13.00	06d	1.52	1.00	1.52	67.00	10a	9.14	2.50	3.66	181.50	13b	27.43	3.75	7.32	342.75
	-35.58	2.00	35.58	15.00		-1.52	2.00	1.52	69.00		-9.14	5.00	3.66	186.50		-27.43	7.50	7.32	350.25
	0.00	1.00	35.58	16.00		0.00	1.00	1.52	70.00		0.00	2.50	3.66	189.00		0.00	3.75	7.32	354.00
03a	53.38	1.00	53.38	17.00	07a	2.29	1.25	1.83	71.25	10b	9.14	2.50	3.66	191.50	13c	27.43	3.75	7.32	357.75
	-53.38	2.00	53.38	19.00		-2.29	2.50	1.83	73.75		-9.14	5.00	3.66	196.50		-27.43	7.50	7.32	365.25
	0.00	1.00	53.38	20.00		0.00	1.25	1.83	75.00		0.00	2.50	3.66	199.00		0.00	3.75	7.32	369.00
03b	53.38	1.00	53.38	21.00	07b	2.29	1.25	1.83	76.25	10c	9.14	2.50	3.66	201.50	13d	18.29	2.50	7.32	371.50
	-53.38	2.00	53.38	23.00		-2.29	2.50	1.83	78.75		-9.14	5.00	3.66	206.50		-18.29	5.00	7.32	376.50
	0.00	1.00	53.38	24.00		0.00	1.25	1.83	80.00		0.00	2.50	3.66	209.00		0.00	2.50	7.32	379.00
03d	35.58	1.00	35.58	25.00	07c	2.29	1.25	1.83	81.25	10d	6.86	3.75	1.83	212.75	14a	36.58	3.75	9.75	382.75
	-35.58	2.00	35.58	27.00		-2.29	2.50	1.83	83.75		-6.86	7.50	1.83	220.25		-36.58	7.50	9.75	390.25
	0.00	1.00	35.58	28.00		0.00	1.25	1.83	85.00		0.00	3.75	1.83	224.00		0.00	3.75	9.75	394.00
04a	71.17	1.00	71.17	29.00	07d	1.83	1.00	1.83	86.00	11a	13.72	3.75	3.66	227.75	14b	36.58	3.75	9.75	397.75
	-71.17	2.00	71.17	31.00		-1.83	2.00	1.83	88.00		-13.72	7.50	3.66	235.25		-36.58	7.50	9.75	405.25
	0.00	1.00	71.17	32.00		0.00	1.00	1.83	89.00		0.00	3.75	3.66	239.00		0.00	3.75	9.75	409.00
04b	71.17	1.00	71.17	33.00	08a	4.57	2.50	1.83	91.50	11b	13.72	3.75	3.66	242.75	14d	27.43	3.75	7.32	412.75
	-71.17	2.00	71.17	35.00		-4.57	5.00	1.83	96.50		-13.72	7.50	3.66	250.25		-27.43	7.50	7.32	420.25
	0.00	1.00	71.17	36.00		0.00	2.50	1.83	99.00		0.00	3.75	3.66	254.00		0.00	3.75	7.32	424.00
04d	53.38	1.00	53.38	37.00	08b	4.57	2.50	1.83	101.50	11c	13.72	3.75	3.66	257.75	15a	45.72	3.75	12.19	427.75
	-53.38	2.00	53.38	39.00		-4.57	5.00	1.83	106.50		-13.72	7.50	3.66	265.25		-45.72	7.50	12.19	435.25
	0.00	1.00	53.38	40.00		0.00	2.50	1.83	109.00		0.00	3.75	3.66	269.00		0.00	3.75	12.19	439.00
05a	88.96	1.25	71.17	41.25	08c	4.57	2.50	1.83	111.50	11d	9.14	2.50	3.66	271.50	16a	54.86	4.50	12.19	443.50
	-88.96	2.50	71.17	43.75		-4.57	5.00	1.83	116.50		-9.14	5.00	3.66	276.50		-54.86	9.00	12.19	452.50
	0.00	1.25	71.17	45.00		0.00	2.50	1.83	119.00		0.00	2.50	3.66	279.00		0.00	4.50	12.19	457.00
05b	88.96	1.25	71.17	46.25	08d	2.29	1.25	1.83	120.25	12a	18.29	2.50	7.32	281.50	16b	54.86	4.50	12.19	461.50
	-88.96	2.50	71.17	48.75		-2.29	2.50	1.83	122.75		-18.29	5.00	7.32	286.50		-54.86	9.00	12.19	470.50
	0.00	1.25	71.17	50.00		0.00	1.25	1.83	124.00		0.00	2.50	7.32	289.00		0.00	4.50	12.19	475.00
05d	71.17	1.00	71.17	51.00	09a	6.86	3.75	1.83	127.75	12b	18.29	2.50	7.32	291.50					
	-71.17	2.00	71.17	53.00		-6.86	7.50	1.83	135.25		-18.29	5.00	7.32	296.50					
	0.00	1.00	71.17	54.00		0.00	3.75	1.83	139.00		0.00	2.50	7.32	299.00					

Table 3.9. Load history data for FCB-2 test. Refer to Figures 3.9(a) and 3.9(b).

Load History for Test FCB-2														
Force Control					Displacement Control									
Cycle #	Load (kN)	Time (min)	Rate (kN/min)	Time Acc. (min)	Cycle #	Displacement (mm)	Time (min)	Rate mm/min	Time Acc. (min)	Cycle #	Displacement (mm)	Time (min)	Rate mm/min	Time Acc. (min)
	0.00	0.00	0.00	0.00		0.00	0.00	0.00	54.00		0.00	0.00	0.00	139.00
01a	17.79	1.00	17.79	1.00	06a	1.83	1.00	1.83	55.00	09b	6.86	3.75	1.83	142.75
	-17.79	2.00	17.79	3.00		-1.83	2.00	1.83	57.00		-6.86	7.50	1.83	150.25
	0.00	1.00	17.79	4.00		0.00	1.00	1.83	58.00		0.00	3.75	1.83	154.00
	17.79	1.00	17.79	5.00		1.83	1.00	1.83	59.00		6.86	3.75	1.83	157.75
01b	-17.79	2.00	17.79	7.00	06b	-1.83	2.00	1.83	61.00	09c	-6.86	7.50	1.83	165.25
	0.00	1.00	17.79	8.00		0.00	1.00	1.83	62.00		0.00	3.75	1.83	169.00
	17.79	1.00	17.79	9.00		1.83	1.00	1.83	63.00		6.86	3.75	1.83	171.50
02a	-35.58	2.00	35.58	11.00	06c	-1.83	2.00	1.83	65.00	09d	-4.57	5.00	1.83	176.50
	0.00	1.00	35.58	12.00		0.00	1.00	1.83	66.00		0.00	2.50	1.83	179.00
	35.58	1.00	35.58	13.00		1.52	1.00	1.52	67.00		9.14	2.50	3.66	181.50
02b	-35.58	2.00	35.58	15.00	06d	-1.52	2.00	1.52	69.00	10a	-9.14	5.00	3.66	186.50
	0.00	1.00	35.58	16.00		0.00	1.00	1.52	70.00		0.00	2.50	3.66	189.00
	53.38	1.00	53.38	17.00		2.29	1.25	1.83	71.25		9.14	2.50	3.66	191.50
03a	-53.38	2.00	53.38	19.00	07a	-2.29	2.50	1.83	73.75	10b	-9.14	5.00	3.66	196.50
	0.00	1.00	53.38	20.00		0.00	1.25	1.83	75.00		0.00	2.50	3.66	199.00
	53.38	1.00	53.38	21.00		2.29	1.25	1.83	76.25		9.14	2.50	3.66	201.50
03b	-53.38	2.00	53.38	23.00	07b	-2.29	2.50	1.83	78.75	10c	-9.14	5.00	3.66	206.50
	0.00	1.00	53.38	24.00		0.00	1.25	1.83	80.00		0.00	2.50	3.66	209.00
	35.58	1.00	35.58	25.00		2.29	1.25	1.83	81.25		6.86	3.75	1.83	212.75
03d	-35.58	2.00	35.58	27.00	07c	-2.29	2.50	1.83	83.75	10d	-6.86	7.50	1.83	220.25
	0.00	1.00	35.58	28.00		0.00	1.25	1.83	85.00		0.00	3.75	1.83	224.00
	71.17	1.00	71.17	29.00		1.83	1.00	1.83	86.00		13.72	3.75	3.66	227.75
04a	-71.17	2.00	71.17	31.00	07d	-1.83	2.00	1.83	88.00	11a	-13.72	7.50	3.66	235.25
	0.00	1.00	71.17	32.00		0.00	1.00	1.83	89.00		0.00	3.75	3.66	239.00
	71.17	1.00	71.17	33.00		4.57	2.50	1.83	91.50		13.72	3.75	3.66	242.75
04b	-71.17	2.00	71.17	35.00	08a	-4.57	5.00	1.83	96.50	11b	-13.72	7.50	3.66	250.25
	0.00	1.00	71.17	36.00		0.00	2.50	1.83	99.00		0.00	3.75	3.66	254.00
	53.38	1.00	53.38	37.00		4.57	2.50	1.83	101.50		13.72	3.75	3.66	257.75
04d	-53.38	2.00	53.38	39.00	08b	-4.57	5.00	1.83	106.50	11c	-13.72	7.50	3.66	265.25
	0.00	1.00	53.38	40.00		0.00	2.50	1.83	109.00		0.00	3.75	3.66	269.00
	88.96	1.25	71.17	41.25		4.57	2.50	1.83	111.50		9.14	2.50	3.66	271.50
05a	-88.96	2.50	71.17	43.75	08c	-4.57	5.00	1.83	116.50	11d	-9.14	5.00	3.66	276.50
	0.00	1.25	71.17	45.00		0.00	2.50	1.83	119.00		0.00	2.50	3.66	279.00
	88.96	1.25	71.17	46.25		2.29	1.25	1.83	120.25		18.29	2.50	7.32	281.50
05b	-88.96	2.50	71.17	48.75	08d	-2.29	2.50	1.83	122.75	12a	-18.29	5.00	7.32	286.50
	0.00	1.25	71.17	50.00		0.00	1.25	1.83	124.00		0.00	2.50	7.32	289.00
	71.17	1.00	71.17	51.00		6.86	3.75	1.83	127.75					
05d	-71.17	2.00	71.17	53.00	09a	-6.86	7.50	1.83	135.25					
	0.00	1.00	71.17	54.00		0.00	3.75	1.83	139.00					

Table 3.10. Load history data for FCB-1 test. Refer to Figures 3.10(a) and 3.10(b).

Force Control										Displacement Control									
Cycle #	Load (kN)	Time (min)	Rate (mm/min)	Time Acc. (min)	Time (min)	Rate (mm/min)	Time Acc. (min)	Cycle #	Time Acc. (min)	Time (min)	Rate (mm/min)	Time Acc. (min)	Time (min)	Rate (mm/min)	Time Acc. (min)	Cycle #	Time Acc. (min)	Time (min)	Rate (mm/min)
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
01a	18	1.00	18	1.00	1.83	1.00	1.83	06a	1.83	1.00	1.83	1.00	1.83	1.00	1.83	16a	1.83	1.00	1.83
01b	-18	2.00	18	3.00	-1.83	2.00	-1.83	06b	-1.83	2.00	-1.83	2.00	-1.83	2.00	-1.83	16b	-1.83	2.00	-1.83
01c	0	1.00	18	4.00	0.00	1.00	0.00	06c	0.00	1.00	0.00	1.00	0.00	1.00	0.00	16c	0.00	1.00	0.00
01d	18	1.00	18	5.00	1.83	1.00	1.83	06d	1.83	1.00	1.83	1.00	1.83	1.00	1.83	16d	1.83	1.00	1.83
01e	-18	2.00	18	6.00	-1.83	2.00	-1.83	06e	-1.83	2.00	-1.83	2.00	-1.83	2.00	-1.83	16e	-1.83	2.00	-1.83
01f	0	1.00	18	8.00	0.00	1.00	0.00	06f	0.00	1.00	0.00	1.00	0.00	1.00	0.00	16f	0.00	1.00	0.00
01g	36	1.00	36	9.00	3.66	1.00	3.66	06g	3.66	1.00	3.66	1.00	3.66	1.00	3.66	16g	3.66	1.00	3.66
01h	-36	2.00	36	11.00	-3.66	2.00	-3.66	06h	-3.66	2.00	-3.66	2.00	-3.66	2.00	-3.66	16h	-3.66	2.00	-3.66
01i	0	1.00	36	12.00	0.00	1.00	0.00	06i	0.00	1.00	0.00	1.00	0.00	1.00	0.00	16i	0.00	1.00	0.00
01j	36	1.00	36	13.00	3.66	1.00	3.66	06j	3.66	1.00	3.66	1.00	3.66	1.00	3.66	16j	3.66	1.00	3.66
01k	-36	2.00	36	14.00	-3.66	2.00	-3.66	06k	-3.66	2.00	-3.66	2.00	-3.66	2.00	-3.66	16k	-3.66	2.00	-3.66
01l	0	1.00	36	16.00	0.00	1.00	0.00	06l	0.00	1.00	0.00	1.00	0.00	1.00	0.00	16l	0.00	1.00	0.00
01m	53	1.00	53	17.00	5.30	1.00	5.30	06m	5.30	1.00	5.30	1.00	5.30	1.00	5.30	16m	5.30	1.00	5.30
01n	-53	2.00	53	19.00	-5.30	2.00	-5.30	06n	-5.30	2.00	-5.30	2.00	-5.30	2.00	-5.30	16n	-5.30	2.00	-5.30
01o	0	1.00	53	20.00	0.00	1.00	0.00	06o	0.00	1.00	0.00	1.00	0.00	1.00	0.00	16o	0.00	1.00	0.00
01p	53	1.00	53	21.00	5.30	1.00	5.30	06p	5.30	1.00	5.30	1.00	5.30	1.00	5.30	16p	5.30	1.00	5.30
01q	-53	2.00	53	23.00	-5.30	2.00	-5.30	06q	-5.30	2.00	-5.30	2.00	-5.30	2.00	-5.30	16q	-5.30	2.00	-5.30
01r	0	1.00	53	24.00	0.00	1.00	0.00	06r	0.00	1.00	0.00	1.00	0.00	1.00	0.00	16r	0.00	1.00	0.00
01s	36	1.00	36	25.00	3.66	1.00	3.66	06s	3.66	1.00	3.66	1.00	3.66	1.00	3.66	16s	3.66	1.00	3.66
01t	-36	2.00	36	27.00	-3.66	2.00	-3.66	06t	-3.66	2.00	-3.66	2.00	-3.66	2.00	-3.66	16t	-3.66	2.00	-3.66
01u	0	1.00	36	28.00	0.00	1.00	0.00	06u	0.00	1.00	0.00	1.00	0.00	1.00	0.00	16u	0.00	1.00	0.00
01v	71	1.00	71	29.00	7.10	1.00	7.10	06v	7.10	1.00	7.10	1.00	7.10	1.00	7.10	16v	7.10	1.00	7.10
01w	-71	2.00	71	31.00	-7.10	2.00	-7.10	06w	-7.10	2.00	-7.10	2.00	-7.10	2.00	-7.10	16w	-7.10	2.00	-7.10
01x	0	1.00	71	32.00	0.00	1.00	0.00	06x	0.00	1.00	0.00	1.00	0.00	1.00	0.00	16x	0.00	1.00	0.00
01y	71	1.00	71	33.00	7.10	1.00	7.10	06y	7.10	1.00	7.10	1.00	7.10	1.00	7.10	16y	7.10	1.00	7.10
01z	-71	2.00	71	35.00	-7.10	2.00	-7.10	06z	-7.10	2.00	-7.10	2.00	-7.10	2.00	-7.10	16z	-7.10	2.00	-7.10
02a	0	1.00	71	36.00	0.00	1.00	0.00	07a	0.00	1.00	0.00	1.00	0.00	1.00	0.00	17a	0.00	1.00	0.00
02b	53	1.00	53	37.00	5.30	1.00	5.30	07b	5.30	1.00	5.30	1.00	5.30	1.00	5.30	17b	5.30	1.00	5.30
02c	-53	2.00	53	39.00	-5.30	2.00	-5.30	07c	-5.30	2.00	-5.30	2.00	-5.30	2.00	-5.30	17c	-5.30	2.00	-5.30
02d	0	1.00	53	40.00	0.00	1.00	0.00	07d	0.00	1.00	0.00	1.00	0.00	1.00	0.00	17d	0.00	1.00	0.00
02e	89	1.25	71	41.25	8.90	1.25	8.90	07e	8.90	1.25	8.90	1.25	8.90	1.25	8.90	17e	8.90	1.25	8.90
02f	-89	2.50	71	43.75	-8.90	2.50	-8.90	07f	-8.90	2.50	-8.90	2.50	-8.90	2.50	-8.90	17f	-8.90	2.50	-8.90
02g	0	1.25	71	45.00	0.00	1.25	0.00	07g	0.00	1.25	0.00	1.25	0.00	1.25	0.00	17g	0.00	1.25	0.00
02h	89	1.25	71	46.25	8.90	1.25	8.90	07h	8.90	1.25	8.90	1.25	8.90	1.25	8.90	17h	8.90	1.25	8.90
02i	-89	2.50	71	48.75	-8.90	2.50	-8.90	07i	-8.90	2.50	-8.90	2.50	-8.90	2.50	-8.90	17i	-8.90	2.50	-8.90
02j	0	1.25	71	50.00	0.00	1.25	0.00	07j	0.00	1.25	0.00	1.25	0.00	1.25	0.00	17j	0.00	1.25	0.00
02k	71	1.00	71	51.00	7.10	1.00	7.10	07k	7.10	1.00	7.10	1.00	7.10	1.00	7.10	17k	7.10	1.00	7.10
02l	-71	2.00	71	53.00	-7.10	2.00	-7.10	07l	-7.10	2.00	-7.10	2.00	-7.10	2.00	-7.10	17l	-7.10	2.00	-7.10
02m	0	1.00	71	54.00	0.00	1.00	0.00	07m	0.00	1.00	0.00	1.00	0.00	1.00	0.00	17m	0.00	1.00	0.00

Table 3.11. Measured steel material properties.

Steel Properties				
Component Description	F_y (MPa)	ϵ_y	F_u (MPa)	F_u/F_y
DBCB - #6 Longitudinal Bars	418	0.00224	610	1.46
DBCB - #8 Diagonal Bars	431	0.00219	692	1.60
SPCB - #9 Longitudinal Bars (MMFX)	860	0.00401	1262	1.47
FCB-2 - Fuse Web	243	0.00139	397	1.64
FCB-2 - Fuse Flange	305	0.00127	495	1.62
FCB-1 - Fuse Web	264	0.00119	418	1.58
SPCB - Web Plate	294	0.00148	403	1.37
SCB, FCB-1 - Main Section Web	245	0.00153	424	1.73
SCB, FCB-1, FCB-2 - Main Section Flange	277	0.00126	473	1.70
#2 Stirrups for DBCB and SPCB	414	0.00239	624	1.51

Table 3.12. Concrete cylinder test data.

7-Day Test					118-Day Test (SCB Test)									
Cylinder Number	Diam. (mm)	Area (mm ²)	Failure Load (kN)	Compressive Strength (Mpa)	Cylinder Number	Diam. (mm)	Area (mm ²)	Failure Load (kN)	Compressive Strength (Mpa)					
1	152	18232	434	23.8	1	152	18232	634	34.8					
2	152	18232	423	23.2	2	152	18232	638	35.0					
3	152	18232	454	24.9	3	152	18232	633	34.7					
4	152	18232	467	25.6	4	152	18232	674	37.0					
5	152	18232	464	25.4	$\mu_{118} = 35.4$			$\sigma_{118} = 1.1$						
6	152	18232	432	23.7	157-Day Test (SPCB Test)									
7	152	18232	443	24.3	Cylinder Number	Diam. (mm)	Area (mm ²)	Failure Load (kN)	Compressive Strength (Mpa)					
8	152	18232	508	27.9										
9	152	18232	518	28.4										
10	152	18232	483	26.5										
11	152	18232	493	27.1	1	152	18232	637	34.9					
12	152	18232	457	25.1	2	152	18232	640	35.1					
13	152	18232	461	25.3	3	152	18232	718	39.4					
14	152	18232	484	26.6	4	152	18232	634	34.8					
15	152	18232	483	26.5	$\mu_{157} = 36.0$			$\sigma_{157} = 2.2$						
16	152	18232	516	28.3	177-Day Test (DBCB Test)									
$\mu_7 = 25.8$			$\sigma_7 = 1.6$		Cylinder Number	Diam. (mm)	Area (mm ²)	Failure Load (kN)	Compressive Strength (Mpa)					
28-Day Test														
Cylinder Number	Diam. (mm)	Area (mm ²)	Failure Load (kN)	Compressive Strength (Mpa)						1	152	18232	696	38.2
										2	152	18232	625	34.3
					3	152	18232	701	38.4					
1	152	18232	630	34.6	4	152	18232	718	39.4					
2	152	18232	646	35.5	$\mu_{177} = 37.6$			$\sigma_{177} = 2.3$						
3	152	18232	639	35.1	212-Day Test (FCB-1 & 2 Test)									
4	152	18232	540	29.6	Cylinder Number	Diam. (mm)	Area (mm ²)	Failure Load (kN)	Compressive Strength (Mpa)					
5	152	18232	637	34.9										
6	152	18232	620	34.0										
7	152	18232	623	34.2										
8	152	18232	639	35.1	1	152	18232	643	35.3					
9	152	18232	599	32.8	2	152	18232	681	37.4					
	152	18232			3	152	18232	609	33.4					
10	152	18232	643	35.3	4	152	18232	732	40.2					
$\mu_{28} = 34.1$			$\sigma_{28} = 1.8$		$\mu_{212} = 36.5$			$\sigma_{212} = 2.9$						

Table 3.13(a). Strain data at first yield in web of SCB (see Figure 3.3 for gage numbers).

Strains at First Yield in Web (SCB)								
Loading Values					F_y		ϵ_y	
Force (kN)	Δ_y (mm)	Cycle #	Shear Angle	E_s (MPa)	Flange (MPa)	Web (MPa)	Flange (μ)	Web (μ)
565	5.2	12A	0.50%	199946	277	245	1386	1228
Strain Gage Location		Fixed Wall		Beam Span		Load Wall		
Gage Numbers		4	3	14	13	2	1	
Top Flange Strains		256	862	912	-285	-396	117	
Gage Numbers		8	7	16	15	6	5	
Web Principal Strains		154	482	1414	1276	335	137	
Gage Numbers		12	11	18	17	10	9	
Bottom Flange Strains		160	-464	-776	351	765	303	
Strains Normalized to Yield Strain								
Strain Gage Location		Fixed Wall		Beam Span		Load Wall		
Gage Numbers		4	3	14	13	2	1	
Top Flange Strains		0.185	0.622	0.658	-0.206	-0.286	0.084	
Gage Numbers		8	7	16	15	6	5	
Web Principal Strains		0.125	0.393	1.152	1.039	0.273	0.112	
Gage Numbers		12	11	18	17	10	9	
Bottom Flange Strains		0.115	-0.335	-0.560	0.253	0.552	0.219	

Table 3.13(b). Strain data at final peak force at last cycle of SCB test (see Figure 3.3 for gage numbers).

Strains at Final Peak Force (SCB)								
Loading Values					F_y		ϵ_y	
Force (kN)	Δ_y (mm)	Cycle #	Shear Angle	E_s (MPa)	Flange (MPa)	Web (MPa)	Flange (μ)	Web (μ)
992	98.6	Mono	10.80%	199946	277	245	1386	1228
Strain Gage Location		Fixed Wall		Beam Span		Load Wall		
Gage Numbers		4	3	14	13	2	1	
Top Flange Strains		867	5538	1885	328	lost	159	
Gage Numbers		8	7	16	15	6	5	
Web Principal Strains		717	2959	43221	45223	2511	767	
Gage Numbers		12	11	18	17	10	9	
Bottom Flange Strains		238	-1178	-215	1164	2784	976	
Strains Normalized to Yield Strain								
Strain Gage Location		Fixed Wall		Beam Span		Load Wall		
Gage Numbers		4	3	14	13	2	1	
Top Flange Strains		0.625	3.995	1.360	0.237	lost	0.115	
Gage Numbers		8	7	16	15	6	5	
Web Principal Strains		0.584	2.410	35.208	36.839	2.045	0.625	
Gage Numbers		12	11	18	17	10	9	
Bottom Flange Strains		0.172	-0.850	-0.155	0.840	2.008	0.704	

Table 3.14(a). Strain data at first yield in diagonal bars of DBCB test (see Figure 3.4 for gage numbers).

Strains at First Yield in Diagonal Bars - DBCB										
Loading Values				E _s (Mpa)	F _y			ε _y		
Force (kN)	Δ _y (mm)	Cycle #	Shear Angle		Diag. Bars (Mpa)	Stirrup (Mpa)	Long. Bars (Mpa)	Diag. Bars (μ)	Stirrup (μ)	Long. Bars (μ)
387	7.8	10A	1.00%	199946	432	414	419	2159	2069	2093
Strain Gage Location					Fixed Wall		Beam Span		Load Wall	
Gage Numbers					n/a	10	14	6	2	n/a
Top Longitudinal Bar Strains					n/a	942	1297	-703	bad	n/a
Gage Numbers					n/a	9	13	7	3	n/a
Diagonal Bar Group #1 Strains					lost	440	-76	823	1032	n/a
Gage Numbers					n/a	11	15	5	1	n/a
Diagonal Bar Group #2 Strains					n/a	1165	2190	-808	bad	n/a
Gage Numbers					n/a	12	16	8	4	n/a
Bottom Longitudinal Bar Strains					n/a	16	446	1739	1285	n/a
Gage Numbers					n/a	n/a	18	17	n/a	n/a
Stirrup Strains					n/a	n/a	1714	-558	n/a	n/a
Strains Normalized to Yield Strain										
Strain Gage Location					Fixed Wall		Beam Span		Load Wall	
Gage Numbers					n/a	10	14	6	2	n/a
Top Longitudinal Bar Strains					n/a	0.450	0.620	-0.336	bad	n/a
Gage Numbers					n/a	9	13	7	3	n/a
Diagonal Bar Group #1 Strains					lost	0.204	-0.035	0.381	0.478	n/a
Gage Numbers					n/a	11	15	5	1	n/a
Diagonal Bar Group #2 Strains					n/a	0.540	1.015	-0.374	bad	n/a
Gage Numbers					n/a	12	16	8	4	n/a
Bottom Longitudinal Bar Strains					n/a	0.008	0.213	0.831	0.614	n/a
Gage Numbers					n/a	n/a	18	17	n/a	n/a
Stirrup Strains					n/a	n/a	0.828	-0.270	n/a	n/a

Table 3.14(b). Strain data at first yield in longitudinal bars of DBCB test (see Figure 3.4 for gage numbers).

Strains at First Yield in Longitudinal Bars - DBCB										
Loading Values				E _s (Mpa)	F _y			ε _y		
Force (kN)	Δ _y (mm)	Cycle #	Shear Angle		Diag. Bars (MPa)	Stirrup (MPa)	Long. Bars (MPa)	Diag. Bars (μ)	Stirrup (μ)	Long. Bars (μ)
471	10.2	11A	1.50%	199946	432	414	419	2159	2069	2093
Strain Gage Location					Fixed Wall		Beam Span		Load Wall	
Gage Numbers					n/a	10	14	6	2	n/a
Top Longitudinal Bar Strains					n/a	1176	1499	-847	bad	n/a
Gage Numbers					n/a	9	13	7	3	n/a
Diagonal Bar Group #1 Strains					lost	485	-758	1012	1312	n/a
Gage Numbers					n/a	11	15	5	1	n/a
Diagonal Bar Group #2 Strains					n/a	1470	3247	-1084	bad	n/a
Gage Numbers					n/a	12	16	8	4	n/a
Bottom Longitudinal Bar Strains					n/a	49	277	2102	1592	n/a
Gage Numbers					n/a	n/a	18	17	n/a	n/a
Stirrup Strains					n/a	n/a	2328	-539	n/a	n/a
Strains Normalized to Yield Strain										
Strain Gage Location					Fixed Wall		Beam Span		Load Wall	
Gage Numbers					n/a	10	14	6	2	n/a
Top Longitudinal Bar Strains					n/a	0.562	0.716	-0.405	bad	n/a
Gage Numbers					n/a	9	13	7	3	n/a
Diagonal Bar Group #1 Strains					lost	0.225	-0.351	0.469	0.608	n/a
Gage Numbers					n/a	11	15	5	1	n/a
Diagonal Bar Group #2 Strains					n/a	0.681	1.504	-0.502	bad	n/a
Gage Numbers					n/a	12	16	8	4	n/a
Bottom Longitudinal Bar Strains					n/a	0.023	0.132	1.004	0.761	n/a
Gage Numbers					n/a	n/a	18	17	n/a	n/a
Stirrup Strains					n/a	n/a	1.125	-0.261	n/a	n/a

Table 3.14(c). Strain data at final peak force during last cycle of DBCB test (see Figure 3.3 for gage numbers).

Strains at Final Peak Force - DBCB										
Loading Values				E _s (MPa)	F _y			ε _y		
Force (kN)	Δ _y (mm)	Cycle #	Shear Angle		Diag. Bars (MPa)	Stirrup (MPa)	Long. Bars (MPa)	Diag. Bars (μ)	Stirrup (μ)	Long. Bars (μ)
471	10.2	11A	1.20%	199946	432	414	419	2159	2069	2093
Strain Gage Location					Fixed Wall		Beam Span		Load Wall	
Gage Numbers					n/a	10	14	6	2	n/a
Top Longitudinal Bar Strains					n/a	1787	lost	-2351	bad	n/a
Gage Numbers					n/a	9	13	7	3	n/a
Diagonal Bar Group #1 Strains					lost	824	-4179	3145	1851	n/a
Gage Numbers					n/a	11	15	5	1	n/a
Diagonal Bar Group #2 Strains					n/a	718	lost	lost	bad	n/a
Gage Numbers					n/a	12	16	8	4	n/a
Bottom Longitudinal Bar Strains					n/a	-850	-2064	447	178	n/a
Gage Numbers					n/a	n/a	18	17	n/a	n/a
Stirrup Strains					n/a	n/a	2328	6219	n/a	n/a
Strains Normalized to Yield Strain										
Strain Gage Location					Fixed Wall		Beam Span		Load Wall	
Gage Numbers					n/a	10	14	6	2	n/a
Top Longitudinal Bar Strains					n/a	0.854	lost	-1.123	bad	n/a
Gage Numbers					n/a	9	13	7	3	n/a
Diagonal Bar Group #1 Strains					lost	0.382	-1.936	1.457	0.857	n/a
Gage Numbers					n/a	11	15	5	1	n/a
Diagonal Bar Group #2 Strains					n/a	0.333	lost	lost	bad	n/a
Gage Numbers					n/a	12	16	8	4	n/a
Bottom Longitudinal Bar Strains					n/a	-0.406	-0.986	0.214	0.085	n/a
Gage Numbers					n/a	n/a	18	17	n/a	n/a
Stirrup Strains					n/a	n/a	1.125	3.006	n/a	n/a

Table 3.15(a). Strain data at end of 1.5% shear angle cycles for SPCB test (point of loss of composite action) (see Figure 3.5 for gage numbers).

Strains at Loss of Composite Action - SPCB										
Loading Values				E _s (MPa)	F _y			ε _y		
Force (kN)	Δ _y (mm)	Cycle #	Shear Angle		Plate (MPa)	Stirrup (MPa)	Long. Bars (MPa)	Plate (μ)	Stirrup (μ)	Long. Bars (μ)
-334	-13.1	11A	1.50%	199946	294	414	860	1469	2069	4303
Strain Gage Location					Fixed Wall		Beam Span		Load Wall	
Gage Numbers					n/a	11	10	n/a	9	8
Top Long. Bar Strains					n/a	-159	-184	n/a	2392	1911
Gage Numbers					7	6	5	4	3	2
Steel Plate Principal Strains					lost	624	lost	261	841	843
Gage Numbers					n/a	15	14	n/a	13	12
Bottom Long. Bar Strains					n/a	1529	lost	n/a	521	-34
Gage Numbers					n/a	n/a	17	n/a	16	n/a
Stirrup Strains					n/a	n/a	lost	n/a	2799	n/a
Strains Normalized to Yield Strain										
Strain Gage Location					Fixed Wall		Beam Span		Load Wall	
Gage Numbers					n/a	11	10	n/a	9	8
Top Long. Bar Strains					n/a	-0.037	-0.043	n/a	0.556	0.444
Gage Numbers					7	6	5	4	3	2
Steel Plate Principal Strains					lost	0.425	lost	0.178	0.573	0.574
Gage Numbers					n/a	15	14	n/a	13	12
Bottom Long. Bar Strains					n/a	0.355	lost	n/a	0.121	-0.008
Gage Numbers					n/a	n/a	17	n/a	16	n/a
Stirrup Strains					n/a	n/a	lost	n/a	1.353	n/a

Table 3.15(b). Strain data at final peak force of last cycle tested for SPCB test (see Figure 3.5 for gage numbers).

Strains at Final Peak Force - SPCB										
Loading Values				F_y				ϵ_y		
Force (kN)	Δ_y (mm)	Cycle #	Shear Angle	E_s (MPa)	Plate (MPa)	Stirrup (MPa)	Long. Bars (MPa)	Plate (μ)	Stirrup (μ)	Long. Bars (μ)
-334	-13.1	11A	1.43%	199946	294	414	860	1469	2069	4303
Strain Gage Location				Fixed Wall		Beam Span			Load Wall	
Gage Numbers				n/a	11	10	n/a	9	8	n/a
Top Long. Bar Strains				n/a	635	4571	n/a	lost	505	n/a
Gage Numbers				7	6	5	4	3	2	1
Steel Plate Principal Strains				lost	849	lost	lost	lost	744	n/a
Gage Numbers				n/a	15	14	n/a	13	12	n/a
Bottom Long. Bar Strains				n/a	321	lost	n/a	lost	749	n/a
Gage Numbers				n/a	n/a	17	n/a	16	n/a	n/a
Stirrup Strains				n/a	n/a	lost	n/a	lost	n/a	n/a
Strains Normalized to Yield Strain										
Strain Gage Location				Fixed Wall		Beam Span			Load Wall	
Gage Numbers				n/a	11	10	n/a	9	8	n/a
Top Long. Bar Strains				n/a	0.148	1.062	n/a	lost	0.117	n/a
Gage Numbers				7	6	5	4	3	2	1
Steel Plate Principal Strains				lost	0.578	lost	lost	lost	0.506	n/a
Gage Numbers				n/a	15	14	n/a	13	12	n/a
Bottom Long. Bar Strains				n/a	0.075	lost	n/a	lost	0.174	n/a
Gage Numbers				n/a	n/a	17	n/a	16	n/a	n/a
Stirrup Strains				n/a	n/a	lost	n/a	lost	n/a	n/a

Table 3.16(a). Strain data first yield in web of fuse of the 50% fuse (FCB-2 test) (see Figure 3.6 for gage numbers).

Strains at First Yield in Fuse Web (FCB-2; 50% Fuse)												
Loading Values				F_y				ϵ_y				
				E_s (MPa)	Main Section		Fuse Section		Main Section		Fuse Section	
Force (kN)	Δ_y (mm)	Cycle #	Shear Angle		Flange (MPa)	Web (MPa)	Flange (MPa)	Web (MPa)	Flange (μ)	Web (μ)	Flange (μ)	Web (μ)
329	4.1	08B	0.45%	199946	277	243	305	243	1386	1214	1524	1214
Strain Gage Location					Fixed Wall		Beam Span			Load Wall		
Gage Numbers					4	3	18	17	16	2	1	
Top Flange Strains					113	254	421	98	-509	-27	-8	
Gage Numbers					8	7	15	14	13	6	5	
Web Principal Strains					55	169	484	1269	498	129	n/a	
Gage Numbers					12	11	21	20	19	10	9	
Bottom Flange Strains					3	-67	-553	128	455	261	115	
Strains Normalized to Yield Strain												
Strain Gage Location					Fixed Wall		Beam Span			Load Wall		
Gage Numbers					4	3	18	17	16	2	1	
Top Flange Strains					0.08	0.18	0.30	0.06	-0.37	-0.02	-0.01	
Gage Numbers					8	7	15	14	13	6	5	
Web Principal Strains					0.04	0.14	0.39	1.05	0.41	0.11	n/a	
Gage Numbers					12	11	21	20	19	10	9	
Bottom Flange Strains					0.00	-0.05	-0.40	0.08	0.33	0.19	0.08	

Table 3.16(b). Strain data at the end of the 50% fuse (FCB-2 test) (see Figure 3.6 for gage numbers).

Strains at End of Test (FCB-2; 50% Fuse)												
Loading Values					F_y				ϵ_y			
					Main Section		Fuse Section		Main Section		Fuse Section	
Force (kN)	Δ_y (mm)	Cycle #	Shear Angle	E_s (MPa)	Flange (MPa)	Web (MPa)	Flange (MPa)	Web (MPa)	Flange (μ)	Web (μ)	Flange (μ)	Web (μ)
n/a	n/a	13	2.00%	199946	277	243	305	243	1386	1214	1524	1214
Strain Gage Location					Fixed Wall		Beam Span			Load Wall		
Gage Numbers					4	3	18	17	16	2	1	
Top Flange Strains					111	305	-45	651	-276	149	114	
Gage Numbers					8	7	15	14	13	6	5	
Web Principal Strains					87	196	86	2074	433	89	n/a	
Gage Numbers					12	11	21	20	19	10	9	
Bottom Flange Strains					3	-67	-553	128	455	261	115	
Strains Normalized to Yield Strain												
Strain Gage Location					Fixed Wall		Beam Span			Load Wall		
Gage Numbers					4	3	18	17	16	2	1	
Top Flange Strains					0.08	0.22	-0.03	0.43	-0.20	0.11	0.08	
Gage Numbers					8	7	15	14	13	6	5	
Web Principal Strains					0.07	0.16	0.07	1.71	0.35	0.07	n/a	
Gage Numbers					12	11	21	20	19	10	9	
Bottom Flange Strains					0.09	0.02	-0.54	0.20	-0.45	0.12	0.07	

Table 3.17(a). Strain data first yield in the web of the 70% fuse (FCB-1 test) (see Figure 3.7 for gage numbers).

Strains at First Yield in Fuse Web (FCB-1; 70% Fuse)												
Loading Values					F_y				ϵ_y			
					Main Section		Fuse Section		Main Section		Fuse Section	
Force (kN)	Δ_y (mm)	Cycle #	Shear Angle	E_s (MPa)	Flange (MPa)	Web (MPa)	Flange (MPa)	Web (MPa)	Flange (μ)	Web (μ)	Flange (μ)	Web (μ)
485	7.9	10A	1.00%	199946	277	265	277	265	1386	1324	1386	1324
Strain Gage Location					Fixed Wall		Beam Span			Load Wall		
Gage Numbers					4	3	18	17	16	2	1	
Top Flange Strains					216	673	856	820	-928	21	-85	
Gage Numbers					8	7	15	14	13	6	5	
Web Principal Strains					132	314	932	1334	723	159	n/a	
Gage Numbers					12	11	21	20	19	10	9	
Bottom Flange Strains					3	-67	-553	128	455	261	115	
Strains Normalized to Yield Strain												
Strain Gage Location					Fixed Wall		Beam Span			Load Wall		
Gage Numbers					4	3	18	17	16	2	1	
Top Flange Strains					0.16	0.49	0.62	0.59	-0.67	0.02	-0.06	
Gage Numbers					8	7	15	14	13	6	5	
Web Principal Strains					0.11	0.26	0.76	1.01	0.59	0.13	n/a	
Gage Numbers					12	11	21	20	19	10	9	
Bottom Flange Strains					0.09	-0.21	-0.80	0.21	0.33	0.38	0.13	

Table 3.17(b). Strain data at the final peak force in the last cycle of test FCB-1 (70% fuse) (see Figure 3.7 for gage numbers).

Strains at Final Peak Force (FCB-1; 70% Fuse)												
Loading Values					F_v				ϵ_v			
Force (kN)	Δ_y (mm)	Cycle #	Shear Angle	E_s (MPa)	Main Section		Fuse Section		Main Section		Fuse Section	
					Flange (MPa)	Web (MPa)	Flange (MPa)	Web (MPa)	Flange (μ)	Web (μ)	Flange (μ)	Web (μ)
712	94.7	21A	10.40%	199946	277	265	277	265	1386	1324	1386	1324
Strain Gage Location					Fixed Wall		Beam Span			Load Wall		
Gage Numbers					4	3	18	17	16	2	1	
Top Flange Strains					518	1515	-4884	-6519	-2119	177	50	
Gage Numbers					8	7	15	14	13	6	5	
Web Principal Strains					364	687	702	2978	1051	358	n/a	
Gage Numbers					12	11	21	20	19	10	9	
Bottom Flange Strains					3	-67	-553	128	455	261	115	
Strains Normalized to Yield Strain												
Strain Gage Location					Fixed Wall		Beam Span			Load Wall		
Gage Numbers					4	3	18	17	16	2	1	
Top Flange Strains					0.37	1.09	-3.52	-4.70	-1.53	0.13	0.04	
Gage Numbers					8	7	15	14	13	6	5	
Web Principal Strains					0.30	0.56	0.57	2.25	0.86	0.29	n/a	
Gage Numbers					12	11	21	20	19	10	9	
Bottom Flange Strains					0.02	-0.36	-10.57	-5.95	-1.61	-0.69	0.27	

Table 3.18. Calculated versus measured shear capacities.

Calculated v. Measured Capacities					
Test Specimen	$V_{calc.}$ (kN)	V_{yield} (kN)	V_{max} (kN)	$V_{yield}/V_{calc.}$	$V_{max}/V_{calc.}$
SCB	578	565	934	0.98	1.62
DBCB	396	387	623	0.98	1.57
SPCB	418	n/a	394	n/a	0.94
FCB-2 (50% fuse)	282	329	554	1.17	1.96
FCB-1 (70% fuse)	461	485	816	1.05	1.77

BEAM TYPE	MAIN OBJECTIVE(S)	FINAL BEAM SECTION
UNENCASED STEEL COUPLING BEAM (SCB-1)	Design and detail beam and connection region using established guidelines. Investigate behavior when used to couple walls with no special wall boundary elements. Potential for decreasing congestion over length of beam's embedment region.	<p>25.4x127 PLATE, TOP AND BOTTOM FLANGES 12x305 WEB 19mm THICK STIFFENER PLATES; BOTH SIDES OF WEB</p>
DIAGONALLY-BRACED REINFORCED CONCRETE COUPLING BEAM (DBC-1)	Design and detail beam per ACI 318-02 provisions with the exception of service stress level limit requirements given by eq. 21-9. Investigate behavior when used to couple walls with no special wall boundary elements. Potential for decreasing congestion over development length of diagonal and longitudinal steel.	<p>(2)-#6 (19mm) LONG. BARS TOP AND BOTTOM, w/ #2 (3.2mm) STIRRUPS @ 76mm o.c. 127x114 DIAGONAL CORE w/ (4)-#8 (25.4mm) BARS AND #3 (9.5mm) TIES @ 76mm o.c., TYPICAL; DIAGONALS INCLINED AT 13° FROM HORIZ. 1'-2" 10"</p>
WEB SHEAR PLATE COUPLING BEAM (SPCB-1)	Extend Subedi's plate reinforced beam model to include the contribution of conventional transverse reinforcement shear resistance. Attractive solution to mitigating steel congestion over the clear span of the beam and along the connection region. Potential for achieving high service shear stress levels.	<p>PLATE 6.4x279 w/ 1/4" Ø (6.4mm) HEADED STUDS BOTH SIDES OF PLATE #2 (3.2mm) STIRRUPS @ 76mm o.c. (2)-#8 (25.4mm) MMFX BARS TOP AND BOTTOM 1'-2" 10"</p>
UNENCASED FUSED STEEL COUPLING BEAM (FCB-1 & FCB-2)	Design and detail main section same as SCB-1. Design central fuse to have 50% shear capacity as that of the main section. In addition to minimizing congestion along the connection region, potential for appreciable reduction in post-earthquake repair/replacement costs.	<p>25.4x127 PLATE, TOP AND BOTTOM FLANGES 12x305 WEB, MAIN BEAM 9.5x305 WEB, FCB-1 6.4x305 WEB, FCB-2 19mm THICK STIFFENER PLATES; BOTH SIDES OF WEB</p>

Figure 3.1. Summary of coupling beam cross sections tested.

Figure 3.2(a). South elevation of test setup.

Figure 3.2(b). South elevation of test setup with a specimen installed.

Figure 3.3(a). Strain gage instrumentation for SCB test (1in=25.4mm).

Figure 3.3(b). Force, displacement, and rotation transducers for SCB Test (1in=25.4mm; 1kip=4.448kN).

Figure 3.4(a). Strain gage instrumentation for DBCB test (1in=25.4mm).

Figure 3.4(b). Force, displacement, and rotation transducers for DBCB Test ($1\text{in}=25.4\text{mm}$; $1\text{kip}=4.448\text{kN}$).

Figure 3.5(a). Strain gage instrumentation for SPCB test ($1\text{in}=25.4\text{mm}$).

Figure 3.5(b). Force, displacement, and rotation transducers for SPCB Test
(1in=25.4mm; 1kip=4.448kN).

Figure 3.6(a). Strain gage instrumentation for 50% fuse (FCB-2) (1in=25.4mm).

Figure 3.6(b). Force, displacement, and rotation transducers for FCB-2 test (50% fuse) (1in=25.4mm; 1kip=4.448kN).

Figure 3.7(a). Strain gage instrumentation for 70% fuse (FCB-1) (1in=25.4mm).

Figure 3.7(b). Force, displacement, and rotation transducers for FCB-1 test (70% fuse) (1in=25.4mm; 1kip=4.448kN).

Figure 3.8(a). First phase of load history in force control (up to $\pm 71kN$) for test SCB.

Figure 3.8(b). Second phase of load history in displacement control (cyclic to $\pm 36.6mm$, monotonic to $-100.6mm$) for test SCB.

Figure 3.9(a). First phase of load history in force control (up to $\pm 89kN$) for test DBCB.

Figure 3.9(b). Second phase of load history in displacement control (up to $\pm 54.9mm$) for test DBCB.

Figure 3.10(a). First phase of load history in force control (up to $\pm 89kN$) for test SPCB.

Figure 3.10(b). Second phase of load history in displacement control (up to $\pm 54.9mm$) for test SPCB.

Figure 3.11(a). First phase of load history in force control (up to $\pm 89kN$) for test FCB-2.

Figure 3.11(b). Second phase of load history in displacement control (up to $\pm 18.3mm$) for test FCB-2.

Figure 3.12(a). First phase of load history in force control (up to $\pm 89kN$) for test FCB-1.

Figure 3.12(b). Second phase of load history in displacement control (up to $\pm 100.6mm$) for test FCB-1.

Figure 3.13. Concrete cylinder compressive strength v. time.

Figure 3.14. Normalized shear vs. shear angle for SCB test.

Figure 3.15. Force-deflection for SCB test.

Figure 3.16. Normalized shear vs. shear angle for DBCB test.

Figure 3.17. Photograph of condition of DBCB at end of 3% rotation cycles.

Figure 3.18. Force-deflection for DBCB test.

Figure 3.19. Photograph of buckled diagonal bars during 5% rotation cycles.

Figure 3.20. Free body diagram of a DBCB showing diagonal bar steel, compression strut, and stirrup forces.

Figure 3.21. Photos taken of SPCB test at the end of the 5% shear angle cycles. Evidence of the steel plate has been sliding relative to the concrete.

Figure 3.22. Force-deflection of SPCB test.

Figure 3.23. Shear vs. shear strain plotted up to the end of the 1% shear angle cycles.

Figure 3.24. Shear vs. shear strain plotted up to the end of the 1.5% shear angle cycles.

Figure 3.25. Shear vs. shear strain plotted up to end of 3% shear angle cycles.

Figure 3.26. Principal strains in web of SPCB test.

Figure 3.27. Normalized shear vs. shear angle for FCB-2 Test.

Figure 3.28. Principal strains in webs of fuse and main sections for FCB-2 (50% fuse).

Figure 3.29. Force-deflection for FCB-2 (50% fuse).

Figure 3.30. Normalized shear vs. shear angle for FCB-1 test (70% fuse).

Figure 3.31. Force-deflection for FCB-1 test (70% fuse).

Figure 3.32. Web splice connection at end of 4% rotation cycles of the FCB-1 test (70% fuse). Evidence of the web splice connection slipping is observed.

Figure 3.33. Photo of FCB-1 (70% fuse) at the end of the 5% rotation cycles. Web/flange welds are fractured in the end of the main beam near the splice connection.

Figure 3.34. Peak-to-peak stiffness of steel beams up to 4% rotation.

Figure 3.35. Plastic energy accumulation of steel beams up to 4% rotation.

Figure 3.36. Total energy dissipated by all specimens up to 4% rotation.

Figure 3.37. Peak-to-peak stiffness of concrete beams up to 4% rotation.

Figure 3.38. Plastic energy dissipation of concrete beams up to 4% rotation.

CHAPTER 4: Design of Practical Diagonally Reinforced Coupling Beams

This chapter discusses the design of diagonally-reinforced coupling beams for practical structures. For the design of diagonally reinforced concrete coupling beams (DBCB) to be compliant with ACI 318-02 often requires that inappropriate assumptions be made during design or results in unconstructible beam details. A discussion of the design requirements for DBCBs is presented along with a number of design examples in an attempt to illustrate the difficulties inherent in designing these elements. Recommendations aimed at simplifying the design of DBCBs are presented.

4.1 Introduction

Conventional longitudinally reinforced concrete coupling beams (Paulay 1971 and 1986; Shiu et al 1978) have been shown capable of dissipating reasonable amounts of energy provided the beam has a flexure-dominant behavior. Conventionally reinforced coupling beams (CCB) have been observed to only exhibit satisfactory performance at nominal gross section shear stresses below $3\sqrt{f_c'}$ (where f_c' is given in psi units; $0.25\sqrt{f_c'}$ in MPa units), provided that the behavior of the beam is dominated by flexure (Aktan and Bertero 1981; Aristizabal-Ochoa 1987). Sliding shear at the face of the wall begins to affect the response of CCBs at stresses between $3\sqrt{f_c'}$ and $6\sqrt{f_c'}$ (psi units; $0.25\sqrt{f_c'}$ and $0.5\sqrt{f_c'}$ in MPa units). The failure of the Mount McKinley Building during the 1964 Anchorage Earthquake (Berg and Stratta 1964) is an often cited example illustrating the shortcomings of conventionally reinforced coupling beams.

Diagonally reinforced concrete coupling beams (DBCB) (Paulay and Binney 1974; Barney et al 1978) have been shown to exhibit greater energy absorption and

deformation capacity than CCBs. Additionally, the diagonal orientation of the primary reinforcing mitigates sliding shear at the face of the wall.

In 1999, ACI 318 *Building Code Requirements for Structural Concrete* first recognized the use of DBCBs in CCW systems. The provisions comprise Section 21.7.7 of the current version of ACI 318-02 (ACI 2002). The notation used throughout this chapter is consistent with that of ACI 318-02. Where applicable, concrete stress, f_c' and $\sqrt{f_c'}$, are given in US standard inch-pound units. The gross section shear stress, v_n , referred to throughout this paper is calculated as:

$$v_n = V_n / b_w h \quad (4.1)$$

Where V_n is the nominal shear capacity of the DBCB and b_w and h are the overall width and depth of the DBCB, respectively.

4.2 Research Objective

The objective of this work is to discuss the design of diagonally reinforced concrete coupling beams (DBCB) in the context of ACI 318-02. It is the contention of this work that regardless of ACI 318 requirements, it is simply not possible to design a practically constructible DCB having a shear stress approaching $10\sqrt{f_c'}$ ($0.83\sqrt{f_c'}$), the ACI 318-prescribed limit. This point will be demonstrated through discussion of provisions for DBCBs and by providing design examples. Current design practice and proposed performance-based design methods both benchmark the design of coupled core walls (CCW) based on maximum allowable shear stress in the coupling beam.

4.3 ACI 318 Provisions for the Design of DBCBs

DBCBs are permitted for coupling beams having a span-to-depth ratio, ℓ_n/h , less than 4 (ACI 318-02 21.7.7.2) and are required for coupling beams having a span-to-depth

ratio less than 2 and a factored shear stress on the gross concrete section exceeding $4\sqrt{f'_c}$ ($0.33\sqrt{f'_c}$) (21.7.7.3). The diagonal bar group elements are effectively treated as compression members subject to seismic load. The diagonal elements must consist of at least four longitudinal bars surrounding a concrete core (21.7.7.4(a) and (c)) and be confined as a seismic column (21.4.4.1 – 21.4.4.3). The diagonal element must have outside dimensions exceeding $b_w/2$ in width and $b_w/5$ in depth (21.7.7.4(a)), where b_w is the overall width of the coupling beam. The diagonal element must be developed for tension into each wall pier (21.7.7.4(d)). Finally, the maximum allowable shear stress on the gross concrete section is $10\sqrt{f'_c}$ ($0.83\sqrt{f'_c}$) (21.7.7.4(b)). The nominal shear strength of a DBCB, V_n , is given as (21.7.7.4(b) Eq. 21-9):

$$V_n = 2A_{vd}f_y\sin\alpha \quad (4.2)$$

Where A_{vd} is the area of steel in one diagonal, f_y is the yield strength of the steel and α is the angle of inclination of the diagonal element.

In addition to the diagonal reinforcement, conventional longitudinal (11.8.5) and transverse (11.8.4) shear reinforcement appropriate for a deep beam is required to confine the entire coupling beam section (21.7.7.4(f)).

The implication inherent in ACI 318-02 Section 21.7.7, that the diagonal elements are compression elements, affects the details of the diagonal element. Significantly, based on this implication, the authors contest that the minimum clear distance between bars in the diagonal element is $1.5d_b$ or 1.5 inches (38.1mm) as required by 7.6.3 rather than the less restrictive limit for spacing between parallel bar layers given by 7.6.1 (d_b or 1.0inch (25.4mm)). This interpretation increases the overall depth of the diagonal element and, as shown below, decreases the efficiency of the element.

4.4 Depth of Diagonal Element, h_x

The required value of h_x will often be greater than the minimum value of h_x , which is $b_w/5$. The diagonal element requires at least four corner reinforcing bars and will most typically require #4 transverse reinforcing to satisfy the confinement requirements of 21.4.4.1. Often, to provide sufficient reinforcing steel, bundled bars will be required, further increasing the depth of the diagonal. When bundled bars are used, it is also important to recognize that the required clear distance between bundles increases (7.6.6.5).

4.5 Angle of Inclination, α

The determination of the angle of inclination for a given beam geometry is of critical importance to the design of DBCBs. The angle of inclination of the centroid of the diagonal element is determined from beam geometry and is affected by concrete cover, c , and, significantly by the overall depth of the diagonal element, h_x . The angle of inclination, α , is determined as:

$$\alpha = \tan^{-1} \left[\left(h - \frac{h_x}{\cos \alpha} - 2c \right) / \ell_n \right] \quad (4.3)$$

Where ℓ_n is the clear span of the DBCB.

Clearly, calculating a value for α , is an iterative process. For most architecturally practical coupling beams, defined as those having a span-to-depth ratio exceeding 2, α will be less than about 20°. A reasonable estimate of the angle of inclination, in this case, may be found to be:

$$\alpha = \tan^{-1} [(h - 1.06h_x - 2c) / \ell_n] \quad (4.4)$$

A change in the angle of inclination is inversely proportional to the change in the resulting required reinforcement. This effect is particularly pronounced at lower angles of

inclination. For example, changing the angle of inclination from 15° to 14° results in a 7% increase in the required diagonal reinforcing steel.

4.6 Width of Diagonal Element, b_x

The minimum width of the diagonal element, b_x , is $b_w/2$. Meeting this minimum value necessitates that the diagonal elements interlock as they intersect at the coupling beam midspan. The total available width in which to locate the diagonal element is considerably smaller than the actual beam width, b_w . The diagonal elements must fit not only within the concrete cover and conventional beam reinforcement required by 21.7.7.4(f), but also within the outer-most curtains of the wall pier boundary element reinforcing. The available width, $b_{\text{available}}$, for locating the diagonal elements is therefore:

$$b_{\text{available}} = b_w - 2[(c + d_b \text{ (of } A_v) + d_b \text{ (of } A_{vh}) + d_b \text{ (of wall reinforcing)})] \quad (5)$$

Where b_w is the width of the DBCB, and d_b is the bar diameter of the reinforcing steel as indicated: A_v and A_{vh} are the vertical and horizontal reinforcement confining the entire DBCB required by 21.7.7.4(f).

The need to interlock the diagonals results in offsetting each diagonal and may result in increased clear distance requirements across the width of the elements in order to facilitate one diagonal passing through the other. Finally, if the wall boundary reinforcement has more than two exterior curtains of reinforcing steel, additional space must be provided to permit the diagonal reinforcing to clear the wall reinforcement as it passes into the wall pier.

Occasionally, concern has been raised that the ACI requirement that the width of the diagonal element, b_x , be larger than $b_w/2$ results in a DBCB that is eccentrically

reinforced. Although this is true, it is not felt to be a great concern since typically a coupling beam will be supported laterally by a floor slab.

Based on the foregoing sections, the design of DBCBs typically progresses in an iterative manner. Clear sketches are often required to determine the permissible geometry.

4.7 DCB Shear Stress

As mentioned above, ACI 318-02 21.7.7.4(b) limits the gross concrete section shear stress to $10\sqrt{f'_c}$ ($0.83\sqrt{f'_c}$). This value is justified in ACI 318 as being based on results of tests reported by Barney et al (1980). Table 4.1 presents the nominal shear capacity, calculated using Equation 4.2, and the observed shear capacity of DBCB tests reported in the available literature. None of the existing tests of DBCBs had reinforcing details that would satisfy the requirements of ACI 318-02 and few of the test specimens were designed to carry $10\sqrt{f'_c}$ ($0.83\sqrt{f'_c}$). Of particular note, the beams reported by Barney et al on which the ACI shear stress limit is reportedly based, contain different reinforcing in each diagonal. The small-scale specimens have 2 #3 bars comprising one diagonal and a single #4 comprising the other (Barney et al 1978).

Regardless of the foregoing discussion, the concrete shear capacity is not considered in the design equation for DBCBs (Equation 4.2). In any event, the concrete shear capacity is only used as a surrogate limit.

4.8 Design of CCWs

Elastic analysis of CCWs used to determine design loads (ICC 2003) often results in coupling beam shear capacities exceeding those permitted by ACI 318. Inherently large redundancy factors (ICC 2003) associated with CCWs, and directional effects also

serve to increase the shear demand placed on coupling beams (Harries et al 2004). To reduce coupling beam stresses so that they fall within the $10\sqrt{f_c'}$ ($0.83\sqrt{f_c'}$) limit, designers often use unrealistically high values of $\sqrt{f_c'}$ and/or reduce the effective stiffness of the coupling beams to unrealistically small values. Neither approach is appropriate and both mask the expected behavior of the CCW system. Nonetheless, using these approaches results in the design coupling beam shear for most CCWs, being very close to $10\sqrt{f_c'}$ ($0.83\sqrt{f_c'}$), although the actual shear demand may be expected to exceed this value. The argument can be made that the $10\sqrt{f_c'}$ ($0.83\sqrt{f_c'}$) limit is not relevant to DCBs in any event since the design is based solely on the reinforcing steel (Equation 4.2).

4.9 Examples of DBCB Design

The contention of the work presented in this chapter is that regardless of ACI 318 requirements, it is simply not possible to design a practically constructible DBCB having a shear stress as great as $10\sqrt{f_c'}$ ($0.83\sqrt{f_c'}$). Based on the foregoing discussion of the ACI provisions for DBCBs, a design exercise was undertaken to demonstrate this point. The intent was to design DBCBs having a range of parameters and evaluate their constructability. All designs are fully compliant with the requirements of ACI 318-02 discussed above. The assumptions and parameters used to design the example DBCBs are given in Table 4.2. Figure 4.1 defines the parameters and shows additional details of the horizontal (A_{vh}) and vertical (A_v) reinforcing in the beam section and the confinement provided the primary diagonal reinforcing (A_{sh}). This reinforcement was the same for all beam sections designed. The parameters selected are representative of DBCBs in practice. Typically, architecturally practical span-to-depth ratios will fall between 3 and 4

(and greater) although shorter spans are also investigated as they are easier to design using the ACI provisions. The values of design concrete shear stress were selected to represent what the authors believe is practical ($6\sqrt{f'_c}$ ($0.5\sqrt{f'_c}$)), the ACI 318 limit ($10\sqrt{f'_c}$ ($0.83\sqrt{f'_c}$)) and a value that may be obtained from an elastic analysis ($14\sqrt{f'_c}$ ($1.17\sqrt{f'_c}$), reported by Harries et al 2004). For all designs, the angle of inclination, α , was maximized; this and the discrete nature of reinforcing bar selection results in designs having nominal shear capacities close to but not exactly the same as the target values selected.

Figures 4.2 and 4.3 show the resulting DBCB designs for $f'_c = 5000$ psi and $f'_c = 8000$ psi (34.5 and 55.2 MPa), respectively. In each figure, the beams are arranged such that the rows represent design concrete shear stress targets ($v_n = 6, 10$ and $14\sqrt{f'_c}$ ($0.5, 0.83$, and $1.2\sqrt{f'_c}$)) and the columns represent span-to-depth ratio ($\ell_n/h = 1, 2, 3$ and 4).

The first observation made in this exercise is that it rapidly becomes necessary to provide bundled bars. ACI 318 limits bundled bars to #11 and smaller (7.6.6) and requires that bundled bar development length be increased (12.4.1). Using bundled bars results in a deeper diagonal element (increased h_x) due to the dimension of the bundle and the increase in required spacing between bundles (d_b is calculated based on a single bar having an area equivalent to the total area of the bundle (7.6.6.5)). As a result, the DBCB has a shallower angle of inclination (Equation 4.3) requiring additional reinforcing steel. Additionally, bundled bars will result in greater congestion in the wall pier and may preclude developing the diagonal bars by bending them or providing hooks in the wall pier as is often done in practice.

In Figures 4.2 and 4.3, the darker shaded entries are those designs that the authors feel are not reasonably constructible. In these cases, it will be exceptionally difficult to fit all of the reinforcing steel into the section. Additionally, in these cases, it has been necessary to remove the center curtain of wall pier reinforcement. The lighter shaded entries are considered very difficult to construct although not entirely impractical. Typically, these cases involve bundled bars which the authors feel may be more difficult to place accurately. It can also be seen that some combinations of shear stress and beam length simply cannot be designed to satisfy ACI 318 provisions. Typically the reason that these designs cannot be completed is that as more steel is required, h_x increases, resulting in a lower angle of inclination, α , which in turn requires the addition of more steel.

Since the design capacity of DCBs is based entirely on the reinforcing steel (Equation 4.2), increasing the concrete strength, as may be done in the design process to satisfy shear stress limits, requires more steel. This option results in greater congestion and therefore less constructible beams.

Figure 4.4 shows an example of a DBCB design which would be difficult to construct. The example has an architecturally practical l_n/h ratio of 3. The diagonal reinforcement consists of an arrangement of 4 bundles of #11 reinforcing bars. (This example is represented by the 5000 psi (34.5 MPa) beam having a shear stress of $10\sqrt{f'_c}$ ($0.83\sqrt{f'_c}$) or the 8000 psi (55.2 MPa) beam having a shear stress of $6\sqrt{f'_c}$ ($0.5\sqrt{f'_c}$), as shown in Figures 4.2 and 4.3, respectively.) The congestion of this detail is evident from Figure 4.4. It can be seen that specially detailed confining steel will be required to confine the diagonal elements near midspan and that congestion in this region may affect the ability to place and consolidate concrete. To address congestion in this region, the use

of “headed bars” arranged horizontally through the entire DBCB section, rather than ties surrounding the diagonal elements has been attempted (HRC 2004). Laboratory tests of this arrangement (Mitchell 2004) have shown essentially identical behavior using this simpler confinement to that exhibited using ACI 318 compliant confinement details.

The required straight bar development length, ℓ_d , of the diagonal elements into the walls (See Figure 4.4) is calculated as (ACI 318-02 21.5.4.2):

$$\ell_d \geq \kappa \left(\frac{3.5f_y}{65\sqrt{f'_c}} \right) d_b \text{ (psi units)} \quad (4.6)$$

$$\ell_d \geq \kappa \left(\frac{3.5f_y}{5.4\sqrt{f'_c}} \right) d_b \text{ (MPa units)} \quad (4.6)$$

where $\kappa = 1.2$ for bundles of 3 bars and 1.33 for bundles of 4 bars (12.4.1).

In the example shown in Figure 4.4, the required straight development length would be $\ell_d=84\text{inches}$ (2130mm) for $f'_c=5000\text{psi}$ (34.5MPa) and $\ell_d=66\text{inches}$ (1680mm) for $f'_c=8000\text{psi}$ (55.2MPa).

Providing deeper DBCBs results in a more efficient beam design (by increasing α); however, it also results in greater story heights which are generally undesirable. Similarly, wider coupling beams are not practical since the diagonal elements must be embedded into the wall piers requiring the wall piers to be at least as wide as the beams.

4.10 Recommendations

It has been shown that it is difficult to design a practically constructible DBCB to have a design shear stress approaching the ACI 318 limit of $10\sqrt{f'_c}$ ($0.83\sqrt{f'_c}$). It has been previously suggested (Harries et al 2004) that a value of $6\sqrt{f'_c}$ ($0.5\sqrt{f'_c}$) is the practical upper limit when considering the constructability of DBCBs. The foregoing discussion

has identified the span-to-depth ratio and the concrete strength as also affecting the constructability. Based on the observations and assessment of the designs shown in Figures 4.2 and 4.3, it is proposed that a constructible DBCB may be designed provided:

$$V_n/b_w h \leq (11.5 - \ell_n/h) \beta_1 \sqrt{f_c'} \text{ (psi units)} \quad (4.7)$$

$$V_n/b_w h \leq 0.083(11.5 - \ell_n/h) \beta_1 \sqrt{f_c'} \text{ (MPa units)} \quad (4.7)$$

where β_1 is defined by ACI 318-02 10.2.7.3 as being equal to 0.85 for concrete strengths up to and including 4000psi (27.6MPa) and is reduced at a rate 0.05 for each 1000psi (6.9MPa) in excess 4000psi. β_1 need not be taken less than 0.65. An approximate value of $V_n/b_w h \leq 6\sqrt{f_c'}$ (psi units; $V_n/b_w h \leq 0.5\sqrt{f_c'}$ in MPa units), however, remains generally appropriate.

The coefficient in Equation 4.7 has been calibrated to recognize that shear stresses approaching $10\sqrt{f_c'}$ ($0.83\sqrt{f_c'}$) are attainable when $\ell_n/h=1$ but that maximum attainable shear stresses fall to approximately $6\sqrt{f_c'}$ ($0.5\sqrt{f_c'}$) when ℓ_n/h approaches 4. The use of the β_1 factor to account for increased concrete strength is selected for its convenience and familiarity to designers.

The ACI requirements of 21.7.7.4 constraining the geometry and confinement requirements of the individual diagonal elements are felt to be unnecessarily restrictive. It is proposed that potential buckling of the diagonal bars can be prevented by confining the entire DBCB section with transverse reinforcement satisfying the requirements of 10.8.4 and 10.8.5 and the spacing requirements of 21.3.3.2. By using such gross section confinement and relaxing the diagonal element geometry requirements, different diagonal arrangements (such as intersecting horizontal planes of reinforcement across the width of

the member) may be used which will allow for improved constructability at higher shear stress levels. This is the approach taken by both the Canadian (CSA 1994) and New Zealand (NZS1995) concrete codes. In taking this approach, the Canadian code increases the development length of the diagonal bars by 150% since the diagonal elements may no longer behave as a single unit.

4.11 Implications for Performance Based Design

As discussed above, force-based design procedures (ICC 2003) using appropriate reduced stiffness factors typically result in very high coupling beam shears in CCW structures. DBCB design shear values have been shown to be on the order of $12\sqrt{f_c'}$ ($1.0\sqrt{f_c'}$) for a 30-story CCW (McNeice and Harries 2004) and as high as $16\sqrt{f_c'}$ ($1.33\sqrt{f_c'}$) for a 10-story CCW (Harries et al 2004). These values exceed ACI limits and could not be reasonably designed for in any event. To address this issue (and others associated with the design of CCWs), Harries et al (2004) have proposed a method for the performance based design (PBD) of CCWs. The premise of the proposed PBD approach is that an efficient CCW design may be initiated through the judicious selection of a coupling beam capacity. In the PBD approach, the coupling beam capacity (and behavior) is selected (rather than determined through analysis) and the design of the wall piers proceeds based on this prescribed beam behavior. This method has been shown (Harries et al 2004 and McNeice and Harries 2004) to result in CCWs that satisfy rational performance criteria

In an efficient CCW design, it is desired that the coupling beam capacity be as great as possible while still being constructible. The constructability of the DBCB is, in this case, one performance criteria that must be satisfied. It is proposed, therefore, that

Equation 4.7, or alternately, the use of $v_n \leq 6\sqrt{f'_c}$ ($0.5\sqrt{f'_c}$), represent a practical starting point for the PBD of CCWs having DBCBs.

4.12 Conclusions

Diagonally reinforced concrete coupling beams (DBCB) are an efficient method of coupling wall piers and forming a coupled core wall (CCW) lateral force resisting system. Design provisions of these beams were introduced in Chapter 21 of ACI 318-99 and comprise Section 21.7.7 of ACI 318-02. It is the contention of this work that regardless of ACI 318 requirements, it is simply not possible to design a practically constructible DBCB having a shear stress approaching $10\sqrt{f'_c}$ ($0.83\sqrt{f'_c}$), the ACI 318-prescribed limit. This is demonstrated through a discussion of the ACI provisions for DBCBs and through DBCB design examples. Equation 4.7, or alternately, the use of $v_n \leq 6\sqrt{f'_c}$ ($0.5\sqrt{f'_c}$) is recommended as a practical upper limit of gross section shear stress for which a practical DBCB may be designed.

The confinement requirements for the individual diagonal elements may be excessive. Confinement of the entire DBCB cross section would allow more versatility in designing the DBCB and is believed to still provide adequate support for the diagonal elements when they serve as a compression strut. The DBCB test discussed in Chapter 3 indicates that ACI-compliant transverse reinforcement is not required at beam chord rotations at or below 3%. Further investigation of this issue is required.

4.13 Epilogue

The author and others have developed a number of alternatives that alleviate the aforementioned difficulties associated with design and construction of diagonally reinforced coupling beams. These options include steel (Harries et al 1997, Shahrooz et

al 1993), steel-concrete hybrid (Shahrooz et al 2001). Additionally, the use of headed bars to provide confinement has been proposed (Mitchell 2004). These alternatives have been successfully implemented in a number of constructions (Harries and Shahrooz 2005 and HRC 2004). Although not yet adopted in practice, current research is investigating a variety of innovative coupling beam forms including steel beams having a fused link (as discussed in Chapter 3), and unbonded post-tensioned coupling beams (Kurama and Shen, 2004).

Table 4.1. Capacity of DBCBs reported in literature.

Researcher	Specimen Designation	ℓ_n/h	$V_n / b_w h \sqrt{f'_c}$ (psi units) ¹	
			calculated (Equation 2)	observed
Galano and Vignoli 2000	P07	1.50	5.4	6.3
	P12	1.50	6.2	7.5
Paulay and Binney 1974	316	1.30	9.9	11.7
	317	1.30	7.4	8.6
	395	1.03	7.1	8.6
Hindi et al 2003	1	2.74	8.4	13.6
Barney et al 1978	C6	2.50	11.6	10.9
	C8	5.00	6.1	5.3
Tassios et al 1996	CB2A	1.00	7.5	9.9
	CB2B	1.00	5.3	6.2

¹ divide reported value by 12 for MPa units

Table 4.2. Assumptions and parameters used to design DBCBs (see Figure 4.1).

Parameter	Value
concrete strength, f'_c	5000 and 8000 psi (34.5 and 55.2 MPa)
DCB and wall width, b_w	24 inches (610 mm)
DCB depth, h	36 inches (915 mm)
span-to-depth ratio, ℓ_n/h	1, 2, 3 and 4
concrete cover, c	1.5 inches (38.1 mm) for beams 0.75 inches (19 mm) for walls
design concrete shear stress, v_n (Equation 1)	$6\sqrt{f'_c}$, $10\sqrt{f'_c}$ and $14\sqrt{f'_c}$ (psi units) $0.5\sqrt{f'_c}$, $0.83\sqrt{f'_c}$ and $1.17\sqrt{f'_c}$ (MPa units)
wall reinforcing steel	three curtains of #8 bars

(a) DCB elevation

(b) DCB dimensions

(c) DCB reinforcing details

Figure 4.1. Diagonal coupling beam details.

Figure 4.2. Diagonal coupling beam details satisfying ACI 318-02 having $f'_c = 5000 \text{ psi}$ (34.5 MPa).

Figure 4.3. Diagonal coupling beam details satisfying ACI 318-02 having $f'_c = 8000\text{psi}$ (55.2MPa).

Figure 4.4. Example of DBCB having bundled #11 bars and $\ell_n/h = 3$.

Chapter 5: Wall Boundary Elements Study

In this chapter, a study investigating the need for special boundary elements in wall piers used in coupled core wall systems is presented. The study entails the design of eight different configurations of buildings with coupled core walls as the lateral force resisting system. In all eight cases, steel coupling beams are used. The equivalent lateral force procedure, per NEHRP 2000, is used for elastic structural analysis to determine the design forces and moments required for the structural components.

The steel coupling beams are designed using the procedure detailed in Chapter 2. As will be seen, the design of the coupling beams is an iterative process, and the final proportioning of the beams is dependent on two factors: (1) the factored capacity of all beams at each level must be greater than the design shears and moments at their respective level and, (2) the beams must be proportioned in a manner that minimizes the wall overstrength factor (this is discussed in detail in this chapter).

The design of the wall piers is dependent on the need for special boundary elements. This is the heart of the study. Three different methods for determining the need for special boundary elements are investigated. Two of the methods are prescribed in ACI 318-02. The first method determines the need for special boundary elements based on the extreme fiber concrete compressive stresses at design loads; the stresses are calculated using gross section properties (ignoring steel) and assumes a linear distribution of stresses over the depth of the section. If the compressive stress exceeds a critical limit, special boundary elements are required. The second method determines the need for special boundary elements based on the location of the neutral axis at design loads. If the depth

of the compression block exceeds a prescribed limit, special boundary elements are required. As will be shown in this chapter, neither of these methods is appropriate for wall piers in coupled core wall systems. Thus, an alternative method is investigated.

The third method is an alternative method. This proposed method determines the need for special boundary elements based on determining the extreme fiber concrete compressive strains at design loads. If the concrete compressive strains exceed a critical limit, special boundary elements will be required. The results of this study suggest that for some configurations of coupled core wall buildings the concrete compressive strains are moderate, and hence, special boundary elements are not required.

5.1 Introduction

The total normal stresses acting on the cross-section of a wall pier in a coupled core wall (CCW) system are a combination of normal stresses resulting from (1) axial forces due to gravity loads, (2) axial forces resulting from frame action due to the coupling effect, and (3) overturning moments due to bending of each wall pier. The stresses resulting from frame action and overturning moment are present only when lateral loads are applied.

In order to maintain sufficient axial load-carrying and flexural capacity of the wall piers, longitudinal reinforcing steel must be transversely restrained to prevent buckling. The surrounding concrete can provide the required restraint to buckling provided that spalling of shell concrete, concrete strength degradation, and wall damage due to shearing effects are delayed. Where sufficient longitudinal reinforcement protection is not expected to be provided by shell concrete, confining steel is provided to confine the core

of concrete in which the reinforcing steel is located. This core of confined concrete is referred to as a “special boundary element.”

ACI 318-02 *Building Code Requirements for Structural Concrete* offers two methods for determining the need for special boundary elements. The first method (ACI 318-02 §21.7.6.3) considers concrete stress and permits expected maximum concrete stresses to be determined based on a linear stress distribution over the depth of a section with properties based on the gross wall pier sections. When the maximum concrete stress computed at factored loads, including earthquake effects, exceeds 20% of the specified concrete strength, special boundary elements are required. The second method (ACI 318-02 §21.7.6.2) considers the location of the neutral axis of a section when subjected to design loads. When the length of the compression zone exceeds a critical value, special boundary elements are required. ACI 318-02 §21.7.6.2 requires special boundary elements under the following condition:

$$c \geq \frac{l_w}{600 \left(\frac{\delta_u}{h_w} \right)} \quad (5.1)$$

Equation 5.1 follows from a displacement-based design approach and assumes that special boundary elements are required when the strain at the extreme compression fiber of a wall section exceeds a critical value. Structural concrete components are capable of resisting extreme fiber compression strains of 0.004 without significant strength degradation, and it has been shown that there are a large class of reinforced concrete buildings utilizing cantilevered shear walls for which special boundary elements are not required (Wallace 1994).

5.2 Objective of Parametric Study

The objective of this parametric study presented in this chapter is to investigate the need for special boundary elements in the wall piers of CCW systems. The degree of reinforcing steel congestion at the wall pier/coupling beam interface can have a significant impact on the constructability and construction costs of buildings utilizing the CCW system, and in some cases, where heavily-reinforced boundaries are needed, can be a determining factor in the decision not to use the CCW system. In this interest, a parametric study is performed to examine the ACI 318-02 procedures for determining the need for special boundary elements in wall piers of coupled core wall systems. Where reasonable, special boundary elements will not be provided in the wall piers in an effort to reduce reinforcing steel congestion.

5.3 Parameters of Study

Eight different configurations of buildings utilizing the CCW system were selected based on building footprints found to be practical. It was decided that unencased built-up steel I-shapes were to be used as the coupling beams on the contention that this type of coupling beam would generate the greatest axial forces in the wall piers due to the superior stiffness of the steel coupling beams over reinforced concrete coupling beams (Harries et al 1997).

All 20-story structures are designed using NEHRP 2000 (FEMA 368), The structures have 2.74m floor-to-floor heights, 178mm thick post-tensioned floor slabs, 610mm thick wall piers with 35MPa concrete strength, and 248MPa steel for the beams. The structures are assumed to be located in San Francisco on a Class C (soft rock) site where $S_s=1.5g$ and $S_1=0.65g$. The site falls into Seismic Use Group 1 and Seismic Design

Category D. Floor dead and live gravity loads are assumed to be $1kPa$ and $2.4kPa$, respectively (see Table 5.1 for a summary of the gravity loads assumed). Cladding dead load is assumed to be $4.4kN/m$ and is applied to the exterior edges of the floor plate.

Two different configurations of CCW systems were investigated: (1) one-cell cores consisting of two C-shape walls, and (2) two-cell cores consisting of two C-shape walls with a central I-Shape wall. Four buildings of each configuration are investigated. Refer to Figure 5.1 and Table 5.2 for the plan dimensions of the eight different structures.

5.4 Structural Analysis and Calculated Design Loads

5.4.1 General Analysis and Design Procedures

The general procedure for analysis and design of the structures, assuming that the equivalent lateral force procedure will be used, is as follows:

Preliminary Load Calculations

- Determine the gravity loads appropriate for the structure.
- Calculate the self-weight of the structure.
- Knowing the site that the structure is built on, determine the Mapped Maximum Considered Earthquake (MCE) spectral response accelerations, determine the site coefficients based on the type of soil at the site, and then determine the Design Spectral Response Acceleration used to generate the design spectral response for the Design Basis Earthquake (DBE).
- Determine the Site Class Factor, Seismic Use Group, and Seismic Design Category.
- Determine the appropriate Response Modification Factor (R), Overstrength Factor (Ω), Importance Factor (I), and Deflection Amplification Factor (C_d).
- Calculate the dynamic weight of the structure (W), and the Seismic Response Coefficient (C_s) to determine the design base shear (V).
- Determine the approximate fundamental period (T_a).
- Calculate how the base shear will be distributed vertically over the height of the building. This is the lateral force (F_x) induced at each level.
- Calculate the horizontal shear distribution (V_x). This the design story shear at each level.
- Generate all appropriate load cases to be applied to the structure.

Modeling and Generating Design Forces

- Select an appropriate analysis software (ETABS Nonlinear v.8.4.5 was used for all the reported elastic analyses).
- Decide modeling parameters and protocol. The boundary conditions, component properties (e.g., the values of EI for base wall piers and upper story wall piers, beam stiffness modifications, etc.), and modeling techniques must be determined.
- Determine the beam proportioning over the height of the building. The proportioning of the beams over the height of the building has a direct effect on the wall overstrength factor and interstory drifts.
- Generate the model and then run the analysis. This is an iterative process. Preliminary beam designs must be assumed and then the structure analyzed. After analysis, the capacity of the beams must be checked in addition to calculating the wall overstrength factor. Iterations on beam design must be made in order to assure adequate beam capacity while at the same time minimizing the wall overstrength factor. Each time the design of the beams is changed, beam stiffnesses are changed which changes the distribution of forces in the structure. Thus, for each change in beam design, a subsequent analysis is required.
- Once the final design is complete, design forces for the wall piers are determined using the governing load combinations.

Coupling Beam Design

- The design procedure for steel coupling beams is detailed in Chapter 2.
- The proportioning of coupling beams over the height of the building is an issue of great debate. When designing based on an elastic analysis, the wall pier design forces must be increased by a wall overstrength factor to ensure that the wall piers remain elastic at base shear levels large enough to yield the beams. The nominal strength of the coupling beams at each level will always be larger than the expected demand due to the capacity reduction factors required to be applied. Thus, the wall pier design forces must accommodate the axial loads generated at nominal beam shear capacities. To minimize the wall overstrength factor, different size beams are used over the height of the structure. For the 20-story structures considered in this study, five different beam sizes were used over the height of the structure. Floors 1-4, 5-8, 9-12, 13-16, and 17-20 had different size beams. This was done to minimize the difference between nominal beam shear capacity and beam shear demand at each floor level. However, this type of beam proportioning over the height of the structure may not be appropriate if designing based on nonlinear analyses (a detailed discussion of this point is presented in Chapter 6).

Wall Pier Design

- The design of the wall piers is the topic of this chapter and discussion is presented in the following sections.

The previous bullets are meant only as an outline to describe the general procedures used in the elastic analysis and design of a coupled core wall building. Example calculations and procedures used in the design of a structure are provided in Appendix B. In the appendix, example calculations of dynamic weight, base shear, vertical distribution of lateral forces, horizontal story shear distribution, and the generation of design spectral response is provided. Specific NEHRP provisions for each step is also provided. It should be noted that the same procedures were used for elastic analyses of all structures considered in the reported study. Where structures are said to be designed based on elastic analysis, the procedures outlined above are detailed in Appendix B are relevant.

5.4.2 Analysis of Structures

For these structures, NEHRP 2000 permits the use of the equivalent lateral force (ELF) procedure for determining the design lateral loads. To facilitate analyses, all structures were modeled and analyzed using ETABS Nonlinear (v. 8.4.5). Appropriate load combinations were considered. The earthquake lateral loads included bi-directional effects and accidental torsion. Appropriate dead and live gravity loads were also incorporated in the design and analysis. Two earthquake load cases were considered. Load case one (LC1) considers an earthquake acting West-to-East where 100% of the earthquake loads are applied in the West-East direction with 30% of the earthquake loads applied in the South-North direction. Load case two (LC2) considers an earthquake acting South-North where 100% of the earthquake loads are applied in the South-North direction with 30% of the earthquake loads acting in the West-East direction. In both cases, accidental torsion is considered and is applied so as to increase the impact of bi-directional effects. See Figure 5.2 for assumed loading directions.

The modeling of the structures consist of the core wall piers, coupling beams, the floor slab, and dummy columns at each corner (Figure 5.3 shows screen snapshots of a typical floor plan and 3-D elevation). The individual walls were modeled as shell elements and the three legs of each C-shape (or four legs of each I-shape) were assigned as ETABS “wall piers.” Doing so allows the output to provide forces acting on each wall pier rather than on the individual shell elements that define the wall piers. To account for cracked sections, the elastic modulus, $E_{c,wall}$, for the base wall pier was reduced by 65% (i.e., $0.35E_{c,wall}$ was input) and the elastic modulus for all wall piers above the base wall pier were reduced by 30% (i.e., $0.7E_{c,wall}$ was input).

The coupling beams were modeled as line elements. Previous research has shown that the fixity of the end of the beam occurs inside the wall pier approximately 1/3 of the calculated beam’s embedment length from the end of the wall pier (Gong and Shahrooz 2001(a)). Hence, the additional flexibility of the beam should be considered and rigid end offsets at the beam ends be incorporated in the model. However, as this is not currently a standard modeling practice in design offices, the ends of the beams were considered to be fixed at the wall pier-beam interface.

The floor plates are modeled as shell elements and assigned as rigid diaphragms. The corner dummy columns were assigned to have infinitesimal cross-sectional area, and were assigned pinned-pinned boundary conditions to ensure no contribution to lateral stiffness.

In addition to the self-weight of the structure, additional area masses were assigned to the floor plate to account for the superimposed gravity dead loads. Additional line masses were assigned to the perimeter edges of the floor plate to account for

cladding weight. The additional masses were assigned to assure that the superimposed gravity dead loads contributed to the dynamic weight of the structure.

The load cases considered in the ETABS analyses involved only lateral load cases. Gravity dead and live loads were calculated by hand and their effects were superimposed on the lateral load effects. This was done to assure that only gravity loads within the tributary areas of the wall piers were considered. Table 5.3 summarizes the calculated dynamic weights, seismic response coefficients, and base shears for each of the eight structures. Table 5.4 summarizes the vertical distributions of lateral forces and horizontal shear distributions calculated using the Equivalent Lateral Force Procedure. Table 5.5 summarizes the calculated redundancy and wall overstrength factors used to compute the design forces for the coupling beams and wall piers, respectively.

5.4.3 Design of the Structures

Design shears and moments for the coupling beams were determined based on analyses of the applied lateral forces with the inclusion of redundancy as required by NEHRP 2000. Wall pier design forces are based on critical load combinations, including lateral and gravity loads, and taking into account wall overstrength factors to ensure elastic wall response beyond yielding of the coupling beams (Harries 2004; Harries and McNiece 2005). Axial loads for the wall piers were computed based on the following load combinations:

$$\text{Tension Wall:} \quad P = 0.9D - 0.2S_{DS}D - \sum V_n \quad (5.2)$$

$$\text{Compression Wall:} \quad P = 1.2D + 0.5L + 0.2S_{DS}D + \sum V_n \quad (5.3)$$

$$\text{Intermediate Wall:} \quad P = 1.2D + 0.5L + 0.2S_{DS}D + \sum V_n \quad (5.4)$$

The sum of the beam shear demands will be zero when there is symmetry in the core walls. With symmetry, the shear forces acting on the wall on the West and East sides of the intermediate wall will be equal and opposite in sign. As all of the structures in this study have symmetry, the term, ΣV_n , in Equation 5.4 is taken as zero.

A summary of design forces for coupling beams and wall piers is provided in Table 5.6 and Table 5.7, respectively. Wall pier design forces, over the height of Structure_01, are provided in Table 5.7. Refer to Appendix C for the design forces of Structure_02-Structure_08. The wall forces are distributed over the height similarly in all structures.

5.4.3.1 Coupling Beam Design

The design procedures used for the steel coupling beams are given in Chapter 2. Appendix B also has detailed example calculations used for design. For all structures, the maximum design beam shears for levels 1-4, 5-8, 9-12, 13-16, and 17-20 were determined and then the beams were appropriately designed for their respective floor. A summary of the resulting beam dimensions is provided in Table 5.8.

5.4.3.2 Design of Wall Pier Longitudinal Steel

The flexural steel for the wall piers was proportioned based on ACI 318-02 provisions. Design axial load-moment interaction curves were generated for all wall piers to ensure sufficient flexural and axial load-carrying capacity; biaxial loading was considered. To facilitate design, Xtract (v. 2.6.2), a fiber cross-sectional analysis software package was utilized. As it is the objective of this work to establish the potential of providing sufficient capacity of the wall piers while not implementing special boundary elements, an unconfined concrete model was assumed and no confined areas of concrete

were assigned. Two curtains of uniformly distributed longitudinal steel were used for design so as not to create any localized areas of concentrated normal forces in the wall section. A summary of the resulting bars sizes and spacing provided in the wall piers is given in Table 5.9.

5.5 Determining Whether Special Boundary Elements are Required.

As discussed previously, the intent of this study is to investigate the applicability of current ACI 318-02 provisions for assessing the need for special boundary elements in wall piers of CCW systems and the possibility of reducing wall pier steel congestion. Accordingly, compression fiber strain was computed at the following locations, which are also shown graphically in Figure 5.3: (1) the end of the leg of the C-shape tension wall where the coupling beam engages the wall pier (Point A), (2) The end of the leg of the I-Shape wall where the coupling beam connects to the wall pier (Point B), and (3) the flange/web intersection of the C-Shape compression wall (Point C). Additionally, points where extreme fiber concrete tension were determined are also shown in the figure (points D, E, and F). In the concrete was assigned to have zero tensile strength in the fiber model. Points D, E, and F are evaluated for cases where the entire depth of the section is in compression. The need for special boundary elements was evaluated through the following procedures.

5.5.1 Stress Calculations Based on ACI 318-02 §21.7.6.3

Provision ACI 318-02 §21.7.6.3 states that special boundary elements are required if extreme fiber concrete stresses, calculated using a linear stress distribution over the depth of a wall and gross section properties, exceeds 20% of the specified

concrete compressive strength. The extreme fiber compressive stress, at design loads, is calculated for all wall piers for both load cases (LC1 and LC2).

For the LC1 and LC2 earthquakes, the extreme fiber compressive stresses are calculated as follows (refer to Figure 5.2 for nomenclature used in the following equations):

$$\text{Tension Wall: } \sigma_{cc} = -\frac{P}{A} + \frac{M_{xx} \times \bar{y}_T}{I_{xx}} + \frac{M_{yy} \times \bar{x}_R}{I_{yy}} \quad (5.5)$$

$$\text{Compression Wall: } \sigma_{cc} = \frac{P}{A} - \frac{M_{xx} \times \bar{y}_T}{I_{xx}} + \frac{M_{yy} \times \bar{x}_R}{I_{yy}} \quad (5.6)$$

$$\text{Intermediate Wall: } \sigma_{cc} = \frac{P}{A} + \frac{M_{xx} \times \bar{y}_T}{I_{xx}} + \frac{M_{yy} \times \bar{x}_R}{I_{yy}} \quad (5.7)$$

Table 5.10 summarizes the resulting calculated stresses using this provision. For illustration, stresses over the height of the structure are shown for Structure_01. Refer to Appendix C for the stresses calculated for Structure_02-Structure_08. The distribution over the building height is similar in all structures. The specified concrete compressive strength is $35MPa$; hence, any compressive stresses exceeding 20% of this value, $7MPa$, will result in mandatory special boundary elements. Additionally, ACI 318-02 §21.7.6.3 requires that where boundary elements are required, they may only be discontinued when the compressive stress falls below 15% of the specified compressive strength, or $5.3MPa$ in this case. As can be seen in Table 5.10, for Structure_01 the extreme fiber compressive stress is greater than $7MPa$ when considering both load cases. For example, structure_01 would require special boundary elements up to and including level 12, where LC2 is the governing load case. The results are similar in Structure_02-Structure_08.

5.5.2 Compression Block Depth Calculations Based on ACI 318-02 §21.7.6.2

ACI 318-02 §21.7.6.2 requires special boundary elements where the depth of the compression zone exceeds the critical value given in Equation 5.1. In order to investigate the application of this provision, the depth of the compression zone is required to be determined. Using the cross-section analyses generated with Xtract, the depth of the compression zone was determined for all wall piers for all structures considering both load cases 1 & 2. To establish the limit prescribed by Equation 5.1, the equation is rearranged to establish the limit based on the ratio of the compression block depth to the length of the wall (c/l_w) in the direction of loading. The ratio of δ_u/h_w can reasonably be taken as 0.007 for practical CCW configurations. For example, the maximum elastic interstory drift in the coupled direction in Structure_01 is 15mm ($C_d=5$) and l_w is equal to 3658mm resulting in $\delta_u/h_w=0.004<0.007$. The δ_u/h_w is less than 0.007 in all eight structures. Therefore, δ_u/h_w is taken as 0.007. Rearranging Equation 5.1, special boundary elements will be required when the ratio of c/l_w exceeds 0.24. Table 5.11 summarizes the compression zone depths at each floor level in structure_01 and structure_02. The remaining structures have similar results over the height of the structures.

As can be seen in Table 5.11, representative values of the depth of the compression zone for a one-cell and two-cell structure are shown (see Appendix C for a complete summary of the results). In most cases, c/l_w exceeds 0.24 with the exception of the tension walls of the one-cell configurations. Thus, using this method, special boundary elements are required over the entire height of all structures.

5.5.3 Discussion

Stress Comparisons - ACI 318-02 §21.7.6.3 –vs.- Fiber Analysis

Extreme fiber compressive stresses were determined from section fiber analysis and compared to the stresses calculated using the provisions of §21.7.6.3. The stresses based on fiber analysis for Structure_01 are also provided in Table 5.10. Comparing the stresses using the two different methods, there is good correlation between the stresses calculated based on gross section properties to the stresses determined from the more rigorous fiber analysis approach for all structures when considering LC2. However, when considering the tension wall stresses resulting from LC1 (point A for LC1), the gross section properties method significantly underestimates the stresses found from the fiber analyses. For example, considering Structure_01 (refer to Figure 5.4 and Table 5.10), the ACI 318 method (§21.7.6.3), using gross section properties and a linear distribution of stress over the depth of the section, indicates that special boundary elements would only be required at the first through third levels. However, the fiber analysis, considering cracked section properties and nonlinear material behavior, indicates that special boundary elements would be required up to and including level 12. Hence, a designer would significantly underestimate the need for special boundary elements following the more approximate method using gross section properties.

Figure 5.4a illustrates the distribution of stresses over the height of structure_01 at points A & C for LC1. Note that there is good correlation at point C but, at point A, the ACI 318-02 §21.7.6.3 method yields much more moderate stresses than that of the more rigorous fiber analysis. The differences in the calculated stresses in the tension wall (point A) can be directly attributed to the fact that cracked sections were considered in

the fiber analyses, but not considered using the method prescribed in ACI 318-02 §21.7.6.3. Relatively good correlation in the compression wall (point C) using the different methods is reasonable considering that the large axial compressive forces acting on the wall significantly diminish the effects of cracked sections. All structures have similar results. Note that the stresses at points D and F represent extreme fiber concrete tensile stress. The concrete model used in the fiber analysis appropriately was assigned to have no tensile capacity. As discussed previously, points D and F were considered in order to allow for cases where the entire depth of the section is in compression.

ACI 318-02 §21.7.6.2

This provision follows from a displacement-based design approach that considers cantilevered shear walls where axial loads due to frame action are not present and, as such, tensile normal stresses due to flexure will usually not be overcome by the compressive stresses resulting from gravity loads alone. The intent of this provision, and the reference used to develop the provision (Wallace 1994), is to ensure that special boundary elements are provided when concrete strains, at factored loads, exceed a critical strain. For strain levels below a critical strain, significant concrete strength degradation and shell spalling are not anticipated.

In the reported study, the critical compressive strain capacity was taken as 0.004. Using the XTRACT cross-section analysis software, the extreme fiber compressive strains at points A, B, and C (see Figure 5.2), under factored loads were computed for each floor of all the eight structures. The resulting concrete compressive strains are provided in Table 5.10. As can be seen in Table 5.10, the concrete strain at point A, at level 1 in structure_05, is 0.0018 which is the largest compressive strain for all of the

structures. This strain is a relatively moderate concrete compressive strain compared to the critical strain equal to 0.004. The extreme fiber concrete compressive strains are plotted over the height of the building for structure_05 in Figure 5.4b. Referring to Figure 5.4b, it is evident that, based on concrete strain capacity, special boundary elements would not be required in the wall piers of this structure. The results are the same for the other seven structures investigated in this study. Hence, special boundary elements are not required in the wall piers in all of the eight structures considered in this study.

5.6 Comparison of Wall Boundary Detailing

Where special boundary elements are required by §21.7.6.2 or §21.7.6.3, detailing requirements of §21.7.6.4 shall be satisfied. The provisions of §21.7.6.4 prescribe the requirements for providing a special boundary element. The boundary element must extend horizontally from the extreme compression fiber a distance not less than the larger of $c-0.1l_w$ and $c/2$ (§21.7.6.4(a)). §21.7.6.4 also requires that the provisions in §§21.4.4.1-21.4.4.3 be satisfied. These provisions require that the total cross-sectional area of transverse hoop reinforcement, A_{sh} , shall not be less than that required by the following equation (note that Equation 5.8 is an ACI 318 equation and, therefore *pound* and *inch* units are used).

$$A_{sh} = 0.09 \left(\frac{sh_c f'_c}{f_{yh}} \right) \quad (5.8)$$

Additionally, the transverse reinforcement must be provided by single or overlapping hoops. Crossties of the same bar size and spacing as that of the hoops must be provided and each end of the crosstie must engage a peripheral longitudinal reinforcing bar. Consecutive crossties must be alternated end-for-end along the longitudinal reinforcement. The spacing of the transverse reinforcement must not exceed the smaller

of (a) one-quarter of the minimum member dimension, (b) six times the diameter of the longitudinal reinforcement, and (c) s_x as defined by

$$s_x = 4 + \left(\frac{14 - h_x}{3} \right) \quad (5.9)$$

In Equation 5.10, s_x shall not exceed 6in (152mm) and need not be less than 4in (102mm). The crossties or legs of overlapping hoops shall not be spaced more than 14in (356mm) on center in the direction perpendicular to the longitudinal axis of the structural member. Thus, in wall piers where two curtains of longitudinal reinforcement are used, and center-to-center spacing of the curtains exceed 356mm, an additional longitudinal reinforcing bar is required.

Where special boundary elements are not required by §21.7.6.2 or §21.7.6.3, detailing requirements of §21.7.6.5 shall be satisfied. §21.7.6.5 contains two provisions that must be satisfied: (a) If the longitudinal reinforcement ratio at the wall boundary is greater than $400/f_y$, boundary transverse reinforcement must satisfy §21.4.4.1(c), §21.4.4.3, and §21.7.6.4(a) and the maximum transverse reinforcement in the boundary shall not exceed 8in (203mm), and (b) when V_u in the plane of the wall exceeds $A_{cv}\sqrt{f'_c}$ horizontal reinforcing terminating at the edges of structural walls without boundary elements shall have a standard hook engaging the edge reinforcement or the edge reinforcement shall be enclosed with U-stirrups having the same size and spacing as the horizontal reinforcement.

The ratio, $400/f_y$, for Gr. 60 steel is $40/60,000psi=0.000667$. Referring to Table 5.9, the smallest wall pier reinforcing ratio, $\rho=0.0081$, is in the wall piers at levels 13-20 in Structure_01. Therefore, the provision defined in §21.7.6.5(a) must be satisfied. Upon

examining the differences between the provisions where, (1) special boundary elements are required and (2) special boundary elements are not required, the only significant difference is that the maximum spacing of the transverse reinforcement is *6in* (152mm) and *8in* (203mm), respectively.

As shown in this study, special wall boundary elements are required when using (1) the method defined in §21.7.6.2 (extreme fiber concrete stress, calculated using uncracked section properties and assuming linear material behavior, exceeds $20\%\sqrt{f'_c}$), and (2) the method defined in §21.7.6.2 (the depth of the compression zone exceeds $0.24l_w$). However, it has been shown in this study, that special boundary elements are not required when considering the compressive strain capacity of structural walls. Recall that this method consisted of comparing the calculated extreme fiber concrete compressive strains at design loads versus a strain capacity equal to 0.004.

As discussed previously, upon examining the differences between the provisions where, (1) special boundary elements are required and (2) special boundary elements are not required, the only significant difference is that the maximum spacing of the transverse reinforcement is *6in* (152mm) and *8in* (203mm), respectively. To compare the provisions, an example problem was worked in where the provisions were applied to designing the wall pier boundary in the tension wall in Structure_01 for LC1 design forces. First, the wall boundary is designed assuming that special boundary elements were required (i.e., the provisions defined in §21.7.6.4). Second, the wall boundary is designed assuming that special boundary elements were not required (i.e., the provisions defined in §21.7.6.5).

Figure 5.5 shows the wall pier boundary designs detailed using the two different assumptions. As can be seen in the figure, the only difference when comparing the two

boundaries is that when special boundary elements are required, the required spacing of the transverse reinforcement is $127mm$; when special boundary elements are not required, the required spacing of the transverse reinforcement is $203mm$. It is the contention of the author that there is no appreciable difference between the two boundaries.

In both cases, additional longitudinal bars are required to be provided at mid-width of the wall to satisfy the maximum longitudinal bar spacing in the boundary zone of $14''$ ($356mm$) required by §21.4.4.4. The addition of the bar provided at mid-width of the wall further increases reinforcing steel congestion; even in wall piers where concrete compressive strains have been found to be moderate relative to the strain capacity of structural walls (0.004). Detailed calculations made to generate the solutions to this example problem are provided in Appendix D.

5.6 Conclusions

- The ACI 318-02 §21.7.6.2 provision for determining the need for special boundary elements, based on the location of the neutral axis, is not appropriate for wall piers in CCW systems. The compressive strains in the compression wall due to axial loads can overcome the tensile normal strains due to flexure resulting in a critical wall pier section that is entirely in compression while compressive strains are moderate and well below an acceptable compressive strain capacity equal to 0.004.
- The ACI 318-02 §21.7.6.3 provision for determining the need for special boundary elements, based on the state of stress at the extreme compression fiber, is not appropriate for wall piers in CCW systems. The axial forces imparted to the wall piers by the coupling beams are significant and the tension wall stresses can

be profoundly underestimated in cases where large net axial tension forces are present.

- The detailing requirements of wall boundary zones as prescribed in §21.7.6.4 and §21.7.7.6.5 are very similar and effectively result in similar boundary element detailing regardless of whether special boundary elements are required or not.
- Further research is required to assess the behavior of wall piers in CCW systems during the load reversals associated with earthquakes to determine if special boundary elements can be eliminated while maintaining sufficient axial load-carrying and flexural integrity in cases where extreme concrete compressive strains are expected to be moderate relative to the strain capacity of structural walls (0.004).

Table 5.1. Superimposed Gravity Loads.

Superimposed Gravity Loads		
Floor Loads		
MEP:	0.24	(kPa)
Columns:	0.24	(kPa)
Partitions:	0.48	(kPa)
Total	0.96	(kPa)
Line Loads		
Cladding	4.38	(kN/m)
Live Load		
	2.39	(kPa)

Table 5.2. Plan dimensions of structures (refer to Figure 5.1).

Structure	Type	X (m)	Y (m)	CX (m)	CY (m)	WL (m)	FL (m)	WLI (m)	BL (m)
01	one-cell	24.4	24.4	9.1	6	3.7	4.7	n/a	1.8
02	two-cell	30.5	24.4	13.4	6.7	2.6	5.5	4.6	1.8
03	two-cell	42.7	30.5	21.9	7.9	4.6	7.9	9.2	1.8
04	one-cell	30.5	30.5	9.1	6	3.7	4.7	n/a	1.8
05	two-cell	42.7	30.5	16.5	9.1	4	7.9	4.9	1.8
06	two-cell	30.5	24.4	14.6	6.7	3.1	5.5	4.9	1.8
07	one-cell	24.4	24.4	6.7	4.9	2.4	3.7	n/a	1.8
08	one-cell	30.4	30.4	6.7	4.9	2.4	3.7	n/a	1.8

Table 5.3. Seismic response coefficients, weights and base shears.

Base Shear Summary			
Structure	C _s	W (kN)	V _B (kN)
01	0.0682	85095	5806
02	0.0682	107984	7368
03	0.0682	177644	12121
04	0.0682	121733	8306
05	0.0682	173543	11842
06	0.0682	109136	7447
07	0.0682	81403	5554
08	0.0682	118148	8062

Table 5.4. ELF calculated lateral forces and story shears.

Equivalent Lateral Forces Summary																
Floor	Structure 01		Structure 02		Structure 03		Structure 04		Structure 05		Structure 06		Structure 07		Structure 08	
	F _x (kN)	V _x (kN)	F _x (kN)	V _x (kN)	F _x (kN)	V _x (kN)	F _x (kN)	V _x (kN)	F _x (kN)	V _x (kN)	F _x (kN)	V _x (kN)	F _x (kN)	V _x (kN)	F _x (kN)	V _x (kN)
20	667	667	846	846	1392	1392	954	954	1360	1360	855	855	638	638	925	925
19	619	1286	786	1632	1293	2685	886	1839	1263	2622	794	1649	592	1230	860	1785
18	573	1859	727	2359	1196	3881	820	2659	1168	3791	735	2384	548	1778	795	2580
17	528	2386	670	3028	1102	4982	755	3414	1076	4867	677	3061	505	2283	733	3313
16	484	2870	614	3642	1010	5992	692	4106	986	5853	620	3681	463	2745	672	3985
15	441	3311	559	4202	920	6912	631	4737	899	6752	565	4246	422	3167	612	4597
14	399	3710	507	4708	833	7746	571	5308	814	7567	512	4758	382	3549	554	5151
13	359	4069	455	5163	749	8495	513	5821	732	8298	460	5219	343	3892	498	5649
12	320	4389	406	5569	668	9163	458	6279	652	8951	410	5629	306	4198	444	6093
11	282	4671	358	5928	589	9752	404	6682	576	9526	362	5991	270	4468	392	6485
10	246	4917	312	6240	514	10266	352	7034	502	10028	316	6307	235	4704	342	6827
9	212	5129	268	6508	442	10707	303	7337	431	10460	271	6578	202	4906	294	7120
8	179	5307	227	6735	373	11080	255	7592	364	10824	229	6807	171	5077	248	7368
7	147	5455	187	6922	308	11387	211	7803	301	11124	189	6996	141	5218	205	7573
6	118	5573	150	7072	246	11634	169	7972	241	11365	151	7147	113	5331	164	7737
5	91	5664	115	7187	190	11823	130	8102	185	11550	116	7264	87	5418	126	7863
4	66	5730	84	7271	138	11961	94	8196	134	11685	85	7348	63	5481	92	7955
3	44	5773	55	7326	91	12052	62	8259	89	11774	56	7404	42	5522	61	8015
2	24	5798	31	7357	51	12103	35	8294	50	11823	31	7435	23	5546	34	8049
1	9	5807	11	7368	19	12122	13	8306	18	11841	12	7447	9	5554	12	8062
	5807		7368		12122		8306		11841		7447		5554		8062	

Table 5.5. Redundancy and wall overstrength factors.

ρ and γ_{wall} Factors Summary		
Structure	ρ	γ_{wall}
01	1.428	1.62
02	1.274	1.37
03	1.301	1.32
04	1.500	1.72
05	1.500	1.32
06	1.324	1.32
07	1.500	1.73
08	1.500	1.85

Table 5.6. Coupling beam design shear demands and factored capacities.

Structure	Floor Level	Design Shear (kN)	Factored Capacity (kN)	Structure	Floor Level	Design Shear (kN)	Factored Capacity (kN)
01	17-20	445	540	05	17-20	424	545
	13-16	753	811		13-16	775	835
	9-12	1032	1081		9-12	1119	1162
	5-8	1109	1151		5-8	1220	1262
	1-4	995	1081		1-4	1082	1162
02	17-20	305	430	06	17-20	237	304
	13-16	542	619		13-16	446	484
	9-12	760	813		9-12	635	650
	5-8	827	859		5-8	681	692
	1-4	757	813		1-4	614	678
03	17-20	243	295	07	17-20	716	865
	13-16	391	431		13-16	1121	1176
	9-12	472	481		9-12	1600	1665
	5-8	474	481		5-8	2104	2144
	1-4	332	363		1-4	2131	2188
04	17-20	744	951	08	17-20	1141	1199
	13-16	1302	1427		13-16	1810	1840
	9-12	1828	1921		9-12	2400	2399
	5-8	2160	2226		5-8	3469	3467
	1-4	2084	2188		1-4	3599	3590

Table 5.7. Wall pier design forces for Structure_01.

Pier Design Forces and Moments														
Structure	Floor	C-Shaped Wall Piers							I-Shaped Walls					
		LC1				LC2			LC1			LC2		
		Pu		Mx (kN-m)	My (kN-m)	Pu (kN)	Mx (kN-m)	My (kN-m)	Pu (kN)	Mx (kN-m)	My (kN-m)	Pu (kN)	Mx (kN-m)	My (kN-m)
		Comp. (kN)	Tens. (kN)											
Structure_01	20	2747	-493	334	4033	1703	1113	522	n/a	n/a	n/a	n/a	n/a	n/a
	19	5494	-986	968	6010	3424	3225	823	n/a	n/a	n/a	n/a	n/a	n/a
	18	8241	-1479	1827	6434	5164	6089	932	n/a	n/a	n/a	n/a	n/a	n/a
	17	10988	-1971	2845	6001	6925	9483	920	n/a	n/a	n/a	n/a	n/a	n/a
	16	14336	-3065	3968	6202	8753	13226	974	n/a	n/a	n/a	n/a	n/a	n/a
	15	17684	-4158	5155	6127	10602	17184	993	n/a	n/a	n/a	n/a	n/a	n/a
	14	21032	-5251	6383	7129	12469	21276	1194	n/a	n/a	n/a	n/a	n/a	n/a
	13	24380	-6344	7639	8404	14353	25463	1454	n/a	n/a	n/a	n/a	n/a	n/a
	12	28329	-8038	8922	9291	16293	29739	1630	n/a	n/a	n/a	n/a	n/a	n/a
	11	32278	-9731	10236	10198	18246	34119	1818	n/a	n/a	n/a	n/a	n/a	n/a
	10	36227	-11424	11590	11156	20212	38634	2024	n/a	n/a	n/a	n/a	n/a	n/a
	9	40176	-13118	12997	12216	22188	43324	2255	n/a	n/a	n/a	n/a	n/a	n/a
	8	44281	-14967	14471	13331	24183	48235	2496	n/a	n/a	n/a	n/a	n/a	n/a
	7	48387	-16817	16023	14724	26182	53410	2794	n/a	n/a	n/a	n/a	n/a	n/a
	6	52492	-18666	17665	16533	28182	58884	3173	n/a	n/a	n/a	n/a	n/a	n/a
	5	56597	-20516	19402	18945	30173	64674	3665	n/a	n/a	n/a	n/a	n/a	n/a
	4	60546	-22210	21235	22375	32131	70784	4351	n/a	n/a	n/a	n/a	n/a	n/a
	3	64495	-23903	23158	27128	34048	77193	5261	n/a	n/a	n/a	n/a	n/a	n/a
	2	68444	-25596	25160	33912	35891	83868	6485	n/a	n/a	n/a	n/a	n/a	n/a
	1	72392	-27290	27227	44022	37619	90755	8165	n/a	n/a	n/a	n/a	n/a	n/a

Table 5.8. Beam dimensions.

Summary of Beam Dimensions												
Structure	Floor Levels	d	h _w	t _w	b _f	t _f	Structure	d	h _w	t _w	b _f	t _f
		(mm)	(mm)	(mm)	(mm)	(mm)		(mm)	(mm)	(mm)	(mm)	(mm)
01	17-20	419	318	13	330	51	05	610	533	8	330	38
	13-16	419	318	19	330	51		610	533	12	330	38
	9-12	419	318	25	330	51		610	533	16	330	38
	5-8	419	318	27	330	64		610	533	18	330	38
	1-4	419	318	25	330	51		610	533	16	330	38
02	17-20	438	337	10	305	51	06	483	406	6	330	38
	13-16	438	337	14	305	51		483	406	9	330	38
	9-12	438	337	18	305	51		483	406	12	330	38
	5-8	438	337	19	305	51		483	406	13	330	38
	1-4	438	337	18	305	51		483	406	12	330	38
03	17-20	368	267	8	229	51	07	610	508	13	356	51
	13-16	368	267	12	229	51		610	508	17	356	51
	9-12	368	267	13	229	51		660	559	22	356	51
	5-8	368	267	13	229	51		660	559	29	356	51
	1-4	368	267	10	229	51		660	559	29	356	51
04	17-20	660	559	13	406	51	08	635	483	19	406	76
	13-16	660	559	19	406	51		635	483	28	406	76
	9-12	660	559	26	406	51		635	483	37	406	76
	5-8	660	559	30	406	64		635	483	54	406	76
	1-4	660	559	29	406	51		635	483	55	406	76

Table 5.9. Wall pier longitudinal steel reinforcement.

Pier Reinforcement Summary																		
Structure	Floors	C-Shape				I-Shape				Structure	C-Shape				I-Shape			
		ρ (%)	Bar Qty.	Bar Size (#)	Bar Spacing (mm)	ρ (%)	Bar Qty.	Bar Size (#)	Bar Spacing (mm)		ρ (%)	Bar Qty.	Bar Size (#)	Bar Spacing (mm)	ρ (%)	Bar Qty.	Bar Size (#)	Bar Spacing (mm)
01	17-20	0.81	92	9	229	n/a	n/a	n/a	n/a	05	0.84	208	7	6	0.71	150	8	203
	13-16	0.81	92	9	229	n/a	n/a	n/a	n/a		0.84	208	7	6	0.71	150	8	203
	9-12	0.81	92	9	229	n/a	n/a	n/a	n/a		0.84	208	7	6	0.71	150	8	203
	5-8	1.26	92	11	229	n/a	n/a	n/a	n/a		0.84	208	7	6	0.71	150	8	203
	1-4	1.82	92	14	229	n/a	n/a	n/a	n/a		0.84	208	7	6	0.71	150	8	203
02	17-20	1.05	106	9	178	1.05	128	10	203	06	1.04	114	9	8	1.32	150	10	203
	13-16	1.05	106	9	178	1.05	128	10	203		1.04	114	9	8	1.32	150	10	203
	9-12	1.05	106	9	178	1.05	128	10	203		1.04	114	9	8	1.32	150	10	203
	5-8	1.39	90	11	178	1.05	128	10	203		1.04	114	9	8	1.32	150	10	203
	1-4	2.00	90	14	178	1.70	128	14	203		1.04	114	9	8	1.32	150	10	203
03	17-20	0.47	126	7	254	0.67	212	8	229	07	1.48	94	10	7	n/a	n/a	n/a	n/a
	13-16	0.47	126	7	254	0.67	212	8	229		1.48	94	10	7	n/a	n/a	n/a	n/a
	9-12	0.47	126	7	254	0.67	212	8	229		1.48	94	10	7	n/a	n/a	n/a	n/a
	5-8	0.47	126	7	254	0.67	212	8	229		2.62	94	14	7	n/a	n/a	n/a	n/a
	1-4	0.47	126	7	254	0.67	212	8	229		3.68	132	14	5	n/a	n/a	n/a	n/a
04	17-20	1.04	118	9	127	n/a	n/a	n/a	n/a	08	1.39	112	9	6	n/a	n/a	n/a	n/a
	13-16	1.04	118	9	127	n/a	n/a	n/a	n/a		1.76	112	10	6	n/a	n/a	n/a	n/a
	9-12	1.31	118	10	127	n/a	n/a	n/a	n/a		3.13	112	14	6	n/a	n/a	n/a	n/a
	5-8	2.17	158	11	127	n/a	n/a	n/a	n/a		5.56	112	18	6	n/a	n/a	n/a	n/a
	1-4	3.72	188	14	127	n/a	n/a	n/a	n/a		5.95	120	18	5.5	n/a	n/a	n/a	n/a

Table 5.10. Extreme fiber concrete compressive stresses and strains.

Extreme Fiber Concrete Stresses and Strains													
Structure	Floor	C-Shaped Walls											
		Concrete Stress								Concrete Strain			
		ACI 21.7.6.3				Fiber Analysis							
		LC1		LC2		LC1		LC2		LC1		LC2	
		Point A (MPa)	Point C (MPa)	Point A (MPa)	Point C (MPa)	Point A (MPa)	Point C (MPa)	Point A (MPa)	Point C (MPa)	Point A (10 ⁻³)	Point C (10 ⁻³)	Point A (10 ⁻³)	Point C (10 ⁻³)
Structure_01	20	1.0	-0.7	0.5	-0.9	2.2	0.0	0.4	-1.3	0.081	-0.162	0.015	-0.045
	19	1.5	-0.8	0.9	-1.6	3.5	0.0	0.9	-1.9	0.126	-0.124	0.031	-0.069
	18	1.7	-0.4	1.4	-2.1	3.9	0.0	1.3	-2.1	0.142	-0.031	0.048	-0.076
	17	1.5	0.1	1.9	-2.5	4.1	0.2	1.8	-2.4	0.148	0.008	0.065	-0.085
	16	1.5	0.6	2.5	-3.1	4.6	0.7	2.3	-2.9	0.166	0.025	0.084	-0.104
	15	1.5	1.2	3.1	-3.6	4.9	1.2	2.9	-3.4	0.177	0.045	0.103	-0.122
	14	1.7	1.5	3.7	-4.3	5.8	1.5	3.4	-4.0	0.209	0.056	0.124	-0.146
	13	2.0	1.7	4.4	-5.1	6.8	1.8	4.1	-4.7	0.237	0.064	0.147	-0.170
	12	2.1	2.1	5.0	-5.8	7.5	2.2	4.7	-5.4	0.272	0.078	0.169	-0.196
	11	2.2	2.5	5.7	-6.6	8.2	2.6	5.3	-6.1	0.296	0.093	0.192	-0.222
	10	2.3	2.9	6.4	-7.4	8.9	3.0	5.9	-6.8	0.322	0.107	0.215	-0.248
	9	2.5	3.3	7.1	-8.2	10.0	3.3	6.6	-7.6	0.363	0.120	0.239	-0.275
	8	2.7	3.6	7.8	-9.0	8.5	3.6	7.1	-8.1	0.307	0.131	0.257	-0.295
	7	2.9	3.9	8.5	-9.9	9.3	3.9	7.8	-8.9	0.337	0.142	0.283	-0.324
	6	3.3	4.2	9.4	-10.8	10.4	4.2	8.5	-9.7	0.380	0.150	0.310	-0.354
	5	3.8	4.2	10.2	-11.8	12.4	4.3	9.3	-10.7	0.452	0.154	0.338	-0.388
	4	4.6	4.0	11.2	-13.0	11.7	4.0	9.9	-11.3	0.426	0.144	0.358	-0.412
	3	5.8	3.4	12.2	-14.3	14.0	3.6	10.7	-12.4	0.514	0.128	0.390	-0.454
	2	7.5	2.3	13.3	-15.9	17.5	2.6	11.7	-13.8	0.649	0.093	0.427	-0.508
	1	10.1	0.3	14.5	-18.0	26.2	0.6	12.8	-15.8	1.050	0.043	0.469	-0.585
Structure_05	20	0.5	0.2	0.6	-0.62	0.5	0.2	0.6	-0.6	0.018	0.008	0.020	-0.021
	19	1.0	0.4	1.3	-1.25	1.0	0.4	1.2	-1.2	0.037	0.015	0.043	-0.041
	18	1.4	0.5	1.9	-1.84	1.5	0.6	1.8	-1.7	0.054	0.030	0.064	-0.061
	17	1.8	0.6	2.5	-2.42	2.0	0.7	2.4	-2.2	0.071	0.024	0.085	-0.080
	16	2.3	0.7	3.3	-3.08	2.7	0.8	3.0	-2.8	0.098	0.028	0.109	-0.102
	15	2.7	0.8	4.0	-3.74	3.5	0.9	3.7	-3.5	0.126	0.032	0.134	-0.124
	14	3.1	0.8	4.7	-4.39	4.2	1.0	4.4	-4.1	0.153	0.036	0.158	-0.146
	13	3.5	0.9	5.4	-5.04	4.9	1.1	5.1	-4.7	0.179	0.040	0.183	-0.168
	12	3.9	1.0	6.2	-5.78	5.9	1.3	5.8	-5.3	0.213	0.046	0.210	-0.192
	11	4.4	1.1	7.1	-6.57	7.0	1.4	6.6	-6.1	0.253	0.049	0.239	-0.219
	10	5.1	0.9	7.9	-7.47	8.5	1.3	7.4	-6.9	0.309	0.046	0.268	-0.248
	9	5.9	0.7	8.8	-8.42	10.1	1.1	8.2	-7.7	0.370	0.041	0.299	-0.280
	8	6.7	0.4	9.8	-9.40	11.9	0.9	9.1	-8.6	0.433	0.034	0.330	-0.312
	7	7.7	0.1	10.7	-10.44	13.8	0.7	10.0	-9.6	0.505	0.025	0.363	-0.347
	6	8.7	-0.3	11.7	-11.53	15.8	0.3	10.9	-10.6	0.584	0.011	0.398	-0.385
	5	9.9	-0.8	12.8	-12.70	18.0	0.0	12.1	-11.7	0.674	-0.009	0.436	-0.425
	4	11.3	-1.4	14.0	-13.94	20.6	0.0	13.0	-12.9	0.780	-0.040	0.477	-0.471
	3	12.9	-2.1	15.3	-15.27	23.3	0.0	14.2	-14.3	0.907	-0.088	0.522	-0.523
	2	14.7	-2.9	16.8	-16.68	27.5	0.0	15.5	-15.8	1.126	-0.155	0.573	-0.583
	1	16.8	-3.7	18.4	-18.17	34.3	0.0	17.0	-17.6	1.883	-0.271	0.630	-0.655

Table 5.11. c/l_w ratios (results of fiber analyses). Limit is 0.24.

c/l _w ratios													
Structure	Floor	Tension Wall		Compression Wall		Structure	Floor	Tension Wall		Compression Wall		I-Shaped Wall	
		LC1	LC2	LC1	LC2			LC1	LC2	LC1	LC2	LC1	LC2
Structure_01	20	0.22	1.00	0.22	1.00	Structure_02	20	0.74	1.00	0.85	1.00	0.49	1.00
	19	0.22	1.00	0.36	1.00		19	0.63	1.00	0.85	1.00	0.51	0.91
	18	0.22	1.00	0.69	1.00		18	0.63	1.00	0.85	1.00	0.59	0.86
	17	0.22	1.00	1.00	1.00		17	0.52	1.00	0.85	1.00	0.74	0.78
	16	0.22	1.00	1.00	1.00		16	0.48	1.00	0.82	1.00	0.79	0.70
	15	0.19	1.00	1.00	1.00		15	0.59	1.00	0.81	1.00	0.90	0.63
	14	0.19	0.97	1.00	1.00		14	0.41	1.00	0.85	1.00	1.00	0.54
	13	0.19	0.93	1.00	1.00		13	0.41	1.00	0.81	1.00	1.00	0.49
	12	0.14	0.92	1.00	0.97		12	0.37	1.00	0.85	1.00	1.00	0.46
	11	0.14	0.92	1.00	0.93		11	0.37	1.00	0.85	1.00	1.00	0.42
	10	0.14	0.88	1.00	0.92		10	0.30	1.00	0.85	1.00	1.00	0.40
	9	0.14	0.88	1.00	0.88		9	0.30	1.00	0.81	1.00	1.00	0.40
	8	0.19	0.87	1.00	0.88		8	0.30	1.00	0.74	1.00	1.00	0.37
	7	0.19	0.83	1.00	0.83		7	0.30	1.00	0.71	1.00	1.00	0.35
	6	0.19	0.83	1.00	0.83		6	0.30	1.00	0.63	1.00	1.00	0.33
5	0.19	0.78	1.00	0.82	5	0.30	1.00	0.59	1.00	1.00	0.33		
4	0.22	0.77	1.00	0.82	4	0.37	0.97	0.52	1.00	1.00	0.40		
3	0.22	0.73	1.00	0.78	3	0.37	0.94	0.48	1.00	0.90	0.39		
2	0.22	0.68	1.00	0.73	2	0.37	0.93	0.41	0.97	0.79	0.39		
1	0.25	0.32	1.00	0.73	1	0.41	0.90	0.41	0.94	0.67	0.39

Figure 5.1. (a) One-cell type CCW (b) two-cell type CCW. (dimensions given in Table 1).

Figure 5.2. Locations of interest for stress calculations and reported stresses and strains for (a) tension wall, (b) compression wall, (c) I-shaped wall, and nomenclature used in equations.

Figure 5.3. Example of ETABS model. (a) floor plan. (b) 3-D elevation. Structure_02 is shown.

Figure 5.4(a). Extreme fiber concrete stresses for Structure_01, LC1.

Figure 5.4(b). Extreme fiber concrete strains for Structure_05, LC1.

(a)

(b)

Figure 5.5. Wall boundary detailing of the base tension wall pier in Structure_01 where special boundary elements are assumed to be (a) required and (b) not required.

Chapter 6: Nonlinear Response-History Analysis

This chapter presents the results of nonlinear response-history analyses of two core wall buildings. Both buildings have the same plan and story dimensions. However, steel coupling beams are used for one of the buildings while the other has diagonally-reinforced concrete coupling beams. The analyses are two-dimensional and consider earthquakes acting in the coupled direction.

The design of the wall piers and coupling beams is based on the results of an elastic analysis using the equivalent lateral force procedure (ELF). The ELF procedure is described in Chapter 5. Each structure was subsequently modeled and nonlinear response analyses were performed.

Each building was subjected to four different ground motion records. Two of the ground motion records are recorded motions from past earthquakes: (1) the north-south direction ground motion recorded during the El Centro earthquake in May of 1940 and (2) the north-south ground motion recorded at the Sylmar hospital station during the Northridge earthquake in January of 1994. The final two ground motions are artificial ground motions based on: (3) ASCE7-02 10% chance of exceedance in 50 years and (4) ASCE7-02 2% chance of exceedance in 50 years. Upon finalizing the analyses, the overall behavior of each of the structures was evaluated and compared.

The wall properties used in the nonlinear models were derived from analytically-generated axial force-moment interaction (P-M) diagrams and moment-curvature (M- ϕ) relationships. The variables for beam hysteretic models were established based on calibration of experimental data (load-deflection responses) as described in Chapter 3.

6.1 Structure Parameters

The structures assumed for the nonlinear analyses are the same building as Structure_01 described in Chapter 5 (for reference, the plan geometry and assumed superimposed gravity loads are provided again in Figure 6.1 and Table 6.1, respectively). The dynamic weight of the structure with diagonally reinforced coupling beams is slightly different than that of the SCB building, hence the base shear and distribution of story lateral forces are different. The force distribution within the structure will also be different due to the difference in stiffness between a steel coupling beam and a diagonally reinforced concrete coupling beam. As a result, wall pier forces for the building with the diagonally reinforced coupling beams will be different. Because of these differences, the walls piers in the DBCB building need to be redesigned.

6.2 Elastic Analysis and Results

The mapped maximum considered earthquake spectral response accelerations, superimposed gravity loads, and resulting design response spectrum for the two buildings are the same as those described in Chapter 5. The resulting seismic response coefficients, dynamic weights, base shears, lateral story forces, and story shear distributions are given in Tables 6.2 and 6.3, respectively.

Again, the ETABS software was used for the elastic analysis of the DBCB building. The redundancy factors for the SCB building and the DBCB building are 1.4 and 1.5, respectively. The wall overstrength factors for the SCB building and the DBCB building are 1.62 and 2.12, respectively. These values are summarized in Tables 6.4.

6.2.1 Coupling Beam Design

The procedures used in the design and modeling of the coupling beams for the building with steel coupling beams are the same as those described in previous chapters. The design of the diagonally reinforced coupling beams is the same as that detailed in Chapter 2. However, the modeling parameters for concrete beams used in the elastic analysis is different than that of the steel coupling beams due to the need for accounting for cracked section properties.

To account for cracked section properties, the concrete modulus of elasticity was reduced by a factor related to the effective moment of inertia of the concrete beam. To determine the effective moment of inertia of the beam, the equation recommended by Paulay and Priestly (1992) for estimating the effective moment of inertia of a diagonally reinforced concrete beam was used. The equation is as follows.

$$I_e = \frac{\alpha I_g}{1 + 3 \left(\frac{L}{h} \right)^2} \quad (6.1)$$

Where, I_e is the effective moment of inertia, I_g is the gross moment of inertia (ignoring steel), L is the clear span length of the beam, h is the nominal height of the beam, and α is 0.4 for DBCBs. Equation 6.1 shows that the effective moment of inertia is a function of the aspect ratio (L/h) of the beam. To account for the effective moment of inertia in ETABS, the value of the concrete modulus of elasticity was reduced by a factor equal to the ratio of I_e/I_g .

The final beam designs for the steel and diagonally-reinforced concrete beams are given in Table 6.5. The values of I_e/I_g used over the height of the DBCB building are given in Table 6.6. Figure 6.2 shows the final DBCB designs. Transverse reinforcement

around the diagonal bar groups is provided as needed to hold the bars in place during construction. Figure 6.3 shows the beam shear demand and capacity distributions over the height of the building for both the SCB and DBCB buildings.

6.2.2 Wall Pier Design

As discussed in Chapter 5, the current ACI provisions for establishing the need for special boundary elements is not appropriate for wall piers in coupled core wall systems. Since the building geometry considered for nonlinear analysis are the same as Structure_01 in Chapter 5, where it is shown that the extreme fiber concrete compressive strains are well below a critical value set at 0.004, it is assumed that special boundary elements will not be necessary for the DBCB building. Additionally, the ACI requirements for transverse reinforcement in the boundaries of the wall piers as defined by §21.7.6.5, where special boundary elements are not required, will not be considered.

Similar to the wall pier longitudinal reinforcement of the structures in Chapter 5, two curtains of uniformly distributed longitudinal steel reinforcement will be provided. The wall pier reinforcement provided for the SCB building is the same as that provided for Structure_01. The reinforcement requirements for the DBCB building is, however, different. The flexural demand on the wall piers in DBCB building will be larger compared to the SCB building due to the larger stiffness of steel coupling beams as compared to diagonally reinforced concrete coupling beams. Again, XTRACT was used to generate the P-M diagrams used to assure adequate axial and flexural capacity. A summary of the wall pier design forces and the longitudinal steel reinforcement provided for each building is shown in Tables 6.7 and 6.8, respectively.

6.2.2.1 Wall Overstrength Factor

The larger wall overstrength factor for the DBCB building (refer to Section 6.2) is a direct result of the difference in the beam capacity factors used for steel coupling beams and diagonally-reinforced coupling beams. The following demand/capacity relationships in the design of the beams must be satisfied according to AISC Manual of Steel Construction (for SCB) and ACI318-12 (for DBCB).

$$V_{u,conc} \leq \phi_{v,conc} V_{n,conc} \quad (6.2)$$

$$V_{u,steel} \leq \phi_{v,steel} V_{n,steel} \quad (6.3)$$

Where, $V_{u,i}$ is the design shear demand for a concrete or steel beam, $V_{n,i}$ is the nominal shear capacity of a concrete or steel beam, $\phi_{v,conc}$ is the shear strength reduction factor for a concrete beam (equal 0.75), and $\phi_{v,steel}$ is the shear strength reduction factor for a steel beam (equal to 0.9). Assuming that the design shear demand for the concrete and steel coupling beams is the same, $V_{u,conc} = V_{u,steel}$ and Equations 6.2 and 6.3 can be set equal to each other resulting in the following relationship between the required nominal strength of a concrete beam relative to a steel beam.

$$\begin{aligned} \phi_{v,conc} V_{n,conc} &= \phi_{v,steel} V_{n,steel} \\ 0.75 V_{n,conc} &= 0.9 V_{n,steel} \end{aligned} \quad (6.4)$$

$$V_{n,conc} = 1.2 V_{n,steel} \quad (6.5)$$

Equation. 6.5 shows that for the same shear demand, the nominal strength of a concrete beam must be at least 20% greater than the nominal strength of a steel beam. The wall overstrength factor is the ratio of the sum of the nominal shear capacities of the beams over the height of the building to the sum of the design shear demands over the

height of the building. Due to the larger capacity reduction factor for a concrete beam relative to a steel beam, the ratio of nominal shear capacities to design shear demand over the height of the building is larger for a building with diagonally-reinforced concrete coupling beams than that of a building with steel coupling beams. Thus, the wall overstrength factor for a DBCB building is greater than that of a SCB building.

6.3 Nonlinear Modeling and Parameters

6.3.1 Determining Beam Hysteretic Rules and Parameters

The experimentally measured hysteretic characteristics were used to characterize the behavior of the coupling beams used in the nonlinear models. To facilitate this process, the Hysteresis module of the Ruaumoko Nonlinear Analysis program (version 2.6) was used. The Hysteresis module uses known mathematical hysteresis rules to “match” measured hysteretic behavior. The procedure is an iterative process where the hysteresis rule parameters are adjusted until the mathematical model generates a force-deflection (F-d) curve that correlates with a given force-deflection curve, which in this case was the experimentally measured force-deflection curves.

6.3.1.1 Steel Coupling Beam Hysteresis

Figure 6.5 shows the Al-Bermani mathematical model and the model parameters used to match the measured steel coupling beam F-d. These were arrived at after an exhaustive number of iterations leading to the results shown in Figure 6.6. Note that the best correlation is achieved when the hysteresis model with the parameters shown in Figure 6.5 is considered to have no strength degradation.

Two mathematical hysteresis rules were considered most appropriate for the steel coupling beam: (1) the Ramberg-Osgood hysteresis rule and (2) the Al-Bermani

Bounding-Surface hysteresis rule. Both rules were investigated as possible mathematical models for the analyses. The Ramberg-Osgood model generated an acceptable correlation to the measured F-d curve; however, initial analyses indicated suspicious results. The calculated force-deflection responses (see Figure 6.4(a)) were unstable and exhibited dramatic and abrupt changes in stiffness with no inelastic behavior. As this behavior could not be rationalized, the Ramberg-Osgood model was abandoned. On the other hand, the Al-Bermani hysteresis rule produces reasonable results (see Figure 6.4(b)).

The aforementioned hysteresis parameters were based on the experimental data from a ½ scale model of the beams in the 20-story buildings to be analyzed. Therefore, the initial stiffness, positive yield force, and negative yield force must be scaled for the full-scale beams. Accordingly, initial stiffness, k_0 , has to be multiplied by 2, and the yield strengths (F_y^+ and F_y^-) have to be multiplied by 4. Additional adjustments are needed to account for the different size beams used over the height of the buildings. The shear capacity of a steel beam is directly proportional to the area of the web. Therefore, the scaling factor is the ratio of the web area of a full-scale equivalent of the test beams, which was the basis of the identified parameters shown in Figure 6.5, to the web area of a particular beam at a given floor level. The web area of the experimental beam is $4,516mm^2$; hence, the web area of the equivalent full-scale beam is $(4)(4,516mm^2)=18,064mm^2$. The web area of the beams at floor levels 1-4 is $8,065mm^2$. Therefore, the full-scale hysteretic parameters, for beams at floor levels 1-4, are scaled by a factor of $8,065/18,064=0.4465$, resulting in values of $k_0=(79)(2)(0.4465)=70kN/m$, $F_y^{(+)}=(712)(4)(0.4465)=1,271kN$, and $F_y^{(-)}=(-578)(4)(0.4465)=-1,033kN$. The parameters of the beams at the other floor levels are scaled in the same manner.

6.3.1.2 Diagonally Reinforced Coupling Beam Hysteresis

The Modified Takeda hysteresis rule was chosen as the mathematical hysteresis model for the diagonally-reinforced concrete beam. The same iterative process used for identifying the SCB hysteresis parameters was followed in generating the required parameters for the DBCB. The measured F-d curve of the DBCB exhibits strength degradation, and as such the required number of iterations needed to arrive at an acceptable correlation was much more intense (relative to the SCB) due to the number of parameters needed for each iteration. The Modified Takeda mathematical model, hysteretic parameters, and F-d correlation plot are illustrated in Figures 6.7, 6.8, and 6.9, respectively.

To account for scaling of the hysteretic parameters for the diagonally reinforced beams, a similar procedure was used as that described for the SCB. However, the shear strength of a diagonally-reinforced concrete beam is directly proportional to $A_{vd}\sin\alpha$ where A_{vd} is the steel area of one diagonal bar group and α is the angle of inclination of the diagonal bar group. For example, $A_{vd}\sin\alpha$ for the experimental beam is 562mm^2 ; hence, for a full-scale beam, $A_{vd}\sin\alpha=(4)(562\text{mm}^2)=2,248\text{mm}^2$. The value $A_{vd}\sin\alpha$ for the beams at floor levels 1-4 is $1,219\text{mm}^2$. Therefore, the hysteretic parameters for the beams at floor levels 1-4 are scaled by $(1,219/2,248)=0.5423$. Thus, the hysteretic parameters for the beams at floor levels 1-4 are $k_0=(57)(2)(0.5423)=57\text{kN/m}$, $F_y^{(+)}=(489)(4)(0.5423)=1,062\text{kN}$, and $F_y^{(-)}=(-489)(4)(0.5423)=-1,062\text{kN}$. The beams at the other floor levels are scaled similarly.

6.3.1.3 Wall Pier Properties

To determine the wall pier properties, axial force moment interaction (P-M) diagrams were used to determine the failure surfaces. In generating the P-M diagrams, the limiting concrete compressive strain and steel tension strain were taken as 0.003 and 0.15, respectively. Thus, the P-M diagrams represent failure surfaces considering these strains. In all following discussions, the term failure implies a combination of axial force and moment outside of the P-M surface which was generated based on these aforementioned strains. M- ϕ diagrams were used to determine the effective moment of inertias and bilinear factors. For both buildings, the design of the longitudinal steel for the wall piers in floor levels 9-20 are the same. The longitudinal steel is different for floor levels 5-8 and 1-4. Thus, different wall pier properties were determined for floor levels 1-4, 5-8, and 9-20. Additionally, to account for hinging at the base of the wall piers, the parameters used to define the base wall pier (level 1), are different than the parameters used for the wall piers at floor levels 2-4.

The nonlinear analyses are in the coupled direction. Therefore, the yield surfaces for the wall piers are based on the P-M diagrams for moments about the y-axis. The C-shape walls are not symmetrical about the y-axis, and hence, the P-M diagrams are not symmetrical. The P-M diagrams for floors 1-4, 5-8, and 9-20 for the SCB building and the DBCB building are shown in Figures 6.10, 6.11, and 6.12, respectively. The critical points used to define the P-M diagrams for the different wall piers are shown in the Figure 6.13.

It is important to recognize that the sign convention used in XTRACT is such that positive moments represent moments which impart axial tension in the web of the C-

shape. Conversely, negative moments represent moments that impart axial compression in the web of the C-shape. Positive axial forces are above the x-axis. The tabulated values shown in Figure 6.13 reflect the difference in the sign conventions used in XTRACT and Ruaumoko.

The wall piers behave differently depending on the sense of the moment imparted to them. As can be seen in the P-M diagrams, the C-shape walls have a larger moment capacity when the web is in compression (negative moment) than when the web is in tension (positive moment). This type of asymmetric behavior has a direct influence on the moment-curvature relationships of the wall depending on the sense of the moment. The positive moment-curvature relationship is much softer than the negative moment-curvature relationship. This is illustrated in the moment-curvature plots for the wall piers shown in Figures 6.14-6.19. In these figures, the $M-\phi$ relationships are shown for both the tension walls and compression walls for floor levels 1-4, 5-8, and 9-20 for both buildings. The axial force used to generate the $M-\phi$ for wall piers 1-4 is the design axial force calculated at level 1. The axial force used to generate the $M-\phi$ for wall piers 5-8 is the design axial force calculated at level 5. Likewise, the design axial force calculated at level 9 was used to generate the $M-\phi$ for wall piers 9-20.

Load reversals are expected during a seismic event and thus, an individual wall pier will act as a tension or a compression wall depending on the direction of loading. However, the modified Takeda hysteresis rule, as implemented in Ruaumoko, does not allow unsymmetrical properties, and only one set of parameters, the elastic and inelastic stiffness properties for the tension wall and compression wall moment-curvatures were

averaged. It is important to note that this method will underestimate the behavior of the compression wall and overestimate the behavior of the tension wall.

Table 6.9 and 6.10 summarize the results of averaging the tension wall and compression wall moment-curvature slopes for the SCB building and DBCB building respectively. To illustrate the procedure used to determine the effective moments of inertia shown in Tables 6.9 and 6.10, the following calculations for the effective moments of inertia and bilinear factors for the wall piers at floor levels 1-4 in the DBCB building are presented.

The first step is to calculate the transformed moment of inertia, EI_g . The concrete modulus of elasticity can be directly calculated and the gross moment of inertia is taken from the XTRACT model used for the design of the wall pier; I_g for these wall piers is $9.56m^4$.

$$E_c = 4697\sqrt{35} = 27,778MN / m^2$$

$$EI_g = (27,778MN / m^2)(9.56m^4) = 265,653MN - m^2$$

Next, the elastic slopes (EI_y) of the tension wall and compression wall are calculated.

$$EI_y(TW) = \frac{M_y}{\phi_y} = \frac{78.634MN - m}{0.001033(1/m)} = 76,122MN - m^2$$

$$EI_y(CW) = \frac{M_y}{\phi_y} = \frac{128.796MN - m}{0.000728(1/m)} = 176,918MN - m^2$$

As discussed previously, the average of the EIs of the tension wall and compression wall will be used since only one set of parameters can be given to a wall pier.

$$EI_{y,AVG} = \frac{76,122MN - m^2 + 176,918MN - m^2}{2} = 126,520MN - m^2$$

The effective moment of inertia for the wall is calculated as

$$I_e = \frac{EI_{y,AVG}}{E} = \frac{126,520MN - m^2}{27,778MN / m^2} = 4.55m^4$$

The moment of inertia for the base wall pier will be determined as the average of the ratios of EI_y/EI_g for the tension wall and compression wall.

$$\frac{EI_y(TW)}{EI_g} = \frac{76,122MN - m^2}{265,653MN - m^2} = 0.2865$$

$$\frac{EI_y(CW)}{EI_g} = \frac{176,918MN - m^2}{265,653MN - m^2} = 0.6659$$

$$\frac{EI_y}{EI_g}(AVG) = \frac{0.2865 + 0.6659}{2} = 0.476$$

This is the factor that is used to determine the effective moment of inertia for the base wall pier. Therefore, the moment of inertia used for the base wall pier is

$$I_e = (0.467)(4.55m^4) = 2.12m^4.$$

The last step needed to finish the calculations for the wall piers at floor levels 1-4 is to determine the bilinear factor. This is the parameter that defines the behavior of the wall beyond the yield point (the post-peak leg of the bilinear moment-curvature relationship). The post-peak slopes (EI_{SH}) of the tension wall and compression wall moment-curvatures are

$$EI_{SH}(TW) = \frac{\Delta M}{\Delta \phi} = \frac{10.222MN - m}{0.00365(1/m)} = 2,801MN - m^2$$

$$EI_{SH}(CW) = \frac{\Delta M}{\Delta \phi} = \frac{14.371MN - m}{0.00467(1/m)} = 3,077MN - m^2$$

The ratios of EI_{SH}/EI_g for the tension wall and compression wall are

$$\frac{EI_{SH}(TW)}{EI_g} = \frac{2,801MN - m^2}{265,653MN - m^2} = 0.0105$$

$$\frac{EI_{SH}(CW)}{EI_g} = \frac{3,077MN - m^2}{265,653MN - m^2} = 0.0116$$

The average of these two values will be taken as the bilinear factor for the wall piers at floor levels 1-4. Thus , the bilinear factor is

$$\frac{0.0105 + .0116}{2} = 0.0111 .$$

The parameters for the other wall piers in the DBCB building and for the wall piers in the SCB building are computed similarly.

6.3.2 Nonlinear Modeling

The Time-History Input Data module of the Ruaumoko Nonlinear Analysis software package was used to generate the nonlinear models for the two buildings being investigated. The core of the buildings were modeled as 2-D equivalent frames where the “columns” of the frame have the properties of the wall piers and are located at the centroids of the wall piers. Figure 6.20 shows the plan dimensions of the core, where the centroids of the walls are located, and a partial elevation of the equivalent frame.

The wall piers were modeled as general quadratic beam-column frame members. The Modified Takeda hysteresis rule was used to define the wall pier hysteresis, and hysteretic strength degradation in each direction was considered to be based on ductility. The Modified Takeda and hysteretic strength reduction parameters were determined based on engineering judgment. When determining the parameters for hysteretic strength reduction, the fact that the wall piers are unconfined (no boundary elements) was considered. The modified Takeda parameters (refer to Figure 6.7) for the wall piers are $\alpha=0$ and $\beta=0.6$. Figure 6.21 shows the hysteretic strength reduction used for the wall piers.

The measured force-deflection curves for the experimental beams were generated based on transverse forces imparted to the beams, and these curves represent combined

effects of shear, flexure, and beam/wall pier interaction. The yield forces assigned to Ruaumoko beam members represent yield forces associated with flexural hinging which is not representative of the measured F-d curve. For this reason, the coupling beams were modeled as spring elements (see Figure 6.22).

The spring element consists of a longitudinal, rotational, and transverse spring. The spring is modeled to have no interaction between the longitudinal, rotational, and transverse spring components. Sufficiently large stiffness and yield forces are assigned to the longitudinal and rotational springs to ensure rigid behavior. The hysteretic parameters determined for the steel and diagonally-reinforced coupling beams are assigned to the transverse spring. Thus, transverse loading governs the hysteretic behavior of the coupling beams. Figures 6.23-6.27 show partial elevations and the spring element parameters for both buildings for floor levels 1-4, 5-8, 9-12, 13-16, and 17-20, respectively (note that the tabulated beam parameters shown in the figures account for having two coupling beams in the equivalent frame). The rigid links on both sides of the coupling beam connecting the beams to the wall piers are modeled as sufficiently rigid beam-column frame members.

The dynamic weight of the structure was input as dynamic loads assigned to the nodes of the wall piers at each floor level. To distribute the dynamic weight of the structure, the total weight of the building was distributed evenly of the 20 floor levels. One-half of the dynamic weight in each floor level was assigned to the left and right wall piers. The nodal dynamic weights assigned are 2,126kN and 2,135kN for the SCB and DBCB buildings, respectively.

A static analysis of the gravity loads is performed prior to the time-history analysis so that residual forces due to gravity loads are considered in the solution of the nonlinear analysis. The gravity loads are assigned to the nodes of the wall piers at each floor level and represent the tributary gravity loads that each wall pier will have at each floor level. The static loads assigned to the node at each wall pier at each floor level are $1,343kN$ and $1,481kN$ for the SCB and DBCB buildings, respectively.

6.4 Nonlinear Analysis

As discussed in the opening of this chapter, each building was subjected to four different ground motions: (1) the North-South El Centro ground motion (1940), (2) the North-South Northridge ground motion (1994), (3) an artificial ground motion representing an ASCE7-02 10% in 50 years ground motion, and (4) an artificial ground motion representing an ASCE7-02 2% in 50 years ground motion. The input excitations are shown in Figures 6.28-6.31. The acceleration spectra for these ground motions are provided in Figure 6.32.

The analyses were based on Newmark inelastic dynamic time-history analyses with diagonal mass matrix mass models. The damping model chosen for the time-history analyses is the Rayleigh initial stiffness model. Small displacements were considered with simplified P-delta effects.

6.5 Analyses Results

The following structural demands and responses that were investigated for each ground motion.

- Base shears imparted to the buildings by the ground motions compared to the base shears determined by the elastic analyses.
- Beam shear demand and nominal capacity distributions over the height of the buildings and with respect to the time of the earthquake record.

- Demands on the wall piers compared to the failure surfaces defined by the axial-moment interaction curves.
- Maximum rotation demands on the coupling beams. These will be related to the beam rotation capacities and associated damage that was measured during the experimental phase.
- Interstory and roof drifts compared to the allowable 2% interstory and roof drift limits defined by NEHRP.

6.5.1 Base Shears

The maximum base shear demand for both buildings, and for all four ground motions, exceeded the code specified base shear. The El Centro earthquake generated base shear approximately 40% greater than the base shears estimated by the equivalent lateral force (ELF) method. In the worst case, the base shear demand in the Northridge earthquake is more than three times greater than the ELF base shear. A summary of the ELF base shears compared to the absolute maximum base shears imparted to the buildings by the ground motions is given in Table 6.11. As can be seen in this table, of the two recorded ground motions, the Northridge earthquake is more severe in terms of base shear than the El Centro earthquake. Of the two ASCE7-02 artificial records, the 2% in 50 years record is more severe in terms of base shear than that of the 10% in 50 years. Figures 6.33 and 6.34 show plots of the base shears over the course of the four events for the SCB and DBCB buildings, respectively.

The natural periods of the SCB building and DBCB building are 2.31sec and 2.63sec, respectively. Referring back to Figure 6.32, which shows the spectra for all of the ground motions, the Northridge spectrum has the highest acceleration of the four records for the periods of the two buildings, with the ASCE7-02 10% in 50 years record having the lowest. Thus, the relative level of base shear imparted to the buildings by each ground motion seems reasonable.

6.5.2 Coupling Beam Shear Demand

EL CENTRO EARTHQUAKE

The maximum base shear occurred at approximately 4sec into this earthquake record for both buildings. At this point of the event, none of the beams had yielded in the SCB building. The beams in the DBCB building at levels 3-20 are just at or beyond yield; the beam at level one is approximately 65% of yield and the beam at level 2 is approximately 90% of yield. Figures 6.35 and 6.36 show plots of the beam shear demands at the time of maximum base shear for the SCB and DBCB buildings, respectively for this event.

At approximately 6sec into the event, the beams in floors 7-20 of the SCB building are just at or beyond yield. The beams at floors 1-6 of the SCB building do not yield over the course of the event. For illustration, Figure 6.37 shows plots of the beam demands over the course of the event for floor levels 1-4 and 9-12. The beams in the DBCB building at floors 4-20 yield several times in the first 12 seconds of the event. The beams at floors 1-3 do not yield over the course of the event. For illustration, Figure 6.38 shows plots of the beam demands over the course of the event for floor levels 1-4 and 9-12.

NORTHRIDGE EARTHQUAKE

The maximum base shear occurred at approximately 4sec into this earthquake record for both buildings. At this point of the event, all of the beams had yielded in the both the SCB and DBCB buildings. Figures 6.35 and 6.36 show plots of the beam shear demands at the time of maximum base shear for the SCB and DBCB buildings, respectively for this event.

All of the beams in the SCB building yield several times over the first 10 seconds of this event. For illustration, Figure 6.39 shows plots of beam shear demands over the course of the event for floors 1-4 and 9-12. All of the beams in the DBCB building yield in the first 8 seconds of the record; the beams at floor levels 5-20 yield several times over this period of the record. For illustration, Figure 6.40 shows plots of the beam demands over the course of the event for floor levels 1-4 and 9-12.

ASCE7-02 10% in 50 YEARS ARTIFICIAL EARTHQUAKE

The maximum base shear occurred at approximately 7sec into this earthquake record for both buildings. At this point of the event, the beams in floors 5-13 are just at or beyond yield in the SCB building. The beams at floors 1-4 are approximately 65-70% of their yield. All of the beams in the DBCB building have yielded at this point in the event. Figures 6.35 and 6.36 show plots of the beam shear demands at the time of maximum base shear for the SCB and DBCB buildings, respectively for this event.

The beams at floors 5-12 in the SCB building are within 90-95% of their yield several times over the first 16 seconds of this event; the beams at floors 1-4 do not yield over the course of this event. For illustration, Figure 6.41 shows plots of beam shear demands over the course of the event for floors 1-4 and 9-12. The majority of the beams in the DBCB building yield several times in the first 16 seconds of the record. For illustration, Figure 6.42 shows plots of the beam demands over the course of the event for floor levels 1-4 and 9-12.

ASCE7-02 2% in 50 YEARS ARTIFICIAL EARTHQUAKE

The maximum base shear occurred at approximately 7sec into this earthquake record for both buildings. At this point of the event, the beams in floors 2-13 in the SCB

building have yielded. The beam at floor 1 is approximately 95% of yield. All of the beams in the DBCB building have yielded at this point in the event. Figures 6.35 and 6.36 show plots of the beam shear demands at the time of maximum base shear for the SCB and DBCB buildings, respectively for this event.

The majority of the beams in the SCB building yield several times over the first 16 seconds of this event. For illustration, Figure 6.43 shows plots of beam shear demands over the course of the event for floors 1-4 and 9-12. All of the beams in the DBCB building yield several times in the first 16 seconds of the record. For illustration, Figure 6.44 shows plots of the beam demands over the course of the event for floor levels 1-4 and 9-12.

6.5.3 Wall Pier Demands

EL CENTRO EARTHQUAKE

The axial force and flexural demands on the walls piers in both the SCB and DBCB buildings are relatively moderate. This behavior is reasonable considering that the El Centro earthquake is the most moderate in, terms of ground acceleration, of the four ground motions investigated. Figures 6.45 and 6.46 (SCB and DBCB, respectively) show the axial load-moment hysteretic behavior of the wall piers plotted onto the bottom portion of the axial force-moment interaction curves generated for the wall piers. As can be seen in the figures, the trace of the demands on the wall piers do not exceed the P-M failure envelope. Thus, the wall piers maintain their axial load-carrying capacities throughout the duration of the event.

NORTHRIDGE EARTHQUAKE

This earthquake placed the largest demand on the wall piers in both structures. Figures 6.47 and 6.48 (SCB and DBCB, respectively) show the axial load-moment hysteretic behavior of the wall piers plotted onto the bottom portion of the axial force-moment interaction curves generated for the wall piers. As can be seen in the figures, the trace of the demands on the wall piers do not exceed the P-M failure envelope. Thus, the wall piers maintain their axial load-carrying capacities throughout the duration of the event. However, the axial loads and moments imparted to the wall piers in the SCB building are near the failure surface defined by the P-M interaction envelope. In the case of the DBCB building, the axial load and moment demands on the wall piers is less severe relative to the defined failure surface. This, in part, is due to the larger wall overstrength factors used in the design of the wall piers in the DBCB building relative to the SCB building.

The flexural demands on the wall piers in both buildings are also significant indicating inelastic response. The axial load-moment hysteretic response indicates that plastic energy is being dissipated by the wall piers. The flexural demands on the wall pier is a result of the plastic hinges being formed in the ends of the coupling beams. Referring to Figures 6.35 and 6.36, the beams at all floor levels in both buildings have yielded several times over the duration of this event. The flexural demand on the wall piers indicates that the shear-carrying capacity of the coupling beams has been exhausted, the coupling effect diminished, and the redistribution of the overturning moments are resisted by wall pier flexure rather than a combination frame action and wall pier flexure, placing a larger flexural demand on the wall piers.

ASCE7-02 10% in 50 YEARS ARTIFICIAL EARTHQUAKE

This earthquake placed a larger axial demand on the wall piers in both structures relative to the El Centro earthquake. Figures 6.49 and 6.50 (SCB and DBCB, respectively) show the axial load-moment hysteretic behavior of the wall piers plotted onto the bottom portion of the axial force-moment interaction curves generated for the wall piers. As can be seen in the figures, the trace of the demands on the wall piers do not exceed the P-M failure envelope. Thus, the wall piers maintain their axial load-carrying capacities throughout the duration of the event.

The flexural demand on the wall piers in both buildings is moderate although, somewhat larger in the wall piers of the DBCB building compared to the SCB building. Referring to Figures 6.35 and 6.36, in the DBCB building, the majority of the beams are well beyond yield and their shear load-carrying ability has peaked. However, the beams in the SCB building still have reserve capacity. The somewhat larger flexural demands in the wall piers of the DBCB building are a result of the diminished coupling action due to the hinging of the coupling beams. Conversely, the coupling action of the SCB building has not been exhausted resulting in smaller flexural demands.

ASCE7-02 2% in 50 YEARS ARTIFICIAL EARTHQUAKE

This earthquake placed a larger axial demand on the wall piers in both structures relative to the El Centro earthquake. Figures 6.51 and 6.52 (SCB and DBCB, respectively) show the axial load-moment hysteretic behavior of the wall piers plotted inside bottom portion of the axial force-moment interaction curves generated for the wall piers. As can be seen in the figures, the trace of the demands on the wall piers do not exceed the P-M failure envelope. Thus, the wall piers maintain their axial load-carrying

capacities throughout the duration of the event. However, the demands are somewhat more severe in the wall piers of the DBCB building than that of the SCB building.

As can be seen in the hysteretic responses, the slopes of the axial load-moment hystereses in the wall piers of the SCB building are more steep relative to the DBCB building. Additionally, the wall piers in the DBCB building exhibit a larger degree of energy-dissipation than that of the wall piers in the SCB building. This indicates that the coupling action of the DBCB building has been diminished and the majority of the overturning moment is resisted through flexure of wall piers rather than a combination of frame action and wall flexure.

6.5.4 Interstory/Roof Drift and Beam Rotation Demands

NEHRP limits the design interstory drift and roof drift to 2% of the floor-to-floor height and overall height of a building, respectively. The floor-to-floor height of the buildings considered in these analyses is $2,743\text{mm}$ corresponding to a 55mm maximum allowable interstory. The overall height of the buildings is 55m corresponding to a $1,098\text{mm}$ maximum allowable roof drift.

Both buildings satisfy the 2% roof drift limit for all four of the ground motions considered. Table 6.12 summarizes the roof drifts, interstory drifts, and maximum beam rotation demands of both buildings for the four ground motions considered. The DBCB building roof drifts are 8-25% larger than the SCB building roof drifts with the largest difference occurring during the El Centro ground motion. The largest roof drifts in both buildings occurred during the ASCE7-02 2% in 50 years earthquake. The roof drifts during this event, for the SCB and DBCB building, are -520mm and -584mm , respectively, corresponding to 47% and 53% of the allowable maximum roof drift.

Figures 6.53 and 6.54 show plots of the drifts for the SCB and DBCB building, respectively. The interstory drifts shown in these figures correspond to the interstory drift at the time when the roof drift is maximum. Considering the larger flexural demands on the wall piers of the DBCB building relative to the SCB building, as discussed in the previous section, it is reasonable to expect that the DBCB building would experience larger lateral drifts relative to the SCB building.

Both buildings satisfy the 2% interstory drift limit for all four of the ground motions considered. The largest interstory drift in the SCB building is -35mm and occurs during both the Northridge (occurs at floor 15). The largest interstory drift in the DBCB building -46mm and occurs at floor 16 during the 2% in 50 years artificial ground record. The interstory drifts in the DBCB building are 27% to 48% greater than the drifts experienced by the SCB building.

The maximum beam rotations occur at the floor levels where the interstory drift is largest. Figure 6.55 shows a schematic of the relationship between interstory drift and beam rotation (Table 6.12 also summarizes the maximum rotations and the floors at which the rotations occurred). Beam rotations were determined considering the both the relative vertical displacement of the beam ends in addition to the nodal rotation. The maximum beam rotations occurred in the upper floors of both buildings. Of the two buildings, the beams of the DBCB building experienced the largest beam rotations. The largest of the beam rotations in the DBCB building is 2.6% and occurred at floor 16 during the ASCE7-02 2% in 50 years event. The largest beam rotation demand in the SCB building occurred during the same event at floor 16 and is 1.5%.

As discussed previously, beam rotation is directly related to interstory drift (refer to Figure 6.55). Referring to the figure, beam rotation increases as interstory drift increases. Hence, the larger beam rotations seen in the DBCB building as compared to the SCB building is reasonably expected due to the larger interstory drifts seen in the DBCB building as compared to the SCB building.

6.6 Discussion

The overall structural behavior of the SCB building is superior to the DBCB building in all of the categories presented in the previous sections. The superior stiffness of the steel coupling beam allows the structure to maintain a higher level of lateral stiffness at higher levels of base shear than that of the diagonally reinforced coupling beam. As the lateral stiffness due to frame action becomes exhausted, overturning moment is redistributed to wall pier flexure which directly leads to larger lateral drifts. Consequently, the SCB building is also superior to the DBCB building when considering lateral drift.

Larger lateral drifts produce larger beam rotations. The DBCB building experienced the largest lateral drifts and thus, experienced the largest beam rotations. As can be seen in Table 6.12, the maximum beam rotations in the DBCB building for the El Centro and ASCE7-02 10% in 50 years is 1% and 1.6 %, respectively. The level of damage to a diagonally reinforced coupling beam at 1.5% rotation is moderate (see Figure 6.56). At this level of rotation, the concrete cover of the beam is still intact with moderate diagonal tension cracking. Thus, from a beam damage perspective, the diagonally reinforced beam can be considered as a viable option for a coupled core wall system for the El Centro and 10% in 50 years events. However, the maximum beam

rotations in the DBCB building for the Northridge and ASCE7-02 2% in 50 years events were 2.6%. The experimentally measured beam chord rotation capacity suggests that the non ACI-compliant DBCB has a maximum rotation capacity of 3%.

At 3% rotation, the experimental data presented in Chapter 3 suggest that the cover of the diagonally reinforced coupling beam is lost. However, the main core of the beam is still intact (see Figure 6.57). Hence, from a beam damage perspective, this type of beam may be considered as a viable option for a coupled core wall system for this building considering the level of beam rotation demand imparted to the structure by the Northridge earthquake.

At 4% rotation, the core of the beam in the test specimen described in Chapter 3 has significantly degraded with the unconfined diagonal bars buckling in compression (see Figure 6.58). From a beam damage perspective, this beam is not considered a viable option for buildings where expected beam chord rotations exceed 4%. The largest beam chord rotation demand occurred during the ASCE7-02 2% in 50 years record and was 2.6%. Hence, considering beam damage, this type of beam can be considered as a viable beam for all four of the ground motions considered.

Addressing the drift requirements is difficult when the design is based on an elastic analysis. Drift is dependent on the overall lateral stiffness of the building. Lateral stiffness can be increased in a coupled core wall structure by providing stiff coupling beams. However, in an effort to minimize wall overstrength factors, the beams are proportioned over the height of the building such that different beam sizes are used to minimize the ratio of nominal shear capacity to shear demand. The less stiff beams in the upper floors directly leads to relatively larger interstory drifts.

To control drift, for example, it may be advantageous to provide the same size coupling beam over the entire height of the building. However, if an elastic analysis procedure is used, the wall overstrength factors would dramatically increase, thereby, increasing the design loads for which the wall piers would be designed. Additionally, the coupling beams over the height of the building may not yield simultaneously at a given level of base shear. In cases where drift limits cannot be met in an elastic design, it may be more appropriate to design based on an inelastic analysis where wall overstrength factors are not considered and the design of wall piers and beams are based directly on the demands determined from the inelastic analysis.

The beam rotations for the SCB building are moderate for all of the four ground motions with the largest demand of 1.5% during the Northridge earthquake. At 2% rotation, there is no observable damage to the beam. From a beam damage perspective, the steel coupling beam is considered to be a viable option for this building for all of the ground motions considered.

The axial load and flexural capacity of the wall piers are sufficient for both buildings and for all four of the ground motions considered. This is illustrated in the comparisons of the wall pier hysteretic responses to the axial load-moment interaction diagrams. However, it is important to consider that the hysteretic characteristics of the wall piers used to define the wall piers in the analytical models are based on averaged parameters; the hysteretic characteristics of the compression wall are underestimated but, more importantly, the hysteretic characteristics of the tension wall are overestimated.

The hystereses of the wall piers in both buildings approach the failure surface of the axial load-moment interaction failure surface in the positive-negative portion of the

graph during the Northridge and ASCE7-02 2% in 50 years events. This quadrant in the graph represents a combination of positive moment and negative axial force defining a tension wall. The sign convention defining positive wall pier moment is shown in the graphs of the P-M interaction surfaces shown in Figures 6.10-6.12. Although the demands near the wall pier capacities are brief, they correspond to peaks in the base shear demands. The effects of horizontal shear on wall piers experiencing axial tension are not considered in these analyses.

The design of the wall piers considered axial load and flexural capacity only; biaxial bending was considered. Recall that both of the load case considered (LC1 and LC2) included bidirectional earthquake loading as well as accidental torsion. Additionally, the decision not to provide special boundary elements was based on extreme fiber concrete compressive strains at design loads determined by an elastic analysis. The relatively moderate axial and flexural demands on the wall piers in the SCB and DBCB buildings imparted by the ASCE7-02 10% in 50 years event, which shows good correlation with the design response spectrum, suggest that the design of the wall piers based on an elastic analysis is reasonable. However, it is important to realize that the nonlinear analyses discussed in this chapter was two dimensional and therefore, only uniaxial bending (in addition to axial loads) is represented in the hysteretic demands shown in Figures 6.45-6.52.

In an effort to gain some level of understanding as to the how well the concrete compressive strains estimated at design loads determined from an elastic analysis compare to the concrete compressive strains at maximum axial loads and moments determined from a nonlinear analysis correlate, only uniaxial bending can be examined

since the nonlinear analysis is two dimensional resulting in uniaxial bending. Thus, the concrete compressive strains in the wall piers, at design loads determined from an elastic analysis, must be recalculated considering only axial forces and moments resulting from lateral loads acting in the coupled direction only (i.e., axial load plus moment about the y-axis of the wall pier). These concrete compressive strains will be compared to concrete compressive strains calculated at maximum axial forces and moments resulting from the nonlinear analysis.

Figure 6.59 shows plots of the hysteretic responses of the wall piers of the SCB building during the 10% in 50 years event plotted on the P-M diagram. The coordinates of the design axial forces and moments (uniaxial moment about the y-axis of the wall pier) are the design forces (resulting from elastic analysis) for which the extreme fiber concrete strains were calculated at Points A and C (refer to Figure 5.2 at the end of Chapter 5) for the compression wall and tension wall, respectively. Recall that the SCB building this is the same structure as Structure_01 presented in Chapter 5). As can be seen in the figure, the hysteretic response of the wall piers does not exceed the magnitudes of axial forces or moments for which the extreme fiber compressive strains were calculated. Hence, it is reasonable to assume that the strain capacity of the walls were not exceeded during this event, and the decision not to provide special boundary elements in the wall piers of this structure would be appropriate.

The axial and flexural demands on the wall piers during the Northridge and ASCE7-02 2% in 50 years events may impart significantly greater concrete compressive strains in the extreme fibers of the wall piers that exceed an acceptable strain capacity. Figure 6.60 shows the wall pier hysteretic response plotted on the P-M diagrams for the

Northridge Earthquake. The coordinates of the design axial forces and moments (uniaxial moment about the wall pier's y-axis) are the design forces (resulting from elastic analysis) for which the extreme fiber concrete strains were calculated at Points A and C (refer to Figure 5.2 at the end of Chapter 5) for the compression wall and tension wall, respectively.

Table 6.13 summarizes the axial forces and uniaxial moments determined from (a) elastic analysis) and (b) maximum axial force and uniaxial moment determined from nonlinear analysis. As can be seen in this table, the largest strain resulting from loads from the nonlinear analysis occurs in the base tension wall and is equal to 0.001132 which is well below the compressive strain capacity of 0.004. Hence, the compressive strain capacity of the wall piers was not exceeded even during ground motion records considered as extreme events relative to the design response spectrum. These resulting strains suggest that determining the need for special bound elements based on concrete compressive strains determined from an elastic analysis would be appropriate.

Table 6.1. Superimposed gravity loads.

Superimposed Gravity Loads		
Floor Loads		
MEP:	0.24	(kPa)
Columns:	0.24	(kPa)
Partitions:	0.48	(kPa)
Total	0.96	(kPa)
Line Loads		
Cladding	4.38	(kN/m)
Live Load		
	2.39	(kPa)

Table 6.2. Seismic response coefficients, dynamic weights, and base shears (both buildings).

Base Shear Summary			
Building	Siesmic Coefficient	Dynamic Weight (kN)	Base Shear (kN)
SCB	0.0682	85,099	5,805
DBCB	0.0682	85,410	5,828

Table 6.3. Lateral forces from ELF procedure (both buildings).

ELF Lateral Forces and Story Shears				
Floor Level	SCB Building		DBCB Building	
	F _x (kN)	V _x (kN)	F _x (kN)	V _x (kN)
20	667	667	669	669
19	619	1286	621	1290
18	573	1859	575	1865
17	528	2386	530	2395
16	484	2870	485	2880
15	441	3311	442	3322
14	399	3710	401	3723
13	359	4069	360	4083
12	320	4389	321	4404
11	282	4671	283	4687
10	246	4917	247	4934
9	212	5129	212	5147
8	179	5307	179	5326
7	147	5455	148	5474
6	118	5573	119	5593
5	91	5664	91	5684
4	66	5730	66	5750
3	44	5773	44	5794
2	24	5798	24	5819
1	9	5807	9	5828
ΣF_x	5807	ΣF_x	5828	

Table 6.4. Redundancy and wall overstrength factors (both buildings).

ρ and γ_{wall} Factors		
Structure	ρ	γ_{wall}
SCB	1.43	1.62
DBCB	1.50	2.12

Table 6.5. Beam design shears and factored capacities (both buildings).

SCB Building			DBCB Building		
Floor Levels	Design Shear (kN)	Factored Capacity (kN)	Floor Levels	Design Shear (kN)	Factored Capacity (kN)
17-20	445	540	17-20	285	330
13-16	753	811	13-16	480	483
9-12	1032	1081	9-12	677	707
5-8	1109	1151	5-8	1096	1119
1-4	995	1081	1-4	745	757

Table 6.6. Effective moment of inertia of DBCBs used in elastic analysis.

Effective DBCB Moment of Inertia			
Floor Level	I _g (in ⁴)	I _e (in ⁴)	Stiffness Reduction
20	11664	3929	0.3368
19	11664	3929	0.3368
18	11664	3929	0.3368
17	11664	3929	0.3368
16	11664	3929	0.3368
15	11664	3929	0.3368
14	11664	3929	0.3368
13	11664	3929	0.3368
12	16000	5197	0.3248
11	16000	5197	0.3248
10	16000	5197	0.3248
9	16000	5197	0.3248
8	27648	8294	0.3000
7	27648	8294	0.3000
6	27648	8294	0.3000
5	27648	8294	0.3000
4	21296	6655	0.3125
3	21296	6655	0.3125
2	21296	6655	0.3125
1	21296	6655	0.3125

Table 6.7. Wall pier design forces from elastic analysis.

Wall Pier Design Forces for SCB Building							
Floor Level	LC1				LC2		
	Pu		M _x (kN-m)	M _y (kN-m)	Pu (kN)	M _x (kN-m)	M _y (kN-m)
	Comp. (kN)	Tens. (kN)					
20	2747	-493	334	4033	1703	1113	522
19	5494	-986	968	6010	3424	3225	823
18	8241	-1479	1827	6434	5164	6089	932
17	10988	-1971	2845	6001	6925	9483	920
16	14336	-3065	3968	6202	8753	13226	974
15	17684	-4158	5155	6127	10602	17184	993
14	21032	-5251	6383	7129	12469	21276	1194
13	24380	-6344	7639	8404	14353	25463	1454
12	28329	-8038	8922	9291	16293	29739	1630
11	32278	-9731	10236	10198	18246	34119	1818
10	36227	-11424	11590	11156	20212	38634	2024
9	40176	-13118	12997	12216	22188	43324	2255
8	44281	-14967	14471	13331	24183	48235	2496
7	48387	-16817	16023	14724	26182	53410	2794
6	52492	-18666	17665	16533	28182	58884	3173
5	56597	-20516	19402	18945	30173	64674	3665
4	60546	-22210	21235	22375	32131	70784	4351
3	64495	-23903	23158	27128	34048	77193	5261
2	68444	-25596	25160	33912	35891	83868	6485
1	72392	-27290	27227	44022	37619	90755	8165
Wall Pier Design Forces for DBCB Building							
Floor Level	LC1				LC2		
	Pu		M _x (kN-m)	M _y (kN-m)	Pu (kN)	M _x (kN-m)	M _y (kN-m)
	Comp. (kN)	Tens. (kN)					
20	2572	-304	336	5306	1733	1118	592
19	5144	-608	972	8191	3478	3240	944
18	7716	-912	1835	9188	5235	6116	1096
17	10289	-1216	2857	9105	7008	9524	1125
16	13332	-1990	3984	9425	8834	13281	1203
15	16375	-2765	5176	9444	10675	17254	1244
14	19418	-3540	6408	10842	12527	21360	1469
13	22461	-4315	7668	12719	14388	25561	1767
12	26194	-5777	8955	14059	16311	29850	1980
11	29928	-7240	10273	15625	18237	34242	2227
10	33660	-8704	11631	17594	20160	38770	2531
9	37392	-10168	13042	20183	22078	43473	2924
8	42396	-12895	14519	20437	24131	48397	2997
7	47401	-15623	16076	21486	26172	53587	3172
6	52405	-18351	17723	23427	28200	59077	3468
5	57409	-21079	19466	26450	30211	64887	3910
4	61297	-22692	21306	32030	32129	71018	4762
3	65185	-24306	23236	39462	34005	77453	5850
2	69072	-25920	25246	49560	35814	84153	7242
1	72960	-27534	27321	63793	37521	91069	9041

Table 6.8. Wall longitudinal reinforcement based on elastic design (both buildings).

Pier Reinforcement Summary					
Structure	Floors	ρ (%)	Bar Qty.	Bar Size (#)	Bar Spacing (mm)
SCB	17-20	0.81	92	9	229
	13-16	0.81	92	9	229
	9-12	0.81	92	9	229
	5-8	1.26	92	11	229
	1-4	1.82	92	14	229
DBCB	17-20	1.04	118	9	205
	13-16	1.04	118	9	205
	9-12	1.04	118	9	205
	5-8	1.31	118	10	205
	1-4	2.33	118	14	205

Table 6.9. Elastic and inelastic EI slopes for all wall piers of the SCB building. See M- ϕ plots in Figures 6.14-6.16.

Elastic Moment-Curvature Slopes - SCB Model							
Wall Gross Moment of Inertia (considering steel)				Average Moment of Inertia			
Floor Levels	E _{c,wall} (kN/m ²)	I _g (m ⁴)	EI _g (kN-m ²)	EI (CW) (kN-m ²)	EI (TW) (kN-m ²)	EI (AVG) (kN-m ²)	I _{AVG} (m ⁴)
L1-4	27,787,983	9.26	257,345,510	169,425,366	46,488,668	107,957,017	3.89
L5-8	27,787,983	9.21	255,906,486	139,428,114	25,779,719	82,603,917	2.97
L9-20	27,787,983	9.16	254,467,462	105,309,021	20,475,489	62,892,255	2.26
Wall Pier Yield Coordinates for SCB Model							
	Compression Wall		Tension Wall		Level 1 Wall Pier		
Floor Levels	M _y (kN-m)	ϕ _y (1/m)	M _y (kN-m)	ϕ _y (1/m)	EI _y (AVG) (kN-m ²)	EI _y (AVG)/EI _g	
L1-4	111,172	0.000656	45,757	0.000984	107,957,017	0.420	
L5-8	93,547	0.000671	31,946	0.001239			
L9-20	67,788	0.000644	22,370	0.001093			
Wall Moment of Inertias used in Models							
	L1	L2-4	L5-8	L9-20			
	I _e (m ⁴)	I _e (m ⁴)	I _e (m ⁴)	I _e (m ⁴)			
	1.63	3.89	2.97	2.26			

Inelastic Moment-Curvature Slopes - SCB Model							
Floor Levels	Compression Wall		Tension Wall		Average of CW and TW		$EI_{SH,AVG}$
	ΔM (kN-m)	$\Delta \phi$ (1/m)	ΔM (kN-m)	$\Delta \phi$ (1/m)	ΔM_{AVG} (kN-m)	$\Delta \phi_{AVG}$ (1/m)	$\Delta M_{AVG}/\Delta \phi_{AVG}$ (kN-m ²)
L1-4	23,102	0.00414	9,748	0.00310	16,425	0.003622	4534704
L5-8	10,331	0.00757	4,077	0.00783	7,204	0.007698	935739
L9-20	6,792	0.01048	0	0.02553	3,396	0.018005	188620
Bilinear Factor							
Floor Levels	EI_g (kip-ft ²)	$EI_g/EI_{SH,AVG}$					
L1-4	257,345,510	0.0176					
L5-8	255,906,486	0.0037					
L9-20	254,467,462	0.0007					

Table 6.10. Elastic and inelastic EI slopes for all wall piers of the DBCB building. See M- ϕ plots in Figures 6.17-6.19.

Elastic Moment-Curvature Slopes - DBCB Model							
Wall Gross Moment of Inertia (considering steel)				Average Moment of Inertia			
Floor Levels	$E_{c,wall}$ (kN/m ²)	I_g (m ⁴)	EI_g (kN-m ²)	EI (CW) (kN-m ²)	EI (TW) (kN-m ²)	EI (AVG) (kN-m ²)	I_{AVG} (m ⁴)
L1-4	27,787,983	9.56	265,739,819	176,833,770	76,087,272	126,460,521	4.55
L5-8	27,787,983	9.36	259,983,722	188,906,331	33,567,914	111,237,123	4.00
L9-20	27,787,983	9.30	258,304,860	107,612,656	41,587,306	74,599,981	2.68
Wall Pier Yield Coordinates for DBCB Model							
		Compression Wall		Tension Wall		Level 1 Wall Pier	
Floor Levels		M_y (kN-m)	ϕ_y (1/m)	M_y (kN-m)	ϕ_y (1/m)	EI_y (AVG) (kN-m ²)	$EI_y(AVG)/EI_g$
L1-4		128,796	0.000728	78,634	0.001033	126,460,521	0.476
L5-8		86,768	0.000459	39,317	0.001171		
L9-20		67,788	0.000630	42,706	0.001027		
Wall Moment of Inertias used in Models							
		L1	L2-4	L5-8	L9-20		
		I_e (m ⁴)	I_e (m ⁴)	I_e (m ⁴)	I_e (m ⁴)		
		2.17	4.55	4.00	2.68		

Inelastic Moment-Curvature Slopes - DBCB Model							
		Compression Wall		Tension Wall		Average of CW and TW	
Floor Levels		ΔM (kN-m)	$\Delta \phi$ (1/m)	ΔM (kN-m)	$\Delta \phi$ (1/m)	ΔM_{AVG} (kN-m)	$\Delta \phi_{AVG}$ (1/m)
L1-4		14,371	0.00467	10,222	0.00365	12,297	0.00416
L5-8		15,035	0.00638	3,430	0.00700	9,233	0.00669
L9-20		10,331	0.01332	4,379	0.00583	7,355	0.00958
Bilinear Factor							
Floor Levels		EI_g (kip-ft ²)	$EI_g/EI_{SH,AVG}$				
L1-4		265,739,819	0.0111				
L5-8		259,983,722	0.0053				
L9-20		258,304,860	0.0030				

Table 6.11. Maximum base shears imparted to buildings during ground motions.

Base Shear				
SCB Building				
Ground Motion	ELF Base Shear (kN)	Maximum Base Shear (kN)	Max/ELF	Approx. Time Maximum Occurred (sec)
El Centro	5,805	8,184	1.41	4
Northridge	5,805	18,059	3.11	4
10% in 50	5,805	10,764	1.85	7
2% in 50	5,805	15,479	2.67	7
DBCB Building				
Ground Motion	ELF Base Shear (kN)	Maximum Base Shear (kN)	Max/ELF	Time Maximum Occurred (sec)
El Centro	5,827	8,429	1.45	4
Northridge	5,827	19,660	3.37	4
10% in 50	5,827	12,099	2.08	7
2% in 50	5,827	17,169	2.95	7

Table 6.12. Maximum Interstory drifts, roof drifts, and beam rotations imparted to buildings during ground motions.

Drift and Beam Rotation Demands							
Allowable Interstory Drift, δ_{ALL}		55	(mm)				
Allowable Roof Drift, Δ_{ALL}		1097	(mm)				
SCB Building							
Record	Interstory Drift δ (mm)	Roof Drift Δ (mm)	δ/δ_{ALL}	Δ/Δ_{ALL}	*Max. Rotation (%)	**Floor Level	Approx. Time Occurred (sec)
El Centro	-17	-259	0.31	0.24	0.6	14	5
Northridge	-35	-530	0.64	0.48	1.5	15	5
10% in 50	-26	-401	0.47	0.37	1.1	14	12
2% in 50	-31	-520	0.56	0.47	1.4	12	15
DBCB Building							
Record	Interstory Drift δ (mm)	Roof Drift Δ (mm)	δ/δ_{ALL}	Δ/Δ_{ALL}	*Max. Rotation (%)	**Floor Level	Approx. Time Occurred (sec)
El Centro	24	325	0.44	0.30	1.00	17	6
Northridge	45	575	0.82	0.52	2.60	17	4
10% in 50	-33	-460	0.60	0.42	1.60	16	15
2% in 50	-46	-584	0.84	0.53	2.60	16	15

*Maximum beam rotation is considered at the floor where interstory drift is maximum.

**The floor level given is the floor level where interstory drift is largest

Table 6.13. Comparison of wall pier concrete compressive strains (elastic analysis vs. nonlinear analysis).

Strain Comparison Under Uniaxial Loading						
	Compression Wall		Tension Wall		Conc. Comp. Strain	
	P (kN)	M_y (kN-m)	P (kN)	M_y (kN-m)	Point C (CW) (10⁻³)	Point A (TW) (10⁻³)
Level 1						
Elastic	72,387	-44,020	-27,288	44,020	-0.0430	-1.0500
Nonlinear	70,942	-110,583	-8,629	72,732	-1.0400	-1.1320
Level 5						
Elastic	56,592	-18,895	-20,514	18,895	-0.1540	-0.4520
Nonlinear	61,290	-43,728	-14,637	38,298	-0.4770	-0.7300
Level 9						
Elastic	48,570	-26,097	-17,346	9,237	0.1200	0.3630
Nonlinear	40,174	-12,215	-13,117	12,215	0.1390	0.3400

Figure 6.1. Building plan dimensions.

Figure 6.2. Final DBCB designs.

Figure 6.3. Beam demand and capacity distributions.

(a)

(b)

Figure 6.4. Beam hysteretic response comparison of (a) Ramberg-Osgood and (b) Al-Bermani hysteresis rules.

Bounding-Surface Hysteresis

Al-Bermani Hysteresis Model					
Hysteresis Parameters					
k_0 (kN/mm)	$F_y^{(+)}$ (kN)	$F_y^{(-)}$ (kN)	r	α	β
79	712	-578	0.1	0.2	0.2

Figure 6.5. Al-Bermani hysteresis rule. Parameters shown are for the experimental steel coupling beam.

Figure 6.6. Hysteresis correlation for the experimental steel coupling beam.

k_0 (kN/mm)	r	F_{y+} (kN)	F_{y-} (kN)	
53	0.01	489	-489	
Strength Degradation				
Ductility at Start	Ductility at Final	Final Strength equal 0.01	α	β
3	6.52	7.52	0.5	0.001
Power Factor and Unloading				
Power Factor N		Unload as		
1		1		

Figure 6.7. Modified Takeda hysteresis rule. Parameters shown are for the experimental diagonally-reinforced coupling beam.

Figure 6.8. Strength degradation for the diagonally-reinforced coupling beam.

Figure 6.9. Hysteresis correlation for the experimental diagonally-reinforced concrete beam.

Figure 6.10. P-M interaction for wall piers of floors 1-4 for SCB and DBCB buildings.

Figure 6.11. P-M interactions for wall piers of floors 5-8 for SCB and DBCB buildings.

Figure 6.12. P-M interactions for wall piers of floors 9-20 for SCB and DBCB buildings.

Quadratic Beam-Column Yield Interaction Surface

Quadratic Beam-Column Yield Surface Parameters - Wall Piers						
Floor Levels	PYC (kN)	P (PB) (kN)	B (MB) (kN-m)	C (PC) (kN)	C (MC) (kN-m)	PYT (kN)
SCB Model						
L1-4	-285,784	-152,566	178,146	-46,526	-160,114	67,788
L5-8	-279,290	-149,408	149,946	-67,876	-136,795	38,177
L9-20	-271,461	-150,476	141,811	-86,336	-131,277	29,370
DBCB Model						
L1-4	-291,566	-148,430	165,673	-54,844	-146,421	34,565
L5-8	-277,155	-145,894	150,082	-82,866	-135,711	34,428
L9-20	-265,546	-143,893	137,337	-92,696	-127,346	22,075

Figure 6.13. Quadratic beam-column yield interaction surface and critical point values for all wall piers for of the SCB and DBCB buildings.

Figure 6.14. Moment-curvature for TW and CW of SCB building at floors 1-4.

Figure 6.15. Moment-curvature for TW and CW of SCB building at floors 5-8.

Figure 6.16. Moment-curvature for TW and CW of SCB building at floors 9-20.

Figure 6.17. Moment-curvature for TW and CW of DBCB building at floors 1-4.

Figure 6.18. Moment-curvature for TW and CW of DBCB building at floors 5-8.

Figure 6.19. Moment-curvature for TW and CW of DBCB building at floors 9-20.

Figure 6.20. Equivalent frame used in nonlinear model.

Strength Degradation Rule For Wall Piers

Figure 6.21. Strength degradation assigned to wall piers in nonlinear model.

Figure 6.22. Ruaumoko spring member.

Spring Hysteresis Parameters - S1-4 Transverse Direction						
Model	F_y^+ kN	F_y^- kN	k_0^- kN/mm	r	α	β
SCB	2542	-2066	140	0.1	0.2	0.2
DBCB	2124	-2144	114	0.01	n/a	n/a

Longitudinal and rotational spring stiffness is infinitely large

Figure 6.23. Elevation of nonlinear model and spring parameters (L1-4).

Spring Hysteresis Parameters - S5-8 Transverse Direction						
Model	F_y^+ kN	F_y^- kN	k_0^- kN/mm	r	α	β
SCB	2706	-2200	150	0.1	0.2	0.2
DBCB	3142	-3142	168	0.01	n/a	n/a

Longitudinal and rotational spring stiffness is infinitely large

FLOORS 5-8

Figure 6.24. Elevation of nonlinear model and spring parameters (L5-8).

Spring Hysteresis Parameters - S9-12 Transverse Direction						
Model	F_y^+ kN	F_y^- kN	k_0^- kN/mm	r	α	β
SCB	2542	-2066	140	0.1	0.2	0.2
DBCB	1984	-1984	106	0.01	n/a	n/a

Longitudinal and rotational spring stiffness is infinitely large

Figure 6.25. Elevation of nonlinear model and spring parameters (L9-12).

Spring Hysteresis Parameters - S13-16 Transverse Direction						
Model	F_y^+ kN	F_y^- kN	k_0^- kN/mm	r	α	β
SCB	1906	-1548	106	0.1	0.2	0.2
DBCB	946	-946	50	0.01	n/a	n/a

Longitudinal and rotational spring stiffness is infinitely large

Figure 6.26. Elevation of nonlinear model and spring parameters (L13-16).

Spring Hysteresis Parameters - S17-20 Transverse Direction						
Model	F_y^+ kN	F_y^- kN	k_0^- kN/mm	r	α	β
SCB	1270	-1032	70	0.1	0.2	0.2
DBCB	928	-928	50	0.01	n/a	n/a

Longitudinal and rotational spring stiffness is infinitely large

Figure 6.27. Elevation of nonlinear model and spring parameters (L17-20).

Figure 6.28. El Centro ground motion record.

Figure 6.29. Northridge ground motion record.

Figure 6.30. ASCE7-02 10% in 50 years artificial ground motion record.

Figure 6.31. ASCE7-02 2% in 50 years artificial ground motion record.

Figure 6.32. Spectra for all ground motions.

Figure 6.33. Base shears for SCB building (all records).

Figure 6.34. Base shears for DBCB building (all records).

Figure 6.35. Beam shear distribution at time of maximum base shear (SCB building - all records).

Figure 6.36. Beam shear distribution at time of maximum base shear (DBCB building - all records).

Figure 6.37. Beam shear distribution vs. time (SCB building - EC record).

Figure 6.38. Beam shear distribution vs. time (DBCB building - EC record).

Figure 6.39. Beam shear distribution vs. time (SCB building - NR record).

Figure 6.40. Beam shear distribution vs. time (DBCB building - NR record).

Figure 6.41. Beam shear distribution vs. time (SCB building - 10% in 50 years record).

Figure 6.42. Beam shear distribution vs. time (DBCB building - 10% in 50 years record).

Figure 6.43. Beam shear distribution vs. time (SCB building - 2% in 50 years record).

Figure 6.44. Beam shear distribution vs. time (DBCB building - 2% in 50 years record).

Figure 6.45. Wall pier hysteretic responses at Levels 1, 5, and 9 (SCB building - EC record).

Figure 6.46. Wall pier hysteretic responses at levels 1, 5, and 9 (DBCB building - EC record).

Figure 6.47. Wall pier hysteretic responses at levels 1, 5, and 9 (SCB building - NR record).

Figure 6.48. Wall pier hysteric responses at levels 1, 5, and 9 (DBCB building - NR record).

Figure 6.49. Wall pier hysteretic responses at levels 1, 5, and 9 (SCB building – 10% in 50 years record).

Figure 6.50. Wall pier hysteretic responses at levels 1, 5, and 9 (DBCB building – 10% in 50 years record).

Figure 6.51. Wall pier hysteretic responses at levels 1, 5, and 9 (SCB building – 2% in 50 years record).

Figure 6.52. Wall pier hysteretic responses at levels 1, 5, and 9 (DBCB building – 2% in 50 years record).

Figure 6.53. Interstory and roof drifts (SCB building).

Figure 6.54. Interstory and roof drifts (DBC building).

Figure 6.55. Schematic of relationship between roof drift and beam rotation.

Figure 6.56. Damage to diagonally-reinforced beam at 1.5 % rotation.

Figure 6.57. Damage to diagonally reinforced beam at 3% rotation.

Figure 6.58. Damage to diagonally reinforced beam at 4% rotation.

Figure 6.59. Comparison of wall pier design axial forces and moments at which concrete compressive strains are determined from elastic analysis to wall pier axial forces and moments generated by 10% in 50 years record (SCB building).

Figure 6.60. Comparison of wall pier design axial forces and moments at which concrete compressive strains are determined from elastic analysis to wall pier axial forces and moments generated by Northridge record (SCB building).

Chapter 7: Conclusions and Recommendations

7.1 Summary

This research project focused on the performance of coupling beams with realistic span-to-depth ratios and the construction difficulties associated with the coupling beams and wall piers in coupled core wall (CCW) systems. To investigate the performance of coupling beams with realistic span-to-depth ratios, a diagonally-reinforced coupling beam (DBCB) with a span-to-depth ratio of 2.6, and a steel coupling beam with an equivalent depth to the DBCB, were tested under reverse cyclic loading. Additionally, a new type of steel coupling beam equipped with a central steel “fuse” section, intended to minimize post-earthquake repair and/or replacement, was designed and subsequently tested under reverse cyclic loading.

The investigation of the potential for minimizing the construction difficulties focused on two possibilities: (1) a new concrete/steel composite coupling beam consisting of a conventionally-reinforced concrete beam equipped with a vertical steel web plate (centrally located in the cross section of the beam) was designed and subsequently tested under reverse cyclic loading to investigate its potential as a viable alternative to the DBCB and (2) eliminating the special boundary elements generally found in wall piers of buildings in high seismic regions thereby, significantly minimizing the steel congestion at the wall pier/beam interface.

The experimental phase of the research involved testing five specimens which were approximately $\frac{1}{2}$ scale of a practical coupled core wall system. The test subassemblies consisted of two reinforced concrete wall piers connected by a coupling

beam. Four different types of coupling beams were tested: (1) a steel coupling beam (SCB) designed with well-established guidelines, (2) a diagonally reinforced coupling beam (DBCB) of which the transverse steel confining the diagonal bar groups does not satisfy ACI 318 provision, (3) a new shear plate coupling beam (SPCB) designed with a design methodology developed during this research specifically for this type of beam, and (4) a new steel coupling beam with a central fuse (FCB) designed with a design methodology developed during this research specifically for this type of beam; two different fuse capacities were tested to investigate the potential for replacement of the fuse. In all cases, the wall piers of the specimens had nominal wall reinforcement with no special boundary elements were provided.

The performance of the steel coupling beams was superior to the concrete coupling beam in stiffness, energy dissipation, and rotation capacity. Of the four different beam tested, the steel coupling beam (SCB) is superior. However, the FCB and DBCB performed relatively well exhibiting acceptable stiffness, energy dissipation, and beam rotation capacities. The SPCB failed to behave as designed because no significant composite action between the concrete and the steel plate was achieved. The less than acceptable performance of the SPCB is directly attributed to specimen construction errors that were not discovered until after testing had been completed. Chapter 2 presents the design procedures of the specimens and the results of the experimental testing phase of this project in detail.

In particular, the diagonally reinforced coupling beam presents several difficulties both in design and construction. Current ACI codes limit the nominal shear stress capacity of a DBCB to $0.83\sqrt{f'_c}$ which is a function only of the compressive strength of

the concrete rather than the strength of the diagonal bar groups for which the shear strength is determined. To evaluate the design of the DBCB, a parametric study was conducted to investigate the nominal shear stress levels at which a diagonally-reinforced beam is practically constructible. Four different L/h ratios (1, 2, 3, and 4) and 3 different levels of shear stress ($0.5\sqrt{f'_c}$, $0.83\sqrt{f'_c}$, and $1.2\sqrt{f'_c}$) were considered. The work of this study shows that nominal shear stress levels of $0.83\sqrt{f'_c}$ are not practically achievable due to the amount of diagonal steel and the angle of inclination of the diagonal bar groups required at this level of shear stress. Based, in part, on the results of the study, design guidelines as a function of the L/h ratio of the beam are presented. The particulars and results of this study are presented in detail in Chapter 4.

Chapter 5 provides details of a parametric study performed to investigate the potential of eliminating the special boundary elements (SBE) generally found in the wall piers of buildings located in high seismic regions. Eight structures with varying floor plan dimensions were analyzed and designed based on elastic analysis procedures. Specifically, the equivalent lateral force procedure was used to determine the design forces for which the wall piers and coupling beams would be designed. Steel coupling beams were used in all structures. Upon arriving at the design forces for the wall piers in the buildings, the need for special boundary elements in the wall piers was investigated using three different methods. Two of the methods investigated are ACI provisions: (1) establishing the need of SBE based on extreme fiber concrete compressive stress in the wall pier at design loads and (2) establishing the need of SBE based on the location of the neutral axis in the wall pier cross section at design loads. The third method is a method that proposes to establish the need for SBE based on the extreme fiber concrete strain in

the wall piers at design loads. Nonlinear material behavior is considered when determining the concrete strains.

The work in this study shows that the two ACI provisions for establishing the need for SBE in wall piers in coupled core wall systems are not appropriate. The ACI method based on extreme fiber concrete stress assumes gross section properties and a linear distribution of stress over the depth of the section. The magnitude of compressive stresses calculated in the tension walls of coupled core wall systems can be significantly underestimated using this method relative to the compressive stresses found from a nonlinear analysis of the wall pier. The significant axial forces generated in the wall piers of a CCW system increase the depth of the compression zone in the cross section of the wall pier relative to a wall pier subjected only to flexure. In the upper floors of a CCW structure, the normal tension stresses generated by flexure can be overcome by the normal compressive stresses generated by axial force resulting in a wall pier cross section in compression over the entire depth of the section. Hence, establishing the need of SBE in wall piers of CCW systems may result in providing SBE in the wall piers over the entire height of the building including the wall piers in the top floors of the structure where flexural and axial demands are reasonably expected to be moderate and at a minimum.

The Wall Boundary Element study presented in Chapter 5 proposes to establish the need for special boundary elements based on extreme fiber concrete compressive strains determined by basic principles incorporating nonlinear material behavior. The need for SBE would be established based on a critical strain limit defined by the strain capacity of unconfined reinforced concrete. It has been shown that reinforced concrete

wall piers experience no significant strength degradation and spalling at compressive strains at or below 0.004. The concrete compressive strains in the wall piers (at design loads) of the eight structures considered in this study were moderate and well below 0.004. Based on the strain capacity of reinforced concrete, special boundary elements would not be required.

To evaluate the overall structural behavior of the types of coupled core systems considered in this research, a series of dynamic nonlinear analyses were performed. Two buildings with the same plan and horizontal dimensions were investigated. One building was fitted with steel coupling beams and the other with diagonally-reinforced coupling beams. The design of the coupling beams and the wall piers are based on an elastic analysis. The building configuration chosen for the analyses is the same building as one of the structures in presented in Chapter 5 where it was shown that the compressive strains were moderate. Hence, the wall piers in the two selected buildings had no special boundary elements.

The properties of the coupling beams used in the nonlinear analyses were calibrated to reasonable replicate the hysteretic behavior of the beams measured during the experimental phase reported in Chapter 3. The properties of the wall piers were determined based on the P-M interactions and M- ϕ relationships of the wall piers designed for the buildings. Subsequent to finalizing the nonlinear models, four different ground motions were introduced to each of the structures. The resulting responses were then compared. The overall behavior of the building fitted with steel coupling beams exhibited superior performance in terms of overall lateral stiffness and energy dissipation in the coupling beams and performed adequately during all four of the ground motions

considered. The building fitted with diagonally reinforced coupling beams also performed acceptably for all four ground motions. The axial load-carrying capacity of the wall piers was not exceeded in either of the buildings during the four ground motions. It is important to note that the hysteretic responses of the wall piers during the two more extreme events exhibit axial tensile forces very near the tensile capacity of the wall piers. As was discussed in Chapter 6, an evaluation of the extreme fiber concrete compressive strains indicate that the strain capacity of the walls was not exceeded even during the two relatively extreme events. Also, an appreciable degree of inelastic response in the wall piers is seen during the two more extreme events in both buildings.

One of the ground motions has a spectrum that correlates very well with the design response spectrum for which the buildings was designed. During this event, extreme combinations of axial force and moment in the hysteretic response of the wall piers do not exceed the design axial forces and moments for which the extreme fiber concrete strains were calculated based on the elastic analyses. This correlation suggests that establishing the need for special boundary elements in the wall piers based on the concrete compressive strains at design loads (determined from elastic analyses) is reasonable.

7.2 Conclusions

1. Steel coupling beams exhibit excellent stiffness, energy dissipation, and beam rotation capacity. For beam rotation demands expected in practical core wall structures, damage to the beam is moderate.
2. Diagonally-reinforced concrete beams with non ACI-compliant transverse steel around the diagonal bar groups exhibit acceptable stiffness and energy

- dissipation. For beam rotation demands at or below 3% rotation, the confined core of the beam remains intact; thus, the damage to the beam is acceptable in this range of beam rotation demands. Construction difficulties are minimized due to less steel congestion over the beam's length.
3. Diagonally-reinforced concrete beams with non ACI-compliant transverse steel around the diagonal bar groups experience significant damage at beam rotation demands of 4% or greater. For coupled core wall systems where beam rotation demands are expected to be 4% or greater, this type of beam is not acceptable.
 4. ACI 318 currently limits the nominal shear capacity of a diagonally-reinforced concrete beam to $0.83\sqrt{f'_c}$ (f'_c in *MPa*). This level of shear stress is not achievable for practically-designed coupling beams for practical span-to-depth ratios, nor is the limit reasonable. The experimental data suggest that a shear force equivalent to $1.2\sqrt{f'_c}$ can be achieved while maintaining an acceptable degree of damage to the beam. Furthermore, the contribution of the stirrups and the compression strut (when the beam's axial growth is constrained) should be considered.
 5. Fuse steel coupling beams exhibit excellent stiffness, energy-dissipation, and beam rotation capacities. When proportioned appropriately, the web of the fuse section of the beam can be well beyond yield while the main section of the beam remains elastic. This beam promises great potential for reducing post-earthquake repair/replacement costs.
 6. The shear plate coupling beam failed to produce any reasonable degree of success more than likely due to construction errors. However, for this reason, the design

- methodology or performance of the beam can not be ascertained with a high degree of certainty.
7. Neither of the current ACI 318 provisions for establishing the need for special boundary elements in wall piers is appropriate for wall piers in coupled core wall systems. Establishing the need for special boundary elements based on the expected compressive strains at design loads and considering nonlinear material behavior has promising potential as a viable solution method.
 8. The wall overstrength factors applied to the design forces for wall piers determined from an elastic analysis are necessary in order to achieve the expected behavior of a core wall system. As was discussed in Chapter 6, the axial load-carrying capacity of a wall pier may very well be exceeded during extreme seismic events when wall overstrength factors are not considered in an elastic design.
 9. Attempts to minimize the magnitude of wall overstrength by providing different size coupling beams over the height of the building create difficulties in meeting interstory and roof drift limits.

7.3 Recommendations

1. The design methodology and performance of the proposed shear plate composite coupling beam has not been verified. Further experimentation is required to investigate the potential of this beam.
2. Although the fuse steel coupling beam performed adequately within a range of reasonably expected beam rotation demands, damage to the main section of the beam was significant at beam rotations beyond 4%. It is recommended that the

end returns be provided to the fillet welds at the flange/web connections to eliminate end effects at the terminus of the welds. The possibility of using full penetration groove welds in lieu of fillet welds is also recommended.

3. When testing the first fuse steel coupling beam, the wall piers were brought back to their original position before the first fuse section was removed and replaced with the second fuse. It is reasonable to expect that the wall piers in a real structure may experience permanent deformations during an earthquake. Further research is recommended to address the application of this type of beam to this circumstance.

References

ACI Committee 318, 2002. *ACI 318-02/ACI 318R-02 Building Code Requirements for Reinforced Concrete and Commentary*, American Concrete Institute. Farmington Hills MI.

AISC, 2002. *Seismic Provisions for Structural Steel Buildings (a supplement to AISC Manual of Steel Construction, 3rd Edition)*, American Institute of Steel Construction.

Aktan, A. E. and Bertero, V. V., 1981, The seismic resistant design of R/C coupled shear walls, Report No. UCB/EERC-81/07, *Earthquake Engineering Research Center*, University of California, Berkeley.

Aktan, A. E. and Bertero, V. V., 1984, Seismic response of R/C frame-wall structures, *ASCE Journal of the Structural Division*, **110** (ST 8), 1803-1821.

Aristizabal-Ochoa, J. D., 1982, Dynamic response of coupled wall systems, *ASCE Journal of the Structural Division*, **108** (ST 8), 1846-1857.

Aristizabal-Ochoa, J.D., 1987. Seismic Behavior of Slender Coupled Wall Systems, *ASCE Journal of the Structural Division*, Vol. 113, No. ST 10, pp 2221-2234.

Barney, G. B., Shiu, K. N., Rabbat, B. G., Fiorato, A. E., Russell, H. G., and Corley, W. G., 1978, *Earthquake Resistant Structural Walls – Test of Coupling Beams*, Report to the NSF, submitted by Portland Cement Association, Research and Development, Skokie, IL.

Barney, G.G. et al, 1980. *Behavior of Coupling Beams under Load Reversals* Portland Cement Association, Research and Development, Skokie, IL.

Berg, V.B. and Stratta, J.L., 1964. *Anchorage and the Alaska Earthquake of March 27, 1964*, AISC, New York, 63 pp.

Canadian Standards Association (CSA), 1994. *CSA A23.3-94 Design of Concrete Structures*, Rexdale, Canada. (reaffirmed 2000)

ETABS Nonlinear version 8.4.5, copyright 1984-2004. Computers and Structures, Inc. Computer Program.

Fintel, M. and Ghosh, S. K., 1982, Case study of aseismic design of a 16-story coupled wall using inelastic dynamic analysis, *ACI Structural Journal*, **79** (3), 171-179.

Fortney, P. J., Shahrooz, B. M., Rassati, G. A., 2004, The Next Generation of Coupling Beams, *Composite Construction V*, July 18-23, 2004, Kruger National Park, South Africa. (submitted for review).

Fortney P.J., Rassati G.A., Shahrooz B.M., Clemente I., Noé S., 2004. Cyclic test on steel plate reinforced coupling beam, *Proceedings of Atti del VI Workshop Italiano sulle Strutture Composte*, Trieste, Italy (in press).

Galano, L., and Vignoli, A., 2000, Behavior of Short Coupling Beams with Different Reinforcement Layouts, *ACI Structural Journal*, Nov-Dec 2000, 876-885.

Gong, B., Shahrooz, B. M., 2001a, Concrete-steel Composite Coupling Beams. I: Component Testing, *Journal of Structural Engineering*, June 2001, 625-631.

Gong, B., Shahrooz, B. M., 2001b, Concrete-Steel Composite Coupling Beams. II: Subassembly Testing and Design Verification, *Journal of Structural Engineering*, June 2001, 632-638.

Headed Reinforcement Corporation (HRC) 2004 *Headed Reinforcement* Sales brochure material and personal correspondence with HRC engineer, November 22, 2004.

Harries, K. A., Mitchell, D., Cook, W. D., and Redwood, R.G., 1993, Seismic Response of Steel Beams Coupling Concrete Walls, *ASCE Journal of Structural Engineering*, **119** (12), 3611-3629.

Harries, K. A., Cook, W. D., Mitchell, D., and Redwood, R.G., 1995, The Use of Steel Beams to Couple Concrete Walls, *Proceedings of the Seventh Canadian Conference on Earthquake Engineering*, Montreal, Canada, 509-516.

Harries, K.A., Mitchell, D., Redwood, R.G., and Cook, W.D. 1997. Seismic Design of Coupling Beams – A Case for Mixed Construction, *Canadian Journal of Civil Engineering*, Vol. 24, No.3, pp 448-459.

Harries, K. A., Gong B., and Shahrooz, B. M., 2000, Behavior and design of Reinforced Concrete, Steel, and Steel-Concrete Coupling Beams, *Earthquake Spectra*, 16 (4), 775-799.

Harries, K. A., 2001, Ductility and Deformability of Coupling Beams in Reinforced Concrete Coupled Walls, *Earthquake Spectra*, 17(3), 457-478.

Harries, K.A., Shahrooz, B.M., Brienens, P., Fortney P.J., Rassati, G.A., 2004. Performance-Based Design of Coupled Core Wall Systems, *Composite Construction in Steel and Concrete V* (under review).

Harries, K.A., Shahrooz, B.M. and Brien, P., 2004. Performance Based Design of Coupled Walls. *Proceedings of the 5th International Conference on Composite Construction*, July 2004, South Africa. (in press).

Harries, K. A., Fortney, P. J., Shahrooz, B. M., and Brien, P., 2004, Design of Practical Diagonally Reinforced Coupling Beams – A Critical Review of ACI 318 Requirements, *ACI Structures Journal*. (under review).

Harries, K.A. and McNiece, D.S., 2005. Performance-Based Design of High-Rise Coupled Wall Systems, *ASCE Journal of the Structural Division* (in press).

Harries, K.A. and Shahrooz, B., 2005. Hybrid Coupled Wall Systems, *Concrete International*. May 2005 (scheduled)

Hindi, R.A., 2001. *A Proposed Damage Model for R/C Bridge Elements Under Cyclic Loading*. Ph.D. dissertation, University of British Columbia.

ICC, 2003. *International Building Code 2003*, International Codes Council.

Kasai, K., and Popov, E. P., 1986, A study of seismically resistant eccentrically braced frames, *Report No. UCB/EERC-86/01*, Earthquake Engineering Research Center, University of California, Berkeley.

Kurama, Y., and Shen, Q., 2004. Post-Tensioned Hybrid Coupled Walls Under Lateral Load, *ASCE Journal of Structural Engineering*. Vol. 130, No. 2, pp 297-309.

Malley, J. O., and Popov, E. P., 1983, Design of links and beam to column connections for eccentrically braced frames, *Report No. UCB/EERC-83/03*, Earthquake Engineering Research Center, University of California, Berkeley.

Mitchell, D., 2004. Report to ACI Subcommittee 318-H – Seismic Provisions, San Francisco, October 25, 2004

New Zealand Standards Association (NZS), 1995. *NZS 3101:1995 Concrete Structures Standard*.

Park, R. and Paulay, T., 1975, *Reinforced Concrete Structures*, John Wiley and Sons, Inc., New York.

Paulay, T., 1971. Coupling Beams of Reinforced Concrete Shear Walls, *ASCE Journal of the Structural Division*, Vol. 97, No. ST 3, pp 843-862.

Paulay, T., and Binney, J. R., 1974, Diagonally Reinforced Concrete Beams for Shear Walls, *ACI Special Publications SP 42 – Shear in Reinforced Concrete*, 579-598.

Paulay, T., 1986. The Design of Ductile Reinforced Concrete Structural Walls for Earthquake Resistance, *Earthquake Spectra*, Vol. 2, No. 4, pp 783-823.

Paulay, T., and Priestly, M.J.N., 1992. Seismic Design of Reinforced Concrete and Masonry Buildings, Wiley Interscience.

Roeder, C. W., and Popov, E. P., 1978, Eccentrically braced steel frames for earthquakes, *ASCE Journal of the Structural Division*, 104 (3), 391-411.

Ruaumoko Nonlinear Analysis version 2.6, copyright March 2000. University of Canterbury, Department of Civil Engineering. Computer Program.

Shahrooz, B. M., Remmetter, M. E., and Qin, F., 1992, Seismic Response of Composite Coupled Structural Walls, *Proceedings of the International Conference on Concrete*, 465-481.

Shahrooz, B. M., Remmetter, M. E., and Qin, F., 1993, Seismic Design and Performance of Composite Coupled Walls, *ASCE Journal of the Structural Division*, 119 (11), 2858-2896.

Shahrooz B.M., Gong B., Tunc G., Deason J.D., 2001. An Overview of Reinforced Concrete Core Wall-Steel Frame Hybrid Structures, *Progress in Structural Engineering and Materials*, Vol. 3, No. 2, pp. 149-158.

Shahrooz, B.M., Fortney, P.J. and Rassati, G.A.. 2003. Seismic Performance of Hybrid Core wall Buildings, *Proceedings of the International Workshop on Steel and Concrete Composite Construction*, Taipei. pp 79-88.

Shiu, N. K., Barney, G. B., Fiorato, A. E. and Corley, W. G., 1978. Reversed Load Tests of Reinforced Concrete Coupling Beams, *Proceedings of the Central American Conference on Earthquake Engineering*, El Salvador, pp 239-249.

Shui, N. K., Barney, G. B., Fiorato, A. E. and Corley, W. G., 1981, *Earthquake Resistant Walls-Coupled Wall Test*, Report to NSF submitted by Portland Cement Association, Research and Development, Skokie, Il.

Subedi, N. K., 1989, Reinforced concrete beams with plate reinforcement for shear, *Proc. Instn. Civ. Engrs.*, Part 2 (87), 377-399.

Tassios, T. P., Moretti, M., and Bezas, A., 1996. On the coupling behavior and ductility of reinforced concrete coupling beams of shear walls, *ACI Structural Journal* Vol. 93, No. 6, pp 711-720.

Wallace J.W., 1994. New Methodology for Seismic Design of RC Shear , *ASCE Journal of the Structural Division*, Vol. 120, No. 3, pp 863-884.

Xtract-cross sectional X sTRuctural Analysis of ComponenTs version 2.6.2, copyright 2002. Imbsen and Associates, Inc. Computer Program.

APPENDIX A: Coupling Beam Damage Records and Miscellaneous Test Measurements

Damage Record for Test SCB

Test 13: $\pm 914.4\text{-mm}$ (0.36-inch) – 1% Shear Angle

At the completion of test 13D, a hairline crack approximately 76-mm long was observed. The crack is located on the West end of the East wall pier; it initiates at the top of the coupling beam's top flange and travels upward.

Test 14 - $\pm 13.7\text{-mm}$ (0.54-inch) – 1.5% Shear Angle

Crack documented at end of cycle 13D has now propagated further upward an additional 60-mm. Crack width remains approximately 1-mm wide along entire length of crack.

At East end of the South face of the West wall pier, a crack is observed subsequent to the completion of test 14D. The crack is half-moon shaped on the face of the wall and extends horizontally across the East end of the wall pier. The crack width is approximately 1-mm wide.

Test 15 - ± 10.3 -mm (0.72-inch) – 2% Shear Angle

At the completion of test 15D, a crack was observed at the bottom side of the flange of the coupling beam on the East end of the West wall pier. The crack is vertical, originates at the horizontal crack observed at the end of test 14D and extends vertically up to the bottom side of the bottom flange of the beam. The crack width is approximately 1-mm wide.

Test 16 - ± 27.4 -mm (1.08-inch) – 3% Shear Angle

At the completion of test 16C, five new cracks were observed. The crack width of all the previously-documented cracks have not increased at this point with the exception of the half-moon shaped crack on the South face of the West wall (first documented at the end of test 14D). The crack width of this crack has increased to approximately 3-mm.

1. The half-moon shaped crack on the South face of the West wall pier documented at the end of test 14D extended upward an additional 101-mm and terminated at the corner of the wall at approximately 250-mm from the top of the beam's top flange.
2. A 203-mm long vertical crack originates at the top of the beam's top flange at the East end of the West wall pier. The crack width is approximately 1-mm wide.
3. A 100-mm vertical crack is observed on the East face of the West wall pier. The crack originates at 230-mm above the top of the beam's top flange and moves upward at an angle to the corner of the wall. The crack width is approximately 1-mm wide.
4. Two vertical cracks on the West end of the East wall pier are also observed. One originates at the top of the beam's top flange and extends approximately 100-mm upward. The second originates at the bottom of the beam's bottom flange and extends approximately 100-mm downward. Both cracks are approximately 1-mm wide.

Test 17 - ± 36.6 -mm (1.44-inch) – 4% Shear Angle

Several vertical cracks were observed at the top of both faces of the wall piers at the subsequent to the completion of test 17. The length of the vertical cracks vary from 100-mm to 600-mm and have a crack width of approximately 1-mm.

The following photos illustrate the final cracked condition of the specimen at the completion of all testing.

Damage Record for Test DBCB

Test 9: $\pm 6.9\text{-mm}$ (0.27-inch) – 0.75% Shear Angle

At the completion of test 9, a diagonal tension crack was observed at midspan of the north face of the coupling beam. The crack is approximately 230-mm long and is 1-mm wide.

Test 10: $\pm 9.2\text{-mm}$ (0.36-inch) – 1.0% Shear Angle

At the completion of test 10D, several diagonal tension cracks are observed on both faces of the coupling beam. Diagonal tension cracks are evident for loading in both directions forming crisscrossing crack patterns. All crack widths are approximately 1-mm wide.

Test 11: $\pm 13.7\text{-mm}$ (0.54-inch) – 1.5% Shear Angle

At the completion of test 11D, diagonal tension cracks are observed on both faces of the coupling beam extending along the entire length of the beam. Diagonal tension cracks are evident for loading in both directions forming crisscrossing crack patterns. All crack widths are approximately 1-mm wide.

Test 12: $\pm 18.3\text{-mm}$ (0.72-inch) – 2.0% Shear Angle

At the completion of test 12D, crisscrossing diagonal tension cracks are formed in dense patterns on both sides of the coupling beam. Crack widths have increased to 5-mm in some cases. Spalling of the concrete cover has occurred at the bottom side of the West end of the beam. At this point, no cracking is evident in the wall piers.

Test 13: $\pm 27.4\text{-mm}$ (1.08-inch) – 3.0% Shear Angle

At the completion of test 13D, side cover on both faces of the coupling beam is loosely attached and ready to fall off of the beam. At this point, all loose concrete is removed in order to protect measuring equipment from falling concrete in following tests.

North and South faces of coupling beam after loose concrete has been removed. A portion of bottom cover has also been removed. Note that the confined core of the beam appears to be intact.

Test 14: $\pm 36.6\text{-mm}$ (1.44-inch) – 4.0% Shear Angle

At the completion of test 14D, confined core of beam degraded severely. Longitudinal and diagonal bars are exposed. During test cycle, diagonal bars in compression buckle enough to come in to contact with transverse reinforcement, deforming the transverse bars. Diagonal bars return to straight while in tension. Beam ends remain in tact at this point forming half-diamond shapes on both faces of beam indicating that connection region at the beam/wall interface is still structurally sound.

Test 15: $\pm 45.7\text{-mm}$ (1.80-inch) – 5.0% Shear Angle

At the completion of test 15, core of beam has degraded further. Diagonal bars have permanent deformation due to buckling during loading.

Test 16: $\pm 54.9\text{-mm}$ (2.16-inch) – 6.0% Shear Angle

Beam has completely degraded by the end of test 16. Most of the core of the beam is gone almost completely exposing longitudinal and diagonal bars. Note that at the beam ends, half-diamond shaped patterns exist where the beam is still in tact indicating that the wall/beam connection region is still structurally sound.

Condition of beam at end of all testing after all loose concrete has been removed.

Damage Record for Test SPCB

Test 7: $\pm 2.3\text{-mm}$ (0.09-inch) – 0.25% Shear Angle

At the completion of test 7D, a diagonal tension crack approximately 250-mm long was observed on the South face of the coupling beam to the East side of midspan of the beam. The crack width is 1-mm wide.

Test 8: $\pm 4.6\text{-mm}$ (0.18-inch) – 0.5% Shear Angle

At the completion of test 8D, two diagonal tension cracks were observed on the North face of the coupling beam; one crack to the East side of midspan approximately 200-mm long, the second crack is to the West side of midspan of the beam and is approximately 180-mm long. A 180-mm long diagonal tension crack was also observed on the South face of the coupling beam near midspan. All cracks observed at this point are approximately 1-mm long.

Test 9: $\pm 6.9\text{-mm}$ (0.27-inch) – 0.75% Shear Angle

At the completion of test 9, one additional diagonal tension crack was observed on the South face of the coupling beam; several additional diagonal tension cracks were observed on the North face of the beam creating crisscrossing patterns. The new cracks are predominantly extensions of previously documented cracks. At this point, crack widths have not increased beyond 1-mm long.

Test 10: ± 9.1 -mm (0.36-inch) – 1.0% Shear Angle

Several more diagonal tension cracks were observed on both faces of the coupling beam at the completion of test 10; the majority of which are located on the North face of the beam. Some crack widths have increased to approximately 2-mm wide.

Test 11: ± 13.7 -mm (0.54-inch) – 1.5% Shear Angle

At the completion of test 11, dense crisscrossing patterns of diagonal tension cracks were observed on both faces of the coupling beam. Some crack widths have increased to 3-4-mm wide. Minor spalling/flaking of concrete was also observed at the interface of the beam/wall connection at the top and bottom sides of the beam.

Test 12: $\pm 18.3\text{-mm}$ (0.72-inch) – 2.0% Shear Angle

At the completion of test 12, concrete cover on both sides of the beam is loose and ready to fall off. Significant spalling is evident on the underside of the beam on the West beam end at the beam/wall interface partially exposing the bottom longitudinal bar. Several cracks have widened and have increased to 5-12-mm wide. Transverse reinforcement is partially exposed in one area at midspan of the beam on the North face. At this point, no cracking is evident in the wall piers.

Test 13: $\pm 27.4\text{-mm}$ (1.08-inch) – 3.0% Shear Angle

At the completion of test 13, a significant amount of side cover has been lost exposing the longitudinal and transverse reinforcement. Some top and bottom cover has also been displaced. The confined core of the beam shows no evidence of degradation. No evidence of cracking in the wall piers is evident.

Test 14: $\pm 36.6\text{-mm}$ (1.44-inch) – 4.0% Shear Angle

At the completion of test 14, the confined core of the beam is beginning to degrade. The longitudinal bars, top and bottom, are all but exposed. The majority of damage at this point is in the form of concrete spalling at the beam ends near the beam/wall pier connection. However, no cracks are evident in the walls piers and beam cracks have not propagated into the wall piers.

Test 15: $\pm 45.7\text{-mm}$ (1.80-inch) – 5.0% Shear Angle

At the completion of test 15, the core of the beam has degraded further, exposing the vertical web shear plate. There is evidence of the steel shear plating sliding relative to the concrete.

Test 16: ± 54.9 -mm (2.16-inch) – 6.0% Shear Angle

At the completion of test 16, the bottom side of the steel web plate is exposed along the entire length of the beam. More evidence of the steel plate sliding relative to the concrete is observed. The core is degraded further; it was observed during loading of test 16, that the concrete core was no longer contributing to the resistance of the load. It was decided that all testing would stop at this point.

Damage Record for Test FCB-1

Test 13: $\pm 27.4\text{-mm}$ (1.08-inch) – 3.0% Shear Angle

At the completion of test 13, three hairline cracks in the wall piers were observed; (1) a 60-mm long crack originating at the corner of the top flange of the beam at the West end of the East wall pier and extending approximately horizontally to the corner of the wall pier, (2) a 200-mm long crack on the North face of the East wall pier at the West beam end, and (3) a 100-mm long vertical crack in the East end of the West wall pier originating at the top of the beam's top flange.

Test 14: $\pm 36.6\text{-mm}$ (1.44-inch) – 4.0% Shear Angle

At the completion of test 14, evidence of web splice connection slippage was observed. A 0.127-mm (0.005-in) feeler gage was used to check the connection. The feeler gage was refused. No additional cracks in the wall piers were observed.

Test 15: ± 45.7 -mm (1.80-inch) – 5.0% Shear Angle

At the completion of test 15A, the initiation of a weld fracture at the web/bottom flange connection in the end of the East main beam section was observed. By the end of test 15D, length of the weld fracture had increased to 90-mm long.

At the completion of test 15A, a weld fracture approximately 60-mm long was observed at the top flange/web connection of the East main beam section. By the end of test 15D, the fracture had increased to approximately 100-mm long.

At the completion of test 15D, clear signs of web yielding trajectories in the webs of the main beam sections as well as the fuse. Both web splice plates showed evidence of slipping.

The top flange splice plate of the East main beam section has permanent deformation.

At the completion of test 15D, several more cracks in the east and west wall piers were observed at the ends and faces of the wall piers near the beam/wall interface. The crack widths were approximately 1-mm wide.

Test 16: $\pm 54.9\text{-mm}$ (2.16-inch) – 6.0% Shear Angle

At the completion of test 16, the East end of the fuse web is moving relative to the web splice plate and bearing failure is evident in the web of the fuse section. The web splice connection at this location has failed.

At the end of test 16, permanent deformation in the top and bottom flanges of the East main beam section is evident.

Note that in this photo, the web bearing failure in the web of the fuse cannot be seen because the web splice plate has moved upward relative to the web of the fuse, hiding the damage.

Weld fractures at the top and bottom flange/web connections, of both main beam sections have increased to 100-mm long.

Test 17: $\pm 64\text{-mm}$ (2.52-inch) – 7.0% Shear Angle

At the completion of test 17, the weld fractures at the flange/web connections of the main beam sections have increased to 130-mm long.

The bottom bolt hole in the East end of the web of the fuse has failed in bearing; more evidence that this connection has failed.

Evidence of permanent deformations are obvious over the entire length and depth of the beam. The top and bottom flanges of all three sections have permanent deformations, as well as the webs of all three sections.

Additional cracks have formed on the South face of the West wall pier. At this point, all cracks widths are approximately 1-mm wide.

Test 18: ± 73.2 -mm (2.88-inch) – 8.0% Shear Angle

At the completion of test 18, complete bolt hole failure is evident in the top bolt hole at the East end of the fuse section.

This photo is taken during test 18 when the load wall has moved upward to peak displacement. The bottom bolt in the East end of the fuse has torn through the web of the fuse.

The weld fracture in the main beam section has increased to 150-mm long.

The degree of permanent deformations over the length and depth of the beam has significantly increased.

Test 19: $\pm 82.3\text{-mm}$ (3.24-inch) – 9.0% Shear Angle

At the completion of test 19, the beam has been all but torn apart. Weld fractures in the main beam sections have increased to 180-mm long and the permanent deformations are significant. At this point during loading, the weld fractures are just opening and closing, although, the system is still carrying some load.

Test 20: $\pm 91.4\text{-mm}$ (3.60-inch) – 10.0% Shear Angle

At the completion of test 20, the flange of the East main beam section has unzipped from the web nearly to the wall pier.

The web of the fuse exhibits severe damage at the bolt holes.

The bottom bolt at the East end of the fuse web splice connection has sheared off.

Test 21: $\pm 100.5\text{-mm}$ (3.96-inch) – 11.0% Shear Angle

All testing was stopped after the completion of test 21. This photo depicts the final condition of the beam. Damage at this point is severe. However, it should be noted that at this point, the beam was still carrying load.

Figure A.1. Load wall rotation during SCB test.

Figure A.2. Load wall rotation during DBCB test.

Figure A.3. Load wall rotation during SPCB test.

Figure A.4. Load wall rotation during FCB test (50% fuse).

Figure A.5. Load wall rotation during FCB test (70% fuse).

Appendix B: Example Calculations Using the ELF Procedure

Calculation of Dynamic Weight of Structure

Structure No. 1
 No. of Stories 20
 Fl/Fl Height 9 (ft)
 Plate Size 80 x 80 (ft x ft)
 Core Size 30 x 19.5 (ft x ft)
 Wall Thickness 24 (in)

Core Opening 403 (ft²)
 Perimeter Length 320 (ft)

Material Densities		
Material Type	Weight (kips/ft ³)	Mass (kips*s ² /ft ⁴)
Steel	0.495	0.015372671
Concrete	0.150	0.004658385

Plate Size			Plate Weight and Mass (per floor)		
Direction	Length (ft)	Area/Vol (ft ² , ft ³)		Weight (kips)	Mass (kips*s ² /ft)
x	80	5997	Total	525	16
y	80	3498			
z	0.5833				

Core Wall Properties and Weights								
Floor Level	Story Height (ft)	Flange Length (ft)	t _f (ft)	Web Length (ft)	t _w (ft)	No. of Walls per Floor	Wall Weight per Floor (kips)	Acc. Weight (kips)
20	9	15.500	2.000	12.000	2.000	2	213	213
19	9	15.500	2.000	12.000	2.000	2	213	427
18	9	15.500	2.000	12.000	2.000	2	213	640
17	9	15.500	2.000	12.000	2.000	2	213	853
16	9	15.500	2.000	12.000	2.000	2	213	1067
15	9	15.500	2.000	12.000	2.000	2	213	1280
14	9	15.500	2.000	12.000	2.000	2	213	1493
13	9	15.500	2.000	12.000	2.000	2	213	1706
12	9	15.500	2.000	12.000	2.000	2	213	1920
11	9	15.500	2.000	12.000	2.000	2	213	2133
10	9	15.500	2.000	12.000	2.000	2	213	2346
9	9	15.500	2.000	12.000	2.000	2	213	2560
8	9	15.500	2.000	12.000	2.000	2	213	2773
7	9	15.500	2.000	12.000	2.000	2	213	2986
6	9	15.500	2.000	12.000	2.000	2	213	3200
5	9	15.500	2.000	12.000	2.000	2	213	3413
4	9	15.500	2.000	12.000	2.000	2	213	3626
3	9	15.500	2.000	12.000	2.000	2	213	3839
2	9	15.500	2.000	12.000	2.000	2	213	4053
1	9	15.500	2.000	12.000	2.000	2	213	4266
Total = 4266

Steel Beam Section Properties and Weights										
Floor Level	Story Height (ft)	Overall Beam Height (in)	Flange Thickness (in)	Flange Width (in)	Web Thickness (in)	Beam Length (ft)	Beam Weight (kips)	Number of Beams / Floor	Total Weight per Floor (kips)	Acc. Beam Weight (kips)
20	9	16.500	2.000	13.000	0.500	6.000	1.2014	2	2.4028	2.4028
19	9	16.500	2.000	13.000	0.500	6.000	1.2014	2	2.4028	4.8056
18	9	16.500	2.000	13.000	0.500	6.000	1.2014	2	2.4028	7.2084
17	9	16.500	2.000	13.000	0.500	6.000	1.2014	2	2.4028	9.6113
16	9	16.500	2.000	13.000	0.750	6.000	1.2659	2	2.5317	12.1430
15	9	16.500	2.000	13.000	0.750	6.000	1.2659	2	2.5317	14.6747
14	9	16.500	2.000	13.000	0.750	6.000	1.2659	2	2.5317	17.2064
13	9	16.500	2.000	13.000	0.750	6.000	1.2659	2	2.5317	19.7381
12	9	16.500	2.000	13.000	1.000	6.000	1.3303	2	2.6606	22.3988
11	9	16.500	2.000	13.000	1.000	6.000	1.3303	2	2.6606	25.0594
10	9	18.000	2.000	13.000	1.000	6.000	1.3613	2	2.7225	27.7819
9	9	18.000	2.000	13.000	1.000	6.000	1.3613	2	2.7225	30.5044
8	9	16.500	2.000	13.000	1.065	6.000	1.3471	2	2.6941	33.1985
7	9	16.500	2.000	13.000	1.065	6.000	1.3471	2	2.6941	35.8927
6	9	16.500	2.000	13.000	1.065	6.000	1.3471	2	2.6941	38.5868
5	9	16.500	2.000	13.000	1.065	6.000	1.3471	2	2.6941	41.2809
4	9	16.500	2.000	13.000	1.000	6.000	1.3303	2	2.6606	43.9416
3	9	16.500	2.000	13.000	1.000	6.000	1.3303	2	2.6606	46.6022
2	9	16.500	2.000	13.000	1.000	6.000	1.3303	2	2.6606	49.2628
1	9	16.500	2.000	13.000	1.000	6.000	1.3303	2	2.6606	51.9234
Total = 51.9234										

Additional Weight per Floor									Additional Mass per Floor	
Floor Level	Story Height (ft)	Floor Area Loads			Line Loads		Total Add'l Weight/Floor (kips)	Acc. Add'l Weight (kips)	Additional Area Mass (kip-s ² /ft ²)	Additional Line Mass (kip-s ² /ft ²)
		MEP (ksf)	Columns (ksf)	Partitions (ksf)	Cladding (klf)	Misc. (klf)				
20	9	0.005	0.005	0.010	0.300	0.000	215.94	215.94	0.00062112	0.009317
19	9	0.005	0.005	0.010	0.300	0.000	215.94	431.88	0.00062112	0.009317
18	9	0.005	0.005	0.010	0.300	0.000	215.94	647.82	0.00062112	0.009317
17	9	0.005	0.005	0.010	0.300	0.000	215.94	863.76	0.00062112	0.009317
16	9	0.005	0.005	0.010	0.300	0.000	215.94	1079.7	0.00062112	0.009317
15	9	0.005	0.005	0.010	0.300	0.000	215.94	1295.64	0.00062112	0.009317
14	9	0.005	0.005	0.010	0.300	0.000	215.94	1511.58	0.00062112	0.009317
13	9	0.005	0.005	0.010	0.300	0.000	215.94	1727.52	0.00062112	0.009317
12	9	0.005	0.005	0.010	0.300	0.000	215.94	1943.46	0.00062112	0.009317
11	9	0.005	0.005	0.010	0.300	0.000	215.94	2159.4	0.00062112	0.009317
10	9	0.005	0.005	0.010	0.300	0.000	215.94	2375.34	0.00062112	0.009317
9	9	0.005	0.005	0.010	0.300	0.000	215.94	2591.28	0.00062112	0.009317
8	9	0.005	0.005	0.010	0.300	0.000	215.94	2807.22	0.00062112	0.009317
7	9	0.005	0.005	0.010	0.300	0.000	215.94	3023.16	0.00062112	0.009317
6	9	0.005	0.005	0.010	0.300	0.000	215.94	3239.1	0.00062112	0.009317
5	9	0.005	0.005	0.010	0.300	0.000	215.94	3455.04	0.00062112	0.009317
4	9	0.005	0.005	0.010	0.300	0.000	215.94	3670.98	0.00062112	0.009317
3	9	0.005	0.005	0.010	0.300	0.000	215.94	3886.92	0.00062112	0.009317
2	9	0.005	0.005	0.010	0.300	0.000	215.94	4102.86	0.00062112	0.009317
1	9	0.005	0.005	0.010	0.300	0.000	215.94	4318.8	0.00062112	0.009317
Total = 4318.8									0.01242236	0.186335

Total Structural Dynamic Weight										
Floor Level	Story Height (ft)	Slab Weight (kips)	Wall Weight (kips)	Beam Weight (kips)	Add'l Weight (kips)	Add'l Mass (kip-s ² /ft)	Total Weight per Floor (kips)	Total Acc. Weight (kips)	Total Mass per Floor (kips-s ² /ft)	Total Acc. Mass (kips-s ² /ft)
20	9	525	213	2.4028	215.94	6.70621	956	956	30	30
19	9	525	213	2.4028	215.94	6.70621	956	1913	30	59
18	9	525	213	2.4028	215.94	6.70621	956	2869	30	89
17	9	525	213	2.4028	215.94	6.70621	956	3826	30	119
16	9	525	213	2.5317	215.94	6.70621	957	4782	30	149
15	9	525	213	2.5317	215.94	6.70621	957	5739	30	178
14	9	525	213	2.5317	215.94	6.70621	957	6695	30	208
13	9	525	213	2.5317	215.94	6.70621	957	7652	30	238
12	9	525	213	2.6606	215.94	6.70621	957	8608	30	267
11	9	525	213	2.6606	215.94	6.70621	957	9565	30	297
10	9	525	213	2.7225	215.94	6.70621	957	10522	30	327
9	9	525	213	2.7225	215.94	6.70621	957	11478	30	356
8	9	525	213	2.6941	215.94	6.70621	957	12435	30	386
7	9	525	213	2.6941	215.94	6.70621	957	13392	30	416
6	9	525	213	2.6941	215.94	6.70621	957	14348	30	446
5	9	525	213	2.6941	215.94	6.70621	957	15305	30	475
4	9	525	213	2.6606	215.94	6.70621	957	16262	30	505
3	9	525	213	2.6606	215.94	6.70621	957	17218	30	535
2	9	525	213	2.6606	215.94	6.70621	957	18175	30	564
1	9	525	213	2.6606	215.94	6.70621	957	19131	30	594
Total = 180						Total = 19131		Total = 594		

Generating Design Spectral Response

Structure No. **1**

Design Response Spectrum Generation

(NEHRP 2000, FEMA 368,369 / March 2001)

Seismic Use Group: **Sect. 1.3**

Assumption: Persons capacity < 5000 people (sect. 1.3.2, item #6)

^^Assign Seismic Use Group I

Occupancy Importance Factor: **Sect. 1.4**

^^Importance Factor (I): **1.00** (see TABLE 1.4)

Ground Motion: **Sect. 4.1**

Maximum Considered Earthquake Ground Motion: Sect. 4.1.1

As given on Map No.s 3 & 4

For 0.2 sec. Spectral Response Accel.

$$S_s = \mathbf{1.50}$$

For 1.0 sec. Spectral Response Accel.

$$S_1 = \mathbf{0.65}$$

Site class Definitions: Sect. 4.1.2.

$S_s = 1.5 > 0.15$, $S_1 = 0.65 > 0.04$, Take as Site Class C:

^^ Site Class C is assigned

Site Coefficients and Adjusted Maximum considered Earthquake Spectral Response acceleration Parameters: Sect. 4.1.2.4

$$S_{MS} = F_a S_s : (\text{eq. 4.1.2.4-1})$$

$$S_{M1} = F_v S_1 : (\text{eq. 4.1.2.4-2})$$

$$\mathbf{F_a = 1.00} \quad (\text{see TABLE: 4.1.2.4a})$$

$$\mathbf{F_v = 1.30} \quad (\text{see TABLE: 4.1.2.4b})$$

$$\mathbf{S_{MS} = 1.500}$$

$$\mathbf{S_{M1} = 0.845}$$

Design Spectral Response Acceleration Parameters: Sect. 4.1.2.5

$$S_{DS} = (2/3)S_{MS} : (\text{eq. 4.1.2.5-1})$$

$$S_{D1} = (2/3)S_{M1} : (\text{eq. 4.1.2.5-2})$$

$$\mathbf{S_{DS} = 1.0000}$$

$$\mathbf{S_{D1} = 0.5633}$$

General Procedure Response Spectrum: Sect. 4.1.2.6

$$T_0 = 0.2(S_{D1}/S_{DS}) : (\text{by NEHRP definition})$$

$$T_s = S_{D1}/S_{DS} : (\text{by NEHRP definition})$$

$$\text{For } T_0 < T$$

$$S_a = 0.6(S_{DS}/T_0)T + 0.4S_{DS} : (\text{eq. 4.1.2.6-1})$$

$$\text{For } T_0 < T < T_s$$

$$S_a = S_{DS} : (\text{eq. 4.1.2.6-2})$$

$$\text{For } T > T_s$$

$$S_a = S_{D1}/T : (\text{eq. 4.1.2.6-3})$$

$$\mathbf{T_0 = 0.1127}$$

$$\mathbf{T_s = 0.5633}$$

Response Spectrum Data Points			
Governing Equation	T	S _a	
eq. 4.1.2.6-1	0.0000	0.4000	
eq. 4.1.2.6-1	0.0100	0.4533	
eq. 4.1.2.6-1	0.0200	0.5065	
eq. 4.1.2.6-1	0.0300	0.5598	
eq. 4.1.2.6-1	0.0400	0.6130	
eq. 4.1.2.6-1	0.0500	0.6663	
eq. 4.1.2.6-1	0.0600	0.7195	
eq. 4.1.2.6-1	0.0700	0.7728	
eq. 4.1.2.6-1	0.0800	0.8260	
eq. 4.1.2.6-1	0.0900	0.8793	
eq. 4.1.2.6-1	0.1000	0.9325	
eq. 4.1.2.6-1	0.1100	0.9858	
eq. 4.1.2.6-2	0.1127	1.0000	S _a =S _{DS}
eq. 4.1.2.6-2	0.1227	1.0000	S _a =S _{DS}
eq. 4.1.2.6-2	0.1327	1.0000	S _a =S _{DS}
eq. 4.1.2.6-2	0.1427	1.0000	S _a =S _{DS}
eq. 4.1.2.6-2	0.1527	1.0000	S _a =S _{DS}
eq. 4.1.2.6-2	0.5633	1.0000	S _a =S _{DS}
eq. 4.1.2.6-3	0.6133	0.9185	
eq. 4.1.2.6-3	0.6633	0.8492	
eq. 4.1.2.6-3	0.7133	0.7897	
eq. 4.1.2.6-3	0.7633	0.7380	
eq. 4.1.2.6-3	0.8133	0.6926	
eq. 4.1.2.6-3	0.8633	0.6525	
eq. 4.1.2.6-3	0.9133	0.6168	
eq. 4.1.2.6-3	0.9633	0.5848	
eq. 4.1.2.6-3	1.0000	0.5633	S _a = S _{D1}
eq. 4.1.2.6-3	2.0000	0.2817	
eq. 4.1.2.6-3	3.0000	0.1878	
eq. 4.1.2.6-3	4.0000	0.1408	
eq. 4.1.2.6-3	5.0000	0.1127	
eq. 4.1.2.6-3	6.0000	0.0939	
eq. 4.1.2.6-3	7.0000	0.0805	
eq. 4.1.2.6-3	8.0000	0.0704

Calculating Lateral Forces and Story Shears

Structure No. **1**

Equivalent Lateral Force Distributions

(NEHRP 2000, FEMA 368,369 / March 2001)

Equivalent Lateral Force Procedure: (Sect. 5.4)

Seismic Base Shear: (Sect. 5.4.1)

$$V = C_s W \quad (\text{eq. 5.4.1})$$

Calculations of Seismic Response coefficient: (Sect. 5.4.1.1)

$$C_s = S_{DS} / (R/I) \quad (\text{eq. 5.4.1.1-1})$$

S_{DS}	1.00
R	6
I	1

$$C_s = \mathbf{0.1667}$$

Period Determination: (Sect. 5.4.2)

$$T \leq T_a C_u \quad (\text{eq. 5.4.2})$$

$$C_u = \mathbf{1.4}$$

Approximate Fundamental Period: (Sect. 5.4.2.1)

$$T_a = C_r h_n^x \quad (\text{eq. 5.4.2.1-1})$$

$h_n =$	180
$C_r =$	0.02 See TABLE 5.4.2.1
$x =$	0.75

$$T_a = \mathbf{0.9828}$$

$$T \leq T_a C_u = \mathbf{1.3760} \quad \mathbf{O.K.}$$

$$T_a = \mathbf{0.9828}$$

Seismic Coefficients Limits Test:**(back to Sect. 5.4.1.1)**

$$C_s \leq S_{D1} / [T(R/I)] \quad (\text{eq. 4.4.1.1-2})$$

$$C_s \geq 0.044 I S_{DS} \quad (\text{eq. 4.4.1.1-3})$$

$$S_{D1} = \mathbf{0.5633} \quad (\text{from Design Response})$$

$$I = \mathbf{1.0000}$$

$$S_{DS} = \mathbf{1.0000} \quad (\text{from Design Response})$$

$$R = \mathbf{6}$$

$$T_x = \quad (\text{Taken from ETABS filename.IEG})$$

$$T_y = \quad (\text{Taken from ETABS filename.IEG})$$

$T_x(\text{final}) =$	$\mathbf{1.3760}$
$T_y(\text{final}) =$	$\mathbf{1.3760}$

$$S_{D1} / [T(R/I)] = \mathbf{0.06823}$$

$$0.044 I S_{DS} = \mathbf{0.04400}$$

$$C_s(\text{Upper}) = \mathbf{0.0682}$$

$$C_s(\text{lower}) = \mathbf{0.1667}$$

$$C_s (\text{final Value}) = \mathbf{0.0682}$$

Return to Seismic Base Shear:**(Sect. 5.4.1)**

$$V = C_s W \quad (\text{i.e., } C_{s\text{final}} * TDL) \quad (\text{eq. 5.4.1})$$

$$V = \mathbf{1305} \quad (\text{kips})$$

Vertical Distribution of Seismic Forces (Sect. 5.4.3)

$$F_x = C_{vx} V \quad (\text{eq. 5.4.3-1})$$

$$C_{vx} = (w_x h_x^k) / (\sum (w_i h_i^k)) \quad (\text{eq. 5.4.3-2})$$

$$k = \mathbf{1.438} \quad (\text{Sect. 5.4.3})$$

Vertical Distribution of Seismic Forces								
Floor Level	Story Height (ft)	Height Above Base (ft)	Total Weight per Floor (kips)	Accumulated Floor Weight (kips)	$w_x h_x^k$ (kip*ft)	C_{vx}	F_x (kips)	Story Shear (kips)
20	9	180	956	956	1673751	0.11481	150	150
19	9	171	956	1913	1554740	0.10664	139	289
18	9	162	956	2869	1438441	0.09867	129	418
17	9	153	956	3826	1324940	0.09088	119	537
16	9	144	957	4782	1214490	0.08331	109	645
15	9	135	957	5739	1106850	0.07592	99	744
14	9	126	957	6695	1002310	0.06875	90	834
13	9	117	957	7652	900991	0.06180	81	915
12	9	108	957	8608	803141	0.05509	72	987
11	9	99	957	9565	708683	0.04861	63	1050
10	9	90	957	10522	617956	0.04239	55	1106
9	9	81	957	11478	531079	0.03643	48	1153
8	9	72	957	12435	448321	0.03075	40	1193
7	9	63	957	13392	369996	0.02538	33	1226
6	9	54	957	14348	296434	0.02033	27	1253
5	9	45	957	15305	228069	0.01564	20	1273
4	9	36	957	16262	165461	0.01135	15	1288
3	9	27	957	17218	109404	0.00750	10	1298
2	9	18	957	18175	61069	0.00419	5	1303
1	9	9	957	19131	22539	0.00155	2	1305
Total = 180		Total = 19131		$\Sigma(w_i h_i^k) = 14578667$		Base Shear Check O.K.		
						Base Shear = 1305		

Calculating Tributary Wall Gravity Loads

Structure No. **1**

LL: 0.05 (ksf)

Wall TA: 1167 (ft²)

Gravity:

Dead Loads

SW As Calc.

Columns: 0.005 (ksf)

MEP: 0.005 (ksf)

Partitions 0.010 (ksf)

Cladding: 0.300 (klf)

Wall Dead and Live Loads								
Floor Level	Slab Weight per Floor (kips/ft ²)	Beam Weight per Floor (kips)	Add'l Weight per Floor (kips)	Wall Weight per Floor (kips)	Wall DL (kips)	Acc. Wall DL (kips)	LL / Floor (kips)	Acc. Wall LL (kips)
20	0.0875	1.2014	17	107	227	227	58	58
19	0.0875	1.2014	17	107	227	455	58	117
18	0.0875	1.2014	17	107	227	682	58	175
17	0.0875	1.2014	17	107	227	910	58	233
16	0.0875	1.2659	17	107	227	1137	58	292
15	0.0875	1.2659	17	107	227	1365	58	350
14	0.0875	1.2659	17	107	227	1592	58	408
13	0.0875	1.2659	17	107	227	1820	58	467
12	0.0875	1.3303	17	107	228	2047	58	525
11	0.0875	1.3303	17	107	228	2275	58	583
10	0.0875	1.3613	17	107	228	2502	58	642
9	0.0875	1.3613	17	107	228	2730	58	700
8	0.0875	1.3471	17	107	228	2958	58	758
7	0.0875	1.3471	17	107	228	3185	58	817
6	0.0875	1.3471	17	107	228	3413	58	875
5	0.0875	1.3471	17	107	228	3640	58	933
4	0.0875	1.3303	17	107	228	3868	58	992
3	0.0875	1.3303	17	107	228	4095	58	1050
2	0.0875	1.3303	17	107	228	4323	58	1108
1	0.0875	1.3303	17	107	228	4551	58	1167
					4551		1167	

Design of a Steel Coupling Beam

Structure No. **1**

Floor Level: **1-4**

Design Forces

Mu: **8072** (k-in)

Vu: **224** (k)

Beam and Material Properties

L: **72.0** (in)

E_s: **29000.0** (ksi)

F_{yw}: **36.0** (ksi)

F_{yt}: **36.0** (ksi)

Local Buckling Parameters

WLB

$$h/t_w \leq \alpha \sqrt{E/F_{wy}}$$

α: **3.05** (taken from AISC Table: C-B5.1)

h/t_w ≤ **86.57**

FLB

$$b_f/t_f \leq \beta \sqrt{E/F_{wy}}$$

β: **0.31** (taken from AISC Table: C-B5.1)

h/t_w ≤ **8.80**

Establishing a Trial Design

Shear Strength (V_n)

$$V_u \leq \phi_v V_n$$

$$\phi_v: \quad 0.9$$

$$V_p = 0.6 F_{yw} A_w$$

$$A_w = h t_w$$

$$V_p = 0.6 F_{yw} h t_w$$

$$V_p = 21.6 * h * t_w$$

$$V_u = \phi V_n = \phi V_p = \quad 224 \quad (k)$$

$$h * t_w = \quad 11.51 \quad (in)^2$$

Assume Values For Web Dimensions

$t_w =$	1.000	(in)	$h * t_w$ Is O.K.
$h =$	12.500	(in)	
WLB Is Satisfied.			

$$V_u = \quad 224 \quad (k)$$

$$\phi V_p = \quad 243 \quad (k)$$

Shear Strength O.K.

Flexural Strength (M_n)

$$M_u \leq \phi_b M_n$$

$$\phi_b: \quad 0.9$$

$$M_n = M_p \leq 1.5 M_y$$

$$M_p = F_y t Z_x$$

$$M_p = 36 * Z_x$$

$$M_y = F_y t S_x$$

$$M_y = 36 * S_x$$

$$M_u = \phi_b M_p$$

$$M_u = \quad 8072 \quad (k-in)$$

$$Z_{x(req)} = \quad 249.14 \quad (in)^3$$

To Ensure Shear Response Domination:

$$L \leq 1.6M_p/V_p$$

$$L = 1.6F_y Z_x / (V_p / \phi_v)$$

$$Z_{x(\text{req})} = 310.75 \quad (\text{in})^3$$

Selecting The Larger Value for $Z_{x(\text{req})}$

$$Z_{x(\text{req})} = 310.75 \quad (\text{in})^3$$

To Satisfy FLB:

$$b_f/t_f \leq \beta \sqrt{E/F_{wy}}$$

Assume Values For Flange Dimensions

$b_f =$	13.000	(in)
$t_f =$	2.000	(in)

FLB Is Satisfied.

For A Wide-Flange Beam, Z_x is:

$$Z_x = 2[(b_f t_f h/2) + h^2 t_w/8]$$

$$Z_{x(\text{prov})} = 364.06 \quad (\text{in})^3$$

Z_x Is Satisfied.

Trial Section Properties

$t_f =$	2.000	(in)
$b_f =$	13.000	(in)
$t_w =$	1.000	(in)
$h =$	12.500	(in)
$Z_x =$	364.063	$(\text{in})^3$
$d =$	16.500	(in)
$I_x =$	2913.34	$(\text{in})^4$
$S_x =$	353.13	$(\text{in})^3$
$A =$	64.500	$(\text{in})^2$

FLB Is Satisfied.

WLB Is Satisfied.

O.K.

Diff. in Z_x 53.31 $(\text{in})^3$

Strength Checks

Shear Strength

$$\begin{aligned} V_u &= 224 & (k) \\ \phi V_p &= 243 & (k) \\ \text{Shear Strength} &= 243 & (k) \end{aligned}$$

<u>Shear Strength IS O.K.</u>
Design Is Within 8%

Flexural Strength

$$\begin{aligned} M_u &= 8072 & (k\text{-in}) \\ \phi M_p &= 11796 & (k\text{-in}) \\ \phi 1.5 M_y &= 17162 & (k\text{-in}) \\ \text{Flexural Strength} &= 17162 & (k\text{-in}) \end{aligned}$$

<u>Flexural Strength IS O.K.</u>
*Plastic Moment Governs.
Design Is Within 53%

WLB

$$\begin{aligned} h / t_w &= 12.50 \\ \alpha(E_s/F_{yw}) &= 86.57 \end{aligned}$$

<u>WLB Is O.K.</u>

FLB

$$\begin{aligned} b_f / t_f &= 6.50 \\ \beta(E_s/F_{yf}) &= 8.80 \end{aligned}$$

<u>FLB Is O.K.</u>

Plastic Section Modulus

$$\begin{aligned} Z_{x(\text{req})} &= 310.75 & (\text{in})^3 \\ Z_{x(\text{prov})} &= 364.06 & (\text{in})^3 \end{aligned}$$

Plastic Section Modulus Is O.K.
Design Is Within 14.65%

Embedment Length

$$V_u = 0.9f_b\beta_1b_f l_e[(0.58 - 0.22\beta_1) / (0.88 + (a/l_e))]$$

$$f_b = 54.0 * \text{SQRT}(f'_c)(t_{\text{wall}}/b_f)0.66$$

$$\begin{aligned} f'_c &= 5000 & (\text{psi}) \\ \beta_1 &= 0.80 \\ b_f &= 13.00 & (\text{in}) \\ t_{\text{wall}} &= 10.00 \\ V_u &= 224 & (k) \\ f_b &= 3.21 & (\text{ksi}) \\ L &= 72.00 & (\text{in}) \end{aligned}$$

$l_e = 36.00$	(in)
---------------	------

Example Calculations of a Diagonally-Reinforced Concrete Beam

Structure No. DBCB

Floor Level: 1-4

Legend

Design Forces

Mu: **6037** (k-in)
Vu: **168** (k)

Minimum Diagonal Dimensions

$b_w \geq$ **12** $b_w/2$
 $b_h \geq$ **4.8** $b_h/5$
 $\phi_v =$ **0.75**

Beam and Material Properties

L: **72.0** (in)
Ec: **4030.5** (ksi)
h: **22.0** (ksi)
bw **24.0** (ksi)
Clear Cover = **1.5** (in)
l/h **3.272727**

Rebar Spacing Requirements

ACI 7.6
7.6.3 $s \geq 1.5db$ and 1.5"
7.6.6 Bundled Bars

$b_w =$ **12** (in)
 $b_h =$ **10** (in)

Rebar Properties

Es (rebar) **29000** (ksi)
Fy (rebar) **60** (ksi)
Stirrup Size= **0.5** (in)

α	$\sin \alpha$	Vn (req) (kips)	A_{vD} (req) (in ²)	Bar Size Used (in ²)	Bar Quantity (req)	Bar Quantity (prov)	A_{vD} (prov) (in ²)	ϕV_n (prov) (kips)	Vn (prov) (kips)	α
7.13	0.1240	223	15.008	1.27	12	12	15.24	170	227	6.1
ϕV_n	170	(kips)								
$\phi V_n > V_u$	YES									

Appendix C: Wall Boundary Element Study Results

Table C.1. Plan dimensions for structures in Wall Boundary Study.

Structure	Type	X (m)	Y (m)	CX (m)	CY (m)	WL (m)	FL (m)	WLI (m)	BL (m)
01	one-cell	24.4	24.4	9.1	6	3.7	4.7	n/a	1.8
02	two-cell	30.5	24.4	13.4	6.7	2.6	5.5	4.6	1.8
03	two-cell	42.7	30.5	21.9	7.9	4.6	7.9	9.2	1.8
04	one-cell	30.5	30.5	9.1	6	3.7	4.7	n/a	1.8
05	two-cell	42.7	30.5	16.5	9.1	4	7.9	4.9	1.8
06	two-cell	30.5	24.4	14.6	6.7	3.1	5.5	4.9	1.8
07	one-cell	24.4	24.4	6.7	4.9	2.4	3.7	n/a	1.8
08	one-cell	30.4	30.4	6.7	4.9	2.4	3.7	n/a	1.8

Table C.2. Equivalent Lateral Force seismic Coefficients, weights, and base shears.

Base Shear Summary			
Structure	C _s	W (kN)	V _B (kN)
01	0.0682	85095	5806
02	0.0682	107984	7368
03	0.0682	177644	12121
04	0.0682	121733	8306
05	0.0682	173543	11842
06	0.0682	109136	7447
07	0.0682	81403	5554
08	0.0682	118148	8062

Table C.3. ELF story forces and story shears.

Equivalent Lateral Forces Summary																
Floor	Structure 01		Structure 02		Structure 03		Structure 04		Structure 05		Structure 06		Structure 07		Structure 08	
	Fx (kN)	Vx (kN)	Fx (kN)	Vx (kN)	Fx (kN)	Vx (kN)	Fx (kN)	Vx (kN)	Fx (kN)	Vx (kN)	Fx (kN)	Vx (kN)	Fx (kN)	Vx (kN)	Fx (kN)	Vx (kN)
20	667	667	846	846	1392	1392	954	954	1360	1360	855	855	638	638	925	925
19	619	1286	786	1632	1293	2685	886	1839	1263	2622	794	1649	592	1230	860	1785
18	573	1859	727	2359	1196	3881	820	2659	1168	3791	735	2384	548	1778	795	2580
17	528	2386	670	3028	1102	4982	755	3414	1076	4867	677	3061	505	2283	733	3313
16	484	2870	614	3642	1010	5992	692	4106	986	5853	620	3681	463	2745	672	3985
15	441	3311	559	4202	920	6912	631	4737	899	6752	565	4246	422	3167	612	4597
14	399	3710	507	4708	833	7746	571	5308	814	7567	512	4758	382	3549	554	5151
13	359	4069	455	5163	749	8495	513	5821	732	8298	460	5219	343	3892	498	5649
12	320	4389	406	5569	668	9163	458	6279	652	8951	410	5629	306	4198	444	6093
11	282	4671	358	5928	589	9752	404	6682	576	9526	362	5991	270	4468	392	6485
10	246	4917	312	6240	514	10266	352	7034	502	10028	316	6307	235	4704	342	6827
9	212	5129	268	6508	442	10707	303	7337	431	10460	271	6578	202	4906	294	7120
8	179	5307	227	6735	373	11080	255	7592	364	10824	229	6807	171	5077	248	7368
7	147	5455	187	6922	308	11387	211	7803	301	11124	189	6996	141	5218	205	7573
6	118	5573	150	7072	246	11634	169	7972	241	11365	151	7147	113	5331	164	7737
5	91	5664	115	7187	190	11823	130	8102	185	11550	116	7264	87	5418	126	7863
4	66	5730	84	7271	138	11961	94	8196	134	11685	85	7348	63	5481	92	7955
3	44	5773	55	7326	91	12052	62	8259	89	11774	56	7404	42	5522	61	8015
2	24	5798	31	7357	51	12103	35	8294	50	11823	31	7435	23	5546	34	8049
1	9	5807	11	7368	19	12122	13	8306	18	11841	12	7447	9	5554	12	8062
	5807		7368		12122		8306		11841		7447		5554		8062	

Table C.4. Redundancy, ρ , and wall overstrength factors, γ_{wall} .

ρ and γ_{wall} Factors Summary		
Structure	ρ	γ_{wall}
01	1.428	1.62
02	1.274	1.37
03	1.301	1.32
04	1.500	1.72
05	1.500	1.32
06	1.324	1.32
07	1.500	1.73
08	1.500	1.85

Table C.5. Beam Design Shears and Factored Capacities.

Beam Shear Demand and Capacity Summary (includes ρ)							
Structure	Floor Level	Design Shear (kN)	Factored Capacity (kN)	Structure	Floor Level	Design Shear (kN)	Factored Capacity (kN)
01	17-20	445	540	05	17-20	424	545
	13-16	753	811		13-16	775	835
	9-12	1032	1081		9-12	1119	1162
	5-8	1109	1151		5-8	1220	1262
	1-4	995	1081		1-4	1082	1162
02	17-20	305	430	06	17-20	237	304
	13-16	542	619		13-16	446	484
	9-12	760	813		9-12	635	650
	5-8	827	859		5-8	681	692
	1-4	757	813		1-4	614	678
03	17-20	243	295	07	17-20	716	865
	13-16	391	431		13-16	1121	1176
	9-12	472	481		9-12	1600	1665
	5-8	474	481		5-8	2104	2144
	1-4	332	363		1-4	2131	2188
04	17-20	744	951	08	17-20	1141	1199
	13-16	1302	1427		13-16	1810	1840
	9-12	1828	1921		9-12	2400	2399
	5-8	2160	2226		5-8	3469	3467
	1-4	2084	2188		1-4	3599	3590

Table C.5. Beam Proportioning.

Beam Proportioning Summary												
Structure	Floor Levels	d	h _w	t _w	b _f	t _f	Structure	d	h _w	t _w	b _f	t _f
		(mm)	(mm)	(mm)	(mm)	(mm)		(mm)	(mm)	(mm)	(mm)	(mm)
01	17-20	419	318	13	330	51	05	610	533	8	330	38
	13-16	419	318	19	330	51		610	533	12	330	38
	9-12	419	318	25	330	51		610	533	16	330	38
	5-8	419	318	27	330	64		610	533	18	330	38
	1-4	419	318	25	330	51		610	533	16	330	38
02	17-20	438	337	10	305	51	06	483	406	6	330	38
	13-16	438	337	14	305	51		483	406	9	330	38
	9-12	438	337	18	305	51		483	406	12	330	38
	5-8	438	337	19	305	51		483	406	13	330	38
	1-4	438	337	18	305	51		483	406	12	330	38
03	17-20	368	267	8	229	51	07	610	508	13	356	51
	13-16	368	267	12	229	51		610	508	17	356	51
	9-12	368	267	13	229	51		660	559	22	356	51
	5-8	368	267	13	229	51		660	559	29	356	51
	1-4	368	267	10	229	51		660	559	29	356	51
04	17-20	660	559	13	406	51	08	635	483	19	406	76
	13-16	660	559	19	406	51		635	483	28	406	76
	9-12	660	559	26	406	51		635	483	37	406	76
	5-8	660	559	30	406	64		635	483	54	406	76
	1-4	660	559	29	406	51		635	483	55	406	76

Table C.6. Wall pier longitudinal steel reinforcement.

Pier Reinforcement Summary																		
Structure	Floors	C-Shape				I-Shape				Structure	C-Shape				I-Shape			
		ρ (%)	Bar Qty.	Bar Size (#)	Bar Spacing (mm)	ρ (%)	Bar Qty.	Bar Size (#)	Bar Spacing (mm)		ρ (%)	Bar Qty.	Bar Size (#)	Bar Spacing (mm)	ρ (%)	Bar Qty.	Bar Size (#)	Bar Spacing (mm)
01	17-20	0.81	92	9	229	n/a	n/a	n/a	n/a	05	0.84	208	7	6	0.71	150	8	203
	13-16	0.81	92	9	229	n/a	n/a	n/a	n/a		0.84	208	7	6	0.71	150	8	203
	9-12	0.81	92	9	229	n/a	n/a	n/a	n/a		0.84	208	7	6	0.71	150	8	203
	5-8	1.26	92	11	229	n/a	n/a	n/a	n/a		0.84	208	7	6	0.71	150	8	203
	1-4	1.82	92	14	229	n/a	n/a	n/a	n/a		0.84	208	7	6	0.71	150	8	203
02	17-20	1.05	106	9	178	1.05	128	10	203	06	1.04	114	9	8	1.32	150	10	203
	13-16	1.05	106	9	178	1.05	128	10	203		1.04	114	9	8	1.32	150	10	203
	9-12	1.05	106	9	178	1.05	128	10	203		1.04	114	9	8	1.32	150	10	203
	5-8	1.39	90	11	178	1.05	128	10	203		1.04	114	9	8	1.32	150	10	203
	1-4	2.00	90	14	178	1.70	128	14	203		1.04	114	9	8	1.32	150	10	203
03	17-20	0.47	126	7	254	0.67	212	8	229	07	1.48	94	10	7	n/a	n/a	n/a	n/a
	13-16	0.47	126	7	254	0.67	212	8	229		1.48	94	10	7	n/a	n/a	n/a	n/a
	9-12	0.47	126	7	254	0.67	212	8	229		1.48	94	10	7	n/a	n/a	n/a	n/a
	5-8	0.47	126	7	254	0.67	212	8	229		2.62	94	14	7	n/a	n/a	n/a	n/a
	1-4	0.47	126	7	254	0.67	212	8	229		3.68	132	14	5	n/a	n/a	n/a	n/a
04	17-20	1.04	118	9	127	n/a	n/a	n/a	n/a	08	1.39	112	9	6	n/a	n/a	n/a	n/a
	13-16	1.04	118	9	127	n/a	n/a	n/a	n/a		1.76	112	10	6	n/a	n/a	n/a	n/a
	9-12	1.31	118	10	127	n/a	n/a	n/a	n/a		3.13	112	14	6	n/a	n/a	n/a	n/a
	5-8	2.17	158	11	127	n/a	n/a	n/a	n/a		5.56	112	18	6	n/a	n/a	n/a	n/a
	1-4	3.72	188	14	127	n/a	n/a	n/a	n/a		5.95	120	18	5.5	n/a	n/a	n/a	n/a

Table C.7. Pier Design forces.

Pier Design Forces and Moments														
Structure	Floor	C-Shaped Wall Piers							I-Shaped Walls					
		LC1				LC2			LC1			LC2		
		Pu		Mx (kN-m)	My (kN-m)	Pu (kN)	Mx (kN-m)	My (kN-m)	Pu (kN)	Mx (kN-m)	My (kN-m)	Pu (kN)	Mx (kN-m)	My (kN-m)
		Comp. (kN)	Tens. (kN)											
Structure_01	20	2747	-493	334	4033	1703	1113	522	n/a	n/a	n/a	n/a	n/a	n/a
	19	5494	-986	968	6010	3424	3225	823	n/a	n/a	n/a	n/a	n/a	n/a
	18	8241	-1479	1827	6434	5164	6089	932	n/a	n/a	n/a	n/a	n/a	n/a
	17	10988	-1971	2845	6001	6925	9483	920	n/a	n/a	n/a	n/a	n/a	n/a
	16	14336	-3065	3968	6202	8753	13226	974	n/a	n/a	n/a	n/a	n/a	n/a
	15	17684	-4158	5155	6127	10602	17184	993	n/a	n/a	n/a	n/a	n/a	n/a
	14	21032	-5251	6383	7129	12469	21276	1194	n/a	n/a	n/a	n/a	n/a	n/a
	13	24380	-6344	7639	8404	14353	25463	1454	n/a	n/a	n/a	n/a	n/a	n/a
	12	28329	-8038	8922	9291	16293	29739	1630	n/a	n/a	n/a	n/a	n/a	n/a
	11	32278	-9731	10236	10198	18246	34119	1818	n/a	n/a	n/a	n/a	n/a	n/a
	10	36227	-11424	11590	11156	20212	38634	2024	n/a	n/a	n/a	n/a	n/a	n/a
	9	40176	-13118	12997	12216	22188	43324	2255	n/a	n/a	n/a	n/a	n/a	n/a
	8	44281	-14967	14471	13331	24183	48235	2496	n/a	n/a	n/a	n/a	n/a	n/a
	7	48387	-16817	16023	14724	26182	53410	2794	n/a	n/a	n/a	n/a	n/a	n/a
	6	52492	-18666	17665	16533	28182	58884	3173	n/a	n/a	n/a	n/a	n/a	n/a
	5	56597	-20516	19402	18945	30173	64674	3665	n/a	n/a	n/a	n/a	n/a	n/a
	4	60546	-22210	21235	22375	32131	70784	4351	n/a	n/a	n/a	n/a	n/a	n/a
	3	64495	-23903	23158	27128	34048	77193	5261	n/a	n/a	n/a	n/a	n/a	n/a
	2	68444	-25596	25160	33912	35891	83868	6485	n/a	n/a	n/a	n/a	n/a	n/a
	1	72392	-27290	27227	44022	37619	90755	8165	n/a	n/a	n/a	n/a	n/a	n/a
Structure_02	20	2456	527	1307	922	2176	731	395	1578	652	3099	1578	2180	936
	19	4966	1000	2038	1847	4373	2541	615	3156	1668	5187	3156	5580	1566
	18	7538	1410	2363	2834	6596	5180	714	4734	2991	6143	4734	10005	1855
	17	10179	1752	2414	3878	8847	8421	729	6312	4562	6280	6312	15258	1897
	16	13073	1842	2628	5039	11169	12070	794	7890	6334	6774	7890	21183	2046
	15	16047	1851	2700	6171	13519	16012	815	9469	8253	6932	9469	27602	2093
	14	19096	1785	2670	7252	15896	20163	806	11047	10288	6819	11047	34407	2059
	13	22211	1653	2546	8299	18293	24481	769	12626	12418	6479	12626	41532	1957
	12	25585	1262	2695	9394	20757	28932	814	14205	14644	6793	14205	48978	2051
	11	29022	809	2772	10409	23239	33537	837	15784	16955	7094	15784	56706	2142
	10	32519	296	3060	11574	25739	38309	924	17363	17043	7848	17363	64728	2370
	9	36070	-271	3398	13709	28253	43275	1026	18943	21846	8729	18943	73062	2636
	8	39712	-930	3709	16056	30793	48465	1120	20522	24439	9576	20522	81737	2892
	7	43386	-1620	4114	18678	33341	53916	1242	22101	27135	10697	22101	90752	3230
	6	47070	-2320	4650	21616	35895	59666	1404	23680	29927	12230	23680	100090	3693
	5	50733	-3000	5378	24915	38443	65749	1624	25260	32798	14386	25260	109694	4344
	4	54278	-3561	6514	28642	40961	72212	1967	26839	35713	17757	26839	119444	5362
	3	57681	-3980	8141	32832	43436	79116	2459	28418	38614	22586	28418	129145	6821
	2	60832	-4147	10527	37543	45833	86536	3179	29997	41422	29661	29997	138537	8957
	1	63560	-3892	14836	42695	48092	94611	4480	31576	44013	38854	31576	147203	11733

Table C.7 (cont.). Pier design forces.

Pier Design Forces and Moments														
Structure	Floor	C-Shaped Wall Piers							I-Shaped Walls					
		LC1				LC2			LC1			LC2		
		Pu		Mx (kN-m)	My (kN-m)	Pu (kN)	Mx (kN-m)	My (kN-m)	Pu (kN)	Mx (kN-m)	My (kN-m)	Pu (kN)	Mx (kN-m)	My (kN-m)
		Comp. (kN)	Tens. (kN)											
Structure_03	20	3685	1135	2645	1003	3473	778	786	2756	1116	7051	2756	3671	2096
	19	7396	2242	4265	2165	6958	3072	1268	5513	2787	12805	5513	9172	3806
	18	11130	3327	5197	3486	10457	6674	1545	8269	4956	16440	8269	16310	4886
	17	14897	4379	5652	4934	13973	11315	1680	11026	7560	18385	11026	24880	5464
	16	18832	5264	6160	6534	17540	16756	1831	13782	10544	20164	13782	34701	5993
	15	22810	6105	6372	8157	21128	22838	1894	16539	13841	20908	16539	45554	6214
	14	26832	6903	6340	9768	24737	29433	1884	19296	17405	20768	19296	57283	6173
	13	30895	7659	6100	11347	28365	36454	1813	22053	21198	19884	22053	69767	5910
	12	35046	8328	5822	13744	32024	43846	1730	24810	25197	18759	24810	82929	5575
	11	39230	8963	5375	16961	35698	51587	1597	27567	29378	17127	27567	96687	5090
	10	43440	9573	5243	20465	39383	59673	1558	30324	33723	18017	30324	110989	5355
	9	47666	10167	6355	24266	43077	68113	1889	33082	38219	22219	33082	125786	6604
	8	51896	10756	7757	28382	46772	76930	2305	35839	42851	27739	35839	141030	8244
	7	56116	11355	9488	32833	50463	86158	2820	38596	47599	34707	38596	156657	10315
	6	60307	11984	11603	37646	54142	95835	3449	41353	52437	43303	41353	172579	12870
	5	64447	12663	14180	42848	57799	106005	4214	44110	57327	53774	44110	188672	15982
	4	68393	13537	17788	48512	61392	116738	5287	46866	62204	67487	46866	204724	20058
	3	72240	14509	22245	54586	64939	128077	6612	49623	67008	83391	49623	220535	24784
	2	75951	15617	28018	61105	68423	140141	8327	52380	71624	101437	52380	235727	30148
	1	79479	16908	36109	68078	71821	153130	10732	55136	75879	120909	55136	249730	35935
Structure_04	20	4092	-1221	523	6096	2171	1743	642	n/a	n/a	n/a	n/a	n/a	n/a
	19	8184	-2443	1484	8371	4378	4948	965	n/a	n/a	n/a	n/a	n/a	n/a
	18	12276	-3664	2760	8133	6619	9198	1015	n/a	n/a	n/a	n/a	n/a	n/a
	17	16368	-4886	4235	6693	8895	14116	899	n/a	n/a	n/a	n/a	n/a	n/a
	16	21518	-7164	5817	7946	11281	19389	1122	n/a	n/a	n/a	n/a	n/a	n/a
	15	26667	-9442	7437	9731	13696	24791	1438	n/a	n/a	n/a	n/a	n/a	n/a
	14	31817	-11720	9055	11610	16132	30182	1785	n/a	n/a	n/a	n/a	n/a	n/a
	13	36967	-13998	10650	13678	18582	35500	2175	n/a	n/a	n/a	n/a	n/a	n/a
	12	43216	-17374	12223	14598	21123	40745	2363	n/a	n/a	n/a	n/a	n/a	n/a
	11	49466	-20751	13789	15723	23673	45962	2590	n/a	n/a	n/a	n/a	n/a	n/a
	10	55714	-24129	15370	17043	26231	51235	2856	n/a	n/a	n/a	n/a	n/a	n/a
	9	61963	-27506	17002	18602	28799	56673	3171	n/a	n/a	n/a	n/a	n/a	n/a
	8	68889	-31559	18721	19668	31425	62404	3393	n/a	n/a	n/a	n/a	n/a	n/a
	7	75816	-35612	20566	21055	34063	68555	3678	n/a	n/a	n/a	n/a	n/a	n/a
	6	82742	-39665	22569	22836	36711	75231	4039	n/a	n/a	n/a	n/a	n/a	n/a
	5	89669	-43718	24750	25165	39365	82499	4502	n/a	n/a	n/a	n/a	n/a	n/a
	4	96510	-47686	27115	28492	42002	90382	5141	n/a	n/a	n/a	n/a	n/a	n/a
	3	103352	-51655	29657	33374	44600	98858	6037	n/a	n/a	n/a	n/a	n/a	n/a
	2	110194	-55623	32359	41016	47107	107862	7357	n/a	n/a	n/a	n/a	n/a	n/a
	1	117036	-59592	35189	53941	49430	117295	9418	n/a	n/a	n/a	n/a	n/a	n/a

Table C.7 (cont.). Pier design forces.

Pier Design Forces and Moments														
Structure	Floor	C-Shaped Wall Piers							I-Shaped Walls					
		LC1				LC2			LC1			LC2		
		Pu		Mx (kN-m)	My (kN-m)	Pu (kN)	Mx (kN-m)	My (kN-m)	Pu (kN)	Mx (kN-m)	My (kN-m)	Pu (kN)	Mx (kN-m)	My (kN-m)
		Comp. (kN)	Tens. (kN)											
Structure_05	20	3928	920	2408	1381	3529	1694	825	2292	701	4296	2292	2659	1471
	19	7931	1765	4544	2845	7087	5267	1557	4585	1933	5768	4585	7333	1976
	18	12006	2537	5440	4413	10675	10364	1863	6877	3612	6512	6877	13705	2230
	17	16163	3228	5523	6095	14296	16664	1892	9169	5658	6531	9169	21466	2237
	16	20745	3494	5973	8048	18035	23895	2046	11462	7996	7394	11462	30336	2533
	15	25430	3657	6095	9994	21811	31846	2088	13755	10565	7668	13755	40082	2627
	14	30208	3727	5907	11901	25619	40364	2023	16049	13320	7545	16049	50534	2584
	13	35067	3717	5485	13784	29454	49351	1879	18342	16230	7189	18342	61577	2463
	12	40401	3232	5735	15833	33412	58750	1965	20636	19280	7738	20636	73146	2651
	11	45812	2670	6157	18348	37394	68538	2109	22930	22458	8329	22930	85204	2853
	10	51296	2034	6856	21887	41397	78714	2349	25223	25762	9197	25223	97740	3150
	9	56844	1334	7723	25762	45419	89294	2646	27516	29193	10313	27516	110758	3533
	8	62563	465	8527	29947	49486	100302	2921	29811	32754	11148	29811	124268	3819
	7	68313	-435	9649	34547	53563	111765	3305	32105	36445	12236	32105	138273	4191
	6	74063	-1336	11235	39608	57642	123708	3848	34400	40264	13624	34400	152761	4667
	5	79768	-2192	13513	45178	61708	136143	4629	36694	44200	15366	36694	167692	5263
	4	85241	-2816	17118	51343	65713	149078	5863	38988	48231	18274	38988	182986	6259
	3	90500	-3226	22124	58076	69650	162514	7578	41282	52324	22232	41282	198516	7615
	2	95392	-3269	29080	65413	73471	176491	9961	43576	56412	28056	43576	214028	9610
	1	99684	-2712	38219	73176	77098	191100	13091	45871	60391	38342	45871	229122	13134
Structure_06	20	2487	643	1426	841	2271	786	429	1615	569	3170	1615	1897	954
	19	5010	1251	2387	1717	4558	2618	719	3230	1501	5073	3230	5003	1527
	18	7570	1821	2808	2659	6862	5257	845	4845	2738	5982	4845	9128	1801
	17	10173	2349	2864	3664	9187	8494	862	6459	4220	6116	6459	14070	1841
	16	13035	2618	3151	4809	11583	12158	949	8075	5896	6764	8075	19659	2037
	15	15954	2829	3257	5928	14001	16133	981	9690	7717	7002	9690	25732	2108
	14	18928	2987	3217	7002	16440	20337	969	11306	9653	6921	11306	32186	2084
	13	21949	3096	3056	8040	18897	24727	920	12921	11683	6570	12921	38954	1978
	12	25222	2955	3191	9128	21420	29275	961	14537	13802	6879	14537	46020	2071
	11	28542	2767	3216	10248	23957	33987	968	16153	16004	6935	16153	53361	2088
	10	31904	2535	3468	12254	26509	38877	1044	17768	18290	7595	17768	60985	2287
	9	35303	2267	3900	14462	29073	43964	1174	19384	20667	8553	19384	68908	2575
	8	38782	1920	4335	16873	31658	49274	1305	21000	23139	9511	21000	77152	2864
	7	42274	1559	4926	19541	34248	54836	1483	22616	25710	10806	22616	85723	3253
	6	45759	1205	5739	22496	36837	60678	1728	24232	28376	12576	24232	94612	3786
	5	49209	887	6863	25768	39415	66823	2066	25848	31125	15008	25848	103780	4519
	4	52566	662	8472	29391	41965	73296	2551	27464	33934	18475	27464	113145	5562
	3	55786	572	10724	33394	44471	80127	3229	29080	36762	23269	29080	122575	7006
	2	58787	703	13936	37818	46902	87387	4196	30696	39535	29888	30696	131821	8998
	1	61445	1176	18764	42631	49216	95266	5649	32312	42099	38778	32312	140370	11675

Table C.7 (cont.). Pier design forces.

Pier Design Forces and Moments														
Structure	Floor	C-Shaped Wall Piers							I-Shaped Walls					
		LC1				LC2			LC1			LC2		
		Pu		Mx (kN-m)	My (kN-m)	Pu (kN)	Mx (kN-m)	My (kN-m)	Pu (kN)	Mx (kN-m)	My (kN-m)	Pu (kN)	Mx (kN-m)	My (kN-m)
		Comp. (kN)	Tens. (kN)											
Structure_07	20	3237	-1326	375	4102	1478	1250	407	n/a	n/a	n/a	n/a	n/a	n/a
	19	6475	-2652	1054	5353	2995	3512	597	n/a	n/a	n/a	n/a	n/a	n/a
	18	9712	-3977	1936	4994	4556	6452	622	n/a	n/a	n/a	n/a	n/a	n/a
	17	12950	-5303	2931	4053	6155	9769	555	n/a	n/a	n/a	n/a	n/a	n/a
	16	16879	-7321	3967	4951	7834	13224	723	n/a	n/a	n/a	n/a	n/a	n/a
	15	20809	-9338	4993	5830	9534	16642	897	n/a	n/a	n/a	n/a	n/a	n/a
	14	24738	-11355	5976	6733	11243	19918	1078	n/a	n/a	n/a	n/a	n/a	n/a
	13	28668	-13373	6903	7803	12949	23008	1288	n/a	n/a	n/a	n/a	n/a	n/a
	12	33684	-16475	7774	7924	14733	25915	1323	n/a	n/a	n/a	n/a	n/a	n/a
	11	38700	-19578	8604	8281	16508	28681	1396	n/a	n/a	n/a	n/a	n/a	n/a
	10	43715	-22681	9416	8814	18276	31386	1500	n/a	n/a	n/a	n/a	n/a	n/a
	9	48731	-25785	10243	9600	20041	34143	1651	n/a	n/a	n/a	n/a	n/a	n/a
	8	54813	-29952	11126	9885	21868	37087	1701	n/a	n/a	n/a	n/a	n/a	n/a
	7	60895	-34120	12107	10427	23705	40357	1802	n/a	n/a	n/a	n/a	n/a	n/a
	6	66977	-38288	13220	11180	25559	44067	1948	n/a	n/a	n/a	n/a	n/a	n/a
	5	73059	-42455	14487	12174	27433	48288	2143	n/a	n/a	n/a	n/a	n/a	n/a
	4	79239	-46720	15914	13467	29327	53046	2392	n/a	n/a	n/a	n/a	n/a	n/a
	3	85418	-50985	17496	15505	31213	58321	2770	n/a	n/a	n/a	n/a	n/a	n/a
	2	91597	-55250	19214	19059	33047	64046	3393	n/a	n/a	n/a	n/a	n/a	n/a
	1	97777	-59515	21040	26013	34729	70131	4526	n/a	n/a	n/a	n/a	n/a	n/a
Structure_08	20	4395	-1893	577	5475	1948	1922	548	n/a	n/a	n/a	n/a	n/a	n/a
	19	8790	-3785	1601	6611	3969	5328	772	n/a	n/a	n/a	n/a	n/a	n/a
	18	13186	-5678	2910	5691	6059	9689	758	n/a	n/a	n/a	n/a	n/a	n/a
	17	17581	-7571	4363	6513	8200	14530	959	n/a	n/a	n/a	n/a	n/a	n/a
	16	23401	-10887	5844	7382	10476	19468	1142	n/a	n/a	n/a	n/a	n/a	n/a
	15	29221	-14203	7270	8299	12761	24226	1328	n/a	n/a	n/a	n/a	n/a	n/a
	14	35041	-17519	8590	9204	15042	28631	1508	n/a	n/a	n/a	n/a	n/a	n/a
	13	40861	-20836	9783	10216	17303	32602	1700	n/a	n/a	n/a	n/a	n/a	n/a
	12	47923	-25393	10842	10562	19604	36131	1747	n/a	n/a	n/a	n/a	n/a	n/a
	11	54985	-29950	11786	11077	21876	39276	1824	n/a	n/a	n/a	n/a	n/a	n/a
	10	62046	-34508	12653	11768	24126	42166	1938	n/a	n/a	n/a	n/a	n/a	n/a
	9	69107	-39065	13505	12868	26357	45008	2125	n/a	n/a	n/a	n/a	n/a	n/a
	8	78544	-45995	14424	13278	28668	48070	2158	n/a	n/a	n/a	n/a	n/a	n/a
	7	87981	-52925	15495	14109	30992	51638	2277	n/a	n/a	n/a	n/a	n/a	n/a
	6	97419	-59854	16792	15226	33353	55960	2462	n/a	n/a	n/a	n/a	n/a	n/a
	5	106856	-66784	18362	16609	35765	61195	2705	n/a	n/a	n/a	n/a	n/a	n/a
	4	116567	-73987	20226	18220	38238	67406	2993	n/a	n/a	n/a	n/a	n/a	n/a
	3	126278	-81190	22374	20623	40744	74559	3414	n/a	n/a	n/a	n/a	n/a	n/a
	2	135989	-88394	24764	24847	43223	82529	4116	n/a	n/a	n/a	n/a	n/a	n/a
	1	145700	-95597	27352	33955	45523	91144	5524	n/a	n/a	n/a	n/a	n/a	n/a

Table C.8. Wall pier extreme fiber concrete stresses and strains.

Extreme Fiber Concrete Stresses and Strains																	
Structure	Floor	C-Shaped Walls										I-Shape Walls					
		Concrete Stress								Concrete Strain				Concrete Stress			
		ACI 21.7.6.3				Fiber Analysis				ACI 21.7.6.3				Fiber Analysis			
		LC1		LC2		LC1		LC2		LC1		LC2		LC1		LC2	
		Point A (MPa)	Point C (MPa)	Point A (MPa)	Point C (MPa)	Point A (MPa)	Point C (MPa)	Point A (MPa)	Point C (MPa)	Point A	Point C	Point A	Point C	Point B (MPa)	Point B (MPa)	Point B (MPa)	Point B (MPa)
Structure 01	20	1.0	-0.7	0.5	-0.9	2.2	0.0	0.4	-1.3	0.081	-0.162	0.015	-0.045	n/a	n/a	n/a	n/a
	19	1.5	-0.8	0.9	-1.6	3.5	0.0	0.9	-1.9	0.126	-0.124	0.031	-0.069	n/a	n/a	n/a	n/a
	18	1.7	-0.4	1.4	-2.1	3.9	0.0	1.3	-2.1	0.142	-0.031	0.048	-0.076	n/a	n/a	n/a	n/a
	17	1.5	0.1	1.9	-2.5	4.1	0.2	1.8	-2.4	0.148	0.008	0.065	-0.085	n/a	n/a	n/a	n/a
	16	1.5	0.6	2.5	-3.1	4.6	0.7	2.3	-2.9	0.166	0.025	0.084	-0.104	n/a	n/a	n/a	n/a
	15	1.5	1.2	3.1	-3.6	4.9	1.2	2.9	-3.4	0.177	0.045	0.103	-0.122	n/a	n/a	n/a	n/a
	14	1.7	1.5	3.7	-4.3	5.8	1.5	3.4	-4.0	0.209	0.056	0.124	-0.146	n/a	n/a	n/a	n/a
	13	2.0	1.7	4.4	-5.1	6.8	1.8	4.1	-4.7	0.237	0.064	0.147	-0.170	n/a	n/a	n/a	n/a
	12	2.1	2.1	5.0	-5.8	7.5	2.2	4.7	-5.4	0.272	0.078	0.169	-0.196	n/a	n/a	n/a	n/a
	11	2.2	2.5	5.7	-6.6	8.2	2.6	5.3	-6.1	0.296	0.093	0.192	-0.222	n/a	n/a	n/a	n/a
	10	2.3	2.9	6.4	-7.4	8.9	3.0	5.9	-6.8	0.322	0.107	0.215	-0.248	n/a	n/a	n/a	n/a
	9	2.5	3.3	7.1	-8.2	10.0	3.3	6.6	-7.6	0.363	0.120	0.239	-0.275	n/a	n/a	n/a	n/a
	8	2.7	3.6	7.8	-9.0	8.5	3.6	7.1	-8.1	0.307	0.131	0.257	-0.295	n/a	n/a	n/a	n/a
	7	2.9	3.9	8.5	-9.9	9.3	3.9	7.8	-8.9	0.337	0.142	0.283	-0.324	n/a	n/a	n/a	n/a
	6	3.3	4.2	9.4	-10.8	10.4	4.2	8.5	-9.7	0.380	0.150	0.310	-0.354	n/a	n/a	n/a	n/a
	5	3.8	4.2	10.2	-11.8	12.4	4.3	9.3	-10.7	0.452	0.154	0.338	-0.388	n/a	n/a	n/a	n/a
	4	4.6	4.0	11.2	-13.0	11.7	4.0	9.9	-11.3	0.426	0.144	0.358	-0.412	n/a	n/a	n/a	n/a
	3	5.8	3.4	12.2	-14.3	14.0	3.6	10.7	-12.4	0.514	0.128	0.390	-0.454	n/a	n/a	n/a	n/a
	2	7.5	2.3	13.3	-15.9	17.5	2.6	11.7	-13.8	0.649	0.093	0.427	-0.508	n/a	n/a	n/a	n/a
	1	10.1	0.3	14.5	-18.0	26.2	0.6	12.8	-15.8	1.050	0.043	0.469	-0.585	n/a	n/a	n/a	n/a
Structure 02	20	0.7	0.0	0.6	-0.7	0.9	0.0	0.6	-1.3	0.031	0.000	0.020	-0.045	0.9	0.5	1.3	0.5
	19	1.3	0.0	1.2	-1.4	1.6	0.0	1.1	-1.9	0.059	-0.001	0.040	-0.069	1.6	1.0	2.1	1.0
	18	1.9	-0.1	1.8	-2.0	2.4	0.0	1.7	-2.1	0.088	-0.003	0.061	-0.076	2.1	1.5	2.5	1.6
	17	2.5	-0.2	2.5	-2.7	3.3	0.0	2.3	-2.4	0.118	-0.008	0.082	-0.085	2.4	2.0	2.6	2.1
	16	3.1	-0.4	3.2	-3.4	4.3	0.0	2.9	-2.9	0.155	-0.013	0.105	-0.103	2.8	2.5	2.9	2.7
	15	3.7	-0.5	3.9	-4.1	5.2	0.0	3.6	-3.4	0.190	-0.017	0.130	-0.122	3.1	3.1	3.1	3.3
	14	4.3	-0.6	4.6	-4.8	6.2	0.0	4.3	-4.0	0.226	-0.019	0.154	-0.145	3.4	3.6	3.3	3.9
	13	4.8	-0.7	5.4	-5.5	7.0	0.0	4.9	-4.7	0.255	-0.020	0.179	-0.169	3.6	4.2	3.4	4.6
	12	5.3	-0.7	6.2	-6.3	8.1	0.0	5.7	-5.4	0.292	-0.018	0.205	-0.195	4.0	4.8	3.7	5.3
	11	5.8	-0.7	7.0	-7.1	9.0	0.0	6.4	-6.1	0.326	-0.015	0.232	-0.221	4.4	5.4	4.1	6.0
	10	6.3	-0.8	7.8	-7.9	10.1	0.0	7.2	-6.8	0.366	-0.026	0.260	-0.247	4.7	6.1	4.4	6.8
	9	7.4	-1.3	8.7	-9.0	11.9	0.0	8.0	-7.6	0.434	-0.056	0.290	-0.274	5.4	6.8	5.1	7.7
	8	8.5	-1.9	9.6	-10.1	13.2	0.0	8.6	-8.1	0.482	-0.082	0.312	-0.294	5.9	7.5	5.6	8.6
	7	9.8	-2.7	10.5	-11.3	15.3	0.0	9.4	-8.9	0.562	-0.134	0.343	-0.322	6.5	8.2	6.2	9.5
	6	11.3	-3.6	11.5	-12.6	17.6	0.0	10.3	-9.7	0.654	-0.210	0.375	-0.353	7.1	9.0	6.9	10.5
	5	13.0	-4.7	12.5	-13.9	20.1	0.0	11.2	-10.7	0.759	-0.318	0.409	-0.386	8.0	9.9	7.9	11.6
	4	14.9	-6.0	13.7	-15.4	20.1	0.0	11.8	-11.3	0.760	-0.388	0.430	-0.411	9.1	10.9	8.6	11.5
	3	17.2	-7.5	14.9	-17.0	22.8	0.0	12.8	-12.4	0.881	-0.549	0.470	-0.452	10.6	11.9	10.2	12.7
	2	19.8	-9.2	16.3	-18.8	25.7	0.0	14.0	-13.8	1.025	-0.762	0.515	-0.506	12.5	13.1	12.7	14.1
	1	22.9	-11.1	18.1	-20.8	28.7	0.0	15.5	-15.8	1.201	-1.024	0.572	-0.583	15.0	14.4	15.9	15.4
Structure 03	20	0.4	0.3	0.5	-0.5	0.3	0.3	0.5	-0.5	0.013	0.010	0.017	-0.018	0.6	0.4	0.7	0.3
	19	0.7	0.5	1.0	-1.0	0.7	0.5	0.9	-1.0	0.025	0.019	0.034	-0.035	1.1	0.7	1.3	0.7
	18	1.1	0.7	1.5	-1.5	1.0	0.7	1.4	-1.4	0.036	0.026	0.051	-0.051	1.6	1.1	1.7	1.1
	17	1.4	0.8	2.0	-2.0	1.3	0.9	1.9	-1.9	0.048	0.032	0.069	-0.067	1.9	1.5	2.0	1.4
	16	1.8	1.0	2.6	-2.5	1.7	1.0	2.5	-2.4	0.060	0.037	0.089	-0.085	2.3	1.9	2.2	1.8
	15	2.1	1.1	3.1	-3.0	1.4	1.2	3.0	-2.8	0.072	0.043	0.109	-0.102	2.5	2.3	2.4	2.2
	14	2.4	1.2	3.7	-3.5	2.3	1.3	3.6	-3.3	0.084	0.048	0.129	-0.119	2.8	2.7	2.6	2.6
	13	2.8	1.4	4.3	-4.0	2.6	1.5	4.1	-3.8	0.095	0.053	0.150	-0.136	3.0	3.1	2.8	3.0
	12	3.2	1.4	4.9	-4.5	3.1	1.5	4.7	-4.3	0.111	0.054	0.170	-0.154	3.1	3.5	3.0	3.5
	11	3.8	1.2	5.5	-5.1	3.7	1.4	5.3	-4.9	0.135	0.051	0.191	-0.175	3.3	3.9	3.1	3.9
	10	4.4	1.1	6.2	-5.8	4.7	1.3	5.9	-5.5	0.169	0.047	0.213	-0.197	3.6	4.4	3.6	4.5
	9	5.1	0.9	6.9	-6.5	5.9	1.2	6.6	-6.1	0.213	0.043	0.238	-0.220	4.1	5.0	3.9	5.1
	8	5.8	0.7	7.6	-7.2	7.4	1.1	7.3	-6.8	0.266	0.038	0.264	-0.245	4.7	5.5	4.4	5.8
	7	6.7	0.5	8.4	-8.0	9.0	0.9	8.0	-7.5	0.327	0.032	0.290	-0.271	5.4	6.1	5.1	6.5
	6	7.6	0.2	9.2	-8.8	10.8	0.6	8.8	-8.3	0.395	0.023	0.318	-0.299	6.2	6.8	5.9	7.3
	5	8.6	-0.2	10.0	-9.6	12.8	0.0	9.6	-9.1	0.470	-0.001	0.347	-0.329	7.0	7.4	6.9	8.2
	4	9.7	-0.6	10.9	-10.5	15.0	0.0	10.4	-10.0	0.552	-0.008	0.378	-0.362	8.1	8.2	8.2	9.2
	3	10.9	-1.0	11.9	-11.5	17.3	0.0	11.3	-11.0	0.643	-0.034	0.412	-0.398	9.3	8.9	9.8	10.2
	2	12.3	-1.5	12.9	-12.5	19.7	0.0	12.3	-12.1	0.744	-0.073	0.448	-0.440	10.6	9.7	11.7	11.3
	1	13.8	-2.0	14.1	-13.6	22.4	0.0	13.3	-13.4	0.861	-0.131	0.489	-0.491	12.0	10.5	13.8	12.3

Table C.8 (cont.). Wall pier extreme fiber concrete stresses and strains.

		Extreme Fiber Concrete Stresses and Strains															
Structure	Floor	C-Shaped Walls								I-Shape Walls							
		Concrete Stress				Concrete Strain				Concrete Stress				Concrete Strain			
		ACI 21.7.6.3				Fiber Analysis				ACI 21.7.6.3				Fiber Analysis			
		LC1		LC2		LC1		LC2		LC1		LC2		LC1		LC2	
		Point A (MPa)	Point C (MPa)	Point A (MPa)	Point C (MPa)	Point A (MPa)	Point C (MPa)	Point A (MPa)	Point C (MPa)	Point A	Point C	Point A	Point C	Point B (MPa)	Point B (MPa)	Point B (MPa)	Point B (MPa)
Structure 04	20	1.5	-1.0	0.6	-1.4	3.0	0.0	0.6	-1.8	0.107	-0.230	0.020	-0.066	n/a	n/a	n/a	n/a
	19	2.0	-1.0	1.2	-2.4	4.3	0.0	1.1	-2.6	0.154	-0.131	0.041	-0.093	n/a	n/a	n/a	n/a
	18	1.9	-0.3	1.9	-3.0	4.5	0.0	1.7	-2.8	0.162	-0.001	0.063	-0.101	n/a	n/a	n/a	n/a
	17	1.4	0.8	2.6	-3.5	4.1	0.9	2.4	-3.2	0.149	0.031	0.085	-0.114	n/a	n/a	n/a	n/a
	16	1.6	1.3	3.4	-4.5	4.9	1.3	3.1	-4.1	0.178	0.049	0.112	-0.147	n/a	n/a	n/a	n/a
	15	1.9	1.6	4.2	-5.5	6.0	1.7	3.9	-5.1	0.215	0.062	0.140	-0.183	n/a	n/a	n/a	n/a
	14	2.2	2.0	5.1	-6.6	7.0	2.1	4.7	-6.1	0.254	0.074	0.169	-0.218	n/a	n/a	n/a	n/a
	13	2.6	2.2	5.9	-7.7	8.2	2.4	5.5	-7.1	0.280	0.085	0.198	-0.255	n/a	n/a	n/a	n/a
	12	2.5	3.0	6.7	-8.8	7.4	3.0	6.1	-8.0	0.268	0.109	0.222	-0.287	n/a	n/a	n/a	n/a
	11	2.4	3.6	7.6	-10.0	7.5	3.7	6.9	-9.0	0.272	0.131	0.249	-0.325	n/a	n/a	n/a	n/a
	10	2.5	4.3	8.4	-11.1	8.0	4.3	7.6	-10.0	0.290	0.154	0.277	-0.363	n/a	n/a	n/a	n/a
	9	2.5	4.8	9.3	-12.3	10.4	4.8	8.4	-11.1	0.377	0.174	0.306	-0.402	n/a	n/a	n/a	n/a
	8	2.4	5.6	10.1	-13.5	6.1	5.3	8.8	-11.6	0.220	0.191	0.319	-0.422	n/a	n/a	n/a	n/a
	7	2.4	6.4	11.1	-14.8	6.1	5.9	9.6	-12.7	0.221	0.215	0.349	-0.463	n/a	n/a	n/a	n/a
	6	2.5	7.0	12.0	-16.2	6.5	6.5	10.5	-13.8	0.236	0.236	0.381	-0.506	n/a	n/a	n/a	n/a
	5	2.7	7.5	13.1	-17.6	7.9	7.0	11.4	-15.0	0.288	0.254	0.415	-0.553	n/a	n/a	n/a	n/a
	4	3.2	7.7	14.3	-19.2	6.0	6.7	11.4	-15.1	0.216	0.242	0.416	-0.554	n/a	n/a	n/a	n/a
	3	4.2	7.6	15.5	-21.0	7.6	6.6	12.4	-16.4	0.276	0.240	0.453	-0.607	n/a	n/a	n/a	n/a
	2	5.9	6.7	16.9	-23.1	10.4	6.0	13.5	-18.0	0.380	0.218	0.492	-0.671	n/a	n/a	n/a	n/a
	1	9.0	4.4	18.5	-26.0	13.5	4.3	14.8	-20.1	0.496	0.157	0.544	-0.759	n/a	n/a	n/a	n/a
Structure 05	20	0.5	0.2	0.6	-0.62	0.5	0.2	0.6	-0.6	0.018	0.008	0.020	-0.021	1.1	0.6	1.6	0.6
	19	1.0	0.4	1.3	-1.25	1.0	0.4	1.2	-1.2	0.037	0.015	0.043	-0.041	1.7	1.1	2.0	1.0
	18	1.4	0.5	1.9	-1.84	1.5	0.6	1.8	-1.7	0.054	0.030	0.064	-0.061	2.1	1.6	2.3	1.5
	17	1.8	0.6	2.5	-2.42	2.0	0.7	2.4	-2.2	0.071	0.024	0.085	-0.080	2.4	2.0	2.4	2.0
	16	2.3	0.7	3.3	-3.08	2.7	0.8	3.0	-2.8	0.098	0.028	0.109	-0.102	2.8	2.6	2.8	2.6
	15	2.7	0.8	4.0	-3.74	3.5	0.9	3.7	-3.5	0.126	0.032	0.134	-0.124	3.2	3.2	3.1	3.2
	14	3.1	0.8	4.7	-4.39	4.2	1.0	4.4	-4.1	0.153	0.036	0.158	-0.146	3.5	3.7	3.3	3.8
	13	3.5	0.9	5.4	-5.04	4.9	1.1	5.1	-4.7	0.179	0.040	0.183	-0.168	3.7	4.3	3.5	4.5
	12	3.9	1.0	6.2	-5.78	5.9	1.3	5.8	-5.3	0.213	0.046	0.210	-0.192	4.1	4.9	3.9	5.2
	11	4.4	1.1	7.1	-6.57	7.0	1.4	6.6	-6.1	0.253	0.049	0.239	-0.219	4.6	5.6	4.3	6.0
	10	5.1	0.9	7.9	-7.47	8.5	1.3	7.4	-6.9	0.309	0.046	0.268	-0.248	5.1	6.3	4.7	6.8
	9	5.9	0.7	8.8	-8.42	10.1	1.1	8.2	-7.7	0.370	0.041	0.299	-0.280	5.6	7.0	5.3	7.7
	8	6.7	0.4	9.8	-9.40	11.9	0.9	9.1	-8.6	0.433	0.034	0.330	-0.312	6.1	7.8	5.7	8.7
	7	7.7	0.1	10.7	-10.44	13.8	0.7	10.0	-9.6	0.505	0.025	0.363	-0.347	6.7	8.5	6.3	9.6
	6	8.7	-0.3	11.7	-11.53	15.8	0.3	10.9	-10.6	0.584	0.011	0.398	-0.385	7.3	9.3	6.9	10.7
	5	9.9	-0.8	12.8	-12.70	18.0	0.0	12.1	-11.7	0.674	-0.009	0.436	-0.425	8.0	10.2	7.6	11.8
	4	11.3	-1.4	14.0	-13.94	20.6	0.0	13.0	-12.9	0.780	-0.040	0.477	-0.471	9.0	11.1	8.6	13.0
	3	12.9	-2.1	15.3	-15.27	23.3	0.0	14.2	-14.3	0.907	-0.088	0.522	-0.523	10.1	12.1	9.9	14.4
	2	14.7	-2.9	16.8	-16.68	27.5	0.0	15.5	-15.8	1.126	-0.155	0.573	-0.583	11.7	13.3	11.7	16.0
	1	16.8	-3.7	18.4	-18.17	34.3	0.0	17.0	-17.6	1.883	-0.271	0.630	-0.655	14.1	14.7	15.2	18.0
Structure 06	20	0.5	0.1	0.5	-0.6	0.6	0.2	0.5	-0.6	0.029	0.006	0.018	-0.020	0.9	0.5	1.1	0.5
	19	1.0	0.3	1.1	-1.2	1.1	0.3	1.0	-1.1	0.040	0.010	0.037	-0.039	1.5	0.9	2.0	0.9
	18	1.5	0.3	1.7	-1.7	1.6	0.3	1.6	-1.6	0.058	0.013	0.056	-0.057	1.9	1.4	2.1	1.3
	17	1.9	0.3	2.3	-2.3	2.1	0.4	2.1	-2.1	0.077	0.014	0.076	-0.075	2.2	1.8	2.2	1.8
	16	2.4	0.3	2.9	-2.9	2.8	0.4	2.7	-2.6	0.103	0.015	0.097	-0.095	2.6	2.3	2.6	2.4
	15	2.9	0.3	3.6	-3.5	3.5	0.4	3.3	-3.2	0.128	0.016	0.120	-0.115	2.9	2.8	2.8	3.0
	14	3.3	0.3	4.3	-4.1	4.2	0.5	3.9	-3.7	0.152	0.018	0.142	-0.135	3.1	3.3	3.0	3.5
	13	3.7	0.3	4.9	-4.7	4.8	0.6	4.6	-4.3	0.175	0.020	0.164	-0.154	3.3	3.8	3.1	4.0
	12	4.1	0.4	5.7	-5.4	5.6	0.7	5.2	-4.9	0.202	0.024	0.189	-0.176	3.7	4.4	3.4	4.7
	11	4.5	0.4	6.4	-6.0	6.4	0.7	5.9	-5.5	0.230	0.027	0.213	-0.197	4.0	5.0	3.7	5.3
	10	5.2	0.2	7.2	-6.9	7.7	0.6	6.6	-6.2	0.279	0.020	0.239	-0.225	4.4	5.6	4.1	6.0
	9	6.0	-0.1	8.0	-7.7	9.2	0.4	7.3	-7.0	0.333	0.002	0.266	-0.254	4.9	6.2	4.5	6.9
	8	6.9	-0.5	8.8	-8.7	10.8	0.0	8.1	-7.9	0.392	-0.003	0.293	-0.286	5.4	6.9	5.0	7.6
	7	7.9	-1.0	9.6	-9.7	12.6	0.0	8.9	-8.9	0.459	-0.023	0.323	-0.320	6.0	7.6	5.6	8.4
	6	9.0	-1.5	10.6	-10.7	14.5	0.0	9.7	-9.9	0.534	-0.054	0.353	-0.359	6.7	8.3	6.3	9.4
	5	10.3	-2.2	11.5	-11.9	16.7	0.0	10.6	-11.1	0.618	-0.099	0.386	-0.402	7.5	9.1	7.2	10.4
	4	11.8	-2.9	12.6	-13.1	19.0	0.0	11.5	-12.4	0.715	-0.169	0.421	-0.453	8.5	10.0	8.4	11.5
	3	13.4	-3.8	13.7	-14.4	21.6	0.0	12.6	-14.0	0.826	-0.274	0.460	-0.512	9.8	11.0	10.0	12.7
	2	15.3	-4.8	15.0	-15.8	24.4	0.0	13.7	-15.9	0.956	-0.438	0.503	-0.587	11.5	12.0	12.3	14.1
	1	17.6	-5.8	16.5	-17.4	27.3	0.0	15.1	-18.3	1.113	-0.686	0.555	-0.681	13.6	13.2	15.4	15.7

Table C.8 (cont.). Wall pier extreme fiber concrete stresses and strains.

Extreme Fiber Concrete Stresses and Strains																					
Structure	Floor	C-Shaped Walls												I-Shape Walls							
		Concrete Stress								Concrete Strain				Concrete Stress				Concrete Strain			
		ACI 21.7.6.3				Fiber Analysis				ACI 21.7.6.3				Fiber Analysis							
		LC1		LC2		LC1		LC2		LC1		LC2		LC1		LC2		LC1		LC2	
		Point A (MPa)	Point C (MPa)	Point A (MPa)	Point C (MPa)	Point A (MPa)	Point C (MPa)	Point A (MPa)	Point C (MPa)	Point A	Point C	Point A	Point C	Point A	Point C	Point B (MPa)	Point B (MPa)	Point B (MPa)	Point B (MPa)	Point B	Point B
Structure_07	20	2.3	-1.8	0.7	-2.0	3.9	0.0	0.6	-2.8	0.140	-0.360	0.023	-0.100	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	19	2.9	-1.9	1.5	-3.1	5.3	0.0	1.3	-2.8	0.191	-0.116	0.048	-0.102	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	18	2.6	-0.9	2.2	-3.7	5.3	0.0	2.0	-3.7	0.190	-0.073	0.073	-0.132	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	17	1.9	0.4	3.0	-4.2	4.7	0.4	2.3	-3.8	0.169	0.016	0.099	-0.135	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	16	2.2	0.8	3.9	-5.4	5.8	0.8	3.6	-4.8	0.208	0.029	0.130	-0.173	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	15	2.6	1.1	4.9	-6.6	6.8	1.2	4.4	-5.9	0.245	0.042	0.159	-0.211	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	14	2.9	1.5	5.8	-7.8	7.8	1.5	5.2	-6.9	0.281	0.054	0.189	-0.249	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	13	3.3	1.7	6.7	-9.0	8.9	1.7	6.1	-8.0	0.323	0.063	0.220	-0.289	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	12	2.9	2.7	7.5	-10.2	8.4	2.6	6.8	-9.0	0.313	0.096	0.245	-0.327	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	11	2.6	3.6	8.3	-11.4	8.3	3.5	7.5	-10.1	0.301	0.125	0.271	-0.366	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	10	2.5	4.4	9.2	-12.6	10.1	4.2	8.2	-11.2	0.366	0.151	0.298	-0.408	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	9	2.5	5.0	10.0	-14.0	10.8	4.7	9.0	-12.4	0.389	0.171	0.327	-0.452	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	8	2.0	6.1	10.8	-15.4	4.7	5.4	9.2	-12.8	0.172	0.196	0.333	-0.467	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	7	1.7	7.1	11.7	-16.8	3.9	6.3	9.9	-14.1	0.141	0.227	0.361	-0.514	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	6	1.5	7.9	12.7	-18.4	4.1	7.0	10.8	-15.3	0.147	0.254	0.392	-0.564	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	5	1.5	8.7	13.9	-20.1	6.8	7.7	11.6	-16.7	0.245	0.278	0.425	-0.618	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	4	1.7	9.3	15.1	-21.9	3.0	7.7	12.0	-17.2	0.107	0.281	0.438	-0.638	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	3	2.4	9.5	16.5	-24.0	4.6	7.9	13.1	-18.7	0.166	0.287	0.478	-0.700	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	2	4.1	8.7	18.1	-26.6	9.3	7.4	14.3	-20.6	0.338	0.266	0.526	-0.779	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	1	7.8	5.9	20.0	-30.2	27.1	5.1	15.9	-23.1	1.102	0.185	0.589	-0.891	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
Structure_08	20	3.1	-2.4	1.0	-2.7	5.3	0.0	0.9	-3.8	0.193	-0.493	0.033	-0.137	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	19	3.6	-2.1	2.0	-4.0	6.8	0.0	1.9	-4.8	0.247	-0.320	0.067	-0.172	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	18	2.9	-0.5	3.1	-4.8	6.5	0.0	2.9	-4.5	0.234	-0.032	0.103	-0.161	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	17	3.2	0.0	4.4	-6.1	9.2	0.0	4.0	-5.5	0.335	-0.001	0.144	-0.200	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	16	3.3	0.8	5.6	-7.7	7.9	0.8	5.0	-6.8	0.285	0.030	0.180	-0.244	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	15	3.5	1.6	6.9	-9.4	8.7	1.6	6.0	-8.2	0.316	0.057	0.218	-0.295	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	14	3.6	2.3	8.1	-11.0	9.3	2.3	7.1	-9.6	0.338	0.083	0.256	-0.346	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	13	3.8	3.0	9.3	-12.6	10.0	2.9	8.1	-11.0	0.362	0.105	0.293	-0.398	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	12	3.3	4.3	10.3	-14.2	6.6	3.8	8.4	-11.5	0.238	0.137	0.305	-0.419	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	11	2.9	5.5	11.2	-15.9	6.0	4.8	9.2	-12.9	0.216	0.173	0.334	-0.469	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	10	2.5	6.5	12.2	-17.6	5.4	5.7	10.0	-14.2	0.197	0.206	0.364	-0.522	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	9	2.5	7.4	13.1	-19.4	5.3	6.4	10.9	-15.7	0.192	0.231	0.396	-0.578	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	8	1.5	9.0	14.1	-21.5	1.9	6.9	10.3	-15.4	0.070	0.251	0.376	-0.568	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	7	0.9	10.5	15.1	-23.7	0.6	8.0	11.1	-17.0	0.021	0.291	0.405	-0.630	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	6	0.4	11.8	16.3	-26.1	0.0	9.0	12.0	-18.7	-0.023	0.327	0.438	-0.697	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	5	0.2	13.0	17.8	-28.6	0.0	9.9	13.0	-20.3	-0.051	0.361	0.475	-0.768	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	4	0.1	14.2	19.3	-31.2	0.0	10.6	13.8	-21.7	-0.057	0.385	0.508	-0.828	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	3	0.5	14.9	21.2	-34.2	0.0	11.1	15.1	-23.6	-0.039	0.405	0.556	-0.913	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	2	2.1	14.5	23.3	-37.8	3.3	10.9	16.5	-25.6	0.118	0.397	0.613	-1.018	n/a	n/a	n/a	n/a	n/a	n/a	n/a	
	1	6.7	11.1	25.9	-42.9	16.7	8.5	18.4	-28.3	0.620	0.309	0.688	-1.172	n/a	n/a	n/a	n/a	n/a	n/a	n/a	

Table C.9. Wall pier steel reinforcement extreme stresses and strains.

Extreme Steel Stresses and Strains													
Structure	Floor	C-Shaped Walls								I-Shape Walls			
		Steel Stress				Steel Strain				Steel Stress		Steel Strain	
		LC1		LC2		LC1		LC2		LC1	LC2	LC1	LC2
		Point A (MPa)	Point C (MPa)	Point A (MPa)	Point C (MPa)	Point A	Point C	Point A	Point C	Point B (MPa)	Point B (MPa)	Point B	Point B
Structure_01	20	16.1	-32.4	3.1	-9.1	0.081	-0.162	0.015	-0.045	n/a	n/a	n/a	n/a
	19	25.2	-24.7	6.3	-13.8	0.126	-0.124	0.032	-0.069	n/a	n/a	n/a	n/a
	18	28.4	-6.1	9.6	-15.2	0.142	-0.031	0.048	-0.076	n/a	n/a	n/a	n/a
	17	29.7	1.1	13.0	-17.0	0.149	0.006	0.065	-0.085	n/a	n/a	n/a	n/a
	16	33.4	4.6	23.6	-20.7	0.168	0.023	0.084	-0.104	n/a	n/a	n/a	n/a
	15	35.8	8.5	20.6	-24.5	0.179	0.043	0.104	-0.122	n/a	n/a	n/a	n/a
	14	42.3	10.5	24.9	-29.1	0.212	0.053	0.125	-0.146	n/a	n/a	n/a	n/a
	13	50.1	12.1	29.3	-34.0	0.251	0.061	0.147	-0.170	n/a	n/a	n/a	n/a
	12	55.3	14.9	33.8	-39.1	0.278	0.075	0.169	-0.196	n/a	n/a	n/a	n/a
	11	60.6	17.7	38.3	-44.3	0.304	0.089	0.192	-0.222	n/a	n/a	n/a	n/a
	10	66.1	20.4	43.0	-49.5	0.332	0.102	0.216	-0.248	n/a	n/a	n/a	n/a
	9	74.7	23.0	47.8	-54.9	0.375	0.115	0.240	-0.275	n/a	n/a	n/a	n/a
	8	62.8	25.0	51.4	-58.9	0.315	0.126	0.258	-0.295	n/a	n/a	n/a	n/a
	7	69.1	27.2	56.5	-64.7	0.347	0.137	0.284	-0.324	n/a	n/a	n/a	n/a
	6	77.8	28.7	62.0	-70.9	0.390	0.144	0.311	-0.354	n/a	n/a	n/a	n/a
	5	92.4	29.3	67.6	-77.6	0.464	0.147	0.339	-0.388	n/a	n/a	n/a	n/a
	4	86.5	27.2	71.6	-82.5	0.434	0.136	0.359	-0.412	n/a	n/a	n/a	n/a
	3	104.1	23.6	78.1	-90.7	0.522	0.118	0.392	-0.454	n/a	n/a	n/a	n/a
	2	131.3	16.5	85.4	-101.6	0.659	0.083	0.429	-0.508	n/a	n/a	n/a	n/a
	1	212.0	1.2	94.0	-117.0	1.064	0.006	0.472	-0.585	n/a	n/a	n/a	n/a
Structure_02	20	6.2	-0.2	4.0	-4.8	0.031	-0.001	0.020	-0.024	9.1	3.7	0.000	0.000
	19	11.9	-0.5	8.1	-9.2	0.059	-0.003	0.040	-0.046	15.1	7.5	0.046	0.019
	18	17.6	-1.2	12.2	-13.5	0.088	-0.006	0.061	-0.067	18.0	11.2	0.076	0.038
	17	23.4	-2.3	16.5	-17.7	0.118	-0.012	0.083	-0.089	18.7	15.1	0.090	0.056
	16	30.8	-3.6	21.2	-22.6	0.154	-0.018	0.107	-0.113	20.9	19.7	0.094	0.076
	15	37.7	-4.6	26.0	-27.3	0.185	-0.023	0.130	-0.136	22.4	24.1	0.105	0.099
	14	44.3	-5.2	30.8	-31.9	0.223	-0.026	0.155	-0.159	23.7	28.5	0.113	0.121
	13	50.6	-5.5	35.7	-36.4	0.254	-0.028	0.179	-0.182	24.6	32.9	0.119	0.143
	12	58.0	-5.4	41.0	-41.5	0.291	-0.027	0.206	-0.208	27.1	38.1	0.124	0.165
	11	64.6	-4.9	46.4	-46.4	0.324	-0.025	0.233	-0.232	29.5	43.4	0.136	0.191
	10	72.6	-5.1	52.0	-51.8	0.364	-0.025	0.261	-0.259	31.7	49.3	0.148	0.218
	9	86.1	-10.9	57.9	-59.2	0.432	-0.055	0.291	-0.296	36.7	55.5	0.159	0.247
	8	95.7	-16.1	62.3	-65.7	0.480	-0.081	0.313	-0.329	40.4	61.9	0.184	0.279
	7	111.6	-26.4	68.5	-74.5	0.560	-0.132	0.344	-0.373	44.8	68.7	0.203	0.311
	6	129.6	-41.4	74.9	-84.3	0.651	-0.208	0.376	-0.422	50.1	76.1	0.225	0.345
	5	150.5	-62.8	81.8	-95.7	0.755	-0.315	0.410	-0.479	60.7	84.2	0.252	0.382
	4	150.7	-76.6	86.0	-103.1	0.756	-0.385	0.432	-0.516	62.3	83.4	0.286	0.423
	3	174.9	-108.7	94.0	-117.1	0.874	-0.548	0.472	-0.586	74.6	92.5	0.313	0.418
	2	203.6	-150.9	103.2	-133.3	1.022	-0.757	0.518	-0.667	92.9	103.3	0.374	0.464
	1	238.6	-202.8	114.7	-152.9	1.197	-1.017	0.576	-0.765	117.7	113.1	0.466	0.518

Table C.9 (cont.). Wall pier steel reinforcement extreme stresses and strains.

Extreme Steel Stresses and Strains													
Structure	Floor	C-Shaped Walls								I-Shape Walls			
		Steel Stress				Steel Strain				Steel Stress		Steel Strain	
		LC1		LC2		LC1		LC2		LC1	LC2	LC1	LC2
		Point A (MPa)	Point C (MPa)	Point A (MPa)	Point C (MPa)	Point A	Point C	Point A	Point C	Point B (MPa)	Point B (MPa)	Point B	Point B
Structure_03	20	2.6	2.0	3.4	-3.6	0.013	0.010	0.017	-0.018	5.3	2.5	0.000	0.000
	19	5.0	3.7	6.8	-7.0	0.025	0.018	0.034	-0.035	9.7	5.1	0.027	0.013
	18	7.3	5.0	10.3	-10.3	0.037	0.025	0.052	-0.052	12.5	7.7	0.049	0.026
	17	9.7	6.0	14.0	-13.6	0.049	0.030	0.070	-0.068	14.3	10.4	0.063	0.039
	16	12.2	7.0	17.8	-17.1	0.061	0.035	0.090	-0.085	16.2	13.2	0.072	0.052
	15	14.6	8.0	21.8	-20.5	0.073	0.040	0.109	-0.103	17.7	16.0	0.081	0.066
	14	16.9	8.9	25.8	-23.9	0.085	0.044	0.130	-0.120	19.2	19.0	0.089	0.080
	13	19.1	9.9	29.9	-27.3	0.096	0.050	0.150	-0.137	20.3	22.1	0.096	0.095
	12	22.4	10.0	34.1	-31.1	0.112	0.050	0.171	-0.156	21.5	25.4	0.102	0.111
	11	27.2	9.2	38.3	-35.2	0.137	0.046	0.192	-0.176	22.6	28.8	0.108	0.127
	10	34.0	8.2	42.8	-39.6	0.171	0.041	0.215	-0.198	26.6	32.7	0.113	0.145
	9	43.2	7.2	47.7	-44.4	0.217	0.036	0.239	-0.222	28.2	37.3	0.133	0.164
	8	54.0	5.9	52.8	-49.5	0.271	0.030	0.265	-0.247	32.2	42.4	0.141	0.187
	7	66.5	4.3	58.2	-54.7	0.339	0.022	0.292	-0.274	37.2	47.9	0.162	0.213
	6	80.4	2.3	63.8	-60.5	0.404	0.012	0.320	-0.303	43.2	54.0	0.186	0.241
	5	95.8	-0.5	69.7	-66.5	0.481	-0.003	0.350	-0.332	50.5	60.5	0.217	0.280
	4	112.8	-4.7	76.0	-73.2	0.566	-0.023	0.381	-0.366	60.1	67.8	0.253	0.304
	3	131.5	-10.4	82.8	-80.7	0.660	-0.052	0.415	-0.404	72.0	75.6	0.301	0.340
	2	152.4	-18.9	90.1	-89.3	0.765	-0.095	0.452	-0.447	86.5	83.7	0.361	0.379
	1	176.4	-31.3	98.5	-99.8	0.885	-0.157	0.492	-0.499	102.9	91.9	0.434	0.420
Structure_04	20	21.3	-45.9	4.0	-13.3	0.107	-0.230	0.020	-0.066	n/a	n/a	n/a	n/a
	19	30.8	-26.1	8.3	-18.7	0.154	-0.131	0.041	-0.094	n/a	n/a	n/a	n/a
	18	32.4	-2.6	12.6	-20.2	0.163	-0.013	0.063	-0.101	n/a	n/a	n/a	n/a
	17	30.0	5.7	17.1	-22.9	0.151	0.029	0.086	-0.115	n/a	n/a	n/a	n/a
	16	36.2	9.1	22.4	-29.5	0.182	0.046	0.112	-0.148	n/a	n/a	n/a	n/a
	15	43.8	11.6	28.0	-36.6	0.220	0.058	0.141	-0.183	n/a	n/a	n/a	n/a
	14	51.8	13.9	33.8	-43.8	0.260	0.070	0.170	-0.219	n/a	n/a	n/a	n/a
	13	60.8	15.9	39.6	-51.2	0.305	0.080	0.199	-0.256	n/a	n/a	n/a	n/a
	12	54.9	20.6	44.4	-57.6	0.275	0.103	0.223	-0.288	n/a	n/a	n/a	n/a
	11	56.2	25.1	49.8	-65.1	0.282	0.126	0.250	-0.326	n/a	n/a	n/a	n/a
	10	60.2	36.2	55.4	-72.8	0.302	0.143	0.278	-0.364	n/a	n/a	n/a	n/a
	9	78.3	33.2	61.2	-80.7	0.393	0.167	0.307	-0.404	n/a	n/a	n/a	n/a
	8	45.8	36.7	63.8	-84.7	0.230	0.184	0.320	-0.424	n/a	n/a	n/a	n/a
	7	46.3	41.3	69.8	-92.9	0.232	0.207	0.350	-0.465	n/a	n/a	n/a	n/a
	6	49.4	45.4	76.2	-101.6	0.248	0.228	0.382	-0.508	n/a	n/a	n/a	n/a
	5	60.2	48.7	83.0	-110.9	0.302	0.245	0.416	-0.555	n/a	n/a	n/a	n/a
	4	44.7	46.3	83.2	-111.1	0.224	0.232	0.417	-0.556	n/a	n/a	n/a	n/a
	3	56.8	45.5	90.6	-121.8	0.285	0.229	0.454	-0.609	n/a	n/a	n/a	n/a
	2	77.5	40.7	98.9	-134.7	0.389	0.204	0.496	-0.674	n/a	n/a	n/a	n/a
	1	111.8	27.6	109.0	-152.2	0.561	0.138	0.547	-0.761	n/a	n/a	n/a	n/a

Table C.9 (cont.). Wall pier steel reinforcement extreme stresses and strains.

Extreme Steel Stresses and Strains													
Structure	Floor	C-Shaped Walls								I-Shape Walls			
		Steel Stress				Steel Strain				Steel Stress		Steel Strain	
		LC1		LC2		LC1		LC2		LC1	LC2	LC1	LC2
		Point A (MPa)	Point C (MPa)	Point A (MPa)	Point C (MPa)	Point A	Point C	Point A	Point C	Point B (MPa)	Point B (MPa)	Point B	Point B
Structure_05	20	3.7	1.5	4.1	-4.2	0.019	0.007	0.021	-0.021	11.5	4.4	0.000	0.000
	19	7.5	2.8	8.6	-8.4	0.038	0.014	0.043	-0.042	15.0	6.0	0.058	0.022
	18	10.9	3.7	12.9	-12.3	0.055	0.019	0.065	-0.062	16.9	11.1	0.075	0.039
	17	14.5	4.3	17.2	-16.2	0.073	0.022	0.086	-0.081	17.5	14.5	0.085	0.055
	16	20.0	4.9	22.0	-20.6	0.100	0.025	0.110	-0.103	20.4	19.0	0.088	0.073
	15	25.6	5.5	26.9	-25.1	0.129	0.028	0.135	-0.125	22.4	23.5	0.103	0.095
	14	31.1	6.2	31.8	-29.4	0.156	0.031	0.160	-0.147	24.0	28.2	0.113	0.118
	13	36.4	6.8	36.7	-33.8	0.183	0.034	0.184	-0.169	25.3	33.0	0.120	0.142
	12	43.5	7.8	42.2	-38.7	0.218	0.039	0.212	-0.194	28.2	38.4	0.127	0.166
	11	51.8	8.2	47.9	-44.1	0.260	0.041	0.240	-0.220	31.2	44.0	0.142	0.192
	10	63.2	7.3	53.8	-50.1	0.317	0.036	0.270	-0.251	34.7	50.3	0.157	0.221
	9	75.6	6.0	60.0	-56.4	0.380	0.030	0.301	-0.282	38.5	57.1	0.174	0.252
	8	88.9	4.3	66.3	-63.0	0.446	0.022	0.333	-0.315	42.0	63.9	0.193	0.286
	7	103.7	2.0	72.9	-70.1	0.520	0.010	0.366	-0.350	46.0	71.2	0.211	0.321
	6	120.1	0.0	80.0	-77.6	0.602	0.000	0.401	-0.388	50.6	79.1	0.231	0.357
	5	138.6	-5.8	87.5	-85.9	0.696	-0.024	0.439	-0.430	55.8	87.6	0.254	0.397
	4	160.4	-12.6	95.9	-95.2	0.805	-0.063	0.481	-0.476	63.3	97.4	0.280	0.440
	3	186.6	-23.0	105.0	-105.8	0.936	-0.115	0.527	-0.529	73.1	108.3	0.318	0.489
	2	232.1	-38.2	115.4	-118.2	1.165	-0.192	0.579	-0.592	87.2	121.0	0.367	0.543
	1	391.3	-62.3	127.2	-133.1	1.963	-0.313	0.640	-0.666	89.5	137.9	0.437	0.607
Structure_06	20	4.1	1.0	3.6	-4.0	0.021	0.005	0.018	-0.020	8.1	3.3	0.000	0.000
	19	7.9	1.8	7.4	-7.8	0.040	0.009	0.037	-0.039	12.9	6.5	0.041	0.016
	18	11.6	2.2	11.3	-11.4	0.058	0.011	0.057	-0.057	15.4	9.7	0.065	0.033
	17	15.3	2.3	15.2	-15.0	0.077	0.011	0.076	-0.075	16.2	13.0	0.077	0.049
	16	20.6	2.3	19.5	-19.1	0.103	0.012	0.098	-0.095	18.5	17.1	0.081	0.065
	15	25.7	2.5	24.0	-23.0	0.129	0.012	0.120	-0.115	20.2	21.3	0.093	0.086
	14	30.4	2.7	28.4	-27.0	0.153	0.014	0.143	-0.135	21.5	25.3	0.101	0.107
	13	34.9	3.0	32.9	-30.8	0.175	0.015	0.165	-0.154	22.5	29.2	0.108	0.127
	12	40.4	3.6	37.7	-35.2	0.203	0.018	0.183	-0.176	24.7	33.7	0.113	0.146
	11	46.0	4.1	42.6	-39.5	0.231	0.020	0.214	-0.198	26.5	38.3	0.124	0.169
	10	55.7	2.5	47.7	-45.0	0.279	0.012	0.239	-0.225	29.4	43.5	0.133	0.192
	9	66.5	0.3	53.1	-50.9	0.339	0.002	0.267	-0.255	32.9	49.1	0.148	0.218
	8	78.4	-2.7	58.7	-57.2	0.393	-0.037	0.294	-0.286	36.4	54.8	0.165	0.246
	7	91.7	-7.2	64.5	-64.2	0.460	-0.036	0.324	-0.321	40.6	61.0	0.183	0.275
	6	106.7	-13.8	70.6	-71.8	0.535	-0.069	0.354	-0.359	45.9	67.8	0.214	0.306
	5	123.5	-23.4	77.2	-80.5	0.620	-0.118	0.387	-0.403	52.4	75.2	0.230	0.340
	4	142.8	-38.3	84.3	-90.7	0.717	-0.192	0.423	-0.453	61.1	83.5	0.263	0.377
	3	165.2	-60.5	92.0	-102.7	0.829	-0.303	0.462	-0.514	73.1	92.9	0.307	0.419
	2	191.3	-94.8	100.8	-117.7	0.960	-0.476	0.506	-0.589	90.1	103.6	0.367	0.466
	1	222.9	-146.1	111.4	-136.8	1.118	-0.737	0.559	-0.684	114.4	115.7	0.452	0.520

Table C.9 (cont.). Wall pier steel reinforcement extreme stresses and strains.

Extreme Steel Stresses and Strains													
Structure	Floor	C-Shaped Walls								I-Shape Walls			
		Steel Stress				Steel Strain				Steel Stress		Steel Strain	
		LC1		LC2		LC1		LC2		LC1	LC2	LC1	LC2
		Point A (MPa)	Point C (MPa)	Point A (MPa)	Point C (MPa)	Point A	Point C	Point A	Point C	Point B (MPa)	Point B (MPa)	Point B	Point B
Structure_07	20	26.8	-70.7	4.6	-19.0	0.134	-0.355	0.023	-0.095	n/a	n/a	n/a	n/a
	19	36.2	-22.6	9.5	-19.9	0.182	-0.114	0.047	-0.100	n/a	n/a	n/a	n/a
	18	36.0	-14.2	14.4	-26.0	0.181	-0.071	0.072	-0.130	n/a	n/a	n/a	n/a
	17	31.7	3.5	19.6	-26.8	0.159	0.017	0.098	-0.134	n/a	n/a	n/a	n/a
	16	38.9	6.1	25.6	-34.4	0.195	0.031	0.129	-0.172	n/a	n/a	n/a	n/a
	15	45.7	8.7	31.4	-41.9	0.229	0.044	0.158	-0.210	n/a	n/a	n/a	n/a
	14	52.2	11.2	37.3	-49.5	0.262	0.056	0.187	-0.248	n/a	n/a	n/a	n/a
	13	59.9	12.9	43.3	-57.4	0.301	0.065	0.217	-0.287	n/a	n/a	n/a	n/a
	12	55.3	19.6	48.3	-64.9	0.278	0.099	0.242	-0.325	n/a	n/a	n/a	n/a
	11	54.3	25.3	53.5	-72.8	0.273	0.128	0.268	-0.364	n/a	n/a	n/a	n/a
	10	65.9	30.5	58.9	-81.1	0.331	0.153	0.296	-0.406	n/a	n/a	n/a	n/a
	9	75.9	34.6	64.6	-89.9	0.381	0.174	0.324	-0.450	n/a	n/a	n/a	n/a
	8	29.8	39.5	65.9	-93.0	0.150	0.198	0.331	-0.465	n/a	n/a	n/a	n/a
	7	23.3	45.7	71.3	-102.4	0.117	0.229	0.358	-0.512	n/a	n/a	n/a	n/a
	6	23.8	51.2	77.3	-112.3	0.120	0.257	0.388	-0.562	n/a	n/a	n/a	n/a
	5	42.2	56.1	83.8	-123.0	0.212	0.281	0.420	-0.615	n/a	n/a	n/a	n/a
	4	16.8	56.6	86.5	-126.9	0.084	0.284	0.434	-0.635	n/a	n/a	n/a	n/a
	3	28.0	57.9	94.4	-139.4	0.177	0.291	0.440	-0.697	n/a	n/a	n/a	n/a
	2	61.0	54.0	103.8	-154.9	0.306	0.271	0.521	-0.774	n/a	n/a	n/a	n/a
	1	205.4	38.3	116.2	-176.9	1.030	0.192	0.583	-0.885	n/a	n/a	n/a	n/a
Structure_08	20	37.0	-96.7	6.4	-26.0	0.186	-0.485	0.032	-0.130	n/a	n/a	n/a	n/a
	19	46.8	-62.6	13.3	-33.2	0.235	-0.314	0.067	-0.166	n/a	n/a	n/a	n/a
	18	43.9	-6.0	20.4	-31.7	0.220	-0.030	0.102	-0.159	n/a	n/a	n/a	n/a
	17	63.3	0.0	28.4	-39.6	0.318	0.000	0.143	-0.198	n/a	n/a	n/a	n/a
	16	53.4	6.4	35.5	-48.4	0.268	0.032	0.180	-0.242	n/a	n/a	n/a	n/a
	15	58.9	11.8	43.0	-58.5	0.296	0.059	0.216	-0.293	n/a	n/a	n/a	n/a
	14	62.6	17.0	50.4	-68.7	0.314	0.085	0.253	-0.344	n/a	n/a	n/a	n/a
	13	66.7	21.5	57.7	-79.1	0.335	0.108	0.290	-0.395	n/a	n/a	n/a	n/a
	12	43.9	27.9	60.1	-83.2	0.220	0.140	0.302	-0.416	n/a	n/a	n/a	n/a
	11	39.0	35.1	66.0	-93.4	0.196	0.176	0.331	-0.467	n/a	n/a	n/a	n/a
	10	34.8	41.6	71.9	-103.8	0.175	0.209	0.361	-0.519	n/a	n/a	n/a	n/a
	9	33.5	46.7	78.1	-115.1	0.168	0.234	0.392	-0.576	n/a	n/a	n/a	n/a
	8	11.0	50.6	74.3	-113.1	0.055	0.254	0.373	-0.566	n/a	n/a	n/a	n/a
	7	0.9	58.6	79.9	-125.5	0.005	0.294	0.401	-0.628	n/a	n/a	n/a	n/a
	6	-8.1	65.9	86.5	-138.7	-0.040	0.331	0.434	-0.694	n/a	n/a	n/a	n/a
	5	-13.9	72.6	93.9	-152.9	-0.060	0.365	0.471	-0.764	n/a	n/a	n/a	n/a
	4	-15.3	77.6	100.3	-164.9	-0.077	0.389	0.503	-0.825	n/a	n/a	n/a	n/a
	3	-10.2	81.8	109.7	-181.7	-0.051	0.410	0.550	-0.909	n/a	n/a	n/a	n/a
	2	18.1	80.3	120.8	-202.5	0.091	0.401	0.606	-1.013	n/a	n/a	n/a	n/a
	1	114.9	63.3	135.7	-232.8	0.576	0.318	0.681	-1.164	n/a	n/a	n/a	n/a

Figure C.1. General Floor Plan Dimensions. See Table C.1 for dimensions of all structures. (a) one-cell core wall systems. (b) two-cell core wall system.

LC1 = Earthquake in coupled direction including directional effects and *accidental torsion.
 LC2 = Earthquake in uncoupled direction including directional effects and *accidental torsion.
 *Accidental torsion amplifies directional effect.

Figure C.2. Locations of interest for stress calculations and reported stresses and strains for (a) tension wall, (b) compression wall, (c) I-shaped wall, and nomenclature used in equations.

Figure C.3. Design response spectrum wall boundary study structures.

Figure C.4. Example of ETABS model. (a) floor plan. (b) 3-D elevation. Structure_02 shown.

Figure C.5. ELF Story forces and story shears for Type A buildings.

Figure C.6. ELF Story forces and story shears for Type B buildings.

Figure C.7. Interstory drift and roof drift for Type A buildings.

Figure C.8. Interstory drift and roof drift for Type B buildings.

Figure C.9. Extreme fiber concrete compressive stress comparison between fiber analysis and using gross section properties (**Structure_01**).

Figure C.10(a). Extreme fiber concrete fiber analysis and using gross section

compressive stress comparison between properties (C-Shapes of Structure_02).

Figure C.10(b). Extreme fiber concrete compressive stress comparison between fiber analysis and using gross section properties (**I-Shapes of Structure_02**).

Figure C.11(a). Extreme fiber concrete compressive stress comparison between fiber analysis and using gross section properties (**I-Shapes of Structure_03**).

Figure C.11(b). Extreme fiber concrete fiber analysis and using gross section

compressive stress comparison between properties (C-Shapes of Structure_03).

Figure C.12. Extreme fiber concrete fiber analysis and using gross section

compressive stress comparison between properties (**Structure_04**).

Figure C.13(a). Extreme fiber concrete compressive stress comparison between fiber analysis and using gross section properties (C-Shapes of Structure_05).

Figure C.13(b). Extreme fiber concrete compressive stress comparison between fiber analysis and using gross section properties (**I-Shapes of Structure_05**).

Figure C.14(a). Extreme fiber concrete compressive stress comparison between fiber analysis and using gross section properties (**I-Shapes of Structure_06**).

Figure C.14(b). Extreme fiber concrete compressive stress comparison between fiber analysis and using gross section properties (C-Shapes of Structure_06).

Figure C.15. Extreme fiber concrete compressive stress comparison between fiber analysis and using gross section properties (**Structure_07**).

Figure C.16. Extreme fiber concrete compressive stress comparison between fiber analysis and using gross section properties (**Structure_08**).

Figure C.17. Extreme fiber concrete strains at design loads (**Structure_01**).

Figure C.18(a). Extreme fiber concrete strains at design loads (C-Shapes of Structure_02).

Figure C.18(b). Extreme fiber concrete strains at design loads (I-Shapes of Structure_02).

Figure C.19(a). Extreme fiber concrete strains at design loads (I-Shapes of Structure_03).

Figure C.19(b). Extreme fiber concrete strains at design loads (C-Shapes of Structure_03).

Figure C.20. Extreme fiber concrete strains at design loads (**Structure_04**).

Figure C.21(a). Extreme fiber concrete strains at design loads (C-Shapes of Structure_05).

Figure C.21(b). Extreme fiber concrete strains at design loads (I-Shapes of Structure_05).

Figure C.22(a). Extreme fiber concrete strains at design loads (I-Shapes of Structure_06).

Figure X.22(b). Extreme fiber concrete strains at design loads (C-Shapes of Structure_06).

Figure C.23. Extreme fiber concrete strains at design loads (**Structure_07**).

Figure C.24. Extreme fiber concrete strains at design loads (**Structure_08**).

Appendix D: Wall Boundary Detailing Example Problem

Appendix D

Example Boundary Detailing Problem **Examining ACI Requirements for Special Boundary Elements**

Where special boundary elements are required by §21.7.6.2 or §21.7.6.3, §21.7.6.4 shall be satisfied. Where special boundary elements are not required by §21.7.6.2 or §21.7.6.3, §21.7.6.5 shall be satisfied. The following example illustrates how the steel reinforcing details of a wall pier boundary would be detailed according to §21.7.6.5 compared to the detailing requirements where it has been determined that special boundary elements are required.

Example Statement

The tension wall at floor level one of a reinforced concrete building using a coupled core wall system (Structure_01 of this study) has a specified concrete compressive strength of 5,000psi (35MPa). The gross section properties (ignoring steel) are $I_{xx}=4,312ft^4$ ($37.214m^4$), $I_{yy}=1,057ft^4$ ($9.123m^4$). Measuring from the extreme bottom-left corner of the section, $x=4.038ft$ (1.231m) and $y=9.75ft$ (2.972m).

- (a) Using ACI 318-02 provisions, §21.7.6.3, determine the stress at the point located as shown in Figure D.1. If the calculated stress exceeds 20% of the specified concrete compressive strength, detail the reinforcing at the boundary element per ACI 318-02 §21.7.6.4.
- (b) Assume that special boundary elements are not required (this has been determined by performing fiber analysis of the section to determine the state of stress and strain at factored loads). The stress and strain have been found to be moderate, and thus, no significant concrete strength degradation or longitudinal reinforcement buckling is expected. Detail the wall boundary area per ACI 318-02 §21.7.6.5. The value of c (the distance from the extreme compression fiber to the neutral axis) was found, as a result of the fiber analysis, to be 32inches (812.8mm).

The factored loads, wall details, and location of interest for stress calculations are presented in Figure D.1.

Part (a)

First, stress calculations are made, per ACI 318-02 §21.7.6.3, considering the extreme edge of one of the legs of the C-shape wall; this location is of interest as this is the location of the beam-end/wall pier interface. Refer to Figure D.1 for the location where the stress is calculated. §21.7.6.3 states that if the calculated concrete compressive stress at factored loads, using gross section properties and a linear distribution of stress over the depth of the section, exceeds 20% of the specified concrete strength, special boundary elements are required. The calculated concrete compressive stress at the location of

interest is calculated as shown below. Since 20% of the specified concrete strength (5ksi, 35MPa) is 1ksi (7MPa) and the calculated compressive stress is 1.474ksi (10.13MPa), which is greater than 1ksi (7MPa), special boundary elements are required and §21.7.6.4 must be satisfied.

$$\sigma_{cc} = \frac{-6,135 \text{ kips}}{79 \text{ ft}^2} + \frac{(32,469 \text{ kip-ft})(7.962 \text{ ft})}{1,057 \text{ ft}^4} + \frac{(20,081 \text{ kip-ft})(9.75 \text{ ft})}{4312 \text{ ft}^4} \times \left(\frac{1 \text{ ft}^2}{(12 \text{ in})^2} \right)$$

$$\sigma_{cc} = 1.474 \text{ ksi (10.13 MPa)}$$

Figure D.1. Wall geometry and reinforcement details for example problem.

Per §21.7.6.4(a), the boundary element must extend horizontally from the extreme compression fiber a distance not less than the larger of $c - 0.1l_w$ and $c/2$, where c is the distance from the extreme compression fiber to the neutral axis and l_w is length of the entire wall or wall segment considered in the direction of shear force.

$$c - 0.1l_w = 32 \text{ in} - (0.1)(144 \text{ in}) = 17.6 \text{ in (447 mm)}$$

$$\frac{c}{2} = \frac{32 \text{ in}}{2} = 16 \text{ in (406 mm)}$$

$$\therefore \text{Boundary element} \geq 17.6 \text{ in (447 mm)}$$

§21.7.6.4(b) does not apply. In this case, a flanged section is not considered.

§21.7.6.4(c) states that §21.4.4.1-§21.4.4.3 must be satisfied except that ACI EQ. (21-3) need not be satisfied.

§21.4.4.1(a) applies to spiral or circular hoop reinforcement and therefore does not apply. §21.4.4.1(b) states that the total cross-sectional area of rectangular hoop reinforcement shall not be less than that required by ACI EQ. (21-4) (note that ACI EQ. (21-3) is excluded per §21.7.6.4(c)).

$$A_{sh} = 0.09s h_c \left(\frac{f'_c}{f_{yh}} \right) \quad \text{ACI EQ. (21-3)}$$

A_{sh} = total area of transverse steel

h_c = boundary element dimension measured c-c of confining reinf.

s = vertical spacing of transverse reinf.

f'_c = concrete compressive strength

f_{yh} = yield strength of transverse reinf.

Since the boundary element's horizontal dimension must be greater than 17.6in (447mm), and the longitudinal reinforcement is spaced at 9in (229mm) o.c., it is assumed that the boundary element will enclose the first 3 pairs of longitudinal bars from the end of the wall. It is also assumed that the transverse reinforcement will be of the same bar size as the distributed horizontal reinforcement. Therefore, the transverse reinforcement will be #6 bars. Also, the transverse reinforcement in the boundary will consist of 3 legs. Hence, $A_{sh} = 2bars \times 0.44in^2/bar = 0.88in^2$ (568mm²). See the preliminary sketch of the boundary element in Figure D.2.

$$A_{sh} = (0.09)s(24in) \left(\frac{5ksi}{60ksi} \right) = 0.88in^2 \text{ (568mm}^2\text{)}$$

$$s \leq 5.49in \quad \text{say } s = 5in \text{ (127mm)}$$

Figure D.2. Preliminary boundary element dimensions.

§21.4.4.1(c) states the following:

- Transverse reinforcement must be provided by either single or overlapping hoops.
- Crossties of the same bar size and spacing as the hoops shall be permitted.
- Each end of a crosstie must engage a peripheral longitudinal bar.
- Consecutive crossties must be alternated end for end along the longitudinal reinforcement.

§21.4.4.1(d) states that if the design strength of the member core satisfies the requirement of the design loading combinations, including earthquake loading effects, ACI EQ. (21-3) need not be satisfied. This requirement is already exempt per §21.7.6.4 (c).

§21.4.4.1(e) considers effects of concrete cover in excess of 4in. For this design, the concrete cover is less than 4in; therefore, this provision does not apply.

§21.4.4.2 prescribes maximum spacing requirements for the transverse reinforcement in the boundary element and is given as follows.

$$s \leq \text{smaller of } \begin{cases} \frac{1}{4} \text{ of the minimum member dimension} \\ 6d_b \text{ (longitudinal bar diam.)} \\ s_x = 4 + \left(\frac{14 - h_x}{3} \right) \end{cases}$$

Where, s_x = longitudinal spacing of transverse reinforcement. s_x must not be greater than 6in and need not be taken smaller than 4in.

h_x = maximum horizontal spacing of hoops or crosstie legs on all faces of column.

The minimum member dimension is 24in (610mm). The longitudinal reinforcement nominal diameter is 1.75in (44.5mm) and $h_x = 9in$ (229mm).

$$s \leq \text{smaller of } \begin{cases} \left(\frac{1}{4} \right) (24in) = 6in \text{ (152mm)} \\ (6)(1.75in) = 10.5in \text{ (267mm)} \\ s_x = 4 + \left(\frac{14 - 9in}{3} \right) = 5.67in \text{ (144mm)} \end{cases}$$

$$\therefore s \leq 5.67in \text{ (144mm)}$$

$$\text{Say } s = 5in \text{ (127mm)}$$

The spacing requirement for §21.4.4.1(b) was calculated as 5.49in which is less than the 5.67in determined from §21.4.4.2. Therefore, the hoops and crossties will be provided at

#6 hoops and crossties at 5in (127mm) o.c.

§21.4.4.3 states that crossties or legs of overlapping hoops shall not be spaced more than 14in o.c. in the direction perpendicular to the longitudinal axis of the structural member.

The resulting reinforcement detail is shown in Figure D.3.

Figure D.3. Wall boundary element detail per ACI 318-02 §21.7.6.4.

Part (b)

Where special boundary elements are not required by §21.7.6.2 or §21.7.6.3, §21.7.6.5 shall be satisfied.

§21.7.6.5(a) states that if the longitudinal reinforcement ratio at the wall boundary is greater than $400/f_y$, boundary transverse reinforcement shall satisfy sections §21.4.4.1(c), §21.4.4.3, and §21.7.6.4(a). The maximum longitudinal spacing of transverse reinforcement in the boundary shall not exceed 8in.

These provisions controlled the reinforcement detailing in Part (a). Recall that the required area of transverse steel specified in §21.4.4.1(b) did not control the detailing in Part (a). Therefore, if the longitudinal reinforcement ratio in the boundary is greater than $400/f_y$, then the detailing will be the same as that in Part (a) with the exception that the longitudinal spacing of the transverse reinforcement can be increased to 8in (203mm).

$$\frac{400}{f_y} = \frac{400}{60,000 \text{ psi}} = 0.006667 < \frac{(2)(2.25 \text{ in}^2)}{(24 \text{ in})(9 \text{ in})} = 0.020833$$

Therefore, §21.7.6.5(a) must be satisfied. Detailing the boundary element like that of Part (a) and increasing the longitudinal spacing of the transverse reinforcement to 8in (203mm) satisfies this provision.

§21.7.6.5(b) states that if the factored design shear is greater than $A_{cv}\sqrt{f'_c}$, the horizontal reinforcement terminating at the edges of structural walls without boundary elements shall have a standard hook engaging the edge reinforcement or the edge reinforcement shall be enclosed in U-stirrups having the same size and spacing as, and spliced to, the horizontal reinforcement.

$$V_u = 652 \text{ kips} > A_{cv}\sqrt{f'_c} = \frac{(114 \text{ in})(24 \text{ in / leg})(2 \text{ legs})\sqrt{5,000 \text{ psi}}}{1,000 \# / \text{ kip}} = 387 \text{ kips}$$

Therefore, §21.7.6.5(b) must be satisfied. The detailing as given in Part (a) satisfies this provision. The resulting reinforcement detailing is shown in Figure D.4.

Figure D.4. Wall boundary element detail per ACI 318-02 §21.7.6.5.