

ISTANBUL TECHNICAL UNIVERSITY ★ GRADUATE SCHOOL OF SCIENCE
ENGINEERING AND TECHNOLOGY

**THE EFFECT OF COUPLING RATIO ON NONLINEAR BEHAVIOR OF
COUPLED SHEAR WALLS**

Ph.D. THESIS

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Department of Civil Engineering

Structural Engineering Programme

JULY 2015

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JULY 2015

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To my precious wife,

FOREWORD

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TABLE OF CONTENTS

	<u>Page</u>
FOREWORD	ix
TABLE OF CONTENTS	xi
ABBREVIATIONS.....	xiii
LIST OF TABLES.....	xv
LIST OF FIGURES.....	xvii
LIST OF SYMBOLS	xxi
SUMMARY	xxiii
ÖZET xxv	
1. INTRODUCTION	1
1.1 Objectives.....	3
1.2 Scope of the Dissertation.....	4
2. LITERATURE REVIEW	5
2.1 Experimental Studies on Solid Walls.....	5
2.2 Analytical Studies on Coupled Wall Behavior	8
2.3 Experimental Studies on Coupling Beam Behavior.....	9
2.4 Studies on Coupled Beam Modeling.....	13
3. DESIGN OF COUPLED WALL SYSTEMS	15
3.1 Force-based Design	15
3.1.1 Turkish Earthquake Code requirements.....	15
3.1.2 ACI requirements	19
3.2 Performance-based Design.....	20
3.3 Displacement-based Design	22
3.3.1 Coupling ratio	23
3.3.2 Effective height	23
3.3.3 Wall yield displacement.....	24
4. MODELING STRATEGY FOR COUPLING BEAMS	27
4.1 Micro Modeling Approach for Coupling Beams	27
4.2 Macro Modeling Alternatives for Diagonally Reinforced Coupling Beams ...	27
4.2.1 Naish model	27
4.2.2 Lu and Chen model	31
4.2.3 Barbachyn model	32
4.3 Verification of Coupling Beam Macro Models with Previous Test Results....	33
4.4 Proposal to Improve the Coupling Beam Modeling.....	40
4.4.1 Modified ultimate shear capacity	40
4.4.2 Contribution of slip and extension to the effective stiffness.....	42
4.4.3 Backbone curves for the improved model	45
4.5 Summary and Remarks	47
5. ANALYSIS OF COUPLED WALL SYSTEMS	49
5.1 Linear Analysis	49

5.2 Nonlinear Analysis	51
5.2.1 Pushover analysis	51
5.2.2 Nonlinear time history analysis	52
5.3 Coupled Wall and Coupling Beam Capacity-deformation Relations.....	52
5.4 Moment and Shear Distribution	55
5.5 Summary and Remarks.....	57
6. NUMERICAL INVESTIGATIONS	59
6.1 Methodology.....	59
6.2 Properties of Case Study Coupled Wall Systems.....	61
6.3 Design of Coupled Wall Systems.....	63
6.3.1 Loads applied to the coupled wall systems	63
6.3.1.1 Gravity loads	63
6.3.1.2 Seismic loads.....	64
6.3.2 Elastic analyses of wall systems.....	65
6.3.3 Design of coupling beams	67
6.3.4 Design of wall piers.....	71
6.3.5 Shear design of wall piers	74
6.4 Nonlinear modeling of investigated coupled wall systems	76
6.4.1 Analysis software	76
6.4.2 Nonlinear model of the considered coupled wall systems	78
6.4.3 Nonlinear wall element	79
6.4.4 Concrete material model in wall elements	81
6.4.5 Steel reinforcement material model in wall elements	82
6.5 Pushover Analyses and Results on 8-Story Building	84
6.6 Nonlinear Dynamic Analyses and Results	87
6.6.1 Cloud approach ground motion records	88
6.6.2 Records selected to match the target spectra.....	89
6.6.3 Coupling beam plastic rotation	91
6.6.4 Wall pier strains	98
6.6.5 Wall shear design verification.....	103
6.6.6 Moment distribution and shear migration phenomena.....	103
6.6.7 Comparison of fixed and flexible base models	121
6.6.8 Alteration of coupling ratio and its effects.....	123
6.7 Summary and Remarks.....	131
7. CONCLUSIONS	133
7.1 Nonlinear Modeling of Diagonally Reinforced Coupling Beams	133
7.2 Effects of Coupling Ratio on Nonlinear Behavior of Coupled Walls	134
7.3 Recommendations for Further Researches	137
REFERENCES	139
APPENDICES.....	145
APPENDIX A	146
APPENDIX B.....	157
CURRICULUM VITAE	169

ABBREVIATIONS

ACI	: American Concrete Institute
ASCE	: American Society of Civil Engineers
ATC	: Applied Technology Council
CP	: Collapse prevention
CR	: Coupling ratio
CRA	: Coupling ratio alteration
CW	: Coupled wall
DDBD	: Direct Displacement-based Design
DOF	: Degree of freedom
EDP	: Engineering demand parameter
FEMA	: Federal Emergency Management Agency
IO	: Immediate occupancy
LRFD	: Load and resistance factor design
LS	: Life safety
MVLE	: Multiple vertical line element
NEES	: Network for Earthquake Engineering Simulation
NLTHA	: Nonlinear time history analysis
OOP	: Object oriented programming
OPENSEES	: Open System for Earthquake Engineering Simulation
OTM	: Over-turning moment
PBD	: Performance-based design
PCA	: Portland Cement Association
PEER	: Pacific Earthquake Engineering Research Center
PGA	: Peak ground acceleration
RSA	: Response spectrum analysis
SDOF	: Single-degree-of-freedom
SEAOC	: Structural Engineers Association of California
TEC	: Turkish Earthquake Code
UCLA	: University California Los Angeles

LIST OF TABLES

	<u>Page</u>
Table 2.1 : Ductility values of previously performed coupling beam tests.	13
Table 4.1 : Plastic hinge rotation limits for diagonally reinforced coupling beams.	29
Table 4.2 : Test beams (Naish et al. 2013, Barney et al. 1980).	35
Table 4.3 : Properties of diagonally reinforced coupling beam specimens.	43
Table 4.4 : Effective stiffness results of coupling beam specimens.	44
Table 4.5 : Proposed coefficients for effective stiffness formula.	45
Table 6.1 : Target coupling ratio ranges for the preliminary designs.	61
Table 6.2 : Geometrical properties of 8-story coupled wall systems.	63
Table 6.3 : Geometrical properties of 16-story coupled wall systems.	63
Table 6.4 : Geometrical properties of 24-story coupled wall systems.	63
Table 6.5 : RSA results of 8-story coupled wall systems.	65
Table 6.6 : RSA results of 16-story coupled wall systems.	66
Table 6.7 : RSA results of 24-story coupled wall systems.	66
Table 6.8 : Minimum embedment length of diagonal reinforcement.	67
Table 6.9 : Coupling beam design summary for 8-story systems.	68
Table 6.10 : Coupling beam design summary for 16-story systems.	69
Table 6.11 : Coupling beam design summary for 24-story systems.	70
Table 6.12 : Linear analysis shear force results.	76
Table 6.13 : Advantages and of OpenSees.	78
Table 6.14 : Confined concrete material properties in OpenSees (2013).	82
Table 6.15 : Ground motion records used in NLTHA.	89
Table 6.16 : Coupling beam allowable plastic rotations (ASCE 41, 2013).	92
Table 6.17 : Coupling beam performance results.	98
Table 6.18 : Strain limit values for performance levels (TEC, 2007).	102
Table 6.19 : Tension and compression strains (50% in 50 years).	102
Table 6.20 : Tension and compression strains (10% in 50 years).	102
Table 6.21 : Tension and compression strains (2% in 50 years).	103
Table 6.22 : Linear and nonlinear analysis shear force results.	103
Table 6.23 : Proposed moment redistribution coefficient.	117
Table 6.24 : Periods of fixed base model and flexible base model.	122
Table 6.25 : NLTHA results fixed base vs. flexible base.	123

LIST OF FIGURES

	<u>Page</u>
Figure 1.1 : Typical (a) elevation view of coupled wall, (b) conventionally reinforced and (c) diagonally reinforced coupling beam.....	2
Figure 2.1 : Cyclic behavior of walls (Pilakoutas and Elnashai, 1995).	6
Figure 2.2 : Section of RW1 tested by Thomsen and Wallace (1995).	7
Figure 2.3 : Section of RW2 tested by Thomsen and Wallace (1995).	7
Figure 2.4 : Experimental results of Thomsen and Wallace (1995).	7
Figure 2.5 : Walls with staggered and ordered openings (Mosoarca, 2014).	9
Figure 2.6 : Tension over the entire height of the deep beam (Paulay, 1971).	10
Figure 2.7 : Double diagonal bars at hinging locations (Barney et al., 1980).	11
Figure 2.8 : Concentrating flexural reinforcements at edges (Xiao et al., 1999). ...	11
Figure 3.1 : Wall reinforcement layout (TEC, 2007).	16
Figure 3.2 : Wall bending moment envelope (TEC, 2007).	17
Figure 3.3 : Minimum requirement of coupled wall system (TEC, 2007).	18
Figure 3.4 : Diagonal reinforcement layout (TEC, 2007).	19
Figure 3.5 : Confinement of individual diagonals (ACI 318, 2011).	20
Figure 3.6 : Confinement of entire beam section (ACI 318, 2011).	20
Figure 3.7 : DDBD Single-DOF representation (Priestley et al., 2007).	22
Figure 3.8 : Design parameters for regular coupled walls (Priestley et al., 2007). .	24
Figure 3.9 : Displacement-based design steps for coupled wall systems.	26
Figure 4.1 : Coupling beam modeling components (Naish et al. 2013).	28
Figure 4.2 : Naish et al. (2013) back bone curves, (a) slip/extension, (b) flexure. .	28
Figure 4.3 : Slip/extension crack and corresponding rotation.	30
Figure 4.4 : Contributions of slip and extension behaviors.	31
Figure 4.5 : Subelements of coupling beam (Lu and Chen, 2005).	31
Figure 4.6 : Lu and Chen (2005) back bone curves (a) slip/extension hinge, (b) flexure hinge and (c) shear hinge.	32
Figure 4.7 : Nine-node strut-and-tie model (Barbachyn et al. 2012).	32
Figure 4.8 : UCLA laboratory test setup for CB24 and CB33 (Naish et al., 2013). 34	34
Figure 4.9 : PCA laboratory test setup for C6.	35
Figure 4.10 : Naish et al. (2013) model schematic developed for OpenSees.	36
Figure 4.11 : Lu and Chen (2005) model schematic developed for OpenSees.	36
Figure 4.12 : CB24 cyclic analysis results (a) Naish et al. (2013), (b) Lu and Chen (2005), (c) Barbachyn et al. (2012); and (d) cumulative dissipated energy.	37
Figure 4.13 : CB33 cyclic analysis results (a) Naish et al. (2013), (b) Lu and Chen (2005), (c) Barbachyn et al. (2012); and (d) cumulative dissipated energy.	38

Figure 4.14 : C6 cyclic analysis results (a) Naish et al. (2013), (b) Lu and Chen (2005), (c) Barbachyn et al. (2012); and (d) cumulative dissipated energy.	39
Figure 4.15 : CB24 results (a) lateral load vs. rotation; and (b) cumulative energy.	41
Figure 4.16 : CB33 results (a) lateral load vs. rotation; and (b) cumulative energy.	42
Figure 4.17 : Bilinearization on moment-curvature of B1501 (Xtract, 2007).....	44
Figure 4.18 : Effective stiffness of diagonally reinforced coupling beams.....	45
Figure 4.19 : Proposed model for diagonally reinforced coupling beams.....	46
Figure 4.20 : Proposed model backbone curve vs. test results CB24.....	46
Figure 4.21 : Proposed model backbone curve vs. test results C6.	47
Figure 5.1 : Spectral acceleration vs. period.....	50
Figure 5.2 : Idealized lateral response of coupled wall (Harries 2001).....	53
Figure 5.3 : (a) Representative coupled wall system behavior, (a) wall piers and coupling beam moments, (b) over-turning moment vs. coupling moment, (c) coupling ratio alteration plot.	55
Figure 5.4 : Moment distribution a) elastic, b) significant yield at tension wall base.	56
Figure 5.5 : Redistribution of wall moments.	57
Figure 6.1 : Case study flow chart.	60
Figure 6.2 : Typical floor plan of the case study buildings.	61
Figure 6.3 : Extracted representative frames of 8, 16 and 24 story coupled walls..	62
Figure 6.4 : Coordinates and seismic parameters of the case study buildings.	64
Figure 6.5 : Response spectra used for coupled wall systems.....	65
Figure 6.6 : The normalized shear distribution of coupling beams.	66
Figure 6.7 : Diagonal reinforcement layout for CW08-CR-L.	68
Figure 6.8 : Diagonal reinforcement layout for CW08-CR-N.....	68
Figure 6.9 : Diagonal reinforcement layout for CW08-CR-H.....	68
Figure 6.10 : Diagonal reinforcement layout for CW16-CR-L.	69
Figure 6.11 : Diagonal reinforcement layout for CW16-CR-N.....	69
Figure 6.12 : Diagonal reinforcement layout for CW16-CR-H.....	69
Figure 6.13 : Diagonal reinforcement layout for CW24-CR-L.	70
Figure 6.14 : Diagonal reinforcement layout for CW24-CR-N.....	70
Figure 6.15 : Diagonal reinforcement layout for CW24-CR-H.....	70
Figure 6.16 : 8-story wall P-M curves a)low, b)normal and c)high coupled.	72
Figure 6.17 : 8-story wall cross-section.....	72
Figure 6.18 : 16-story wall P-M curves a)low, b)normal and c)high coupled.....	73
Figure 6.19 : 16-story wall cross-section.....	73
Figure 6.20 : 24-story wall P-M curves a)low, b)normal and c)high coupled.....	74
Figure 6.21 : 24-story wall cross-section.....	74
Figure 6.22 : P-M interaction curve of a coupled wall for linear design procedure.	75
Figure 6.23 : OpenSees framework.	78
Figure 6.24 : Nonlinear modeling for the investigated coupled wall systems.....	79
Figure 6.25 : Fiber section (Taucer et al., 1991).	80
Figure 6.26 : OpenSees fiber model validation for flexure (Orakçal et al., 2006)..	80
Figure 6.27 : OpenSees fiber model validation for flexure (Vecchio et al., 2013).	81
Figure 6.28 : Force-based element with 5 integration points.	81
Figure 6.29 : Concrete02 model (Kent and Park, 1971).....	82

Figure 6.30 : Reinforcement steel model.	84
Figure 6.31 : Pushover load pattern.....	84
Figure 6.32 : Pushover - moment vs. maximum interstory drift CW08-CR-L.	85
Figure 6.33 : Pushover - moment vs. maximum interstory drift CW08-CR-N.....	86
Figure 6.34 : Pushover - moment vs. maximum interstory drift CW08-CR-H.....	86
Figure 6.35 : Pushover analysis – moments vs. maximum interstory drift ratio.....	87
Figure 6.36 : Nonlinear dynamic analyses.	88
Figure 6.37 : Distribution of the PGA values of the selected 50 records.....	89
Figure 6.38 : Estimation of the scale factor.....	91
Figure 6.39 : Chord rotation for coupling beams (ASCE 41, 2013).	92
Figure 6.40 : Plastic beam rotations – CW8-CR-L.	93
Figure 6.41 : Plastic beam rotations – CW8-CR-N.....	93
Figure 6.42 : Plastic beam rotations – CW8-CR-H.....	94
Figure 6.43 : Plastic beam rotations – CW16-CR-L.	95
Figure 6.44 : Plastic beam rotations – CW16-CR-N.....	95
Figure 6.45 : Plastic beam rotations – CW16-CR-H.....	96
Figure 6.46 : Plastic beam rotations – CW24-CR-L.	96
Figure 6.47 : Plastic beam rotations – CW24-CR-N.....	97
Figure 6.48 : Plastic beam rotations – CW24-CR-H.....	97
Figure 6.49 : Wall deformed shape.	99
Figure 6.50 : CW08-CR-L shear force and axial strain history 10%in50years.....	99
Figure 6.51 : CW08-CR-N shear force and axial strain history 10%in50years.	99
Figure 6.52 : CW08-CR-H shear force and axial strain history 10%in50years. ...	100
Figure 6.53 : CW08-CR-L wall strain distribution 10%in50years record no.1. ...	100
Figure 6.54 : CW08-CR-N wall strain distribution 10%in50years record no.1...	101
Figure 6.55 : CW08-CR-H wall strain distribution 10%in50years record no.1....	101
Figure 6.56 : Wall axial force vs. time for CW08-CR-N.....	104
Figure 6.57 : Wall bending moment vs. time for CW08-CR-N.....	105
Figure 6.58 : Wall shear force vs. time.	105
Figure 6.59 : Wall A bending moment vs. Wall B bending moment.....	106
Figure 6.60 : CW24-CR-N Wall A mean of maximum story shear forces.	107
Figure 6.61 : CW24-CR-N Wall B corresponding story shear forces.....	107
Figure 6.62 : CW24-CR-N wall piers story shear comparison.	108
Figure 6.63 : Moment ratio between walls vs. CR – CW08 – 50% in 50 years....	109
Figure 6.64 : Moment ratio between walls vs. CR – CW08 – 10% in 50 years....	109
Figure 6.65 : Moment ratio between walls vs. CR – CW08 – 2% in 50 years.....	110
Figure 6.66 : Moment ratio between walls vs. CR – CW16 – 50% in 50 years....	110
Figure 6.67 : Moment ratio between walls vs. CR – CW16 – 10% in 50 years....	111
Figure 6.68 : Moment ratio between walls vs. CR – CW16 – 2% in 50 years.....	111
Figure 6.69 : Moment ratio between walls vs. CR – CW24 – 50% in 50 years....	112
Figure 6.70 : Moment ratio between walls vs. CR – CW24 – 10% in 50 years....	112
Figure 6.71 : Moment ratio between walls vs. CR – CW24 – 2% in 50 years.....	113
Figure 6.72 : Moment redistribution 50% in 50 years.....	114
Figure 6.73 : Moment redistribution vs. coupling ratio 50% in 50 years.....	114
Figure 6.74 : Moment redistribution 10% in 50 years.....	115
Figure 6.75 : Moment redistribution vs. coupling ratio 10% in 50 years.....	115
Figure 6.76 : Moment redistribution 2% in 50 years.....	116
Figure 6.77 : Moment redistribution vs. coupling ratio 2% in 50 years.....	116
Figure 6.78 : Shear migration and wall pier base shear demands.	118
Figure 6.79 : Shear ratio between walls vs. CR – CW08 – 10% in 50 years.....	119

Figure 6.80 : Shear ratio between walls vs. CR – CW16 – 10% in 50 years.	119
Figure 6.81 : Shear ratio between walls vs. CR – CW24 – 10% in 50 years.	120
Figure 6.82 : Shear redistribution 10%in50 years.	120
Figure 6.83 : Shear redistribution vs. coupling ratio.	121
Figure 6.84 : Mathematical model representation of flexible base.	122
Figure 6.85 : Coupling beam plastic rotation fixed base vs. flexible base.	123
Figure 6.86 : Coupling moment and overturning moment vs. time – CW08-CR-N ground motion no.36.....	124
Figure 6.87 : CR vs. normalized time CW08-CR-N ground motion no.36.....	124
Figure 6.88 : Coupling ratio alteration vs. beam plastic rotation CW08-CR-L. ...	125
Figure 6.89 : Coupling ratio alteration vs. beam plastic rotation CW08-CR-N....	126
Figure 6.90 : Coupling ratio alteration vs. beam plastic rotation CW08-CR-H. ...	126
Figure 6.91 : Coupling ratio alteration vs. beam plastic rotation CW16-CR-L. ...	127
Figure 6.92 : Coupling ratio alteration vs. beam plastic rotation CW16-CR-N....	127
Figure 6.93 : Coupling ratio alteration vs. beam plastic rotation CW16-CR-H. ...	128
Figure 6.94 : Coupling ratio alteration vs. beam plastic rotation CW24-CR-L. ...	128
Figure 6.95 : Coupling ratio alteration vs. beam plastic rotation CW24-CR-N....	129
Figure 6.96 : Coupling ratio alteration vs. beam plastic rotation CW24-CR-H. ...	129
Figure 6.97 : Coupling ratio alteration vs. plastic rotation – low coupling.	130
Figure 6.98 : Coupling ratio alteration vs. plastic rotation – normal coupling.....	130
Figure 6.99 : Coupling ratio alteration vs. plastic rotation – high coupling.	131
Figure A.1 : Average vs. target spectrum CW08 - 50% in 50 years.	146
Figure A.2 : Average vs. target spectrum CW08 - 10% in 50 years.	146
Figure A.3 : Average vs. target spectrum CW08 - 2% in 50 years.	147
Figure A.4 : Average vs. target spectrum CW16L - 50% in 50 years.....	147
Figure A.5 : Average vs. target spectrum CW16L - 10% in 50 years.....	148
Figure A.6 : Average vs. target spectrum CW16L - 2% in 50 years.....	148
Figure A.7 : Average vs. target spectrum CW16N - 50% in 50 years.	149
Figure A.8 : Average vs. target spectrum CW16N - 10% in 50 years.	149
Figure A.9 : Average vs. target spectrum CW16N - 2% in 50 years.	150
Figure A.10 : Average vs. target spectrum CW16H - 50% in 50 years.	150
Figure A.11 : Average vs. target spectrum CW16H - 10% in 50 years.	151
Figure A.12 : Average vs. target spectrum CW16H - 2% in 50 years.	151
Figure A.13 : Average vs. target spectrum CW24L - 50% in 50 years.....	152
Figure A.14 : Average vs. target spectrum CW24L - 10% in 50 years.....	152
Figure A.15 : Average vs. target spectrum CW24L - 2% in 50 years.....	153
Figure A.16 : Average vs. target spectrum CW24N - 50% in 50 years.	153
Figure A.17 : Average vs. target spectrum CW24N - 10% in 50 years.	154
Figure A.18 : Average vs. target spectrum CW24N - 2% in 50 years.	154
Figure A.19 : Average vs. target spectrum CW24H - 50% in 50 years.	155
Figure A.20 : Average vs. target spectrum CW24H - 10% in 50 years.	155
Figure A.21 : Average vs. target spectrum CW24H - 2% in 50 years.	156

LIST OF SYMBOLS

A	: Cross-sectional area of coupling beam
a	: Coefficient for proposed effective stiffness formula
A_c	: Area of confined concrete in diagonal bundle
A_{cw}	: Cross-sectional area of concrete beam web
A_d	: Reinforcement area per each diagonal bundle
A_g	: Gross area of wall pier
A_p	: Area of floor plan
A'_s	: Equivalent area of diagonal reinforcement for section analysis
A_{vd}	: Cross-sectional area of each diagonal group of bars
b_w	: Width of beam web
C_y	: Axial compression capacity of diagonal bundle
d	: Effective depth of beam
d_b	: Nominal diameter of diagonal reinforcement bar
d_b	: Nominal diameter of diagonal reinforcement bar
E	: Modulus of elasticity
F_a	: Site amplification coefficient for short period spectral acceleration
f'_c	: Concrete compressive strength at 28 days
f'_{cc}	: Confined concrete compressive strength
f_{ctd}	: Design tensile strength of concrete
F_e	: Design base shear force in direct displacement based design
F_v	: Site amplification coefficient for 1-sec period spectral acceleration
f_u	: Ultimate rupture strength of reinforcement
f_y	: Yield strength of steel reinforcement
f_{yd}	: Design yield strength of steel reinforcement
G	: Shear modulus
h	: Depth of beam
H_{CF}	: Height of contraflexure point
H_{cr}	: Critical height
H_e	: Effective height
H_n	: Total height of wall in direct displacement based design
H_w	: Height of wall
I_{eff}	: Effective section moment of inertia
I_{gross}	: Gross section moment of inertia
K	: Stiffness
K_e	: Effective stiffness
K_i	: Elastic stiffness
L, l_n	: Clear span of coupling beam
l_w	: Length of wall
M	: Wall bending moment obtained from linear analysis
M'	: Wall modified bending moment after redistribution applied
M_c	: Bending moment at compression wall

M'_c	: Bending moment at compression wall after redistribution
m_e	: Effective mass
m_i	: Bending moment of wall pier i
M_T	: Bending moment at tension wall
M'_T	: Bending moment at tension wall after redistribution
M_u	: Ultimate moment capacity
M_y	: Bending moment at yield capacity
r	: Post-yield stiffness reduction factor
P	: Axial load considered for seismic load cases
P_G	: Axial force due to dead load
P_Q	: Axial force due to live load
R_C	: Potential strength contribution from coupling beams
R_T	: Potential strength of entire structure
R_W	: Potential strength contribution from wall pier
S_{MS}	: Short period design spectral acceleration
S_{M1}	: 1-second period design spectral acceleration
S_S	: Short period spectral acceleration
S_1	: 1-second period spectral acceleration
T_e	: Effective period
T_y	: Axial tension capacity of diagonal bundle
V_i	: Coupling beam shear force at i -th story
V_n	: Nominal shear capacity of beam
x	: Depth of neutral axis
α	: Angle between diagonal bars and longitudinal axis of beam
Δ	: Relative displacement of beam end
Δ_d	: Maximum displacement in Direct Displacement-based design
Δ_i	: Vertical displacement of i -th node
Δ_y	: Relative displacement at yield
δ_{ext}	: Deformation due to extension of reinforcement at wall interface
δ_{exty}	: Yield deformation due to extension of reinforcement at wall interface
δ_s	: Deformation due to slip/extension at yield
ε_{ij}	: Strain in the left side of the wall
ε_{kl}	: Strain in the right side of the wall
λ	: Reduced mechanical factor in light-weight concrete
θ	: Beam chord rotation
μ	: Displacement ductility
ϕ_{yW}	: Yield curvature at the wall base
ϕ_{y1}	: Wall curvature corresponding to the overturning moment in DDBD
ϕ_{y2}	: Curvature corresponding to the coupling beam moment in DDBD

THE EFFECT OF COUPLING RATIO ON NONLINEAR BEHAVIOR OF COUPLED WALL SYSTEMS

SUMMARY

Coupled walls result from the need to provide openings in window or door locations. Besides, coupled wall systems are exceptional lateral load resisting systems that not only reduce the deformation demands of the building, but also extend the plastic deformation vertically. This dissertation investigates the dependency on the initial coupling ratio on the actual behavior of rectangular coupled wall systems. In addition, a comprehensive review of the mathematical modeling for diagonally reinforced concrete coupling beams is carried out in the state-of-the-art.

First, a simple yet accurate macro modeling procedure for coupling beams was investigated through a compressive literature survey for capturing the nonlinear behavior properly. Further enhancement to existing models were proposed to have better accuracy in cyclic behaviors comparing to previously performed test results. In addition, the simplicity in the modeling was improved by eliminating the slip/extension rotation hinges.

In the second part of this research, 8, 16 and 24-story prototype reinforced concrete coupled wall buildings located in a high seismic zone were numerically studied. Totally, nine coupled wall prototype structures with identical wall piers, having varying coupling ratio values and number of stories were designed. Elastic analyses using the response spectrum analysis procedure were utilized to establish initial proportions in the coupled wall system, and to determine the design forces and moments. Nonlinear static and dynamic analyses were also conducted to investigate the coupled wall behavior, adequacy of design, and alteration in the coupling beams' efficiency. An additional unique aspect of this work is about investigating the distribution of force demands between wall piers depending on the initial coupling ratio.

The results of this study indicate that low coupled systems do not always satisfy the required performance targets under specific earthquake levels. Moreover, the coupling beams at high-degree coupled systems exhibit comparatively low ductile behavior with limited plastic deformations along elevation.

Although linear analysis results show that the bending moment demands at identical wall piers are the same; this cannot be expected to be valid for nonlinear cases. As coupling beams alter the axial demand on wall piers, they also change their moment capacities. It was observed that the disparity in the moment demands becomes larger as the coupling ratio gets higher. Therefore, further recommendations have been provided on moment redistribution to be utilized during the linear analysis.

Finally, the change in the coupling ratio as plastic deformations increase in the system has been investigated in this study. It was observed that the coupling ratio degradation rate seems more critical in the systems where low coupling ratio is selected initially.

BAĞ KİRİŞ KATKI ORANININ BOŞLUKLU PERDELERİN DOĞRUSAL OLMAYAN DAVRANIŞINA ETKİSİ

ÖZET

Binaların betonarme perde elemanlarında genellikle binanın kullanım amacına veya mimari gereksinimlere göre kapı veya pencere boşlukları bırakılması gerekmektedir. Bırakılan boşlukların düşeyde sürekli ve düzgün olması durumunda aralarında kalan her iki perde parçasını birbirine bağlayan yatay elemanlara bağ kiriş, perde sistemin geneline ise bağ kirişli (boşluklu) perde denir. Deprem davranışı altında konsol perdelerde plastikleşme sadece perde tabanında gerçekleşirken, bağ kirişli boşluklu perdelerde uygun tasarım yapıldığı takdirde plastik davranış perde yüksekliği boyunca yayılabilir. Böylece daha sünek davranış elde edilerek enerjinin sönmelenmesine katkıda bulunabilirler.

Boşluklu perde sisteminin yatay yükler altında davranışı sırasında bağ kirişlerinde oluşan kesme kuvvetleri bağlandığı perdelerin birine eksenel basınç kuvveti olarak aktarılırken, diğerine eksenel çekme kuvveti olarak aktarılır. Böylece sisteme dış yükler tarafından aktarılan devrilme momentinin bir kısmı bağ kirişlerin iki ucundaki kesme kuvvetlerinin toplamından oluşan perde normal kuvvetlerinin momenti ile karşılanmış olur. Karşılanan bu momentin yapıya etkiyen toplam devrilme momentine oranına, bağ kiriş katkı oranı denir. Bağ kiriş katkı oranı, perdelerin ve bağ kirişlerin rijitliğine bağlı olup, doğrusal elastik analiz yöntemine göre yapılan tasarımda eleman boyutlandırılmalarının ve analizlerin tamamlanmasından önce hesaplanamamaktadır. Pratik uygulamalarda analizden elde edilen bu bağ kiriş katkı oranının yönetmeliklerin bağ kirişli perde için gerektirdiği minimum değeri sağlaması durumunda, tasarım yapıldıktan sonra tekrar irdelenmemektedir. Öte yandan, bağkiriş katkı oranının düşük veya yüksek olmasına bağlı olarak perdelerin iç kuvvetlerinde ve doğrusal olmayan davranışında büyük farklar oluşmaktadır. Bu tez çalışmasında boşluklu perdelerin doğrusal olmayan davranışında bağ kiriş katkı oranının etkileri kapsamlı bir şekilde incelenmiştir.

Tez toplam 7 bölümden oluşmaktadır. Birinci bölümde tezin amaç ve kapsamı verildikten sonra ikinci bölümde, literatürde yer alan daha önce boşluklu perde sistemleri ve bağ kiriş elemanlar ile ilgili teorik, deneysel ve sayısal çalışmalar bulguları ile birlikte özetlenmiştir.

Tezin üçüncü bölümünde, bağ kirişli perde sistemlerinin tasarım yöntemleri incelenmiştir. Deprem Bölgelerinde Yapılacak Binalar Hakkında Yönetmelik (2007) ve ACI 318 (2011)'in vermiş olduğu kuvvet bazlı tasarım kriterleri özetlenmiştir. Günümüzde halen bir çok yönetmelik, yapı elemanlarını kuvvet bazlı yöntemle göre tasarım yaptırmaktadır. Fakat, bu yaklaşımda perde ve bağ kiriş elemanlarının aynı anda akması temel alındığından, yapının gerçek doğrusal olmayan davranışını yansıtmamaktadır. İkinci olarak bu bölümde son yıllarda yaygınlaşan performansla göre tasarım ilkeleri incelenmiştir. Bu tasarım yöntemine göre ön tasarımı yapılan boşluklu perde yapı sisteminin daha önceden belirlenen performans hedeflerini sağlaması gerekmektedir. Kuvvet bazlı yöntemden farklı olarak, performansla göre

tasarımda yapı elemanları farklı zamanlarda akmaya geçebilmektedir. Ayrıca elemanların hasar sınırları içerisinde plastik şekil değiştirme yapmasına izin verilmektedir. Bu bölümde son olarak Priestley ve diğerlerinin (2007) önerdiği Direkt Yer Değiştirmeye Göre Tasarım yöntemi incelenmiştir. Yukarıda verilen iki yöntemden farklı olarak direkt yer değiştirmeye göre tasarımda, boşluklu perdelerde bağ kiriş katkı oranı tasarımın ilk adımı olarak belirlenebilmektedir. Öte yandan bu yöntemde bağ kirişlerin tamamının aynı özellikte olduğu ve yapı yüksekliğince aynı kesme kuvvetini paylaştığı kabul edilmektedir.

Tezin dördüncü bölümü çapraz donatılı betonarme bağ kirişlerin doğrusal olmayan matematik modelinin oluşturulmasını kapsamaktadır. Modelleme teknikleri mikro ve makro olarak iki grupta sınıflandırılmaktadır. Mikro modellemede yapı mekaniğinin prensiplerinden yararlanarak sonlu elemanlar yöntemi ile gerilme ve birim şekil değiştirme değerleri hesaplanır ve değerlendirilmesi yapılır. Bu model tekniği ile gerçeğe oldukça yakın sonuç alınabilmekle birlikte, analiz süresi ve sonuçların yorumlanabilmesi oldukça uzun sürmektedir. Öte yandan, son yıllarda yapılan çalışmalarda çubuk ve elasto-plastik mafsallarla oluşturulan çeşitli makro modellemelerin de yeterli seviyede iyi sonuçlar verebildiği gösterilmiştir.

Sunulan çalışmada çapraz donatılı bağ kirişlerin makro modellenmesi ile ilgili değişik yöntemleri kullanan Naish ve diğerleri (2013), Lu ve Chen (2005) ve Barbachyn ve diğerlerinin (2012) geliştirdiği modeller irdelenmiştir. Lu ve Chen (2005) modelinde bağ kirişi elastik kısım, uçlarda yayılı plastik mafsallarla, donatı sıyrılma ve uzamasını temsil eden dönme redörleri, ortada kayma mafsalları olmak üzere toplam dört adet alt elemandan oluşmaktadır. Naish ve diğerlerinin (2013) geliştirdiği modelde ise, eleman uçlarında eğilme moment mafsalları yayılı plastik olarak verilmiştir, buna karşılık moment-dönme ilişkisinde dayanım azalması tanımlanmıştır. Barbachyn ve diğerlerinin (2012) geliştirdiği model ise diğerlerinden farklı olarak betonarme kafeste basınç (strut) ve çekme (tie) çubukları kullanılmıştır. Bu yöntemde çok sayıda düğüm noktası oluşturulduğundan pratik değildir, ayrıca kiriş-perde birleşimlerinde donatı uzama ve sıyrılması tanımlanmamıştır.

Yukarıda bahsedilen üç farklı makro model yaklaşımı daha önceden gerçekleştirilmiş deneysel sonuçlar ile karşılaştırıldığında Naish ve diğerleri (2013) tarafından oluşturulan modelin en iyi sonucu verdiği gözlemlenmiştir. Dolayısıyla tezin son bölümünde yer alan sayısal incelemede boşluklu perdelerin oluşturulmasında bağ kirişleri için bu model baz alınmıştır. Ek olarak bağ kirişlerde çapraz donatının uzama ve sıyrılmasına bağlı olarak kesitin etkin eğilme rijitliğinin belirlenmesi için parametrik bir inceleme yapılmıştır. Elde edilen sonuçlara göre açıklığın derinliğe oranına ve dayanımına bağlı olarak bağ kirişlerin etkin eğilme rijitliklerinin çatlamamış kesitin eğilme rijitliğinin %8-11 aralığında olduğu gözlemlenmiştir. Yapılan bu sayısal incelemeler ile bağ kiriş modellemesinde eleman ucunda bulunan donatı uzama/sıyrılması için oluşturulan elastik dönme redörleri kaldırılarak model daha basit hale getirilebilmiştir.

Tezin beşinci bölümünde boşluklu perdelerin tasarımında kullanılan doğrusal ve doğrusal olmayan analiz yöntemlerini özetlemektedir. Günümüzde boşluklu perdeler yönetmeliklerde verilen doğrusal elastik analiz yöntemleri ile tasarlanabilmektedir. Öte yandan boşluklu perdelerin tasarım aşamasında doğrusal olmayan analizler ile çalışılması perdeler ve bağ kirişlerde plastik davranışa geçiş aşamalarını irdeleme fırsatını vermektedir. Bu bölümde ayrıca doğrusal olmayan davranış altında çekme ve

basınç perdeleri arasındaki moment ve kesme iç kuvvetlerinin tekrar dağılımı prensibi anlatılmaktadır.

Altıncı bölümde, boşluklu perdelerin doğrusal olmayan davranışında bağ kiriş katkı oranının etkilerini incelemek üzere sayısal incelemeler yer almaktadır. Örnek olarak 8, 16 ve 24 katlı ve bağ kiriş katkı oranları düşük, normal ve yüksek olmak üzere toplam 9 adet tipik dikdörtgen kesitli boşluklu perdeli yapı sistemleri incelenmiştir. Perde elemanları için eşdeğer çubuk elemanlar kullanılarak yapılan doğrusal elastik analiz sonuçlarına göre, aynı kat adedine sahip yapılar aynı kütleyle sahip olmalarına karşılık, bağ kiriş katkı oranı yüksek olan yapıların etkin periyodu çok daha fazla kısa olduğundan taban kesme kuvvetlerinin diğer yapılara göre %30 kadar daha olduğu gözlemlenmiştir. Analiz sonuçlarından elde edilen iç kuvvetlere göre boşluklu perdelerin ön tasarımı yönetmeliklere uygun olarak yapılmıştır.

Ön tasarımı tamamlanan 9 adet boşluklu perde sistemlerinin elastik ötesi davranışlarını irdelemek üzere OpenSees (2013) analiz programı ile doğrusal olmayan modelleri oluşturulmaktadır. Programda perde elemanları tek doğrultuda çalışan fiber elemanlardan oluşturulmuştur. Bağ kirişleri ise, tezin üçüncü bölümünde açıklanan Naish ve diğerlerinin (2013) geliştirdiği modelleme için önerilen iyileştirmeler de kullanılarak modellenmiştir. İlk olarak perde ve bağ kirişlerin doğrusal olmayan davranışını daha iyi anlayabilmek ve irdeleyebilmek için sadece 8 katlı yapılarda uygulanmak üzere doğrusal olmayan itme analizi uygulanmıştır. İtme analizi sonucunda bağ kiriş katkı oranı yüksek olan sistemde basınç perdesinin çekme perdesine göre çok daha fazla eğilme momenti ve kesme kuvveti aldığı gözlemlenmiştir.

Daha sonra boşluklu perde yapılarının tamamına uygulanmak üzere zaman tanım alanında doğrusal olmayan analizler yapılmıştır. Hedef performanslara göre değerlendirme yapılabilmesi için 50 yılda aşılma olasılığı %50 olan deprem için; Hemen Kullanım (HK), aşılma olasılığı %10 olan tasarım depremi için; Can Güvenliği (CG) ve aşılma olasılığı %2 olan deprem için Göçmeme Güvenliği (GG) hedef performansları değerlendirilmiştir. Her yapı tipi ve her deprem seviyesi için 20şer kaydın ortalaması alınmak suretiyle toplam 420 kayıt elde edilmiştir.

Sonuç olarak elde edilen bağ kirişi plastik dönme değerlerine göre normal seviyede bağ kiriş katkı oranı olan yapılar her üç deprem performansını uygun şekilde sağlamaktadır. Öte yandan bağ kiriş katkı oranı düşük olan sistemlerde bazı performans seviyelerinin sağlanmadığı görülmüştür. Bununla beraber, bağ kiriş katkı oranı yüksek olan sistemlerin yeterince sünek davranış sergilemediği ve bağ kirişlerinde beklenenden daha az plastik dönme meydana geldiği gözlemlenmiştir.

Yapılan zaman tanım alanında doğrusal olmayan analizlerde ayrıca perdelerin her ikisinde oluşan iç kuvvetlerin değişimleri irdelenmiştir. Bağ kirişlerinden dolayı çekme kuvveti altında olan perde elemanı basınç altında olan perdeye göre çok daha erken akmaya başladığından, moment istemlerinin geri kalanını basınç perdesi taşımak durumunda kalmaktadır. Bağ kiriş katkı oranı yüksek olan sistemlerde her iki perde tabanındaki moment oranının 3.25'e kadar çıktığı gözlemlenmiştir. Öte yandan, bağ kiriş katkı oranı düşük olan sistemlerde bu oranın 1.40'ın altına düştüğü gözlemlenmiştir. Buna bağlı olarak yönetmelikte yer verilen doğrusal elastik analizde boşluklu perdelerde uygulanan eğilme momentinin tekrar dağılım kuralı irdelenmiş olup, bu doğrultuda önerilerde bulunulmuştur. Analiz sonuçlarına göre, bağ kiriş katkı oranı %35'in altında bulunan yapılar için yönetmelikte yer verilen moment tekrar dağılım oranının %30'dan %15'e düşürülebileceği önerilmiştir.

Yatay yükler altında çekme perdesinde oluşan plastikleşme aynı zamanda çekme perdesinden basınç perdesine kesme kuvvetlerin iletilmesine sebep olmaktadır. Literatürde bu olaya “kesme göçü” de denilmektedir. Yapılan sayısal incelemelerde, bağ kiriş katkı oranına bağlı olarak perdeler arasındaki kesme göçünün toplam taban kesme kuvvetine olan oranının %16’ya kadar çıktığı belirlenmiştir.

Tez çalışmasında ek olarak 9 adet örnek yapının tamamına, PGA değerleri giderek artan 50 adet deprem kaydı kullanılarak oluşturulan bulut yaklaşımı kullanılarak, zaman tanım alanında analizleri yapılmıştır. Bu sayede yapılara uygulanan her bir deprem kaydı için artan bağ kiriş plastik dönme değerlerine karşılık bağ kiriş katkı oranındaki değişim gözlemlenmiştir. Bu sayısal çalışmanın sonucunda bağ kiriş katkı oranının doğrusal elastik analizde belirlendiği gibi sabit kalmadığı ve başlangıçta seçilen tasarım değerinin %40’ına kadar azalma meydana gelebileceği görülmüştür.

1. INTRODUCTION

Parallel to the fast urbanization, a rapid increase in the construction of tall structures has taken place. Reinforced concrete walls are common as lateral load bearing members in reinforced concrete multi-story buildings since they are generally preferred conventionally to resist seismic demands. Coupled structural walls form an efficient structural mechanism to resist seismic actions by meeting architectural requirements at the same time in tall buildings. By connecting the considerable lateral stiffness of structural walls with properly proportioned coupling beams that can accommodate most of the energy-dissipative mechanism during the response to earthquake motions, an exceptional behavior of structural system can be achieved.

Since plastic hinges are intended to form not only at the base of the walls but also at both ends of coupling beams, energy dissipation is distributed over a more extensive region of the structure with a result of having higher damping than is the case with cantilever walls. The shear forces on coupling beams *push down* on one wall and *pull up* on the other. The coupled system resists over-turning moment (OTM) through the development of an axial force couple, resulting from the accumulation of the effect of beam shears, as well as flexural reactions in the individual wall. The proportion of OTM resisted by the coupling beams is defined as the coupling ratio (CR). This coupling ratio of the system directly affects the system reaction to lateral forces. Although there exist numerous academic studies on wall behavior, there is a lack of investigation on their relation with coupling ratio.

After using conventional reinforcement in early structures, the use of diagonal reinforcement in coupling beams with aspect ratio (clear length to total depth) less than four was first introduced into ACI 318-99 in 1999. Two bundles of diagonal reinforcement steel bars are placed such that they intersect at the center of the beam (Figure 1.1). These two groups of diagonal bars and the concrete they cover are counted as a truss, with one group serving as the tension member and the other as the compression member. To increase the compressive and the deformation capacities of

the diagonal truss members as well as to prevent buckling of the diagonal bars, the use of transverse reinforcement around the diagonal bar groups is required (ACI 318-99).

The quantity of transverse reinforcement required is the same as that used for columns, which is substantially more than used in most of the prior coupling beam test schedules. Nominal transverse reinforcement is also required around the entire beam cross section.

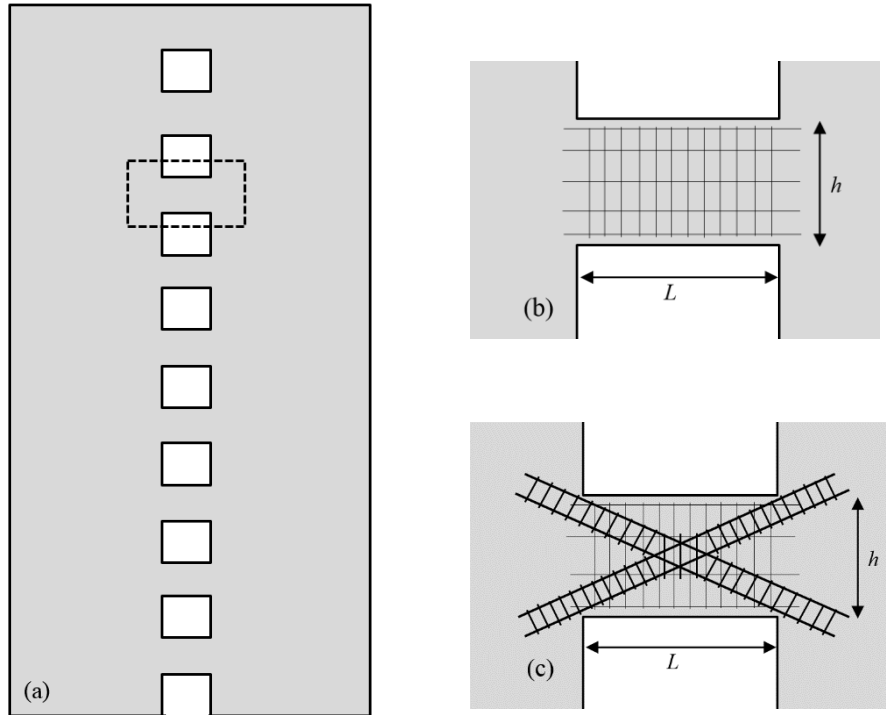


Figure 1.1 : Typical (a) elevation view of coupled wall, (b) conventionally reinforced and (c) diagonally reinforced coupling beam.

For the analysis and design of tall buildings, it is essential that the nonlinear behavior of structural walls and coupling beams are correctly understood and represented in the numerical models. Inelastic response of diagonally reinforced concrete coupling beams can be represented either by using small finite elements with comprehensive expression of each local behavior or by using macro models. The micro modeling is based on stress versus strain relationship of steel and concrete, which is more accurate, detailed and suitable compared to the case of analyzing the behavior of structural elements in experiments. However, micro modeling requires extensive knowledge and time, which limits its use to some fields far from practical applications. While very accurate, micro models remain limited in their use mainly because they must be conducted by experienced engineers in order to achieve reliable results. Moreover, the software required for performing analysis is generally expensive. Thus recent studies

are focused on macro modeling instead, as it is simpler in practice, yet gives adequately accurate results [(Lu and Chen, 2005), (Barbachyn et al., 2012), (Naish et al., 2013)].

1.1 Objectives

A comprehensive theoretical and parametrical research on non-linear behavior of coupled wall system in multi-story buildings will be carried out in this Ph.D. Thesis. The core of the study will be on illustrating the actual behavior for various types of coupled wall systems and compare the facts of results with up-to-date design codes and guidelines.

Previously with the current design procedure, the coupling ratio percentage is calculated with initial stiffness force-based design using elastic analysis. On the other hand, further studies have revealed that elastic analysis is not sufficient to provide accurate information for evaluating the distribution of forces in coupled walls and coupling beams.

Since the progression of plastic behavior on coupling beams and wall piers do not occur simultaneously, the coupling ratio of the coupled wall system cannot be expected as constant during the whole process of ground motion.

Thus, the aim of this research study is to investigate the effect of coupling ratio on nonlinear behavior of coupled shear walls. Variable behavior of the beam-coupling ratio over the increasing deformations will be examined and studied comprehensively with adequate number of mathematical model samples varied with affecting parameters such as aspect ratio. Delivering a recommendation on different coupling ratios alteration coefficients for design practice is also one of the main objectives of this PhD thesis.

Considering the interaction of axial force and bending moment in each wall member, axial forces transmitted by coupling beams increase or decrease the initial moment capacities of each pier. Since the coupling ratio is the key parameter affecting on axial force alteration under an earthquake, another aim of this research is to determine the role of coupling ratio on the disparities of moment capacities. Then, redistribution of moment demands allowance given with the design codes will be compared and evaluated. Depending on the possible increase of bending moment capacity in the

compression wall, the percentage amount of additive shear demand allocated will also be compared with the shear demands of the tension wall.

1.2 Scope of the Dissertation

The 8, 16 and 24-story coupled wall systems with different levels of coupling ratio located in a high seismic zone are investigated in detail to achieve the above mentioned study objectives.

For the elastic analyses, using the response spectrum analysis procedure will be utilized to establish the preliminary design of the coupled wall systems and to determine the design forces and moments. Nonlinear static and dynamic analyses will be conducted to investigate the coupled wall structural behavior, the disparities of demand forces between each wall piers, and the decrement rate of the coupling ratio during a seismic action.

The cross-sections of the investigated structural walls are rectangular and their analysis models have been developed in two-dimensional workspace. Therefore, torsional and out-of-plane effects in the structural system have been ignored. Additionally, the shear behavior of the wall piers has been assumed as linear. Soil-structure interaction was not considered during the designs and analyses.

2. LITERATURE REVIEW

This chapter presents the summary of key researches that had been previously carried out studies on solid walls, coupling beams and coupled wall systems. Experimental testing schedules for diagonally and conventionally reinforced coupling beams and load deformation relations for modeling issues are also investigated and presented.

2.1 Experimental Studies on Solid Walls

As it is known, if the height over length ratio (aspect ratio) of a flexure-controlled wall is higher than 2, then they are classified as slender walls. In the literature, there exists numerous amount of studies carried out about solid slender wall analysis, design and behavior.

Some of the key references are summarized below:

One of the comprehensive studies conducted on solid walls was performed by Oesterle et al. (1979). The study included sixteen 1/3-scaled solid walls with rectangular, barbell and flanged cross-sections. The length and thickness of all walls were 1.9m and 0.1m, respectively. The walls were 4.57m tall. The concrete strengths were carried between 22 MPa and 53 MPa.

Oesterle et al. (1979) observed that the cross-section type affects the behavior of the walls. The rectangular wall had lower out-of-plane stiffness due to the smaller width, causing out-of-plane buckling of the boundary elements under load reversals. Moreover, it was noted that the rectangular walls exhibit significantly lower bending capacity compared to flanged and barbell sections with the same lengths. The large boundary zones in the walls with barbell shape cross-section had large dowel action, which prevents horizontal shear failures. These enlarged boundary zones also minimized wall instability. The increased area of reinforcing steel in the boundary zones also provides a higher flexural capacity for the wall. The flanged section (H-shaped) performed similar flexural capacity compared to the barbell shaped sections.

The barbell shape or flanged sections are recommended for higher lateral load demands.

Pilakoutas and Elnashai (1995) investigated rectangular concrete walls having the same flexural reinforcement but different horizontal reinforcements. The aim of the research was to quantify the ductility and the energy dissipation potential of these walls. The test schedule consisted of six 1/2.5-scaled rectangular solid walls, which had an aspect ratio of 2.

Based on the test results, Pilakoutas and Elnashai (1995) noted that the strengths of specimen were not changed by shear reinforcement, if the required shear capacity had already been provided. The authors concluded that the shear deformations could not be considered toward overall energy dissipation, even if the shear deformations were significant (Figure 2.1).

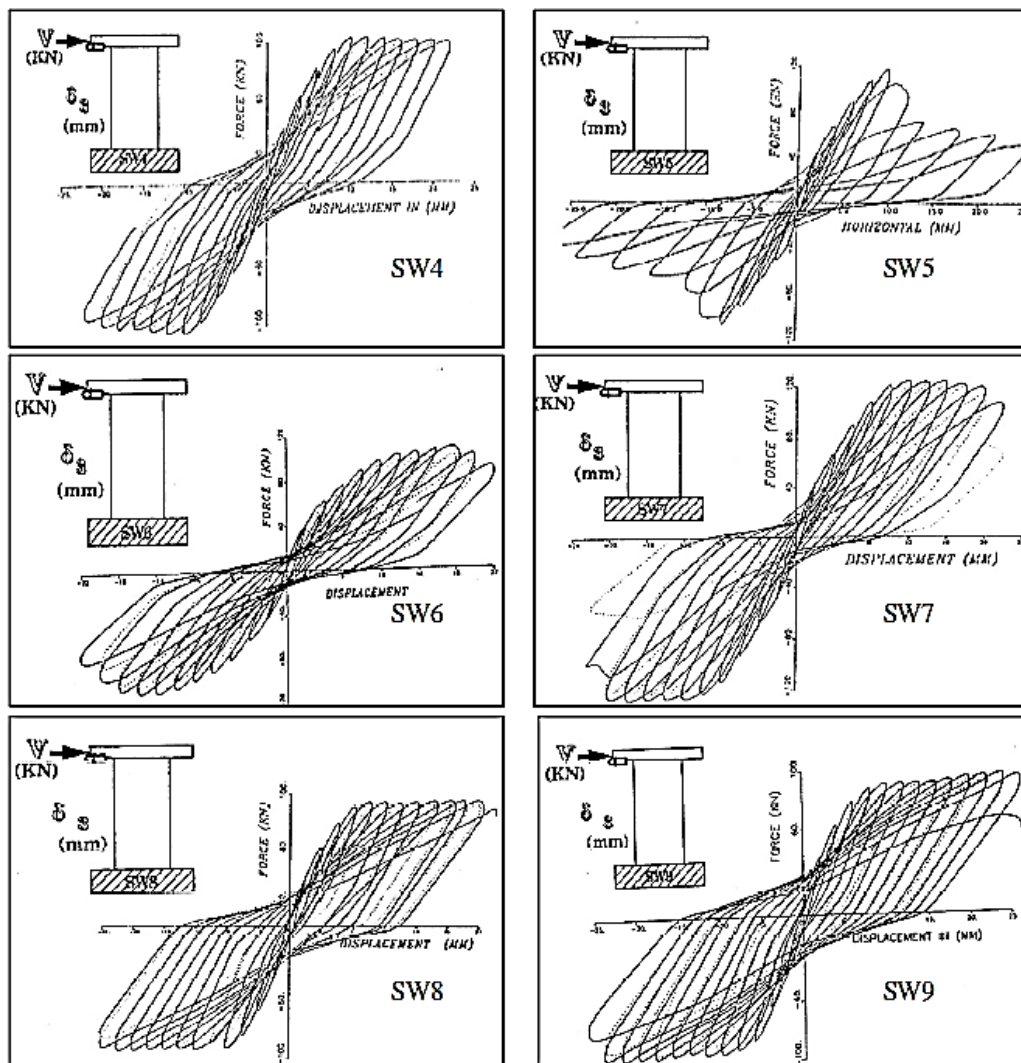


Figure 2.1 : Cyclic behavior of walls (Pilakoutas and Elnashai, 1995).

Thomsen and Wallace (1995) also made research on the behavior of structural walls with rectangular cross-sections. The wall dimensions were selected to be appropriate with typical office buildings. They used a displacement-based design procedure to estimate the inter-story drift ratios of the prototype building. The rectangular walls investigated were identified as RW1 and RW2. The reinforcement details and cross-section dimensions have been summarized in Figure 2.2 and Figure 2.3. The main difference between RW2 from RW1 is the spacing of transverse reinforcement steel inside the boundary elements.

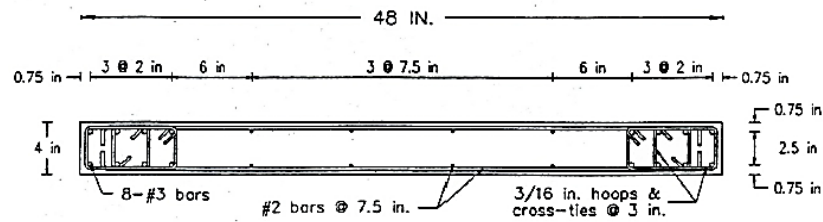


Figure 2.2 : Section of RW1 tested by Thomsen and Wallace (1995).

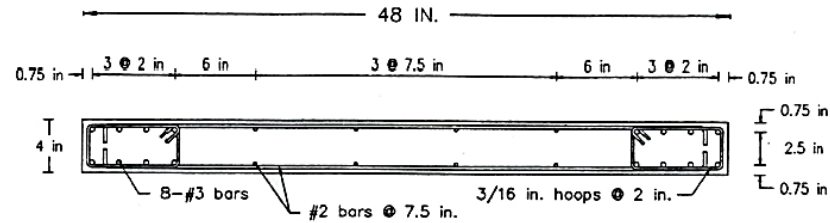


Figure 2.3 : Section of RW2 tested by Thomsen and Wallace (1995).

Under lateral loads, RW1 failed by buckling of all eight longitudinal bars in the boundary region at drift value of 1.5%. On the other hand, the reduced spacing of transverse reinforcement in RW2 delayed the bar buckling until a 2.5% lateral drift (Figure 2.4).

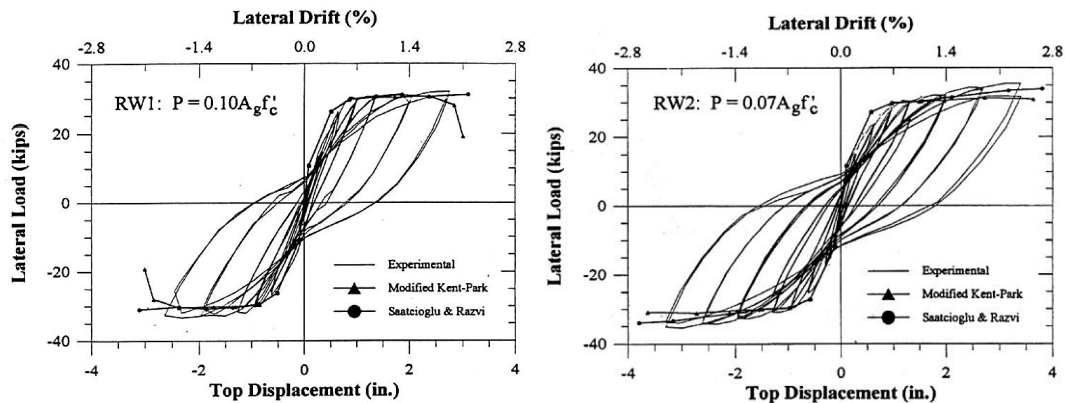


Figure 2.4 : Experimental results of Thomsen and Wallace (1995).

2.2 Analytical Studies on Coupled Wall Behavior

This subchapter summarizes several key studies, which investigated the behavior of coupled wall systems where coupling beams are connected to shear walls to provide system stiffness and ductility.

In University of Canterbury, Santhakumar and Paulay (1976) tested two coupled wall systems consisting of seven stories. While one of the coupled walls had conventionally reinforced coupling beams, the other had diagonally reinforced coupling beams. Wall failure was initiated in the tension wall in conventionally reinforced coupling beam case. In both cases, shear transfer from tension wall to compression wall was emphasized. It was observed that axial force difference due to coupling effect caused shear force demand on the compression wall became three times larger than the tension wall.

Further studies on coupled wall systems by Paulay (1980) revealed the importance of diagonal reinforcement layout to prevent shear failures especially in squat beams with small aspect ratios. Besides, he emphasized the fact that the use of diagonal reinforcement is necessary to provide adequate ductility in the coupled wall systems.

Kaya (1996) conducted analytical studies on coupled shear walls to determine the effective rigidity values and ductility values. In the parametric numerical analyses, the width of infinitely rigid portions of the coupling beams connected to the wall piers were also investigated. It was recommended that the zone between the ends of the coupling beams and the centerline of the wall pier should be taken as entirely rigid. Parametric studies also indicated that the coupled shear walls are more ductile than the solid shear walls.

Harries et al. (2000) conducted several studies on the design and behavior of coupling beams and their impact on the behavior of coupled wall systems. Well-designed coupling beams above the second floor of multi-story buildings are generally believed to develop plastic hinges simultaneously, with similar end rotations over the entire height of the structure. This mechanism obviously will allow energy dissipation to be distributed in the coupling beams over the building height, rather than primarily focused in the wall piers at the base of the building, thus producing better performance. Ideally, the mechanism through which the energy is dissipated should involve plastic hinges first in most of the beams and then at the base of the walls. The variables that

are used to achieve this performance are strength, stiffness, ductility and energy dissipation capacity.

Carbonari et al. (2008) investigated the effect of soil-pile-structure interaction (SSI) on the nonlinear response of coupled wall and frame systems. Globally, the soil-structure-interaction increased the lateral deformability of the whole structure; however, no significant differences were observed between the fixed base and the model with soil-structure interaction feature, since the plastic hinge development produced a shear redistribution between walls.

Mosoarca (2014) performed experimental tests on failure modes of coupled wall systems with ordered and staggered opening layouts as shown in Figure 2.8. It was found that the walls with staggered openings fail in a different way than those with ordered openings. The walls with staggered openings were more rigid and had a higher bearing capacity in comparison with the wall with ordered openings.

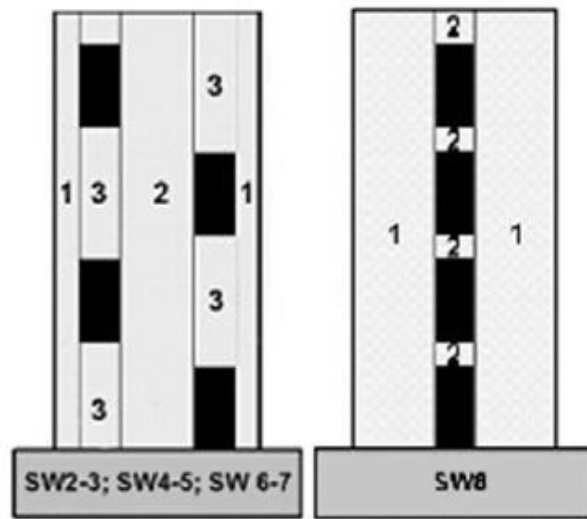


Figure 2.5 : Walls with staggered and ordered openings (Mosoarca, 2014).

2.3 Experimental Studies on Coupling Beam Behavior

The onset of coupled wall and coupling beam studies refer to the early 1970s. Paulay (1971) initiated tests at the University of Canterbury, investigating the behavior of deep concrete beams in shear walls.

Conventionally reinforced coupling beams, having aspect ratio (span to depth) of 1.02, were evaluated under monotonic loading. The test results invalidate the design

concepts of doubly reinforced beams since both top and bottom steel reinforcements can be in tension (Figure 2.6).

For the next experimentation, diagonal reinforcement layout was proposed for deep coupling beams having aspect ratio of 1.02 [(Binney, 1972), (Paulay and Binney, 1974)]. Under cyclic loading, diagonally reinforced coupling beams exhibited larger ductility values with less strength degradation. Almost in all tests, diagonal bundles were failed under buckling, therefore confinement around diagonal reinforcement bundles were proposed to extend the ductility.

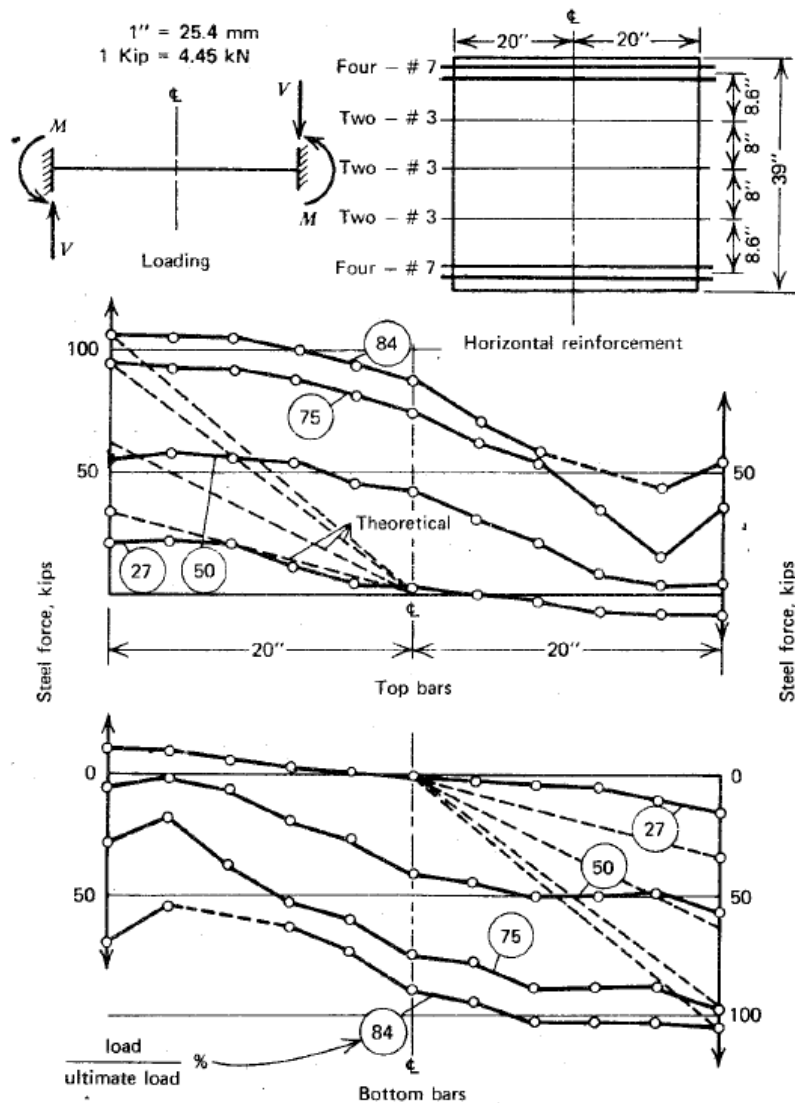


Figure 2.6 : Tension over the entire height of the deep beam (Paulay, 1971).

Barney et al. (1980), investigated relatively more slender coupling beams with aspect ratios 2.5 and 5.0 by also including alternative reinforcement details other than conventional and diagonal layouts (Figure 2.7). Based on test results, enhancing the coupling beam behavior by using diagonals within hinging locations was not efficient

due to its complexity and cost. Besides, full-length diagonally reinforced samples exhibited better ductility, strength and energy dissipation. The investigated beams reached displacement ductility values of 9.0 and 10.2 for aspect ratios of 2.5 and 5.0, respectively.

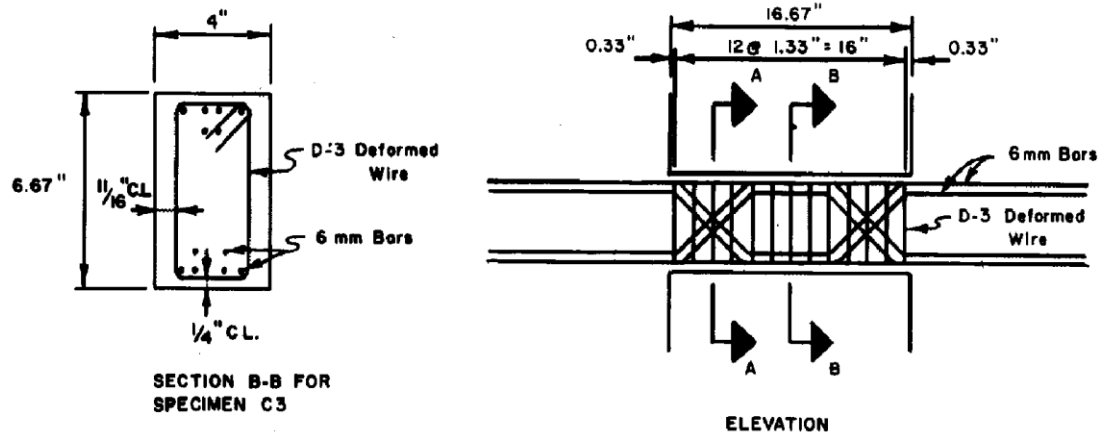


Figure 2.7 : Double diagonal bars at hinging locations (Barney et al., 1980).

In late 1990s, Xiao et al. (1999) investigated the behavior of high strength concrete coupling beams with conventional longitudinal reinforcement subjected to cyclic loading protocols. One of the main purposes of the study was to observe the difference between flexural reinforcement configurations by concentrating the flexural reinforcement at the beam edges (Figure 2.8).

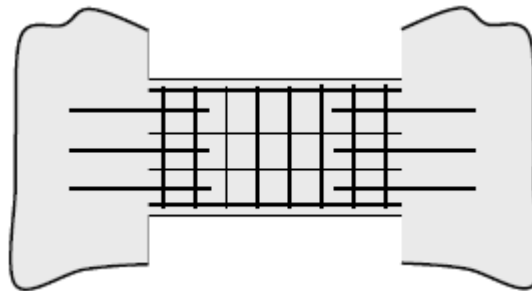


Figure 2.8 : Concentrating flexural reinforcements at edges (Xiao et al., 1999).

Galano and Vignoli (2000) performed studies on short coupling beams with different reinforcement layouts. Four different reinforcement configurations were tested with beams having span-to-depth ratio of 1.50: conventional straight reinforcement scheme, diagonal scheme without confining ties, diagonal scheme with confining ties and inclined bars reform a rhombic configuration. In rhombic configuration, the bars were bent to the horizontal over the last portion of the embedded length. According to the test results, rhombic configurations exhibited slightly better ductility and energy

dissipation rather than full-length diagonals. On the other hand, in rhombic option congestion of reinforcement occurs at wall edges. The displacement ductility of diagonal schemes are 7.0 and 5.0 for volumetric steel ratios of 0.39% and 0.31%.

Kwan and Zhao (2002) conducted experimental studies on five 1:2-scale conventionally reinforced and one diagonally reinforced coupling beams. Their aspect ratios vary between 1.00 and 2.00. On conventionally reinforced beams, displacement ductility increased with higher aspect ratio values and lower shear demands. On the other hand, diagonally reinforced beam with span to depth ratio (aspect ratio) of 1.17 performed better than the beams with conventional reinforcements.

Fortney (2005) also included one diagonally reinforced coupling beam in his experimental test schedule. The beam having aspect ratio of 2.56 was detailed with hoop bars around diagonal bundles. The observed displacement ductility value prior to strength degradation was 4.0 with maximum chord rotation of 5.8%.

In University of California Los Angeles, Naish et al. (2013) tested totally eight half-scaled coupling beams with 2.40 and 3.33 aspect ratios which were representative for residential and commercial buildings, respectively. The diagonally reinforced samples were separated into two groups in terms of confinement strategy: confinement around diagonal reinforcement bundles and full section confinement. It was observed that coupling beams with full section confinement (without additional hoops or ties around diagonal bundles) option performed better behavior in terms of strength and ductility compared to confined diagonal bundles. Test specimens reached significantly higher rotation values compared to ASCE 41-06 specified values prior to strength degradation. Fragility curve studies also revealed larger rotation limit states than code-specified performance limits.

Displacement ductility μ_d values obtained from some of the key experimental studies are summarized on Table 2.1. Here, L and h are the span length and the depth of the beams, respectively. In general, diagonally reinforced coupling beams exhibited larger ductility values compared to conventionally reinforced beams.

Table 2.1 : Ductility values of previously performed coupling beam tests.

Coupling Beam	L/h	Concrete [MPa]	Steel [MPa]	Reinforcement Layout	Analysis	μ_d
Paulay	1.02	-	-	conventional	monotonic	-
Barney C2	2.50	20.70	414	conventional	cyclic	7.8
Barney C5	2.50	20.70	414	conventional	cyclic	7.8
Barney C7	5.00	20.70	414	conventional	cyclic	10.0
Barney C1	2.50	20.70	414	double diagonal	cyclic	9.0
Barney C3	2.50	20.70	414	double diagonal	cyclic	9.0
Barney C4	2.50	20.70	414	double diagonal	cyclic	9.0
Barney C6	2.50	20.70	414	diagonal	cyclic	9.0
Barney C8	5.00	20.70	414	diagonal	cyclic	10.2
Tassios 01	1.00	22.60	507	diagonal	cyclic	5.6
Tassios 02	1.66	27.60	507	diagonal	cyclic	5.2
Xiao 01	3.00	68.90	482	conventional	cyclic	6.0
Xiao 02	4.00	68.90	482	conventional	cyclic	6.2
Galano 01	1.30	40.00	-	diagonal	cyclic	7.0
Galano 02	1.30	40.00	-	diagonal	cyclic	5.0
Kwan 01	1.17	37.90	517	conventional	cyclic	4.0
Kwan 02	1.17	37.90	517	conventional	cyclic	4.3
Kwan 03	1.40	37.90	517	conventional	cyclic	5.0
Kwan 04	1.75	37.90	517	conventional	cyclic	5.0
Kwan 05	2.00	37.90	517	conventional	cyclic	6.0
Fortney	2.56	37.90	431	diagonal	cyclic	4.0
Naish CB24F	2.40	47.20	482	diagonal	cyclic	9.5
Naish CB24D	2.40	47.20	482	diagonal	cyclic	8.7
Naish CB33F	3.33	47.20	482	diagonal	cyclic	9.0
Naish CB33D	3.33	47.20	482	diagonal	cyclic	8.8
Naish FB33	3.33	41.34	482	conventional	cyclic	9.8

2.4 Studies on Coupled Beam Modeling

For numerical analysis of coupled wall systems, coupling beams can be represented by using small finite elements with comprehensive expression of each local behavior based on stress and strain relationship [(Zhao et al., 2004), (Mohr et al., 2007), (Bower and Rassati, 2008), (Brena et al., 2009)].

Zhao et al. (2004) investigated the behavior of deep coupling beams using a nonlinear finite element method they recently developed. The concrete and steel are modeled together by a four-node isometric quadrilateral plane stress element with two extra non-conforming bending nodes intended to remove shear locking. The constitutive model of the element consists of two parts, one for the concrete and one for the steel. The concrete takes into consideration the biaxial behavior of the material, assuming isotropic behavior before cracking and orthotropic after cracking. The steel model assumes that the reinforcing bars are perfectly bonded to the concrete and uniformly smeared throughout the element. A trilinear stress-strain behavior is assumed.

The above method was used to analyze four monotonically loaded, conventionally reinforced coupling beams. The analytical models include the test specimen and the entire loading frame. The author was able to predict the ultimate strength with an average over-estimation of only 6.2%.

While very accurate, those sophisticated modeling techniques are generally time consuming in engineering practice.

Macro modeling techniques on the other hand, are simpler in practice and give adequately accurate results. Naish et al. (2013) recommends modeling diagonally reinforced concretes by using elastic frame element, which has linear rotation springs at each end to stimulate the slip/extension deformations and plastic rotational springs for flexural behavior. Lu and Chen (2005) suggested a nonlinear model for reinforced concrete members, which is composed of three sub elements: an elastic element with spread plastic hinges, an interface bond-slip sub-element and a shear sub-element. Alternatively, Barbachyn et al. (2012) proposed an approach with nine-node strut-and-tie model consisting of concrete and reinforcing steel elements.

3. DESIGN OF COUPLED WALL SYSTEMS

3.1 Force-based Design

In the conventional design approach, structures are proportioned such that their ultimate capacity is greater than or equal to the required ultimate load demands. The required strength is obtained from the most critical combination of factored loads, while the ultimate capacity of an element is obtained by applying reduction to the characteristic strength of materials or nominal strength of elements. The force-based design method with these loads and resistance factors (LRFD) is currently the basis of most codes including ACI (2011) and Turkish Earthquake Code (2007). Generally, applying a force-based design approach is prescriptive in nature; that is; the governing code prescribes how the design will be executed.

In dealing with coupled wall systems, the force-based approach assumes that under the effect of an earthquake loads the wall piers and the coupling beams yield simultaneously at the code-specified base shear. It has already been shown that this is an unrealistic behavior considering the different stiffness of the lateral load carrying mechanisms. Additionally, the traditional force-based of coupled walls often results in coupling beams with shear demands exceeding the permitted shear strength specified by the design code (Harries et al., 2005). For reducing these high shear stresses so that they fall within code limits, designers often use unrealistically high values of concrete strength, and/or reduce the effective stiffness of the coupling beams to very small values in their analyses. Neither approach is appropriate nor will capture the expected behavior of coupled wall systems (Harries and McNeice, 2006).

3.1.1 Turkish Earthquake Code requirements

The minimum length to width ratio requirement per Turkish Earthquake Code (TEC) (2007) for reinforced concrete wall elements is 7.0. The wall thickness shall not be less than 1/20 the story height and 200 mm unless both equations given below are satisfied:

$$\sum A_g / \sum A_p \geq 0.002 \quad (3.1)$$

$$V_b / \sum A_g \leq 0.5 f_{ctd} \quad (3.2)$$

where A_g is the section area of structural element at any story behaving as a structural wall in the direction parallel to the earthquake direction considered, and A_p is the plan area in a story. V_b is the base shear and f_{ctd} is the design tensile strength of concrete.

Special boundary zones must be provided on both ends of a wall element unless the wall is not squat. Those special boundary zones are provided with additional reinforcement bars and also confined with additional transverse reinforcements. The reinforcement layout of a typical wall is summarized in Figure 3.1.

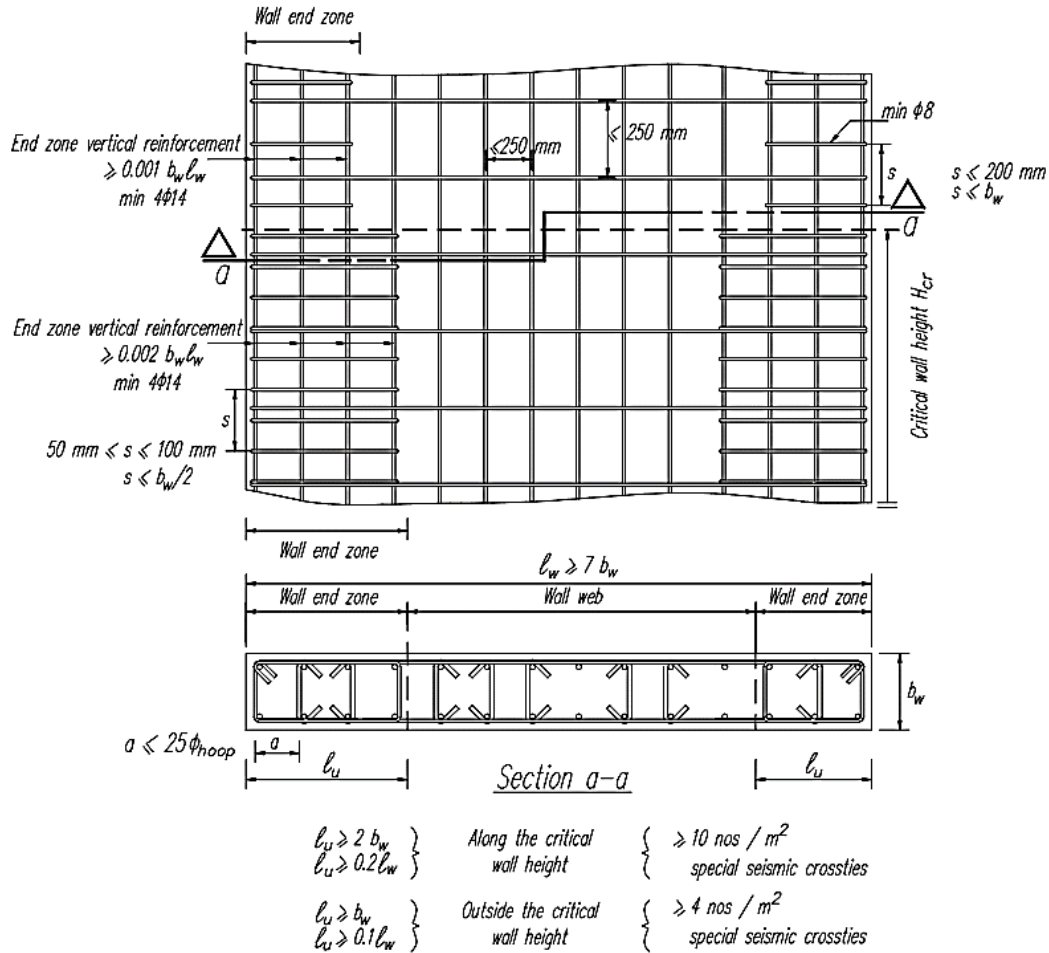


Figure 3.1 : Wall reinforcement layout (TEC, 2007).

Since plastic hinges generally are expected at the base of wall piers, the reinforcement requirements are increased from base up to a certain height, which is also called ‘critical height’ as H_{cr} as expressed in the following equation.

$$2l_w \geq H_{cr} \geq \max[l_w; H_w/6] \quad (3.3)$$

Here l_w and H_w are the length and the total height of the wall, respectively.

The bending moment envelopes in elevation should be idealized as shown in Figure 3.2 for walled or framed-walled systems. It needs to be constant along the critical height zone and goes linearly up to the top.

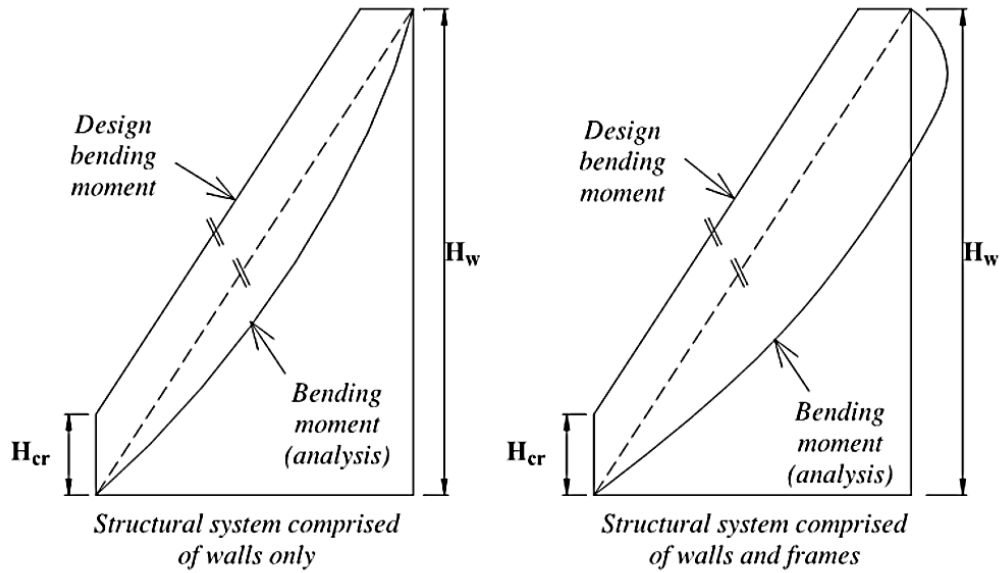


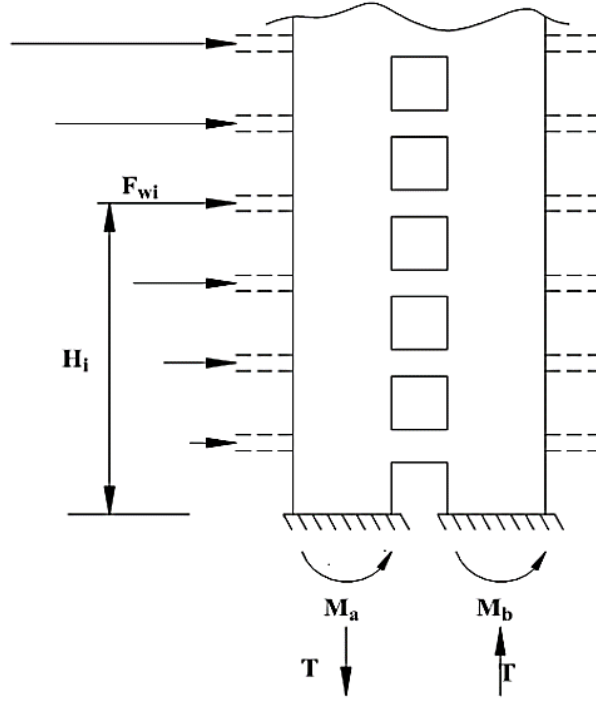
Figure 3.2 : Wall bending moment envelope (TEC, 2007).

For the coupled wall systems, it is also aimed that coupling moments contribute at least 1/3 of the system overturning moment. In other words, the sum of the wall moments should not exceed 2/3 of the overturning moment as shown in Figure 3.3.

The coupling beams have to be provided with diagonal reinforcement layout if both of the following conditions are not satisfied:

$$L/h \geq 3.0 \quad (3.4)$$

$$V_d \leq 1.5f_{cta}b_wd \quad (3.5)$$



$$(M_a + M_b) \leq 2/3 \sum F_{wi} H_i \quad F_{wi} : \text{Seismic load acting at } i\text{'th storey on coupled structural wall system}$$

Figure 3.3 : Minimum requirement of coupled wall system (TEC, 2007).

where L/h is the beam span-to-depth ratio, so called as the aspect ratio of coupling beams. V_d is the design shear force demand on the beam; b_w and d are the width and effective depth of the coupling beam, respectively. The diagonal reinforcement should be placed as shown in Figure 3.4. l_b is the minimum development length required for a reinforcing steel bar as specified in Turkish Standard TS500 (2000).

The minimum required reinforcement area, A_d , per each diagonal bundle is determined with the following equation, which is dependent on the angle of the diagonal layout.

$$A_d \geq V_d / (2f_{yd} \sin \alpha) \quad (3.6)$$

f_{yd} is the design strength of reinforcing steel material and α is the angle between the diagonal bundle and the horizontal plane.

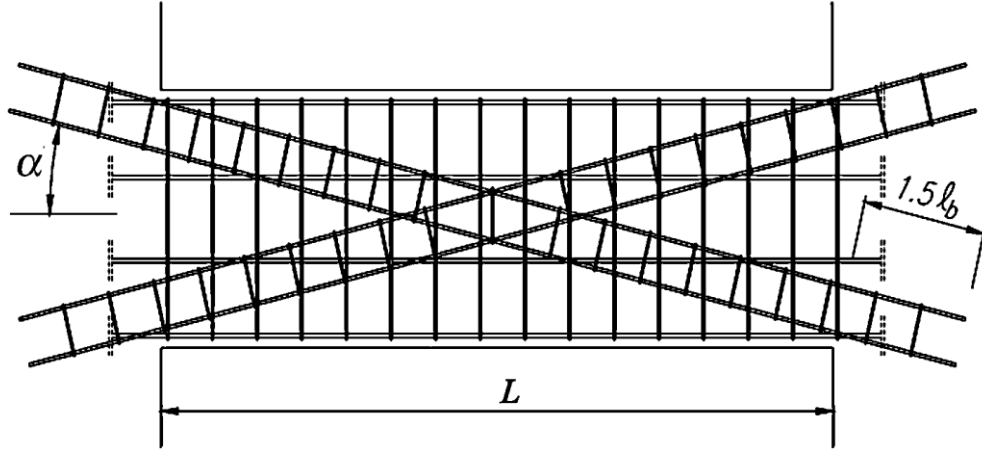


Figure 3.4 : Diagonal reinforcement layout (TEC, 2007).

3.1.2 ACI requirements

Depending on the span to depth ratio (aspect ratio), ACI 318 (2011) classifies coupling beams into three groups:

1- Aspect ratio ≥ 4 : Those beams are deemed shallow for the use of diagonal reinforcement layout efficiently. Therefore, those type of coupling beams are treated as conventional beams, which have the flexural reinforcement placed at the top and the bottom of the member.

2- Aspect ratio < 2 and with $V_u > 0.33\lambda\sqrt{f'_c}A_{cw}$: Here, V_u represents the factored shear force at coupling beam section; λ is the modification factor reflecting the reduced mechanical properties if lightweight concrete is used. f'_c and A_{cw} are the concrete compression strength and the coupling beam section area, respectively. The coupling beams falling in this group are required to be reinforced with two intersecting groups of diagonally reinforcement bundles as required in Turkish Earthquake Code (2007) explained in previous section.

3- Other: Remaining coupling beams, which do not fall in the two groups above, may be reinforced either with conventional layout or with diagonal layout, whichever is appropriate.

ACI 318 (2007) gives the opportunity to develop diagonally reinforced coupling beams with two alternatives: The first alternative is to confine only diagonal bundles by using hoops or crossties as shown in Figure 3.5. For this design, maximum allowed hoop spacing along the diagonal reinforcement is selected as the smaller of s_o and $6d_b$

of the diagonal bars, where s_o is the center-to-center spacing and d_b is the nominal diameter of the diagonal reinforcement bar.

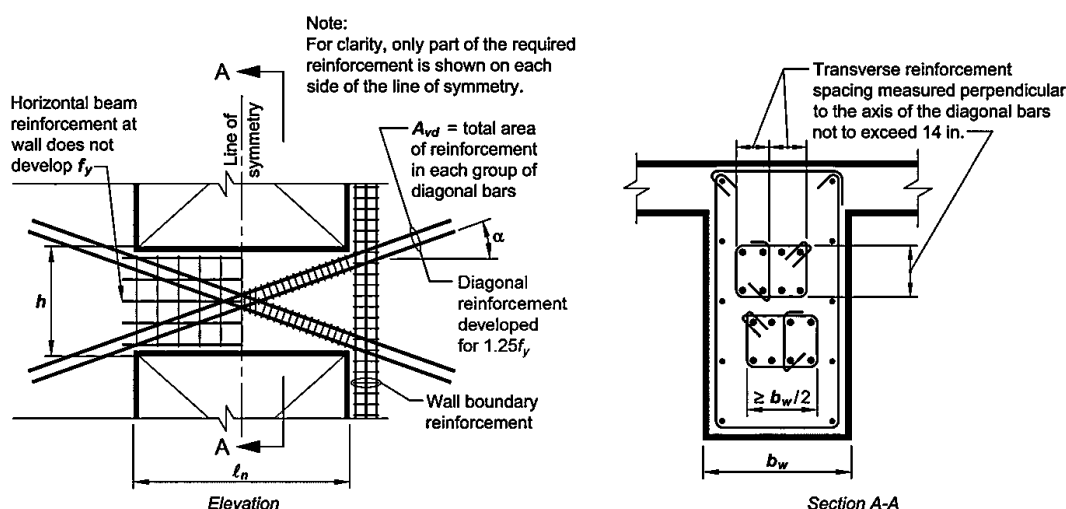


Figure 3.5 : Confinement of individual diagonals (ACI 318, 2011).

The other alternative is intended to overcome the construction difficulties raised by the congestion of reinforcements. By this option, hoops and crossties confine the entire beam section as shown in Figure 3.6.

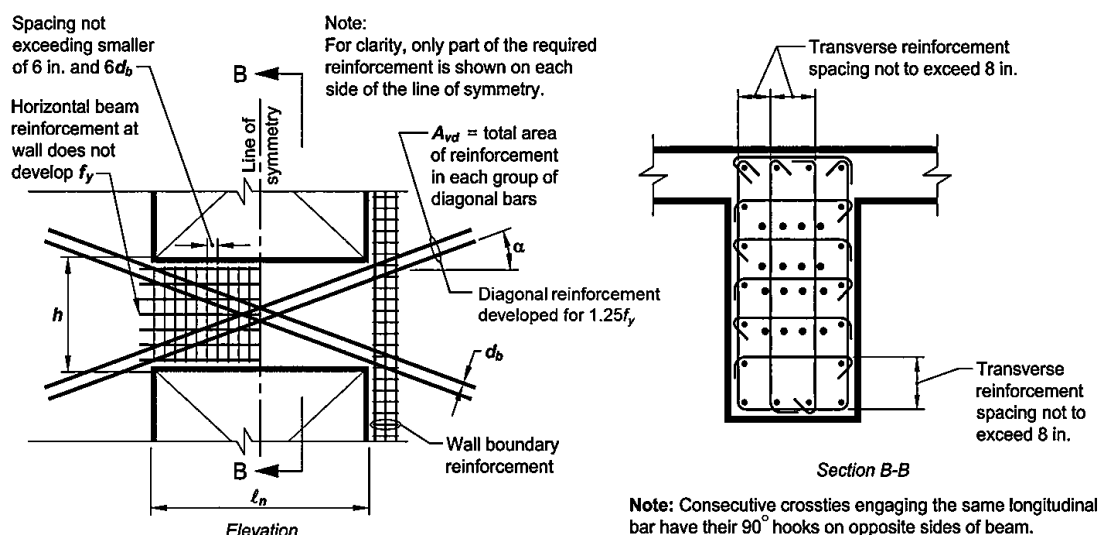


Figure 3.6 : Confinement of entire beam section (ACI 318, 2011).

3.2 Performance-based Design

Performance-based design (PBD) (SEAOC, 1995) has been an upward trend during the last decade in the practice of structural engineering, especially in relation to design under high seismic conditions or blast effects. Widely considered as evaluation

methodology (FEMA, 2000), performance-based tools are being recommended as alternatives to force-based methodologies and introduced as a design tool, typically in relation to specific member types or structural systems (El-Tawil et al., 2009). Commonly, PBD permits controlled plasticity in key structural members as long as anticipated structural and element performance criteria are satisfied. PBD allows the engineer to choose how the structural system will respond and provides the framework for selecting performance objectives for the structure. Performance targets are generally displacement-based objectives, which they can refer any aspect of building performance.

In PBD, multiple performance targets may be required to achieve rather than a discrete objective. In most building codes applications, for instance, the target performance of a structure is that it will satisfy Immediate Occupancy (IO) requirements for an expected service level earthquake (often defined as having a probability of exceedance of 50% in 50 years), Life Safety (LS) requirements at the design level earthquake (10% in 50 years) and Collapse Prevention (CP) requirements at the maximum considered earthquake (2% in 50 years). Allowable deformations or damage levels for CP are greater than those for LS which are, in return, greater than those for IO. Therefore, a spectrum of performance results considering both the seismic load and structural performance (ASCE 41, 2013).

The most critical step in the procedure of PBD is the preference of ‘allowed behavior’ for the structure or individual components for a given earthquake event. ASCE 41 (2013) provides these as the building’s acceptance criteria, as tabular information based on plastic hinge rotation capacities for common structural systems and their members.

For coupled wall systems, Paulay (2002) provides a method and a rational procedure for determining inelastic deformation capacity of diagonally reinforced coupling beams and wall piers. Paulay’s rationale is based on fundamental mechanics of diagonally reinforced coupling beams and appears to estimate well experimentally observed behavior of diagonally reinforced coupling beams reported in the literature as reviewed by McNeice (2004) and Naish et al. (2013).

3.3 Displacement-based Design

The design procedure known as Direct Displacement-Based Design (DDBD) has been studied and developed over the past fifteen years with the aim of mitigating the inadequacies in current force-based design (Priestley et al., 2007). The main phenomena behind the approach is to design a structure which would accomplish a defined performance limit state under a given seismic intensity. The design procedure evaluates the capacities required at designated plastic regions to achieve the design aims in terms of displacement objectives.

The design method assumes a single-degree-of-freedom representation, which is applicable to all structural types, including coupled structural walls. The assumed bi-linear behavior of the lateral force-displacement response of the single-degree-of-freedom is shown in Figure 3.7. An initial elastic stiffness K_i is followed by a post yield stiffness of rK_i . While the force-based seismic design characterizes a structure in terms of elastic, pre-yield properties, DDBD method evaluate the structure by secant stiffness K_e at maximum displacement Δ_d . Although being very practical, direct displacement-based design method was not used in this dissertation since the non-linear behavior of coupled walls will be evaluated as multi-degree-of-freedom systems.

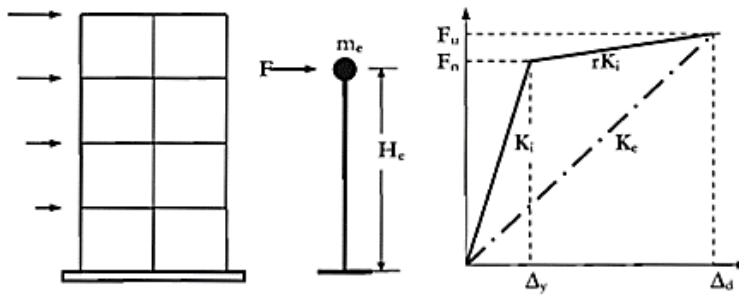


Figure 3.7 : DDBD Single-DOF representation (Priestley et al., 2007).

With the design displacement at maximum response determined, the effective period T_e at maximum displacement response, measured at the effective height H_e can be determined from a set of displacement spectra for different levels of damping. The effective stiffness K_e of the equivalent single-degree-of-freedom system at maximum displacement can be found by inverting the normal equation for the period of a single-degree-of-freedom oscillator, given as in the equation below:

$$K_e = 4\pi^2 m_e / T_e^2 \quad (3.7)$$

where m_e is the effective mass of the structure participating in the fundamental mode of vibration from Figure 3.7. The design force, which is also the design base shear force, can be calculated as shown below:

$$F_e = K_e \Delta_d \quad (3.8)$$

3.3.1 Coupling ratio

In the Direct Displacement-Based Design (DDBD) principles, it is suggested assuming a coupling ratio a priori as the first step of the design procedure. The presumed coupling ratio value adopted for design procedure will normally be in the range of 25% to 75%, and it must be chosen such that at wall boundary zone vertical reinforcement bars are within acceptable limits. Presumed coupling ratio value should also not be so large that the shear forces in the coupling beams induce a tension force at the wall base that might exceed the gravity load.

3.3.2 Effective height

It is evaluated by trial analysis that the displacement profile of the walls at design response can be assumed as linearly increasing with height (Priestley, 2007). Given the discrete nature of the lateral floor forces, the ratio of effective height to total height varies with the number of stories, n . Assuming that floor masses and story heights are equal, the ratio of effective height H_e to total height H_n can be expressed below as follows:

$$H_e/H_n = \left(\sum_{i=1}^n (m_i \Delta_i H_i) / \sum_{i=1}^n (m_i \Delta_i) \right) / H_n \quad (3.9)$$

where m_i is the mass and Δ_i is the displacement at i -th story. Then, Eq. (3.9) is plotted against number of stories, n as shown in Figure 3.8(a). For $8 < n < 20$. A value of 0.7 is a reasonable approximation due to the results shown on the figure. It can also be observed from the plot that the effective height ratio is inversely proportional with the amount of floor numbers exist in a structure (Priestley, 2007).

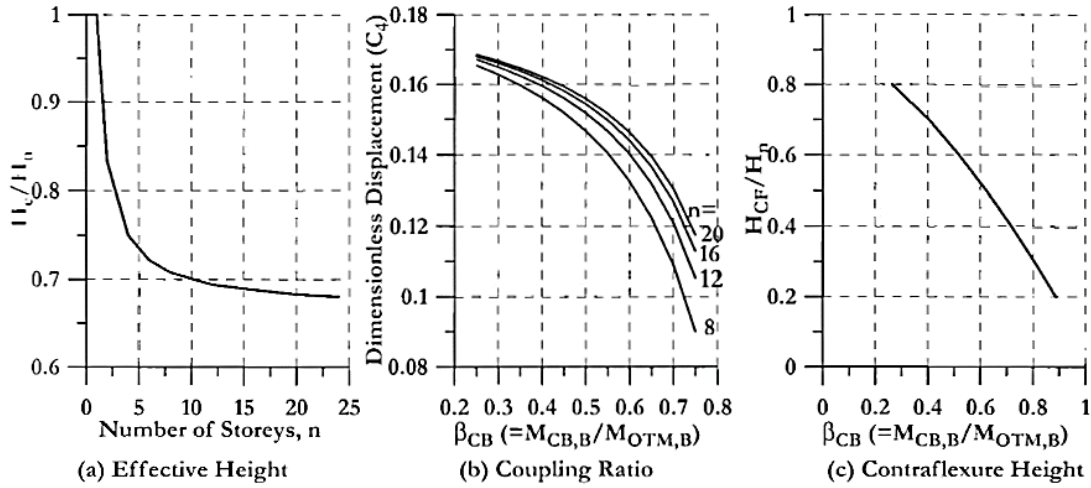


Figure 3.8 : Design parameters for regular coupled walls (Priestley et al., 2007).

3.3.3 Wall yield displacement

The yield displacement at the effective height can be found by subtracting the displacement associated with the coupling-beam moment profile, Δ_{y2} from the displacement associated with the total overturning moment profile, Δ_{y1} .

$$\Delta_y = \Delta_{y1} - \Delta_{y2} = \left(\frac{0.175}{1 - \beta_{CB}} - \left(\frac{\beta_{CB}}{1 - \beta_{CB}} \right) \left(\frac{0.1225 + 0.188n}{n} \right) \right) \phi_{yW} H_n^2 = C_4 \phi_{yW} H_n^2 \quad (3.10)$$

where ϕ_{yW} is the average yield curvature at the wall base and β_{CB} is the coupling Ratio CR.

Note that the equation above has been expressed in terms of the full wall height H_n rather than the effective height. The dimensionless coefficient C_4 has been plotted for the practical range of coupling ratio between 0.25 and 0.75 as given in Figure 3.8(b) for walls with various number of stories.

Figure 3.8(c) includes an additional parameter that is required in the design process that is, the ratio of the height of the point of contraflexure H_{CF} in the wall moment profile to the total wall height H_n . This defines the height at which the drift will be maximum. Contraflexure elevation ratio is also inversely proportional with presumed coupling ratio.

Design procedure starts with an engineer's judgement over a coupling ratio following the choice of a structural geometry. The steps of the procedure for designing coupled walls with direct displacement-based design are summarized in Figure 3.9.

The actual coupling ratio of the wall system is investigated and compared with the initially assumed value. The nonlinear behavior of the system is then examined by a simple pushover analysis with uniform and triangular loading patterns. As a numerical application, the design of an 8-story coupled wall system has been performed to examine the change of the coupling ratio and its verification using DDBD (Toprak et al., 2012).

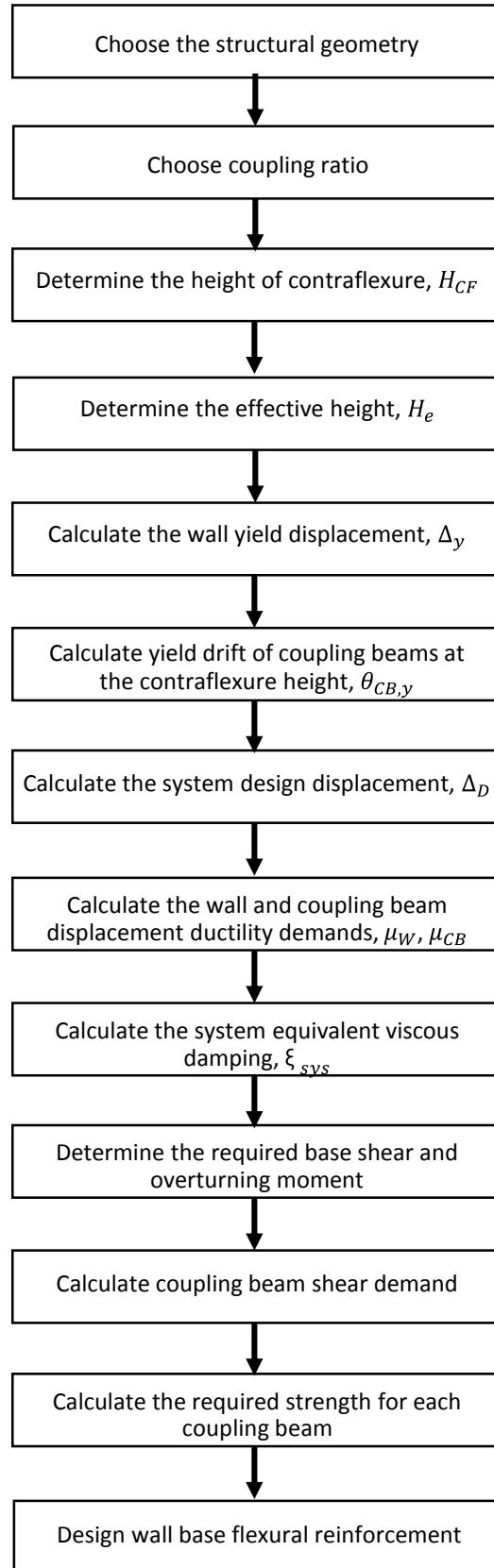


Figure 3.9 : Displacement-based design steps for coupled wall systems.

4. MODELING STRATEGY FOR COUPLING BEAMS

4.1 Micro Modeling Approach for Coupling Beams

For the analysis and design of tall buildings, it is essential that the nonlinear behavior of structural walls and coupling beams are correctly understood and represented in the numerical models. The inelastic response of diagonally reinforced concrete coupling beams can be represented by using either finite elements with comprehensive expression of each local behavior or macro models. Micro-modeling is based on constitutive material models for steel and concrete, which are more accurate, detailed and suitable for cases of analyzing the behavior of structural elements derived from experiments (Zhao et al., 2004, Mohr et al., 2007, Bower and Rassati, 2008, Brena et al., 2009). While very accurate, micro models remain limited in their use mainly because they must be constructed by experienced engineers in order to achieve reliable results. Moreover, the software required for performing analysis are generally comparatively expensive and time consuming.

4.2 Macro Modeling Alternatives for Diagonally Reinforced Coupling Beams

Recent studies are focused on macro models, since they are simpler in practice comparing to aforementioned micro modeling approaches, yet give satisfactory results. A complete model for coupling beams should represent flexure, shear and interface bond-slip mechanisms together. Available models are described below in detail.

4.2.1 Naish model

Naish et al. (2013) proposed a mathematical model for diagonally reinforced coupling beams that consists of an elastic beam cross-section with effective initial stiffness of $EI_{eff} = 0.5EI_{gross}$, elastic-rotation springs (hinges) at each beam-end to simulate the effects of slip/extension deformations, and rigid plastic rotational hinges at each beam-end to simulate the effects of non-linear deformations as shown in Figure 4.1.

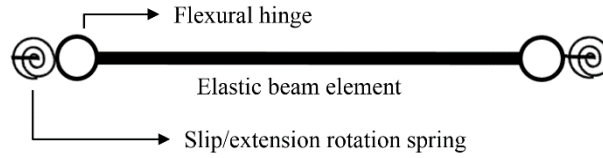


Figure 4.1 : Coupling beam modeling components (Naish et al. 2013).

The stiffness of the slip/extension hinges are defined using the model given by Alsiwat and Saatcioglu (1992), whereas the nonlinear flexural hinges use backbone relations that are developed from previously performed test results as given in Figure 4.2. In the figure, M_u represents the bending moment capacity corresponding to ultimate strength of the beam. Naish et al. (2013) assume the ultimate strength of the beam as 110% of the nominal strength specified by ACI 318-11 (2011). Hysteretic loops and fragility curves derived from the corresponding test results show that properly detailed diagonally-reinforced coupling beam can exhibit more ductile behavior than the limit states given in ASCE 41-06 (2007). Based on the studies by Naish et al. (2013), the increased allowable plastic rotation values are recently recommended in the updated standard (ASCE 41-13, 2014). On the other hand, the form of the back-bone curve is kept as in the previous one. Acceptable plastic hinge rotations for each performance level limit state are summarized in Table 4.1, where IO stands for the Immediate Occupancy, LS is Life Safety and CP is the Collapse Prevention performance levels.

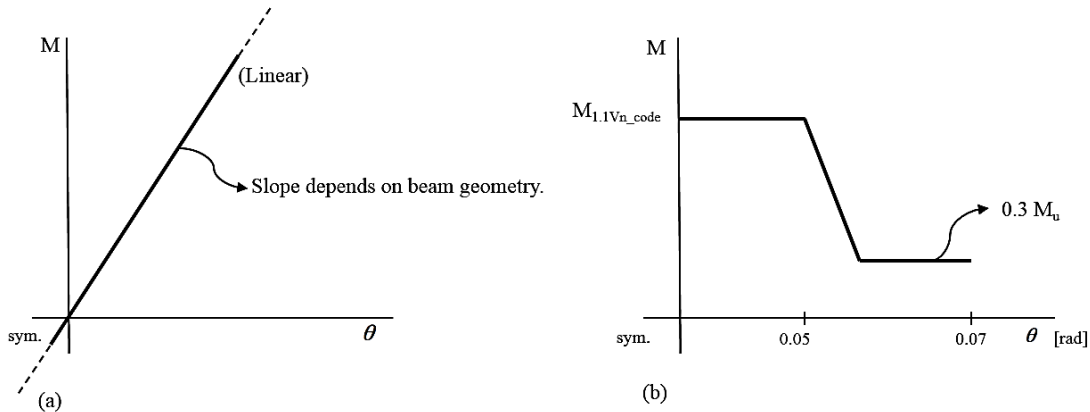


Figure 4.2 : Naish et al. (2013) back bone curves, (a) slip/extension, (b) flexure.

One of the main components of plastic deformation in reinforced concrete is the slip. Reinforcement slip in coupled wall system primarily occurs at the beam-wall interface where flexural behavior is located.

Table 4.1 : Plastic hinge rotation limits for diagonally reinforced coupling beams.

	Acceptable plastic hinge rotation [rad]		
	IO	LS	CP
ASCE 41-06 (2006)	0.006	0.018	0.030
Naish et al. (2013) aspect ratio <2.0	0.006	0.018	0.050
ASCE 41-13 (2014)	0.006	0.030	0.050

Formation of a flexural crack at the interface causes the reinforcement crossing the crack to have larger strains. The widening of this crack may produce plastic strains, which are likely to cause significant extension in the reinforcement. The combined behavior of reinforcement extension and additional rigid-body deformation due to slippage is termed as “*slip and extension*” state in reinforced concrete. As shown in Figure 4.3, Alsiwat and Saatcioglu (1992) state that the phenomenon of “*slip and extension*” behavior of the flexural reinforcement contributes almost 50% of the beam deformation prior to yield. The equations given below are utilized for the case of determining the slip/extension rotations at the yield point of the flexural reinforcement.

$$\delta_{tot} = \delta_s + \delta_{exty} \quad (4.1)$$

$$\theta = \frac{\delta_{toty}}{d - x} \quad (4.2)$$

$$K = \frac{M_y}{\theta} \quad (4.3)$$

In the expressions above, δ_s and δ_{exty} represents the slip of the reinforcement and the extension of the bar due to accumulation of strain along its length at yield, respectively. δ_{tot} stands for the total displacement of the bar at the beam-wall interface due to the slip/extension of the bar at yield, and θ is the rotational angle of crack at the beam-wall interface due to the slip/extension of the bar at yield point, while K is the stiffness of the corresponding moment-rotation hinge that is implemented in the mathematical model (Figure 4.3). Here, d represents the effective depth, which can be measured as the distance from the beam top edge to the diagonal centerline at wall interface. x is

the depth of the compression zone and it is recommended not to assume larger than the one-third of the total depth of the beam.

Although ASCE 41-13 (2014) prescribes the effective flexural stiffness to be $0.3EI_{gross}$, Naish et al. (2013) recommend the equivalent stiffness as 15-20% of the gross section flexural stiffness in coupling beams by accounting the slip and extension behavior.

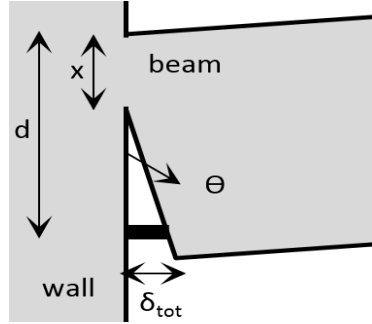


Figure 4.3 : Slip/extension crack and corresponding rotation.

Turkish Earthquake Code (2007) specifies that the diagonal bundles must be extended into the wall with a minimum 1.5 times of the anchorage length l_b . This anchorage length can be estimated with the following equation:

$$l_b = \left(0.12 \frac{f_{yd}}{f_{ctd}} \phi \right) \geq 20\phi \quad (4.4)$$

where f_{yd} is the design yield strength of the steel reinforcement and f_{ctd} is the design tensile strength of the concrete, ϕ is the diameter of an individual diagonal reinforcement. When the code requirement is adopted with a minimum anchorage length of 30ϕ , then it is expected that the contribution of slip would be negligible against extension of the reinforcement as shown in Figure 4.4.

Naish et al. (2013) do not use shear hinges in addition to flexural hinges and slip/extension rotation springs, because shear deformations in diagonal reinforced coupling beams are expected to be very small relative to slip/extension deformations.

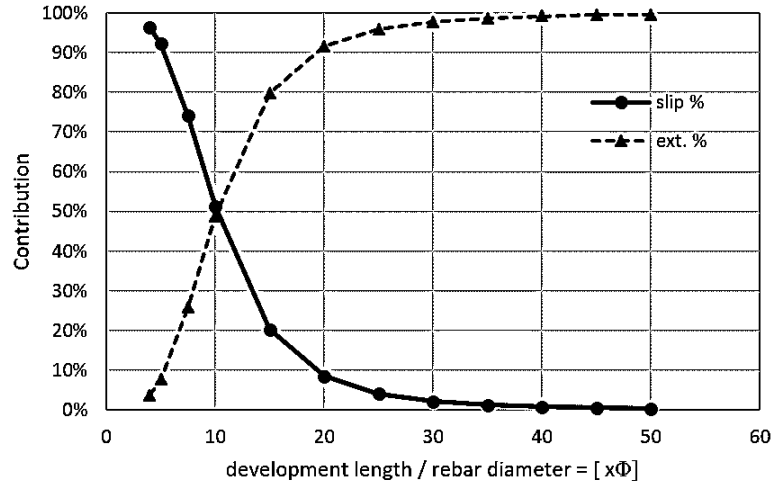


Figure 4.4 : Contributions of slip and extension behaviors.

4.2.2 Lu and Chen model

Lu and Chen (2005) suggested a nonlinear model for diagonally reinforced coupling beam members, which is composed of one elastic element with spread flexural plastic hinges and two interface bond-slip sub-elements called slip/extension rotation springs at both ends and a shear sub-element (shear hinge), as shown in Figure 4.5. This sub-element approach based on the previous study by D'Ambrisi and Filippou (1999) in which sub-elements have parallel connections; therefore, they have the same force vector at both ends and their individual displacements are added together for total deformation.

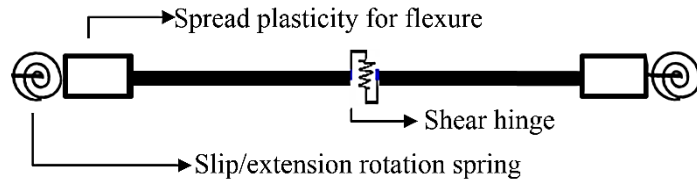


Figure 4.5 : Subelements of coupling beam (Lu and Chen, 2005).

For the interface bond-slip sub-element, a bilinear relationship is used with hardening equal to 0.04. In the shear sub-element a bilinear relationship is also employed for the force-displacement relationship with 0.01 hardening. The slip/extension hinge, the flexure hinge along with the shear hinge backbones are illustrated in Figure 4.6. There, M_y represents the yield moment of the component, G is the shear modulus of the concrete material. A and L are the cross-section area and the length of the coupling beam, respectively.

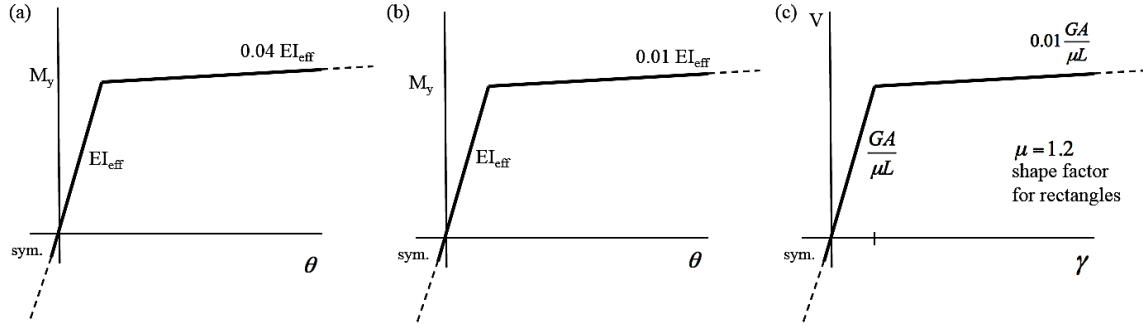


Figure 4.6 : Lu and Chen (2005) back bone curves (a) slip/extension hinge, (b) flexure hinge and (c) shear hinge.

4.2.3 Barbachyn model

Barbachyn et al. (2012) proposed an approach with a nine-node two-dimensional strut-and-tie model consisting of concrete, reinforced concrete, and steel elements as shown in Figure 4.7. The compression and tension behavior of diagonally placed reinforcement naturally leads to this model configuration.

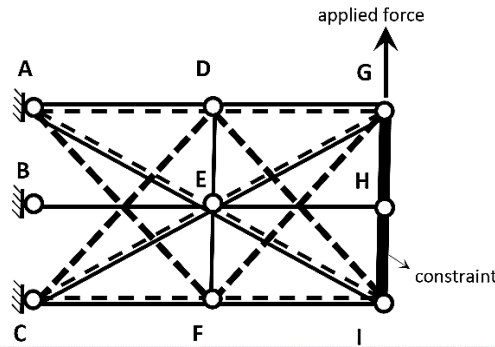


Figure 4.7 : Nine-node strut-and-tie model (Barbachyn et al. 2012).

The steel elements (solid lines) represent the reinforcement resisting both in tension and compression, whereas the concrete elements (dashed elements) are assumed to resist only compression by ignoring the tension strength of concrete. The RC elements (solid-dashed lines) have both steel and concrete members. The goal was to build a simple, yet sufficiently accurate model that can isolate the primary lateral load-resisting components of a coupling beam with a strut and tie model, thus allowing each component to be evaluated separately.

As shown in Figure 4.7, Nodes A, C, G and I are located at the corners of the model where centroids of the diagonal bar groups intersect with the top and bottom longitudinal bar groups in beam. Generally, these nodes are located inside the adjacent wall piers at concrete cover distance, thus providing a more realistic representation of

the beam, rather than simply being placed at the beam ends. The other nodes are placed equidistantly vertically and horizontally between the four corner nodes with one additional node at the center so that a symmetric nine-node array is formed.

The main diagonal elements between nodes A-E-I and nodes C-E-G are of reinforced concrete, providing the primary lateral load resistance of the beam. The steel area is equal to the total area of the developed bars in each diagonal bar group. The concrete strut area is taken as the core area enclosed by the centerline of the steel hoop confining each diagonal bar group.

Four strut elements extend between nodes A-F, C-D, G-F, and I-D to model the concrete in compression from the beam ends to the mid-span. Barbachyn et al. (2012) assumed that the concrete in these elements have the same area as the main diagonal struts.

4.3 Verification of Coupling Beam Macro Models with Previous Test Results

The recent studies on coupling beams are focused on macro models due to their simplicity and adequate accuracy for design applications. In order to select a proper macro modeling for the analyses to be performed in this thesis, three distinct macro modeling alternatives explained in the previous section were examined and modeled with OpenSees (2013) software for the verifications with previously performed coupling beam test results.

For the verification of macro models explained above, three test specimen were selected and re-evaluated: CB24 and CB33 specimen tested at University of California Los Angeles (UCLA) (Naish et al. 2013) and C6 as part of a test campaign for Portland Cement Association (Barney et al., 1980).

The test setup in UCLA Structural/Earthquake Engineering Research Laboratory is shown in Figure 4.8, where the test specimen was placed in a vertical position with end blocks simulating all boundary zones at each beam end. The top and bottom surfaces of the end blocks were grouted and post-tensioned to the steel reaction frame at top and to the laboratory strong floor at bottom to minimize slip between the surfaces as well as to provide for fixed end conditions. Two vertical hydraulic actuators on each side of the beam specimen were used to ensure zero rotation at the top of the specimen,

while maintaining constant (zero) axial force in the beam. The test result were digitally obtained from Network for Earthquake Engineering Simulation (NEES) website.

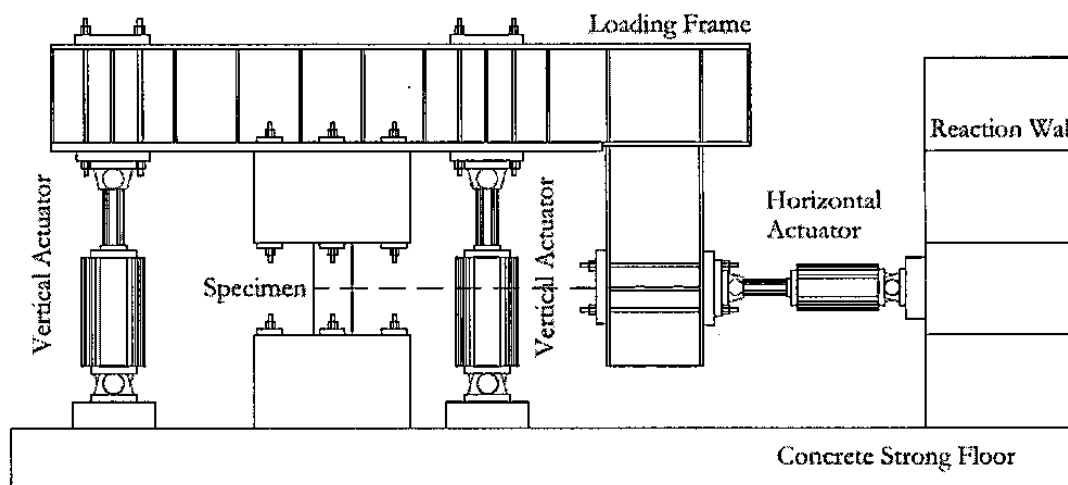


Figure 4.8 : UCLA laboratory test setup for CB24 and CB33 (Naish et al., 2013).

Regarding the reinforcement details: each bundle of diagonal reinforcement in beams CB24 and CB33 includes 6#7 bars ($\phi=25.4\text{mm}$), while 10 and 12 additional longitudinal bars of size #3 ($\phi=9.53\text{mm}$), are located in the perimeter of the beam, respectively. The clear cover of the concrete is 19 mm measured from the outer edge of the stirrups in both CB24 and CB33.

The test setup of Portland Cement Association (PCA) performed for specimen C6 is shown in Figure 4.9. The configuration of the test specimen was selected so that applied load and deformations represented on beams in coupled walls subjected to lateral forces. The specimen consisted of two coupling beams framing into abutment walls at each end. End conditions imposed by abutments represented those at the beam-wall interaction of coupled walls. The researchers state that the axial forces in coupling beams were not considered in the investigation.

The primary reinforcement of specimen C6 consists of full-length diagonals with 2#3 bars ($\phi=9.53\text{mm}$) in one direction and 1#4 bar ($\phi=12.7\text{mm}$) in the other. Symmetry was maintained by passing the #4 bar ($\phi=12.7\text{mm}$) between the #3 bars ($\phi=9.53\text{mm}$) at mid-span. Two 6-mm diameter longitudinal bars were provided for hoops in place.

Their initial stiffness estimation, overall hysteretic behavior plots overlap and cumulative energy dissipations under low and high drifts have been evaluated in comparison with the experimental cyclic response results. The dimensions of the

beams, the reinforcement area, as well as the material strengths are summarized in Table 4.2.

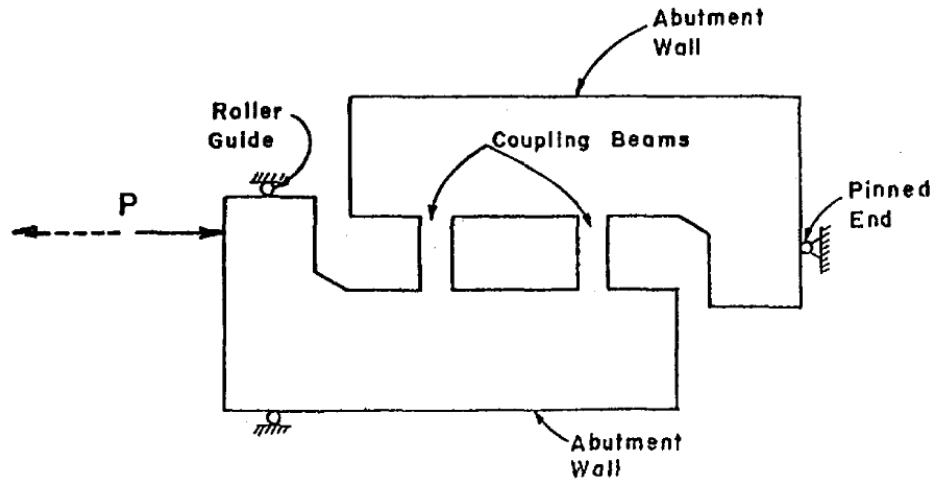


Figure 4.9 : PCA laboratory test setup for C6.

Table 4.2 : Test beams (Naish et al. 2013, Barney et al. 1980).

Beam	Test Institute	Aspect Ratio (l/h)	Span (L) [mm]	Depth (h) [mm]	Diagonal [mm ²]	f'_c [MPa]	f_y [MPa]	f_u [MPa]
CB24	UCLA	2.40	914	381	2321	47.2	480	617
CB33	UCLA	3.33	1524	457	2321	47.2	480	617
C6	PCA	2.50	423	169	127 & 142	18.1	488	722

OpenSees (2013) analysis software has been used to construct the aforementioned coupled beam macro model alternatives. Zero-length elements were introduced in the mathematical model to represent the slip/extension springs and flexure-rotation hinges of coupling beam macro model proposed by Naish et al. (2013). The cracked stiffness of the elastic beam element between the nodes 3 and 4 is taken as half the gross one, as shown in Figure 4.10. The effective stiffness is reduced to a lower value by these slip/extension linear rotational springs. The backbone of flexure hinges is taken as previously defined in Figure 4.2.

Again, zero-length elements were introduced in OpenSees (2013) to represent the shear and slip/extension sub-elements for the Lu and Chen (2005) coupling beam macro model shown in Figure 4.11. The shear hinge is placed between the nodes 2 and 3 in order to avoid meshing the beam element in the middle. Spread plastic sub-

element is represented by an element, which is based on a force-based formulation that allows the formation of plastic hinges at any location. The behavior at a section is described by a stress resultant plasticity model. Number of sections and their locations are determined by the Gauss-Lobatto rule used to integrate the element's force-deformation relationship (Scott et al., 2006).

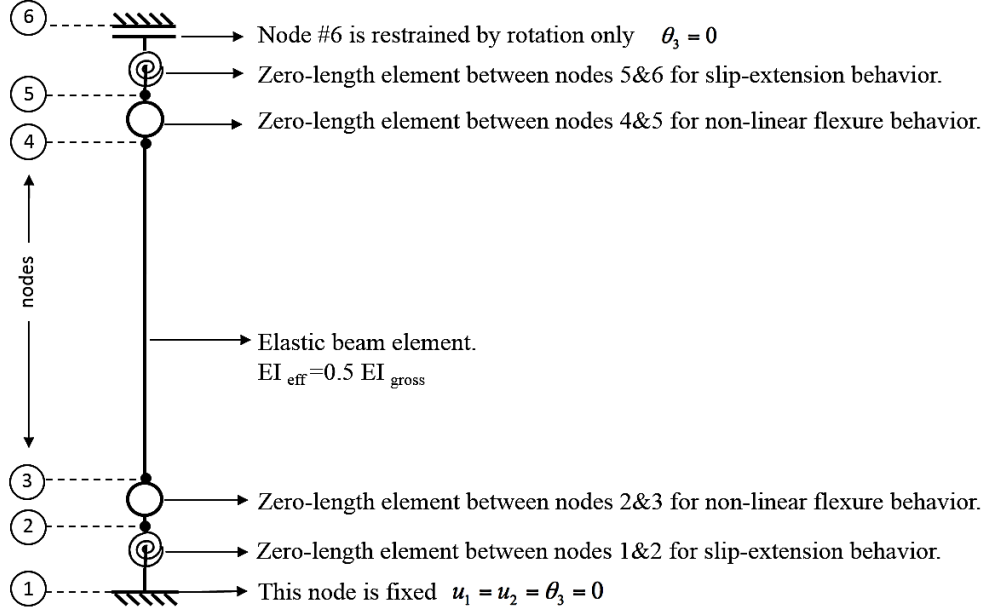


Figure 4.10 : Naish et al. (2013) model schematic developed for OpenSees.

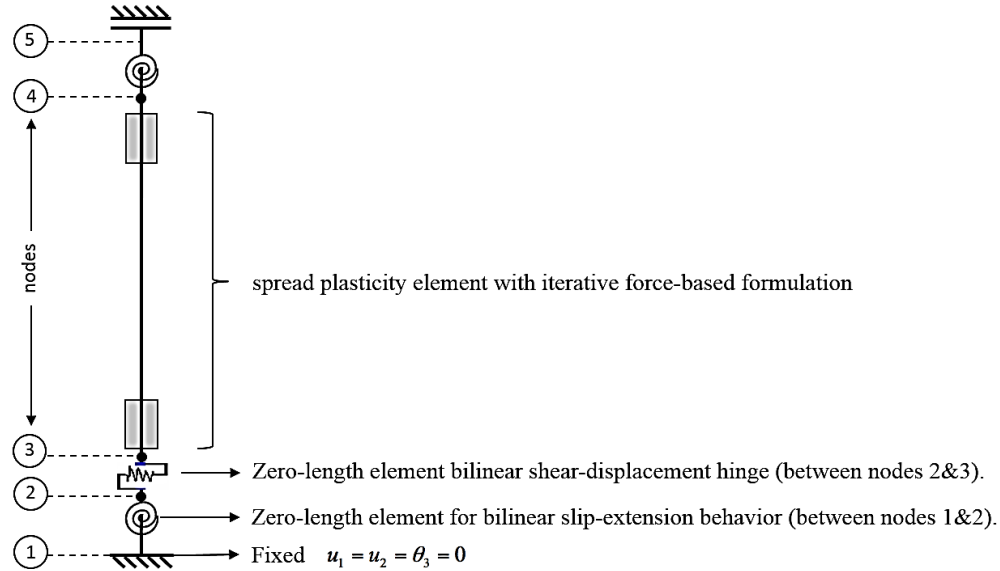


Figure 4.11 : Lu and Chen (2005) model schematic developed for OpenSees.

The three test specimen were also modeled with strut-and-tie method proposed by Barbachyn et al. (2012) in order to check that macro model with the test results.

The lateral load versus beam-end rotation cyclic responses, as obtained by the numerical analyses, are plotted and illustrated in Figure 4.12, Figure 4.13 and Figure

4.14. Analyses results for all three models consistently show that, while being much simpler to implement as compared to micro modeling methods, macro models are able to provide reliable results for the response behavior of diagonally reinforced coupling beams with different aspect ratios. With strut-and-tie method on coupling beam modeling (Barbachyn et al. 2012), there is a limitation on controlling the parameters; for instance, an engineer is not allowed to directly introduce the backbone, but can only define uniaxial material properties. Moreover, the strut-and-tie model does not account for the slip/extension behavior. The cumulative dissipated energy of the model developed by Barbachyn et al. (2012) is not shown in Figure 4.12 and Figure 4.13 since their results extremely exceed the plot range.

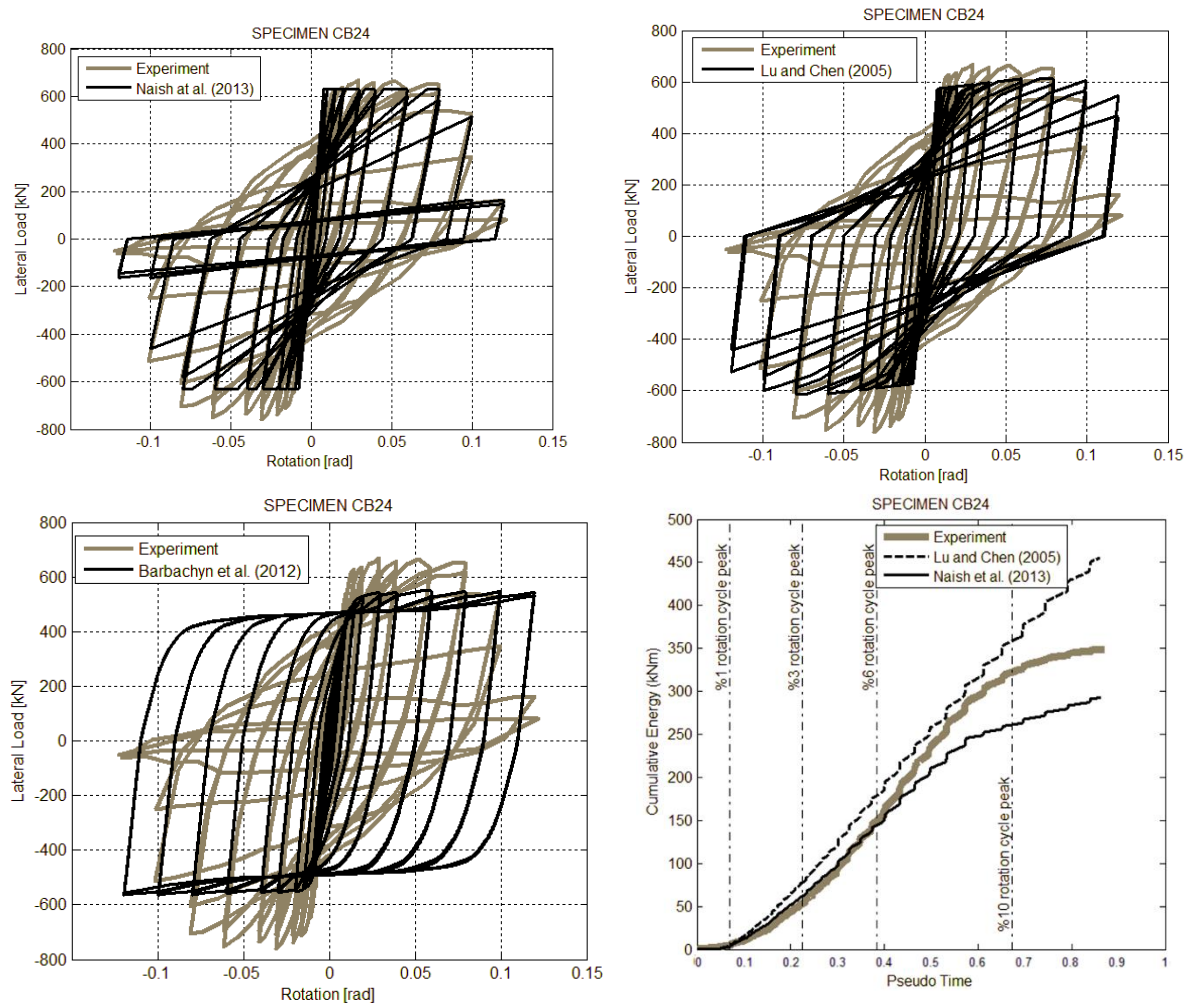


Figure 4.12 : CB24 cyclic analysis results (a) Naish et al. (2013), (b) Lu and Chen (2005), (c) Barbachyn et al. (2012); and (d) cumulative dissipated energy.

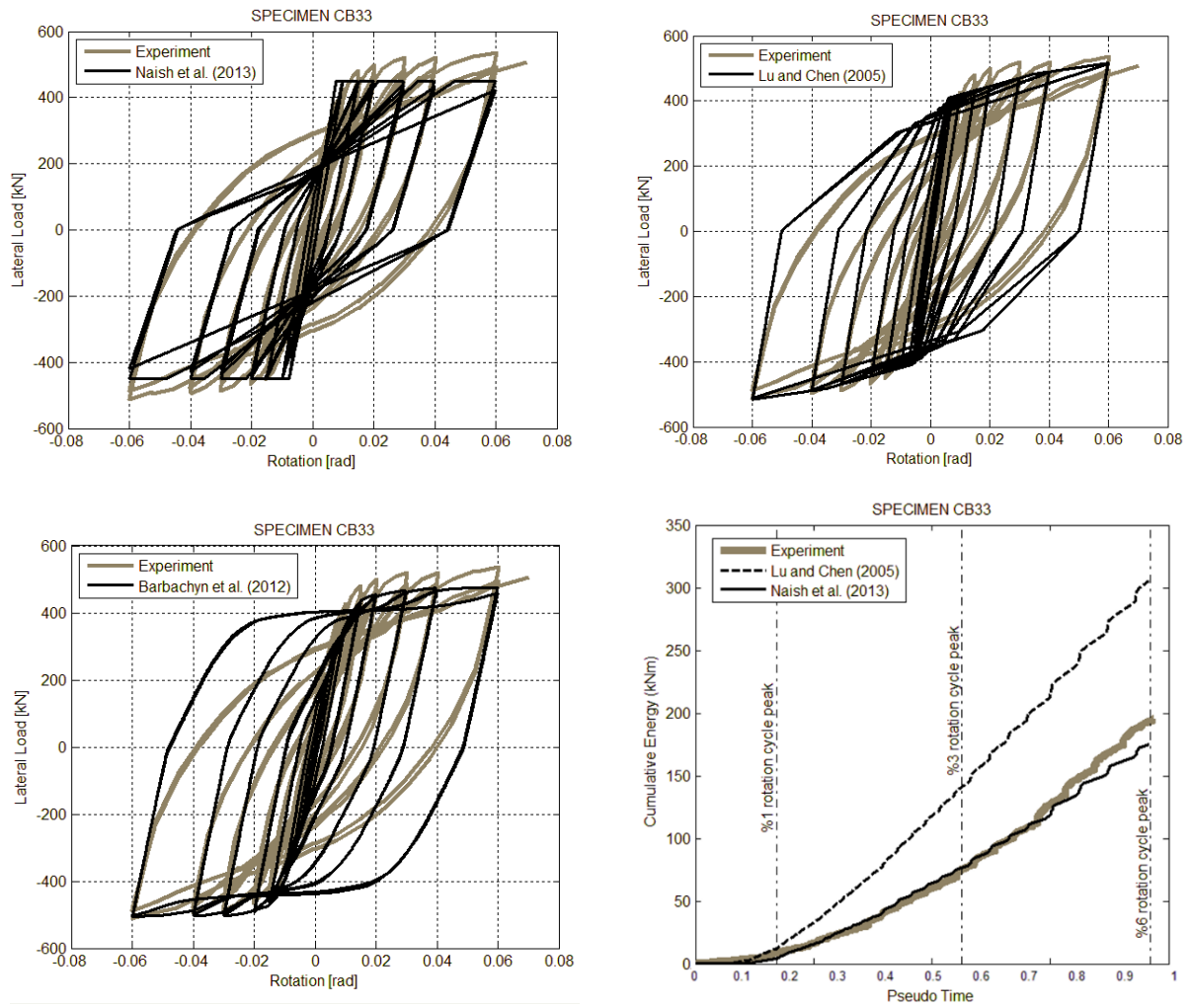


Figure 4.13 : CB33 cyclic analysis results (a) Naish et al. (2013), (b) Lu and Chen (2005), (c) Barbachyn et al. (2012); and (d) cumulative dissipated energy.

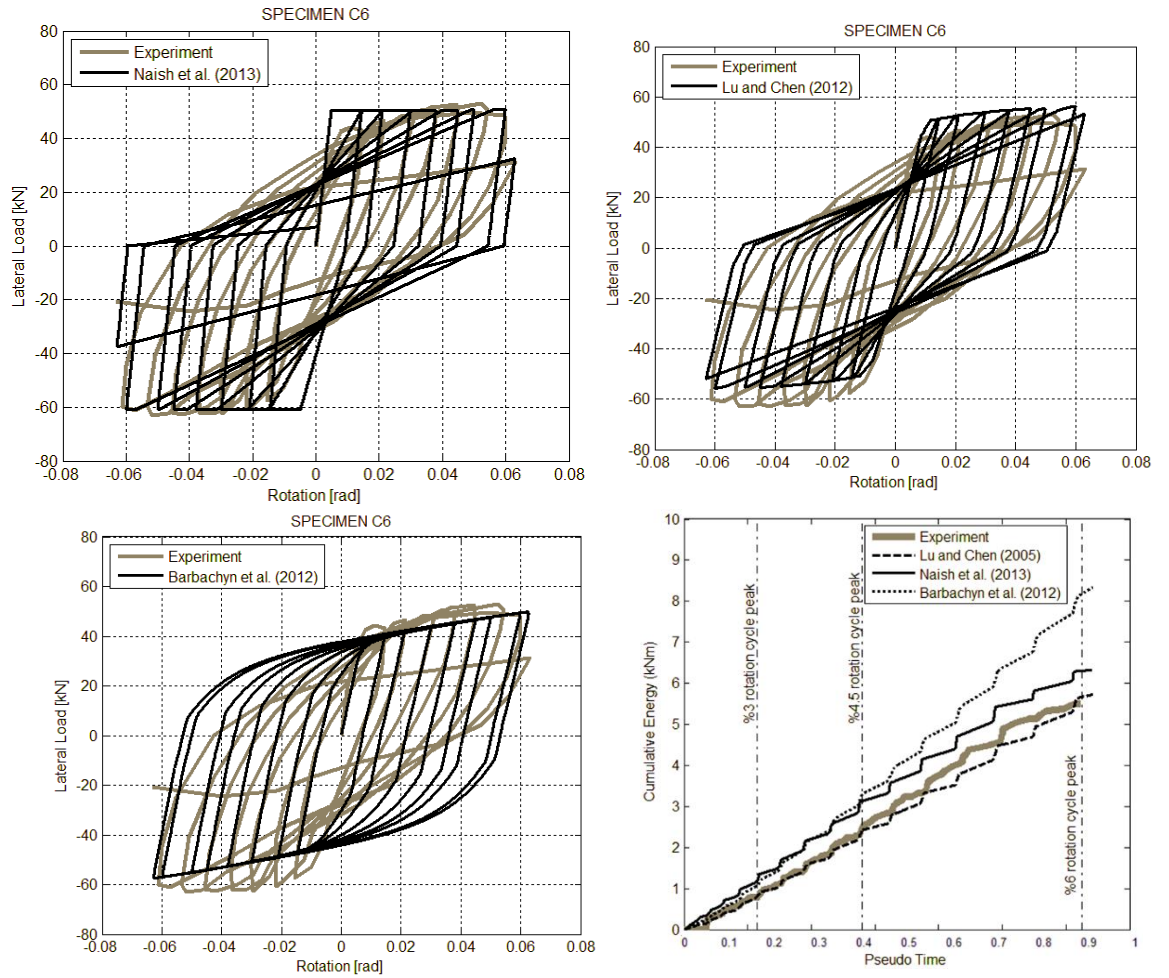


Figure 4.14 : C6 cyclic analysis results (a) Naish et al. (2013), (b) Lu and Chen (2005), (c) Barbachyn et al. (2012); and (d) cumulative dissipated energy.

The amount of dissipated energy in hysteretic loop of beam element models proposed by Lu and Chen (2005) and Naish et al. (2013) is very similar for rotations up to 6%, where collapse prevention performance is defined as in ASCE-41 (2007). For higher rotation values, Naish et al. (2013) managed to incorporate strength degradation resulting in a more accurate estimate for the actual cumulative energy for all three with different aspect ratios.

To improve the precision of these macro models, a further investigation and some modification proposals regarding ultimate shear on backbone curve have been carried out and presented in the following sections.

4.4 Proposal to Improve the Coupling Beam Modeling

The previously examined models have been scrutinized, and an improvement in Naish model is proposed. The improvement of the coupling beam modeling has been performed about the ultimate capacity and the initial stiffness of diagonally reinforced coupling beams and two modifications have been proposed. Both proposals affect the development of the back-bone curve to be used in the nonlinear analyses.

4.4.1 Modified ultimate shear capacity

The beam shear strength V_n is determined by using ACI 318 (2011) code as given in the following equation (4.5), which is based on previous studies by Paulay and Binney (1974).

$$V_n = 2A_{vd} f_y \sin \alpha \leq 0.83A_{cw} \sqrt{f'_c} [MPa] \quad (4.5)$$

Where A_{vd} and A_{cw} represent the diagonal reinforcement bundle area and coupling beam gross cross-sectional area, respectively. α represents the angle between the diagonal reinforcement and the beam horizontal line. f_y and f'_c represent steel yield stress and concrete compressive strength, respectively.

Paulay and Binney (1974) predicted the theoretical strength of diagonally reinforced coupling beams using a simple model which assumes all load to be resisted by diagonal compression and tension carried by the diagonal reinforcement. Obviously, this assumption is quite conservative when the concrete core is surrounded by diagonal bars in each direction which are well confined by ties as seismic codes recommend. Moreover, Hindi and Hassan (2007) point that the underestimation of the strength of coupling beams may possibly lead to designs where plastic hinges occur in the shear walls earlier than the coupling beams. Therefore, including the concrete contribution, they expressed shear strength as a combination of two components as shown by equation (4.6, 4.7 and 4.8).

$$V_n = T_y \sin \alpha + C_y \sin \alpha \quad (4.6)$$

$$T_y = A_{vd} f_y \quad (4.7)$$

$$C_y = A_{vd} f_y + A_c f'_{cc} \quad (4.8)$$

Here, T_y and C_y represent diagonal tension and compression axial force capacities of confined diagonal bundles, respectively, while f'_{cc} is confined concrete compression strength. Depending on the material and the geometric properties of the confined diagonal bundle, the compression component can significantly increase the strength of the coupling beam.

Given the fact that Hindi and Hassan (2007) tested the validity of equation (4.6) only under monotonic loading, the strength of coupling beam models CB24 and CB33 were also re-evaluated under cyclic loadings by altering their ultimate shear capacity in accordance with the equation (4.6), instead of using the equation (4.5), in the context of this study. The modified shear capacity and the cumulative energy curves for the beam models CB24 and CB33 are illustrated in 0 and Figure 4.16 comparing the two capacity interpretations.

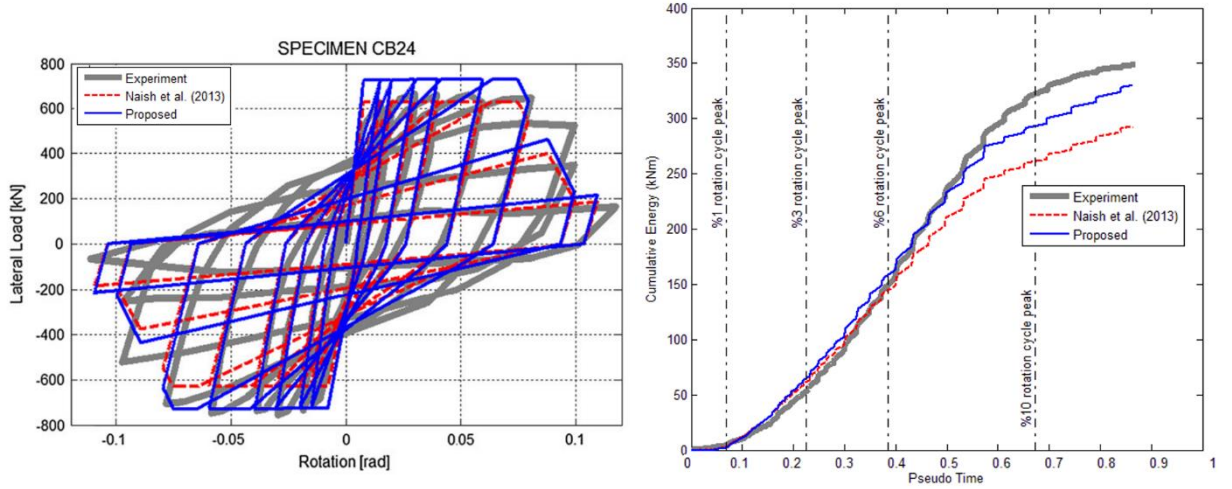


Figure 4.15 : CB24 results (a) lateral load vs. rotation; and (b) cumulative energy.

As it is seen, although both models have backbone curves with the same rotation limits based on the approach by Naish et al. (2013), their ultimate shear capacities are expected to be different since the proposed model has larger shear capacity due to the inclusion of the concrete contribution in the diagonal core.

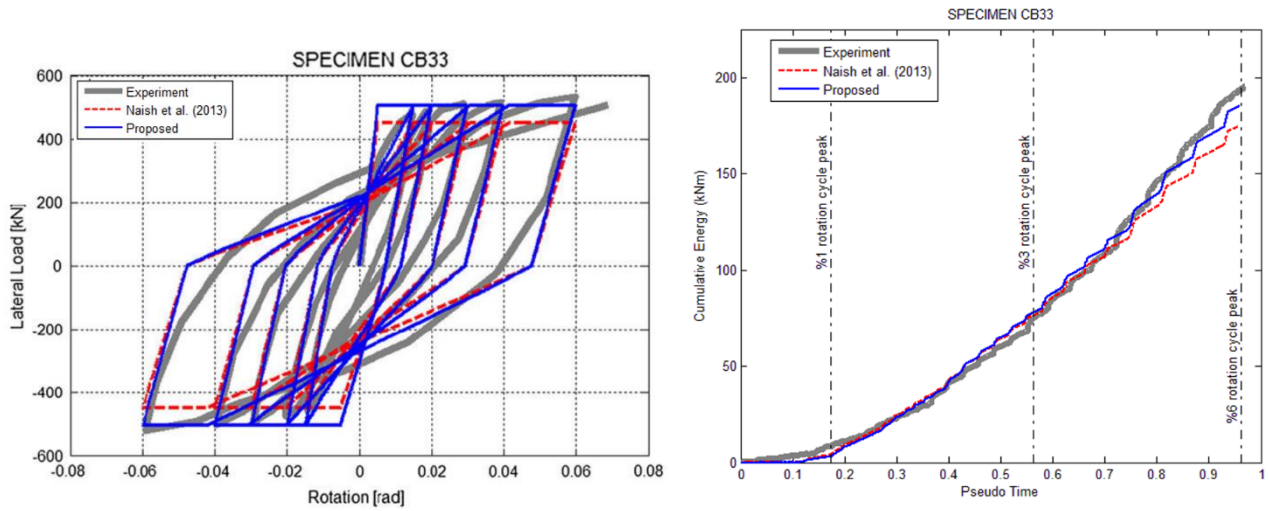


Figure 4.16 : CB33 results (a) lateral load vs. rotation; and (b) cumulative energy.

It can be clearly seen that the modified model captures the experimental cyclic response better in terms of shear capacity also in the region where strength degradation starts. The cumulative energy curves agree well for rotations up to 5-6%. On the other hand, the difference in energy becomes more significant at higher rotations although the modified model matches better to the test results. It can be claimed that with the modified model, the possible conservative beam design issue, which may lead to the occurrence of plastic region in the shear walls before hinges occur in the coupling beams, has been overcome.

4.4.2 Contribution of slip and extension to the effective stiffness

As it has already been described in the previous sections, the nonlinear model proposed by Naish et al. (2013) consists of three type of components: linear line element, plastic hinges for flexural behavior, rotation hinges for slip/extension behavior. Obviously, calculating all necessary parameters for each component types consumes time in the engineering practice.

A simpler model for diagonally reinforced coupling beams is proposed herein by implementing the linear part of slip/extension spring behavior into elastic part of the beam. As already mentioned, the use of $EI_{eff} = 0.5EI_{gross}$ along with the slip/extension spring behavior was found to result in an effective flexural stiffness very close to $0.15EI_{gross}$ (Wallace, 2010). A parametric study has been carried out in order to investigate the change of effective stiffness, when slip/extension behavior is included, with respect to various aspect ratios and reinforcement ratios. The coupling beams with

aspect ratio values of 1.5, 2.0, 3.0 and 3.5 with varied diagonal reinforcement ratio values have been examined for the parametric study, the properties of which are summarized in Table 4.3.

The sectional analyses using Xtract (2007) were executed for determining the change of elastic beam effective stiffness coefficient through bilinearization of the moment-curvature curve based on equating the areas under the bilinear approximation and the moment-curvature curve. Because the diagonal bundles are not perpendicular to the section plane, only horizontal component of the diagonal angle α is accounted for the sectional analysis to compute the flexural capacity as expressed in (4.9). Here, A'_s is the equivalent area of the reinforcement bundle to be considered in the section analysis, and A_{vd} is the actual total area of the diagonal reinforcement. The bilinearization of the moment-curvature diagram for the beam 1501 is illustrated in Figure 4.17.

$$A'_s = A_{vd} \cos \alpha \quad (4.9)$$

Table 4.3 : Properties of diagonally reinforced coupling beam specimens.

Beam	width [mm]	depth [mm]	length [mm]	aspect ratio	EI_{gross} [kNm ²]	diagonal reinforcement	$V_n[kN]$	$V_n/(Ac\sqrt{f'c'}[MPa])$
B1501	600	667	1000	1.50	504460	8Φ18	948	0.37
B1502	600	667	1000	1.50	504460	8Φ20	1171	0.46
B1503	600	667	1000	1.50	504460	8Φ22	1417	0.56
B1504	600	667	1000	1.50	504460	8Φ24	1686	0.67
B1505	600	667	1000	1.50	504460	8Φ26	1979	0.78
B2001	600	500	1000	2.00	212500	10Φ16	755	0.40
B2002	600	500	1000	2.00	212500	10Φ18	955	0.50
B2003	600	500	1000	2.00	212500	10Φ20	1180	0.62
B2004	600	500	1000	2.00	212500	10Φ22	1427	0.75
B3001	600	333	1000	3.00	62774	8Φ16	427	0.34
B3002	600	333	1000	3.00	62774	8Φ18	540	0.43
B3003	600	333	1000	3.00	62774	8Φ20	667	0.53
B3004	600	333	1000	3.00	62774	8Φ22	807	0.64
B3005	600	333	1000	3.00	62774	8Φ24	961	0.76
B3501	600	285	1000	3.50	39354	10Φ14	355	0.33
B3502	600	285	1000	3.50	39354	10Φ16	464	0.43
B3503	600	285	1000	3.50	39354	10Φ18	587	0.54
B3504	600	285	1000	3.50	39354	10Φ20	725	0.67
B3505	600	285	1000	3.50	39354	10Φ22	877	0.81

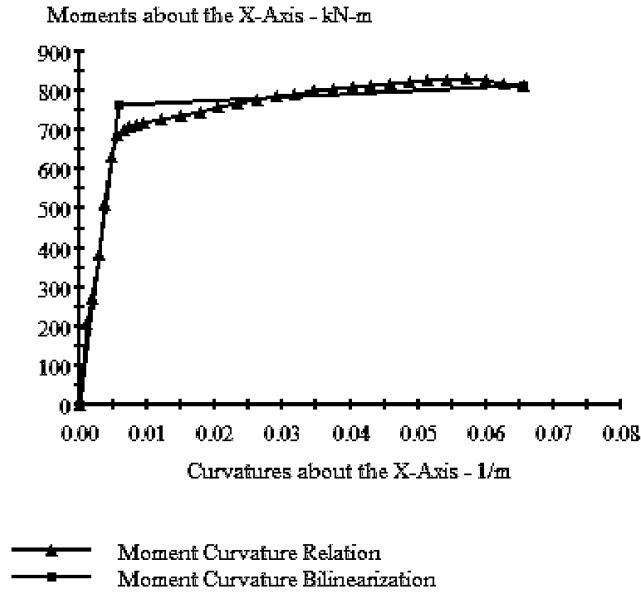


Figure 4.17 : Bilinearization on moment-curvature of B1501 (Xtract, 2007).

The results in Table 4.4 show that the amount of crack opening caused by slip and extension behavior significantly increases as the aspect ratio reduces. It is found that the resultant effective stiffness, obtained by adding rotations caused by flexure, slip and extension, ranges between 8% and 11% of the uncracked cross section stiffness:

Table 4.4 : Effective stiffness results of coupling beam specimens.

Beam	M_y [kNm]	$\phi_{y,flex}$	$\Theta_{y,flex.}$	$\Theta_{y,slip/ext.}$	$\Theta_{y,total}$	EI_{eff} [kNm ²]	EI_{eff}/EI_{gross}
B1501	766	0.0057	0.03%	0.29%	0.32%	39982	0.079
B1502	861	0.0057	0.03%	0.33%	0.35%	40486	0.080
B1503	963	0.0057	0.03%	0.36%	0.39%	41048	0.081
B1504	1075	0.0058	0.04%	0.39%	0.43%	41909	0.083
B1505	1197	0.0059	0.04%	0.42%	0.46%	43038	0.085
B2001	480	0.0087	0.04%	0.37%	0.41%	19736	0.093
B2002	550	0.0089	0.04%	0.41%	0.46%	20052	0.094
B2003	624	0.0090	0.05%	0.46%	0.51%	20484	0.096
B2004	705	0.0092	0.06%	0.51%	0.56%	20979	0.099
B3001	230	0.0146	0.06%	0.62%	0.68%	5624	0.090
B3002	265	0.0149	0.07%	0.70%	0.77%	5758	0.092
B3003	301	0.0151	0.08%	0.78%	0.85%	5868	0.093
B3004	339	0.0153	0.09%	0.85%	0.94%	6004	0.096
B3005	381	0.0156	0.10%	0.93%	1.03%	6160	0.098
B3501	183	0.0180	0.08%	0.68%	0.75%	4051	0.103
B3502	214	0.0184	0.09%	0.77%	0.86%	4143	0.105
B3503	246	0.0187	0.10%	0.87%	0.97%	4222	0.107
B3504	282	0.0192	0.12%	0.96%	1.08%	4331	0.110
B3505	319	0.0197	0.14%	1.06%	1.20%	4450	0.113

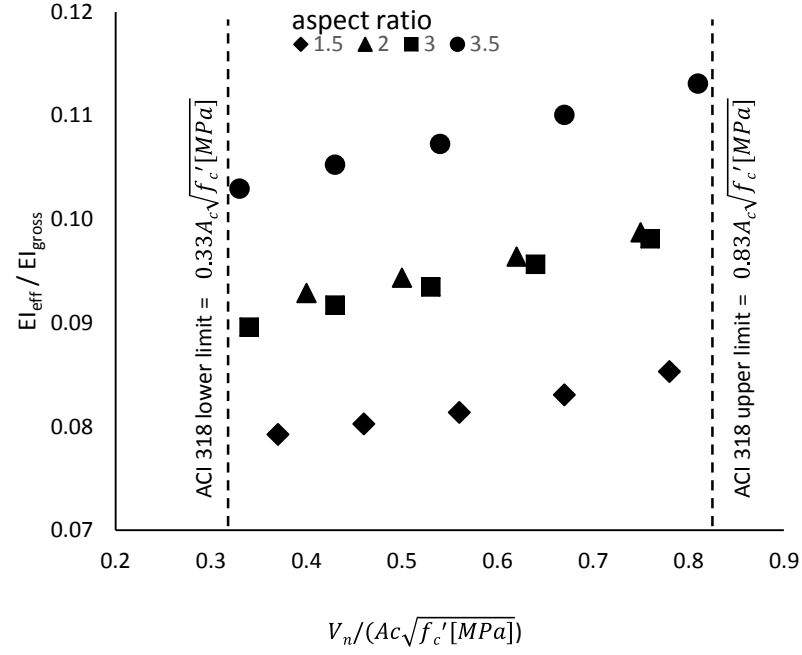


Figure 4.18 : Effective stiffness of diagonally reinforced coupling beams.

It is observed that the change of effective stiffness coefficient in coupling beams varied between 0.08 and 0.11 almost linearly for aspect ratios 1.5 and 3.5, respectively. Therefore, the following Equation (4.10) is proposed for estimating the effective stiffness of diagonally reinforced coupling beams with slip/extension behavior:

$$El_{eff}/El_{gross} = a + 2\left(\frac{V_n}{A_c \sqrt{f'_c} [MPa]} - 0.33\right) \quad (4.10)$$

The proposed values of a and in Eqn (4.10) is also tabulated in Table 4.5. Linear interpolation can be used to estimate the parameter a in between two boundary values.

Table 4.5 : Proposed coefficients for effective stiffness formula.

aspect ratio	a
$l/h = 1.50$	0.08
$2.00 \leq l/h \leq 3.00$	0.09
$l/h = 3.50$	0.10

4.4.3 Backbone curves for the improved model

After modifying the ultimate shear capacity and the initial stiffness determination procedure, the new form of the capacity backbone curve is developed. The backbone curve parameters which will be used later in the next chapters of this dissertation for numerical solutions are illustrated in Figure 4.19.

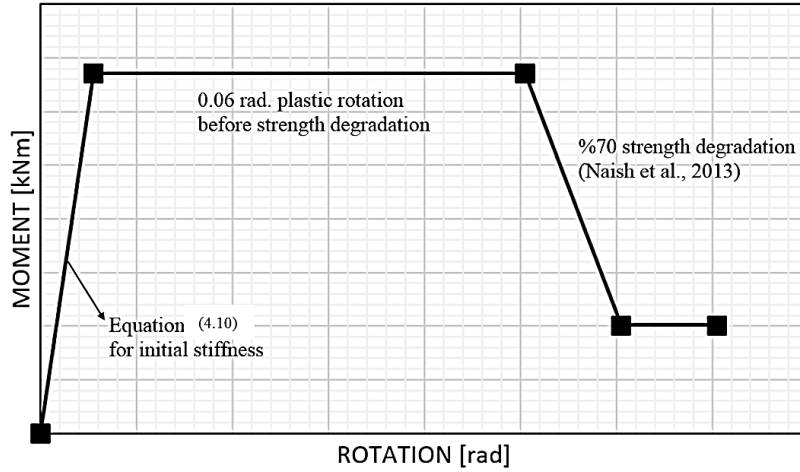


Figure 4.19 : Proposed model for diagonally reinforced coupling beams.

The proposed model backbone is then plotted on the cyclic load-deformation curves for CB24 and C6 as shown in Figure 4.20 and Figure 4.21, respectively in order to compare this new proposal with the previously performed test result. In these plots, the simplified bilinear curve (dashed line) represent the beam capacity curve which is determined with Turkish Earthquake Code (TEC, 2007) procedure. The code effective stiffness coefficient is 0.4 and it does not consider the concrete compression capacity in the diagonal bundles for coupling beam strength estimation.

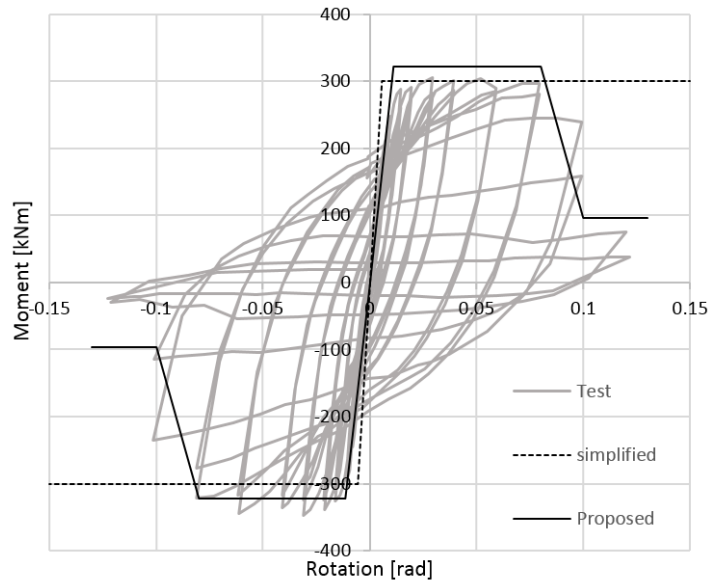


Figure 4.20 : Proposed model backbone curve vs. test results CB24.

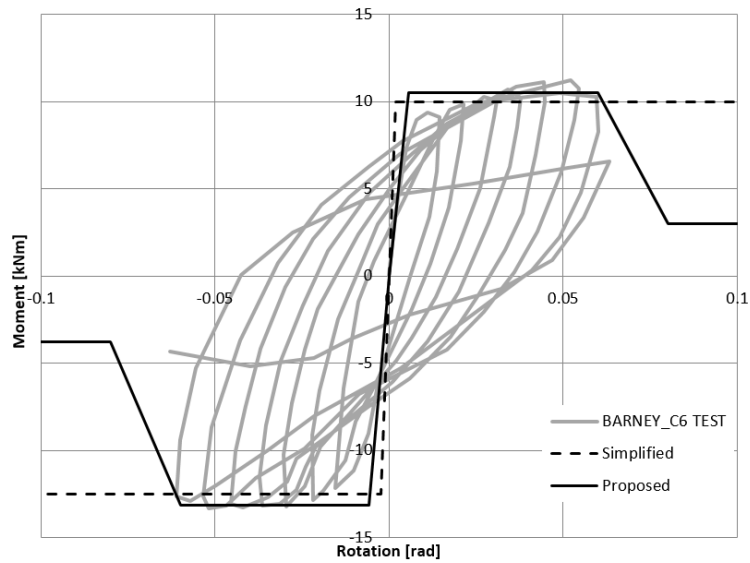


Figure 4.21 : Proposed model backbone curve vs. test results C6.

In both cases, the proposed model represented the actual behavior better than the code-based curve (simplified). The proposed model also considers the strength degradation, while the simplified bilinear does not.

4.5 Summary and Remarks

Three different macro-modeling alternatives introduced by Naish et al (2013), Lu and Chen (2005) and Barbachyn (2012) has been reviewed and modeled for verifying the results of the analyses with that of the experiments.

Numerical investigations show that Lu and Chen (2005) and Naish et al. (2013) proposals were almost identical up to the rotation of 6%, where collapse prevention performance is defined in ASCE-41 (2006). For higher rotation values, Naish et al. (2013) was also able to exhibit strength degradation; thus, presenting a more accurate approach to actual cumulative energy for all three different aspect ratios.

Although there is a limitation on controlling parameters for the strut-and-tie method (Barbachyn et al. 2012), that model can also provide satisfactory estimates for the ultimate shear capacity, however, it lacks somewhat as it does not account for the slip/extension behavior.

After reviewing the existing modeling alternatives in literature, a proposal for the effective stiffness of the coupling beams is introduced to improve their simplicity for engineering practice and also capture a better overall behavior.

When compared with the results of other analyses in terms of lateral load vs. rotation; and cumulative energy curves, it is observed that the proposed model which considers the concrete compression capacity in the beam strength formula captures the experimental cyclic response better in terms of shear capacity also at strength degradation region thus resulting in more reasonable designs.

Analyzing 19 different coupling beams with different aspect ratios, it is observed that the alteration of effective stiffness coefficient in coupling beams varies between 0.08 and 0.11 changing almost linearly as a function of the aspect ratio and the beam strength. Based on this fact, a new formulation is proposed for modeling the effective stiffness of a diagonally reinforced coupling beam with slip/extension behavior.

After modifying the ultimate shear capacity and the initial stiffness determination procedure, the proposed model backbone is then plotted on the cyclic load-deformation curves for CB24 and C6 in order to compare this new proposal with the previously performed test results. It is observed that the proposed model captures the initial stiffness and the strength degradation reasonably.

5. ANALYSIS OF COUPLED WALL SYSTEMS

This section gives an overview of the analysis methods for structural systems in general. The analysis types mainly fall into two categories: linear analysis and nonlinear analysis.

Like other wall structures, the coupled wall systems are usually designed with code-based linear approaches in the engineering practice. On the other hand, the nonlinear methods give the design engineer an opportunity to understand the plastic sequence of the coupled wall system components. This chapter reviews the nonlinear consequences of coupled wall systems and emphasizes the necessity of using the nonlinear approaches.

5.1 Linear Analysis

The concept employed in equivalent static lateral force procedures is to place equivalent static loads on a structure with magnitudes and direction that closely approximate the effects of dynamic loading caused by earthquakes. Concentrated lateral forces due to dynamic loading assumed to act on floor and roof levels of the buildings, where masses of each story assumed to be concentrated. Furthermore, equivalent lateral forces tend to be larger at higher elevations in a structure. Thus, the greatest lateral displacements occur at the top level of a structure. These effects are modeled in equivalent static lateral force procedures of many codes by placing a force at each story level in a structure.

Response-spectrum analysis (RSA) is a linear-dynamic analysis method which measures the contribution from each natural mode of vibration to indicate the likely maximum seismic response of an essentially elastic structure. Response-spectrum analysis provides insight into dynamic behavior by measuring pseudo-spectral acceleration as a function of structural period for a given time history and level of damping.

This procedure is one of the useful tools of earthquake engineering for analyzing the performance of structures under earthquake loading, since many systems behave as single degree of freedom systems. Thus, if you can find out the natural frequency of the structure, then the peak response of the building can be estimated by reading the value from the ground response spectrum for the appropriate frequency. In most building codes in seismic regions, this value forms the basis for calculating the forces that a structure must be designed to resist (seismic analysis). A response spectrum is a plot of the maximum response amplitude (displacement, velocity or acceleration) versus time period of many linear single degree of freedom oscillators to a give component of ground motion. The resulting plot can be used to select the response of any linear SDOF oscillator, given its natural frequency of oscillation. One such use is in assessing the peak response of buildings to earthquakes. The design spectral acceleration parameters S_{MS} and S_{M1} defined by Turkish Seismic Code for Coastal and Harbor Constructions, Railways and Airfields (2008) are calculated by using the following equations.

$$S_{MS} = F_a \times S_S \quad (5.1)$$

$$S_{M1} = F_v \times S_1 \quad (5.2)$$

Here, F_a and F_v are the site coefficients (1.0 for soil class B condition). The short S_S and 1-second period spectral acceleration S_1 parameters are given as tabulated hazard coordinates in the code annex. The spectral response spectrum is plotted based on the back-bone given in Figure 5.1, where S_{ae} is the design spectral acceleration.

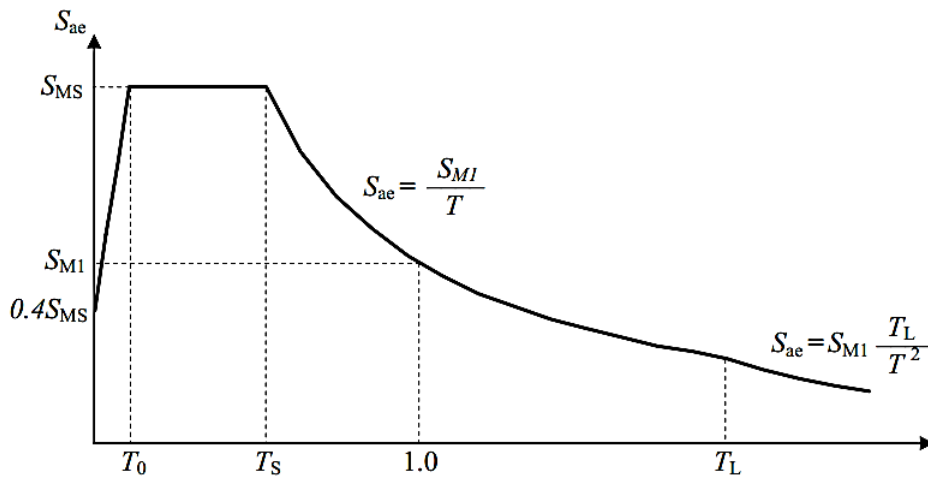


Figure 5.1 : Spectral acceleration vs. period.

5.2 Nonlinear Analysis

Although structures are usually designed for earthquake resistance using elastic analysis procedures in practice, the most of them will exhibit significant plastic deformations under large seismic actions. Enabled by advancements in computing technologies and available test data, nonlinear analyses provide means for calculating structural response beyond the elastic range, including strength and stiffness reduction. As such, nonlinear analysis can play an important role in the evaluation of structures (NIST, 2010) to understand their real behavior.

The first important guideline on the application of nonlinear analysis were those published in FEMA 273 NEHRP Guidelines for the Seismic Rehabilitation of Buildings (FEMA 1997) and ATC 40 Seismic Evaluation of Retrofit of Concrete Buildings (ATC 1996). Later on, the nonlinear analysis concepts were developed and also have been introduced with other guidelines and supplements [(FEMA 440, 2005), (ASCE 41, 2007), (FEMA P440a, 2009), (ATC 58, 2009), (PEER/ATC 72-1, 2010)]. The most recent developments for assessing and retrofitting of existing buildings are introduced with revised ASCE 41 (2013), which also defines diagonally reinforced coupling beam plastic rotation limits with their updated values.

5.2.1 Pushover analysis

When a structure deforms while its elements are yielding progressively, the force-deformation relationship cannot be determined by simple elastic analysis. In such a case, the need for other methods of analysis becomes apparent. One relatively simple method of nonlinear procedure is the nonlinear static analysis, also known as the pushover analysis. This method is intended to provide a simple way for determining a backbone nonlinear response for a structure subjected to incrementally increasing lateral forces and/or displacements. For such an analysis, the structural system is pushed through the initial elastic response and then through sequential yielding of elements to the eventual formation of a failure mechanism until its collapse.

The nonlinear static procedure was documented as an acceptable method of analysis in FEMA 356 (2000). The so-called pushover analysis has become popular because it avoids the complexity of a response history analysis, but still includes significant aspects of degradation that are critical to seismic behavior (El-Tawil et al. 2009). However, the conventional pushover procedure presented in FEMA 356 (2000) does

not directly account for the presence of higher modes, which may be particularly critical in taller buildings, and therefore the procedure should be limited to low and mid-rise buildings whose behavior is dominated by the first mode response.

5.2.2 Nonlinear time history analysis

Previous studies conducted by Cornell and Jalayer (2002) classify nonlinear dynamic analysis into four basic types:

1. The first method is called as “single stripe analysis” where a set of dynamic analyses are carried out with the proper accelerograms scaled to the target level.
2. The second method is the “cloud approach” where the records are not scaled to the same acceleration level. The analyses are instead done by a suite of accelerograms, exactly as recorded, from a common scenario, e.g. very similar magnitudes and distances.
3. The third method is referred as “multi-stripe” approach where records are run at a suite of spectral acceleration stripes.
4. Incremental dynamic analysis (IDA), proposed by Vamvatsikos and Cornell (2002), is a method where a single record is run with varying scaling coefficients.

The above given second approach has been used to validate the pushover analysis results of the case study buildings as given in the next chapter. The main idea is to create a cloud of scattered points resulting from the incremental dynamical analyses over the conventional pushover representation, i.e. base shear – top displacement plot.

Previous studies revealed that, when a structure (i.e. coupled wall system) is subjected to as many as possible number of records with varying PGA values which would also reveal the uncertainty associated with the earthquake shaking. This uncertainty that may be caused due to the disagreement between nonlinear static and dynamic procedures can be overcome by using the cloud analysis (Bal 2008).

5.3 Coupled Wall and Coupling Beam Capacity-deformation Relations

In coupled wall systems, the frame action, that is: the axial forces in the walls resulting from the accumulated shear in coupling beams, is stiffer than the flexural response of the individual wall piers. As a result, the coupling beams exhibit greater ductility demands than the wall piers do. Harries (2001) represents the idealized response of a

coupled wall structure as shown in Figure 5.2 as the sum of the individual pier flexural responses and the frame response resulting from the coupling action of the beams. In the figure, μ_c and μ_w represents the ductility levels of coupling moments and walls, respectively.

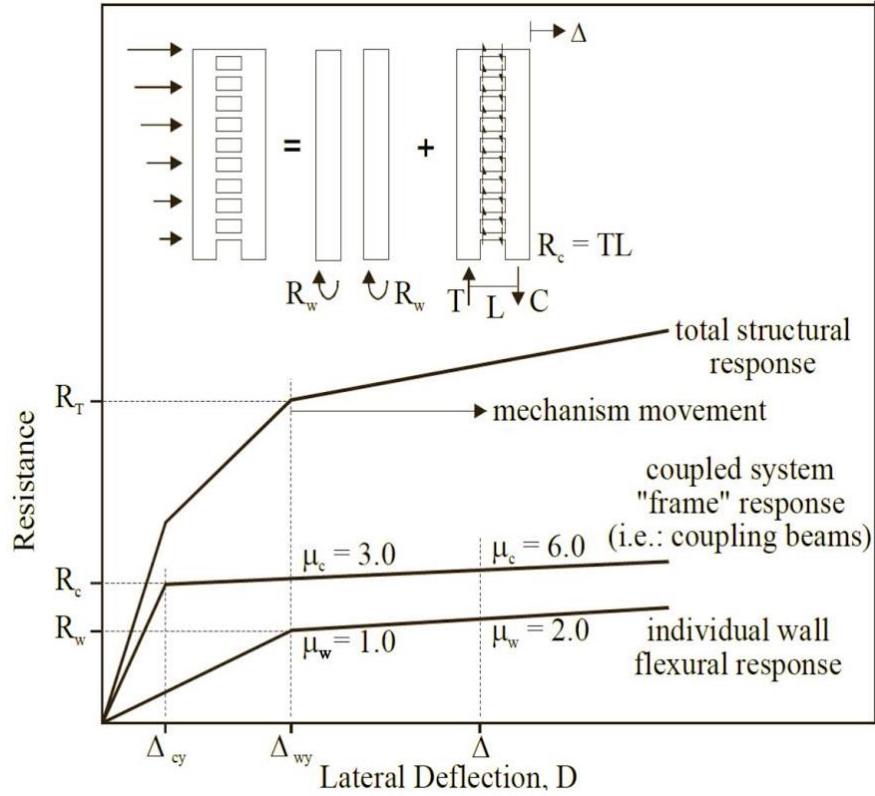


Figure 5.2 : Idealized lateral response of coupled wall (Harries 2001).

To allow the structure to achieve its full plastic capacity at its maximum potential strength, $R_T = \sum R_w + R_c$, the wall piers must form hinges at their bases. At this stage, the coupling beams must already have undergone significant inelastic deformations. As the structure continues to behave in a ductile manner, the ductility demand on the coupling beams continues to exceed that of the walls. If the beams are unable to achieve the ductility ratios demanded from them, potential strength of the entire structure, R_T , is not realized and is limited by the sum of the wall capacities, $\sum R_w$.

On the other hand, tension wall and compression wall will exhibit different behavior due to their axial load – bending moment interaction capacities. Moreover, the tension wall will experience its initial crack at early stages, consequently reducing its stiffness relative to the compression wall. Figure 5.3 (a) represents a typical coupled wall

system behavior that achieves ductility conditions at desired levels. At stage 1, tension wall's stiffness is reduced due to initial cracks. After point 1, coupling ratio is expected to be increased temporarily because the beams will produce coupling moment at a faster rate compared to the tension wall until coupling beam yield point, which is 2. In an efficient coupled wall system design, coupling beams yield prior to wall pier yield, which is illustrated with point 3. The coupling ratio supposed to drop down until wall piers yield. The flat region in coupling ratio (CR) is expected after wall yield (point 3) where the system will not be able to drag additional force demands but deformations until the point 4. Following the coupling beam degradation (point 4), CR will significantly decrease until all the beams' strengths degrade.

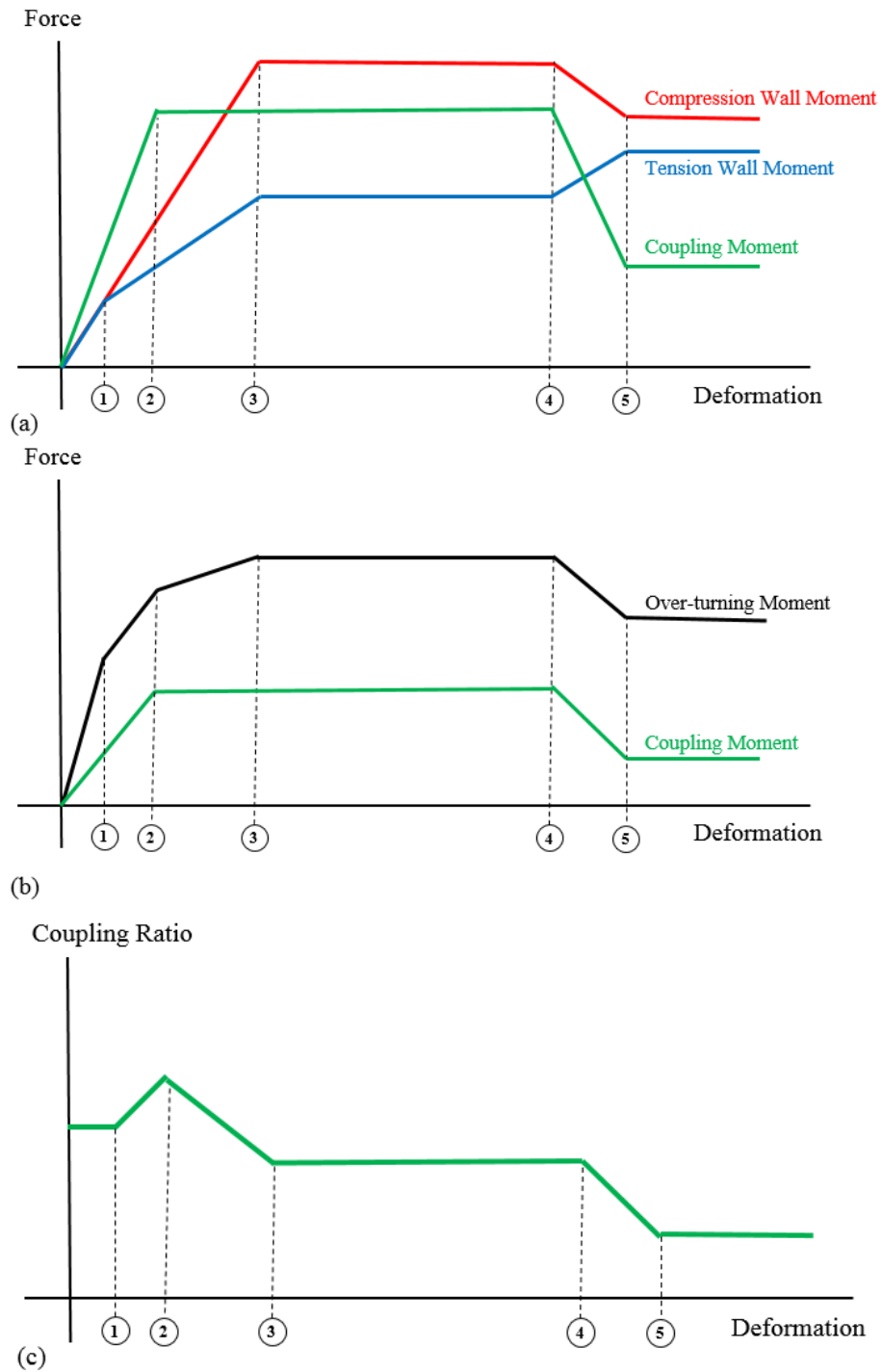


Figure 5.3 : (a) Representative coupled wall system behavior, (a) wall piers and coupling beam moments, (b) over-turning moment vs. coupling moment, (c) coupling ratio alteration plot.

5.4 Moment and Shear Distribution

When a coupled wall system consists of two identical walls piers, their shear and bending moment demands will be the same under the linear static analysis or linear dynamic analysis methods. In reality, this is valid until a wall pier reaches its bending

moment capacity as shown in Figure 5.4a. M_T and M_C are the bending moment demands in tension wall and compression wall, respectively. M_{Ty} represents the moment capacity of the wall in coupling tension and M_{Cy} is the moment capacity of the wall under coupling compression.

Different from the linear analysis procedures, static nonlinear analysis (pushover) and dynamic nonlinear (time history) analysis methods take into account the component capacities to exhibit post-yield behavior of the system. As external forces (overturning moment) increases, the wall under coupling tension will reach its bending capacity; on the other hand, the wall under coupling compression will still remain elastic. From this point, any exceeding bending moment demand will be resisted by the wall under coupling compression. As a result, the wall in coupling compression drags significantly larger bending moments and shear forces than the tension wall even if they are identical (Figure 5.4b).

This can also be observed overturning moment demand peaks in cyclic analysis or analysis with ground motions (time history); however, the roles of being compression or tension for each wall changes with each cycle.

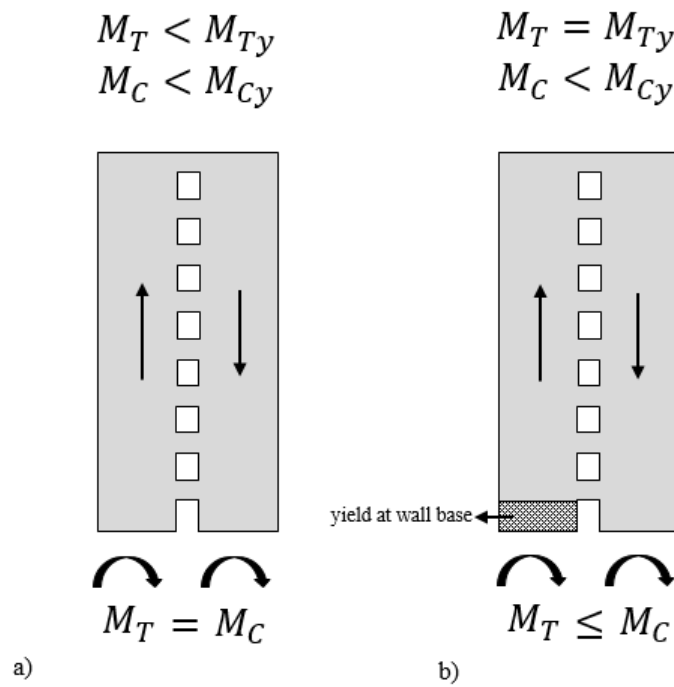


Figure 5.4 : Moment distribution a) elastic, b) significant yield at tension wall base.

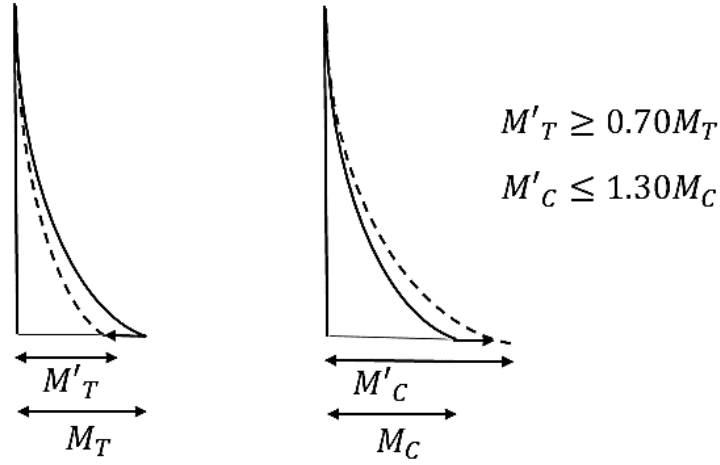


Figure 5.5 : Redistribution of wall moments.

5.5 Summary and Remarks

An overview of analysis methods for structural systems is presented in this chapter. In order to obtain an effective energy dissipation in coupled wall systems, it is very essential that the coupling beams yield before wall piers yield. Therefore, a nonlinear analysis method is strongly recommended for verification of the preliminary design, which is generally done with linear analysis.

The nonlinear behavior of a coupled wall system also exhibits the disparity of the wall pier flexure capacities, even if the wall piers are identical. At that level the redistribution of the moments start from the tension wall towards the compression wall. Considering the axial force and bending moment interaction, the tension wall experience flexural yield before the compression wall; therefore, a redistribution in force demands are observed between each wall piers. Parallel to the moment redistribution, some shear forces transfer to the compression wall as well which is called as “shear migration” phenomena.

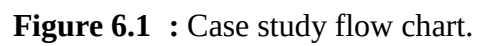
6. NUMERICAL INVESTIGATIONS

6.1 Methodology

In this chapter, behavior of nine coupled wall systems with various coupling ratio levels and story numbers are comprehensively investigated and their response under increasing drift values are observed by using non-linear analysis approaches. For each case, the disparity between moment and shear demand forces are noted. Additionally, the difference between initially determined elastic coupling ratio and plastic coupling ratio values are summarized.

Initially, the preliminary dimensions of totally nine coupled wall systems with 8, 16 and 24 stories have been developed to have low, normal and high level initial (elastic) coupling degrees. The design of structural members then has been carried out under design-level earthquake with return period of 475 years. Modal superposition method of analysis is used with acceleration response spectra for the linear analyses. Development of non-linear elements and components were performed following design of wall piers and coupling beams.

Later on, the nonlinear model of each structural system has been developed on OpenSees (2013) by using aforementioned macro modeling approaches. Non-linear static analyses (pushover) were also carried out only for 8-story low-rise models in order to observe the plastic behavior sequence of specific structural key elements. These pushover analyses also helped to understand the change in the wall pier force demands and coupling ratio alteration phenomena. The pushover analysis method could not be performed for 16 and 24 story systems, because the total mass participation ratio of higher modes exceeded 30%. Then, a series of nonlinear time history analyses (NLTHA) are performed. Consequently, the decrease of the coupling ratio levels are estimated and the proposed coupling ratio alteration (CRA) plots are developed for the investigated structures. Additional NLTHA are performed in order to assess the design of the investigated mathematical models. The analysis and design procedure applied in the thesis is summarized with the flowchart given in Figure 6.1.



6.2 Properties of Case Study Coupled Wall Systems

The case study structures are assumed as office buildings being located in a zone with high seismicity. Coupling ratio alteration (CRA) and its effect on the nonlinear behavior of the coupled wall systems will be examined under low, normally and high coupled wall systems with various numbers of stories. The desired target ranges for low, normal and high coupling degrees are defined as in Table 6.1.

Table 6.1 : Target coupling ratio ranges for the preliminary designs.

	Elastic CR Range
Low Coupled System (CR_L)	25%-35%
Normally Coupled System (CR_N)	45%-55%
High Coupled System (CR_H)	65%-75%

It is assumed that the two pairs of coupled walls on 4 sides of the structure resist all of the lateral loads applied in east-west direction of the structure. The typical floor plan of the structure is given in Figure 6.2.

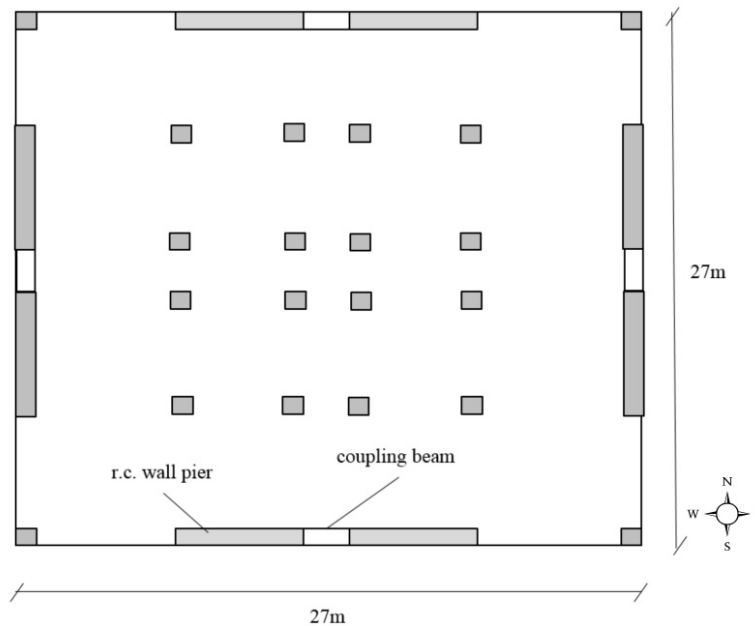


Figure 6.2 : Typical floor plan of the case study buildings.

The building plan area is 729 m² with outside dimensions of 27m by 27m. The story heights are 3.5m at each level, resulting in an overall building heights of 28m, 56m and 84m for 8, 16 and 24 story coupled wall systems, respectively as shown in Figure 6.3. A 20cm thick floor slab is assumed to be utilized for all floors.

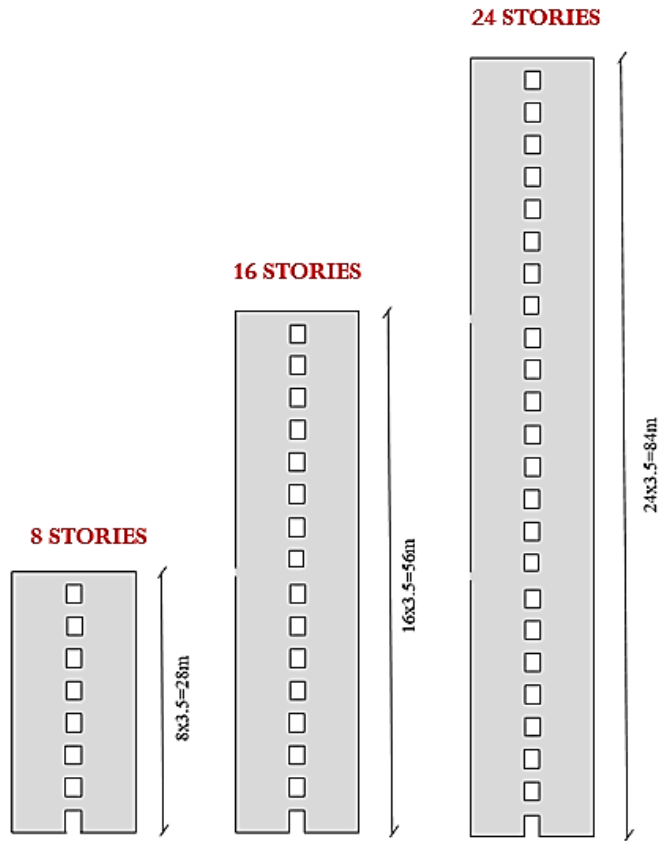


Figure 6.3 : Extracted representative frames of 8, 16 and 24 story coupled walls.

The characteristic compression strength of concrete is 40MPa and the characteristic yield strength of reinforcement steel is 420MPa.

For simplicity, following assumptions have been adopted in order to investigate the coupling ratio alteration:

- the distributed mass is constant along the height
- the stiffness of the walls is constant along the height
- the two coupled walls have the same dimensions and reinforcement
- the strength of coupling beams is constant along the height
- diagonal reinforcement layout design is utilized for the coupling beams
- the shear strength of wall piers is always higher than the flexural strength in order to avoid brittle failure

Table 6.2 : Geometrical properties of 8-story coupled wall systems.

	CW08-CR-L	CW08-CR-N	CW08-CR-H
Number of Stories	8	8	8
Wall Length [m]	6	6	6
Wall Thickness [m]	0.4	0.4	0.4
Beam Width [m]	0.4	0.4	0.4
Beam Depth [m]	0.4	0.6	1.0
Beam Length [m]	1.5	1.5	1.5
Beam Aspect Ratio	3.75	2.5	1.5
Coupling Ratio	0.31	0.50	0.67

Table 6.3 : Geometrical properties of 16-story coupled wall systems.

	CW16-CR-L	CW16-CR-N	CW16-CR-H
Number of Stories	16	16	16
Wall Length [m]	6	6	6
Wall Thickness [m]	0.6	0.6	0.6
Beam Width [m]	0.25	0.3	0.5
Beam Depth [m]	0.375	0.5	1.0
Beam Length [m]	1.5	1.5	1.5
Beam Aspect Ratio	4	3	1.5
Coupling Ratio	0.34	0.51	0.68

Table 6.4 : Geometrical properties of 24-story coupled wall systems.

	CW24-CR-L	CW24-CR-N	CW24-CR-H
Number of Stories	24	24	24
Wall Length [m]	7	7	7
Wall Thickness [m]	0.8	0.8	0.8
Beam Width [m]	0.25	0.4	0.4
Beam Depth [m]	0.5	0.65	1
Beam Length [m]	2	2	2
Beam Aspect Ratio	4	3.08	2
Coupling Ratio	0.35	0.55	0.69

6.3 Design of Coupled Wall Systems

6.3.1 Loads applied to the coupled wall systems

6.3.1.1 Gravity loads

The dead loads applied on the structure have been determined according to the provisions of TS498. A 20cm thick reinforced concrete slab having specific weight of 25kN/m³ was assumed, resulting 5kN/m² distributed load for all stories. In addition to self-weight of the concrete elements, a superimposed dead load equal to 1.5kN/m² was added to account for cladding, partitions, finishes and mechanical loads.

The live loads were determined according to TS 498 specification. The building is assumed to be used as office and therefore 2kN/m^2 for an office space loading has been used. 5kN/m^2 live load is assumed for corridor in the central area of the structure.

6.3.1.2 Seismic loads

The seismic loads for the building structure in this study have been determined according to the provisions of Chapters 11 and 12 of ASCE 7-10. The site class for the building location is assumed as Site Class B. Spectral response acceleration parameters for the short (0.2 sec) and 1-second periods are determined by using Earthquake Regulations for Coastal Structures, Railways and Airfields (2008). The coordinates of the case study buildings are given in the Figure 6.4. The short S_s and 1-second period spectral acceleration S_1 parameters are $1.05g$ and $0.48g$, respectively, where g is the acceleration of gravity.

Coordinates		For Short Period			For 1.0 s Period		
		Spectral Acceleration (g) – S_s			Spectral Acceleration (g) – S_1		
Latitude	Longitude	50%	10%	2%	50%	10%	2%
41	29	0.62	1.05	1.52	0.23	0.48	0.85



Figure 6.4 : Coordinates and seismic parameters of the case study buildings.

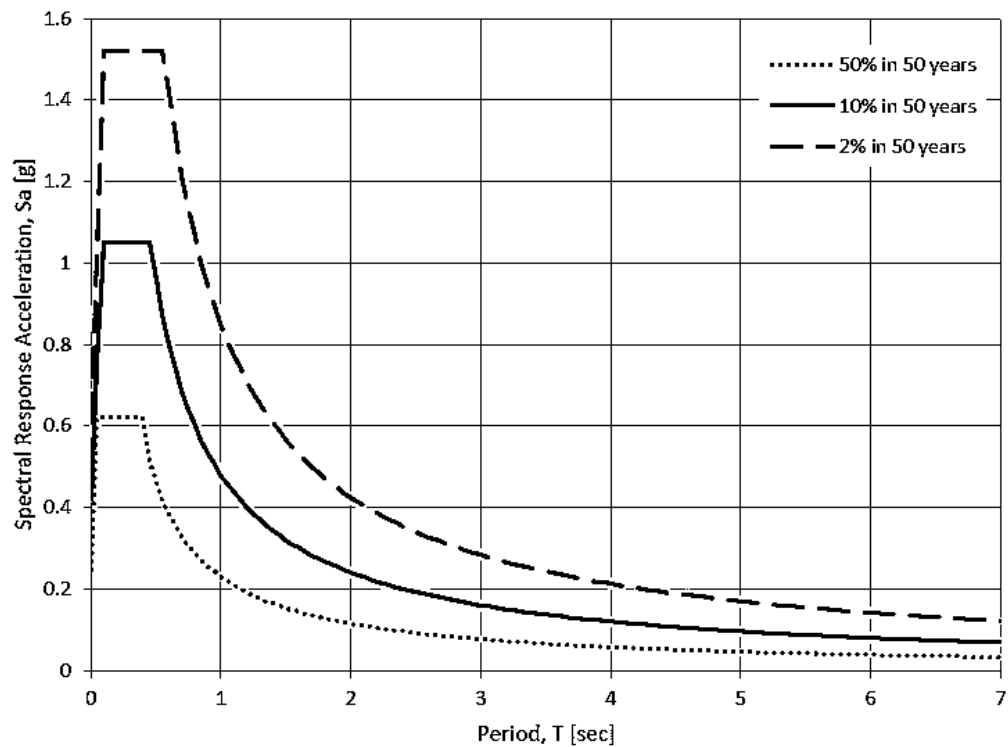


Figure 6.5 : Response spectra used for coupled wall systems.

6.3.2 Elastic analyses of wall systems

Design of the coupled wall system was performed with initial structural analysis using spectral acceleration procedure. The target spectrum of design earthquake level was selected with the probability of exceedance of 10% in 50 years to determine the code-prescribed base shear. The Spectra with three different levels of earthquakes are shown in Figure 6.5.

The response spectrum analysis (RSA) results are summarized as shown in Table 6.5, Table 6.6 and Table 6.7.

Table 6.5 : RSA results of 8-story coupled wall systems.

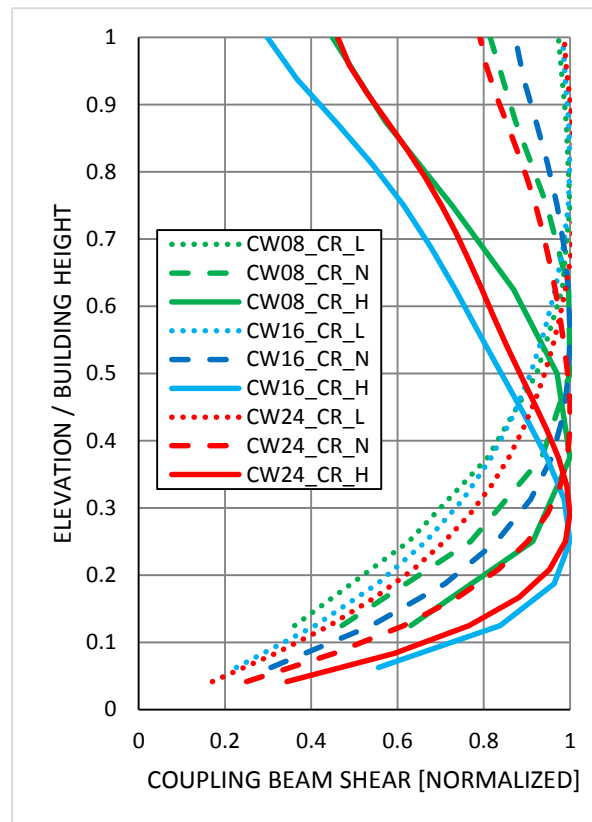
	CW08_CR_L	CW08_CR_N	CW08_CR_H
1st Period [secs]	0.787	0.62	0.467
2nd Period [secs]	0.163	0.148	0.123
Wall A Min Axial [kN]	3460	1750	1378
Wall B Max Axial [kN]	9540	11621	14750
Wall Shear [kN]	1210	1506	1982
Base Shear [kN]	2420	3012	3964
Wall Moment[kNm]	15923	14878	13097
Coupling Moment [kNm]	14615	30323	54108
System OTM [kNm]	46461	60079	80302
CR	0.31	0.50	0.67

Table 6.6 : RSA results of 16-story coupled wall systems.

	CW16_CR_L	CW16_CR_N	CW16_CR_H
1st Period [secs]	2.27	1.82	1.16
2nd Period [secs]	0.46	0.43	0.28
Wall A Min Axial [kN]	10692	8236	1237
Wall B Max Axial [kN]	16507	18963	25962
Wall Shear [kN]	1265	1356	1818
Base Shear [kN]	2530	2712	3636
Wall Moment[kNm]	21867	20297	14894
Coupling Moment [kNm]	22728	42528	62324
System OTM [kNm]	66462	83122	92112
CR	0.34	0.51	0.68

Table 6.7 : RSA results of 24-story coupled wall systems.

	CW24_CR_L	CW24_CR_N	CW24_CR_H
1st Period [secs]	2.7	2.04	1.59
2nd Period [secs]	0.6	0.49	0.39
Wall A Min Axial [kN]	15918	11566	6542
Wall B Max Axial [kN]	24881	29234	34257
Wall Shear [kN]	1594	1830	2174
Base Shear [kN]	3188	3660	4348
Wall Moment[kNm]	38689	35789	31166
Coupling Moment [kNm]	42255	86126	140571
System OTM [kNm]	119633	157704	202903
CR	0.35	0.55	0.69

**Figure 6.6** : The normalized shear distribution of coupling beams.

6.3.3 Design of coupling beams

The desired behavior for the coupling beams in a coupled wall system is to yield almost simultaneously at the code-prescribed base shear (El-Tawil et al. 2009). To achieve this objective, the coupling beams should optimally be designed to meet the shear demand distribution. On the other hand, except for high coupled systems, beam shear force distributions along elevation is almost constant after one third of the height, therefore beams are designed for the most critical shear force along elevation (Figure 6.6). Typically for a coupled wall, the most highly stressed coupling beams are located at approximately between one third and mid-height of the structure.

Diagonally reinforced coupling beams have been designed based on Turkish Earthquake Code (TEC, 2007). However, aspect ratio (span/depth) limitations were adopted from ACI 318 (2011), which allows diagonal reinforcements up to aspect ratio of 4.0.

The embedment length requirements of diagonal bars into the wall are compared both for Turkish Earthquake Code (TEC, 2007) and ACI 318 (2011) as in Table 6.8. In this thesis study, the embedment length is chosen as 45ϕ for all coupling.

Table 6.8 : Minimum embedment length requirements of diagonal reinforcements.

Diagonal Reinforcement Embedment Length	
ACI 318 (2011)	$1.25\phi f_y / 2.1\sqrt{f'_c} = 39.5\phi$
TEC (2007)	$1.5\phi(0.12 f_{yd} / f_{cta}) = 44.7\phi$

The summary of the coupling beam designs for different levels of coupling ratio are given in Table 6.9, Table 6.10 and Table 6.11.

Table 6.9 : Coupling beam design summary for 8-story systems.

	CW08-CR-L	CW08-CR-N	CW08-CR-H
Shear Demand [kN]	292	594	1177
$1.5(b_w)(d)f_{ctd}$ [kN]	352	528	880
Diagonal angle - α [deg]	9.09	16.36	29.26
$Asd=V/(2f_{yd} \sin(\alpha))$ [m ²]	0.002533	0.002889	0.003297
Diameter [mm]	25	25	28
Diagonal Amount	6	6	6

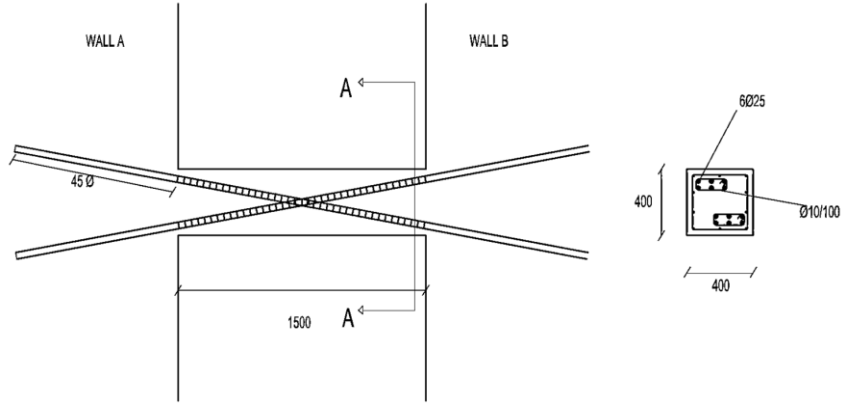


Figure 6.7 : Diagonal reinforcement layout for CW08-CR-L.

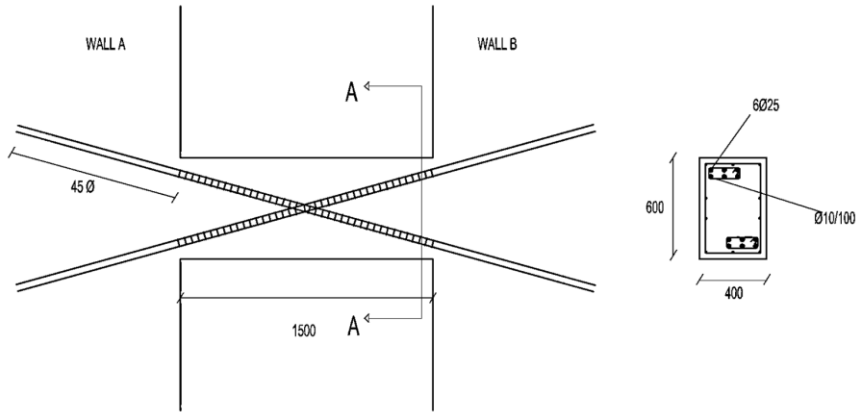


Figure 6.8 : Diagonal reinforcement layout for CW08-CR-N.

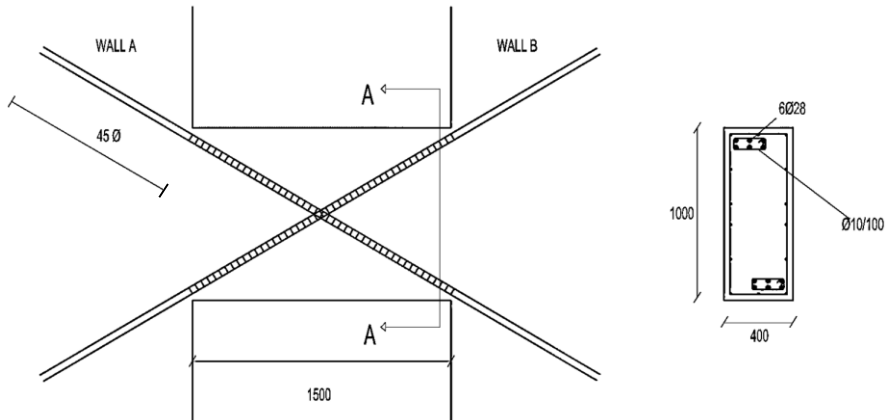


Figure 6.9 : Diagonal reinforcement layout for CW08-CR-H.

Table 6.10 : Coupling beam design summary for 16-story systems.

	CW16-CR-L	CW16-CR-N	CW16-CR-H
Shear Demand [kN]	192	409	1183
$1.5(b_w)(d)f_{ctd}$ [kN]	206.25	330	1100
Diagonal angle - α [deg]	8.16	12.78	29.26
$As_d = V / (2f_{yd} \sin(\alpha))$ [m ²]	0.002239	0.002541	0.003314
Diameter [mm]	25	25	25
Diagonal Amount	5	6	7

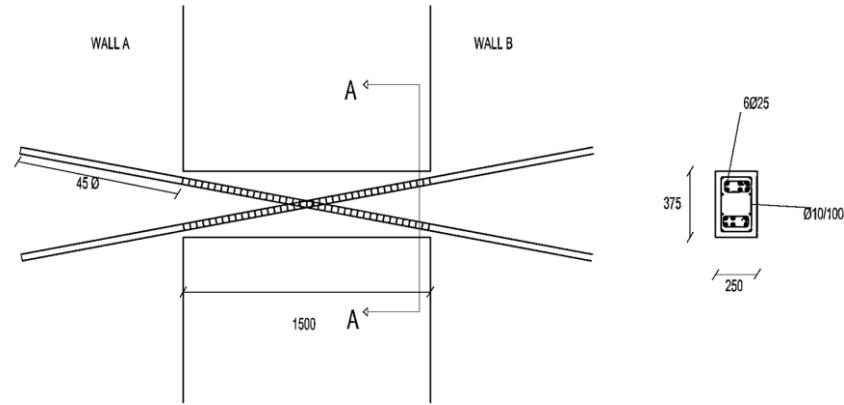


Figure 6.10 : Diagonal reinforcement layout for CW16-CR-L.

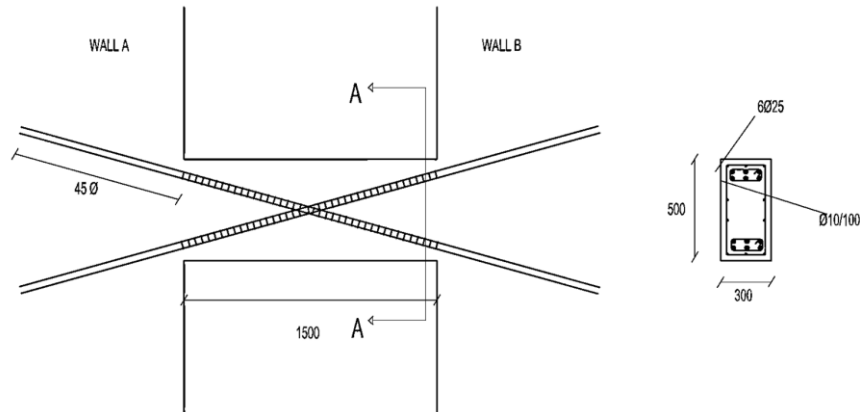


Figure 6.11 : Diagonal reinforcement layout for CW16-CR-N.

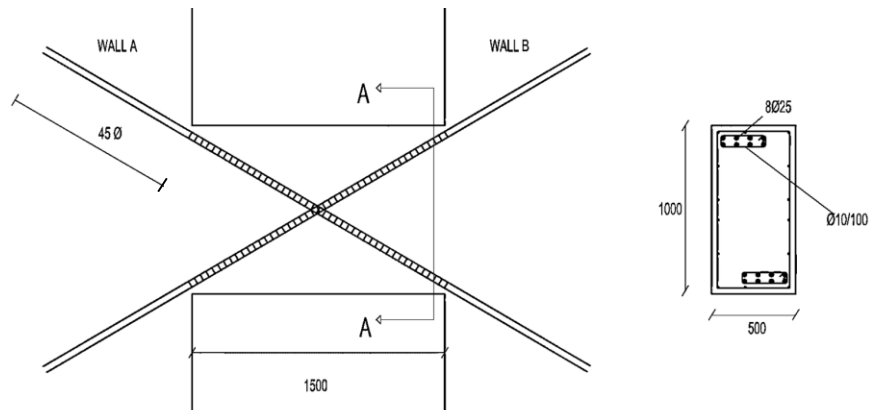


Figure 6.12 : Diagonal reinforcement layout for CW16-CR-H.

Table 6.11 : Coupling beam design summary for 24-story systems.

	CW24-CR-L	CW24-CR-N	CW24-CR-H
Shear Demand [kN]	292	456	856
$1.5(b_w)(d)f_{ctd}$ [kN]	275	572	880
Diagonal angle - α [deg]	9.65	13.77	22.79
$Asd=V/(2f_{yd} \sin(\alpha))$ [m ²]	0.001921	0.002686	0.003025
Diameter [mm]	25	25	22
Diagonal Amount	4	6	8

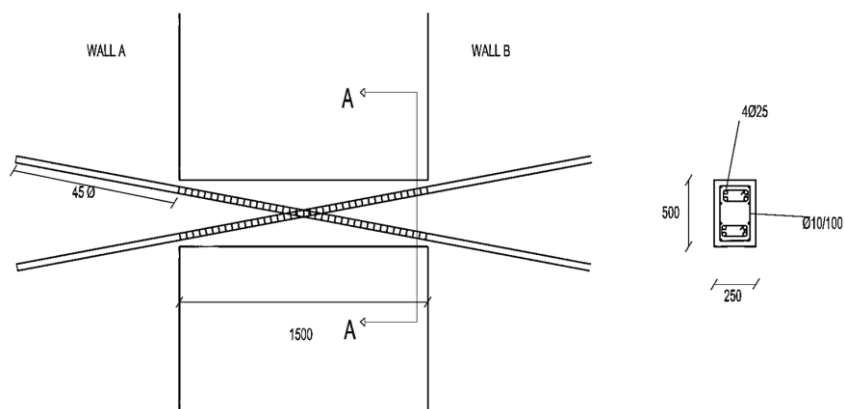


Figure 6.13 : Diagonal reinforcement layout for CW24-CR-L.

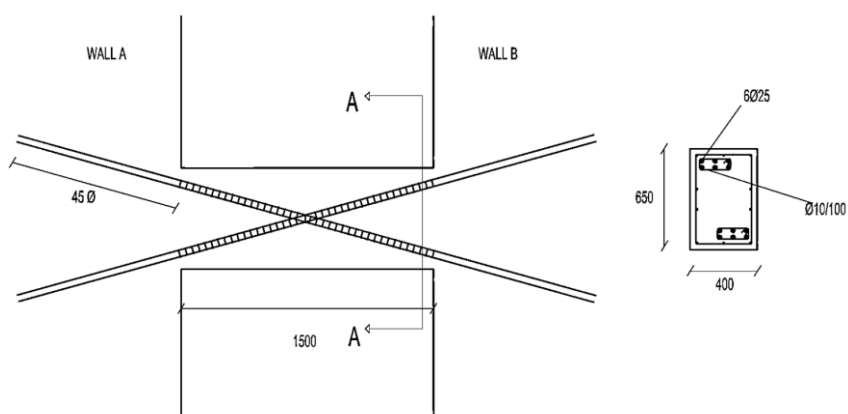


Figure 6.14 : Diagonal reinforcement layout for CW24-CR-N.

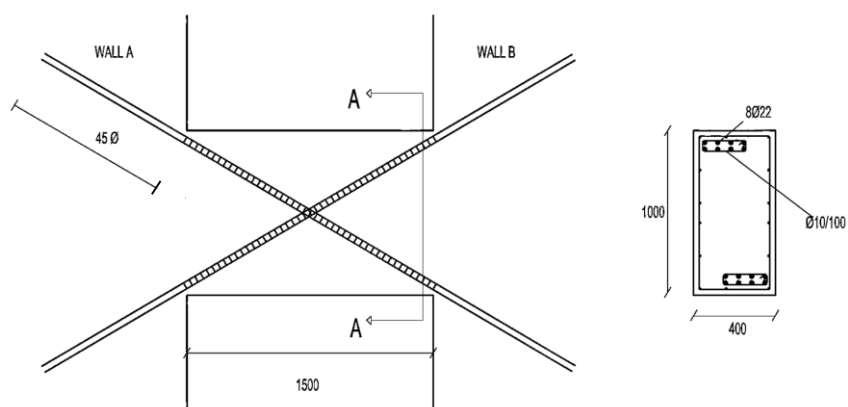


Figure 6.15 : Diagonal reinforcement layout for CW24-CR-H.

6.3.4 Design of wall piers

The wall piers are designed in accordance with Chapter 3 of Turkish Earthquake Code (TEC, 2007). They are assumed to resist the entire seismic load in the direction of coupling (East-West). The 28-day concrete compression strength used for the design is 40 MPa and the yield strength of the shear and flexural reinforcement is selected to be 420 MPa.

Two axial load combinations were considered for the seismic load cases:

$$\text{Tension from coupling beams:} \quad P = 0.9P_G - \sum V_i \quad (6.1)$$

$$\text{Compression from coupling beams:} \quad P = 1.0P_G + 1.0P_Q + \sum V_i \quad (6.2)$$

Here, P_G is the axial compression due to dead load; $1.0P_Q$ is the axial compression due to live load; V_i is the coupling beam shear force at i-th story.

The longitudinal reinforcement layouts and boundary zones of wall piers are designed in compliance with Turkish Earthquake Code (TEC, 2007) requirements. The axial force-bending moment interaction capacity curves are developed by using fiber-based section analysis software XTRACT (2010). The force demands are checked on axial force-bending moment interaction capacity curves in order to avoid over-strength in capacities as shown in Figure 6.16, Figure 6.18 and Figure 6.20.

The reinforcement layouts of the 8-story, 16-story and 24-story walls are shown in Figure 6.17, Figure 6.19 and Figure 6.21, respectively.

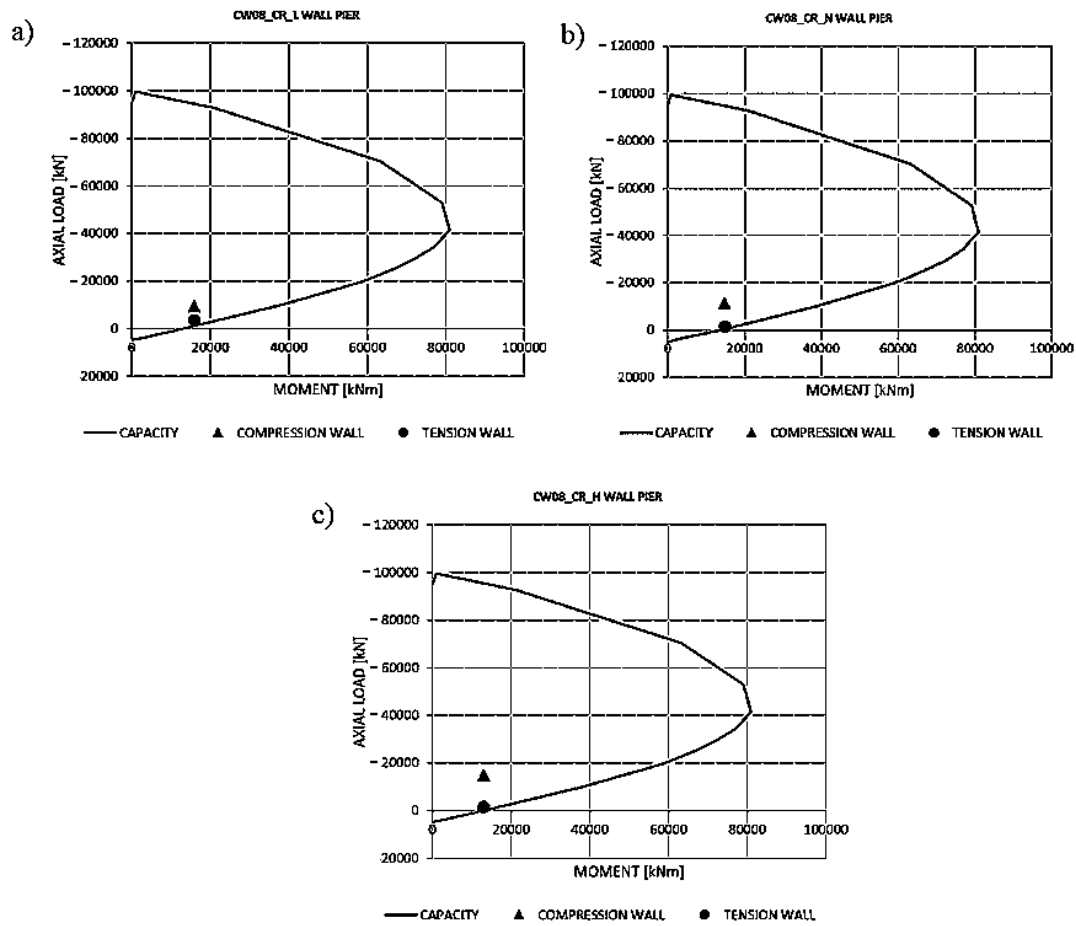


Figure 6.16 : 8-story wall P-M curves a)low, b)normal and c)high coupled.

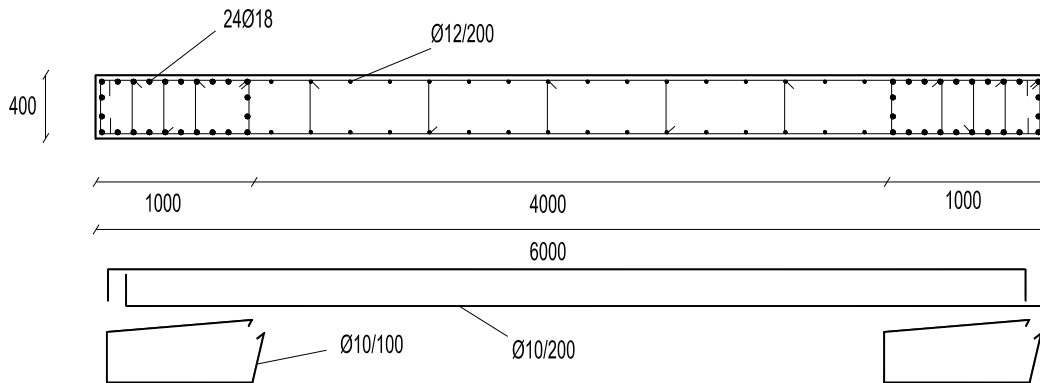


Figure 6.17 : 8-story wall cross-section.

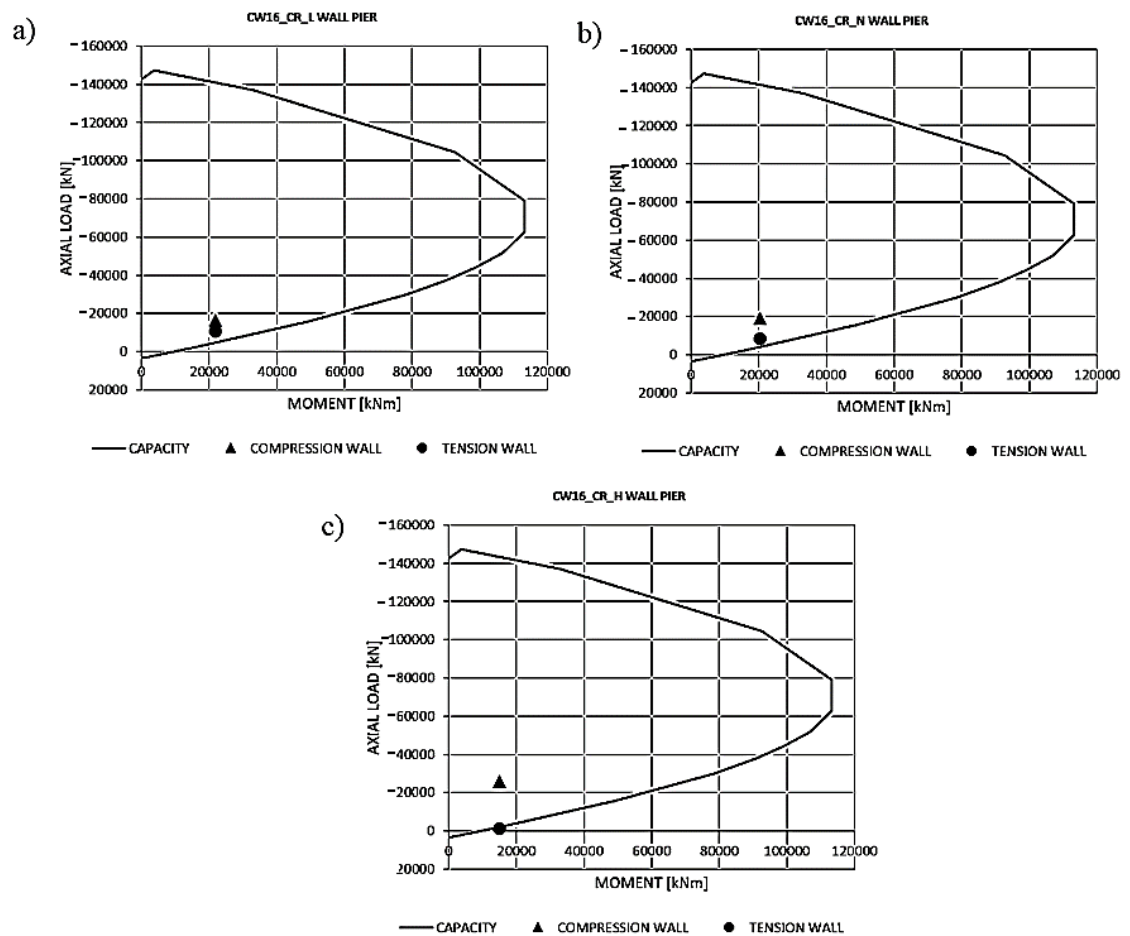


Figure 6.18 : 16-story wall P-M curves a)low, b)normal and c)high coupled.

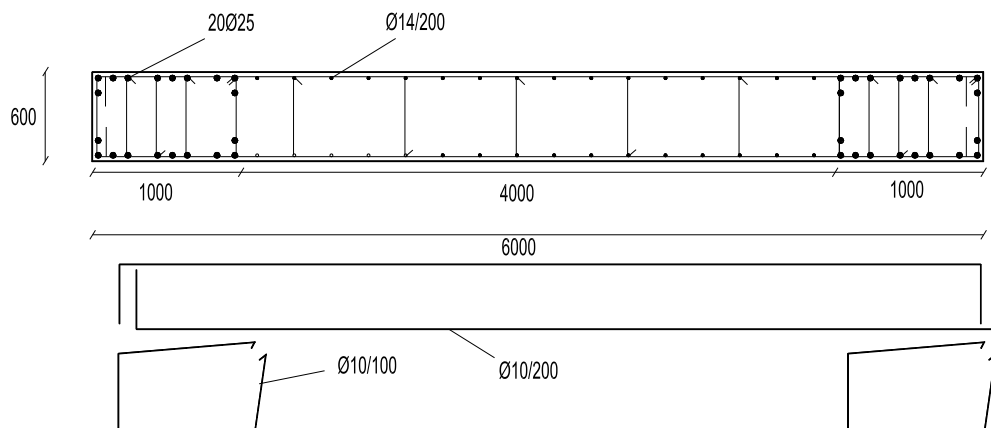


Figure 6.19 : 16-story wall cross-section.

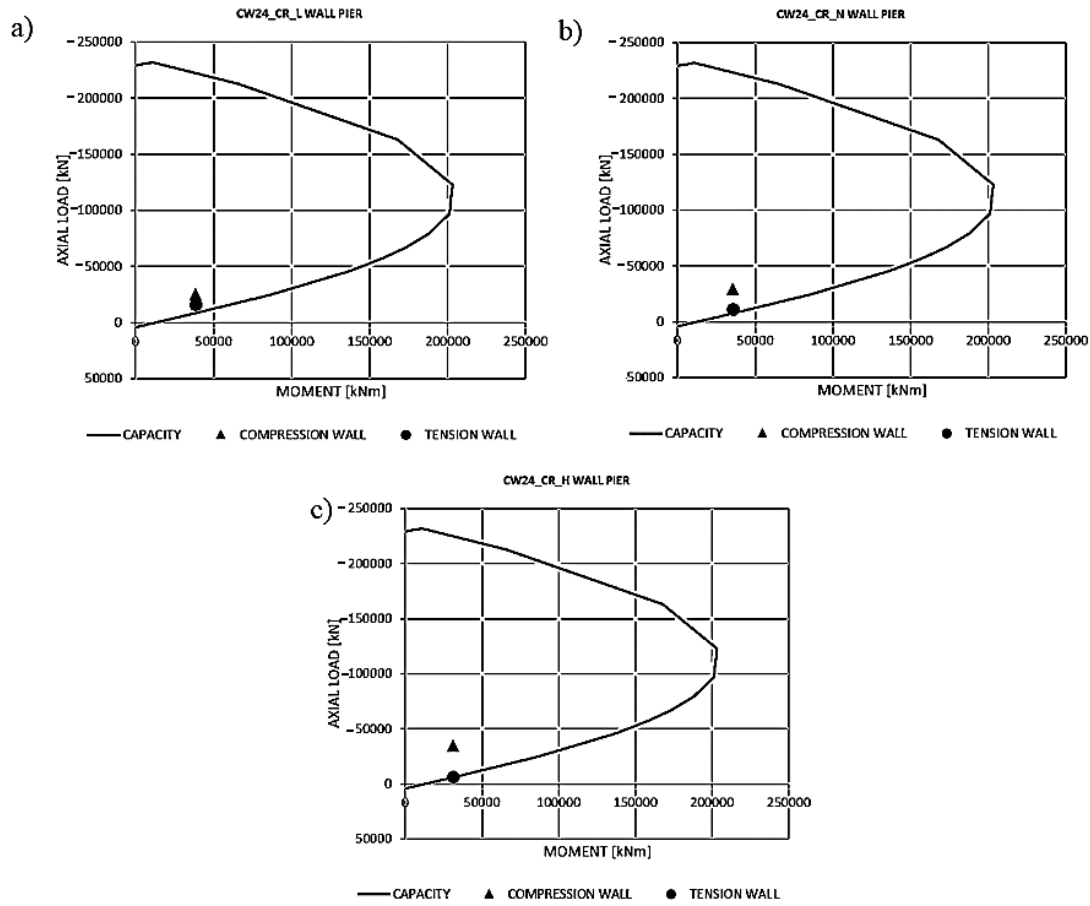


Figure 6.20 : 24-story wall P-M curves a)low, b)normal and c)high coupled.

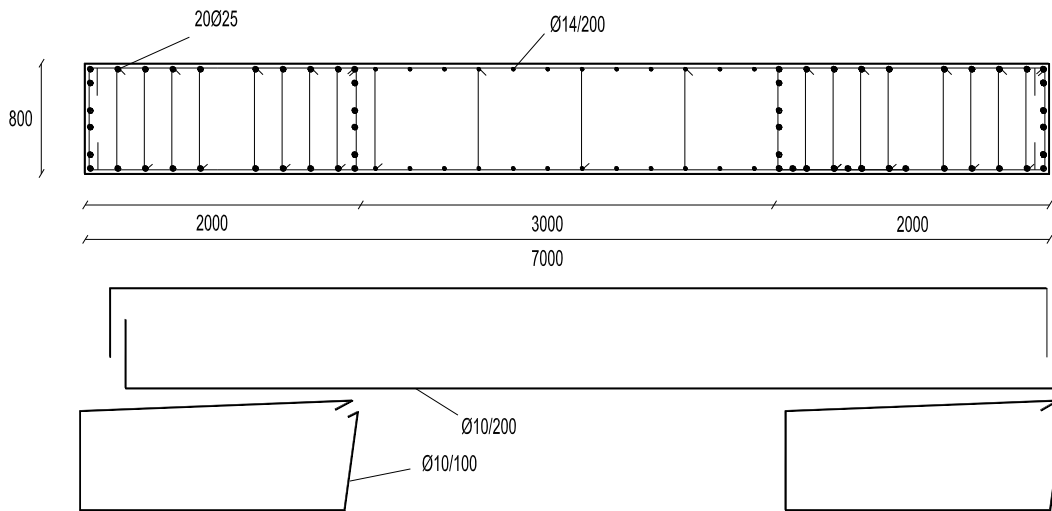


Figure 6.21 : 24-story wall cross-section.

6.3.5 Shear design of wall piers

In Chapter 3, it is explained how to determine the design shear of a wall element in accordance with Turkish Earthquake Code (TEC, 2007). Shear forces obtained from linear analysis must be scaled up by using dynamic shear amplification factor in order

to take into account the higher mode effects. In addition, the maximum probable overturning moment that could be developed for each wall pier must also be estimated.

Although the flexure design of a pier in a coupled wall system is carried out for the lowest axial force case, which might occur under coupling tension action, the same wall would possess significantly larger capacity when coupling beams provide additional compression forces. This situation might exhibit the coupled walls to have larger over-strength values than solid walls without coupling beams (Figure 6.22). At point 1, the coupled wall pier is under gravity load only. Although the considered wall pier is generally designed under load combination where coupling beams transfer axial tension, the same wall also experiences the compression forces from coupling beams (Point 2). The reserve capacity in the compression wall may cause a significant amount of over-strength (Point 3).

Over-strength values and the design shear forces are tabulated in Table 6.12. Although over-strength coefficients increases the design shear forces significantly, Turkish Earthquake Code (TEC, 2007) demand upper boundary limits the design force with demands under $R_a=2$.

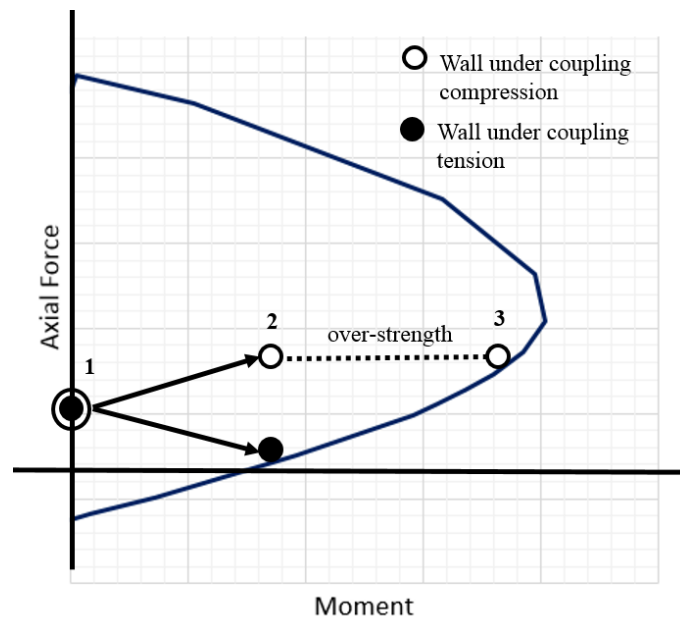


Figure 6.22 : P-M interaction curve of a coupled wall for linear design procedure.

Table 6.12 : Linear analysis shear force results.

	Linear Analysis Ra=6 [kN]	Over- strength	Code Dynamic Amplification	Design Shear Ra=2 upper limit considered [kN]	Max. Allowed Shear (0.83 Ac (fc') ^{0.5})[kN]	Transverse Rein. Ratio Provided
CW08-CR-L	1210	2.42	1.5	3630	12599	0.0025
CW08-CR-N	1506	2.71	1.5	4518	12599	0.0039
CW08-CR-H	1982	3.14	1.5	5946	12599	0.0064
CW16-CR-L	1265	2.25	1.5	3795	18898	0.0025
CW16-CR-N	1356	2.7	1.5	4068	18898	0.0025
CW16-CR-H	1818	2.91	1.5	5454	18898	0.0025
CW24-CR-L	1594	2.22	1.5	4782	29397	0.0025
CW24-CR-N	1830	2.57	1.5	5490	29397	0.0025
CW24-CR-H	2174	2.81	1.5	6522	29397	0.0025

6.4 Nonlinear modeling of investigated coupled wall systems

As it has already been discussed in Chapter 5, the nonlinear behavior of a coupled wall system reveals the disparity of the wall pier flexure capacities, even if the wall piers are identical. In that case, the wall pier under coupling tension experience flexural yield before the compression wall; therefore a redistribution in internal forces (moment and shear) will be expected between each wall piers.

6.4.1 Analysis software

The utilized software OpenSees (2013), the Open System for Earthquake Engineering Simulation, is an object-oriented, open source software framework created at the National Science Foundation sponsored Pacific Earthquake Engineering (PEER) Center.

Object-oriented programming (OOP) is a programming paradigm that represents concepts as “objects” that have data fields (attributes that have data) and associated procedures known as methods. Objects, which are instances of classes, are used to interact with one another to design applications and computer programs. In addition to achieving computational efficiency, OpenSees is designed to be flexible, extensible, and portable.

It allows users to create finite element applications for simulating the response of structural and geotechnical systems subjected to earthquakes. OpenSees is primarily written in C++ (Stroustrup, 2010) and uses several FORTRAN (Backus et al., 1957)

numerical libraries for linear equation solving. Users of OpenSees create applications by writing scripts in the Tcl programming language.

Tcl is known as a powerful yet succinct program scripting language created by John Ousterhout (1994). Ousterhout created Tcl while he was a professor at University of California, Berkeley. It originally derives from “Tool Command Language” and it is used on embedded systems platforms, both in its full form and in several other small-footprint versions.

The goal of OpenSees development is to improve the modeling and computational simulation in earthquake engineering through open-source development, which requires extensive extensibility and maintainability.

OpenSees is not a “code” by the usual definition of a program to solve a specific class of problems. Rather, it is comprised of a set of classes and objects that represent different models, which perform computations for solving the governing equations and provide access to processing results (McKenna, 2011).

Besides, OpenSees is highly modular. The package diagrams in Figure 6.23 show the framework’s finite element method and reliability parts. Unlike traditional commercial and research applications, this modularity allows developers to extend the code in the analysis reliability and output specific parts of the software. In Figure 6.23, every class in italics is an abstract class and developers can provide their own subclass to personalize their applications. This is particularly important in earthquake engineering, where researchers are studying new algorithms and integration schemes, such as collapse analysis and hybrid testing.

Specifically, ModelBuilder is the object in the program responsible for building the Element, Node, LoadPattern, timeSeries, Load and Constraint objects. The Domain object is responsible for storing the objects created by the ModelBuilder and for providing the Analysis and Recorder with access to these objects. The Analysis object is responsible for performing the analysis, which may vary from a simple static linear analysis to a transient non-linear analysis. In OpenSees, each analysis object is composed of a SolutionAlgorithm, an Integrator, a ConstraintHandler, a DOF_Numberer, a SystemOfEqn and an AnalysisModel. The Recorder is to define the unknown to be monitored during the analysis. Benefits of using OpenSees are summarized in Table 6.13.

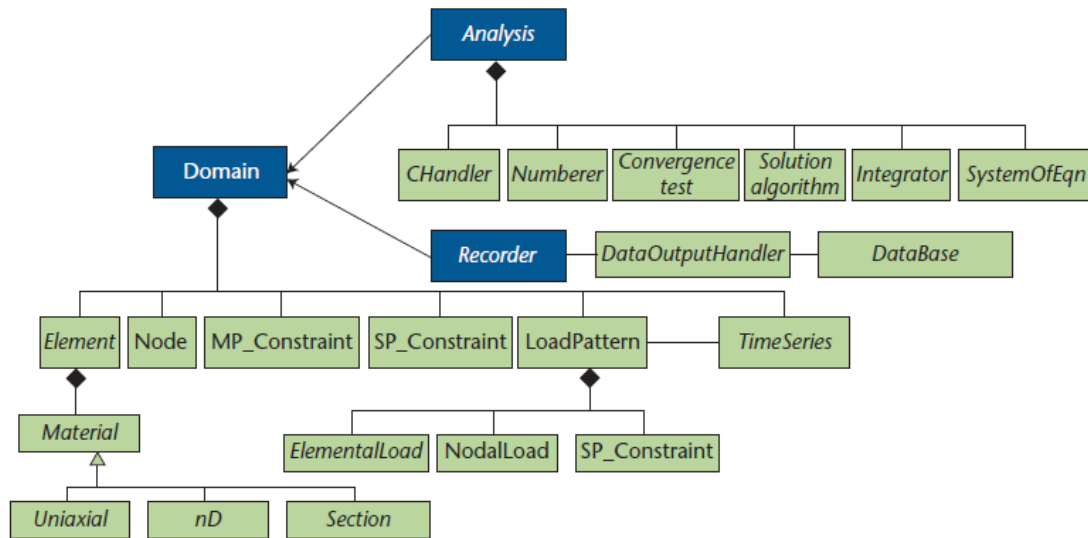


Figure 6.23 : OpenSees framework.

Table 6.13 : Advantages and of OpenSees.

1	Open Source
2	Free
3	Specialized element and material models (Fiber Column-Beam El.)
4	Easy to introduce new materials and elements to library
5	C++ Object Oriented Design
6	Reliability Analysis Capabilities
7	Hybrid analysis feature over 3 rd party software (i.e. Matlab)

6.4.2 Nonlinear model of the considered coupled wall systems

The nonlinear behavior of the nine different coupled wall systems with 8,16 and 24 stories was analytically investigated by using OpenSees (2013). The walls are modeled using force-based beam-column element with fiber sections available in the software. The coupling beams are modeled with elastic beam with flexure hinges at both beam-wall interface ends. Coupling beam nonlinear hinge and elastic line element properties have been arranged appropriately as defined in Chapter 4 in order to capture slip/extension and flexural behaviors. Wall elements and coupling beams are connected with rigid line elements. The general representation of the nonlinear coupled wall system modeling is given in Figure 6.24.

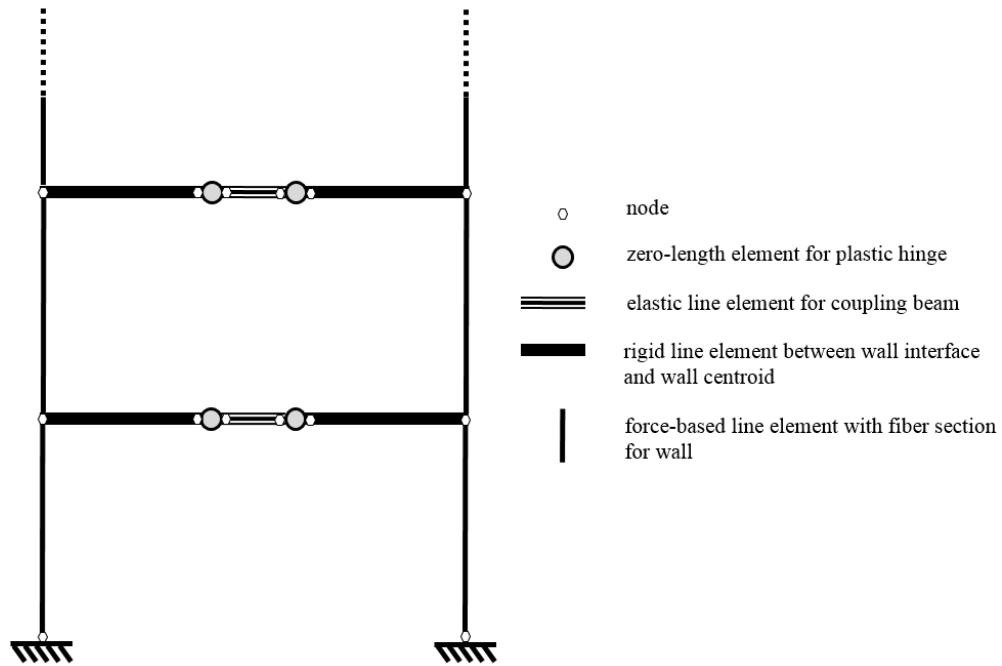


Figure 6.24 : Nonlinear modeling for the investigated coupled wall systems.

6.4.3 Nonlinear wall element

The fiber section model, originally developed by Taucer et al. (1991), has been recognized as one of the well-known methods for static and dynamic analysis of frames and reinforced concrete shear walls. In this model, there are a number of control cross-sections along the element. Each cross-section is subdivided into concrete and steel fibers where uniaxial stress-strain laws are used to describe the response of the material in the longitudinal direction (parallel to the element axis). These sections are located at the control points of numerical integration scheme (Figure 6.25). The characteristics of the element, such as element displacement and element forces are derived from integrating their sectional components along the element. The corresponding sectional components, such as section forces, are calculated from integration of the response of fibers (Taucer et al., 1991).

The force-based elements, characterized by exact force interpolation functions, permit the use of one element for the representation of the non-linear response of a wall member or a wall's interstory. In contrast, displacement-based beam-column elements follow the standard finite element formulation, in which the element displacement field is derived from nodal displacements.

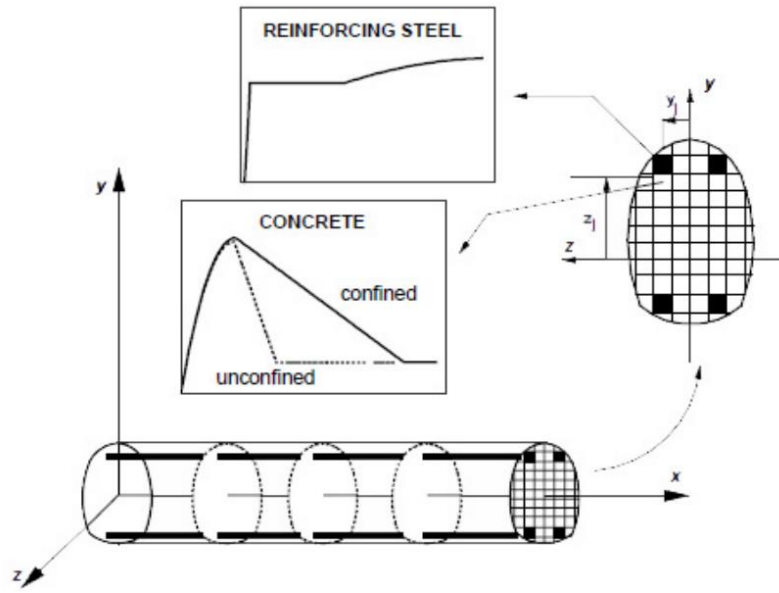


Figure 6.25 : Fiber section (Taucer et al., 1991).

As result, the displacement field is approximate, thus several integration points are required to represent the deformations in a plastic hinge region (Martinelli, 2007). Numerical and experimental researches made by Orakçal et al. (2006) and Vechhio et al. (2013) are one of the numerous amounts of studies, which include OpenSees fiber section flexural behavior validation with test results.

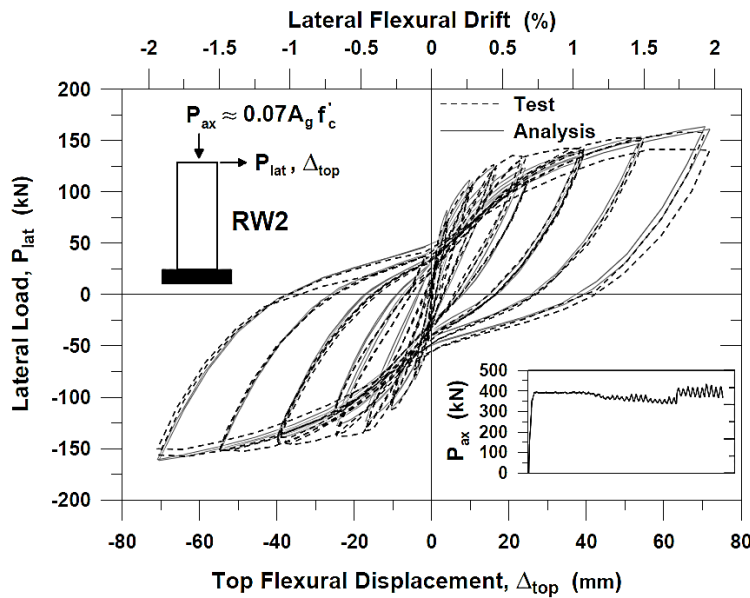


Figure 6.26 : OpenSees fiber model validation for flexure (Orakçal et al., 2006).

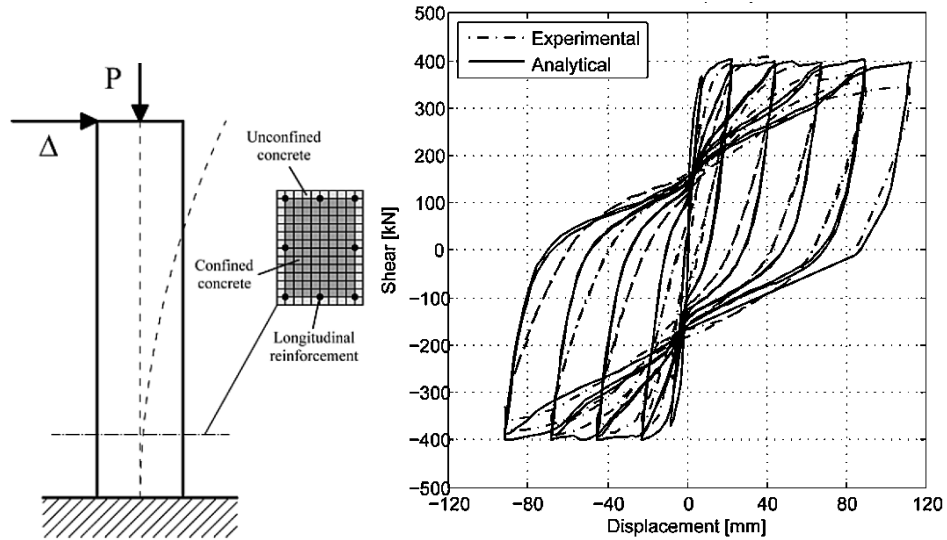


Figure 6.27 : OpenSees fiber model validation for flexure (Vecchio et al., 2013).

Consequently, the force-based element was expected to capture flexure response more accurately than a displacement-based element. In this dissertation, the distributed plasticity beam-column element with force formulation is selected for wall element modeling (Figure 6.28). The shear behavior is assumed elastic for these sections.

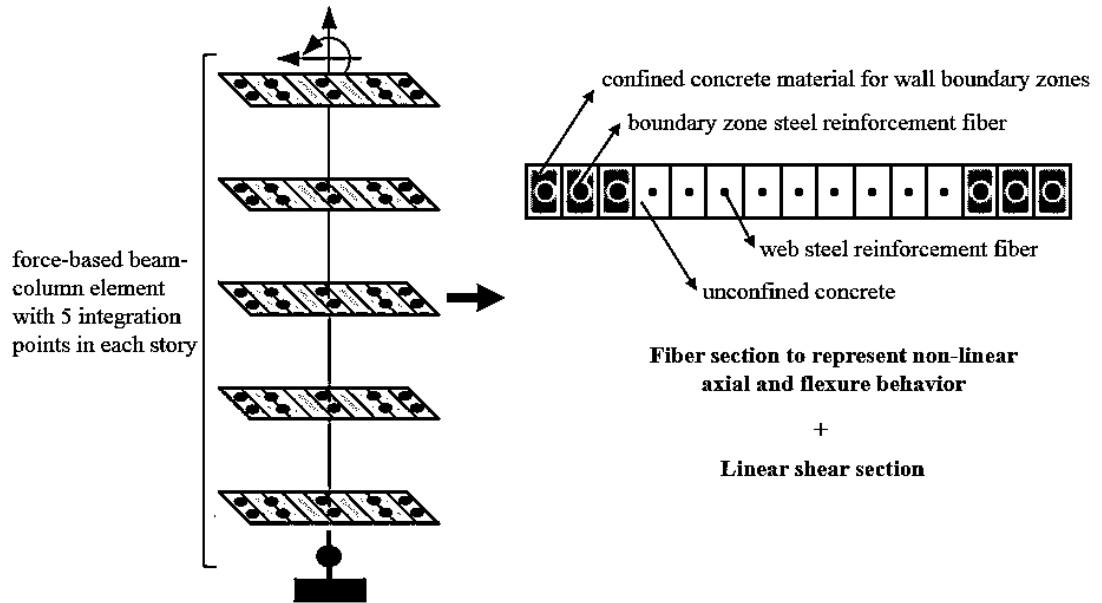


Figure 6.28 : Force-based element with 5 integration points.

6.4.4 Concrete material model in wall elements

Several types of uniaxial concrete materials are available in OpenSees. In this study for concrete fibers, the modified Kent-Park model (concrete02) were used (Kent and Park, 1971). The modified Kent-Park model consists of an ascending parabolic branch and a descending linear part for strains greater than the strain corresponding to the

peak stress (Figure 6.29). Here, f_{pc} is concrete compression strength at 28 days; eps_{c0} is the concrete strain at maximum strength; f_{pcU} is the concrete crushing strength; eps_U is the concrete strain at crushing strength; λ is the ratio between unloading slope at eps_U and initial slope; f_t is the tensile strength and E_{ts} is the tension softening stiffness. Expected material strength of concrete is used for nonlinear analysis models. Concrete material properties are summarized in Table 6.14.

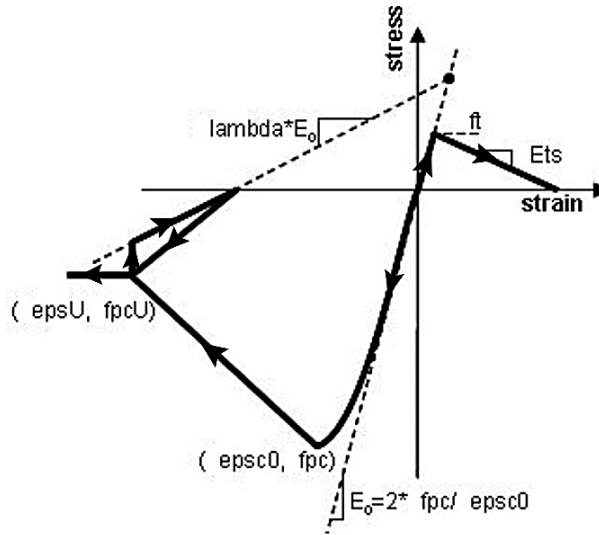


Figure 6.29 : Concrete02 model (Kent and Park, 1971).

Table 6.14 : Confined concrete material properties in OpenSees (2013).

	Parameter	Value
compression strength	f_{pc}	-65000 kN/m ²
strain at maximum strength	eps_{c0}	-0.005
crushing strength	f_{pcU}	-45300 kN/m ²
strain at concrete crushing	eps_U	-0.014
unloading slope coefficient	λ	0.5
tensile strength	f_t	6500 kN/m ²
tensile softening stiffness	E_{ts}	3252 kN/m ²

6.4.5 Steel reinforcement material model in wall elements

The material model used for the steel reinforcement was originally developed by Menegotto and Pinto (1973). This model is very practical for numerical analyses and also exhibits very accurate results compared to previously performed test results (Yassin, 1994). The stress and strain relation of Menegotto and Pinto (1973) steel reinforcement model is described below.

$$\sigma^* = b\varepsilon^* + \frac{(1-b)\varepsilon^*}{(1+\varepsilon^*R)^{1/R}} \quad (6.3)$$

$$\varepsilon^* = \frac{\varepsilon_s - \varepsilon_r}{\varepsilon_0 - \varepsilon_r} \quad (6.4)$$

$$\sigma^* = \frac{\sigma_s - \sigma_r}{\sigma_0 - \sigma_r} \quad (6.5)$$

Here, σ_r and ε_r represents the stress and strain values at the point where the last strain reversal with stress of equal sign took place (point A in Figure 6.30). σ_0 and ε_0 are the stress and strain values where two asymptotes of branch under consideration meet (point B in Figure 6.30). b is the strain hardening ratio, that is the ratio between E_1 and E_0 . The hardening is neglected for simplicity.

R is a parameter which influences the shape of the transition curve and allows a good representation of the Bauschinger effect. Menegotto and Pinto (1973) suggests The expression of R takes the form:

$$R = R_0 + \frac{a_1\xi}{a_2+\xi} \quad (6.6)$$

Where ξ is updated following a strain reversal (Figure 6.30). R_0 is the value of the parameter R during first loading and a_1 , a_2 are experimentally determined parameters to be defined together with R_0 .

Steel01 model in OpenSees (2013) was used for reinforcing steel material. The stress-strain curve relation is idealized as a bilinear. The expected yield stress of S420 reinforcing steel material is 491400kN/m² (117% of the characteristic yield stress) and the hardening is neglected for simplicity.

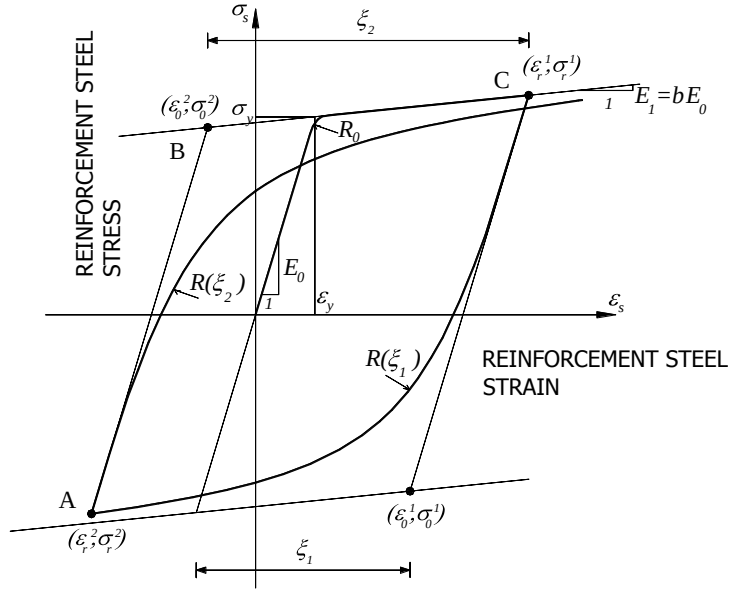


Figure 6.30 : Reinforcement steel model.

6.5 Pushover Analyses and Results on 8-Story Building

Pushover is not a conventional method for coupled wall systems since they are often tall structures with higher modes dominating the overall response. The pushover analysis is used here in order to understand the plastic behavior progression and the force redistribution sequence of the structural system.

First, pushover analysis is conducted for the 8-story systems by applying an incrementally increasing lateral loads in order to observe the plastic hinge propagation sequence and compare it with nonlinear dynamic analysis featured in the following chapters. The pushover force pattern is illustrated in Figure 6.31.

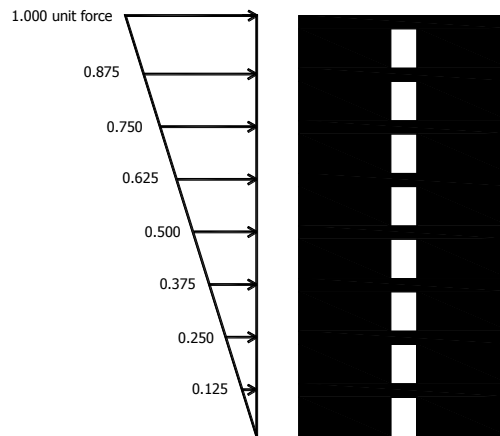


Figure 6.31 : Pushover load pattern.

Prior to the application of lateral loads, appropriate gravity loads tributary to the wall piers have been applied. These loads are treated as forces and do not affect the seismic mass (i.e. dynamic properties) of the structure. While the walls resist 100% of lateral loads, they carry only tributary gravity loads; therefore, the loads tributary to the walls are significantly less than the seismic mass assigned to the walls. The presence of gravity loads is important as these resist uplift in tension wall. First, the gravity loads are applied, then the lateral pushover loads are applied incrementally.

Overturning moment and coupling ratio versus maximum interstory drift ratio pushover curves for the analyses of 8 story buildings with low, normal and high level coupling ratio are presented in Figure 6.32, Figure 6.33 and Figure 6.34. Although walls in all systems are identical, it is observed that the stiffness and strength properties of coupling beams effect the system's behavior and its capacity. As coupling ratio increases, the overturning resistance of the system also increases. Changing the coupling degree also affects the sequences of plastic hinge propagations. In low and normal coupled systems (CW08-CR-L and CW08-CR-N), all of the coupling beams yield before walls. In these wall systems, hinges initially propagates at mid-story levels. It is observed that the first story coupling beam yields after all other beams yield.

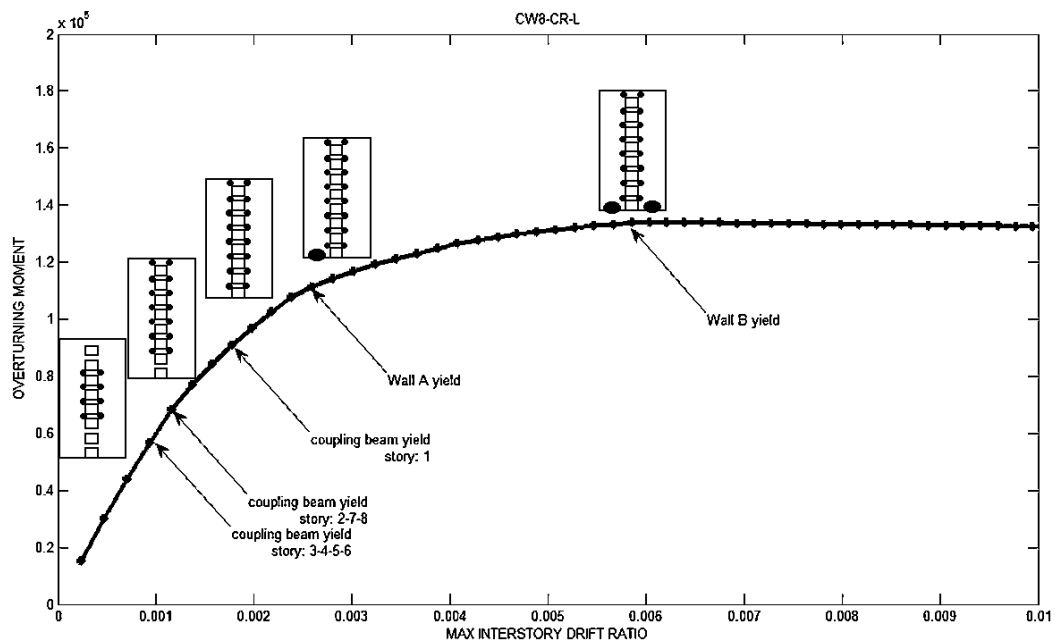


Figure 6.32 : Pushover - moment vs. maximum interstory drift ratio CW08-CR-L.

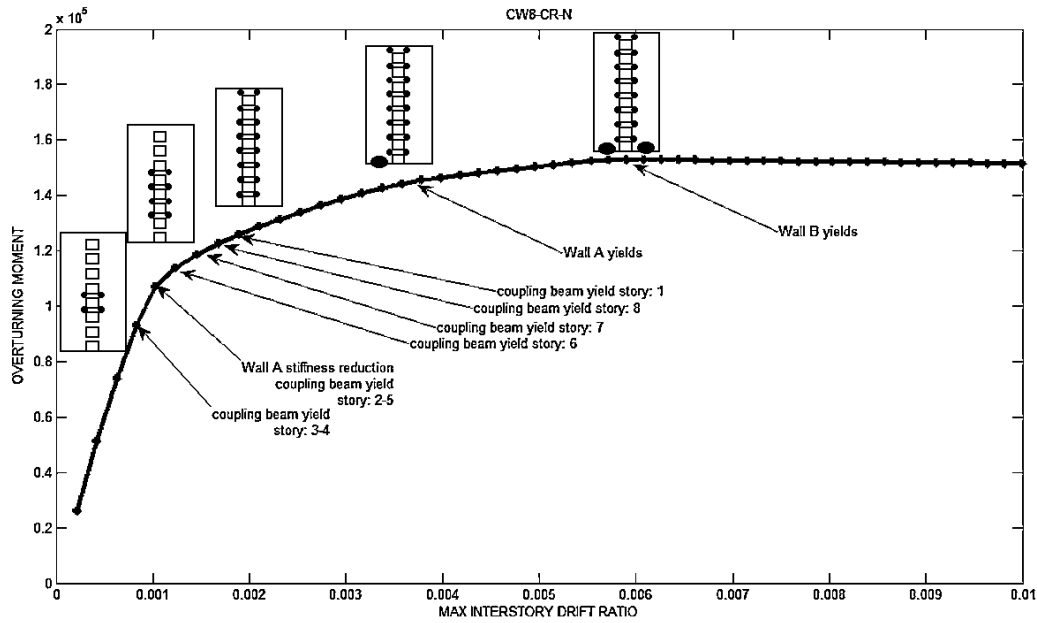


Figure 6.33 : Pushover - moment vs. maximum interstory drift ratio CW8-CR-N.

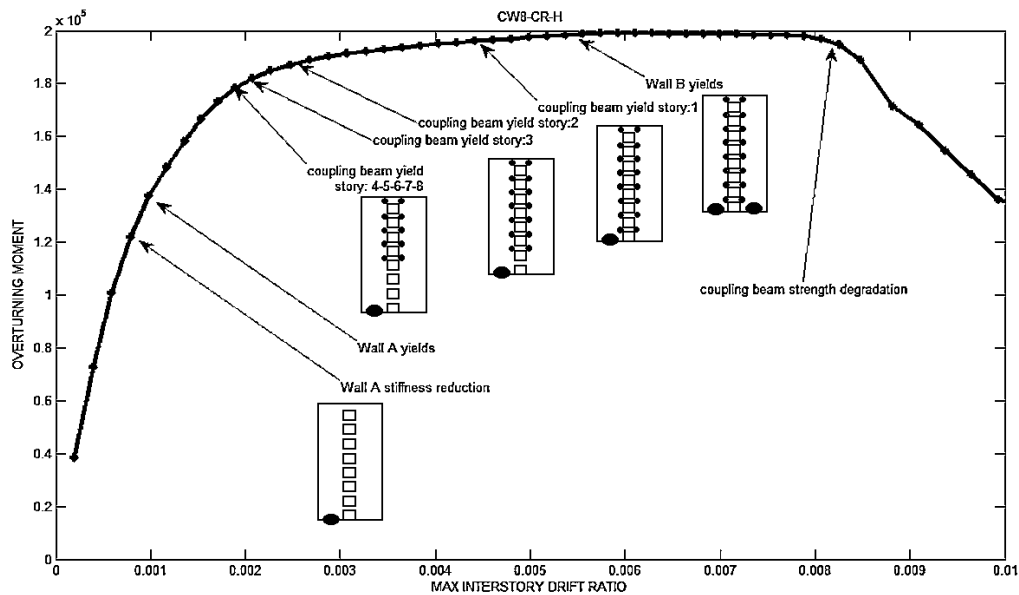


Figure 6.34 : Pushover - moment vs. maximum interstory drift ratio CW8-CR-H.

Since the axial force in tension wall (Wall A) is always lower than the compression wall (Wall B), the flexural demand in Wall A reaches to its capacity sooner, causing it to yield before Wall B. For high coupled system (CW8-CR-H), the tension wall (Wall A) yields even before the coupling beams.

The flexural moment curves of walls and coupling moments are illustrated for all three wall systems in Figure 6.35. The coupling ratio curves are extracted by dividing the coupling moment to the system over-turning moment.

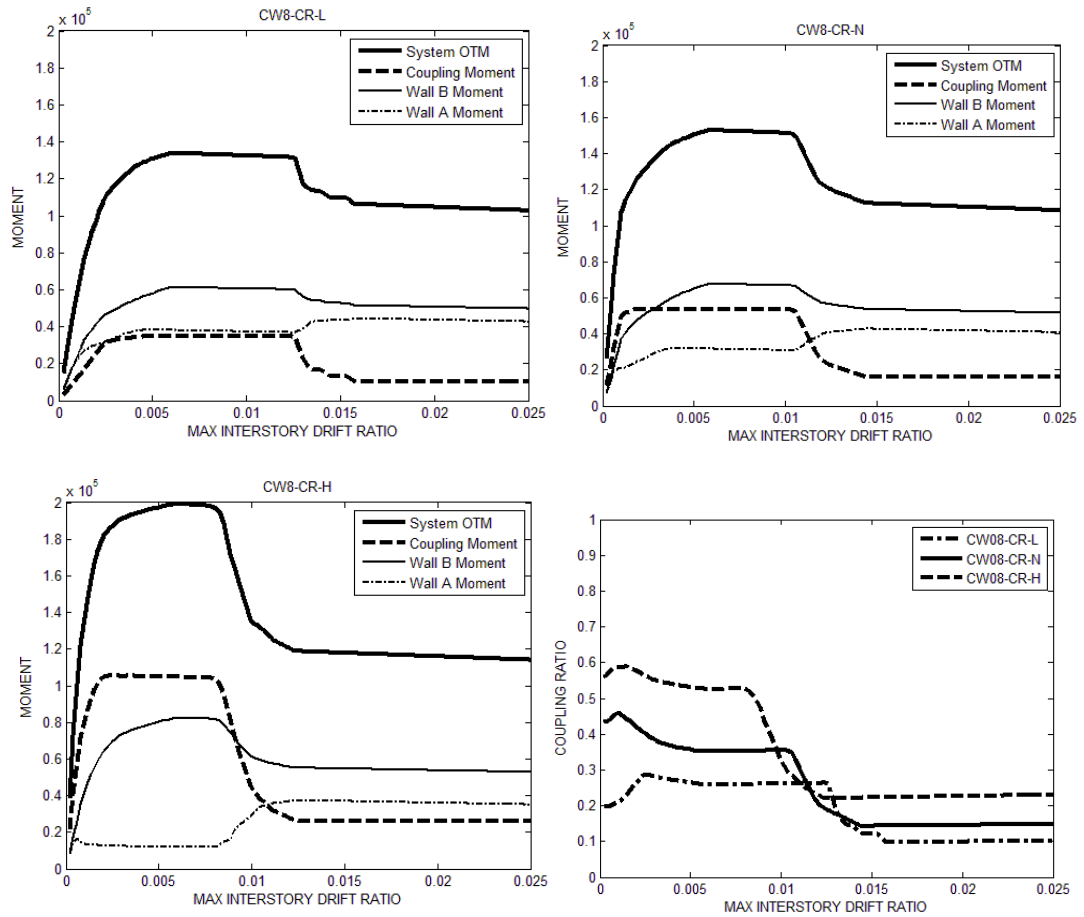


Figure 6.35 : Pushover analysis – moments vs. maximum interstory drift ratio.

Depending on the initial coupling ratio, which was adopted during the design with the linear analysis procedures, the flexural capacities of both walls differ from each other. Once the beams begin to yield, the coupling action begins to degrade and the lateral forces once resisted by coupling frame action are redistributed over the wall piers.

6.6 Nonlinear Dynamic Analyses and Results

In this research work, two types of nonlinear time history analyses were conducted: single-stripe analysis, where the records are run at a suite spectral acceleration stripes, and the cloud approach.

The single stripe analysis method is used for evaluating the performance level requirements and also for investigating the distribution of internal forces for different earthquake levels. Each individual coupled wall system was conducted with nonlinear time history analyses by using 20 ground motion records, which were scaled to the corresponding target earthquake level spectrum.

The cloud analysis uses 50 ground motion records which are not scaled and have varying peak ground acceleration (PGA) values. The main purpose of this analysis procedure is to observe an engineering demand parameter (EDP) of interest under increasing peak ground acceleration values. In this case, the alteration of coupling ratio has been investigated under increasing deformation caused by varying PGA values. Nonlinear dynamic analyses used in this research is summarized in Figure 6.36.

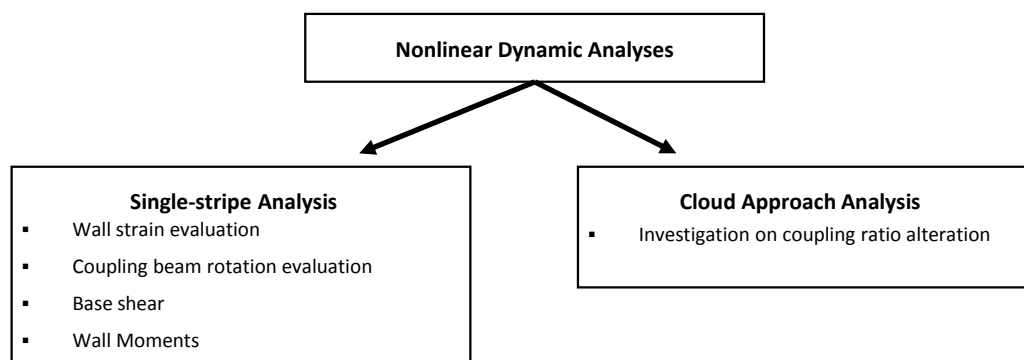


Figure 6.36 : Nonlinear dynamic analyses.

Modal damping was applied as 5% in all nonlinear dynamic analysis cases. The base restraints are assumed to be fixed and soil-structure interaction was not considered within the scope of this research.

6.6.1 Cloud approach ground motion records

As described in Chapter 5, cloud approach uses the records which are not scaled to a specific acceleration level. Instead, the analyses are performed by a set of accelograms from a common scenario, e.g. similar magnitudes and distances. In this dissertation, the “cloud approach” is used in order to investigate the alteration patterns of coupling ratio values versus obtained maximum plastic hinge rotations of beams. The properties of selected 50 records and their PGA distribution is given in Table 6.15 and Figure 6.37.

Although the records follow a regularly sorted PGA levels, the response is expected to have high record-to-record variability. This is because PGA is an intensity measure that is correlated with the structural response parameters.

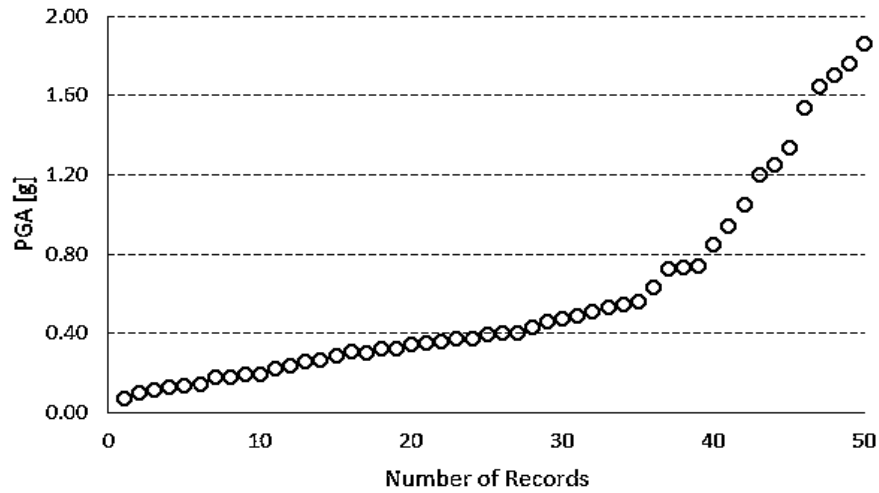


Figure 6.37 : Distribution of the PGA values of the selected 50 records.

Table 6.15 : Ground motion records used in NLTHA.

No.	Event	Station	Magntd	SF	PGA[g]	No.	Event	Station	Magntd.	SF	PGA [g]
1	Landers	NYA090	7.3	3	0.072	26	Landers	BRSW090	7.3	3	0.405
2	Landers	RIM015	7.3	3	0.105	27	Kocaeli	IZN090	7.4	3	0.408
3	Landers	SIL090	7.3	3	0.12	28	Landers	ABY090	7.3	3	0.438
4	Kocaeli	MSK000	7.4	3	0.132	29	Landers	DSP090	7.4	3	0.462
5	Landers	BAD270	7.3	3	0.138	30	Kocaeli	FAT090	7.4	3	0.477
6	Landers	SIL000	7.3	3	0.06	31	Kocaeli	ATK090	7.4	3	0.492
7	Landers	29P090	7.3	3	0.18	32	Landers	DSP000	7.3	3	0.513
8	Kocaeli	KUT090	7.4	3	0.18	33	Kocaeli	CNA000	7.4	3	0.537
9	Landers	PMN000	7.3	3	0.09	34	Kocaeli	ATS090	7.4	3	0.552
10	Landers	EUC022	7.3	3	0.201	35	Kocaeli	FAT000	7.4	3	0.561
11	Landers	PSA000	7.3	3	0.228	36	Kobe	SHI090	6.9	3	0.636
12	Landers	29P000	7.3	3	0.24	37	Kobe	SHI000	6.9	3	0.729
13	Landers	HOS180	7.3	3	0.261	38	Landers	YER270	7.3	3	0.735
14	Landers	BOR090	7.3	3	0.27	39	Kocaeli	ATS000	7.4	3	0.747
15	Kocaeli	IZN180	7.4	3	0.294	40	Landers	CLW-LN	7.3	3	0.849
16	Kocaeli	BUR000	7.4	3	0.309	41	Friuli	A-TMZ270	6.5	3	0.945
17	Kocaeli	ATK000	7.4	3	0.3075	42	Friuli	A-TMZ000	6.5	3	1.053
18	Landers	BAK050	7.3	3	0.324	43	Chalfant	A-ZAK360	6.2	3	1.2
19	Kocaeli	ZYT090	7.4	3	0.33	44	Landers	-	7.3	3	1.251
20	Landers	ABY000	7.3	3	0.345	45	Chalfant	A-ZAK270	6.2	3	1.341
21	Landers	BOR000	7.3	3	0.357	46	Northridge	-	6.7	3	1.542
22	Duzce	MDR000	7.1	3	0.36	47	Cape	RIO360	7.1	3	1.647
23	Landers	MCF090	7.3	3	0.375	48	Northridge	-	6.7	3	1.704
24	Landers	MCF000	7.3	3	0.378	49	Victoria	CPE315	6.4	3	1.761
25	Kocaeli	GYN000	7.4	3	0.396	50	Victoria	CPE045	6.4	3	1.863

6.6.2 Records selected to match the target spectra

Coupled wall systems, which have been designed under 475-year return period earthquake as stated in previous chapters, will now be assessed under different levels of earthquakes with return periods of 72-years, 475-years and 2475-years. At least 20 ground motions were matched target spectral acceleration for each level of earthquake.

The ground motion records are selected and matched to the target spectrum with the following principles:

1. The duration of strong motion part of the acceleration is at least 15 seconds or 5 times of the first natural vibration period of the structure.
2. The average of spectral acceleration values of seismic ground motions corresponding to period zero is not less than the peak ground acceleration of the target spectra.
3. The average of spectral acceleration values are at least 90 % of target spectral acceleration for the periods between $0.2T_1$ and $2T_1$, where T_1 is the fundamental period of the investigated structure.

The spectrum matching procedures in the codes are often set to satisfy the needs of short and mid-rise structures. This is the reason why peak ground acceleration (PGA), a ground motion parameter, which is shown to be of little use in correlating with structural damages, is set as one of the criteria.

The criteria #2 given above practically cannot be applied to tall structures, for instance. Tall structures fall into the long-period range of the spectra, thus matching the average spectra in the long period range yields in underestimating the PGA. Consequently, the second rule mentioned above is ignored in our spectra matching procedure.

The sets of records have been developed for each different coupling ratio levels and for different number of stories, since the fundamental periods would also be changed depending on the structural properties. Sample spectral acceleration graphs for the coupled wall system are given in the appendices.

The record selection has been done by using the PEER NGA database where 7025 recorded motions are available. The scale factor needed is calculated by equating the area below and above the target spectrum as shown in Figure 6.38. The scale factors are set such that the average of 20 records does not go below the target spectrum in certain percentages and most of the cases the average spectrum is not allowed to go below the target spectrum at all. Similarly, the average spectrum is not allowed to go above 30% of the target spectrum in any point within the period window. In order to control the difference of the positive and negative peaks, where the positive peaks refer to the peaks above the target spectrum and vice versa, another criterion is also applied to check the individual records. According to these points, the individual record is not allowed to go below the target spectrum less than 50%, or above more than 200-300% in any of the peaks.

Estimation of the Scale Factor

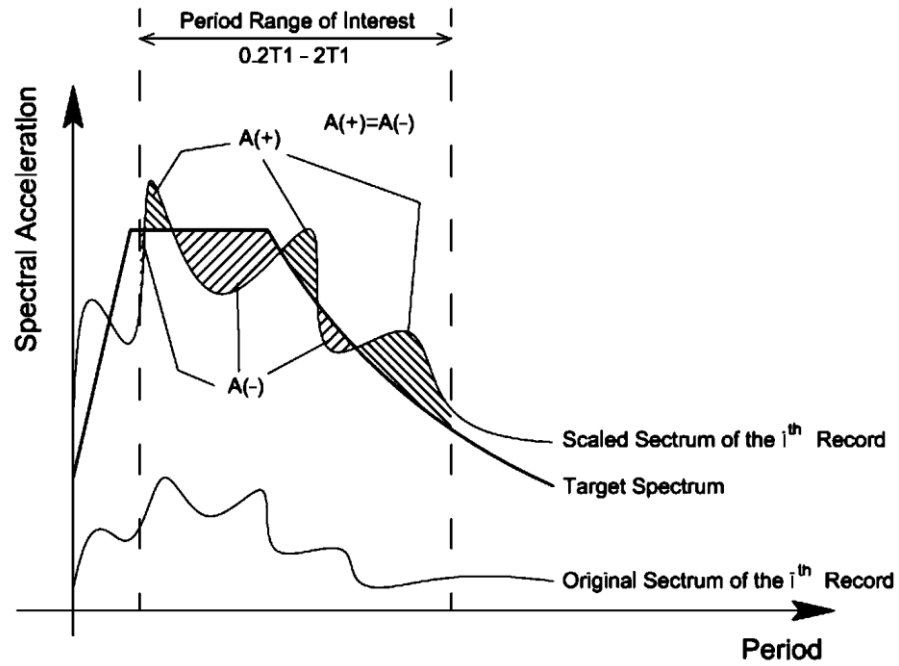


Figure 6.38 : Estimation of the scale factor.

Since the assumed location of the case study building site is at least 40km far from the fault line, the near-fault effect was not considered within the scope of this study. On the other hand, the previous studies show that the estimated increased spectral values can be 2.5 times higher than the original ones. Moreover, it was observed that the effects are more pronounced in terms of drift in a distance of 0-7.5 km (Smyrou, 2014).

6.6.3 Coupling beam plastic rotation

The most widely used deformation parameter to construct backbone curves for coupling beams is the chord rotation of the coupling beam. The chord rotation θ , is the deformation of the beam observed on both ends of the member as illustrated in Figure 6.39. Here, L is the span length of the coupling beam, and Δ is the distance between the normal lines of wall-beam interface at each end. The backbone curves have been developed based on the studies described in Chapter 4.

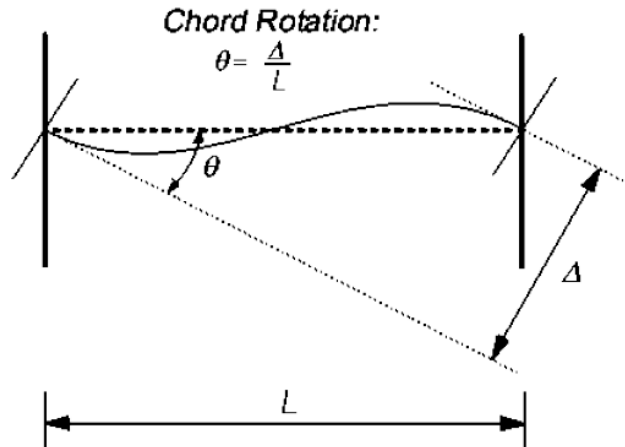


Figure 6.39 : Chord rotation for coupling beams (ASCE 41, 2013).

Allowable plastic rotations for each performance levels are listed in Table 6.16, which are reproduced from ASCE 41 (2013).

Table 6.16 : Coupling beam allowable plastic rotations (ASCE 41, 2013).

Performance Level	Plastic Rotation [rad]
Immediate Occupancy	0.006
Life Safety	0.030
Collapse Prevention	0.050

The mean of coupling beam plastic rotations obtained along the height are shown for 8-story systems in Figure 6.40, Figure 6.41 and Figure 6.42 for different levels of coupling ratio. The vertical axis represent the building story level. In the plots, three different earthquake levels are indicated for each story level. “Triangle” markers represent the absolute mean rotations of coupling beams under service level (50% in 50 years) earthquake. “Square” and “circle” markers represent design level (10% in 50 years) and maximum considered earthquake (2% in 50 years), respectively. The plots also include the allowable plastic rotation boundary lines for Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) performance levels.

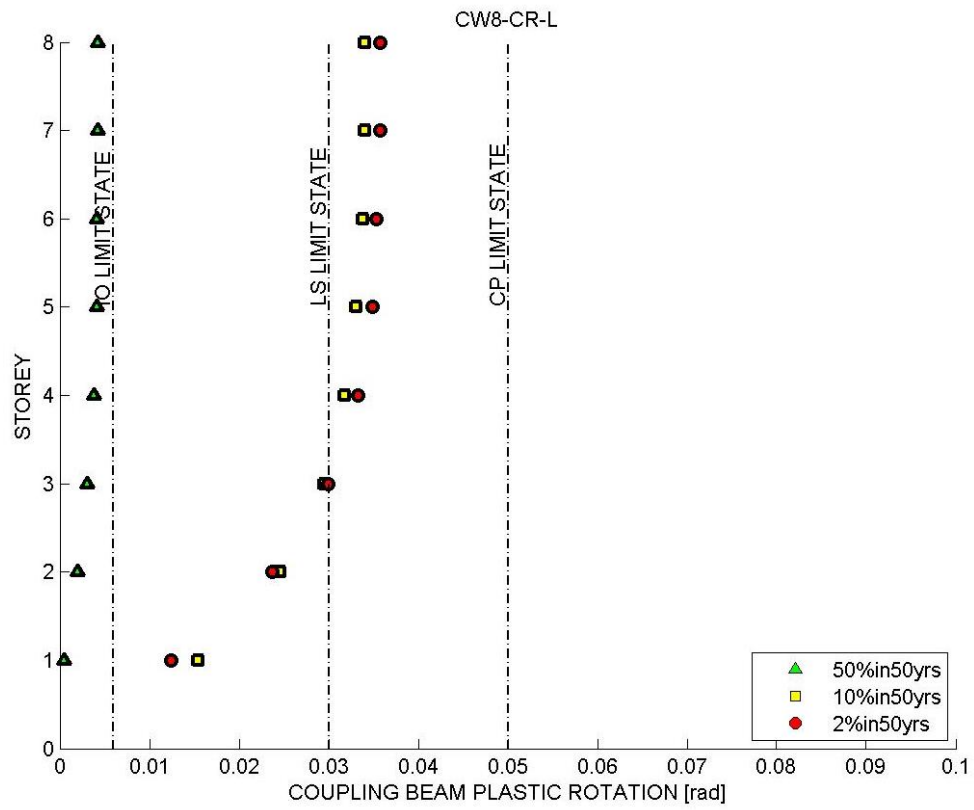


Figure 6.40 : Plastic beam rotations – CW8-CR-L.

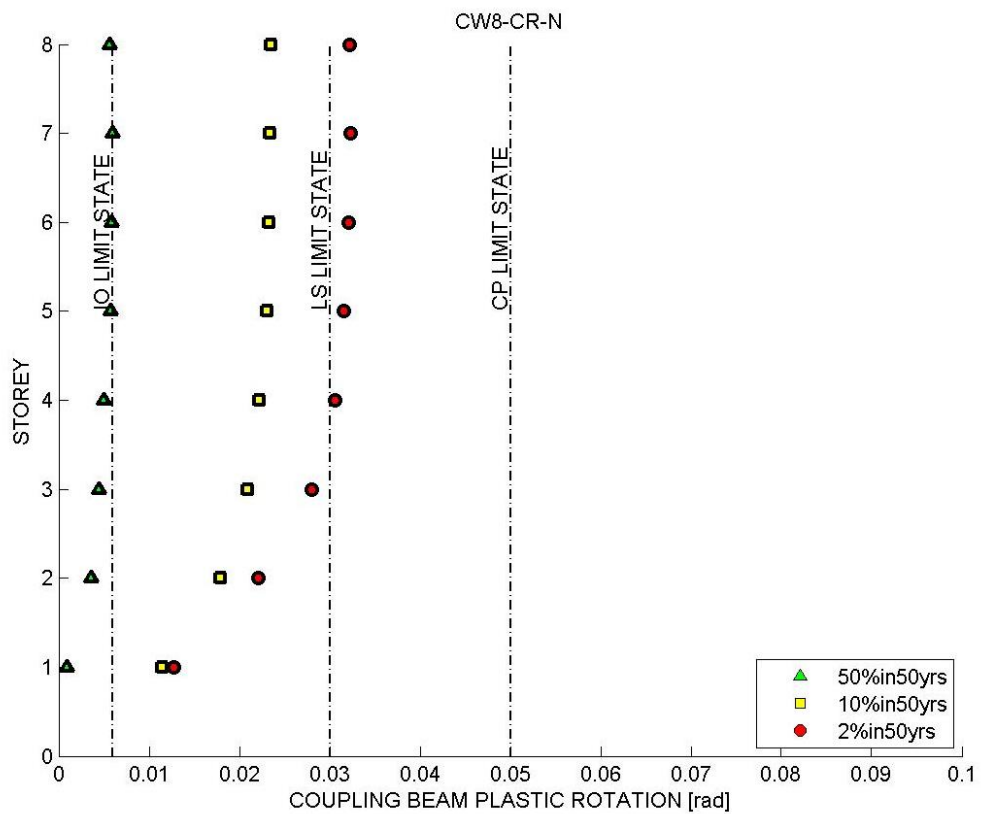


Figure 6.41 : Plastic beam rotations – CW8-CR-N.

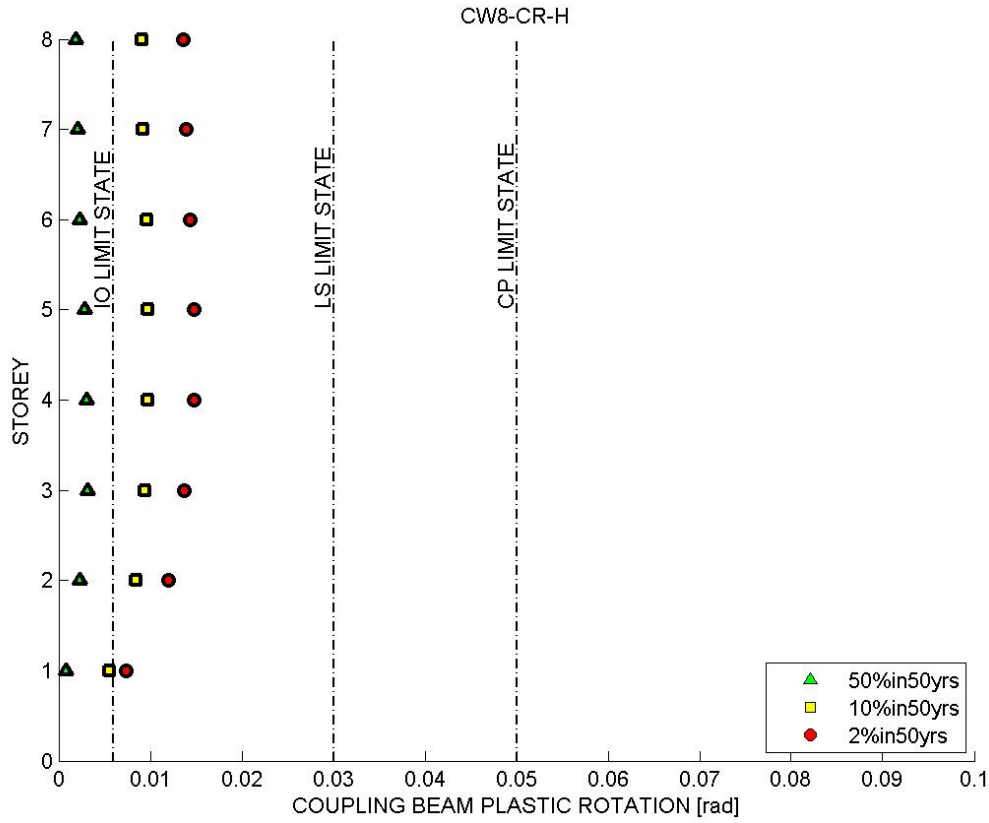


Figure 6.42 : Plastic beam rotations – CW8-CR-H.

It is observed that the 8-story building with low initial coupling ratio (CW08-CR-L) cannot satisfy the Life Safety (LS) performance level since the beams above 4th floor exceed the 0.030rad plastic rotation limit for 10% in 50 years (Figure 6.40). On the other hand, the 8-story building with initial coupling ratio at normal rate (CW08-CR-N) satisfies all performance levels for corresponding earthquake levels (Figure 6.41). The beams of the building with higher coupling ratio (CW08-CR-H) seem not to exhibit much plastic deformations as anticipated, satisfying all performance levels with even much lower than Life Safety limit state (Figure 6.42), which means, the system with higher coupling ratio (CW08-CR-H) has significantly lower ductility than the other two systems (CW08-CR-L and CW08-CR-N).

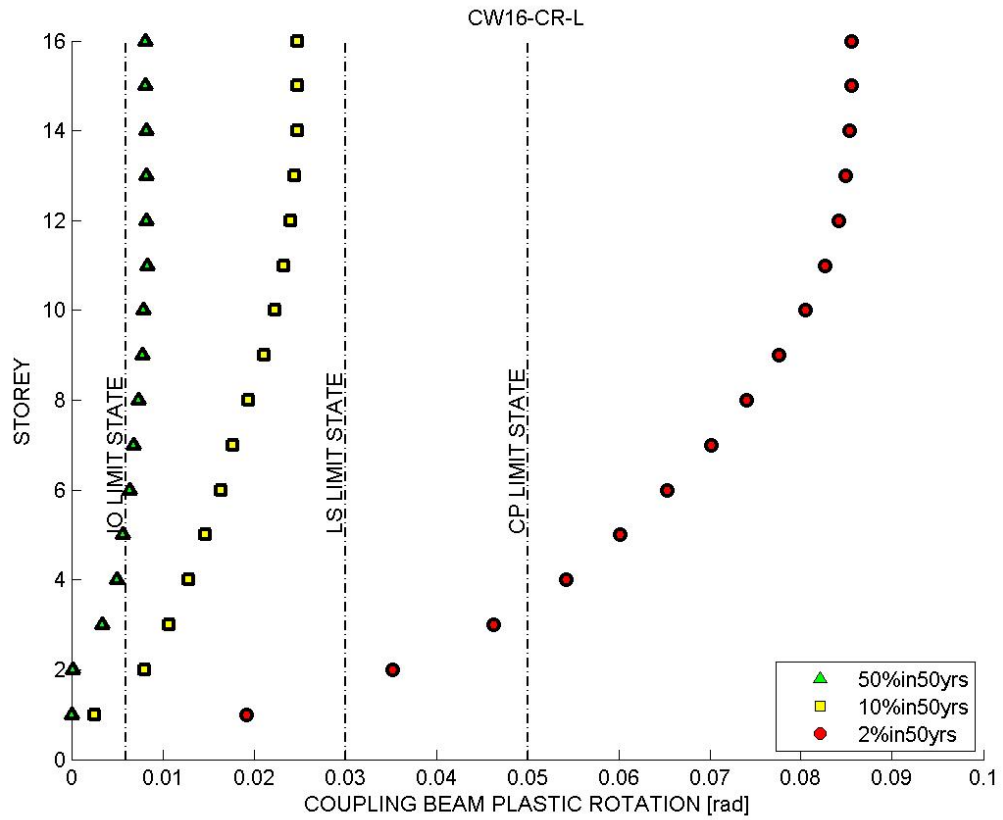


Figure 6.43 : Plastic beam rotations – CW16-CR-L.

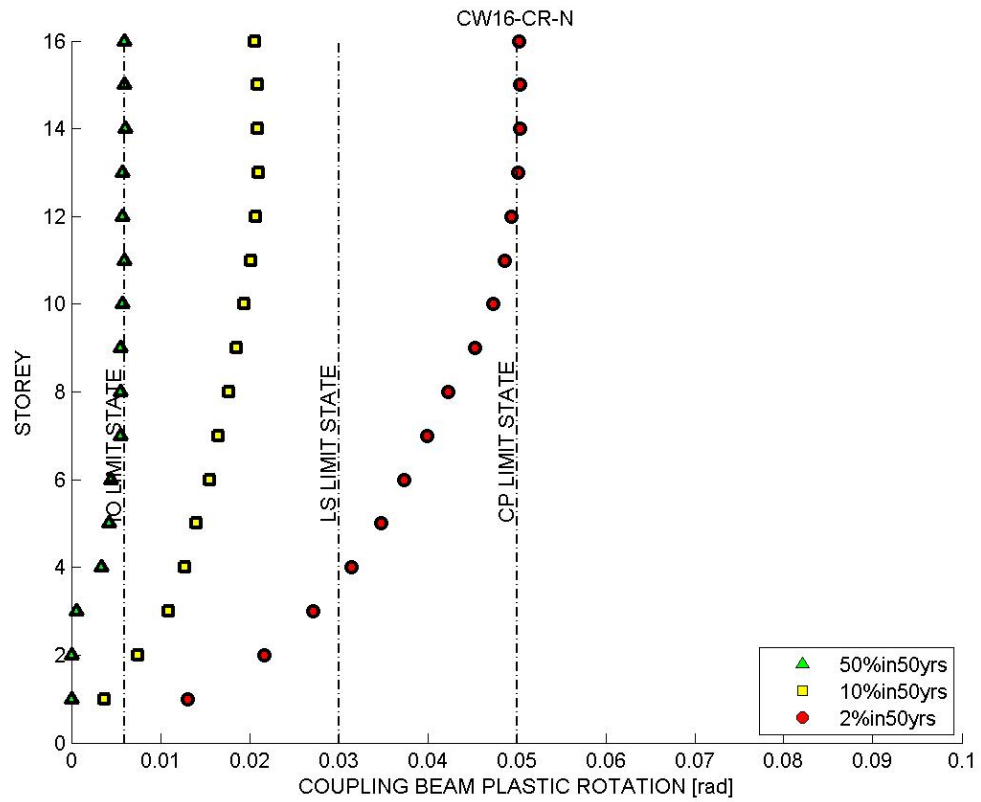


Figure 6.44 : Plastic beam rotations – CW16-CR-N.

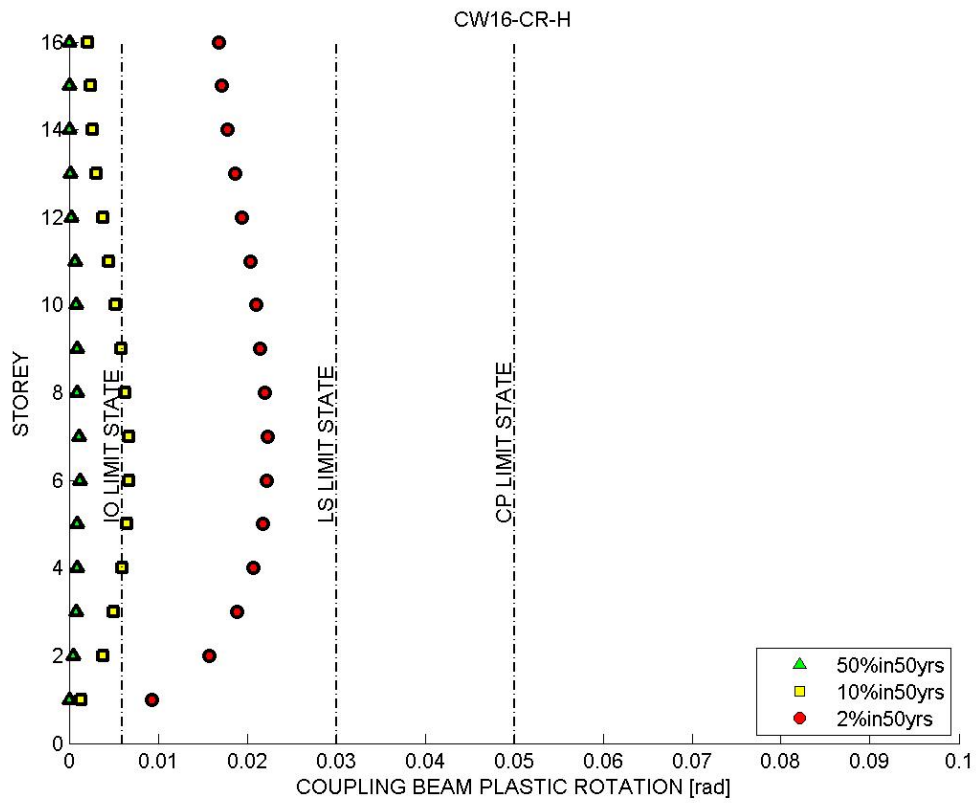


Figure 6.45 : Plastic beam rotations – CW16-CR-H.

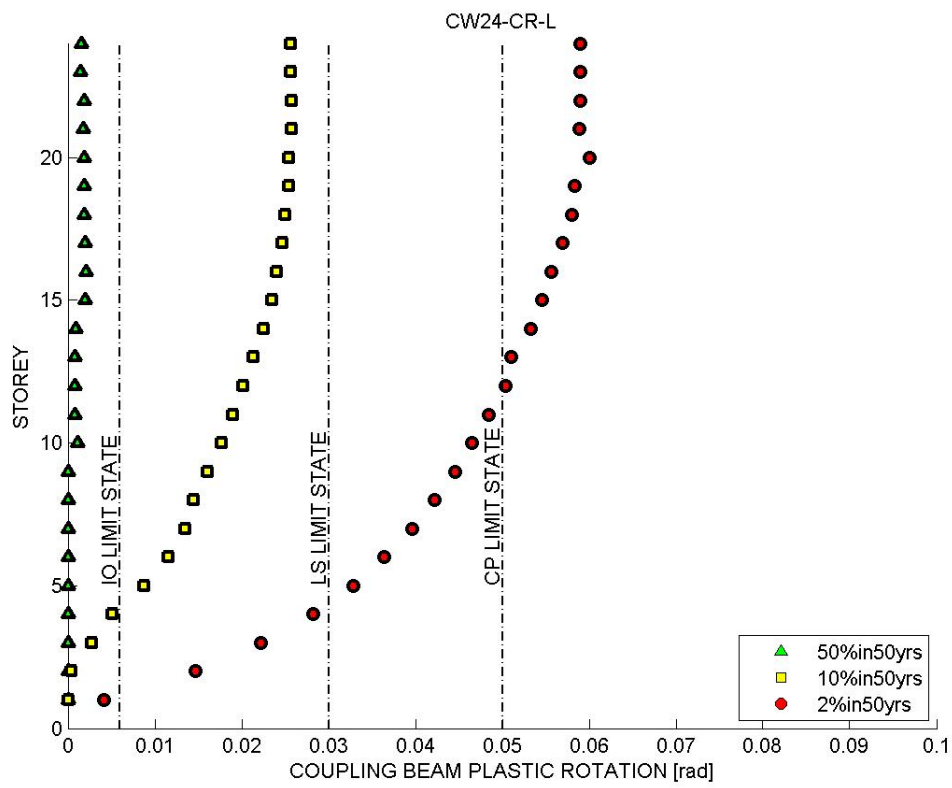


Figure 6.46 : Plastic beam rotations – CW24-CR-L.

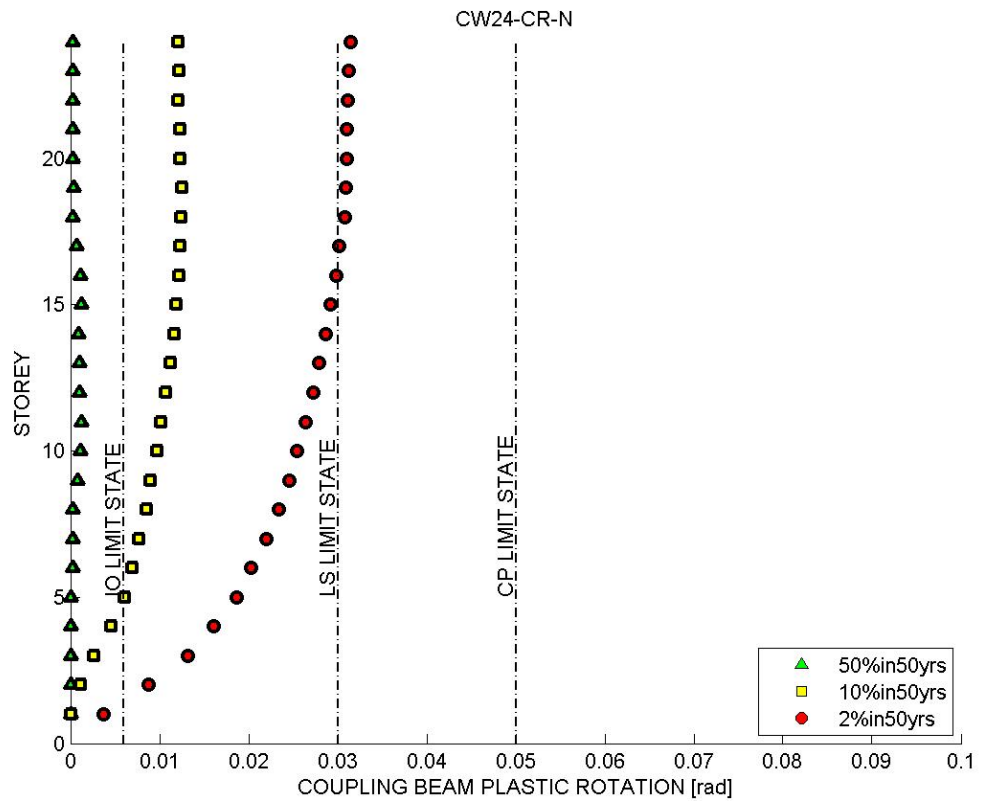


Figure 6.47 : Plastic beam rotations – CW24-CR-N.

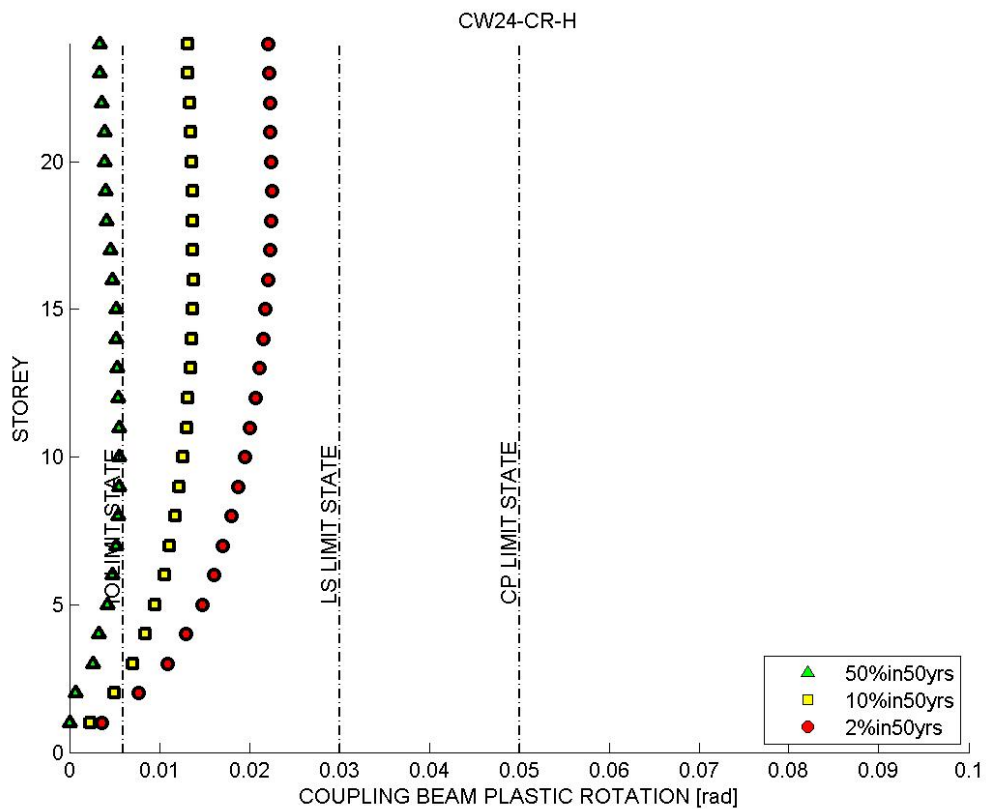


Figure 6.48 : Plastic beam rotations – CW24-CR-H.

Although the coupling beam plastic rotations in CW16-CR-L satisfy the Life Safety (LS) performance under design earthquake level, the structure cannot satisfy the Immediate Occupancy (IO) and Collapse Prevention (CP) performance requirements under service level earthquake and maximum considered earthquake, respectively. The system with normal coupling degree (CW16-CR-N) satisfies all performance levels for corresponding earthquake levels. Again, the beams of the building with higher coupling ratio (CW16-CR-H) seem not to exhibit as much plastic deformation as anticipated. The performance results for 24-story coupled wall systems seem to be similar with 16-story buildings. Coupling beam performance result summary is tabulated in Table 6.17.

Table 6.17 : Coupling beam performance results.

	CW08			CW16			CW24		
	CR-L	CR-N	CR-H	CR-L	CR-N	CR-H	CR-L	CR-N	CR-H
IO	✓	✓	✓	×	✓	✓	✓	✓	✓
LS	×	✓	✓*	✓	✓	✓*	✓	✓	✓
CP	✓	✓	✓*	×	✓	✓*	×	✓	✓*

* very low plastic rotation, over-design

6.6.4 Wall pier strains

The mathematical models of the wall elements in a coupled wall system consist of fiber elements as described in previous chapters. Those fibers mainly represent reinforcement steel and concrete behavior depending on the uniaxial stress values.

Wall strains have been calculated at each end of each wall pier and mean results are extracted for ground floors. Calculations are based on the vertical displacements of fiber ends and geometrical properties. Considering the deformed shape of a wall pier, as shown in Figure 6.49, strain in the left and right side of the wall was obtained as follows:

$$\text{left } \varepsilon_{ij} = (\Delta_i - \Delta_j)/H_w \quad (6.7)$$

$$\text{right } \varepsilon_{lk} = (\Delta_l - \Delta_k)/H_w \quad (6.8)$$

Wall shear force and wall axial strain transition during a representative design level earthquake (10% in 50 years) event is plotted for low, normal and high level coupled wall systems, respectively, as shown in Figure 6.50, Figure 6.51 and Figure 6.52.

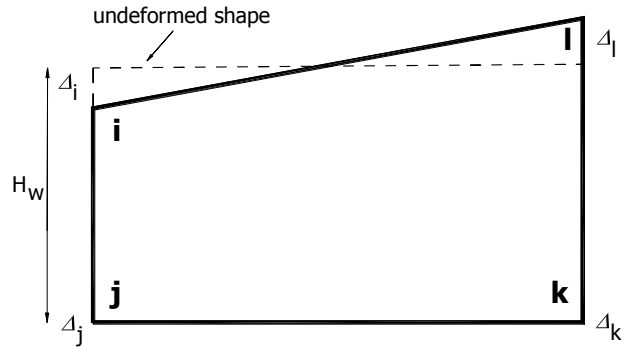


Figure 6.49 : Wall deformed shape.

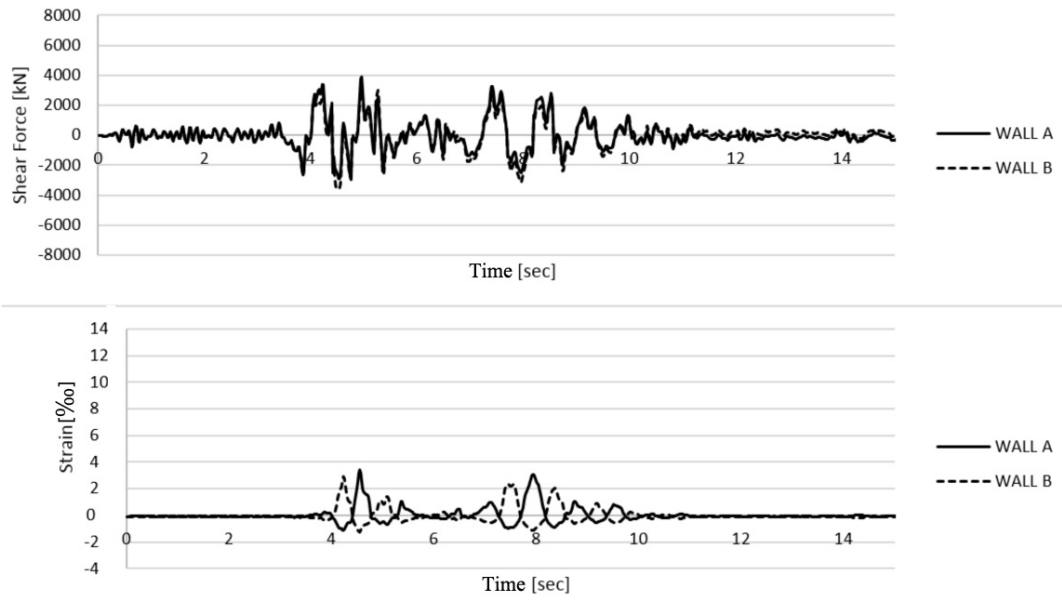


Figure 6.50 : CW08-CR-L shear force and axial strain history 10%in50years.

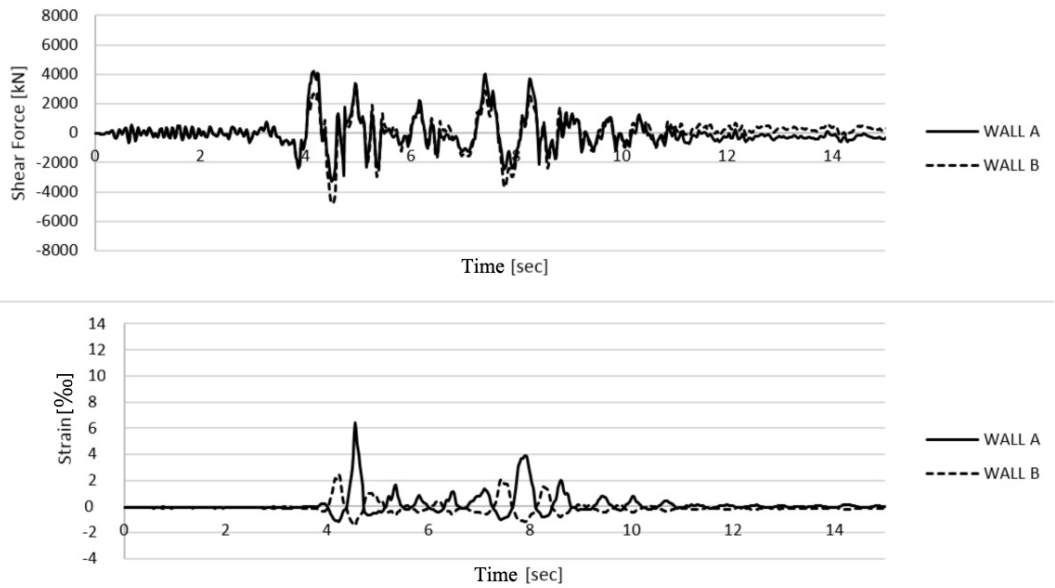


Figure 6.51 : CW08-CR-N shear force and axial strain history 10%in50years.

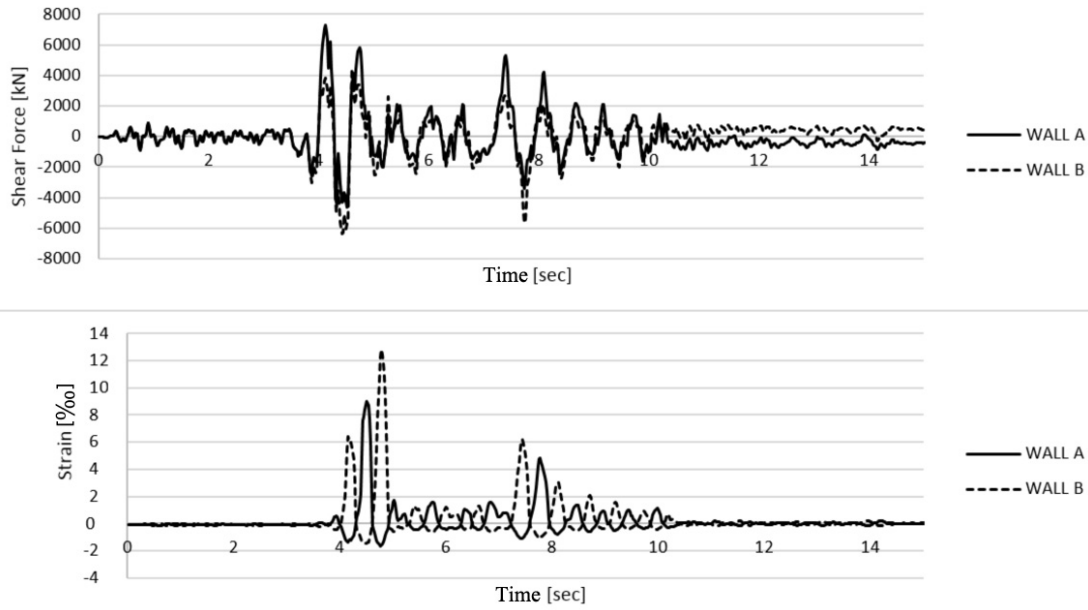


Figure 6.52 : CW08-CR-H shear force and axial strain history 10%in50years.

Although all wall system alternatives are approximately equal in mass values, their seismic demand forces are significantly different depending on their coupling ratios. Since a high-coupled wall system exhibits a stiffer behavior, consequently the base shear demand becomes larger compared to the systems with lower coupling ratios because of the lower periods. The change in the demand forces also affects the wall strain values. Figure 6.53, Figure 6.54 and Figure 6.55 represent the axial strain distribution of wall piers for different coupling degree alternatives at an instant when the overturning moment was maximum between time frame 4.40-4.60 seconds for 8 story case study buildings. The yield strain in the plots is estimated from the expected yield strength of the vertical steel reinforcement ($491\text{MPa}/200000\text{MPa}=0.0025$).

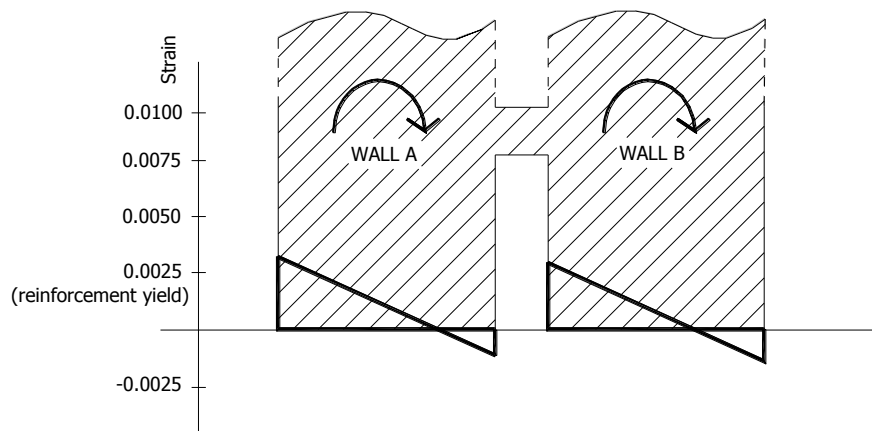


Figure 6.53 : CW08-CR-L wall strain distribution 10%in50years record no.1.

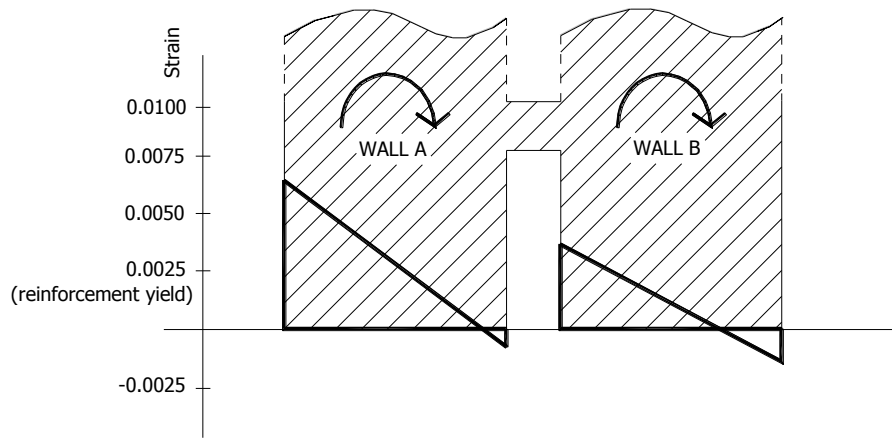


Figure 6.54 : CW08-CR-N wall strain distribution 10%in50years record no.1.

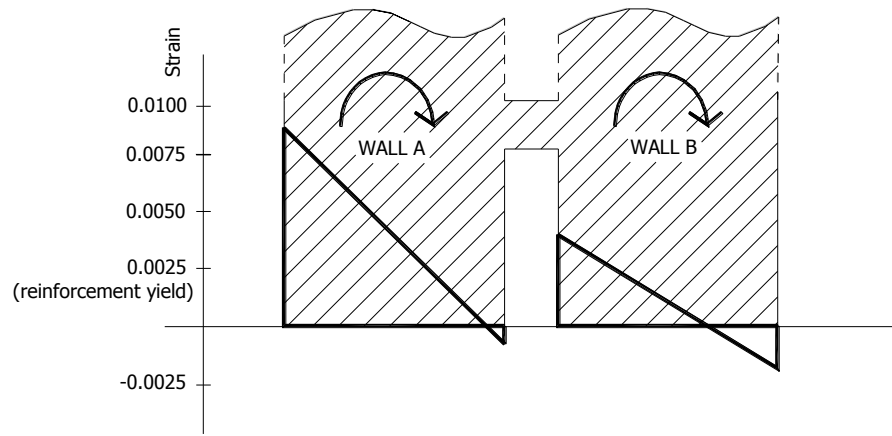


Figure 6.55 : CW08-CR-H wall strain distribution 10%in50years record no.1.

Assuming that the earthquake direction is from East to West, in all three cases, Wall A was the pier under tension force coming from coupling beams. It is noted that the strain distribution between Wall A and Wall B was not significantly different when the coupling degree is low (CR08-CR-L). On the other hand, the extreme axial force changes and the higher lateral demand forces in CR08-CR-H system caused Wall A to experience significantly larger tension strain values relative to Wall B as shown in Figure 6.55.

The Turkish Earthquake Code (TEC, 2007) specifies the strain limit values depending on the performance targets as summarized in Table 6.18.

The mean of the maximum strain values extracted from 20 analyses per each earthquake levels for all case study buildings have been presented in Table 6.19, Table 6.20 and Table 6.21. It was observed that only the high-coupled systems slightly exceed the Immediate Occupancy performance level limits under 50% in 50 years earthquake levels.

Table 6.18 : Strain limit values for performance levels (TEC, 2007).

	Immediate Occupancy	Life Safety	Collapse Prevention
Maximum Tension Strain in Steel Reinforcement	0.010	0.040	0.060
Maximum Compression Strain in Concrete	0.0035	0.0135	0.018

Table 6.19 : Tension and compression strains (50% in 50 years).

	50% in 50 years (mean of 20 analyses)	
	Max. Tension Strain in Steel	Min. Compression Strain in Concrete
CW08-CR-L	0.00277	-0.00101
CW08-CR-N	0.00516	-0.00118
CW08-CR-H	0.01029	-0.00132
CW16-CR-L	0.00300	-0.00127
CW16-CR-N	0.00540	-0.00145
CW16-CR-H	0.01053	-0.00158
CW24-CR-L	0.00323	-0.00154
CW24-CR-N	0.00563	-0.00171
CW24-CR-H	0.01076	-0.00185

Table 6.20 : Tension and compression strains (10% in 50 years).

	10% in 50 years (mean of 20 analyses)	
	Max. Tension Strain in Steel	Min. Compression Strain in Concrete
CW08-CR-L	0.00344	-0.00125
CW08-CR-N	0.00642	-0.00147
CW08-CR-H	0.01280	-0.00164
CW16-CR-L	0.00373	-0.00158
CW16-CR-N	0.00671	-0.00180
CW16-CR-H	0.01309	-0.00197
CW24-CR-L	0.00402	-0.00191
CW24-CR-N	0.00700	-0.00213
CW24-CR-H	0.01338	-0.00230

Although most of the wall systems satisfy the strain performance limits in principle, they all experienced steel reinforcement yield at strain value of 0.0025. This yielding event limits the amount of bending moment carried by the walls, and hence the shear force being sustained by the tension wall. Therefore, the amount of exceeding demand force is then transferred to the compression wall. This phenomenon is called shear migration and will be explained in more detail in the following sections.

Table 6.21 : Tension and compression strains (2% in 50 years).

	2% in 50 years (mean of 20 analyses)	
	Max. Tension Strain in Steel	Min. Compression Strain in Concrete
CW08-CR-L	0.00459	-0.00167
CW08-CR-N	0.00856	-0.00196
CW08-CR-H	0.01708	-0.00219
CW16-CR-L	0.00498	-0.00211
CW16-CR-N	0.00895	-0.00240
CW16-CR-H	0.01746	-0.00263
CW24-CR-L	0.00536	-0.00255
CW24-CR-N	0.00934	-0.00284
CW24-CR-H	0.01785	-0.00307

6.6.5 Wall shear design verification

As it has already been discussed in previous sections, the over-strength coefficients increases the design shear forces of coupled wall piers significantly; however, Turkish Earthquake Code (TEC, 2007) demand upper boundary limits the design force with demands under $R_a=2$. As it is observed in Table 6.22, the nonlinear time history analysis (NLTHA) demand shear values do not exceed the existing capacities with expected material properties.

Table 6.22 : Linear and nonlinear analysis shear force results.

	Linear Analysis $R_a=6$ [kN]	Over-strength	Code Dynamic Amplification	Design Shear $R_a=2$ upper limit considered [kN]	Max. Allowed Shear $(0.83 A_c (f_c')^{0.5})$ [kN]	NLTHA Wall Shear [kN]
CW08-CR-L	1210	2.42	1.5	3630	12599	4059
CW08-CR-N	1506	2.71	1.5	4518	12599	5330
CW08-CR-H	1982	3.14	1.5	5946	12599	7443
CW16-CR-L	1265	2.25	1.5	3795	18898	3393
CW16-CR-N	1356	2.7	1.5	4068	18898	5284
CW16-CR-H	1818	2.91	1.5	5454	18898	5595
CW24-CR-L	1594	2.22	1.5	4782	29397	4002
CW24-CR-N	1830	2.57	1.5	5490	29397	5296
CW24-CR-H	2174	2.81	1.5	6522	29397	7262

6.6.6 Moment distribution and shear migration phenomena

As explained in Chapter 4, the bending moment demands between two identical wall piers in a coupled wall system in nonlinear analysis differ than in the elastic analysis case. The shear force in coupling beams are transferred to walls as tension force on

one pier while as compression force to the other pier. Figure 6.56 represents only the first 10 seconds of axial force changes on piers on CW08-CR-N. The difference between each wall pier's axial forces is almost symmetrical due to the equilibrium in vertical forces. The difference in axial forces reaches its peak value at $t=4.54$ sec, when the moment capacity disparity between each wall also reaches its maximum. This phenomenon also generally occurs during the same time period when the overturning moment on the system gets its maximum value. While the wall with lower axial force (Wall A in this case) reached its bending moment capacity of 35188 kNm, the compression wall (Wall B in this case) could exceed up to 62120 kNm due to higher axial forces (Figure 6.57). The moment demand ratio between each wall for this ground motion is 1.77.

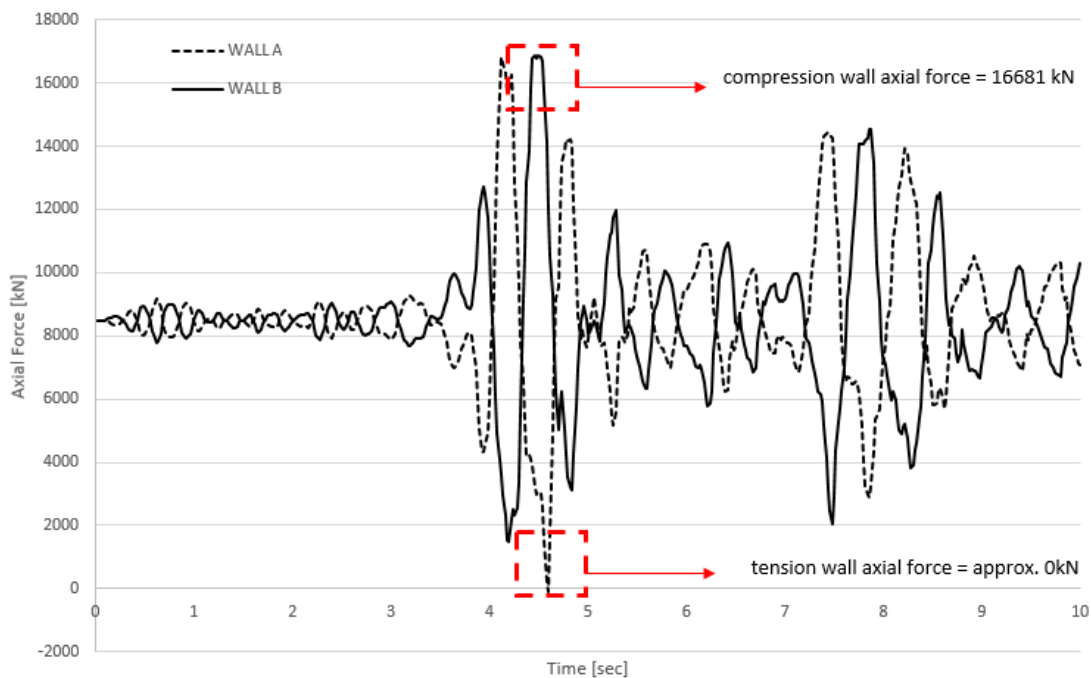


Figure 6.56 : Wall axial force vs. time for CW08-CR-N.

The distribution of the moments between each wall also will affect the distribution of shear force demand, obviously. In that case, the compression wall drags the exceeding shear force that the tension wall cannot bear. On the other hand, the shear force ratio between the walls is limited to 1.28 (Figure 6.58). The main reason of a lower value in the shear ratio is the dynamic shear amplification in the tension wall. When plastic hinge at the bottom of the tensile wall occurs prior to compression wall, the higher mode effects cause tensile wall having a lower M/V ratio where, M and V are the wall moment and shear demands, respectively.

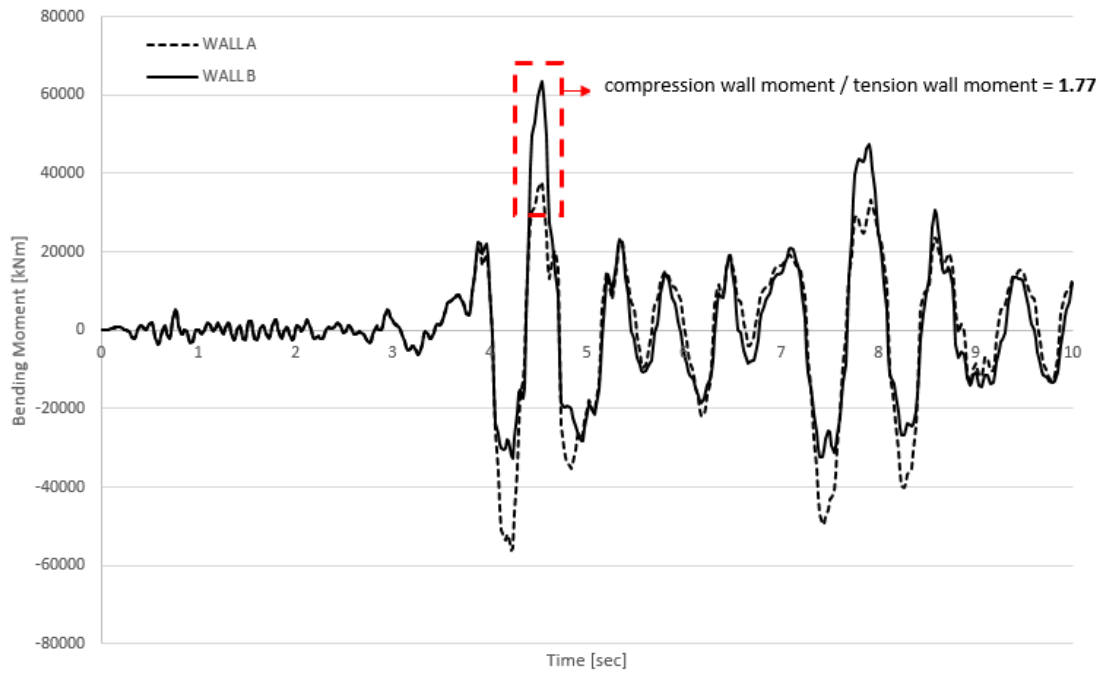


Figure 6.57 : Wall bending moment vs. time for CW08-CR-N.

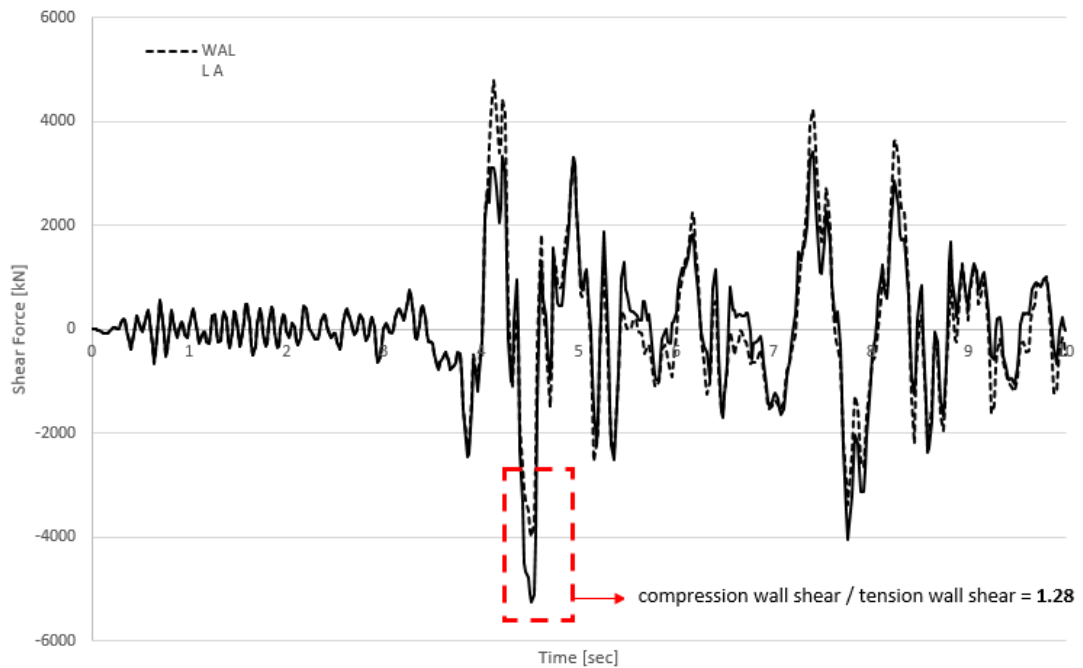


Figure 6.58 : Wall shear force vs. time.

Depending on the contribution of coupling beams, the difference in the pier axial force may vary. For instance, an increase in the coupling ratio also increases the disparity of the force demands between wall piers. Figure 6.59 shows the history of each pier's bending moment demands with two different coupling degrees. Initially, the line is linear thus follows 45 degrees until both wall piers maintain their elastic behavior

under bending. As plastic hinge occurs at the bottom of the tension wall, the bending moment demand at compression wall exceeds the tension wall. When both walls are under positive bending moment, Wall B is in the role of compression wall and drags larger demand forces comparing to the tension Wall A. Under negative bending moment, walls change their roles and Wall A starts to bear larger bending moments as a compression wall. The intensity of this difference increases with higher coupling ratio in coupled wall system.

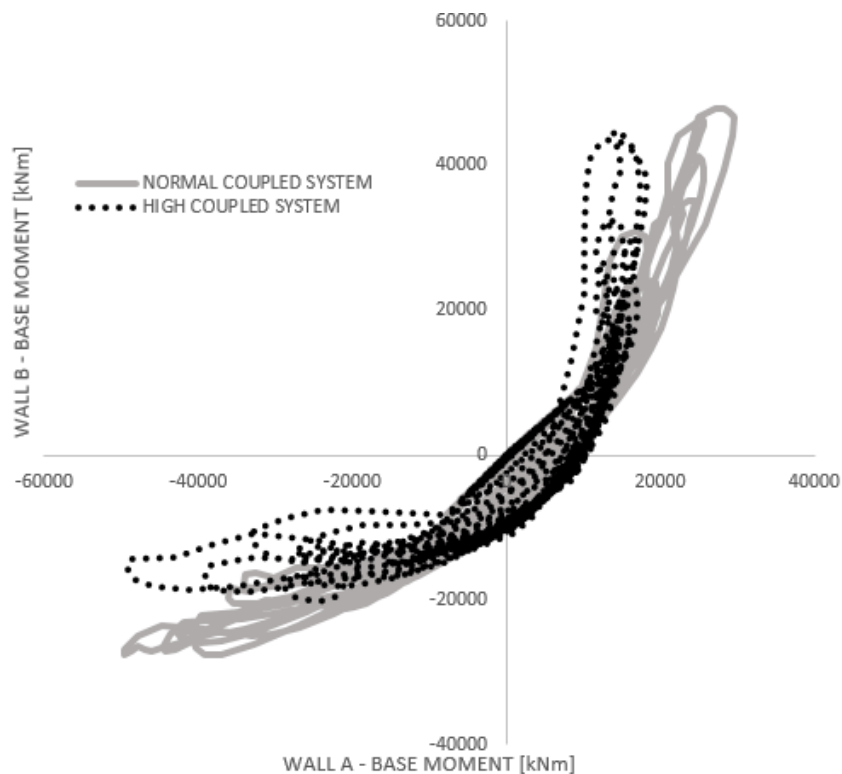


Figure 6.59 : Wall A bending moment vs. Wall B bending moment.

After the initial plastic hinge at tension wall occurs, the plot in Figure 6.59 slightly shifts away from the origin. Although there is not any external lateral load, piers still maintain residual moment forces, one has negative and another has positive. Residual force occurs because walls generally have plastic rotations only from one direction: clockwise for Wall A and counter-clockwise for Wall B. This opposite direction plastic deformations make walls lean to each other causing compression forces in the coupling beams, consequently bending moment at wall pier bottom.

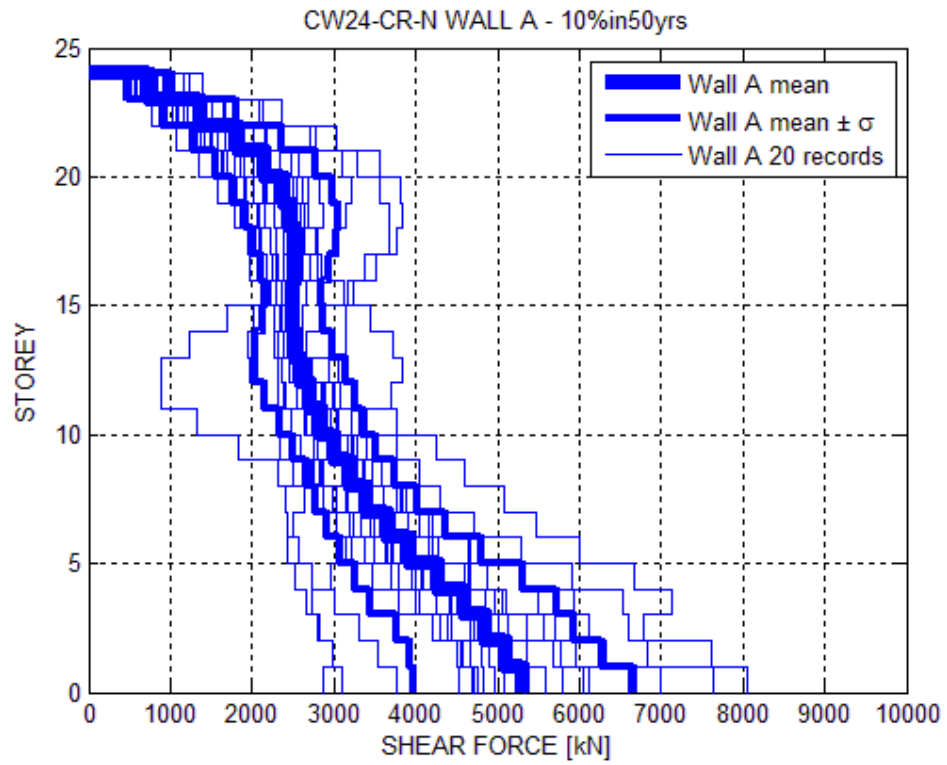


Figure 6.60 : CW24-CR-N Wall A mean of maximum story shear forces.

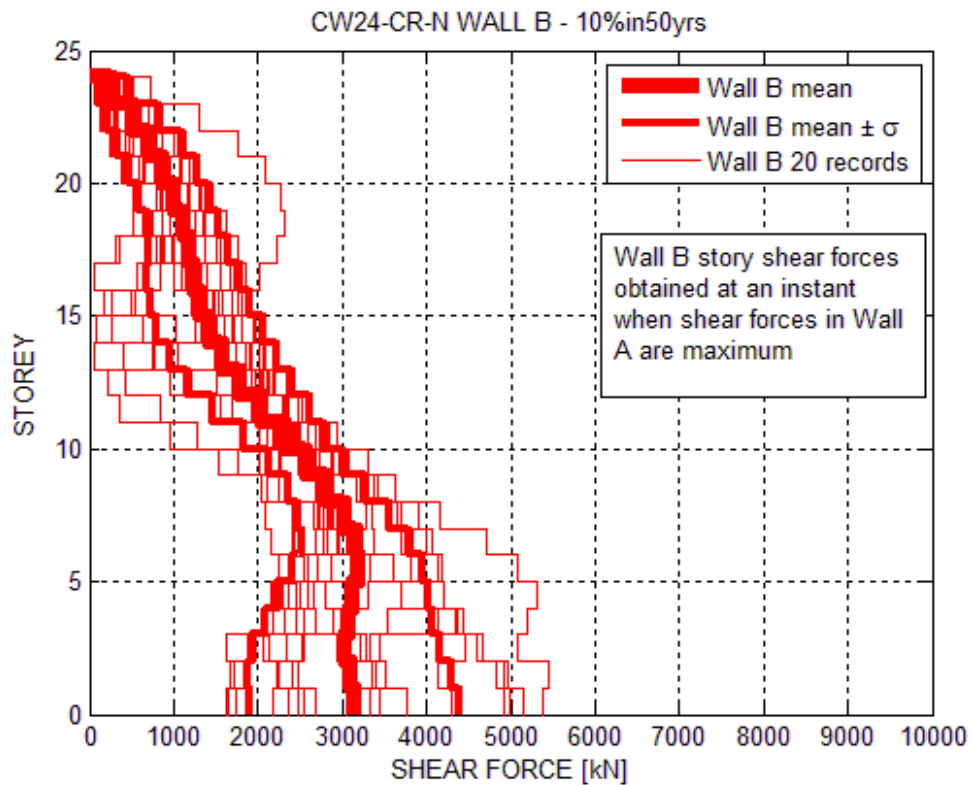


Figure 6.61 : CW24-CR-N Wall B corresponding story shear forces.

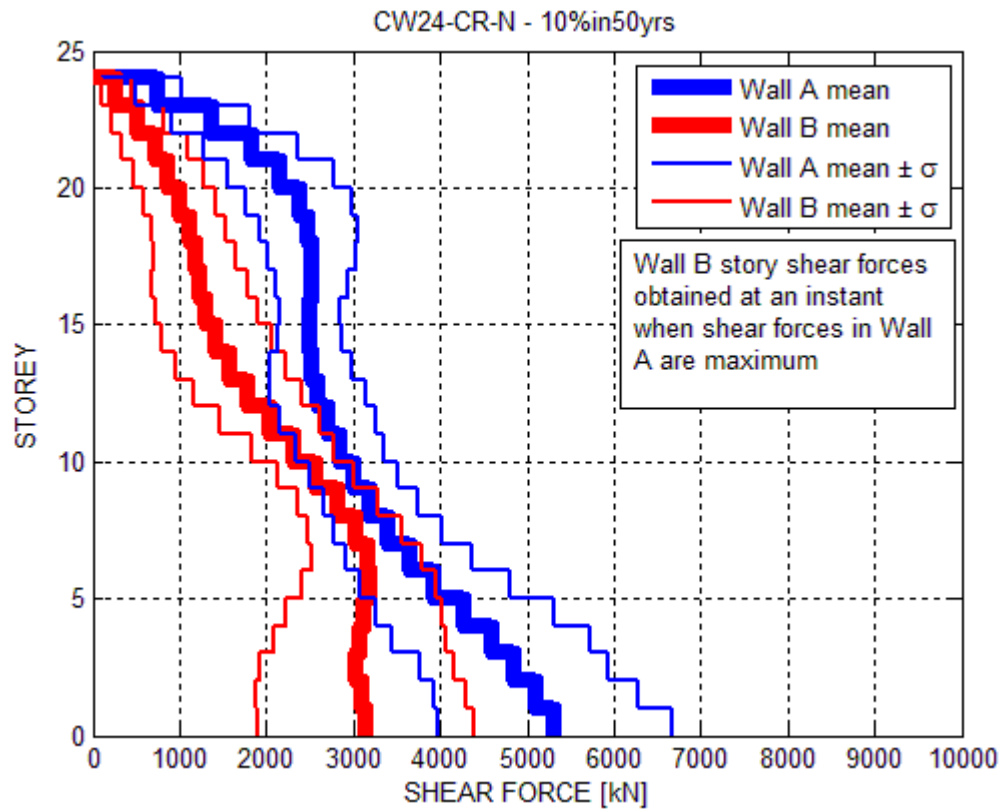


Figure 6.62 : CW24-CR-N wall piers story shear comparison.

In each analysis case, the ratio between the compression wall moment over the tension wall moment is obtained at an instant where the maximum overturning moment acts on the system occurs. The graphs illustrated from Figure 6.63 to Figure 6.71 shows the extracted ratio values for each wall system's initial coupling ratio. Averages of each 20 record analysis results are obtained to observe the trend line for varied initial coupling ratio values.

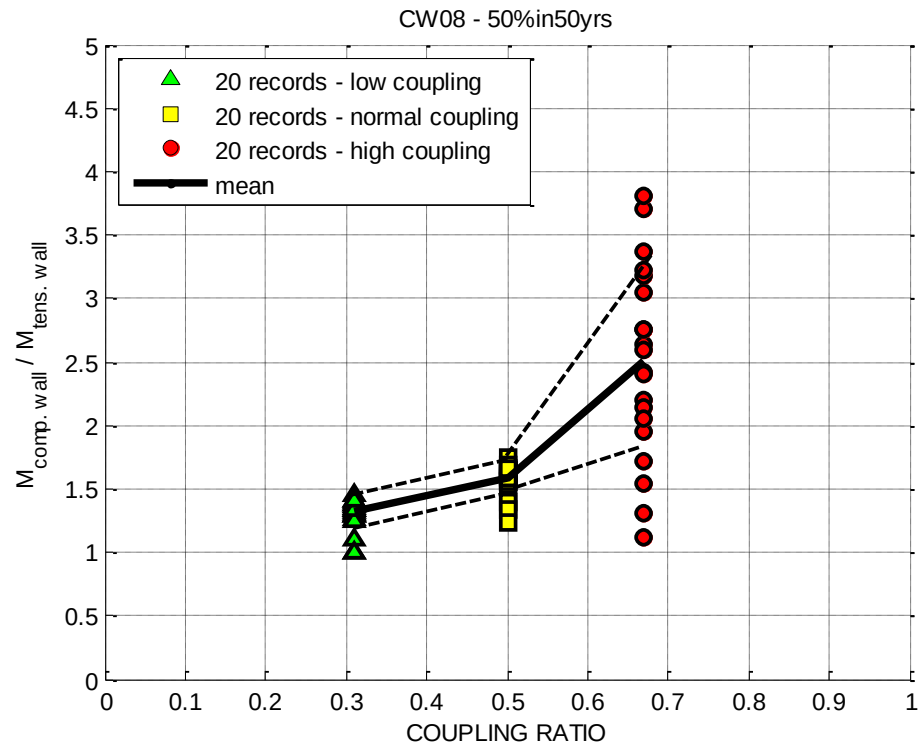


Figure 6.63 : Moment ratio between walls vs. CR – CW08 – 50% in 50 years.

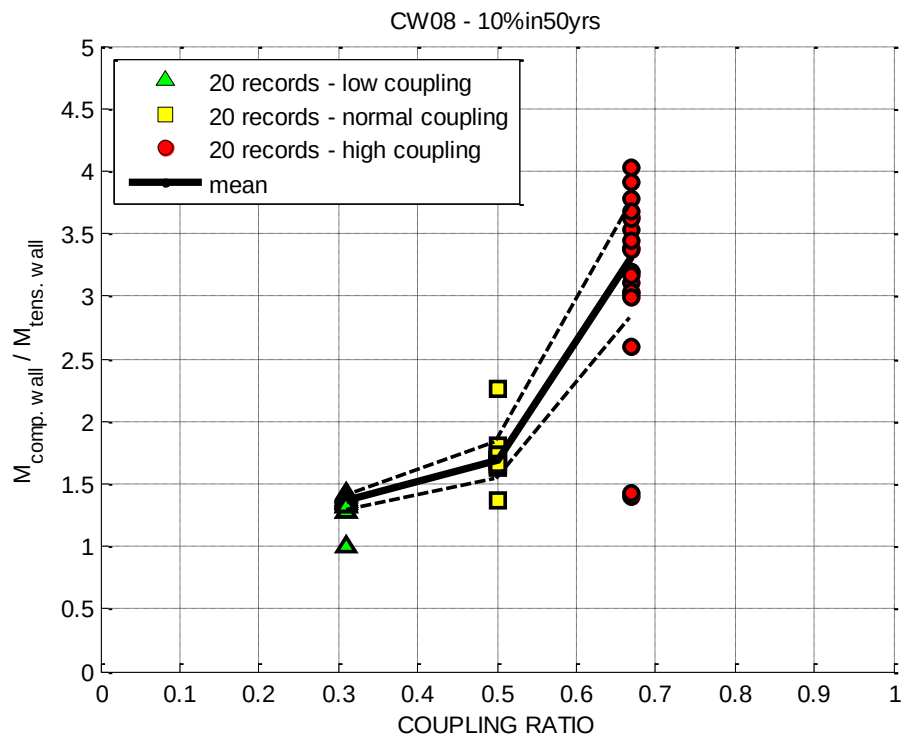


Figure 6.64 : Moment ratio between walls vs. CR – CW08 – 10% in 50 years.

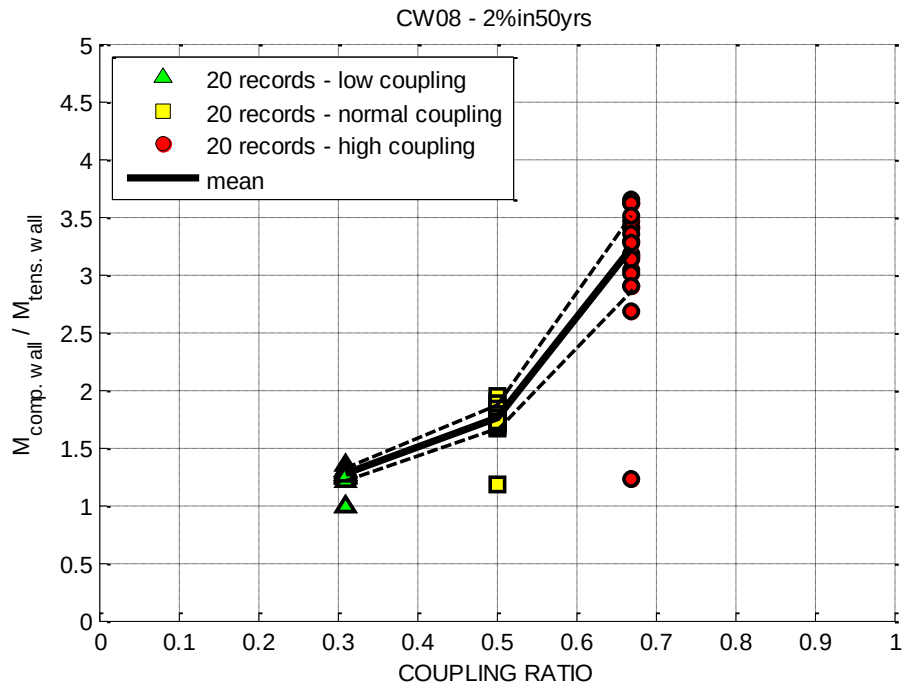


Figure 6.65 : Moment ratio between walls vs. CR – CW08 – 2% in 50 years.

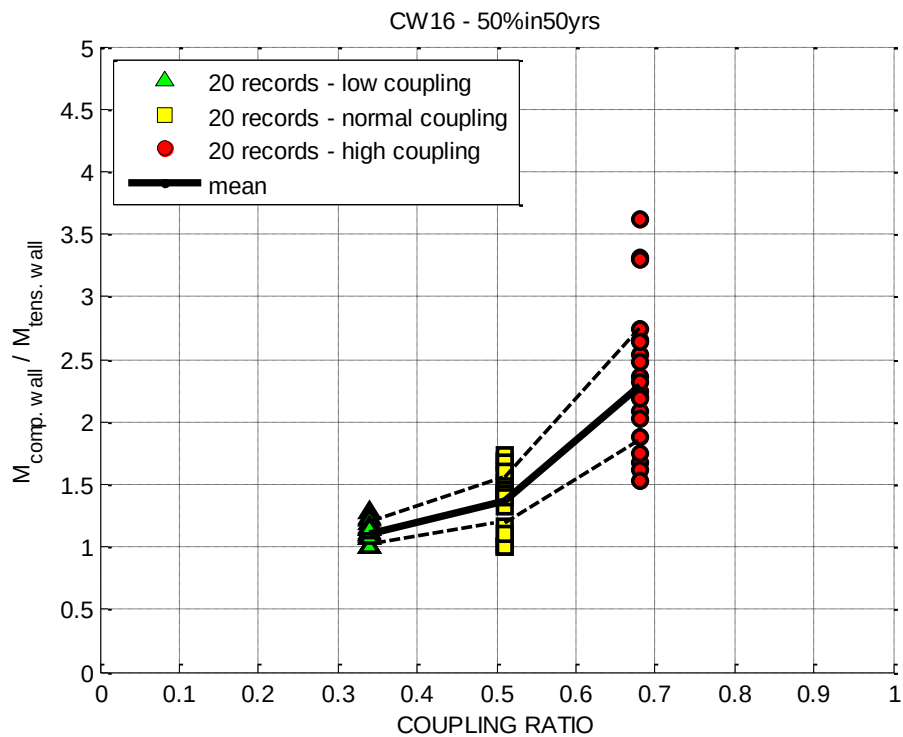


Figure 6.66 : Moment ratio between walls vs. CR – CW16 – 50% in 50 years.

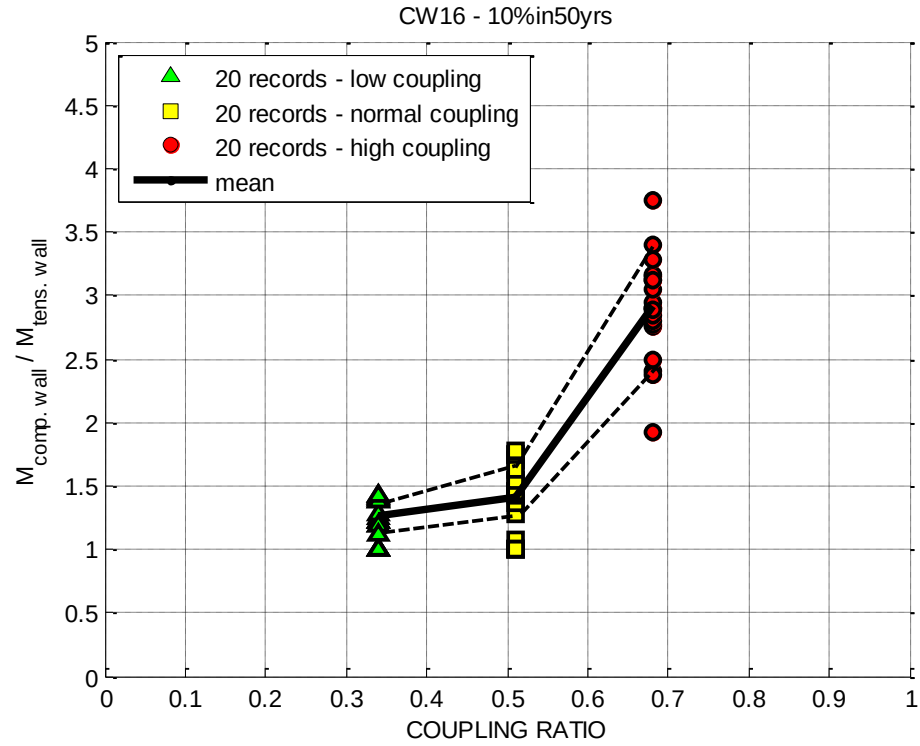


Figure 6.67 : Moment ratio between walls vs. CR – CW16 – 10% in 50 years.

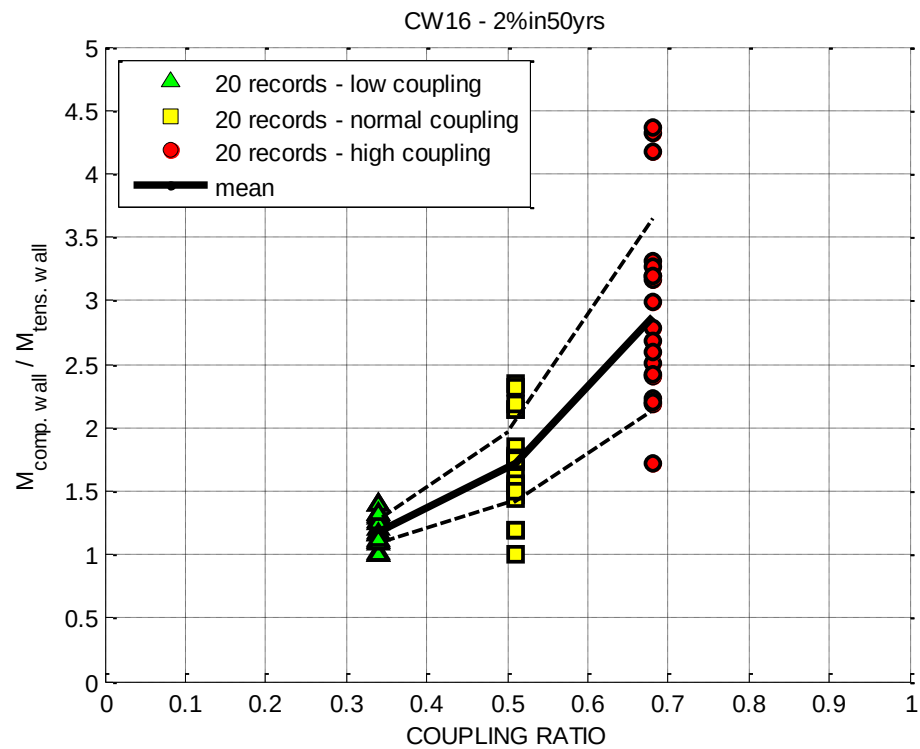


Figure 6.68 : Moment ratio between walls vs. CR – CW16 – 2% in 50 years.

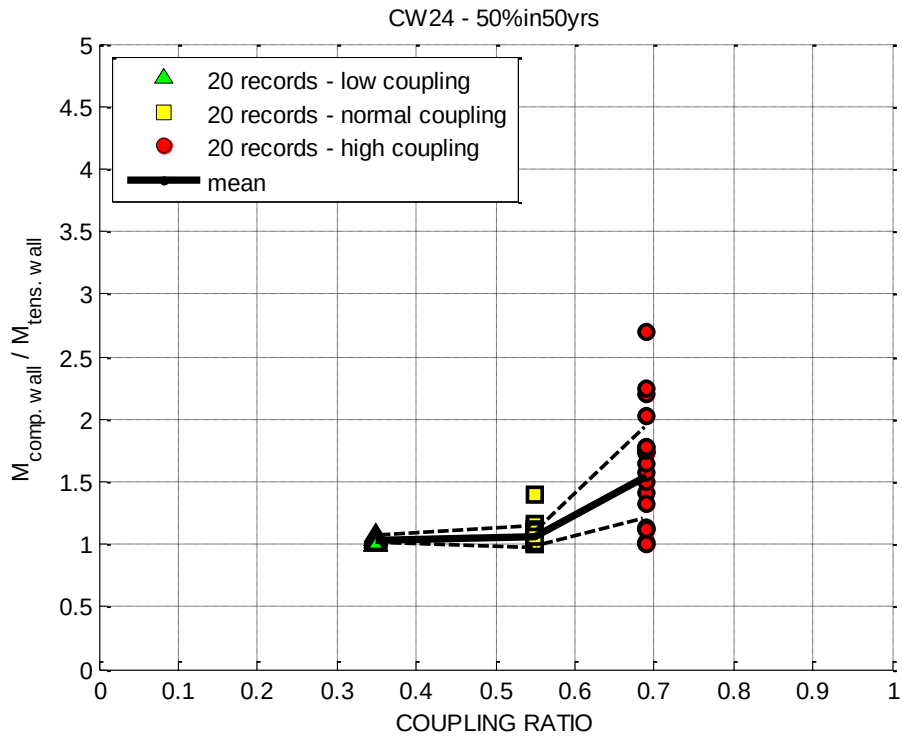


Figure 6.69 : Moment ratio between walls vs. CR – CW24 – 50% in 50 years.

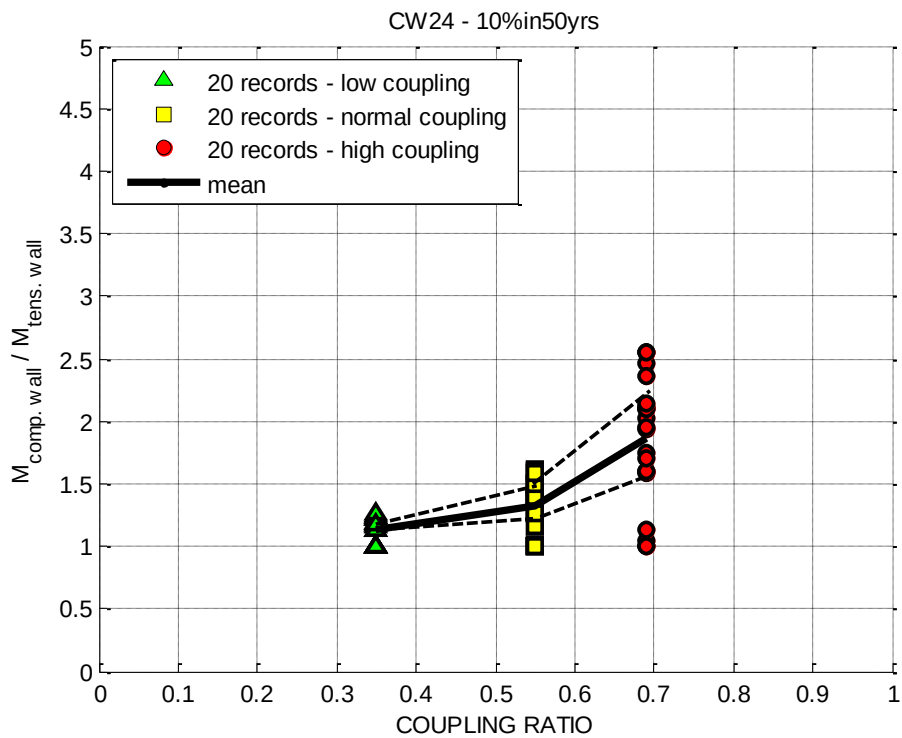


Figure 6.70 : Moment ratio between walls vs. CR – CW24 – 10% in 50 years.

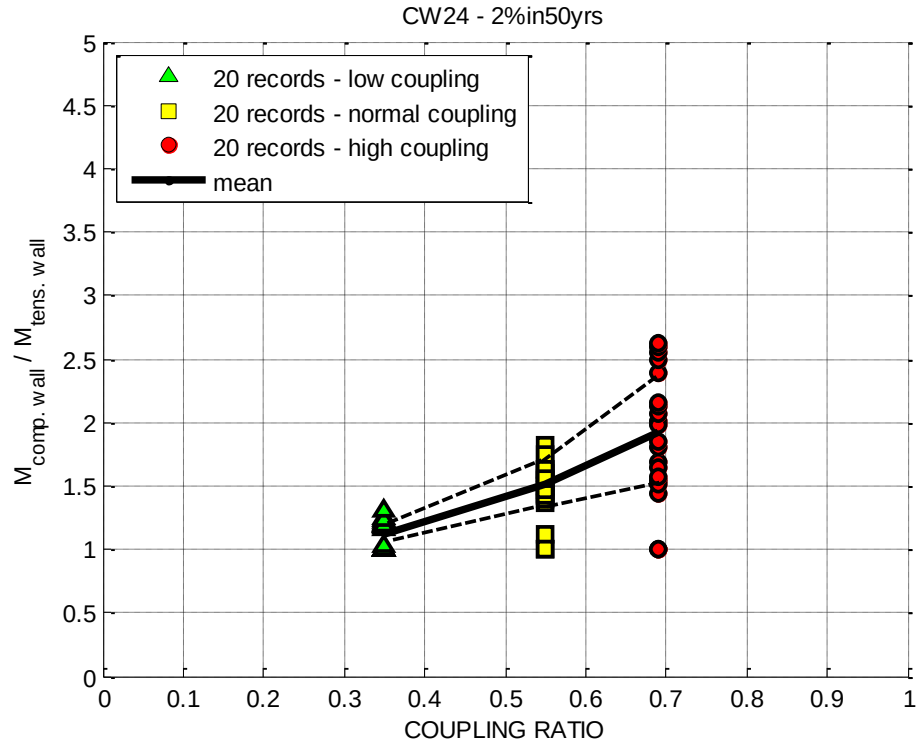


Figure 6.71 : Moment ratio between walls vs. CR – CW24 – 2% in 50 years.

In all cases, the moment demand ratio between compression wall and tension wall increase as the initial coupling ratio increases. This increment becomes significant on wall systems with high coupling (CW08-CR-H, CW16-CR-H, CW24-CR-H). Under design level earthquake with probability of exceedance of 10% in 50 years, the ratio on CW08-CR-H is 3.3, while it is approximately 1.8 on 24 story CW24-CR-H wall system, which means as the system gets higher this ratio decreases.

It has already been explained in Chapter 5 that the distribution of wall demand moments forces between identical wall piers would be assumed as incorrectly the same if the analysis is linear. To overcome this issue, Paulay (1981) recommended making a 30% redistribution of moment demands between each piers to take account the actual moment disparities which occur during the nonlinear analyses. Based on the nonlinear time history analysis results extracted above, the amount of percentage for the redistribution were refined depending on the coupling ratio and the number of story. These three-dimensional surface plots below summarize the interpolation for each earthquake levels (Figure 6.72, Figure 6.74 and Figure 6.76).

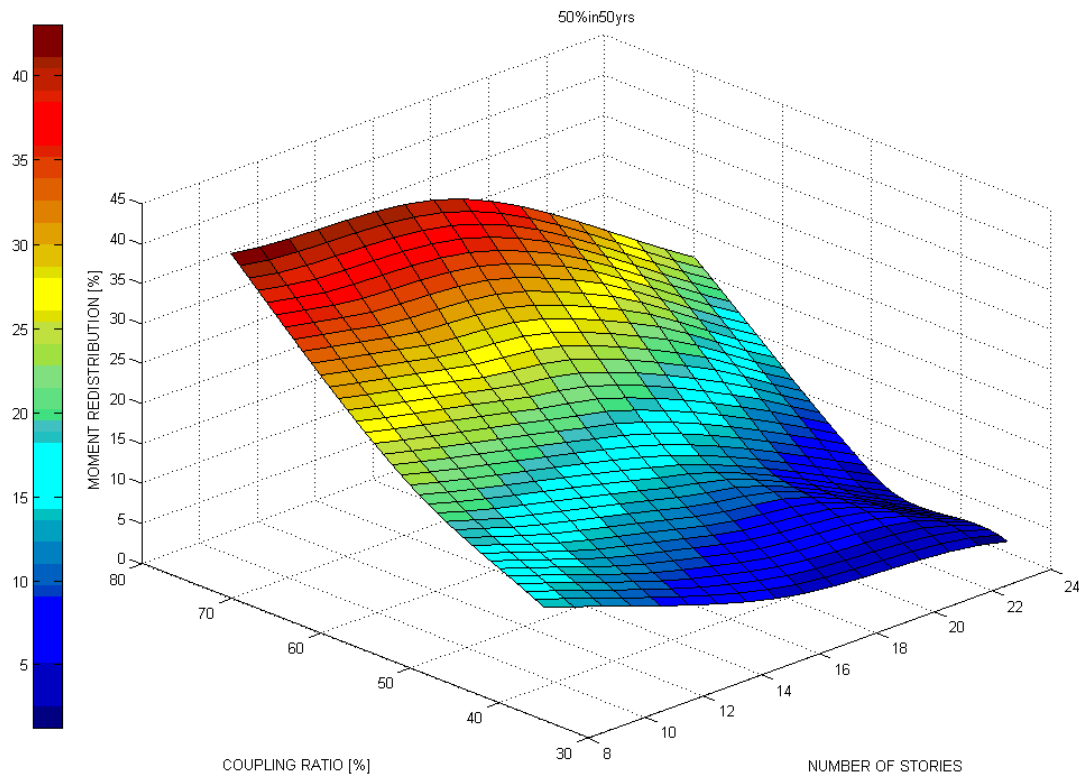


Figure 6.72 : Moment redistribution 50% in 50 years.

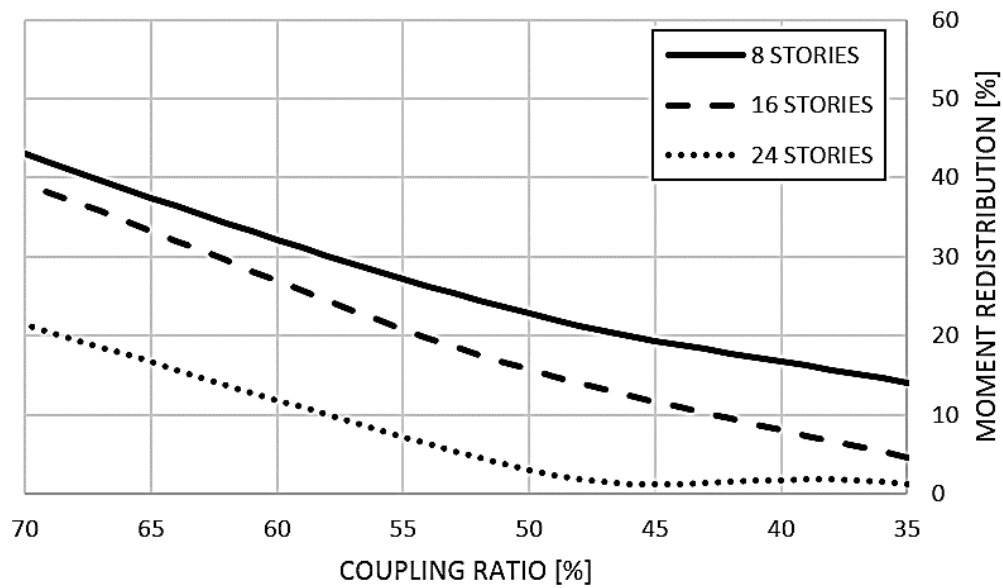


Figure 6.73 : Moment redistribution vs. coupling ratio 50% in 50 years.

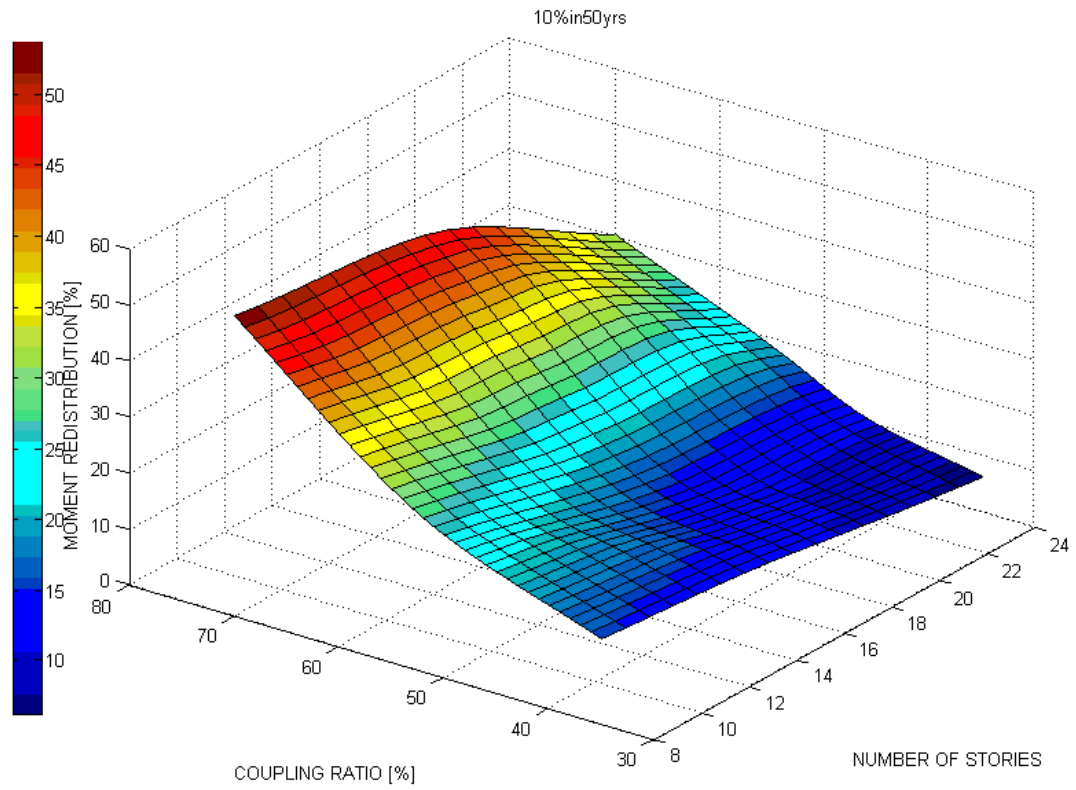


Figure 6.74 : Moment redistribution 10% in 50 years.

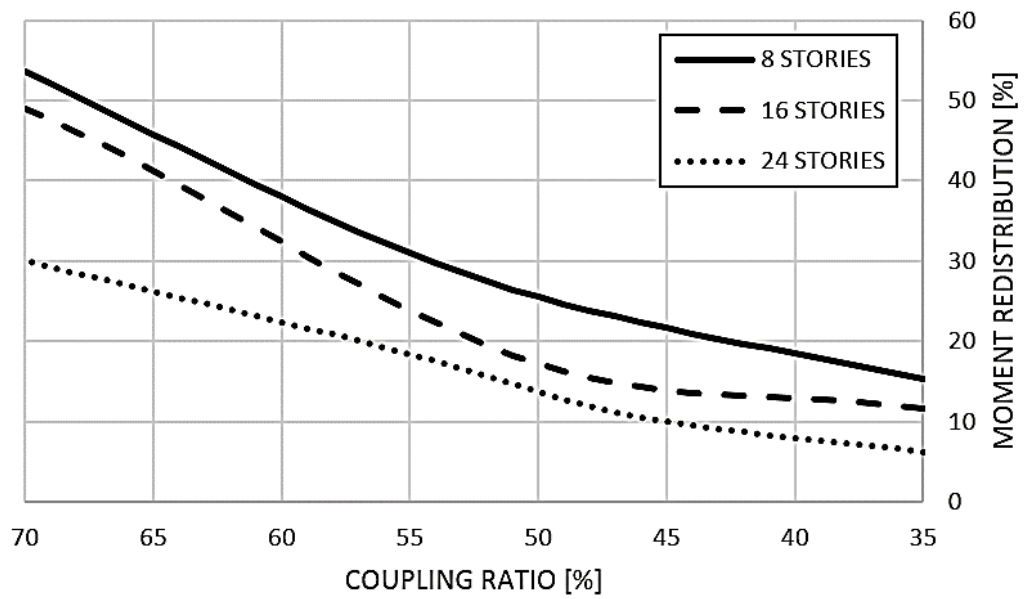


Figure 6.75 : Moment redistribution vs. coupling ratio 10% in 50 years.

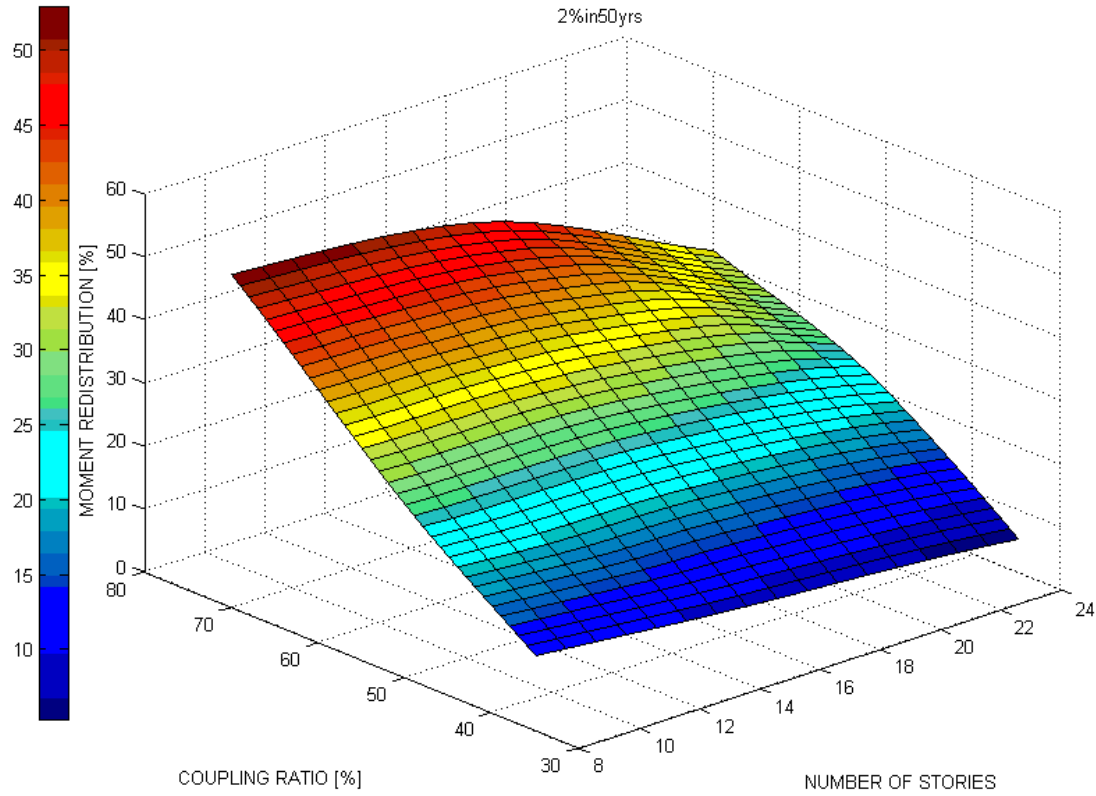


Figure 6.76 : Moment redistribution 2% in 50 years.

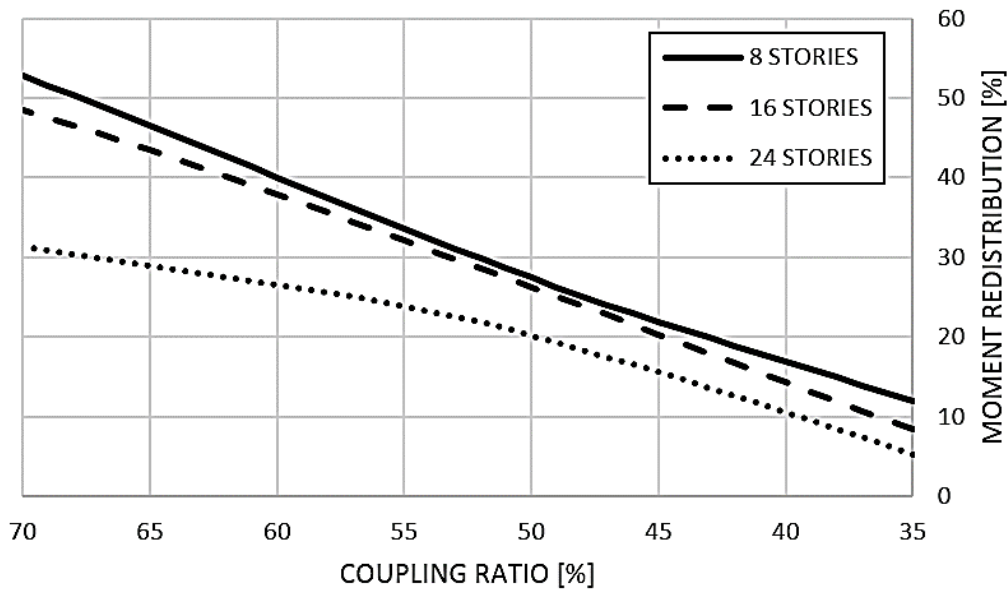


Figure 6.77 : Moment redistribution vs. coupling ratio 2% in 50 years.

The design level earthquake (10% in 50years) analysis result (Figure 6.74) shows that the moment redistribution falls behind 45% in wall systems which have coupling ratio higher than 65%. On the contrary, the amount of redistribution drops down 15% for low-coupled systems. Based on this fact, although the 30% of redistribution in moment demands are allowed in the code-based linear analysis procedures, it is not

recommended to apply moment re-distribution more than 15% on wall systems with low-coupling degrees. Otherwise, probable unnecessary moment re-distribution in the design phase may trigger much higher shear migrations than anticipated. In the light of this information, it can be suggested to estimate the flexural demand on any wall by using the following combinations during the preliminary design:

$$P = 0.9P_G - \sum V_i \quad ; \quad M' = (1 - r)M \quad (6.9)$$

$$P = 1.0P_G + 1.0P_Q + \sum V_i \quad ; \quad M' = (1 + r)M \quad (6.10)$$

Here, P_G is the axial compression due to dead load; $1.0P_Q$ is the axial compression due to live load; V_i is the coupling beam shear force at i -th story; M is the wall bending moment obtained from linear analysis; M' is the wall modified bending moment after redistribution applied. The proposed moment redistribution ratio r is tabulated in the Table 6.23:

Table 6.23 : Proposed moment redistribution coefficient.

Coupling Ratio	r
25%-35%	0.15
45%-75%	0.30

Since bending moment and shear are always interrelated in a wall behavior, flexure yielding in the tension wall also limits the shear demand forces acting on it. On the other hand, the exceeding demand force then is borne by the compression wall. This shear redistribution event between wall piers is also called “shear migration” (Paulay, 1981).

The shear migration can also be rephrased as the difference in the shear demands of the two walls in a coupled wall system if most of the lateral forces are carried by the wall piers. However, this migration event does not always happen when the total base shear is maximum. The second condition to observe a shear migration is a moment redistribution event, which is an unavoidable occurrence between the walls (Vuran, 2014).

The shear migration and its relation with the wall pier shear demands and system base shear is illustrated in Figure 6.78.

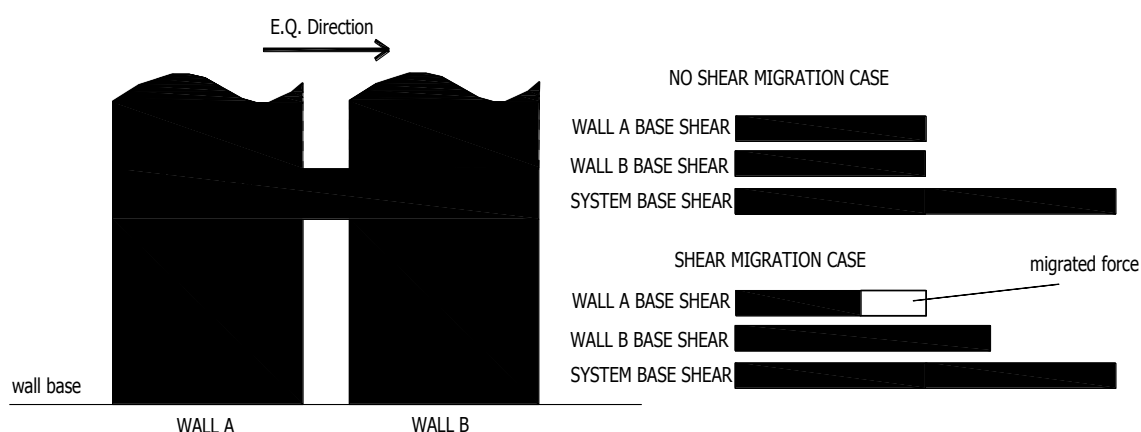


Figure 6.78 : Shear migration and wall pier base shear demands.

It has already been explained in the previous chapter that the maximum change in the axial forces is the key to achieve the highest disparity values in bending moments at wall bases, since the flexure capacity of a wall depends strongly on the acting axial force. In other words, the most critical shear migration would most probably occur when large coupling effect coincides around the time domain of peak base shear.

Figure 6.79, Figure 6.80 and Figure 6.81 show the maximum ratio of shear demand values between the compression and tension walls observed in time window frame of ± 0.1 seconds at around the maximum system base shear. The scattered datum extracted from each individual nonlinear time history analysis and the markers connected with dashed line represent the mean of all 20 analyses conducted for each wall system.

The percentage of the migrated shear over total base shear can be estimated easily by examining the change between the shear demand ratio of compression wall and tension wall. It is observed that the shear redistribution (or shear migration) percentage can be over 15% for wall systems with high coupling degree (Figure 6.82). Especially in 8 and 16 story wall systems, it is observed that the percentage of migrated shear over total system base shear climbed up to 17%.

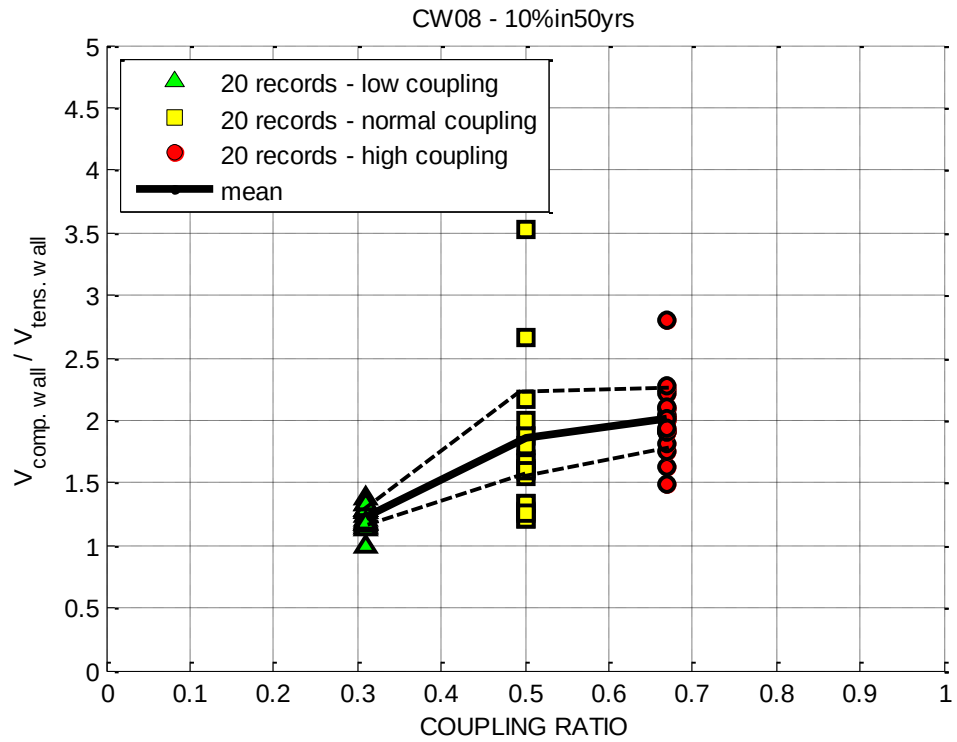


Figure 6.79 : Shear ratio between walls vs. CR – CW08 – 10% in 50 years.

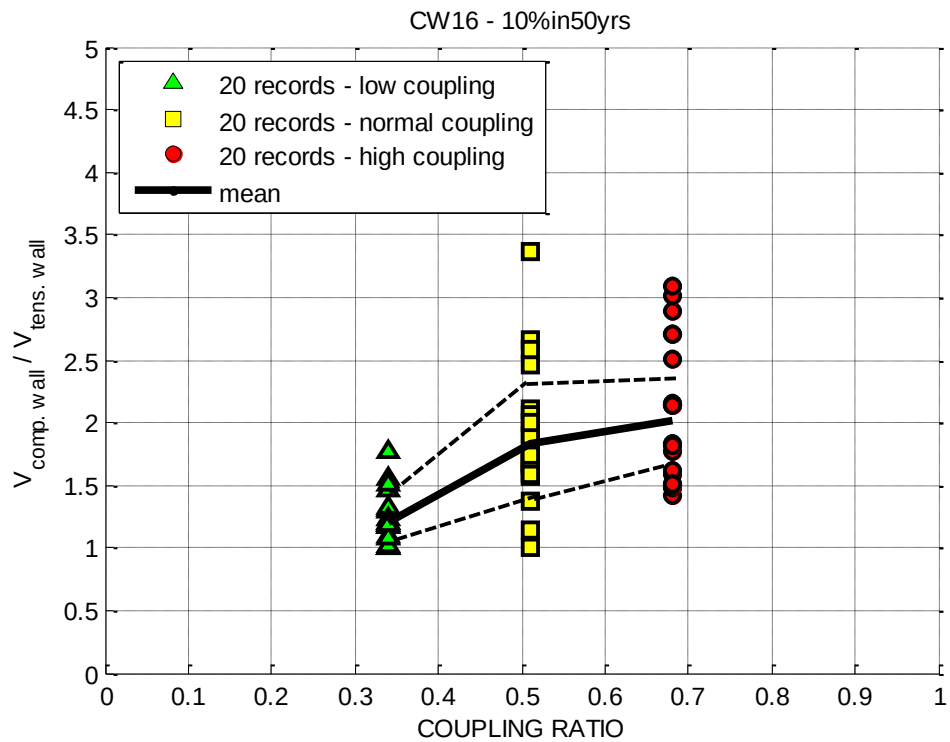


Figure 6.80 : Shear ratio between walls vs. CR – CW16 – 10% in 50 years.

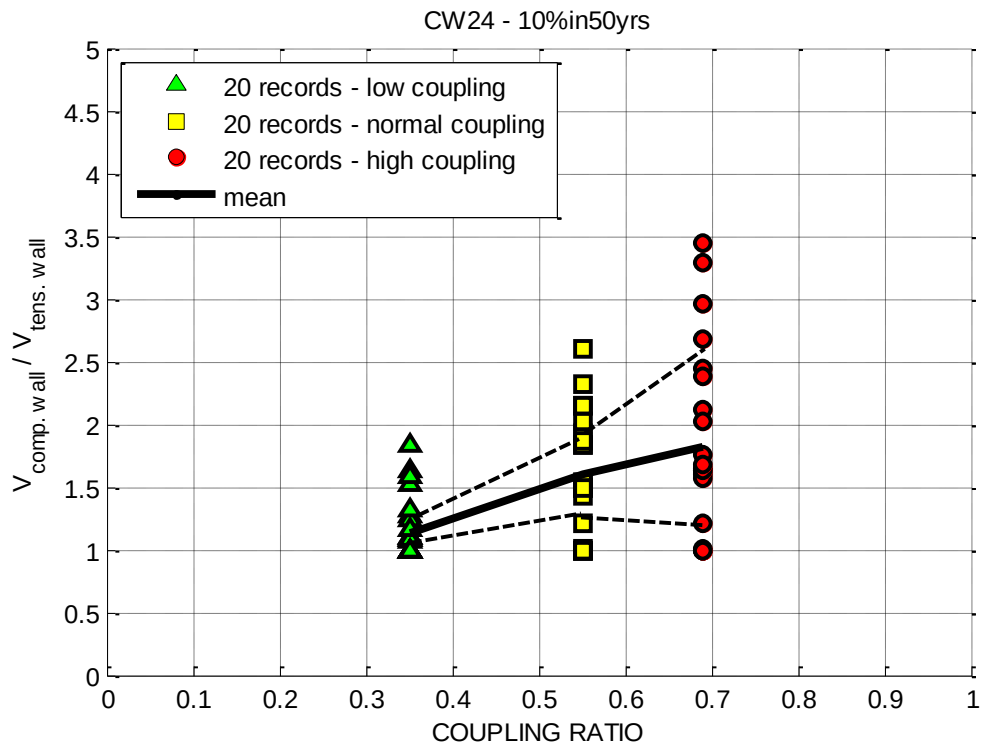


Figure 6.81 : Shear ratio between walls vs. CR – CW24 – 10% in 50 years.

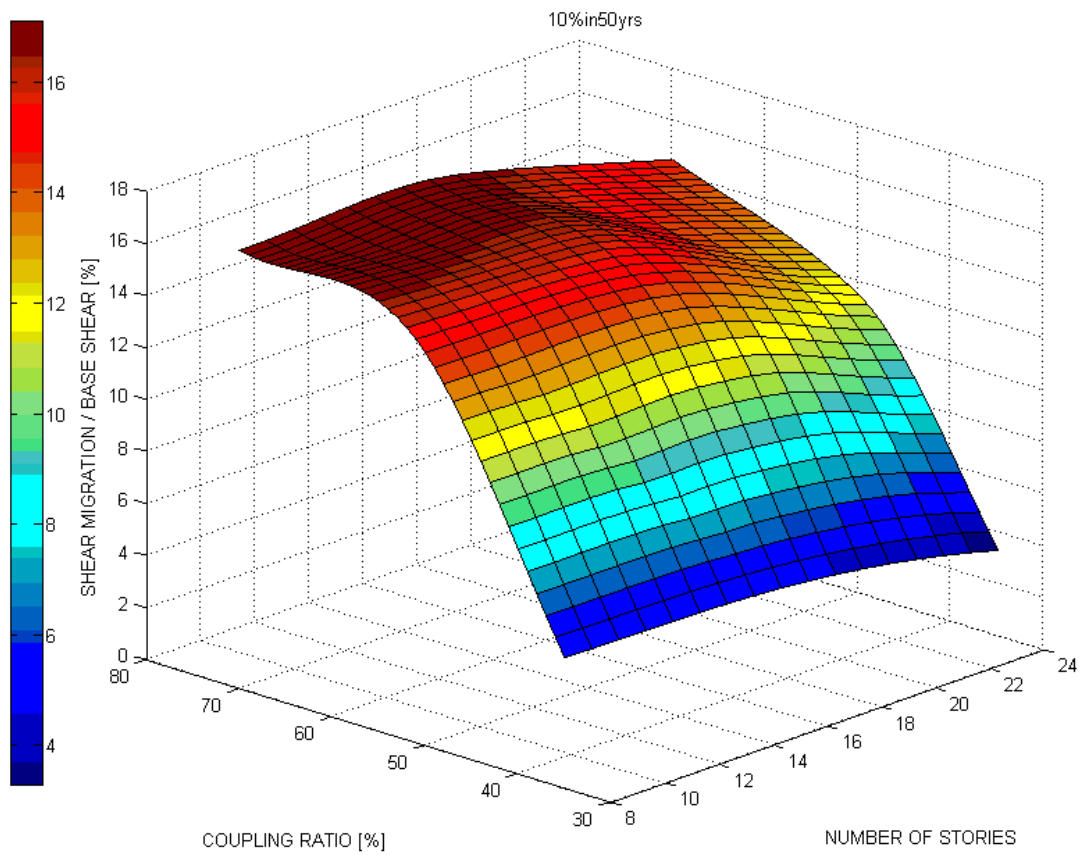


Figure 6.82 : Shear redistribution 10%in50 years.

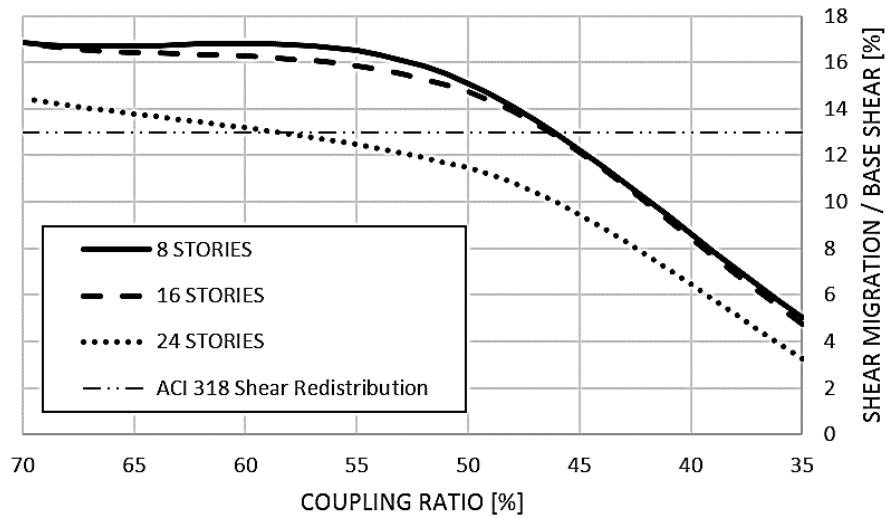


Figure 6.83 : Shear redistribution vs. coupling ratio.

The average unit shear strength assumed for the total available cross-sectional area of both wall piers is limited by ACI 318-11 (2011) as $0.66\sqrt{f'_c}$ with the additional requirement that the unit shear strength assigned to any single pier should not exceed $0.83\sqrt{f'_c}$. This upper limit of strength to be assigned to one of the coupled wall pier is imposed to limit the degree of redistribution of shear force, naturally. In other words, ACI 318-11 (2011) ensures the safety of wall piers against a shear redistribution of up to $(0.83 - 0.66)/(0.66 + 0.66) = 13\%$.

In the analyses presented herein, it is observed that the shear redistribution or the “shear migration” between two identical rectangular walls can climb up to 17% of the total base shear in high-coupled systems.

6.6.7 Comparison of fixed and flexible base models

Fixed-based restraining approach without considering the soil-structure interaction is a common practice in building structure modeling. This reduces the complexity of the problem (i.e. the number of additional degrees of freedom for accounting for the interaction) and lowers the time spent in the computations. In reality, building foundations are flexible. Depending on the soil conditions and foundation properties, the difference between fixed-base model and flexible base model may be significant.

In this section, the mat foundation in the numerical analyses are considered as flexible and the results of 16-story with normal coupling ratio (CW16-CR-N) have been compared with its fixed-based model results for validation. The outside dimensions of 1.5m thick mat foundation is 28.5m by 28.5m. Only half-width is considered since

there exist two coupled wall systems in one direction. The vertical subgrade modulus and horizontal subgrade modulus is assumed as 15000 kN/m^3 and 5000 kN/m^3 , respectively. The subgrade moduli is utilized to obtain spring constants to be modeled under elastic foundation. The mathematical model strategy is illustrated in Figure 6.84.

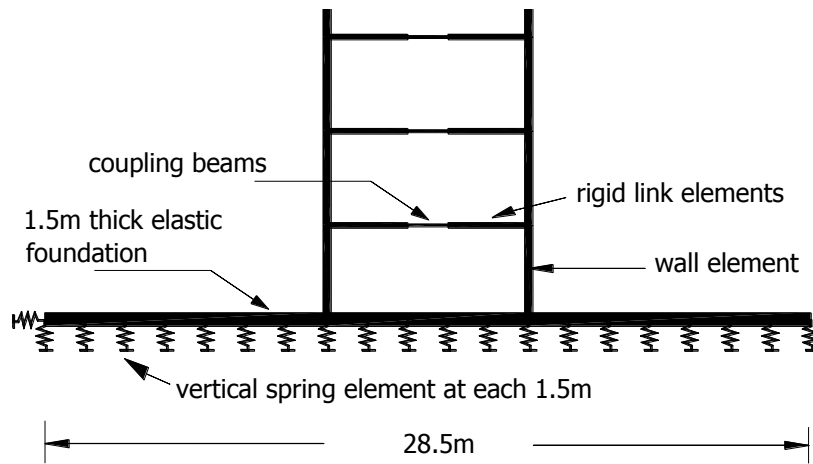


Figure 6.84 : Mathematical model representation of flexible base.

The period durations corresponding to structure's first three in-plane modes have been estimated in eigenvalue analysis (Table 6.24).

Table 6.24 : Periods of fixed base model and flexible base model.

	Fixed base model	Flexible base model
1st Period [sec]	1.61	1.73
2nd Period [sec]	0.36	0.42
3rd Period [sec]	0.14	0.16

The nonlinear time history analyses have been carried under design level earthquake (10% in 50 years) with 20 records. The average of maximum plastic rotations of coupling beams have been obtained and plotted in Figure 6.85. Although there exists light difference in mid-story levels, the plastic rotation demands for both base options are in good agreement with each other.

Coupling beam plastic rotations, wall strains, moment redistribution and shear migration rates have been compared with previously extracted fixed-base model analyses results (Table 6.25). The overall comparison shows that the fix-based option considered in this dissertation exhibit very close trend to the flexible based condition studied in this section.

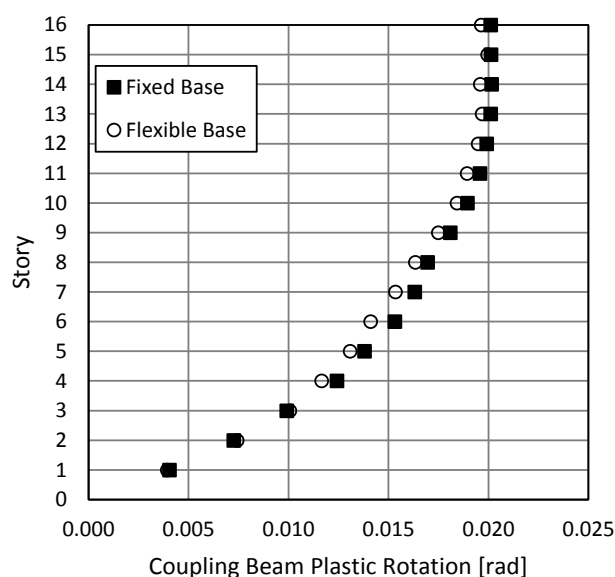


Figure 6.85 : Coupling beam plastic rotation fixed base vs. flexible base.

Table 6.25 : NLTHA results fixed base vs. flexible base.

	Fixed base model	Flexible base model
Coupling beam plastic rotation [rad]	0.021	0.020
Max. tension strain in steel	0.0067	0.0059
Min. compression strain in concrete	-0.0018	0.0014
Moment redistribution [%]	17.4	16.6
Shear migration [%]	14.5	13.8

6.6.8 Alteration of coupling ratio and its effects

The change in the contribution of coupling ratio under lateral loads has already been explained earlier in pushover analysis results section. Further investigation has been carried out with dynamic nonlinear analysis cases. Figure 6.86 shows the time history of overturning moment and coupling moment for the wall system CW08-CR-N under ground motion number 36. When all elements are still elastic at $t=10.8\text{sec}$, it was measured that the coupling ratio is approximately 0.47. Later on, the coupling beams started to yield as they reached their maximum capacity. At $t=13.5\text{ sec}$, the coupling ratio is decreased down to 0.40.

The ratio between the coupling moment over the system overturning moment was calculated for every significant demand peaks. The alteration in the coupling ratio then plotted versus the normalized time for CW08-CR-N wall under ground motion 36 as shown in Figure 6.87. The initial coupling ratio value in the analysis was 0.49 where all elements are elastic. At the end of the analysis, coupling ratio reduced down to 0.22

which is even lower than the minimum required coupling ratio for designing coupled wall systems (TEC, 2007).

In order to investigate the alteration in the coupling ratio during an earthquake, nonlinear time history analyses have been conducted by using “cloud approach”. As aforementioned, the cloud approach in this dissertation uses 50 ground motion records which are not scaled to the same acceleration level. On the contrary, the records from a common scenario are selected to have different peak ground acceleration values in order to provide an incremental dynamic analysis.

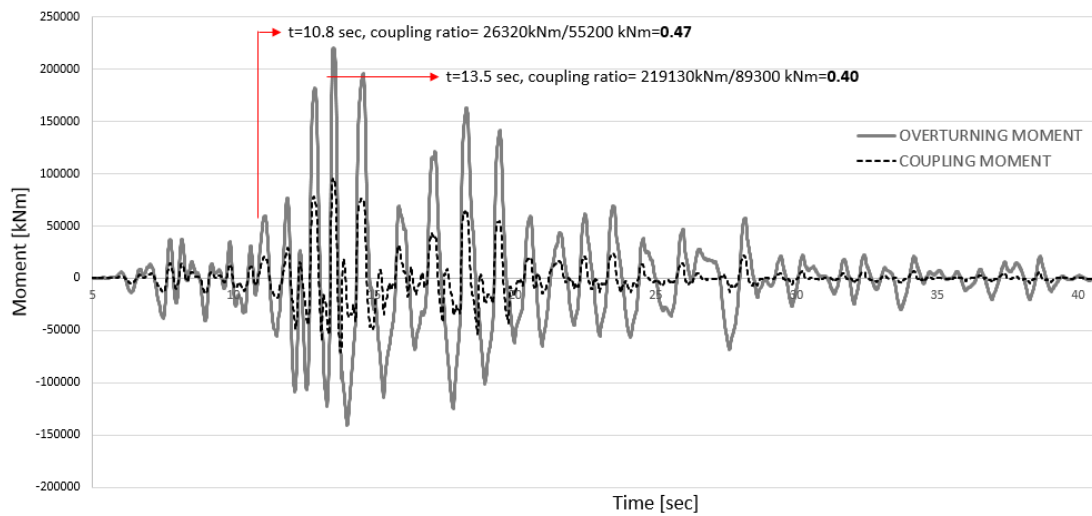


Figure 6.86 : Coupling moment and overturning moment vs. time – CW08-CR-N ground motion no.36.

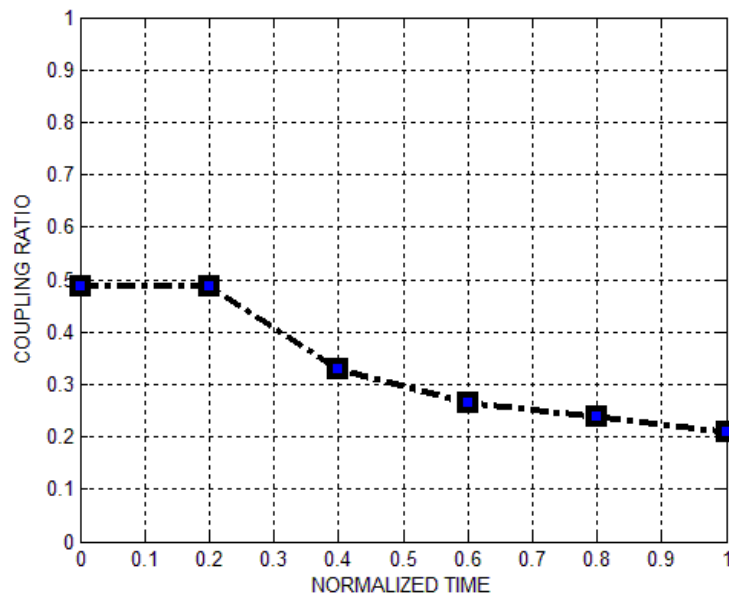


Figure 6.87 : CR vs. normalized time CW08-CR-N ground motion no.36.

From each individual nonlinear dynamic analysis, maximum demand beam plastic rotation and the corresponding system coupling ratio have been obtained. The drop rate in the coupling ratio, in other words, the ratio of the initial coupling degree (elastic) to the post-earthquake coupling degree (plastic) is plotted as shown in Figure 6.88 to Figure 6.96.

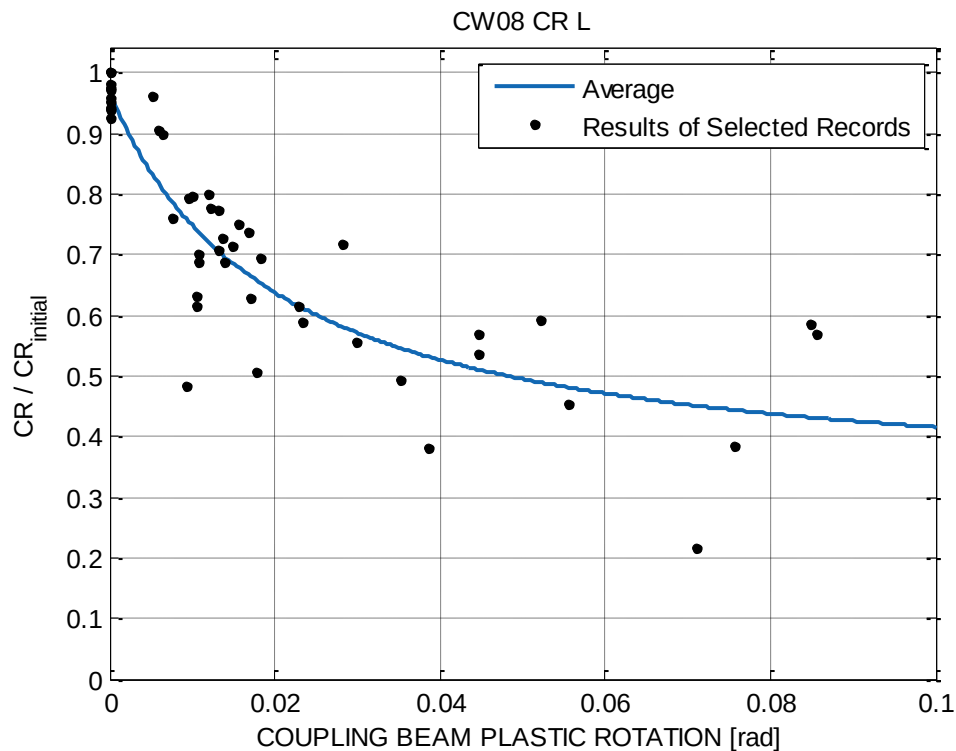


Figure 6.88 : Coupling ratio alteration vs. beam plastic rotation CW08-CR-L.

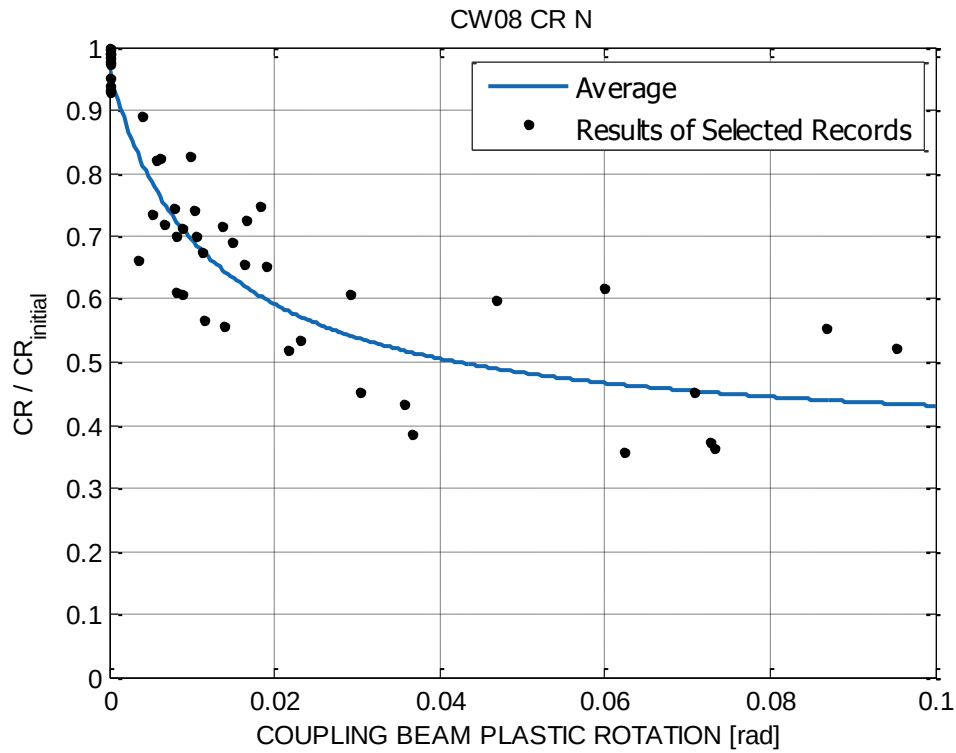


Figure 6.89 : Coupling ratio alteration vs. beam plastic rotation CW08-CR-N.

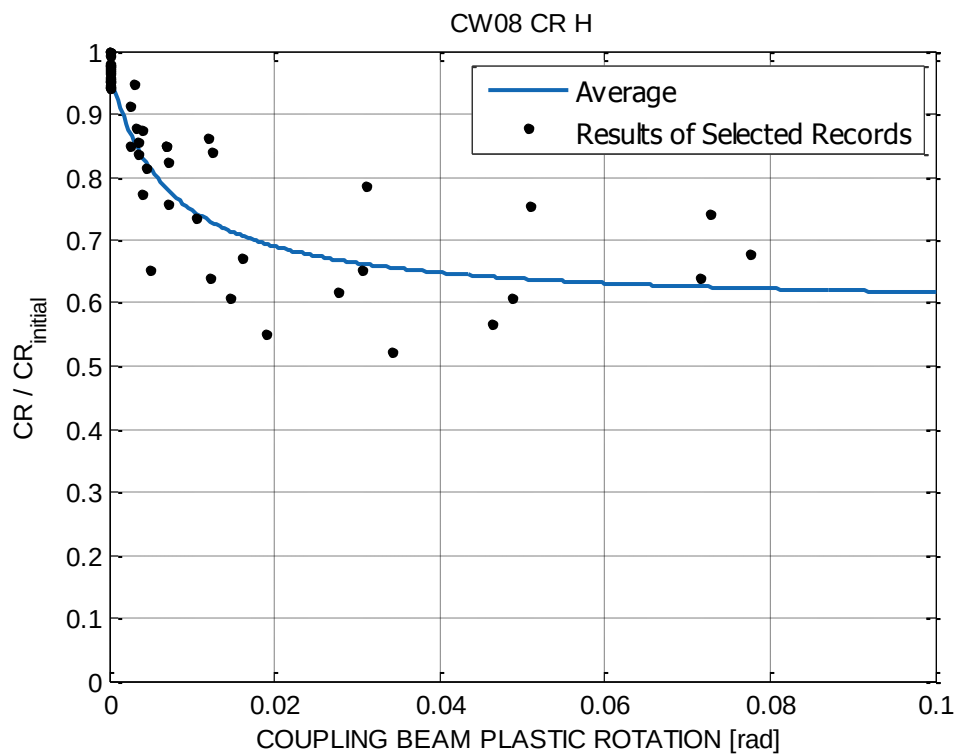


Figure 6.90 : Coupling ratio alteration vs. beam plastic rotation CW08-CR-H.

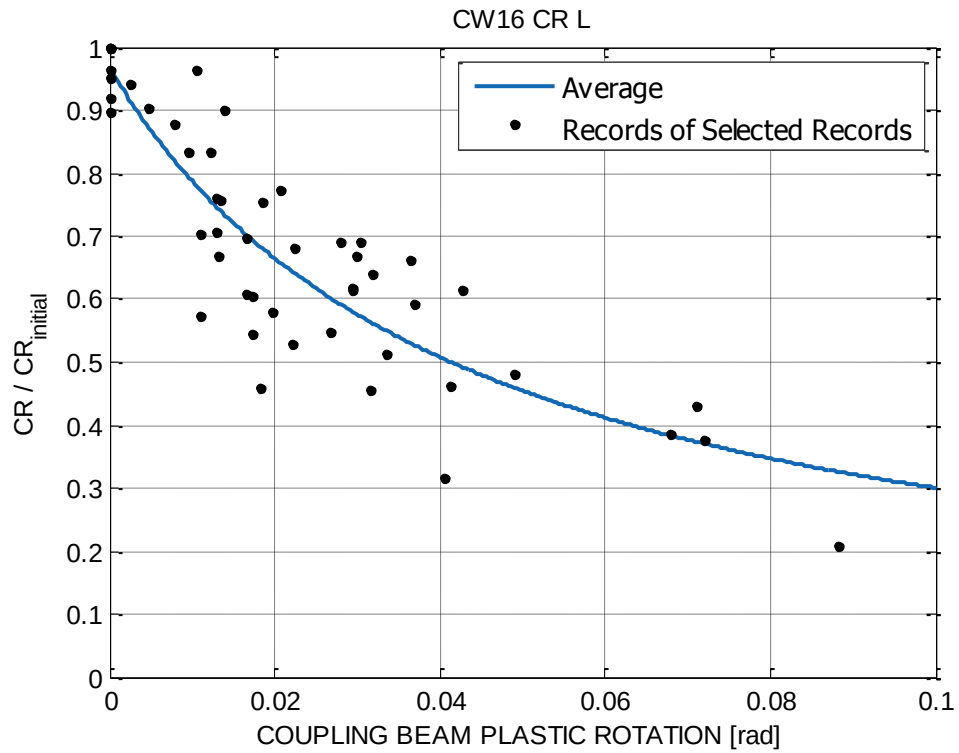


Figure 6.91 : Coupling ratio alteration vs. beam plastic rotation CW16-CR-L.

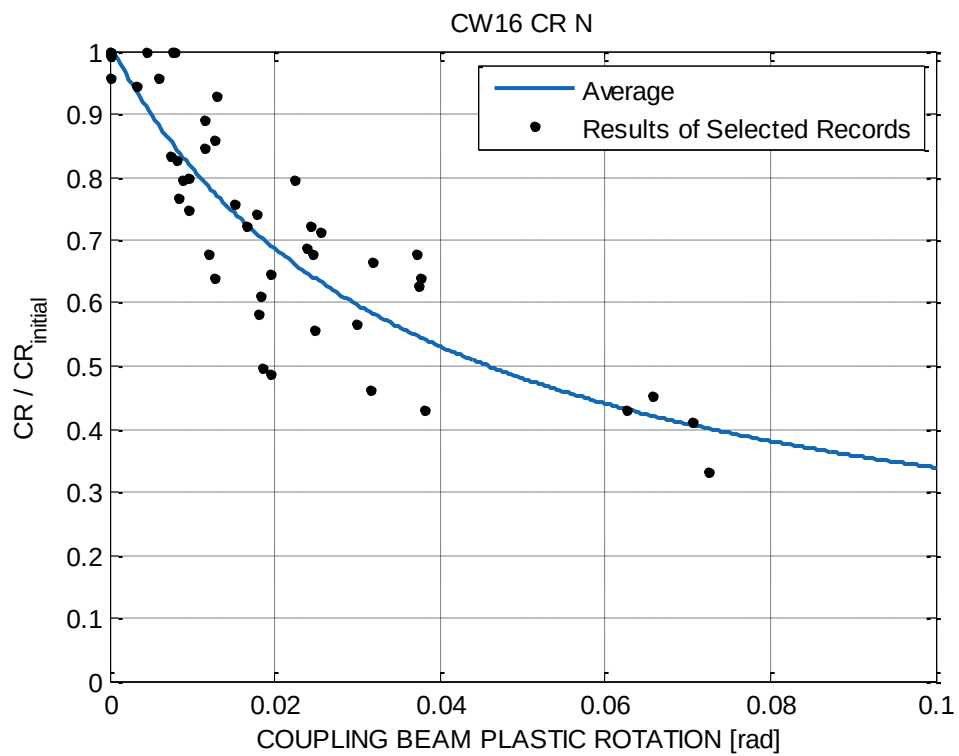


Figure 6.92 : Coupling ratio alteration vs. beam plastic rotation CW16-CR-N.

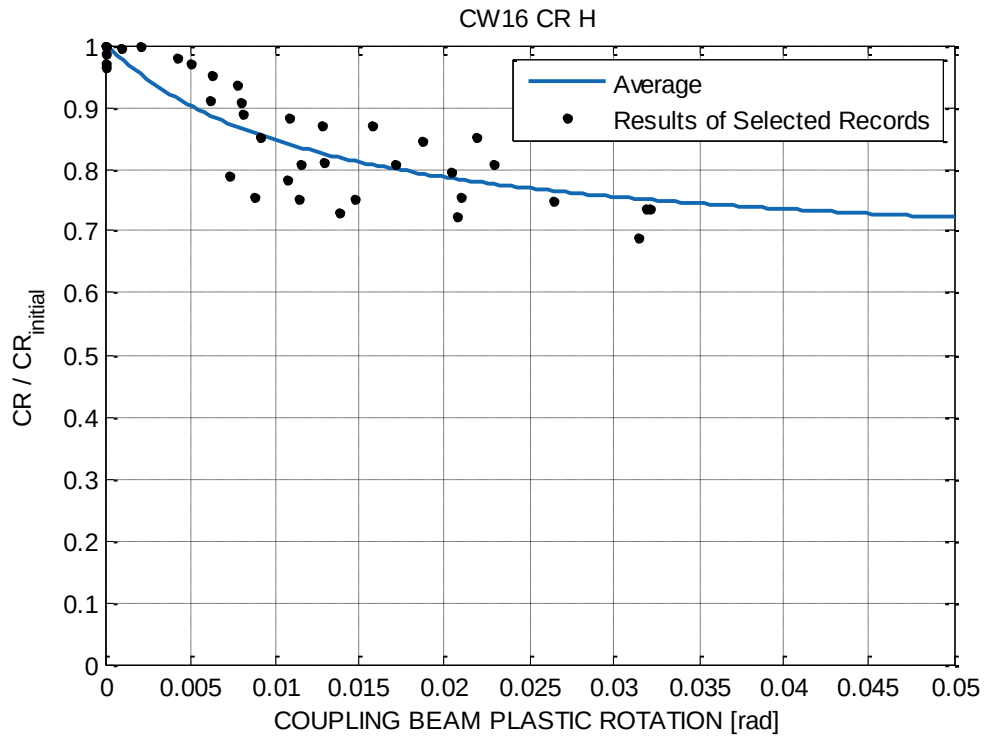


Figure 6.93 : Coupling ratio alteration vs. beam plastic rotation CW16-CR-H.

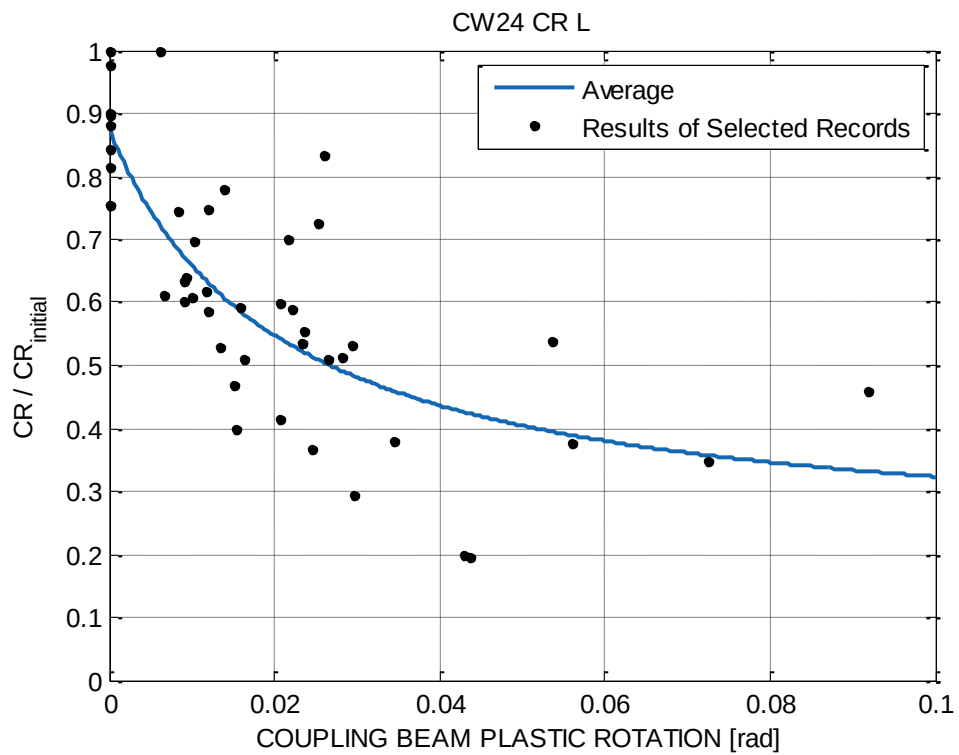


Figure 6.94 : Coupling ratio alteration vs. beam plastic rotation CW24-CR-L.

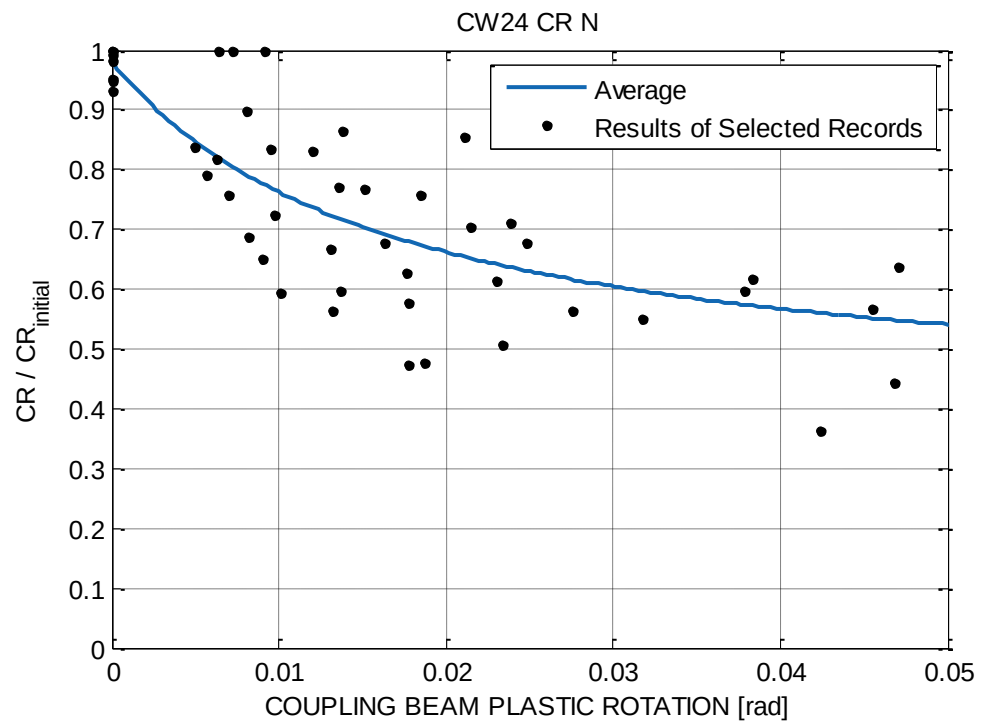


Figure 6.95 : Coupling ratio alteration vs. beam plastic rotation CW24-CR-N.

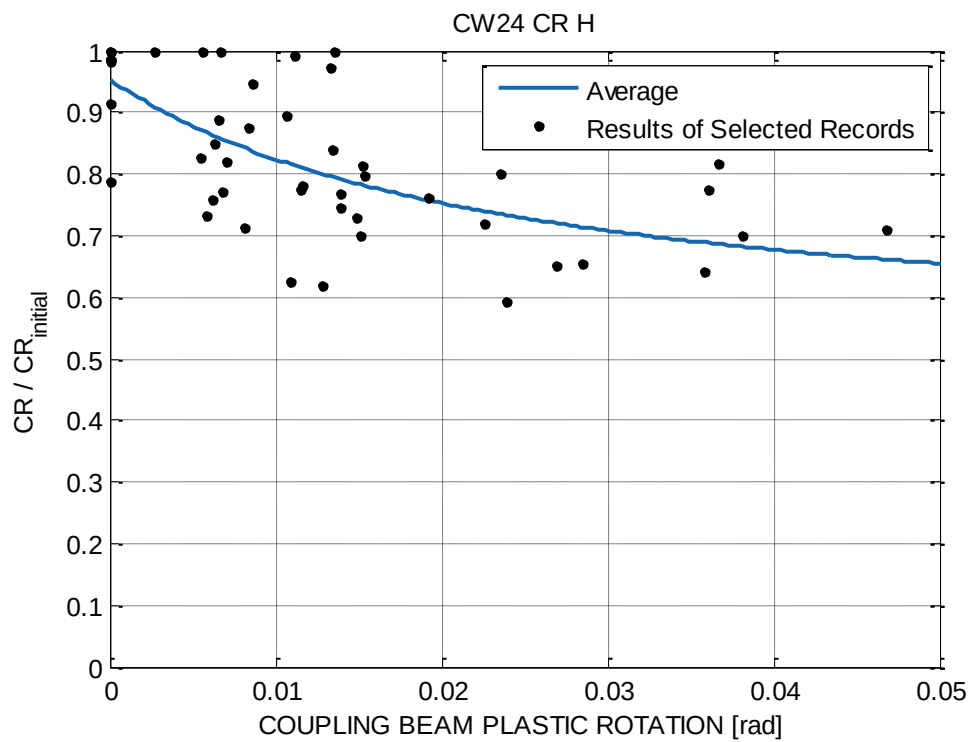


Figure 6.96 : Coupling ratio alteration vs. beam plastic rotation CW24-CR-H.

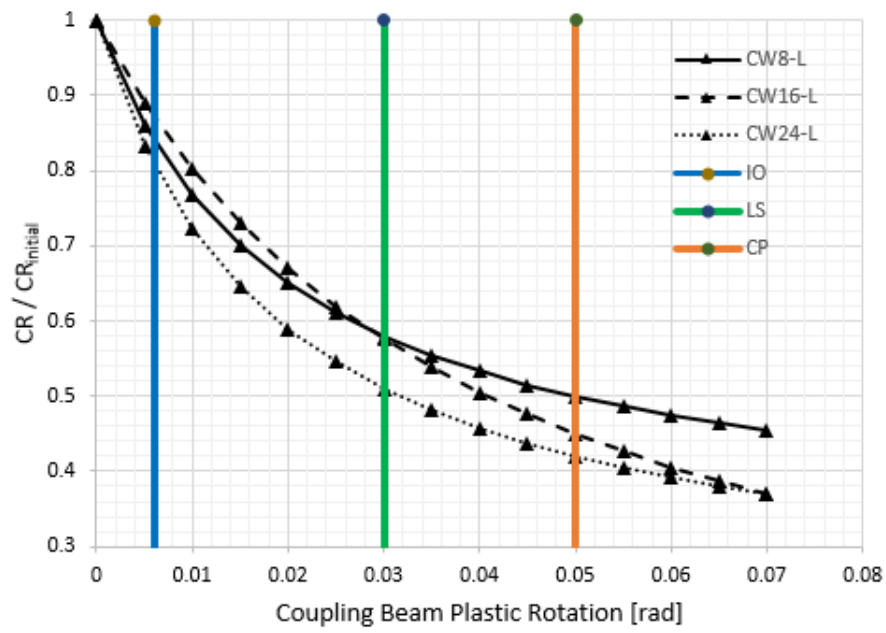


Figure 6.97 : Coupling ratio alteration vs. plastic rotation – low coupling.

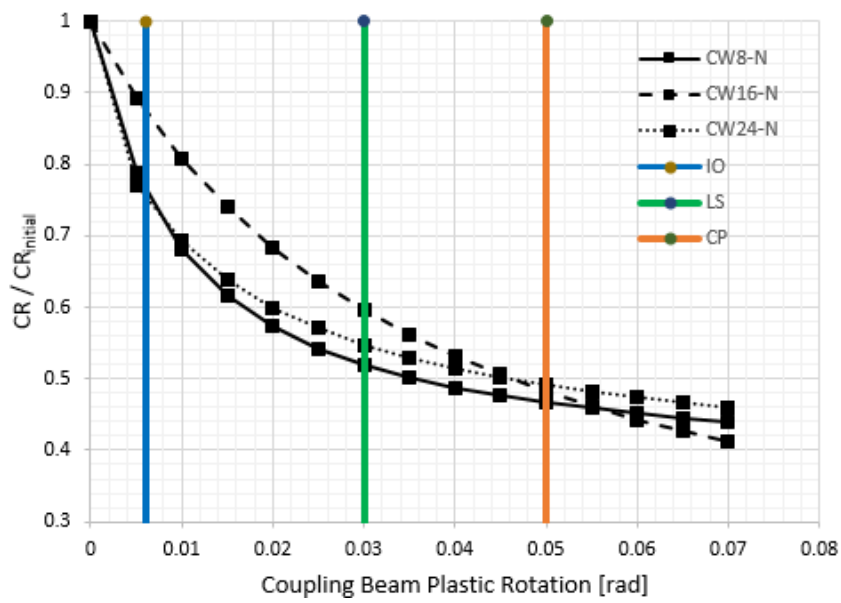


Figure 6.98 : Coupling ratio alteration vs. plastic rotation – normal coupling.

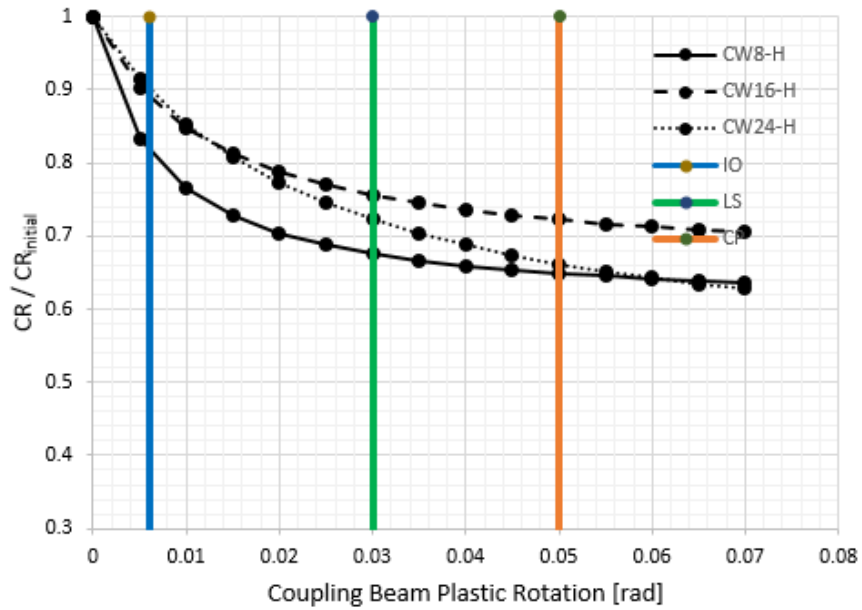


Figure 6.99 : Coupling ratio alteration vs. plastic rotation – high coupling.

The coupling beam plastic rotation versus normalized coupling ratio graphs show that the alteration in the coupling ratio in low-coupled and normal-coupled wall systems are in good agreement, although the number of stories changes. It is observed that around 60% of reduction in the coupling ratio is observed for systems with low and normal coupling ratio. Different than others, the high coupled systems has only a drop rate of 35%.

6.7 Summary and Remarks

In this chapter, numerical case studies about coupled wall systems are presented. These studies investigate the effect of coupling ratio on behavior of selected coupled wall systems. Nine models, with varying coupling ratio rates, of 8-16-24-story building have been designed with linear approach, then loaded with 20 ground motions at service level, design level earthquake and maximum considered earthquake (MCE) level.

In preliminary design stage, where linear analysis method was adopted, the system with 0.70 coupling ratio dragged up to 30% more base shear demand than the systems with 0.50 coupling ratio because of the higher period of the stiffer system. The nonlinear analyses show that the wall pier exposed to tension which coupling beams pull up (tension wall) yields sooner than the other wall which coupling beam push down (compression wall).

Under design level earthquake, it has been observed that the moment demand ratio between wall piers increased up to 325% in high coupling ratio systems. On the other hand, in low coupling degree systems, this ratio does not exceed 135%. The moment re-distribution percentage in the system can easily be determined by making substitutions from the ratio between wall moments. In this case, the moment re-distribution in low-coupled systems does not exceed 15%, which is lower than the code-based allowed 30% moment-redistribution.

It is also observed that the shear redistribution (shear migration) between two identical rectangular walls climbs up to 16% of the total base shear in high-coupled systems.

The strain distribution in wall piers are also changed depending on the coupling ratio. The disparity in the strains between tension and compression wall piers becomes more significant when the coupling ratio is gets higher. Although, none of the case buildings exceed the upper limits for specified performance levels, yielding in the boundary zone vertical reinforcement is sufficient to trigger the shear migration event.

It should always be noted that the case study wall piers and coupling beams in this dissertation have been subjected to real earthquake records. The trade-off of this procedure is the record-to-record variability. This is why all results regarding several nonlinear analyses are presented with associated variability or confidence levels, which are found satisfactory for the purposes of this thesis.

7. CONCLUSIONS

Coupled wall systems are exceptional lateral load resisting systems that not only reduce the deformation demands on the building, but also extend the plastic deformation vertically. The work presented in this dissertation is aimed at contributing to the state of the art of coupled wall systems by investigating the nonlinear behavior of various multi-story building with different levels of coupling ratio. This chapter summarizes the research findings extracted from numerical studies both from diagonally reinforced coupling beam modeling techniques and coupled wall system responses. Some recommendations for further forthcoming researches are also presented at the end of this section.

7.1 Nonlinear Modeling of Diagonally Reinforced Coupling Beams

After performing a comprehensive literature survey through various previously completed analytic and experimental studies performed earlier, a simple yet accurate modeling procedure for coupling beams has been selected for capturing the nonlinear behavior properly. Further enhancement to the existing selected models is proposed in the thesis to have better accuracy in cyclic behaviors comparing to the previously performed test results. Furthermore, the simplicity in the modeling is some more improved by eliminating the slip/extension rotation hinges to be adapted on the case study structures with the parametrical study.

The obtained results on coupling beam modeling could be summarized as in the following:

- A nonlinear coupling beam mathematical model must represent diagonal reinforcement slip/extension behavior, which generally reduces the beam bending stiffness drastically. This behavior component can be implemented to the model by adding linear rotational spring to the both ends of the coupling beam. On the other hand, estimating the stiffness of slip/extension rotational spring elements requires excessive calculations, which may cause time delays in engineering practice. Therefore, a modification for the calculation of the

cracked section moment of inertia for the beam is proposed to eliminate the slip/extension rotation hinges in the model. For this proposal, a parametric study was carried out on 19 different coupling beams, which have various reinforcement ratios and aspect ratio (1.50 to 3.50) to investigate their slip/extension of diagonal reinforcements. It is observed that the change of effective stiffness coefficient in coupling beams varied between 0.14 and 0.04 almost linearly for aspect ratios 1.5 and 3.5, respectively. An approximate formula, which depends on the beam strength and the aspect ratio, was proposed for the sake of simplicity in the modeling.

- The necessity of contributing the concrete compression strength in the diagonals when estimating the coupling beam capacity had already been verified by Hindi and Hassan (2007) with monotonic pushover analyses, in their analytic studies. Within this dissertation scope, this validation study was extended to several other coupling beams utilized in cyclic experimental analysis. It was observed that modifying the coupling beam strength by including the concrete compression in the diagonals gives a better result in the hysteresis loops and cumulative energy when compared with the previously performed test results. Furthermore, this proposed modeling will avoid the danger of underestimating coupling beam strength that might cause wall piers yield sooner than predicted in numerical analyses.

7.2 Effects of Coupling Ratio on Nonlinear Behavior of Coupled Walls

Later in the dissertation work, the nonlinear behavior of various coupled wall systems have been investigated having different values of coupling ratio, which is defined as the unitless measure of coupling moment contribution to system over-turning moment. Three distinct coupling levels (low, normal and high) are considered with 8, 16 and 24 story coupled wall systems. Totally 9 coupled wall systems are designed under design earthquake loads with 475 year return period. In order to study the impacts of coupling ratio over the system behavior, linear, static nonlinear (pushover) and dynamic nonlinear (time history) analyses were carried out. Linear analyses are utilized for the initial design steps. Pushover analyses have been performed only for 8-story wall systems. For nonlinear time history analyses, 20 different ground motion records per each earthquake levels (72 years, 475 years, 2475 years) were matched with target

spectra for each 9 individual coupled wall system. In order to observe the effects and the alterations in their coupling ratios separate nonlinear time history analyses were carried out based on cloud approach, which uses 50 selected records with incremental peak ground accelerations. The obtained results on the behavior of the investigated systems could be summarized as in the following:

- Although the cross-sectional dimensions of structural elements are generally determined at the early stages of a building design, depth of coupling beams must be considered carefully. The changes in the aspect ratio of coupling beams increase their moment contributions (coupling ratio) which significantly affect the stiffness of the overall structure. While systems with low coupling ratio have longer fundamental periods, obviously, for the systems with higher coupling ratios the fundamental periods are shorter. Consequently, due to this fact the coupled wall systems with high coupling ratio drag more shear demand as compared with the lower coupled systems. It was observed in linear analyses that the base shear in systems with 0.70 coupling ratio dragged up to 30% more base shear demand than the systems with 0.50 coupling ratio, this is true for even the wall piers were designed as identical in both cases.
- Under seismic actions, coupling beams will have shear forces due to their coupling actions. These shear forces are transferred to wall piers as axial forces in opposite directions: one in tension, another in compression. The disparity in the axial force between each identical wall piers causes them to have different moment capacities concurrently. Therefore, the wall pier exposed to tension which coupling beams pull up (tension wall) yields sooner than the other wall which coupling beams push down (compression wall).
- Since the overturning moment does not change momentarily, the compression wall have to undertake the residual bending moment which the tension wall cannot bear. Although these increments are generally not evident in systems with low coupling ratios, the moment demand ratios between compression and tension walls become significant in normal and high-coupled wall systems. Under a design level earthquake (return period of 475 years), it has been observed that the moment demand ratio between walls increased up to 325%, 290% and 190% in high coupling ratio systems of 8, 16 and 24 story case study systems, namely CW08-CR-H, CW16-CR-H and CW24-CR-H, respectively.

Thus as a suggestion, although the 30% of redistribution in moment demands are allowed in the code-based linear analysis procedures, it is not recommended to apply moment re-distribution more than 15% on wall systems with low-coupling degrees. Otherwise, probable unnecessary moment re-distribution in the design phase may trigger much higher shear migrations than anticipated. This augmentation in the moment in a wall pier also increases the demand in the shear, which might cause brittle failure. Besides, apart from dynamic shear amplification, walls in coupling compression experience additional shear stresses depending on the inadequacy of moment capacity on the wall under coupling tension.

- Another result obtained about the disparity of the moment capacities is the shear redistribution between the wall piers. The average unit shear strength assumed for the total available cross-sectional area of both wall piers is limited by ACI 318-11 (2011) as $0.66\sqrt{f'_c}$ with the additional requirement that the unit shear strength assigned to any single pier should not exceed $0.83\sqrt{f'_c}$. This upper limit of strength to be assigned to one of the coupled wall pier is imposed to limit the degree of redistribution of shear force, naturally. In other words, ACI 318-11 (2011) ensures the safety of wall piers against a shear redistribution of up to $(0.83 - 0.66)/(0.66 + 0.66) = 13\%$. In the analytical investigations, it is observed that the shear redistribution or the “shear migration” between two identical rectangular walls can climb up to 17% of the total base shear in high-coupled systems.
- The amount of coupling moment coming from the coupling beams is limited with their strength. Thus, the maximum coupling moment is obtained at an instant when all coupling beams reach their yielding capacities. On the other hand, the wall piers will continue to have more bending moments at their bases while they are still elastic. The ratio of coupling moment to system overturning moment, also called coupling ratio, will start to decrease following the yields of the coupling beams. Therefore, the contribution of coupling beams to the overall system moment cannot be expected as constant during the whole process of ground motion. The actual coupling ratio of a system that experienced at a seismic action can be reduced down to 40% of its initial condition, which has to be predicted in the elastic design phase.

7.3 Recommendations for Further Researches

The current work presents an exploration of the coupling ratio alteration and investigation of demand forces redistribution between wall piers during their nonlinear behavior under seismic action. For better understanding of these phenomena, further research about the following subjects is recommended:

- In this dissertation, both wall piers (tension and compression walls) had identical cross-sectional dimensions. The cases where each walls have different sizes may also be investigated to extend this study.
- Coupled wall systems consist of “C”, “T” or “H” sectioned wall piers, which are generally accommodated in the core of high-rise buildings, may also be studied for the generalization of the findings obtained in this study.
- In addition to diagonally reinforced coupling beams, steel reinforced concrete (SRC) coupling beams must also be examined in future researches, since the application of hybrid coupled wall systems (reinforced concrete walls and steel coupling beams) becomes a new trend in the modern engineering practice.
- Soil-structure interaction (SSI) studies can be carried out in order to investigate how the moment redistribution and shear migration rates are effected.

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APPENDICES

APPENDIX A: Earthquake Ground Motion Records

APPENDIX B: OpenSees Codes

APPENDIX A : Earthquake Ground Motion Records

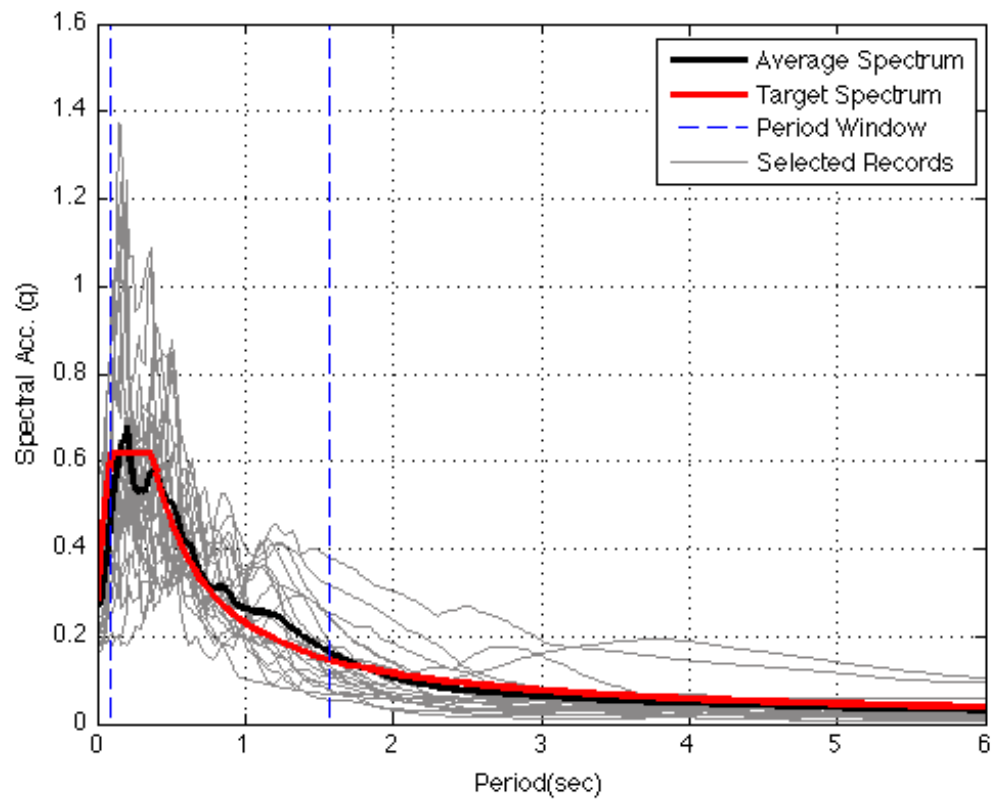


Figure A.1 : Average vs. target spectrum CW08 - 50% in 50 years.

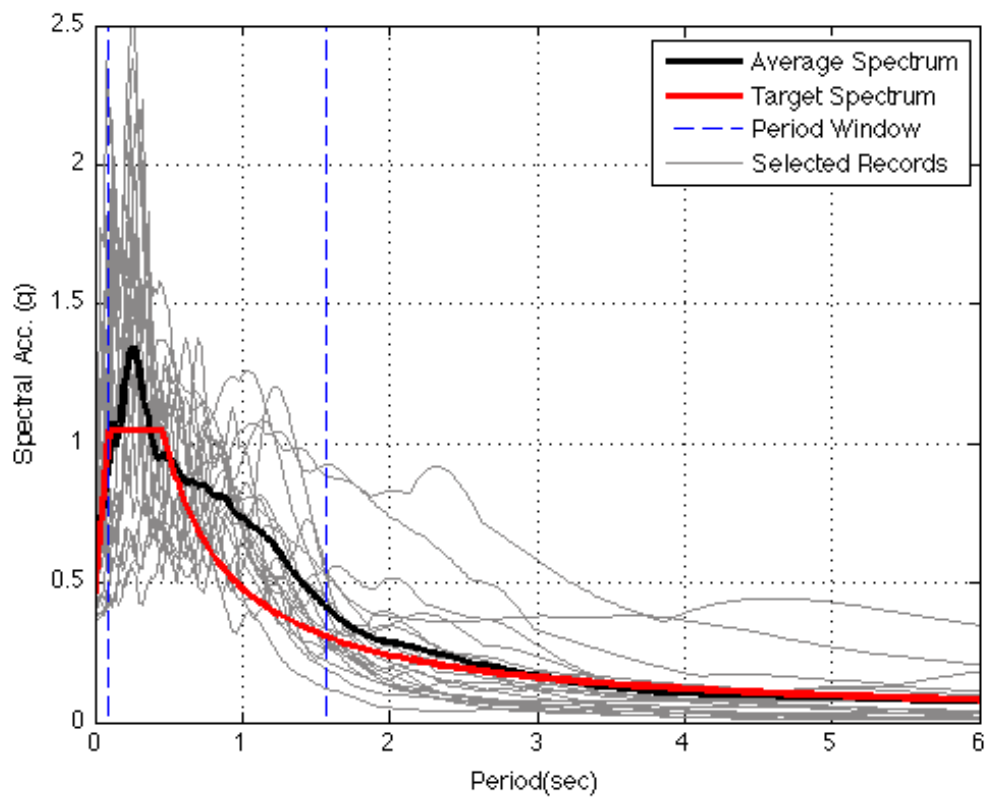


Figure A.2 : Average vs. target spectrum CW08 - 10% in 50 years.

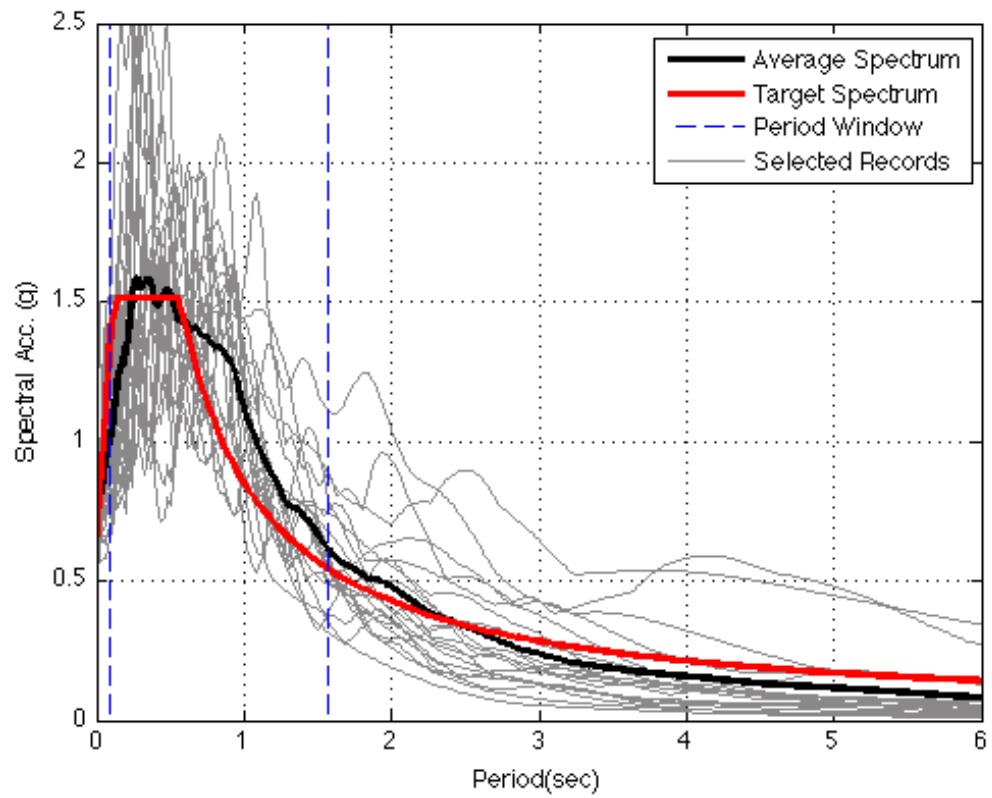


Figure A.3 : Average vs. target spectrum CW08 - 2% in 50 years.

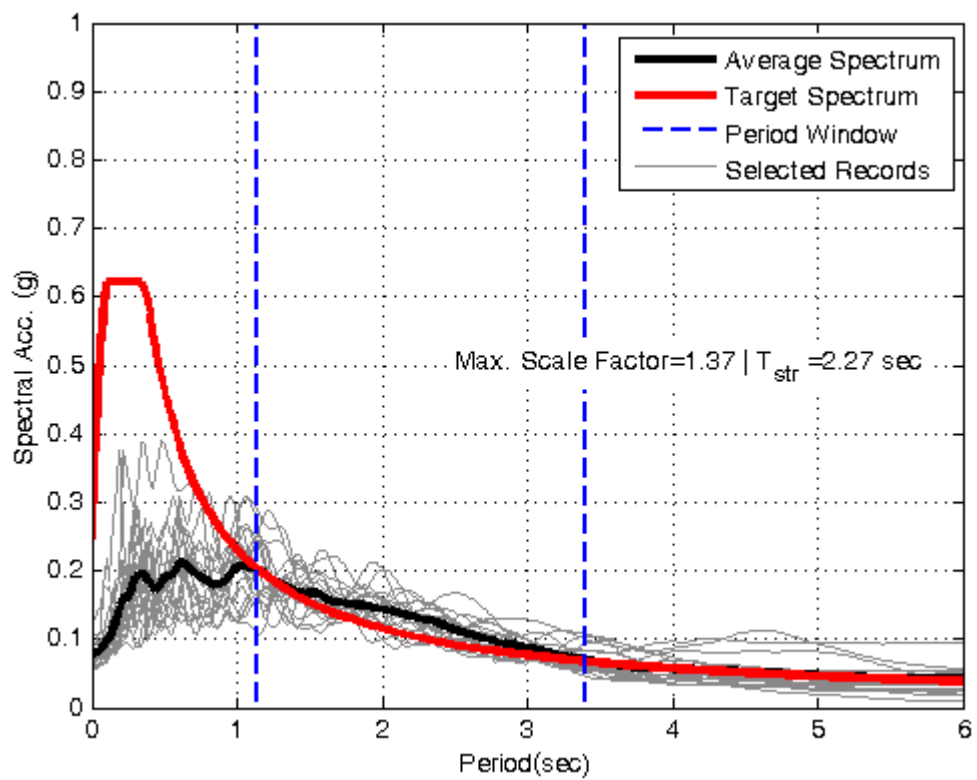


Figure A.4 : Average vs. target spectrum CW16L - 50% in 50 years.

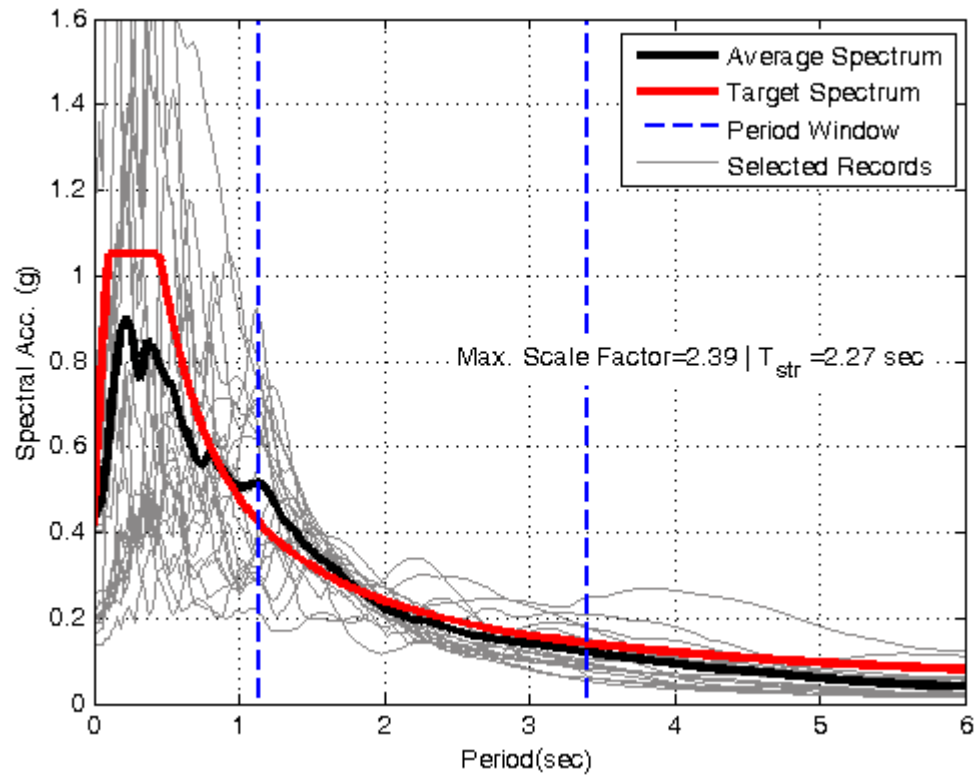


Figure A.5 : Average vs. target spectrum CW16L - 10% in 50 years.

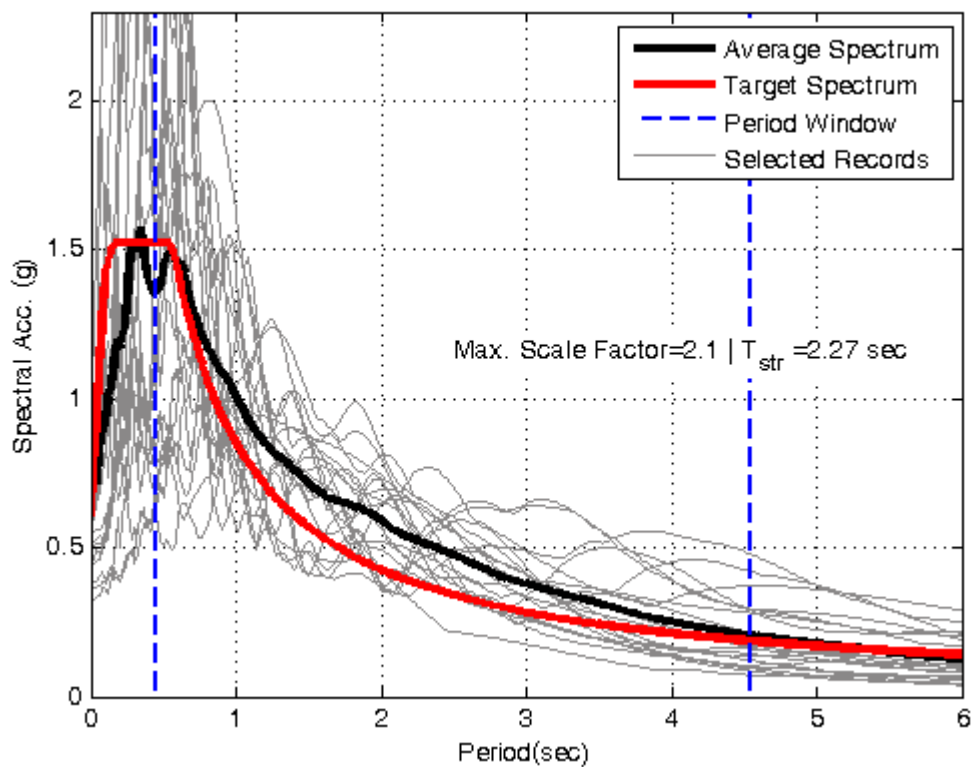


Figure A.6 : Average vs. target spectrum CW16L - 2% in 50 years.

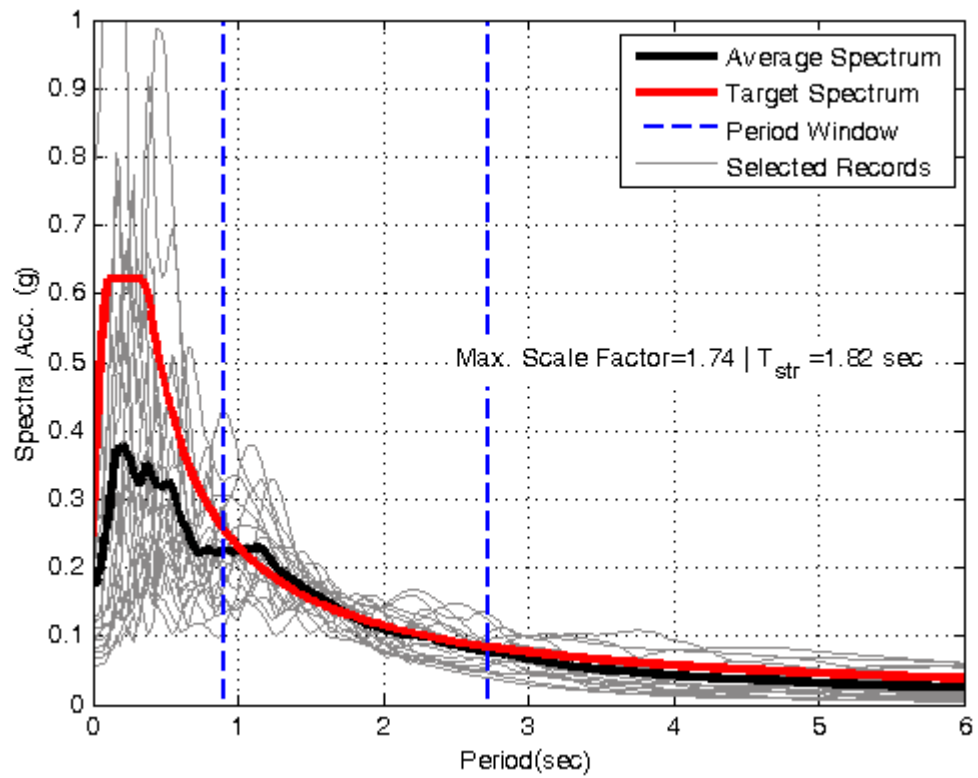


Figure A.7 : Average vs. target spectrum CW16N - 50% in 50 years.

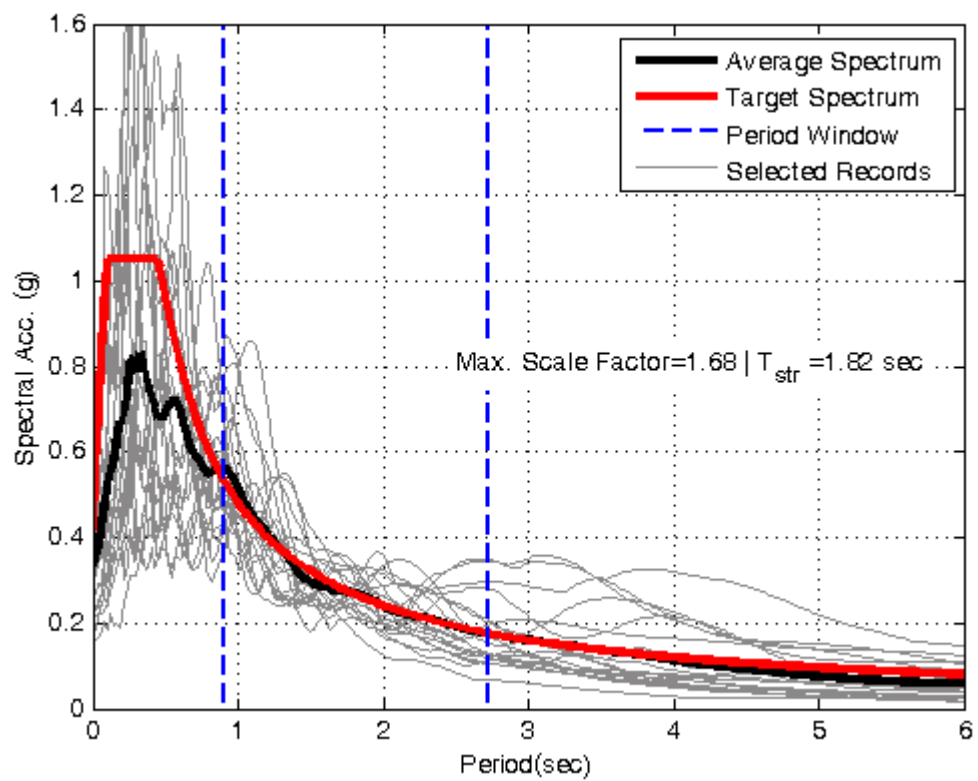


Figure A.8 : Average vs. target spectrum CW16N - 10% in 50 years.

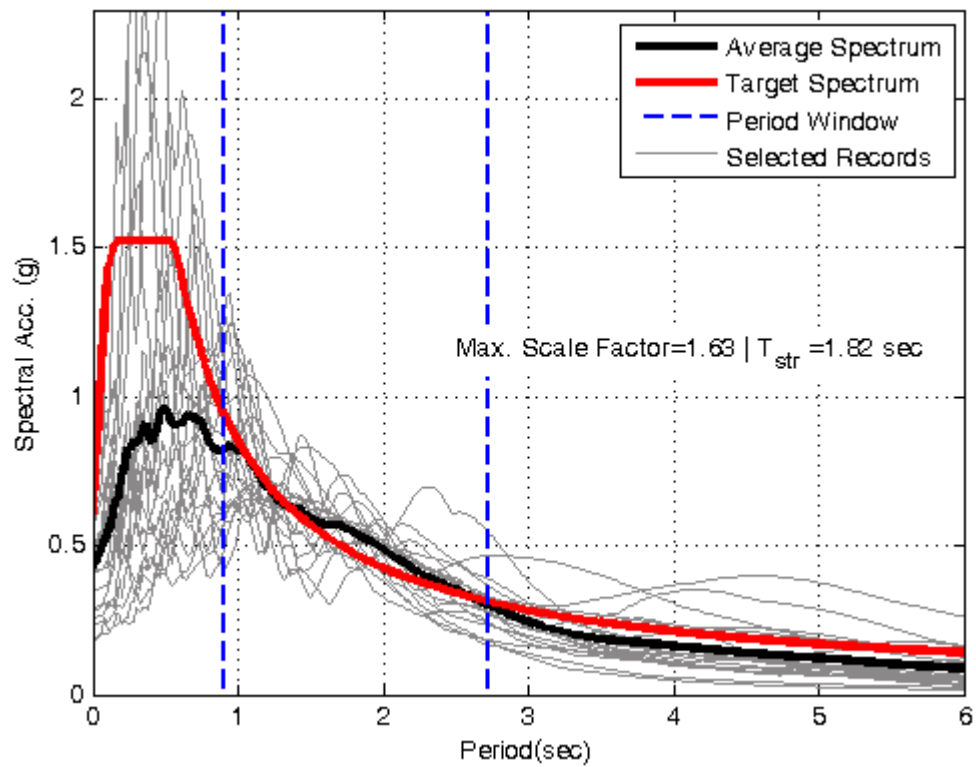


Figure A.9 : Average vs. target spectrum CW16N - 2% in 50 years.

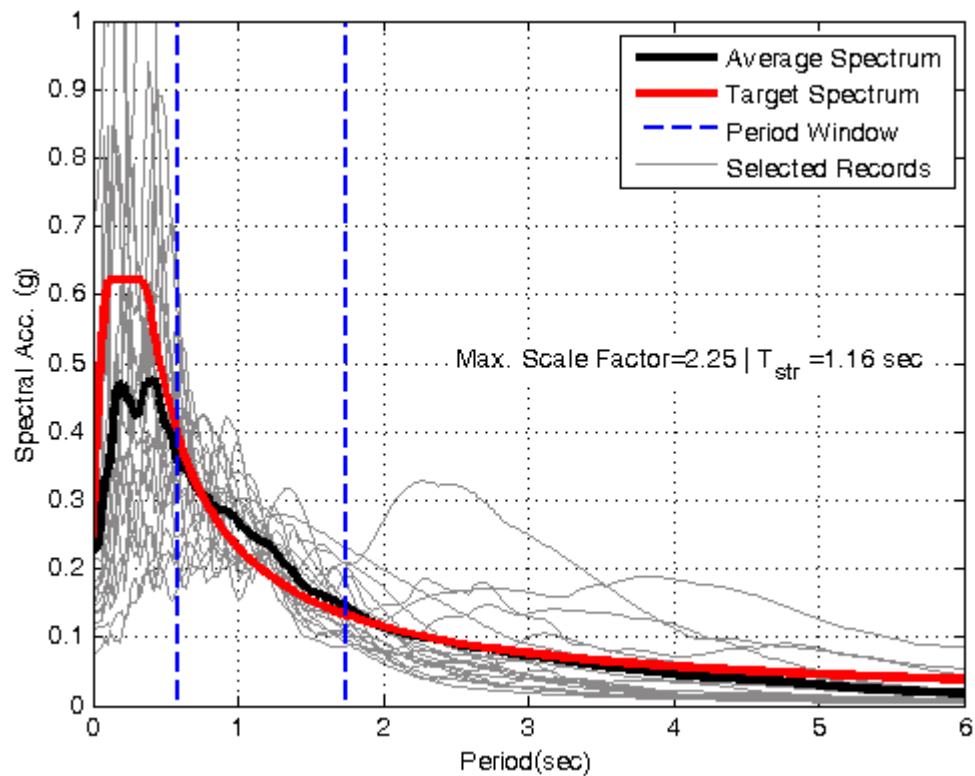


Figure A.10 : Average vs. target spectrum CW16H - 50% in 50 years.

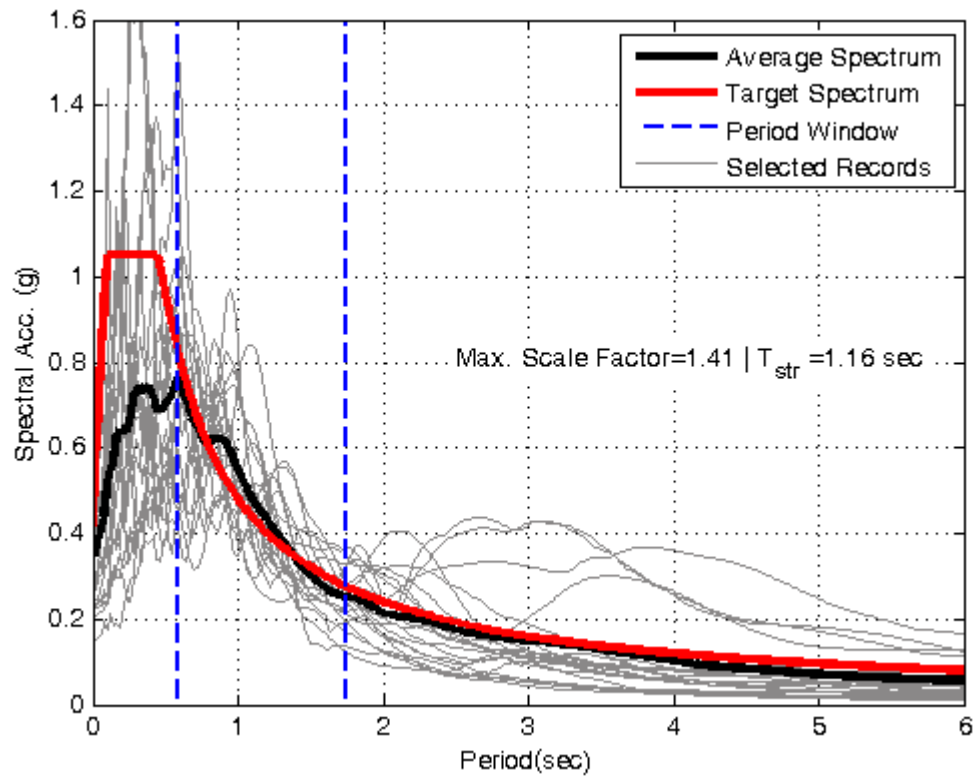


Figure A.11 : Average vs. target spectrum CW16H - 10% in 50 years.

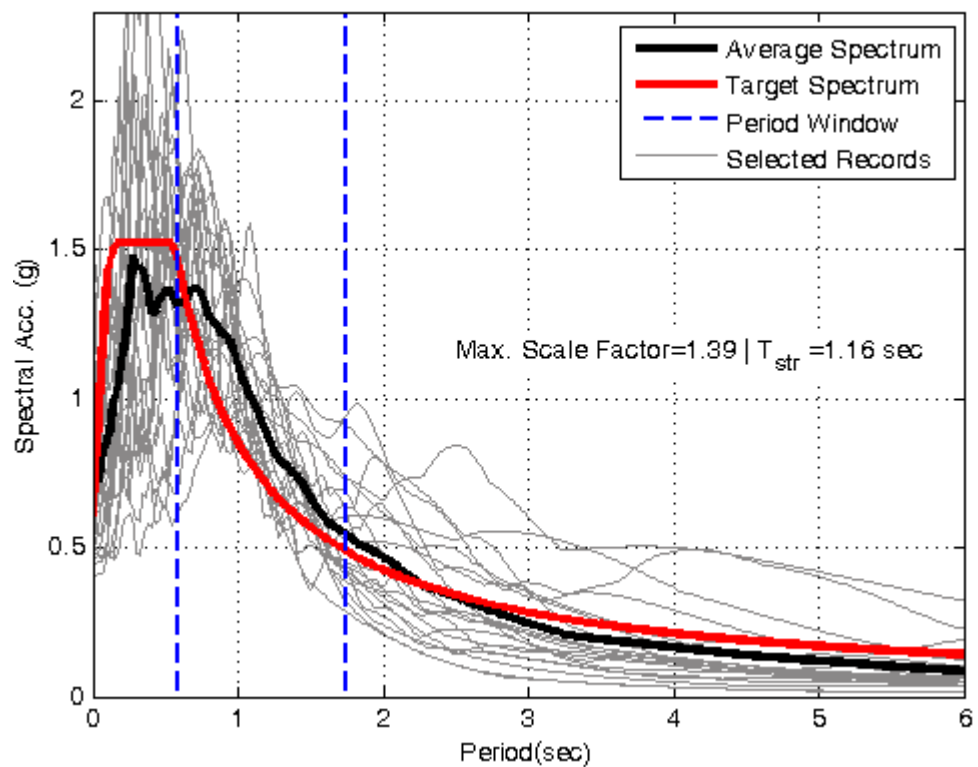


Figure A.12 : Average vs. target spectrum CW16H - 2% in 50 years.

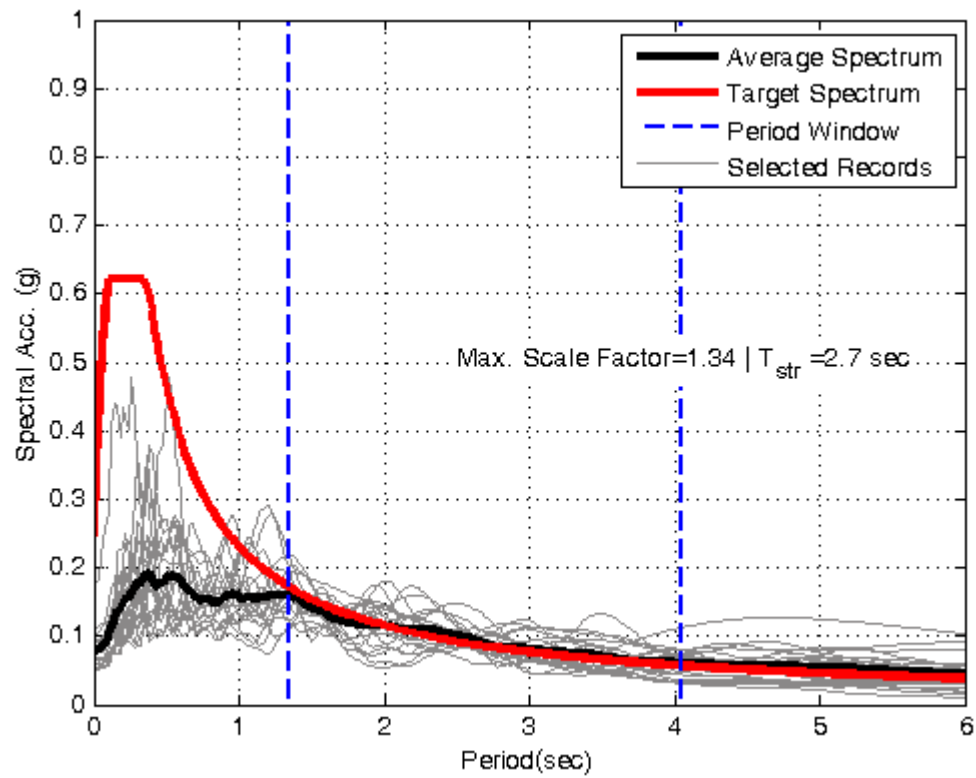


Figure A.13 : Average vs. target spectrum CW24L - 50% in 50 years.

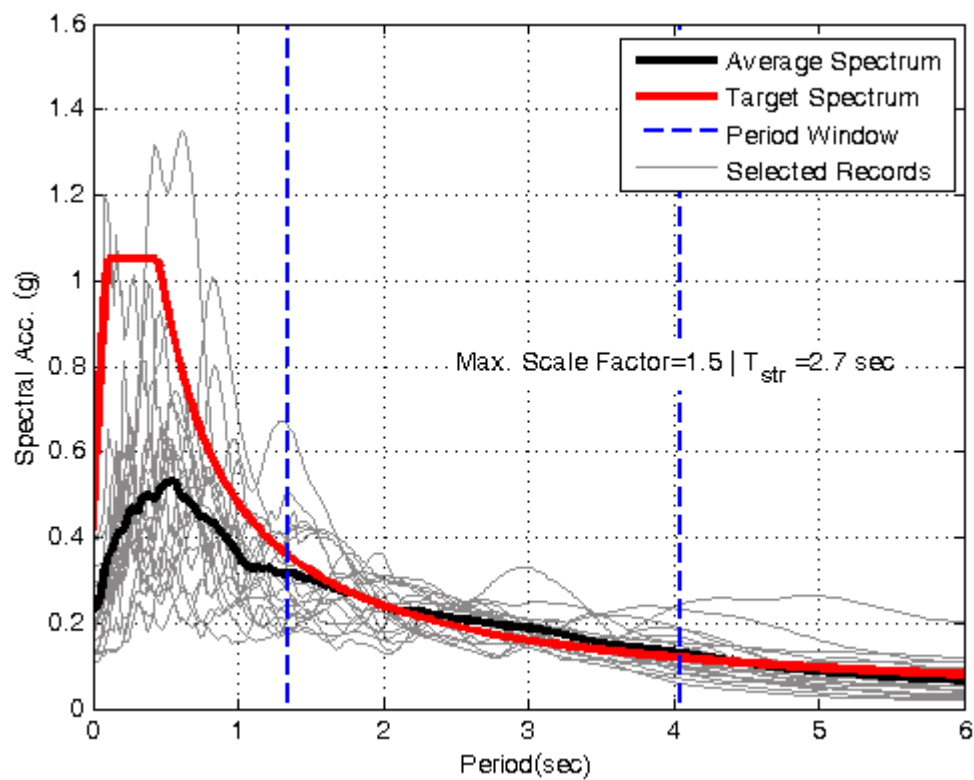


Figure A.14 : Average vs. target spectrum CW24L - 10% in 50 years.

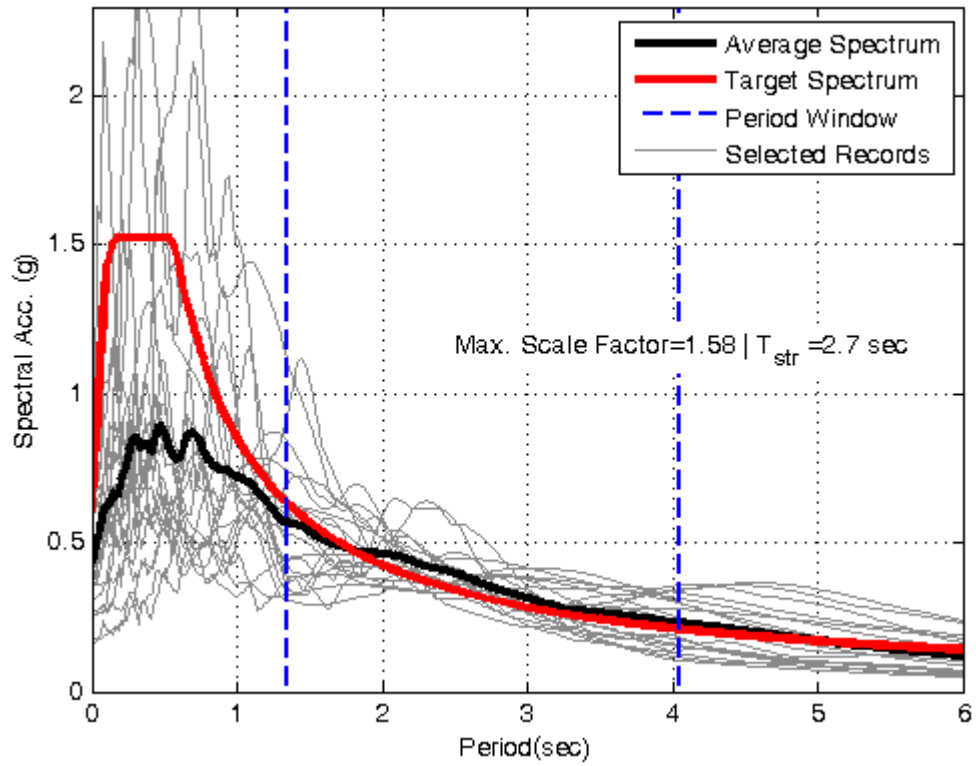


Figure A.15 : Average vs. target spectrum CW24L - 2% in 50 years.

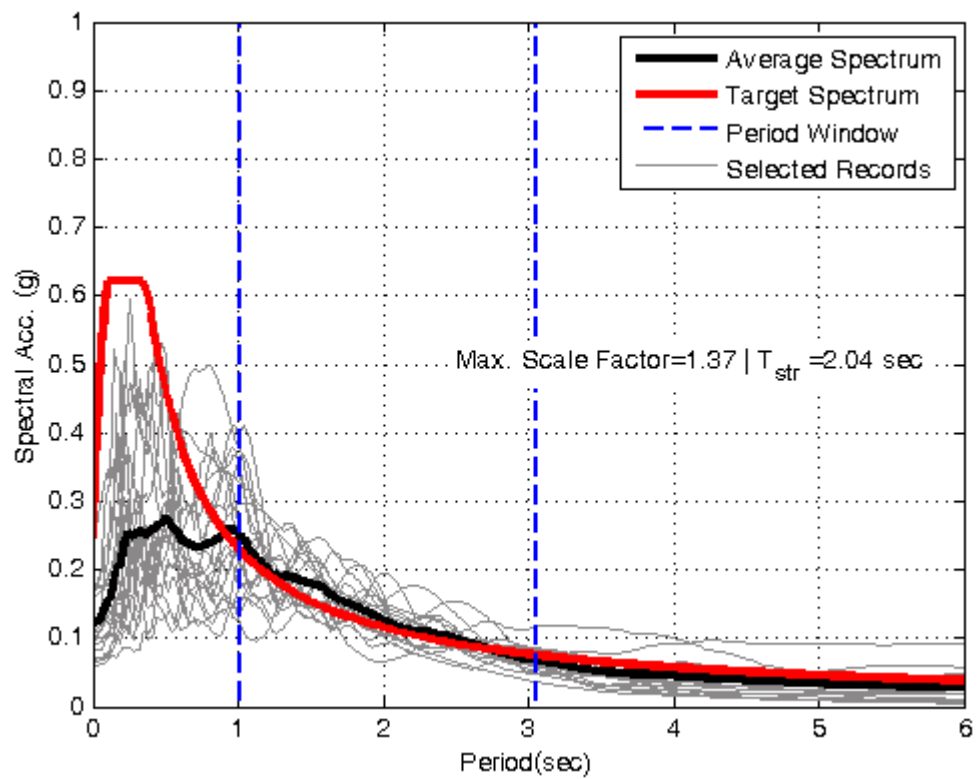


Figure A.16 : Average vs. target spectrum CW24N - 50% in 50 years.

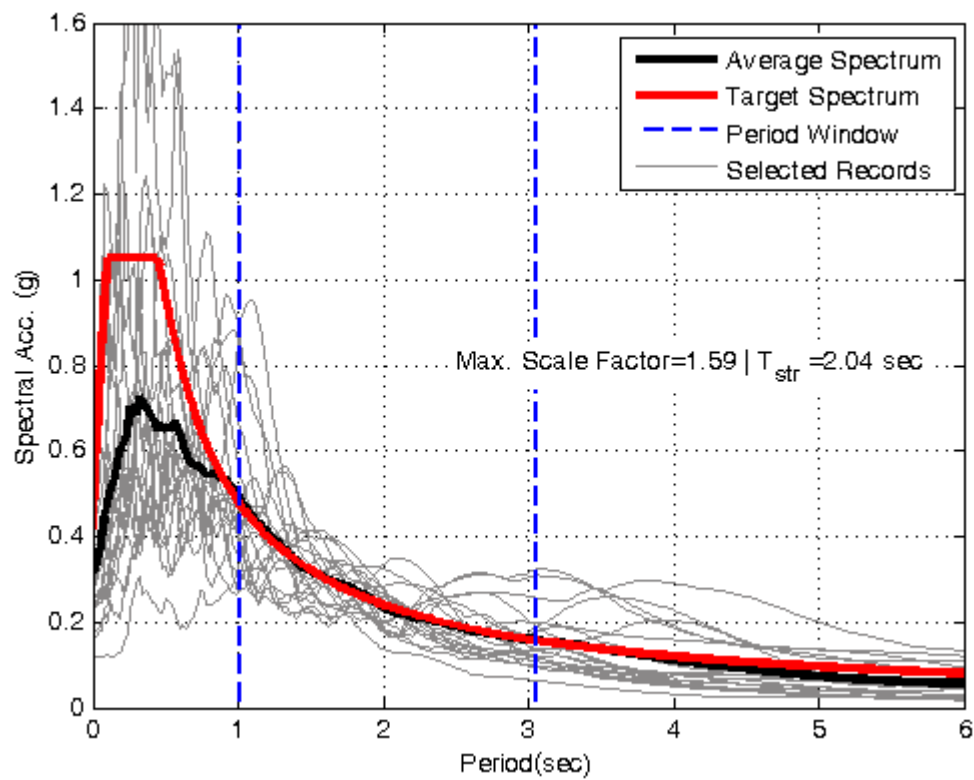


Figure A.17 : Average vs. target spectrum CW24N - 10% in 50 years.

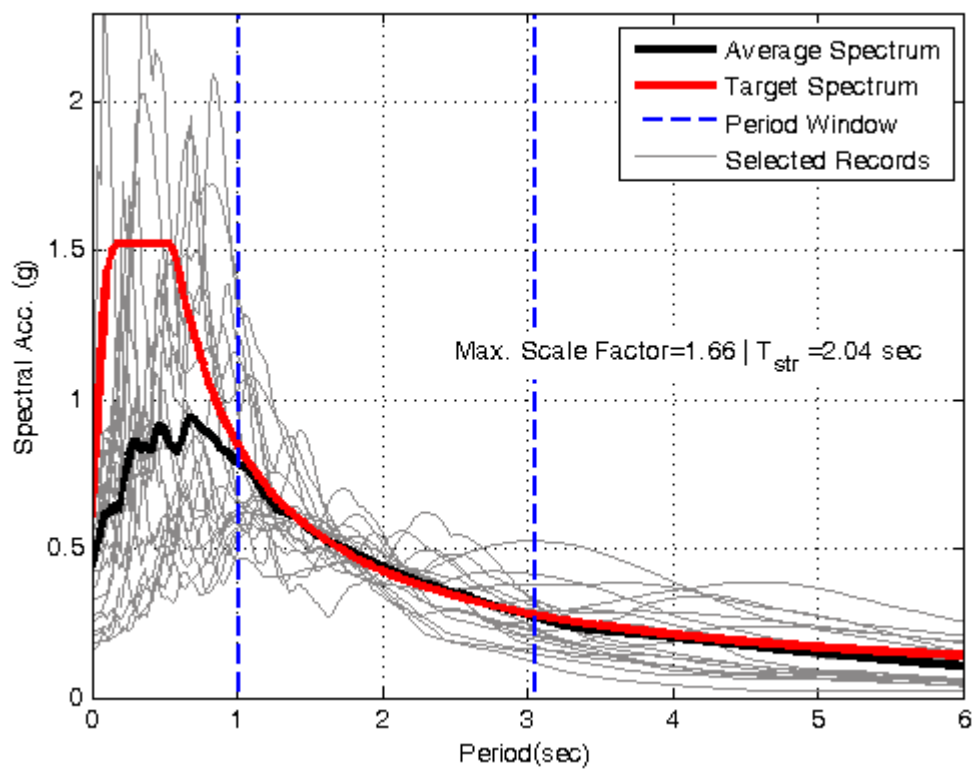


Figure A.18 : Average vs. target spectrum CW24N - 2% in 50 years.

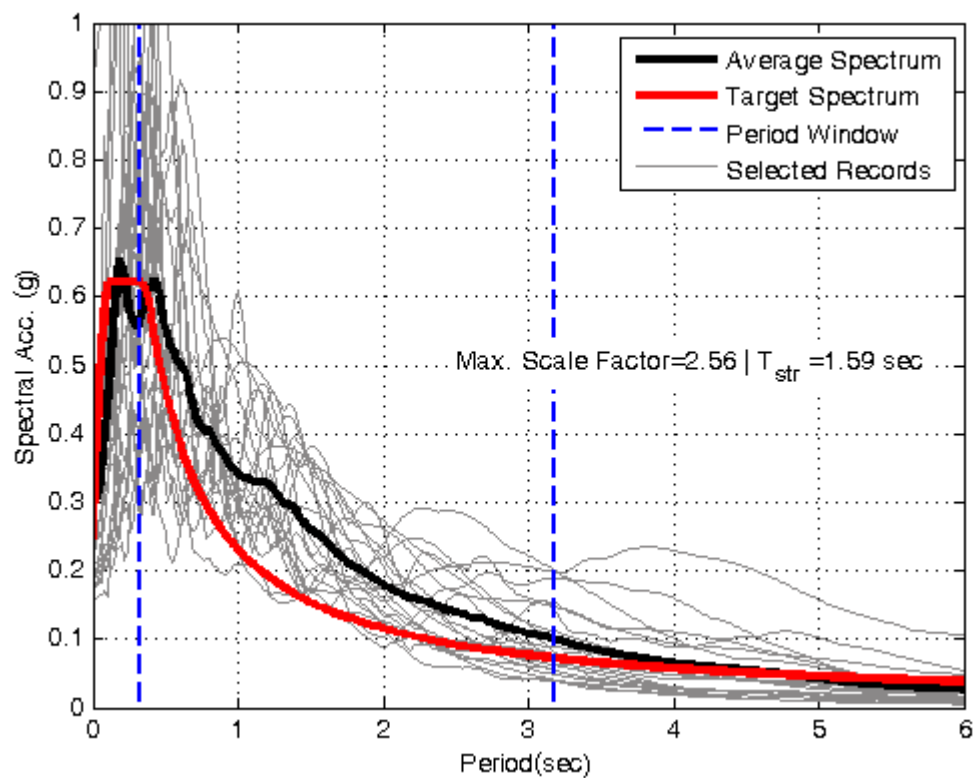


Figure A.19 : Average vs. target spectrum CW24H - 50% in 50 years.

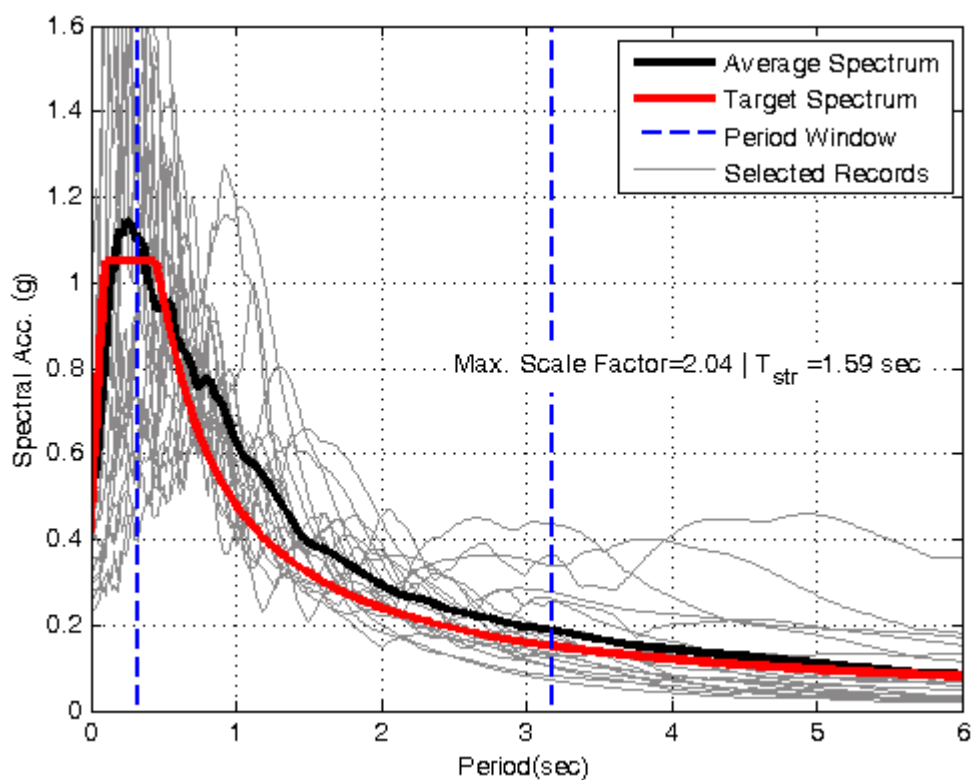


Figure A.20 : Average vs. target spectrum CW24H - 10% in 50 years.

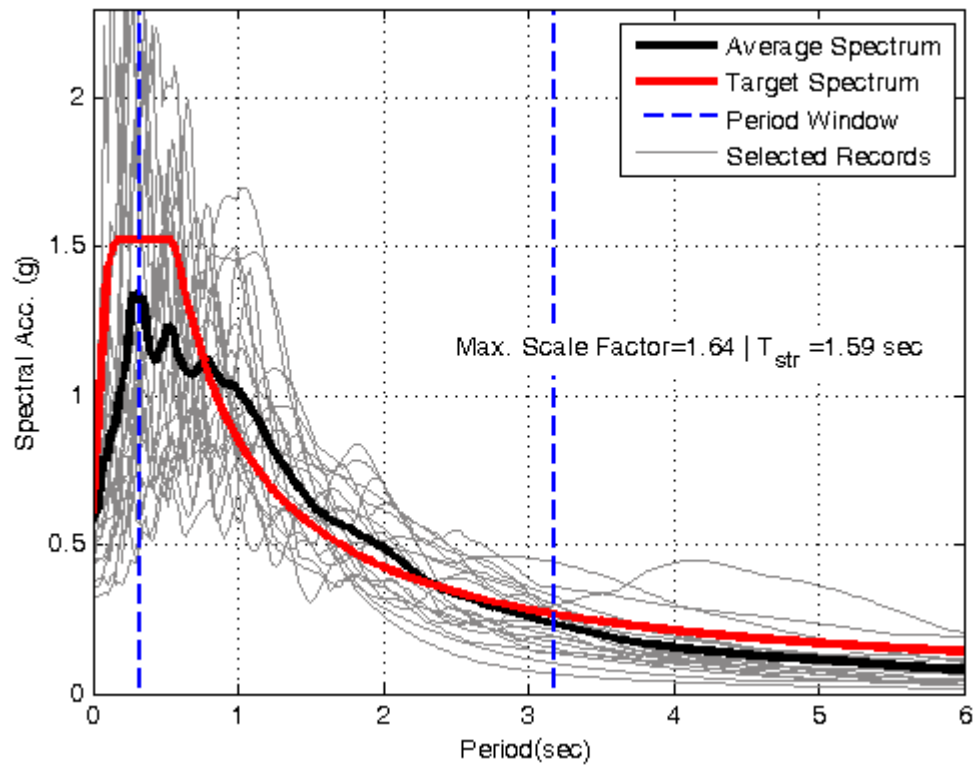


Figure A.21 : Average vs. target spectrum CW24H - 2% in 50 years.

APPENDIX B : OpenSees Codes

NodeCoord.tcl

```
# Node  tag  xCrd  yCrd
node    1  +0.000000E+00 +0.000000E+00
node    2  +0.000000E+00 +3.500000E+00
node    3  +0.000000E+00 +7.000000E+00
node    4  +0.000000E+00 +1.050000E+01
node    5  +0.000000E+00 +1.400000E+01
node    6  +0.000000E+00 +1.750000E+01
node    7  +0.000000E+00 +2.100000E+01
node    8  +0.000000E+00 +2.450000E+01
node    9  +0.000000E+00 +2.800000E+01
node   10  +3.000000E+00 +3.500000E+00
node   11  +3.000000E+00 +7.000000E+00
node   12  +3.000000E+00 +1.050000E+01
node   13  +3.000000E+00 +1.400000E+01
node   14  +3.000000E+00 +1.750000E+01
node   15  +3.000000E+00 +2.100000E+01
node   16  +3.000000E+00 +2.450000E+01
node   17  +3.000000E+00 +2.800000E+01
node   18  +3.000000E+00 +3.500000E+00
node   19  +3.000000E+00 +7.000000E+00
node   20  +3.000000E+00 +1.050000E+01
node   21  +3.000000E+00 +1.400000E+01
node   22  +3.000000E+00 +1.750000E+01
node   23  +3.000000E+00 +2.100000E+01
node   24  +3.000000E+00 +2.450000E+01
node   25  +3.000000E+00 +2.800000E+01
node   26  +4.500000E+00 +3.500000E+00
node   27  +4.500000E+00 +7.000000E+00
node   28  +4.500000E+00 +1.050000E+01
node   29  +4.500000E+00 +1.400000E+01
node   30  +4.500000E+00 +1.750000E+01
node   31  +4.500000E+00 +2.100000E+01
node   32  +4.500000E+00 +2.450000E+01
node   33  +4.500000E+00 +2.800000E+01
```

```

node 34 +4.500000E+00 +3.500000E+00
node 35 +4.500000E+00 +7.000000E+00
node 36 +4.500000E+00 +1.050000E+01
node 37 +4.500000E+00 +1.400000E+01
node 38 +4.500000E+00 +1.750000E+01
node 39 +4.500000E+00 +2.100000E+01
node 40 +4.500000E+00 +2.450000E+01
node 41 +4.500000E+00 +2.800000E+01
node 50 +7.500000E+00 +0.000000E+00
node 51 +7.500000E+00 +3.500000E+00
node 52 +7.500000E+00 +7.000000E+00
node 53 +7.500000E+00 +1.050000E+01
node 54 +7.500000E+00 +1.400000E+01
node 55 +7.500000E+00 +1.750000E+01
node 56 +7.500000E+00 +2.100000E+01
node 57 +7.500000E+00 +2.450000E+01
node 58 +7.500000E+00 +2.800000E+01

```

SPCConstraint.tcl

```
# SPC tag Dx Dy Rz
```

```
fix 1 1 1 1
```

```
fix 50 1 1 1
```

NodeMass.tcl

```
# Mass tag mx my mlz
```

```

mass 2 +1.760000E+02 +0.000000E+00 +0.000000E+00
mass 3 +1.760000E+02 +0.000000E+00 +0.000000E+00
mass 4 +1.760000E+02 +0.000000E+00 +0.000000E+00
mass 5 +1.760000E+02 +0.000000E+00 +0.000000E+00
mass 6 +1.760000E+02 +0.000000E+00 +0.000000E+00
mass 7 +1.760000E+02 +0.000000E+00 +0.000000E+00
mass 8 +1.760000E+02 +0.000000E+00 +0.000000E+00
mass 9 +1.760000E+02 +0.000000E+00 +0.000000E+00
mass 51 +1.760000E+02 +0.000000E+00 +0.000000E+00
mass 52 +1.760000E+02 +0.000000E+00 +0.000000E+00
mass 53 +1.760000E+02 +0.000000E+00 +0.000000E+00
mass 54 +1.760000E+02 +0.000000E+00 +0.000000E+00
mass 55 +1.760000E+02 +0.000000E+00 +0.000000E+00
mass 56 +1.760000E+02 +0.000000E+00 +0.000000E+00
mass 57 +1.760000E+02 +0.000000E+00 +0.000000E+00

```

mass 58 +1.760000E+02 +0.000000E+00 +0.000000E+00

MPCConstraint.tcl

Equal DOF: 10: mNodeTag sNodeTag dof
equalDOF 27 35 1 2

Equal DOF: 11: mNodeTag sNodeTag dof
equalDOF 28 36 1 2

Equal DOF: 12: mNodeTag sNodeTag dof
equalDOF 29 37 1 2

Equal DOF: 13: mNodeTag sNodeTag dof
equalDOF 30 38 1 2

Equal DOF: 14: mNodeTag sNodeTag dof
equalDOF 31 39 1 2

Equal DOF: 15: mNodeTag sNodeTag dof
equalDOF 32 40 1 2

Equal DOF: 16: mNodeTag sNodeTag dof
equalDOF 33 41 1 2

Equal DOF: 2: mNodeTag sNodeTag dof
equalDOF 19 11 1 2

Equal DOF: 3: mNodeTag sNodeTag dof
equalDOF 20 12 1 2

Equal DOF: 4: mNodeTag sNodeTag dof
equalDOF 21 13 1 2

Equal DOF: 5: mNodeTag sNodeTag dof
equalDOF 22 14 1 2

Equal DOF: 6: mNodeTag sNodeTag dof
equalDOF 23 15 1 2

```
# Equal DOF: 7:  mNodeTag  sNodeTag  dof
equalDOF    24    16 1 2
```

```
# Equal DOF: 8:  mNodeTag  sNodeTag  dof
equalDOF    25    17 1 2
```

```
# Equal DOF: 9:  mNodeTag  sNodeTag  dof
equalDOF    26    34 1 2
```

```
# Equal DOF: MPCConstraint01:  mNodeTag  sNodeTag  dof
equalDOF    18    10 1 2
# Materials.tcl
```

```
# Material "ElasticDefault":  matTag  E  eta
uniaxialMaterial Elastic    1 +2.900000E+04 +0.000000E+00
```

```
# Material "wall_hys":  matTag  s1p  e1p  s2p  e2p  s3p  e3p  s1n  e1n  s2n  e2n  s3n
e3n  pinchX  pinchY  D1  D2  beta
uniaxialMaterial Hysteretic    2 +3.300000E+04 +1.800000E-04 +3.500000E+04 +1.100000E-03
+1.320000E+04 +1.300000E-03 -3.300000E+04 -1.800000E-04 -3.500000E+04 -1.100000E-03 -
1.320000E+04 -1.300000E-03 +1.000000E+00 +1.000000E+00 +0.000000E+00 +0.000000E+00
+0.000000E+00
```

```
# Material "wall_axial":  matTag  E  eta
uniaxialMaterial Elastic    3 +8.160000E+07 +0.000000E+00
```

```
# Material "Wall_shear":  matTag  E  eta
uniaxialMaterial Elastic    4 +3.400000E+07 +0.000000E+00
```

```
# Material "cb_sec_nl":  matTag  Fy  E  b  a1  a2  a3  a4
uniaxialMaterial Steel01    5 +2.000000E+02 +1.224000E+08 +1.000000E-04 +0.000000E+00
+1.000000E+00 +0.000000E+00 +1.000000E+00
```

```
# Material "cb_axial":  matTag  E  eta
uniaxialMaterial Elastic    6 +8.160000E+06 +0.000000E+00
```

```
# Material "cb_shear":  matTag  E  eta
uniaxialMaterial Elastic    7 +3.400000E+06 +0.000000E+00
```

```
# Material "wall_sec_nl":  matTag  Fy  E  b  a1  a2  a3  a4
uniaxialMaterial Steel01    8 +3.300000E+04 +1.224000E+08 +1.000000E-04 +0.000000E+00
+1.000000E+00 +0.000000E+00 +1.000000E+00
```

```

# Material "C40_con": matTag fc' epsc0 fcu' epsu
uniaxialMaterial Concrete01 9 -4.500000E+04 -5.000000E-03 -4.200000E+04 -1.600000E-02

# Material "C40_hys_con": matTag s1p e1p s2p e2p s3p e3p s1n e1n s2n e2n s3n
e3n pinchX pinchY D1 D2 beta
uniaxialMaterial Hysteretic 10 +4.000000E+02 +2.000000E-03 +4.400000E+02 +5.000000E-03
+4.398000E+02 +1.600000E-02 -4.000000E+04 -2.000000E-03 -4.400000E+04 -5.000000E-03 -
4.398000E+04 -1.600000E-02 +1.000000E+00 +1.000000E+00 +0.000000E+00 +0.000000E+00
+0.000000E+00

# Material "C40_hys_unc": matTag s1p e1p s2p e2p s3p e3p s1n e1n s2n e2n s3n
e3n pinchX pinchY D1 D2 beta
uniaxialMaterial Hysteretic 11 +4.000000E+02 +2.000000E-03 +3.000000E+02 +4.000000E-03
+3.000000E+00 +5.000000E-03 -4.000000E+04 -2.000000E-03 -3.000000E+04 -4.000000E-03 -
3.000000E+02 -5.000000E-03 +1.000000E+00 +1.000000E+00 +0.000000E+00 +0.000000E+00
+0.000000E+00

# Material "S420": matTag Fy E b a1 a2 a3 a4
uniaxialMaterial Steel01 12 +4.200000E+05 +2.000000E+08 +1.000000E-03 +0.000000E+00
+0.000000E+00 +0.000000E+00 +0.000000E+00

# Material "C40": matTag fc' epsc0 fcu' epsu
uniaxialMaterial Concrete01 13 -5.200000E+04 -2.000000E-03 +0.000000E+00 -5.000000E-03

# Material "C40_con2": matTag fc' epsc0 fcu' epsu lambda ft Ets
uniaxialMaterial Concrete02 14 -6.500000E+04 -5.000000E-03 -4.530000E+04 -1.400000E-02
+5.000000E-01 +6.500000E+03 +3.252000E+03

# Material "CB_hys": matTag s1p e1p s2p e2p s3p e3p s1n e1n s2n e2n s3n e3n
pinchX pinchY D1 D2 beta
uniaxialMaterial Hysteretic 15 +6.710000E+02 +3.920000E-10 +6.710000E+02 +4.500000E-02
+2.010000E+02 +5.500000E-02 -6.710000E+02 -3.920000E-10 -6.710000E+02 -4.500000E-02 -
2.010000E+02 -5.500000E-02 +0.000000E+00 +0.000000E+00 +0.000000E+00 +0.000000E+00
+0.000000E+00

# Material "S420_a": matTag Fy E b R0 cR1 cR2 <a1 a2 a3 a4> <sig0>
uniaxialMaterial Steel02 16 +4.914000E+05 +2.000000E+08 +1.000000E-07 +1.850000E+01
+9.250000E-01 +1.500000E-01 +0.000000E+00 +1.000000E+00 +0.000000E+00 +1.000000E+00
+0.000000E+00

# Element "wall_nl": eleTag NodeI NodeJ NIP secTag geoTranTag <-mass massDens>
<-iter maxIters tol>
element nonlinearBeamColumn 1 1 2 5 7 1 -mass +0.000000E+00 -iter 10
+1.000000E-12

```

```
# Element "wall_nl":  eleTag  NodeI  NodeJ  NIP  secTag  geoTranTag  <-mass massDens>
<-iter maxIters tol>

element nonlinearBeamColumn    2    2    3    5    7    1  -mass +0.000000E+00 -iter  10
+1.000000E-12
```

```
# Element "wall_nl":  eleTag  NodeI  NodeJ  NIP  secTag  geoTranTag  <-mass massDens>
<-iter maxIters tol>

element nonlinearBeamColumn    3    3    4    5    7    1  -mass +0.000000E+00 -iter  10
+1.000000E-12
```

```
# Element "wall_nl":  eleTag  NodeI  NodeJ  NIP  secTag  geoTranTag  <-mass massDens>
<-iter maxIters tol>

element nonlinearBeamColumn    4    4    5    5    7    1  -mass +0.000000E+00 -iter  10
+1.000000E-12
```

```
# Element "wall_nl":  eleTag  NodeI  NodeJ  NIP  secTag  geoTranTag  <-mass massDens>
<-iter maxIters tol>

element nonlinearBeamColumn    5    5    6    5    7    1  -mass +0.000000E+00 -iter  10
+1.000000E-12
```

```
# Element "wall_nl":  eleTag  NodeI  NodeJ  NIP  secTag  geoTranTag  <-mass massDens>
<-iter maxIters tol>

element nonlinearBeamColumn    6    6    7    5    7    1  -mass +0.000000E+00 -iter  10
+1.000000E-12
```

```
# Element "wall_nl":  eleTag  NodeI  NodeJ  NIP  secTag  geoTranTag  <-mass massDens>
<-iter maxIters tol>

element nonlinearBeamColumn    7    7    8    5    7    1  -mass +0.000000E+00 -iter  10
+1.000000E-12
```

```
# Element "wall_nl":  eleTag  NodeI  NodeJ  NIP  secTag  geoTranTag  <-mass massDens>
<-iter maxIters tol>

element nonlinearBeamColumn    8    8    9    5    7    1  -mass +0.000000E+00 -iter  10
+1.000000E-12
```

```
# Element "RIGID":    eleTag  NodeI  NodeJ  A    E    Iz  geoTranTag  <alpha d>  <-mass
massDens>

element elasticBeamColumn    9    2    18 +1.000000E+03 +3.400000E+10 +1.000000E+08  1
+0.000000E+00 +0.000000E+00 -mass +0.000000E+00
```

```
# Element "RIGID":    eleTag  NodeI  NodeJ  A    E    Iz  geoTranTag  <alpha d>  <-mass
massDens>

element elasticBeamColumn   10    3    19 +1.000000E+03 +3.400000E+10 +1.000000E+08  1
+0.000000E+00 +0.000000E+00 -mass +0.000000E+00
```

```

# Element "RIGID":  eleTag  NodeI  NodeJ  A  E  Iz  geoTranTag  <alpha d>  <-mass
massDens>

element elasticBeamColumn  11   4   20 +1.000000E+03 +3.400000E+10 +1.000000E+08  1
+0.000000E+00 +0.000000E+00 -mass +0.000000E+00

# Element "RIGID":  eleTag  NodeI  NodeJ  A  E  Iz  geoTranTag  <alpha d>  <-mass
massDens>

element elasticBeamColumn  12   5   21 +1.000000E+03 +3.400000E+10 +1.000000E+08  1
+0.000000E+00 +0.000000E+00 -mass +0.000000E+00

# Element "RIGID":  eleTag  NodeI  NodeJ  A  E  Iz  geoTranTag  <alpha d>  <-mass
massDens>

element elasticBeamColumn  13   6   22 +1.000000E+03 +3.400000E+10 +1.000000E+08  1
+0.000000E+00 +0.000000E+00 -mass +0.000000E+00

# Element "RIGID":  eleTag  NodeI  NodeJ  A  E  Iz  geoTranTag  <alpha d>  <-mass
massDens>

element elasticBeamColumn  14   7   23 +1.000000E+03 +3.400000E+10 +1.000000E+08  1
+0.000000E+00 +0.000000E+00 -mass +0.000000E+00

# Element "RIGID":  eleTag  NodeI  NodeJ  A  E  Iz  geoTranTag  <alpha d>  <-mass
massDens>

element elasticBeamColumn  15   8   24 +1.000000E+03 +3.400000E+10 +1.000000E+08  1
+0.000000E+00 +0.000000E+00 -mass +0.000000E+00

# Element "RIGID":  eleTag  NodeI  NodeJ  A  E  Iz  geoTranTag  <alpha d>  <-mass
massDens>

element elasticBeamColumn  16   9   25 +1.000000E+03 +3.400000E+10 +1.000000E+08  1
+0.000000E+00 +0.000000E+00 -mass +0.000000E+00

# Element "wall_nl":  eleTag  NodeI  NodeJ  NIP  secTag  geoTranTag  <-mass massDens>
<-iter maxIters tol>

element nonlinearBeamColumn  17   50   51   5   7   1 -mass +0.000000E+00 -iter  10
+1.000000E-12

# Element "wall_nl":  eleTag  NodeI  NodeJ  NIP  secTag  geoTranTag  <-mass massDens>
<-iter maxIters tol>

element nonlinearBeamColumn  18   51   52   5   7   1 -mass +0.000000E+00 -iter  10
+1.000000E-12

# Element "wall_nl":  eleTag  NodeI  NodeJ  NIP  secTag  geoTranTag  <-mass massDens>
<-iter maxIters tol>

element nonlinearBeamColumn  19   52   53   5   7   1 -mass +0.000000E+00 -iter  10
+1.000000E-12

```

```

# Element "wall_nl":  eleTag  NodeI  NodeJ  NIP  secTag  geoTranTag  <-mass massDens>
<-iter maxIters tol>

element nonlinearBeamColumn    20    53    54    5    7    1  -mass +0.000000E+00 -iter  10
+1.000000E-12

# Element "wall_nl":  eleTag  NodeI  NodeJ  NIP  secTag  geoTranTag  <-mass massDens>
<-iter maxIters tol>

element nonlinearBeamColumn    21    54    55    5    7    1  -mass +0.000000E+00 -iter  10
+1.000000E-12

# Element "wall_nl":  eleTag  NodeI  NodeJ  NIP  secTag  geoTranTag  <-mass massDens>
<-iter maxIters tol>

element nonlinearBeamColumn    22    55    56    5    7    1  -mass +0.000000E+00 -iter  10
+1.000000E-12

# Element "wall_nl":  eleTag  NodeI  NodeJ  NIP  secTag  geoTranTag  <-mass massDens>
<-iter maxIters tol>

element nonlinearBeamColumn    23    56    57    5    7    1  -mass +0.000000E+00 -iter  10
+1.000000E-12

# Element "wall_nl":  eleTag  NodeI  NodeJ  NIP  secTag  geoTranTag  <-mass massDens>
<-iter maxIters tol>

element nonlinearBeamColumn    24    57    58    5    7    1  -mass +0.000000E+00 -iter  10
+1.000000E-12

# Element "CB hinge":  eleTag  NodeI  NodeJ  SecID  <-orient x1 x2 x3 y1 y2 y3>  <-
doRayleigh>

element zeroLengthSection    25    18    10    6  -orient +1.000000E+00 +0.000000E+00
+0.000000E+00 +0.000000E+00 +1.000000E+00 +0.000000E+00 -doRayleigh

# Element "CB hinge":  eleTag  NodeI  NodeJ  SecID  <-orient x1 x2 x3 y1 y2 y3>  <-
doRayleigh>

element zeroLengthSection    26    19    11    6  -orient +1.000000E+00 +0.000000E+00
+0.000000E+00 +0.000000E+00 +1.000000E+00 +0.000000E+00 -doRayleigh

# Element "CB hinge":  eleTag  NodeI  NodeJ  SecID  <-orient x1 x2 x3 y1 y2 y3>  <-
doRayleigh>

element zeroLengthSection    27    20    12    6  -orient +1.000000E+00 +0.000000E+00
+0.000000E+00 +0.000000E+00 +1.000000E+00 +0.000000E+00 -doRayleigh

# Element "CB hinge":  eleTag  NodeI  NodeJ  SecID  <-orient x1 x2 x3 y1 y2 y3>  <-
doRayleigh>

element zeroLengthSection    28    21    13    6  -orient +1.000000E+00 +0.000000E+00
+0.000000E+00 +0.000000E+00 +1.000000E+00 +0.000000E+00 -doRayleigh

```

```

# Element "CB hinge":  eleTag  NodeI  NodeJ  SecID  <-orient x1 x2 x3 y1 y2 y3>  <-
doRayleigh>
element zeroLengthSection  29    22    14    6 -orient +1.000000E+00 +0.000000E+00
+0.000000E+00 +0.000000E+00 +1.000000E+00 +0.000000E+00 -doRayleigh

# Element "CB hinge":  eleTag  NodeI  NodeJ  SecID  <-orient x1 x2 x3 y1 y2 y3>  <-
doRayleigh>
element zeroLengthSection  30    23    15    6 -orient +1.000000E+00 +0.000000E+00
+0.000000E+00 +0.000000E+00 +1.000000E+00 +0.000000E+00 -doRayleigh

# Element "CB hinge":  eleTag  NodeI  NodeJ  SecID  <-orient x1 x2 x3 y1 y2 y3>  <-
doRayleigh>
element zeroLengthSection  31    24    16    6 -orient +1.000000E+00 +0.000000E+00
+0.000000E+00 +0.000000E+00 +1.000000E+00 +0.000000E+00 -doRayleigh

# Element "CB hinge":  eleTag  NodeI  NodeJ  SecID  <-orient x1 x2 x3 y1 y2 y3>  <-
doRayleigh>
element zeroLengthSection  32    25    17    6 -orient +1.000000E+00 +0.000000E+00
+0.000000E+00 +0.000000E+00 +1.000000E+00 +0.000000E+00 -doRayleigh

# Element "CB":  eleTag  NodeI  NodeJ  A  E  Iz  geoTranTag  <alpha d>  <-mass massDens>
element elasticBeamColumn  33    10    34 +2.400000E-01 +3.400000E+07 +8.930000E-04  1
+0.000000E+00 +0.000000E+00 -mass +0.000000E+00

# Element "CB":  eleTag  NodeI  NodeJ  A  E  Iz  geoTranTag  <alpha d>  <-mass massDens>
element elasticBeamColumn  34    11    35 +2.400000E-01 +3.400000E+07 +8.930000E-04  1
+0.000000E+00 +0.000000E+00 -mass +0.000000E+00

# Element "CB":  eleTag  NodeI  NodeJ  A  E  Iz  geoTranTag  <alpha d>  <-mass massDens>
element elasticBeamColumn  35    12    36 +2.400000E-01 +3.400000E+07 +8.930000E-04  1
+0.000000E+00 +0.000000E+00 -mass +0.000000E+00

# Element "CB":  eleTag  NodeI  NodeJ  A  E  Iz  geoTranTag  <alpha d>  <-mass massDens>
element elasticBeamColumn  36    13    37 +2.400000E-01 +3.400000E+07 +8.930000E-04  1
+0.000000E+00 +0.000000E+00 -mass +0.000000E+00

# Element "CB":  eleTag  NodeI  NodeJ  A  E  Iz  geoTranTag  <alpha d>  <-mass massDens>
element elasticBeamColumn  37    14    38 +2.400000E-01 +3.400000E+07 +8.930000E-04  1
+0.000000E+00 +0.000000E+00 -mass +0.000000E+00

# Element "CB":  eleTag  NodeI  NodeJ  A  E  Iz  geoTranTag  <alpha d>  <-mass massDens>
element elasticBeamColumn  38    15    39 +2.400000E-01 +3.400000E+07 +8.930000E-04  1
+0.000000E+00 +0.000000E+00 -mass +0.000000E+00

# Element "CB":  eleTag  NodeI  NodeJ  A  E  Iz  geoTranTag  <alpha d>  <-mass massDens>

```

```

element elasticBeamColumn 39 16 40 +2.400000E-01 +3.400000E+07 +8.930000E-04 1
+0.000000E+00 +0.000000E+00 -mass +0.000000E+00

```

```

# Element "CB": eleTag NodeI NodeJ A E Iz geoTranTag <alpha d> <-mass massDens>
element elasticBeamColumn 40 17 41 +2.400000E-01 +3.400000E+07 +8.930000E-04 1
+0.000000E+00 +0.000000E+00 -mass +0.000000E+00

```

```

# Element "CB hinge": eleTag NodeI NodeJ SecID <-orient x1 x2 x3 y1 y2 y3> <-
doRayleigh>
element zeroLengthSection 41 34 26 6 -orient +1.000000E+00 +0.000000E+00
+0.000000E+00 +0.000000E+00 +1.000000E+00 +0.000000E+00 -doRayleigh

```

```

# Element "CB hinge": eleTag NodeI NodeJ SecID <-orient x1 x2 x3 y1 y2 y3> <-
doRayleigh>
element zeroLengthSection 42 35 27 6 -orient +1.000000E+00 +0.000000E+00
+0.000000E+00 +0.000000E+00 +1.000000E+00 +0.000000E+00 -doRayleigh

```

```

# Element "CB hinge": eleTag NodeI NodeJ SecID <-orient x1 x2 x3 y1 y2 y3> <-
doRayleigh>
element zeroLengthSection 43 36 28 6 -orient +1.000000E+00 +0.000000E+00
+0.000000E+00 +0.000000E+00 +1.000000E+00 +0.000000E+00 -doRayleigh

```

```

# Element "CB hinge": eleTag NodeI NodeJ SecID <-orient x1 x2 x3 y1 y2 y3> <-
doRayleigh>
element zeroLengthSection 44 37 29 6 -orient +1.000000E+00 +0.000000E+00
+0.000000E+00 +0.000000E+00 +1.000000E+00 +0.000000E+00 -doRayleigh

```

```

# Element "CB hinge": eleTag NodeI NodeJ SecID <-orient x1 x2 x3 y1 y2 y3> <-
doRayleigh>
element zeroLengthSection 45 38 30 6 -orient +1.000000E+00 +0.000000E+00
+0.000000E+00 +0.000000E+00 +1.000000E+00 +0.000000E+00 -doRayleigh

```

```

# Element "CB hinge": eleTag NodeI NodeJ SecID <-orient x1 x2 x3 y1 y2 y3> <-
doRayleigh>
element zeroLengthSection 46 39 31 6 -orient +1.000000E+00 +0.000000E+00
+0.000000E+00 +0.000000E+00 +1.000000E+00 +0.000000E+00 -doRayleigh

```

```

# Element "CB hinge": eleTag NodeI NodeJ SecID <-orient x1 x2 x3 y1 y2 y3> <-
doRayleigh>
element zeroLengthSection 47 40 32 6 -orient +1.000000E+00 +0.000000E+00
+0.000000E+00 +0.000000E+00 +1.000000E+00 +0.000000E+00 -doRayleigh

```

```

# Element "CB hinge": eleTag NodeI NodeJ SecID <-orient x1 x2 x3 y1 y2 y3> <-
doRayleigh>
element zeroLengthSection 48 41 33 6 -orient +1.000000E+00 +0.000000E+00
+0.000000E+00 +0.000000E+00 +1.000000E+00 +0.000000E+00 -doRayleigh

```

Element "RIGID": eleTag NodeI NodeJ A E Iz geoTranTag <alpha d> <-mass
massDens>

element elasticBeamColumn 49 26 51 +1.000000E+03 +3.400000E+10 +1.000000E+08
1 +0.000000E+00 +0.000000E+00 -mass +0.000000E+00

Element "RIGID": eleTag NodeI NodeJ A E Iz geoTranTag <alpha d> <-mass
massDens>

element elasticBeamColumn 50 27 52 +1.000000E+03 +3.400000E+10 +1.000000E+08
1 +0.000000E+00 +0.000000E+00 -mass +0.000000E+00

Element "RIGID": eleTag NodeI NodeJ A E Iz geoTranTag <alpha d> <-mass
massDens>

element elasticBeamColumn 51 28 53 +1.000000E+03 +3.400000E+10 +1.000000E+08
1 +0.000000E+00 +0.000000E+00 -mass +0.000000E+00

Element "RIGID": eleTag NodeI NodeJ A E Iz geoTranTag <alpha d> <-mass
massDens>

element elasticBeamColumn 52 29 54 +1.000000E+03 +3.400000E+10 +1.000000E+08
1 +0.000000E+00 +0.000000E+00 -mass +0.000000E+00

Element "RIGID": eleTag NodeI NodeJ A E Iz geoTranTag <alpha d> <-mass
massDens>

element elasticBeamColumn 53 30 55 +1.000000E+03 +3.400000E+10 +1.000000E+08
1 +0.000000E+00 +0.000000E+00 -mass +0.000000E+00

Element "RIGID": eleTag NodeI NodeJ A E Iz geoTranTag <alpha d> <-mass
massDens>

element elasticBeamColumn 54 31 56 +1.000000E+03 +3.400000E+10 +1.000000E+08
1 +0.000000E+00 +0.000000E+00 -mass +0.000000E+00

Element "RIGID": eleTag NodeI NodeJ A E Iz geoTranTag <alpha d> <-mass
massDens>

element elasticBeamColumn 55 32 57 +1.000000E+03 +3.400000E+10 +1.000000E+08
1 +0.000000E+00 +0.000000E+00 -mass +0.000000E+00

Element "RIGID": eleTag NodeI NodeJ A E Iz geoTranTag <alpha d> <-mass
massDens>

element elasticBeamColumn 56 33 58 +1.000000E+03 +3.400000E+10 +1.000000E+08
1 +0.000000E+00 +0.000000E+00 -mass +0.000000E+00

CURRICULUM VITAE

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2011 – Present : Arup – Senior Structural Engineer

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PUBLICATIONS ON THE THESIS:

- **Toprak E.,** Gulay F.G., Bal, I.E., 2015 : “Review on the macro-modeling alternatives and a proposal for modeling coupling beams in tall buildings”, *Bulletin of Earthquake Engineering*.
- **Toprak E.,** Gulay F.G., Bal, I.E., 2012: “Numerical Evaluation of Plastic Region Propagation in Coupled Wall Systems”, *15th World Conference on Earthquake Engineering*, Lisbon, Portugal.
- **Toprak E.,** Gulay F.G., Bal, I.E., 2012: “Coupling Ratio Verification of Direct Displacement based Design Procedure for RC Coupled Walls”, *The 2012 International Conference on Advances in Coupled Systems Mechanics ASCM'12*, Seoul, Korea.

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