



**STRUCTURAL BEHAVIOUR OF PRECAST CONCRETE
FRAMES SUBJECT TO COLUMN REMOVAL SCENARIOS**

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TABLE OF CONTENT

Acknowledgement.....	i
Table of Content.....	iii
Abstract.....	ix
List of Figures	xi
List of Tables	xxi
List of Symbols	xxiii
Chapter 1 Introduction.....	1
1.1 Research Background	1
1.2 Alternate Load Paths	2
1.3 Objectives and Scope of Research	4
1.4 Layout of the Thesis	6
Chapter 2 Literature Review	9
2.1 Overview	9
2.2 Design Approaches against Progressive Collapse.....	9
2.2.1 Indirect design approach.....	10
2.2.2 Direct design approach	11
2.2.3 Relationship between indirect and direct approaches.....	12
2.3 Experimental Tests under Progressive Collapse Scenarios	14
2.3.1 Quasi-static tests under column removal scenarios.....	14
2.3.2 Dynamic tests.....	27
2.4 Engineered Cementitious Composites (ECC)	30
2.4.1 Material properties	30
2.4.2 Structural performance under various loading conditions.....	31
2.4.3 Interactions between ECC and reinforcement.....	33

2.4.4 Bond stress of reinforcement embedded in ECC	35
2.5 Component-Based Joint Models	37
2.5.1 Procedure of joint characterisation.....	37
2.5.2 Joint models under cyclic loading.....	38
2.5.3 Joint models under progressive collapse	39
2.6 Summary.....	41
Chapter 3 Experimental Tests of Precast Concrete Beam-Column Sub-Assemblages..	43
3.1 Introduction.....	43
3.2 Test Programme	44
3.2.1 Prototype structure	44
3.2.2 Specimen design	45
3.2.3 Test setup.....	48
3.2.4 Instrumentations.....	50
3.3 Material Properties	51
3.4 Experimental Results of Sub-Assemblages	52
3.4.1 Load-displacement history of beam-column sub-assemblages.....	52
3.4.2 Resistances of beam-column sub-assemblages.....	54
3.4.3 Components of vertical load.....	57
3.4.4 Rotational capacities of beam-column sub-assemblages.....	59
3.4.5 Crack patterns and failure modes of precast beams	63
3.4.6 Horizontal shear transfer between precast beam units and cast-in-situ concrete topping	66
3.4.7 Strains of beam longitudinal reinforcement	68
3.5 Discussions and Suggestions	70
3.6 Conclusions.....	71

Chapter 4 Experimental Tests of Precast Beam-Column Sub-Assemblages with Engineered Cementitious Composites.....	73
4.1 Introduction.....	73
4.2 Experimental Programme on Sub-Assemblages	74
4.2.1 Specimen design.....	74
4.2.3 Material properties	77
4.3 Resistances of Beam-Column Sub-Assemblages	79
4.3.1 Effect of ECC	80
4.3.2 Effect of reinforcement detailing	82
4.3.3 Effect of top reinforcement ratios	83
4.3.4 Effect of bottom reinforcement ratios	84
4.4 Crack Patterns and Failure Modes of Sub-Assemblages	85
4.5 Horizontal Reaction Forces and Bending Moments	88
4.6 Deformation Capacities of Beam-Column Sub-Assemblages	91
4.7 Local Rotations in the Plastic Hinge Region	93
4.8 Interactions between Steel Reinforcement and ECC.....	96
4.9 Conclusions	100
Chapter 5 Experimental Study on Precast Concrete Frames with Different Horizontal Restraints	103
5.1 Introduction.....	103
5.2 Test Programme	104
5.2.1 Frame design and detailing	104
5.2.2 Test setup	108
5.2.3 Instrumentations	110
5.3 Material Properties	111
5.4 Experimental Results of Precast Concrete Frames.....	112

5.4.1 Load-displacement curves	112
5.4.2 Effect of reinforcement detailing on frame behaviour	114
5.4.3 Effect of boundary conditions on frame behaviour.....	116
5.4.4 Pseudo-static resistances of precast concrete frames	116
5.4.5 Load paths of horizontal reaction forces to the support	118
5.4.6 Crack patterns and failure modes of precast beams	120
5.4.7 Behaviour of side columns and joints	122
5.4.8 Variation of steel strains in beams and columns.....	128
5.5 Summary.....	133
Chapter 6 Experimental Study on Exterior Precast Concrete Frames	135
6.1 Introduction.....	135
6.2 Experimental Programme	135
6.2.1 Specimen design and detailing.....	135
6.2.2 Material properties	138
6.3 Test Results of Exterior Frames	139
6.3.1 Load-displacement curves	139
6.3.2 Resistances of precast concrete frames	141
6.3.3 Failure modes of precast frames	142
6.3.4 Lateral deflections of side columns.....	149
6.3.5 Shear strength of beam-column joints	150
6.3.6 Flexural strength of side columns subjected to horizontal tension	152
6.3.7 Variation of steel strain in side joints	155
6.4 Summary.....	158
Chapter 7 Analytical Model for Compressive Arch Action of Beam-Column Sub-Assemblages	161
7.1 Introduction.....	161

7.2 Development of the Analytical Model.....	162
7.2.1 Constitutive models	163
7.2.2 Equilibrium condition.....	167
7.2.3 Compatibility condition.....	171
7.2.4 Solution procedure	175
7.3 Validation of the Analytical Model.....	178
7.3.1 Prediction of CAA capacity and horizontal reaction force	179
7.3.2 Prediction of load-displacement curve	180
7.3.3 Variation of bending moments	183
7.3.4 Estimate of neutral axis depth and reinforcement strain.....	183
7.4 Limitations of the Analytical Model	185
7.5 Parametric Studies	186
7.5.1 Effect of concrete models	186
7.5.2 Effect of tensile strength of ECC	190
7.5.3 Effect of tensile strain capacity of ECC	191
7.5.4 Effect of stiffness of horizontal restraint	192
7.5.5 Effect of reinforcement ratio.....	195
7.6 Conclusion	197
Chapter 8 Component-Based Joint Model for Precast Concrete Beam-Column Sub-Assemblages	201
8.1 Introduction.....	201
8.2 Beam-Column Joint Model.....	202
8.3 Properties of Tensile Spring.....	203
8.3.1 Zero strain with zero slip	204
8.3.2 Zero strain with non-zero slip	212
8.3.3 Non-zero strain with zero slip.....	222

8.4 Properties of Compressive Spring.....	226
8.4.1 Determination of compression force	227
8.4.2 Bond stress in compression	230
8.4.3 Force-slip relationship of compressive spring	231
8.5 Shear Panel Spring	232
8.6 Validation of Joint Model.....	232
8.6.1 Parameters of springs	232
8.6.2 Comparisons with experimental results.....	234
8.7 Discussions	235
8.8 Conclusions.....	236
Chapter 9 Conclusions and Future Work	239
9.1 Conclusions.....	239
9.2 Future Works	244
References	247
Publications	259
Appendix A Quantification of Boundary Conditions	261
A.1 Precast Concrete Beam-Column Sub-Assemblages	261
A.1.1 Horizontal reaction forces.....	261
A.1.2 Stiffness of horizontal restraints.....	263
A.2 Precast Beam-Column Sub-Assemblages with ECC.....	266
A.2.1 Horizontal reaction forces.....	266
A.2.2 Stiffness of horizontal restraints.....	267
A.3 Precast Concrete Frames.....	273
A.3.1 Horizontal reaction forces.....	273
A.3.2 Stiffness of horizontal restraints.....	275

ABSTRACT

From time to time, structural collapse incidents throughout the world prompt research works on the robustness of building structures to mitigate progressive collapse. Among the design approaches, alternate path method tends to prevent the spread of local damage through mobilisation of compressive arch action (CAA) and catenary action in the bridging beam and the floor system. However, development of alternate load path is contingent on structural ductility and integrity at large deformations.

This study aims to investigate the ability of precast concrete joints to develop CAA and catenary action. An experimental programme was conducted on beam-column sub-assemblages and frames under column removal scenarios. The middle beam-column joint and double-span beam over the removed column were extracted from a typical precast concrete structure and scaled down to one-half models. Two enlarged end column stubs were designed in the sub-assemblages, to which horizontal restraints were connected. In addition, engineered cementitious composites (ECC), with strain-hardening behaviour and superior strain capacity in tension, were utilised in the cast-in-situ structural topping and beam-column joint. In the precast frames, side columns were curtailed between the column inflection points below and above the bridging beam to represent realistic boundary conditions to the bridging beam.

In comparison with flexural resistance, development of CAA and catenary action substantially enhanced the progressive collapse resistance of beam-column sub-assemblages. Besides, the effects of reinforcement detailing in the joint, longitudinal reinforcement ratios in the beam, and horizontal interface preparation between the precast beam unit and cast-in-situ concrete topping, on the resistance and deformation capacity of sub-assemblages were studied experimentally under relatively rigid boundary condition. Furthermore, a comparison was made between sub-assemblages with conventional concrete and ECC to highlight the effect of ECC on structural behaviour of sub-assemblages under column removal scenarios. In the experimental tests on precast concrete frames, the influence of joint detailing and boundary conditions on progressive collapse resistance of the frames was investigated. Special attention was placed on the behaviour of side columns subjected to CAA and catenary action. Design recommendations were made in accordance with experimental results.

Based on previous studies, an analytical model was proposed to predict the CAA of beam-column sub-assemblages. The tensile strength of ECC and stress-strain model of concrete were considered in the model. A series of parametric studies was conducted through the analytical model to identify dominant parameters on the resistance of sub-assemblages subjected to CAA. In addition, the pseudo-static resistance was calculated through the energy balance method. In the catenary regime, the component-based joint was developed for precast concrete joints to provide an efficient and explicit representation of joint behaviour. Interactions between the structural members and beam-column joint were modelled by zero-length springs with specific constitutive relationships. The average bond stress for calculating the force-slip relationship of a spring was evaluated. Furthermore, a tension spring representing pull-out failure of embedded reinforcement was derived for precast concrete joints. Finally, the model was calibrated by experimental results of precast and reinforced concrete beam-column sub-assemblages.

LIST OF FIGURES

Fig. 2.1: Design approaches to resist progressive collapse (DOD 2013)	9
Fig. 2.2: Location restriction for internal and peripheral ties (DOD 2013).....	10
Fig. 2.3: Reinforcement details in beams (Orton et al. 2009).....	15
Fig. 2.4: Vertical and axial loads in beams (Orton 2007).....	16
Fig. 2.5: Failure modes of beams (Orton 2007).....	17
Fig. 2.6: Reinforcement detailing of beam-column sub-assemblages (Yu and Tan 2013b)	18
Fig. 2.7: Test setup for beam-column sub-assemblages (Yu and Tan 2013b).....	18
Fig. 2.8: Variations of vertical load and horizontal reaction force with middle joint displacement (Yu and Tan 2013b)	19
Fig. 2.9: Failure mode of sub-assemblage S1 (Yu and Tan 2013b).....	19
Fig. 2.10: Reinforcement details of reinforced concrete beam-column assemblies (Sadek et al. 2011)	21
Fig. 2.11: Test setup and instrumentation for beam-column assemblies (Sadek et al. 2011)	21
Fig. 2.12: Vertical load-middle joint displacement histories (Sadek et al. 2011).....	22
Fig. 2.13: Test setup and instrumentation for beam-column assemblies (Yi et al. 2008)	24
Fig. 2.14: Variation of load cell reaction force versus middle column displacement (Yi et al. 2008).....	24
Fig. 2.15: Effect of middle column displacement on horizontal displacement of columns at first floor level (Yi et al. 2008).....	25
Fig. 2.16: Beam-to-column connection details for SMF building (Main et al. 2014)....	26
Fig. 2.17: Vertical load versus vertical displacement of centre column (Main et al. 2014)	26
Fig. 2.18: Failure mode at connections to centre column (Main et al. 2014)	27

Fig. 2.19: Location of column removal (circled) (Sasani and Sagioglu 2010)	28
Fig. 2.20: Vertical displacements of second and seventh floor joints above removed column (Sasani and Sagioglu 2010)	28
Fig. 2.21: Axial compressive force in column C3 on different floors (Sasani and Sagioglu 2010).....	28
Fig. 2.22: Typical plan of the building and location of column removal (Sasani et al. 2007)	29
Fig. 2.23: Variations of axial forces in column B5 (Sasani et al. 2007)	29
Fig. 2.24: Bending moment diagram and deformed shape of axis 5 (Sasani et al. 2007)	30
Fig. 2.25: Uniaxial tensile stress-strain curves of ECC with 2% PVA fibres (Li 2003) 31	
Fig. 2.26: Load-deformation responses of columns subjected to reversed cyclic loading (Fischer and Li 2002a)	32
Fig. 2.27: Damage properties of beams (Fukuyama et al. 2000).....	32
Fig. 2.28: Load-deflection curves and failure modes of concrete/ECC composite beams (Yuan et al. 2013)	33
Fig. 2.29: Load-deformation responses of specimens in uniaxial tension (Fischer and Li 2002b).....	33
Fig. 2.30: Interface condition in reinforced concrete and ECC (Fischer and Li 2002b) 34	
Fig. 2.31: Total load in specimens versus average strain (Moreno et al. 2014)	35
Fig. 2.32: Cracks in specimens prior to fracture of reinforcement (Moreno et al. 2014)	35
Fig. 2.33: Bond stress-reinforcement slip response (Bandelt and Billington 2014)	36
Fig. 2.34: Reinforced concrete beam-column joint models under cyclic loads.....	38
Fig. 2.35: Bond and bar stress distribution along a reinforcing bar embedded under pull-out force (Lowes et al. 2004).....	39
Fig. 2.36: Beam-column joint models under column removal scenarios	40

Fig. 2.37: Bond and bar stress distribution along a reinforcing bar under axial tension (Yu and Tan 2010b)	41
Fig. 3.1: The prototype precast concrete structure	44
Fig. 3.2: Reinforcement detailing of precast concrete beam-column sub-assemblages ..	46
Fig. 3.3: Test setup for beam-column sub-assemblages	48
Fig. 3.4: Restraints on beam-column sub-assemblages	49
Fig. 3.5: Schematic of hinge rotation and beam deformation measurement	50
Fig. 3.6: Layout of strain gauges on longitudinal reinforcement	50
Fig. 3.7: Stress-strain curves of concrete and reinforcement	52
Fig. 3.8: Vertical load-middle joint displacement curves of beam-column sub-assemblages	52
Fig. 3.9: Horizontal reaction-middle joint displacement curves of beam-column sub-assemblages	53
Fig. 3.10: Free body diagram of the single-span beam	57
Fig. 3.11: Contributions of axial force and bending moments to vertical load of sub-assemblages	58
Fig. 3.12: Deformed profiles of beam-column sub-assemblages	60
Fig. 3.13: Partial hinge at the curtailment point of top bars	62
Fig. 3.14: Rotations of partial hinges at the curtailment point	62
Fig. 3.15: Rotations of plastic hinges at the end column stub of sub-assemblages	63
Fig. 3.16: Crack patterns of beam-column sub-assemblages	63
Fig. 3.17: Failure modes of sub-assemblages at the middle joint	65
Fig. 3.18: Failure modes of beam-column sub-assemblages at the end column stub	66
Fig. 3.19: Horizontal cracking across the concrete interface	66
Fig. 3.20: Strains of beam longitudinal reinforcement in MJ-B-0.88/0.59R	68
Fig. 3.21: Strains of beam longitudinal reinforcement in the beam of sub-assemblage MJ-B-0.88/0.59R	69

Fig. 3.22: Strains of beam longitudinal reinforcement at the middle joint.....	70
Fig. 4.1: Geometric properties of precast beam-column sub-assemblages	74
Fig. 4.2: Load-deflection curve of ECC plates under four-point bending.....	78
Fig. 4.3: Stress-strain relationships of concrete and steel bar.....	79
Fig. 4.4: Variations of vertical loads and horizontal reaction forces of sub-assemblages of bottom reinforcement with 90° bend.....	81
Fig. 4.5: Variations of vertical loads and horizontal reaction forces of sub-assemblages with lap-spliced bottom reinforcement	81
Fig. 4.6: Crack patterns and failure modes of CMJ-B-1.19/0.59.....	86
Fig. 4.7: Development of multi-cracking in the structural topping of EMJ-B-1.19/0.59	86
Fig. 4.8: Failure modes of sub-assemblage EMJ-B-1.19/0.59	87
Fig. 4.9: Failure modes at the end column stub of sub-assemblages	87
Fig. 4.10: Failure modes of EMJ-L-0.88/0.88	88
Fig. 4.11: Force equilibrium of deformed sub-assemblage	88
Fig. 4.12: Variations of horizontal reaction forces in sub-assemblages.....	89
Fig. 4.13: Variations of bending moments in EMJ-B-1.19/0.59	90
Fig. 4.14: Interaction of bending moment and beam axial force	90
Fig. 4.15: Rotations in plastic hinge regions of sub-assemblages	94
Fig. 4.16: Layout of strain gauges on beam longitudinal reinforcement	96
Fig. 4.17: Variations of steel strains in EMJ-L-1.19/0.59.....	97
Fig. 4.18: Strains of reinforcement H16 at the end column stub of EMJ-L-1.19/0.59 ..	98
Fig. 4.19: Curvatures of steel bars along the embedment length.....	99
Fig. 5.1: Geometry and reinforcement detailing of precast concrete frames.....	107
Fig. 5.2: Test setup for precast concrete frames	109
Fig. 5.3: Restraints on precast concrete frames	110
Fig. 5.4: Layout of LVDTs on precast concrete frames	111

Fig. 5.5: Material stress-strain curves of reinforcement and concrete	112
Fig. 5.6: Vertical load-middle joint displacement curves of precast frames	113
Fig. 5.7: Horizontal reaction force-middle joint displacement curves of precast frames	113
Fig. 5.8: Neutral axis depth of beam sections at the face of the side column.....	115
Fig. 5.9: Pseudo-static load-middle joint displacement curves of precast concrete frames	117
Fig. 5.10: Load paths of horizontal reaction forces to the support.....	119
Fig. 5.11: Crack patterns and failure modes of IF-B-0.88-0.59.....	120
Fig. 5.12: Crack patterns and failure modes of EF-B-0.88-0.59.....	121
Fig. 5.13: Crack patterns and failure modes of IF-L-0.88-0.59	121
Fig. 5.14: Crack patterns and failure modes of EF-L-0.88-0.59.....	122
Fig. 5.15: Crack patterns and failure modes of side columns	124
Fig. 5.16: Lateral deflections of side columns	125
Fig. 5.17: Column deflection-middle joint displacement curves of exterior frames....	126
Fig. 5.18: Shear distortion of side beam-column joints.....	127
Fig. 5.19: Strain gauge layout along the bottom bars of precast beams	128
Fig. 5.20: Variations of steel strains in the middle beam-column joint of IF-L-0.88-0.59	129
Fig. 5.21: Strains of beam bottom reinforcement embedded in the right column.....	130
Fig. 5.22: layout of strain gauges in side beam-column joint.....	131
Fig. 5.23: Variations of reinforcement strains in side columns	132
Fig. 5.24: Strains of horizontal hoops in side joint zone	133
Fig. 6.1: Geometry and reinforcement detailing of precast concrete frames.....	138
Fig. 6.2: Vertical loads and horizontal reaction forces of exterior frames EF-B-1.19/0.59 and EF-L-1.19/0.59	139

Fig. 6.3: Vertical loads and horizontal reaction forces of exterior frames EF-B-1.19/0.59S and EF-L-1.19/0.59S	140
Fig. 6.4: Failure modes of middle beam-column joints	143
Fig. 6.5: Crack patterns of bridging beams	144
Fig. 6.6: Propagation of shear cracks in the side joint of EF-B-1.19/0.59	146
Fig. 6.7: Crack patterns and failure modes of side beam-column joints	148
Fig. 6.8: Lateral deflections of side columns.....	149
Fig. 6.9: Shear forces and bending moments on side column	151
Fig. 6.10: Shear forces in side beam-column joints	151
Fig. 6.11: Variations of bending moments at column sections.....	154
Fig. 6.12: Layout of strain gauges in the side joint	155
Fig. 6.13: Variations of reinforcement strains in side columns	156
Fig. 6.14: Strains of horizontal hoops in side beam-column joints.....	157
Fig. 6.15: Actions in side beam-column joint.....	158
Fig. 7.1: Geometric configuration of horizontally restrained beams	162
Fig. 7.2: Plastic hinge mechanism of beam-column sub-assemblages	163
Fig. 7.3: Stress-strain relationship of steel bars	163
Fig. 7.4: Constitutive model for concrete in compression.....	165
Fig. 7.5: Stress-strain relationship of ECC	167
Fig. 7.6: Free-body diagram of the single-span beam.....	167
Fig. 7.7: Force equilibrium of beam sections	168
Fig. 7.8: Configuration of beam at small deformation stage	171
Fig. 7.9: Compatibility condition of beam at large deformation stage	172
Fig. 7.10: Solution procedure for the analytical model.....	177
Fig. 7.11: Comparisons of analytical and experimental vertical load-middle joint displacement curves of reinforced concrete sub-assemblages.....	180

Fig. 7.12: Comparisons of analytical and experimental vertical load-middle joint displacement curves of precast beam-column sub-assemblages.....	181
Fig. 7.13: Comparisons of analytical and experimental horizontal reaction force-middle joint displacement curves of precast beam-column sub-assemblages.....	182
Fig. 7.14: Variations of bending moments at the middle joint and end support.....	183
Fig. 7.15: Variations of numerical strains of steel reinforcement and concrete with middle joint displacement	185
Fig. 7.16: Variations of neutral axis depths with middle joint displacement	185
Fig. 7.17: Comparison of stress-strain models for concrete	187
Fig. 7.18: Comparisons of load-displacement curves with different concrete models	187
Fig. 7.19: Comparisons of bending moments at the middle joint and end support.....	188
Fig. 7.20: Comparisons of neutral axis depths with different concrete models	189
Fig. 7.21: Comparisons of load-displacement curves with different tensile strengths of ECC	190
Fig. 7.22: Comparisons of neutral axis depths with different tensile strengths of ECC	191
Fig. 7.23: Comparisons of load-displacement curves with different tensile strain capacities of ECC	192
Fig. 7.24: Comparisons of load-displacement curves with different horizontal restraints	193
Fig. 7.25: Pseudo-static resistances of sub-assemblages with different horizontal restraints.....	194
Fig. 7.26: Comparisons of neutral axis depths with different horizontal restraints	194
Fig. 7.27: Comparisons of load-displacement curves with different reinforcement ratios	195
Fig. 7.28: Comparisons of neutral axis depths with different reinforcement ratios	196
Fig. 7.29: Pseudo-static resistances of sub-assemblages with different reinforcement ratios	197

Fig. 8.1: Component-based joint model for beam-column sub-assemblages	203
Fig. 8.2: Variation of bond stresses along embedment length of a reinforcing bar	207
Fig. 8.3: Relationship of average bond stress with force and stress at the loaded end of reinforcing bars	208
Fig. 8.4: Relationships of applied force and loaded end slip for steel bars	209
Fig. 8.5: Comparisons of experimental and analytical force-slip relationships.....	212
Fig. 8.6: Bond-slip model for embedded reinforcing bars at elastic stage	214
Fig. 8.7: Bond stress distribution along a reinforcing bar at the peak pull-out force....	215
Fig. 8.8: Comparisons between experimental and analytical results under pull-out loads	217
Fig. 8.9: Variations of bond stress along embedment length at different loading stages	218
Fig. 8.10: Bond stresses and slips of an embedded reinforcing bar at elastic ascending stage	218
Fig. 8.11: Bond stresses and slips of an embedded reinforcing bar at plastic ascending stage	219
Fig. 8.12: Bond stresses and slips of an embedded reinforcing bar at descending stage	221
Fig. 8.13: Bond stress distribution for an elastic steel bar under axial tension	223
Fig. 8.14: Bond stress distribution for a yielded steel bar at loaded end.....	224
Fig. 8.15: Bond stress distribution for a yielded steel bar at mid-point of embedment length.....	225
Fig. 8.16: Relationship of applied force and loaded end slip for reinforcing bar T13 under axial tension.....	226
Fig. 8.17: Neutral axis depth at beam end	228
Fig. 8.18: Enhancement factors and compression forces at middle joint and end column stub.....	229
Fig. 8.19: Force-slip relationships of compressive springs	231

Fig. 8.20: Comparisons of experimental and numerical results of precast concrete beam-column sub-assemblages	234
Fig. 8.21: Comparisons of experimental and numerical results of reinforced concrete sub-assemblages	235
Fig. A.1 Horizontal reaction forces of precast concrete beam-column sub-assemblages	262
Fig. A.2 Horizontal force-displacement relationships of MJ-B-0.52/0.35S	263
Fig. A.3 Horizontal force-displacement relationships of MJ-L-0.52/0.35S	263
Fig. A.4 Horizontal force-displacement relationships of MJ-B-0.88/0.59R.....	264
Fig. A.5 Horizontal force-displacement relationships of MJ-L-0.88/0.59R.....	264
Fig. A.6 Horizontal force-displacement relationships of MJ-B-1.19/0.59R.....	264
Fig. A.7 Horizontal force-displacement relationships of MJ-L-1.19/0.59R.....	265
Fig. A.8 Horizontal reaction forces of precast beam-column sub-assemblages with ECC	267
Fig. A.9 Horizontal force-displacement relationships of CMJ-B-1.19/0.59.....	268
Fig. A.10 Horizontal force-displacement relationships of EMJ-B-1.19/0.59.....	268
Fig. A.11 Horizontal force-displacement relationships of EMJ-B-0.88/0.59.....	269
Fig. A.12 Horizontal force-displacement relationships of EMJ-L-1.19/0.59	269
Fig. A.13 Horizontal force-displacement relationships of EMJ-L-0.88/0.59	269
Fig. A.14 Horizontal force-displacement relationships of EMJ-L-0.88/0.88	270
Fig. A.15 Equivalent connection gap at the beam centroid	271
Fig. A.16 Bending moment-rotation relationships of end column stubs	272
Fig. A.17 Horizontal reaction forces of precast concrete frames.....	274
Fig. A.18 Horizontal force-displacement relationships of IF-B-0.88-0.59	275
Fig. A.19 Horizontal force-displacement relationships of IF-L-0.88-0.59.....	275
Fig. A.20 Horizontal force-displacement relationships of exterior frames	276

LIST OF TABLES

Table 2.1: Difference between earthquake and progressive collapse (DOD 2013)	11
Table 2.2: Occupancy categories (DOD 2013)	13
Table 2.3: Occupancy category and design requirements (DOD 2013)	13
Table 3.1: Geometric property of beam-column sub-assemblages	47
Table 3.2: Material properties of reinforcing bars	51
Table 3.3: Compressive strength of concrete	51
Table 3.4: Test results of beam-column sub-assemblages	54
Table 3.5: Components of vertical load sustained by sub-assemblages	59
Table 3.6: Rotations of plastic hinges and beam-column sub-assemblages	60
Table 3.7: Failure modes of beam-column sub-assemblages	64
Table 4.1: Reinforcement details of precast beam-column sub-assemblages	75
Table 4.2 Mixture proportions of ECC	77
Table 4.3 Strength of ECC in tension and compression	78
Table 4.4: Material properties of reinforcing and concrete	79
Table 4.5: Resistances of beam-column sub-assemblages	80
Table 4.6: Rotation angles of beam-column sub-assemblages	92
Table 5.1: Details of precast concrete frames	105
Table 5.2: Material properties of concrete and reinforcement	112
Table 5.3: Resistances and deformations of precast concrete frames	114
Table 5.4: Pseudo-static resistances of precast concrete frames	118
Table 5.5: Failure modes of precast concrete frames	120
Table 6.1: Geometry and reinforcement details of precast concrete frames	136
Table 6.2: Material properties of steel reinforcement	139
Table 6.3: Compressive and splitting tensile strengths of concrete	139

Table 6.4: Experimental results of precast concrete frames at CAA stage	141
Table 6.5: Resistances and deformations of precast concrete frames at catenary action stage	142
Table 6.6: Maximum shear forces in side beam-column joints	152
Table 6.7: Maximum bending moments at column sections	154
Table 7.1: Boundary conditions of beam-column sub-assemblages	178
Table 7.2: Comparisons of experimental and analytical results	179
Table 7.3: Reinforcement ratios in beam-column sub-assemblages	195
Table 8.1: Failure modes of embedded bars subjected to pull-out force	204
Table 8.2: Average bond stress predicted by Shima's model.....	206
Table 8.3: Material properties of embedded reinforcement (Bigaj 1995)	211
Table 8.4: Bond stress of embedded steel bars under pull loading condition	215
Table 8.5: Material properties of embedded bars	216
Table 8.6: Material properties and embedment length for T13 rebar (Yu 2012)	226
Table 8.7: Material and geometric properties of beam sections	229
Table 8.8: Parameters of springs in joint model.....	233
Table A.1 Horizontal stiffness of precast concrete beam-column sub-assemblages	265
Table A.2 Horizontal stiffness of beam-column sub-assemblages with ECC	270
Table A.3 Rotational stiffness of beam-column sub-assemblages with ECC	273
Table A.4 Horizontal stiffness of precast concrete frames.....	277

LIST OF SYMBOLS

a_{s1}, a_{s2}	Distances from the centroid of tension reinforcement to the extreme tension fibre at the faces of end the column stub and the middle joint, respectively
a'_{s1}, a'_{s2}	Distances from the centroid of compression reinforcement to the extreme compression fibre at the faces of the end column stub and the middle joint, respectively
b	Width of beam section
c	Neutral axis depth at the beam end
c_1, c_2	Neutral axis depths at the faces of the end column stub and the middle joint, respectively
d	Diameter of steel reinforcement
f'_c	Compressive strength of concrete
f_s	Stress of steel reinforcement at the loaded end
f_{sc}	Tensile stress of steel reinforcement at the centre of its embedment length when subjected to axial tension
f'_{scr}	Compressive stress of reinforcement at the critical state when the tensile reinforcement attains its yield strain and concrete reaches its ultimate compressive strain simultaneously
f_y, f'_y	Yield strengths of reinforcement in tension and compression, respectively
h	Depth of beam section
h_t	Thickness of ECC topping
k_{bt}, k_{bb}	Properties of top and bottom springs at the joint interface
l	Clear span of beam
l_1, l_2	Distances from the inflection point of the beam to the left and right ends, respectively
l_{1p}, l_{2p}	Horizontal distances between the linear variable differential transducers in the plastic hinge region near the end column stub

l_b, l_t	Lengths of the column segments below and above the side joint
l_d	Length of debonded region
l_e	Length of elastic steel reinforcement
l_j	Diagonal length of the joint panel
l_s	Length of the straight portion of embedded reinforcement in front of the hook
l_y	Length of yielded steel segment
l_{AB}	Length of steel segment AB under axial tension
l_{CD}, l_{CG}, l_{EF}	Length of elastic steel segments CD, CG and EF under axial tension
l_{GD}	Length of inelastic steel segment GD under axial tension
q	Self-weight of beam
s	Slip of reinforcement relative to concrete
s_1	Slip of reinforcement when the maximum bond stress is attained
s_2	Slip of reinforcement when the local bond stress starts to decrease
s_3	Slip of reinforcement at the onset of frictional bond stress
s_d	Slip of reinforcement at the section where debonding occurs
s_f, s_l	Slip of reinforcement at the free and loaded ends, respectively
s_y	Slip at the section where the steel bar attains its yield strength
s_D	Slip at the end of steel segment CD for axial tension
s_F	Slip at the end of steel segment EF for axial tension
t	Horizontal movement of end support

t_0	Connection gap
u_d	Vertical displacement of middle joint
A_s, A'_s	Areas of reinforcement in the tension and compression zones
C_c, C_s	Compression forces sustained by concrete and steel reinforcement at the beam end, respectively
C_{c1}, C_{c2}	Compressive forces in concrete at the faces of the end column stub and the middle joint, respectively
C_{s1}, C_{s2}	Forces in the compressive reinforcement at the faces of the end column stub and the middle joint, respectively
E_c	Tangent modulus of elasticity of concrete
E_s	Modulus of elasticity of steel bars
E_h	Hardening modulus of steel bars
E_{sec}	Secant modulus of elasticity of concrete
F_d	Force at the section where debonding occurs
F_y	Yield force of steel bars
K_a, K_r	Stiffness of horizontal and rotational restraints, respectively
M_1, M_2	Bending moments at the faces of the end column stub and the middle joint, respectively
M_{bb}	Moment resistance of beam end section
M_c	Moment capacity of side column under combined axial compression force and bending moment
M_{cd}, M_{ce}	Bending moments at column sections D-D and E-E corresponding to the top and bottom faces of beam
N	Axial force in the beam
N_c, N_t	Maximum horizontal compression force at the compressive arch action stage and tension force at the catenary action stage

N_{cr}	Axial compression force in the beam at a critical state when the tensile reinforcement attains its yield strain and extreme compression fibre reaches its crushing strain simultaneously
P	Vertical load on the middle joint
P_c, P_f, P_t	Capacities of flexural action, compressive arch action and catenary action, respectively
P_d	Pseudo-static resistance of precast concrete frames
P_{dc}, P_{dt}	Pseudo-static resistances of precast concrete frames at the compressive arch action and the catenary action stages
R_b, R_t	Horizontal reaction forces in the bottom pin support and top horizontal restraint
T	Tension force sustained by the top longitudinal reinforcement in the beam
T_{s1}, T_{s2}	Tension forces sustained by reinforcement at the faces of the end column stub and the middle joint, respectively
T_{t1}	Tension force sustained by engineered cementitious composites in the structural topping
T_y	Yield force of top reinforcement in the beam
V_{jc}	Horizontal shear force in the side beam-column joint under compressive arch action
V_{jf}	Shear force in the side joint under flexural action
δ	Vertical displacement of middle joint
δ_1, δ_2	Deformations of the joint panel in the diagonal directions
$\delta_{LE-1}, \delta_{LE-2}, \delta_{LE-3}, \delta_{LE-4}$	Readings of linear variable differential transducers in the plastic hinge region near the end column stub
$\delta_{SD-3}, \delta_{SD-4}$	Measurements of linear variable differential transducers SD-3 and SD-4 corresponding to the top and bottom faces of the beam
ε	Strain of steel segment
ε_b	Axial compressive strain of beam

ε_c	Strain corresponding to the compressive strength of concrete
ε_{c0}	Compressive strain of engineered cementitious composites corresponding to the compressive strength
$\varepsilon_{c1}, \varepsilon_{c2}$	Strain of extreme compression fibres at the end column stub and the middle joint
ε_{cu}	Ultimate compressive strain of engineered cementitious composites
ε_d	Strain of reinforcement at the section where debonding occurs
ε_l	Steel strain at the loaded end of reinforcement at post-yield stage
ε_m'	Maximum compressive strain that reinforcement has attained
$\varepsilon_s, \varepsilon_s'$	Tensile and compressive strains of steel reinforcement, respectively
$\varepsilon_{s1}, \varepsilon_{s1}'$	Tensile strain of top reinforcement and compressive strain of bottom reinforcement at the end column stub, respectively
$\varepsilon_{s2}, \varepsilon_{s2}'$	Tensile strain of bottom reinforcement and compressive strain of top reinforcement at the middle joint
$\varepsilon_{tc}, \varepsilon_{tu}$	First cracking and ultimate tensile strains of engineered cementitious composites, respectively
$\varepsilon_y, \varepsilon_y'$	Yield strains of steel reinforcement in tension and compression, respectively
$\varepsilon_{ET-1}, \varepsilon_{ET-2}$	Readings of strain gauges ET-1 and ET-2 at the face of the end column stub
φ	Rotation angle of beam
γ	Shear distortion of side beam-column joint
γ_a	Normalised stiffness of horizontal restraint
γ_c	Ratio of the total compression force in the compression zone to the force sustained by the compressive reinforcement
γ_θ	Ratio of rotations in the plastic hinge region

$\kappa_{RC-1/2}$	Curvature of beam top reinforcement at the face of the end column stub
σ_{c0}	Compressive strength of engineered cementitious composites
σ_{c1}, σ_{c2}	Compressive stresses of concrete at the faces of the end column stub and the middle joint, respectively
σ_{cu}, σ_{cu}	Ultimate compressive strength of engineered cementitious composites
σ_s, σ'_s	Tensile and compressive stresses of steel reinforcement, respectively
σ_{t1}	Tensile stress of ECC topping
σ_{tc}, σ_{tu}	First cracking and ultimate tensile strengths of engineered cementitious composites, respectively
θ_1, θ_2	Rotations measured in the plastic hinge region near the end column stub
θ_c, θ_p	Chord rotation of the bridging beam and plastic hinge rotation at the end column stub
θ_r	Rigid-body rotation of side joint
τ_1	Maximum bond stress
τ_2	Frictional bond stress
τ_d	Bond stress at the section where debonding occurs
τ_f, τ_l	Bond stress at the free and loaded ends of steel bars at the elastic stage
τ_y	Post-yield bond stress of steel reinforcement
τ_{ye}	Bond stress at the section where the steel bar attains its yield strength
$\tau_{CD}, \tau_{CG}, \tau_{EF}$	Bond stress along elastic steel segments CD, CG and EF under axial tension
Θ	Rotation of end support due to insufficient stiffness
Θ_0	Free rotation angle of end support due to connection gap

CHAPTER 1 INTRODUCTION

1.1 Research Background

Progressive collapse is defined by ASCE 7-05 (ASCE 2006) as “the spread of local damage from an initiating event, from element to element resulting, eventually, in the collapse of an entire structure or a disproportionately large part of it; also known as disproportionate collapse”. The partial collapse of Ronan Point Apartment initiated the research interest of engineering communities to seek design methods to mitigate progressive collapse. Following the aftermath of the partial collapse of Ronan Point Apartment, provisions for preventing disproportionate collapse were formulated in the U.K. and formed the basis of subsequent research (Izzudin et al. 2008). In recent years, the disastrous collapse of the Alfred P. Murrah Federal Building in Oklahoma City and the World Trade Centre in New York refocused the intellectual debates on clarifying the load redistribution mechanisms of building structures when subjected to local failure. Thereafter, design guidelines were released by U.S. government agencies, such as Department of Defence (DOD 2013) and General Service Administration (GSA 2003), to prevent progressive collapse of various types of building structures.

With respect to progressive collapse design, two categories of approaches, namely, indirect and direct design, have been proposed by Ellingwood and Leyendecker (1978) and incorporated in the design guides (DOD 2013; GSA 2003). In the indirect method, progressive collapse resistance of structures is implicitly addressed through the provisions for minimum level of strength, continuity and ductility in the form of tie force requirements (DOD 2013). Regarding the direct approach, alternate path method and enhanced local resistance are used to give explicit considerations of maintaining the overall structural robustness when accidental loading condition occurs (NIST 2007). Indeed, both tie forces and alternate path method seek to limit the extent of damage through mobilisation of alternate load paths in bridging members, such as the beam and the floor or roof system, under large deformation conditions. Tie forces are basically contributed by catenary or tensile membrane action in the floor or roof system, possibly accompanied by catenary action in internal

beams (Stevens et al. 2011; Stevens et al. 2009), while alternate path method is primarily implemented through the formation of compressive arch action (CAA) and catenary action in bridging beams.

Experimental tests on beam-column sub-assemblages under middle column removal scenarios indicate that the bridging beam is able to develop significant CAA to resist progressive collapse (Lew et al. 2011; Su et al. 2009; Yu and Tan 2013b). In comparison with catenary action, development of CAA requires relatively small vertical displacement (less than one beam depth) (Gurley 2008), which makes it more attractive to structural engineers. However, in order to mobilise effective CAA in the beam to mitigate progressive collapse, adequate horizontal restraints have to be provided for the beam (Yu and Tan 2013a), in particular, at the structure perimeter.

1.2 Alternate Load Paths

Following the removal of a supporting column, CAA and catenary action develop sequentially in the bridging beam over the “damaged” column, if adjacent structural members provide adequate horizontal restraints for the beam. CAA features axial compression force in the beam at relatively small vertical displacement and it substantially contributes to the flexural resistance of the beam (Park and Gamble 2000; Yu and Tan 2013a). At large deformation stage, tensile strength of the beam is mobilised to sustain vertical load; catenary action kicks in as the last line of defence to mitigate disproportionate propagation of the initial damage. Development of catenary action in a damaged structure requires certain degree of structural integrity after undergoing considerable vertical deformations (Khandelwal and El-Tawil 2007). It is likely to cause premature failure of beam-column joints due to limited rotation capacity under column removal scenarios (Stevens et al. 2011). Thus, beam-column joints, in particular, in precast concrete structures, have to satisfy the rotation requirements in order to develop catenary action.

Greater reinforcement ratios in the beam and special reinforcement detailing in the beam-column joint have shown to increase the rotation capacity of reinforced concrete structures under progressive collapse scenarios (Yu and Tan 2013c; Yu and Tan 2014). Besides, ductile concrete material, such as engineered cementitious

composites (ECC), is expected to enhance the ductility and robustness due to its strain-hardening behaviour, ultra-high strain capacity and damage tolerance in tension (Li 2003). Furthermore, compatible deformations between steel reinforcement and ECC reduce the required embedment length of steel bars (Fischer and Li 2002b). Its potential application to column removal scenarios has to be investigated in terms of its enhancement to progressive collapse resistance and rotation capacity of the beam-column joint.

Similar to reinforced concrete one-way slabs (Park and Gamble 2000), development of CAA and catenary action in the bridging beam is sensitive to horizontal restraints under column removal scenarios. The progressive collapse resistance of beam-column sub-assemblages is substantially increased if nearly rigid horizontal restraints are applied (Su et al. 2009; Yu and Tan 2013c). By reducing the stiffness of horizontal restraints, lower CAA and catenary action can be expected. Thus, flexible boundary conditions of the bridging beam have to be considered in precast concrete structures, in which realistic adjacent columns to the local damage are designed instead of enlarged column stubs.

At the structural level, development of CAA and catenary action in the beam imposes additional horizontal compression and tension forces on adjacent columns. In case of interior column removal scenario, horizontal forces in both the CAA and catenary action stages can be equilibrated by the surrounding floor system as a rigid diaphragm. However, at the perimeter of the structure, shear failure of the beam-column joints (Choi and Kim 2011) and flexural failure of the columns (Lew et al. 2011) may occur as a result of additional horizontal forces. Thus, in the flexural and shear design of adjacent columns, a certain level of horizontal force has to be considered to prevent potential failure under column removal scenarios. In the UFC 4-023-03 (DOD 2013), it is suggested that lateral stability and second-order effects be implicitly taken into account in the provisions of lateral loading in load case combinations, which originate from the seismic design code (ASCE 2007). Nonetheless, the magnitude of the horizontal forces depends on lateral stiffness of the columns. By enlarging the cross sections of columns, horizontal forces acting on the columns can be increased considerably. Therefore, more practical and explicit considerations have to be taken against potential failure of the columns.

In addition to experimental tests, the component-based joint model can also be used to simulate the behaviour of precast concrete structures subject to column removal scenarios. In the model, reinforced concrete beams and columns are simplified as fibre elements, and beam-column joints are represented by a panel, in which shear distortion is considered (Lowes et al. 2004; Mitra and Lowes 2007). The interactions between structural members and joints are modelled by zero-length inelastic springs. The constitutive relationships of springs are defined as a function of material and geometric properties. The method offers a direct approach to representing the complicated mechanisms in the beam-column joints and to predicting the joint response accurately (Lowes and Altoontash 2003). In precast concrete structures, the main challenge lies in the modelling of pull-out failure of embedded steel reinforcement in the joint due to inadequate anchorage length.

Thus far, limited attention is paid onto the behaviour of precast concrete structures under column removal scenarios. As an assembly of precast concrete units, precast concrete structure requires integral and robust beam-column joints when subjected to progressive collapse. Welded joints are vulnerable to column loss due to the reduced ductility in heat-affected zones (Main et al. 2014). Cast-in-situ concrete joints, which connect precast concrete beam and column units through special reinforcement detailing, exhibit equivalent behaviour to monolithic reinforced concrete joints under cyclic loading conditions (CAE 1999; FIB 2003). However, further experimental and analytical investigations are needed on the behaviour of these types of joints under column removal scenarios.

1.3 Objectives and Scope of Research

Under column removal scenarios, the ability of precast concrete structures to develop effective alternate load paths to mitigate progressive collapse remains questionable due to a lack of experimental and analytical results. Therefore, an experimental programme is proposed in the current study and four series of experimental tests are conducted under quasi-static loading condition. The primary objectives of the research are as follows:

(1) To investigate the resistance and deformation capacity of precast concrete beam-column sub-assemblages under quasi-static loading. In the experimental programme, enlarged column stubs are designed for the bridging beam over the removed column, and relatively rigid horizontal restraints are applied to the column stubs. The effect of joint detailing and reinforcement ratios on structural behaviour of sub-assemblages is studied. Horizontal shear transfer between the precast beam units and cast-in-situ concrete topping is examined through different treatment of concrete interface. For this series of tests, horizontal restraints to the column stubs are assumed to be fully effective.

(2) To study the behaviour of beam-column sub-assemblages with ECC in the structural topping and beam-column joint in place of conventional concrete. The enhancement of ECC to the resistance of sub-assemblages under progressive collapse scenarios is quantified through this comparison study. Furthermore, interactions between ECC and steel reinforcement are qualitatively analysed.

(3) To explore the effect of boundary conditions and reinforcement detailing on the development of CAA and catenary action and to gain a deeper insight into the behaviour of side columns under column removal scenarios. Experimental tests are conducted on precast concrete frames with realistic side columns under quasi-static loading condition. Special attention is placed on the flexural and shear resistance of side columns when subjected to horizontal compression and tension forces from the bridging beam.

(4) To develop an analytical model to predict the CAA of beam-column sub-assemblages with ECC structural topping. Different from conventional concrete, ECC exhibits strain-hardening behaviour and superior strain capacity in tension. Thus, its tensile strength has to be considered in the analytical model. The stress-strain model of concrete is used in place of the equivalent rectangular concrete stress block to consider crushing of concrete at CAA stage. The model is calibrated by experimental results at the fibre, member cross section and structural levels.

(5) To develop a component-based model for precast concrete beam-column joints. Force transfer between structural members and beam-column joints is simplified as a series of nonlinear springs at the joint interface. Properties of the springs are derived

based on the bond-slip model of embedded reinforcement. A method to determine the force-slip relationship of steel reinforcement with insufficient embedment length is proposed such that pull-out failure of reinforcement can be incorporated into the component model.

This study provides necessary information on the deformation capacity of precast concrete beam-column joints under column removal scenarios to examine the failure criteria in UFC 4-023-03 and to assess robustness of buildings arising from this research. Observations from experimental tests reveal the major characteristics of precast concrete joints in resisting progressive collapse, such as pull-out failure of embedded reinforcement, horizontal interfacial cracking between precast units and cast-in-situ concrete topping. Furthermore, an attempt is made to investigate the flexural and shear failures of connecting columns subject to CAA and catenary action through equilibrium of lateral forces acting on the columns. Analytical and component-based joint models are proposed to help engineers to evaluate the CAA and catenary action capacities. In the component model, fracture and pull-out failure of steel reinforcement embedded in the joint play a major role and new nonlinear springs are developed for fracture and pull-out failure. The work will facilitate progressive collapse analysis of precast concrete structures by future researchers and structural engineers.

However, current experimental and analytical studies only focus on the quasi-static behaviour of two-dimensional precast concrete structures under column removal scenarios. Potential enhancement of precast concrete planks or cast-in-situ concrete slabs to the collapse resistance of structures is not considered. The out-of-plane deflection and torsion of the bridging beam in the realistic structures is prevented in the experimental tests through lateral restraints.

1.4 Layout of the Thesis

The thesis is divided into nine chapters. The content of the following chapters are briefly described as follows:

Chapter Two presents an overview of design approaches that have been incorporated in the design codes and guidelines to mitigate progressive collapse. Experimental

tests on reinforced concrete buildings and numerical models for beam-column joints are also reviewed under column removal scenarios. Moreover, material properties and structural applications of ECC are introduced.

In Chapter Three, experimental tests on precast concrete beam-column sub-assemblages are described in detail. The resistance and deformation capacity of sub-assemblages are provided. The effect of reinforcement detailing in the joint and reinforcement ratios on the CAA and catenary action is discussed. Conclusions and design recommendations are drawn based on the experimental results of beam-column sub-assemblages.

Chapter Four introduces the experimental programme on precast beam-column sub-assemblages with cast-in-situ ECC in the structural topping and beam-column joint. Behaviour of ECC sub-assemblages subjected to CAA and catenary action is presented. A comparison is made between conventional concrete and ECC sub-assemblages in terms of the resistance, crack pattern and failure mode. Preliminary conclusions on the enhancement of ECC to the resistance of sub-assemblages are obtained.

In Chapter Five, the quasi-static resistance of interior and exterior precast concrete frames is addressed. The influence of joint detailing and boundary conditions on the behaviour of frames is quantified. Besides, the pseudo-static resistance is calculated based on the energy balance method. Chapter Six focuses on the robustness of exterior precast concrete frames subjected to middle column removal scenarios. Attempt is made to evaluate the flexural and shear resistance of side columns when CAA and subsequent catenary action are mobilised in the bridging beam.

In Chapter Seven, an analytical model is proposed to predict the CAA of beam-column sub-assemblages under column removal scenarios. In the model, the tensile strength of ECC in tension and stress-strain model of concrete in compression are taken into account. The model is calibrated against experimental results of reinforced concrete and ECC sub-assemblages. Furthermore, a series of parametric studies is conducted to identify dominant factors on CAA of sub-assemblages.

Chapter Eight addresses the component-based joint model for precast concrete beam-column joints subject to column removal scenarios. In the model, boundary

conditions of embedded reinforcement are classified according to the anchorage length and loading conditions, and average bond stresses at the elastic and post-yield stages are evaluated. A new model is proposed to estimate the force-slip relationship of reinforcement with inadequate embedment length. The model can predict the vertical load-middle joint displacement curves and horizontal reaction force-middle joint displacement curves with reasonably good accuracy. In Chapter Nine, conclusions of current research are drawn and future work is listed.

CHAPTER 2 LITERATURE REVIEW

2.1 Overview

This chapter presents an overview of previous studies related to progressive collapse. First, design approaches included in the design codes and guidelines against progressive collapse are reviewed. Thereafter, experimental tests on reinforced and precast concrete structures are discussed to shed light on the load transfer mechanisms in the bridging beam over the “damaged” column. Finally, the component-based joint model is introduced under cyclic loadings and column removal scenarios.

2.2 Design Approaches against Progressive Collapse

The partial collapse of Ronan Point Apartment in 1968 triggered off intensive research works on robustness of precast concrete beam-column joints to resist progressive collapse and eventually formed the basis of design approaches against progressive collapse proposed by Ellingwood and Leyendecker (1978). Generally, indirect and direct design approaches are classified, as shown in **Fig. 2.1**. These approaches have been widely accepted in the design codes (ACI 2005; BSI 2004) and guidelines (DOD 2013; GSA 2003) following the tragic collapse of the Alfred P. Murrah Federal Building and the World Trade Centre twin towers. In the design of building structures against progressive collapse, tie forces, alternate path method and enhanced local resistance method can be selected in accordance with occupancy category.

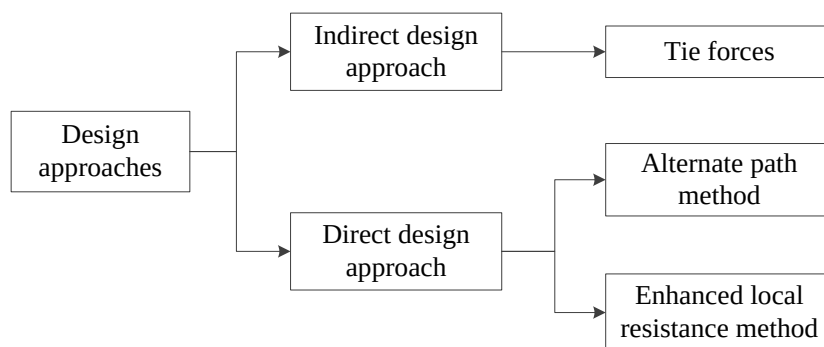


Fig. 2.1: Design approaches to resist progressive collapse (DOD 2013)

2.2.1 Indirect design approach

Indirect design approach is defined as a threat-independent approach to evaluate the ability of a structure to redistribute vertical loads through the specification of minimum levels of strength, continuity, and ductility (Stevens et al. 2011). In the UFC 4-023-03 (DOD 2013) and BS EN 1991-1-7:2006 (BSI 2006), tie forces, which mechanically tie a structure together, are expressed in quantitative terms to enhance structural robustness and redundancy. Three horizontal ties, namely, transverse, longitudinal and peripheral, should be provided in the floor or roof system so as to transfer the vertical loads through catenary or tensile membrane action to undamaged horizontal members (Stevens et al. 2009). Vertical ties are required in the columns and structural walls. The main concern about the aforementioned tie forces lies in the potential inability of structural members and joints to provide horizontal tie forces after experiencing significant rotation (Stevens et al. 2009). Therefore, UFC 4-023-03 (DOD 2013) requires that the primary load-bearing members, such as beams, girders, and spandrels, are exempted from tie forces unless these members and their joints can satisfy the specified rotation demands (see Fig. 2.2).

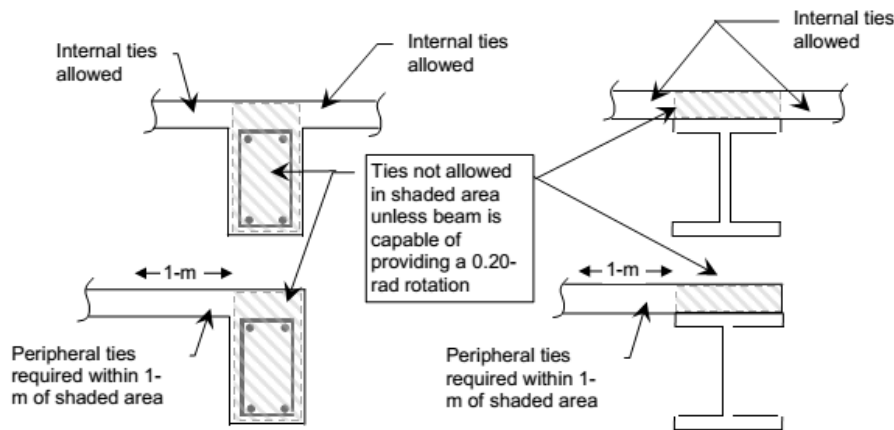


Fig. 2.2: Location restriction for internal and peripheral ties (DOD 2013)

Progressive collapse caused by failure of corner and penultimate supporting members cannot be mitigated through tie forces due to insufficient lateral restraints to develop catenary action (Stevens et al. 2011). Furthermore, the effectiveness of tie forces in preventing the spread of local damage largely depends on initial damage to a single supporting element at one time, and it fails to survive local failure caused by simultaneous removal of two adjacent columns (Hansen et al. 2006). Therefore, direct

design approach has to be employed in case that catenary or membrane action in the floor system cannot be successfully mobilised.

2.2.2 Direct design approach

Direct design approach represents explicit considerations of resistance to progressive collapse in the design (ASCE 2006). It stems from the design methods defined by Ellingwood and Leyendecker (1978) and includes alternate path method and enhanced local resistance method.

2.2.2.1 Alternate path method

Table 2.1: Difference between earthquake and progressive collapse (DOD 2013)

Event	Earthquake	Progressive collapse
Extent	Entire structure	Localized to the column/wall removal area
Load types	Horizontal and temporary	Vertical and permanent
Damage distribution	Distributed throughout the structure	Localized and prevented from progressing
Connection and member response	Cyclic loads with increasing magnitude, without axial loading	One half cycle of loading, in conjunction with a significant axial load

The alternate path method allows a local failure to occur while seeking to restrain its extensive propagation through the mobilisation of alternate load paths in the bridging structural members (ASCE 2006). In this method, threat-independent column removal scenarios are considered, and their extent and locations are prescribed in the UFC 4-023-03 (DOD 2013). Analysis procedures, including elastic static, inelastic static and inelastic dynamic analyses, are adopted to assess structural behaviour after the removal of supporting members. However, the acceptance criteria for members and joints in the UFC 4-023-03 (DOD 2013) are largely imported from ASCE 41-06 (ASCE 2007) under cyclic loading conditions. Significant differences exist between the two guidelines in terms of extent, load types, damage distribution, connection and member response (DOD 2013), as shown in **Table 2.1**. Therefore, the criteria for seismic design are considered too conservative when applied to progressive collapse scenarios (Foley et al. 2007).

The alternate path method consists of diverse vertical load redistribution mechanisms, whereas significant discrepancies exist between current design standards and guidelines. The Building Regulation 2000 (ODPM 2004) and the General Services Administration (GSA 2003) do not indicate whether the alternate path method involves a flexural or a catenary mechanism (Gurley 2008), while development of flexural and catenary actions is implicitly incorporated in the UFC 4-023-03 (DOD 2013). Indeed, following the nominal removal of a supporting column, flexural mechanism is mobilised at relatively small deflections (comparable to in-service deflections) to meet the gravity load requirements (Gurley 2008). With increasing vertical deformations, the load-resisting mechanism is shifted to catenary action. It is possible to calculate the resistance provided by the flexural mechanism but not by the catenary mechanism, as the latter greatly relies on damage locations and available lateral restraints from adjacent structures. Therefore, it is imperative to propose an analytical model to quantify the resistance of building structures when different load redistribution mechanisms are mobilised.

2.2.2.2 Enhanced local resistance method

In the enhanced local resistance method, structural members at the specified locations are intentionally strengthened to reduce the likelihood or extent of the initial damage (DOD 2013). Shear capacity of structural members is designed to exceed their flexural capacity so as to ensure a more ductile and controllable failure mode. This method is oriented to the threat that can be quantified through a risk analysis or specified through performance-based design requirements. It also acts as an effective alternative to provide a level of protection for structures unable to mitigate the failure of corner or penultimate supporting members through tie forces (Stevens et al. 2011).

2.2.3 Relationship between indirect and direct approaches

As a matter of fact, both tie forces and alternate path method attempt to avert the propagation of local failures via the activation of catenary action, which represents the last line of defence against progressive collapse. However, development of catenary action highly relies on the stiffness of lateral restraints and the extent of localised damage in the affected spans. Thus, in accordance with the level of

occupancy and building function or criticality (see **Table 2.2**), the prerequisite of the mobilisation of catenary action prompts the combined application of the three methods in the UFC 4-023-03 (DOD 2013), as shown in **Table 2.3**.

Table 2.2: Occupancy categories (DOD 2013)

Nature of occupancy	Occupancy category
Buildings in Occupancy Category I in Table 2-2 pf UFC 3-301-01 Low occupancy buildings, as defined by UFC 4-010-01	I
Buildings in Occupancy Category II in Table 2-2 pf UFC 3-301-01 Inhabited buildings with less than 50 personnel, primary gathering buildings, billeting, and high occupancy family housing	II
Buildings in Occupancy Category III in Table 2-2 pf UFC 3-301-01	III
Buildings in Occupancy Category IV in Table 2-2 pf UFC 3-301-01 Buildings in Occupancy Category V in Table 2-2 pf UFC 3-301-01	IV

Table 2.3: Occupancy category and design requirements (DOD 2013)

Occupancy category	Design requirement
I	No specific requirements
II	Option 1: tie force for the entire structure and enhanced local resistance for the corner and penultimate columns or walls at the first storey. or Option 2: alternate path for specified column and wall removal locations.
III	Alternate path for specified column and wall removal locations; enhanced local resistance for all perimeter first storey columns or walls
IV	Tie forces; Alternate path for specified column and wall removal locations; enhanced local resistance for all perimeter first storey columns or walls.

For occupancy category II structures, the limitation of tie forces in mitigating damage caused by failure of corner or penultimate supporting members requires the application of enhanced local resistance method to these members to reduce the possibility of initial damage (Stevens et al. 2008). In addition, tie forces are difficult to be used in existing or non-ductile floor systems. These limitations enable the application of alternate path method as an alternative to evaluating existing structures (DOD 2013). Concerning occupancy category III, considerations are given to the increased probability of deliberate attacks and the greater extent of local damage, which exceeds the assumption in the alternate path method that only one supporting member is removed in an event (DOD 2013). Therefore, additional protection is provided by the enhanced local resistance method to minimise the likelihood of column/wall failure at the perimeter. With regard to occupancy category IV buildings,

the addition of tie forces supplements the flexural and catenary action resistances through alternate path method. Additionally, the enhanced local resistance method is applied to reduce the possibility of simultaneous removal of two adjacent columns or walls (DOD 2013). However, there are several limitations associated with the alternate path method. It does not give any guidance on dealing with transfer structures such as plate girders or deep beams, neither does it give any guidance if more than one column is to be removed.

2.3 Experimental Tests under Progressive Collapse Scenarios

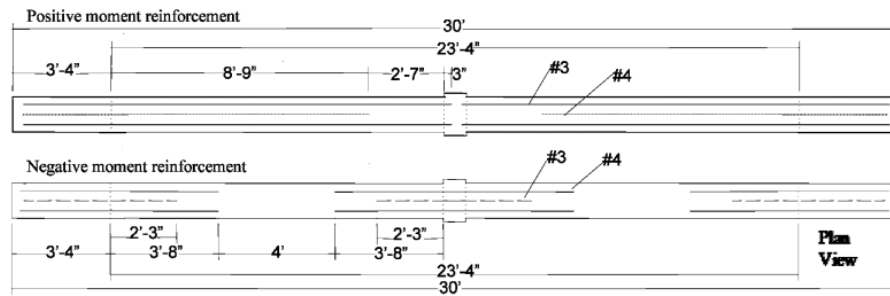
Typically, accompanying flexural action, axial compression increases gradually in axially-restrained beams, which represents the onset of compressive arch action (CAA) (Izzudin and Elghazouli 2004a). At comparatively large deformations, crushing of concrete in the flexural compression zone slowly reduces the beam axial force. At the moment when the beam axial force change from compression to tension, catenary action sets in as the last line of defence against progressive collapse (Su et al. 2009), provided structural joints are sufficiently ductile and continuous and there is sufficient axial restraint on both ends of the beam (Gurley 2008). Catenary action refers to a chain-like mechanism in structural members subjected to large deformations. It is dominant when the deflection is comparable or greater than the depth of the section (Izzudin and Elghazouli 2004b). At catenary stage, tensile strength of the bridging beam is activated to transfer vertical loads from the damaged region to the undamaged parts. However, the capacity of structural joints to undergo large deformations while maintaining their continuity to carry tension forces is not taken into account in conventional design practice. Consequently, experimental tests were performed to investigate the deformation capacity and integrity of joints subject to column removal scenarios.

2.3.1 Quasi-static tests under column removal scenarios

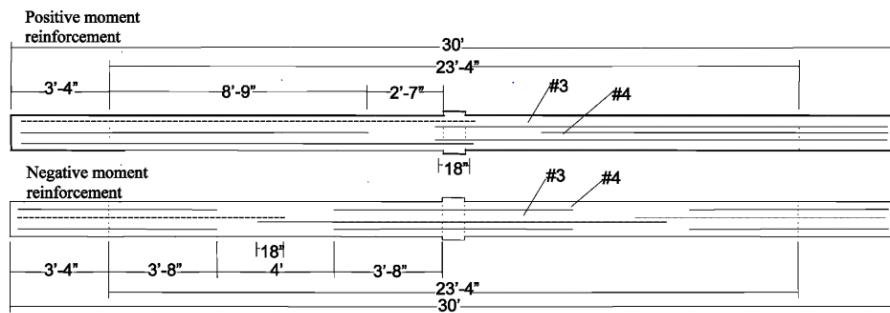
2.3.1.1 Orton's tests on reinforced concrete beams

In order to evaluate the resistance of existing reinforced concrete beams against progressive collapse and the efficiency of carbon fibre-reinforced polymer (CFRP)

in providing continuity, Orton (2007) tested eight reinforced concrete beams under middle column removal scenarios. Vertical and axial restraints were provided at the beam ends. Three point loads were applied to represent uniformly distributed load on the beam.



(a) NR-2

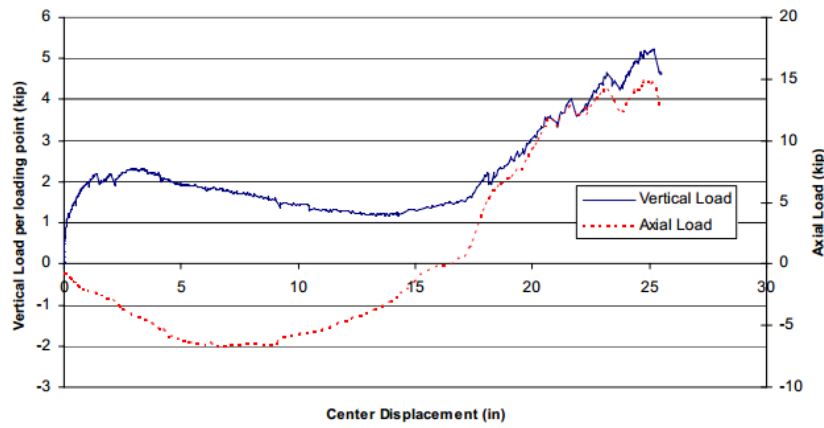


(b) CR-1

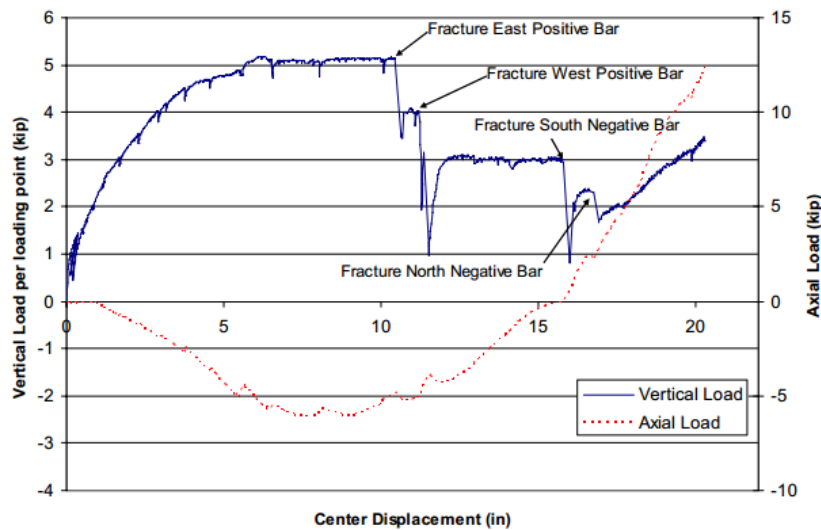
Fig. 2.3: Reinforcement details in beams (Orton et al. 2009)

Among all the beams, NR-2 represented a reinforced concrete beam with discontinuous bottom reinforcement in the middle joint but not strengthened by CFRP (see **Fig. 2.3(a)**). **Fig. 2.4(a)** shows the measured vertical load and axial force in the beam. The maximum vertical load in the compression phase was only 10.3 kN (2.3 kip) at each loading point, accounting for 23% of the required load for progressive collapse resistance by General Service Administration, U.S. Catenary effect commenced in the beam beyond the descending branch of vertical load. Localisation of cracks at the middle joint face enabled the beam to develop large vertical deflection prior to failure, as shown in **Fig. 2.5(a)**. Eventually, the beam attained its catenary action capacity of 23.4 kN (5.2 kip). In beam CR-1, continuous longitudinal reinforcement was provided in the beam and embedded in the joint, as shown in **Fig. 2.3(b)**. Correspondingly, the maximum vertical load was increased to

22.5 kN (5 kip) in the compression phase, 117% greater in comparison with NR-2. However, lower vertical load was obtained in the catenary regime of CR-1 than NR-2, due to premature fracture of negative moment reinforcement (see **Fig. 2.5(b)**), even though CR-1 was able to mobilise a greater tension force in the beam than NR-2, as shown in **Fig. 2.4(b)**. Further experimental tests on reinforced concrete beams retrofitted by CFRP also demonstrate that negative moment reinforcement and CFRP played a crucial role in developing catenary action in the beam under column removal scenarios (Orton et al. 2009). By contrast, positive moment reinforcement and CFRP were favourable to the development of flexural action at relatively small vertical displacement.

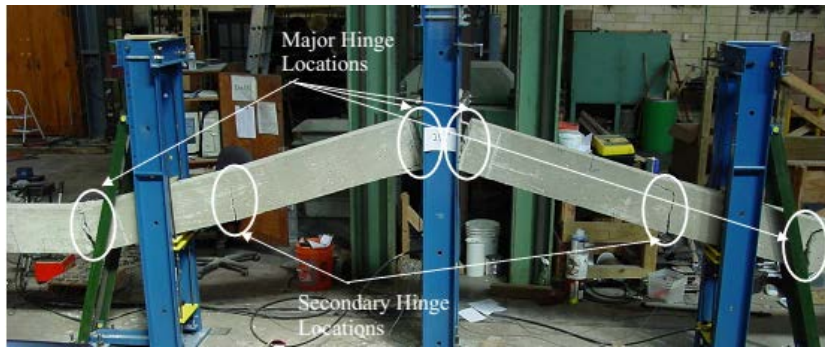


(a) NR-2



(b) CR-1

Fig. 2.4: Vertical and axial loads in beams (Orton 2007)



(a) NR-2



(b) CR-1

Fig. 2.5: Failure modes of beams (Orton 2007)

2.3.1.2 Yu's tests on beam-column sub-assemblages and frames

To identify the load redistribution mechanisms in the bridging beam after column removal and to investigate the resistance of reinforced concrete structures to mitigate progressive collapse, Yu and Tan (2013b) tested two beam-column sub-assemblages designed per ACI 318-05 with seismic and non-seismic detailing (see **Fig. 2.6**). **Fig. 2.7** shows the test setup for the sub-assemblages. To capture the reaction forces at each support, restraints on the end stub were decomposed into two horizontal restraints and one vertical restraint. The reaction forces were measured through load cells. Two transverse frames were used to prevent out-of-plane deflections of the specimens. During each test, displacement-controlled vertical load was applied on the middle column stub through a servo-hydraulic actuator.

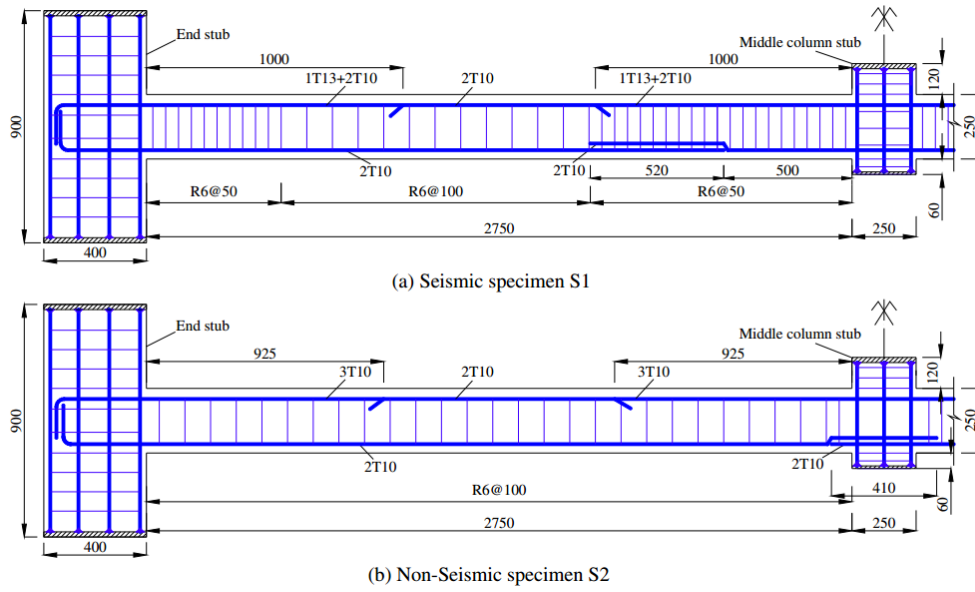


Fig. 2.6: Reinforcement detailing of beam-column sub-assemblages (Yu and Tan 2013b)

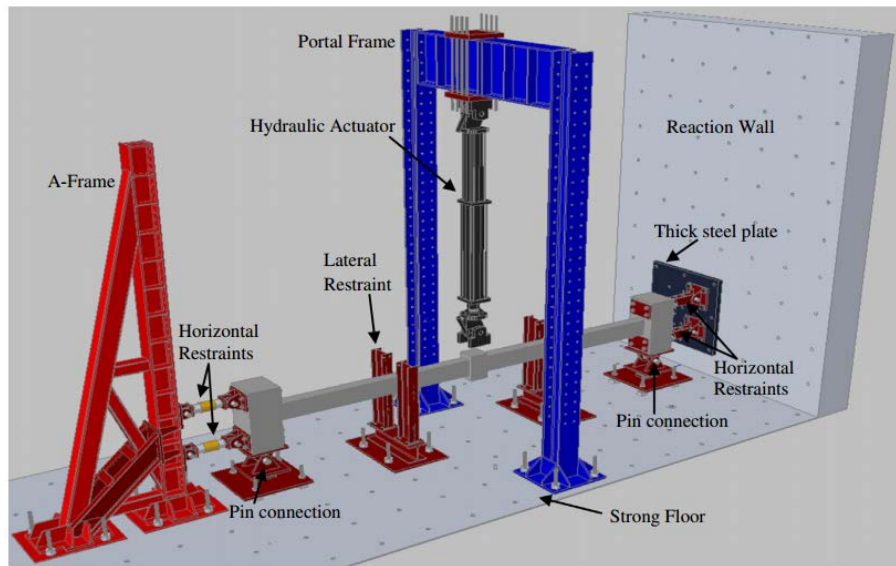


Fig. 2.7: Test setup for beam-column sub-assemblages (Yu and Tan 2013b)

Fig. 2.8 shows the measured vertical load and horizontal reaction force. Under column removal scenarios, three phases of force redistribution mechanisms, namely, flexural action, CAA, and catenary action, were classified in the bridging beam (see **Fig. 2.8(a)**). At the early stage, the specimens behaved in flexural action and cracks formed and spread in the vicinity of the middle joint. After the formation of plastic hinges at the critical sections, CAA kicked in to sustain additional vertical load and axial compressive force developed in the beam, as shown in **Fig. 2.8(b)**. Beyond the

CAA capacity of the beam, crushing of concrete in the flexural compression zones induced a descending branch of the vertical load (see **Fig. 2.8(a)**). At large deformation stage, catenary action was mobilised to resist the vertical load on the middle joint, with axial tension force developed in the beam. Final failure of the sub-assemblages was caused by fracture of beam top reinforcement near the end stub, as shown in **Fig. 2.9**.

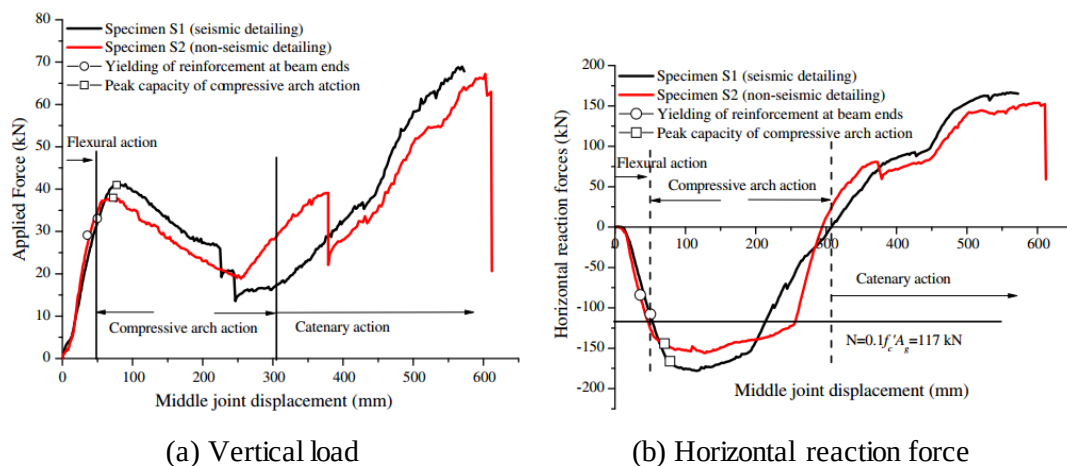


Fig. 2.8: Variations of vertical load and horizontal reaction force with middle joint displacement (Yu and Tan 2013b)

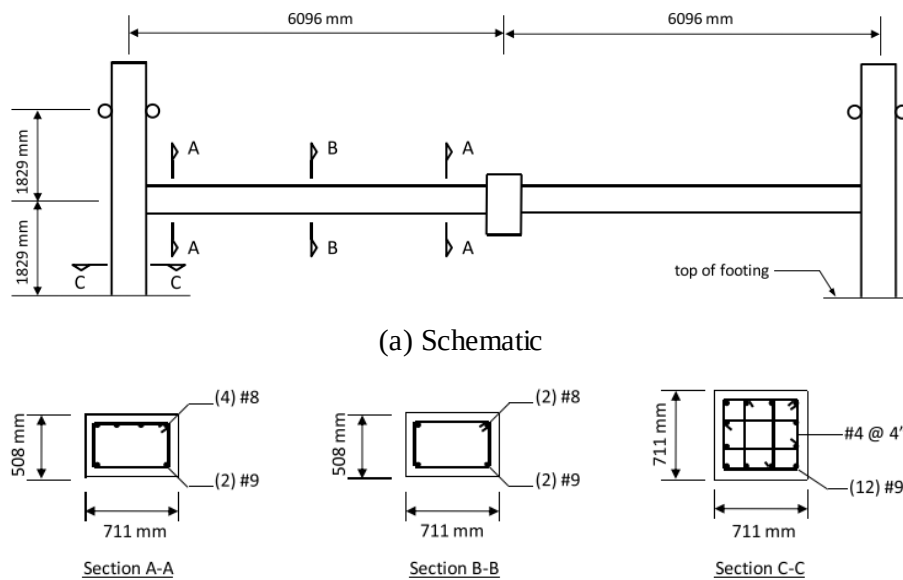


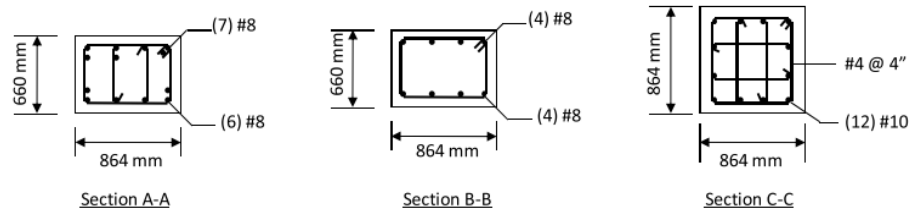
Fig. 2.9: Failure mode of sub-assemblage S1 (Yu and Tan 2013b)

At CAA stage, sub-assemblage S1 developed the maximum load of 41.6 kN. Once catenary action started, vertical load was gradually increased to 68.9 kN, 65% greater than the CAA capacity. Thus, under relatively rigid boundary conditions, the sub-assemblage was able to mobilise effective catenary action to mitigate progressive collapse. A comparison between sub-assemblages with seismic and non-seismic detailing indicates that contrary to previous suggestions (Corley 2002; Corley 2004; Hayes et al. 2005a), seismic detailing contributed little to the collapse resistance of sub-assemblages under column removal scenarios.

Further experimental tests were conducted on beam-column sub-assemblages to investigate the effect of reinforcement ratios and span-depth ratios on the CAA and catenary action (Yu and Tan 2013c). Most of the sub-assemblages were demonstrated to be capable of developing significant catenary action, except those with comparatively short span which exhibited premature shear failure prior to commencement of catenary action. It is reported that CAA substantially contributed to structural resistance of sub-assemblages with lower longitudinal reinforcement ratios and smaller span-depth ratios. By increasing the reinforcement ratio and span-depth ratio, the contribution of catenary action to structural resistance became more significant. However, at the frame level, only limited catenary action developed in the beam with conventional reinforcement detailing, due to consecutive fracture of reinforcing bars at the column face (Yu 2012). Therefore, special reinforcement detailing, such as an additional middle layer of reinforcement in the beam, partial debonding of embedded reinforcement in the middle joint, and partial hinge at the beam ends, were utilised in an attempt to enhance collapse resistance of reinforced concrete frames at catenary action stage (Yu and Tan 2014).

2.3.1.3 Sadek's tests on reinforced concrete assembly





(c) Section properties of SMF assembly

Fig. 2.10: Reinforcement details of reinforced concrete beam-column assemblies (Sadek et al. 2011)

Sadek et al. (2011) carried out experimental tests on reinforced concrete beam-column assemblies. Two full-scale specimens were designed in accordance with ACI 318-02. One represented a portion of an intermediate moment frame (IMF) for Seismic Design Category C, and the other for a fraction of a special moment frame (SMF) for Seismic Design Category D. **Fig. 2.10** shows the details of the reinforced concrete assemblies.

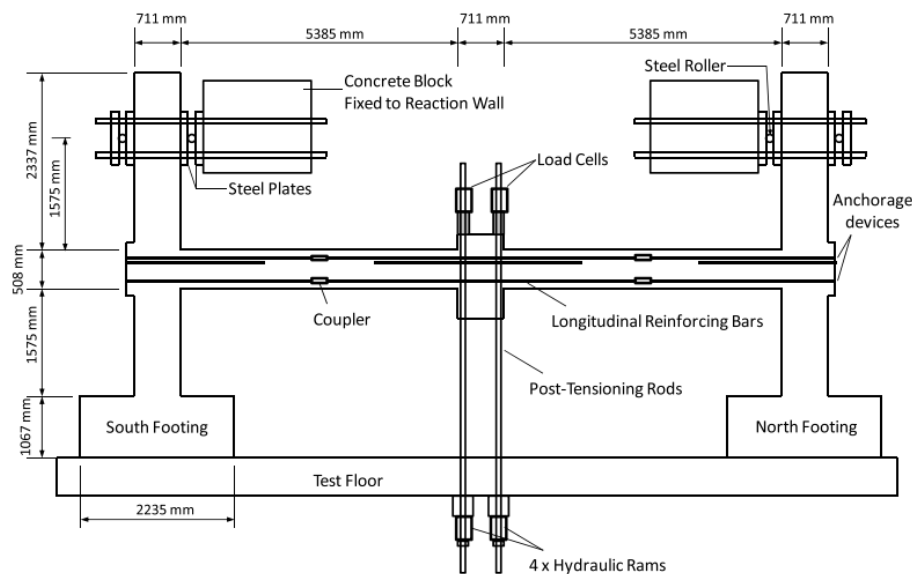


Fig. 2.11: Test setup and instrumentation for beam-column assemblies (Sadek et al. 2011)

Fig. 2.11 shows the test setup for the assemblies. The footings of exterior columns were anchored to the testing floor, and horizontal restraints were applied to the top of exterior columns. Thereafter, displacement-controlled vertical load was applied to the middle column stub at a rate of 25 mm/min. **Fig. 2.12** shows the vertical load-middle column displacement history of the assemblies. With increasing vertical displacement, flexural resistance of the beam-column assemblies was mobilised gradually up to an

initial peak of vertical load. A further increase in vertical displacement resulted in crushing of concrete in the compression zones, which in turn reduced the vertical load. At large deformation stage, development of catenary action in the bridging beams enhanced the resistance of the assemblies. Once fracture of bottom reinforcement occurred in the middle joint, the vertical load dropped sharply, indicating failure of the assemblies. For each specimen, catenary action provided a greater vertical load resistance than flexural action. It indicates that beam-column assemblies with seismic design were capable of mitigating collapse by means of catenary action at large deformations. Compared to IMF, SMF was able to resist 2.25 times greater vertical load at failure. It implies the applicability of seismic design in mitigating progressive collapse, which agrees well with the conclusion drawn by Hayes et al. (2005b).

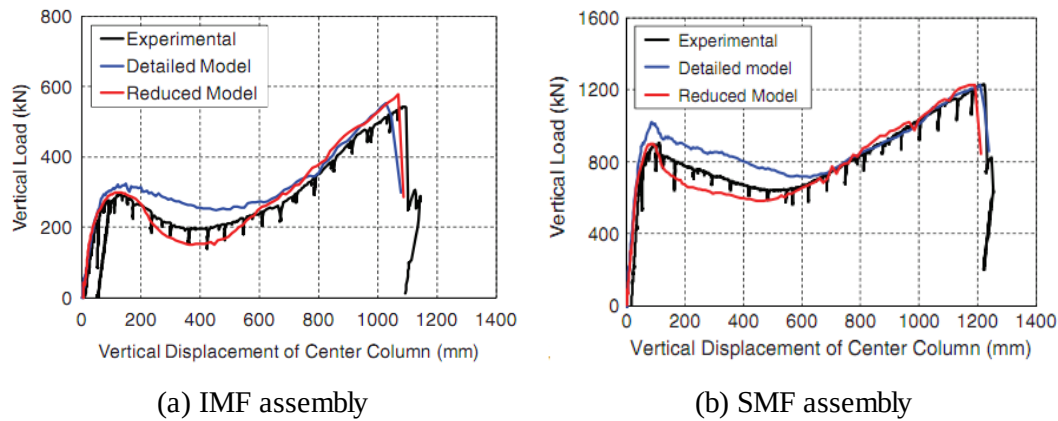


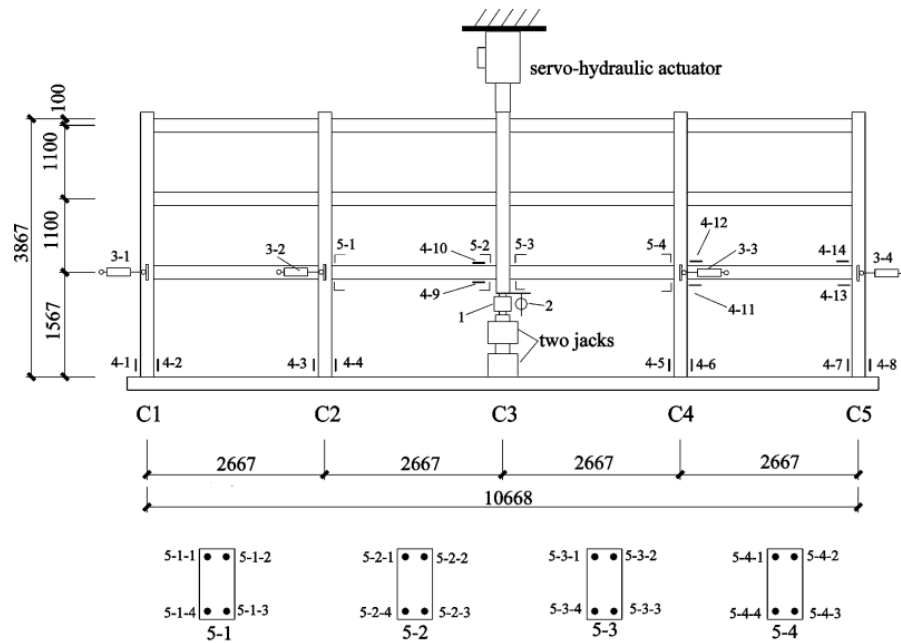
Fig. 2.12: Vertical load-middle joint displacement histories (Sadek et al. 2011)

Besides failure of the bridging beams under column removal scenarios, severe cracking were also observed on the exterior columns, in particular, the joint zone (Lew et al. 2011). However, the horizontal reaction forces were not measured in the tests, and it was not possible to evaluate the flexural strength of the column and the shear strength of the joint when subjected to catenary action.

2.3.1.4 Yi's tests on reinforced concrete frame

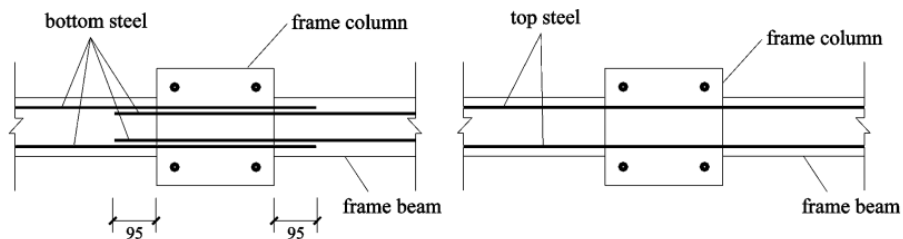
A four-span and three-storey one-third scale planar frame was tested by Yi et al. (2008) to study the behaviour of reinforced concrete frame subjected to column loss. The model frame was designed in accordance with the Chinese concrete design code. **Fig. 2.13** shows the test setup and instrumentations for the frame. The frame was built

on a foundation beam fixed to the reaction floor. Prior to testing, a constant vertical load of 109 kN was applied to the top of the middle column by a servo-hydraulic actuator and it was supported by two jacks on the first floor. Then the mechanical jacks were lowered step-by-step to apply vertical load to the middle column until steel bars fractured near the end of the first floor beam. During loading, change in the axial force in the middle column was recorded with a load cell mounted on the top of the mechanical jacks. Linear variable differential transducers were affixed to specific locations to measure the vertical and lateral displacements of the model frame.



1: load cell; 2: micrometer; 3-1 to 3-4: displacement transducers;
4-1 to 4-12: concrete strain gauges; 5-1 to 5-4 section having steel strain gauges

(a) Details of model frame and instrumentation



(b) Reinforcement detail in beam-column joint

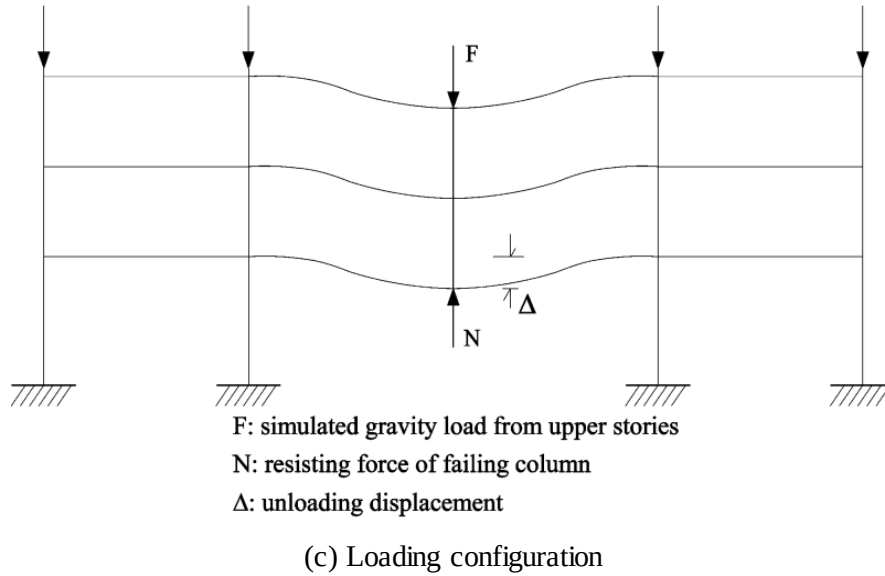


Fig. 2.13: Test setup and instrumentation for beam-column assemblies (Yi et al. 2008)

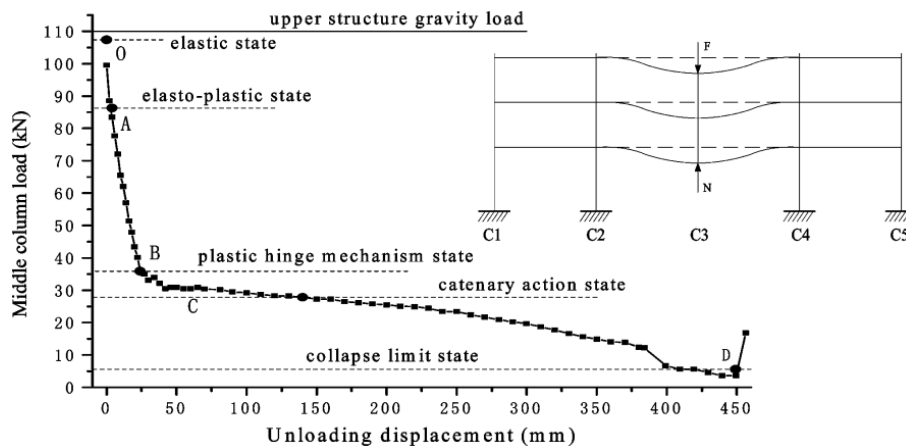


Fig. 2.14: Variation of load cell reaction force versus middle column displacement (Yi et al. 2008)

Five states were identified on the basis of the relationship of the measured load by the load cell and the vertical displacement, as shown in **Fig. 2.14**. At a displacement less than 5 mm, beams in the frame were at the elastic stage. Then the inelastic stage continued until yielding of steel bars occurred at the interface of the middle column on the first floor. Formation of plastic hinges at the beam ends indicated the start of plastic stage mechanism. Softening stage induced by crushing of concrete in the plastic hinge region was not significant, possibly due to less flexural stiffness of adjacent columns. A further increase in vertical displacement mobilised catenary

action and tension cracks propagated across the whole beam section. Eventually, rupture of bottom steel bars near the middle column led to the collapse of the frame.

In addition to the vertical deflection of the middle column, horizontal displacement of the column on the first floor was also measured, as shown in **Fig. 2.15**. Negative deflections represent the measured point moving away from the middle column, and positive values refer to deflections towards the middle column. Hence, at the initial stage, net compression force existed in the first-floor beams, which pushed the columns out. At catenary action stage, axial tension force in the beam pulled the columns towards the middle column. The results agree well with the measured horizontal force on the beam-column sub-assemblages (Yu and Tan 2013c). The significant lateral deflections of the columns at the first floor level demonstrated the deleterious effect of beam catenary action on the stability of adjacent columns.

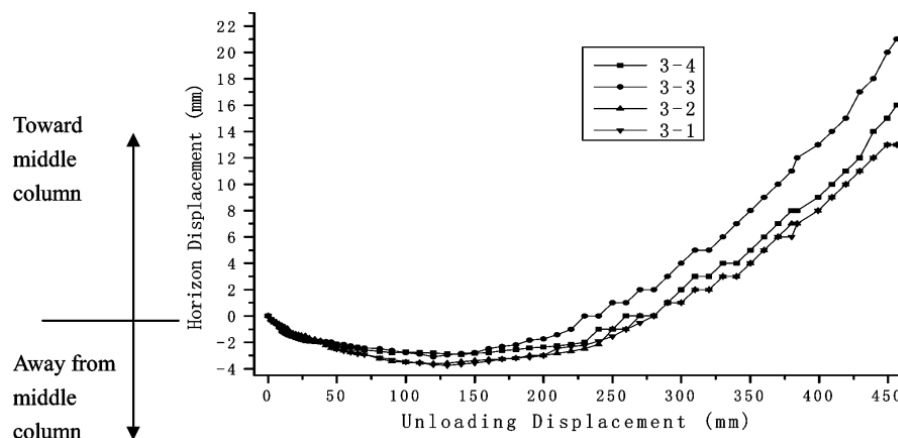


Fig. 2.15: Effect of middle column displacement on horizontal displacement of columns at first floor level (Yi et al. 2008)

2.3.1.5 Main's tests on precast concrete assembly

To examine the effectiveness of precast concrete buildings in resisting progressive collapse, Main et al. (2014) tested a precast concrete beam-column subassembly with welded connection. **Fig. 2.16** shows the details of the beam-to-column connection. Steel angles were embedded in the spandrel beam and welded to the longitudinal reinforcement. The beam was connected to the external column through steel link plates welded to the steel angles in the beam and the steel plate in the column.

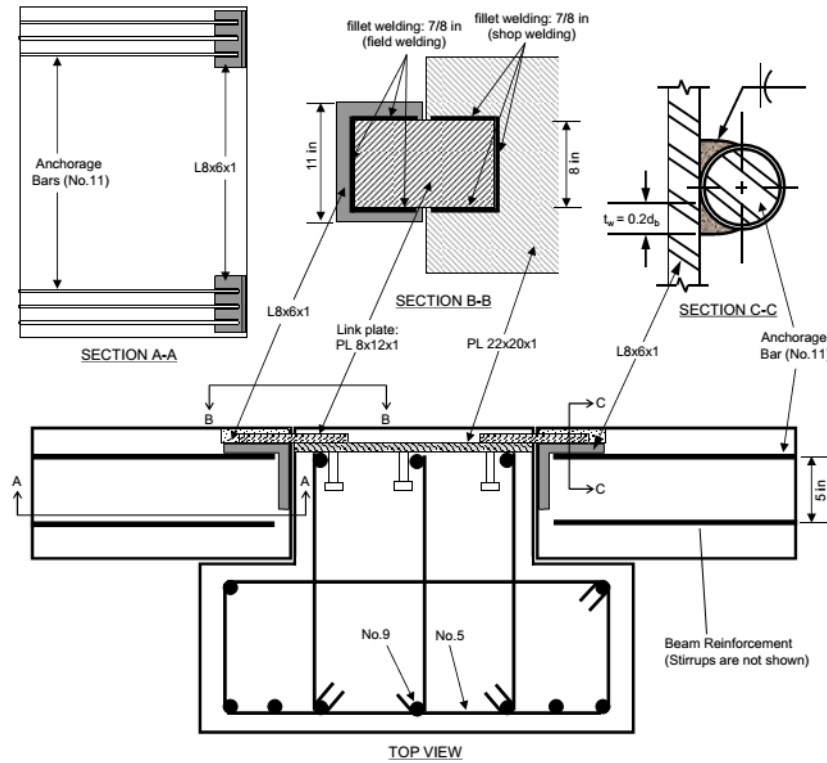


Fig. 2.16: Beam-to-column connection details for SMF building (Main et al. 2014)

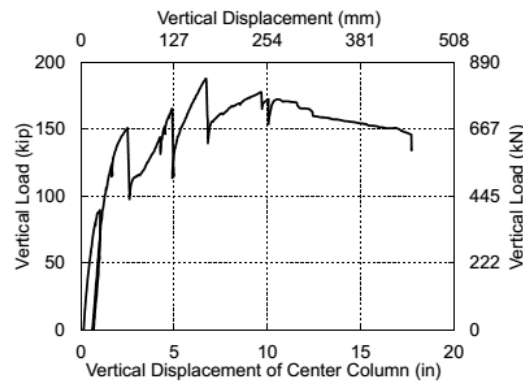


Fig. 2.17: Vertical load versus vertical displacement of centre column (Main et al. 2014)

Fig. 2.17 shows the vertical load-displacement curve of the subassembly. Limited vertical load resistance of the subassembly was obtained due to premature fracture of beam longitudinal reinforcement at the weld location (see **Fig. 2.18**). In nature, the fracture resulted from a reduction in the ductility of the reinforcing bars in the heat-affected zone. Following the rupture of reinforcement, arch action in the specimen slightly increased the vertical load. However, significant plastic deformation of the link plates and severe cracking and spalling of concrete eventually hindered the

development of vertical load. Test results indicate that this type of welded connection in precast concrete structures was not able to develop efficient alternate load path under column removal scenarios.

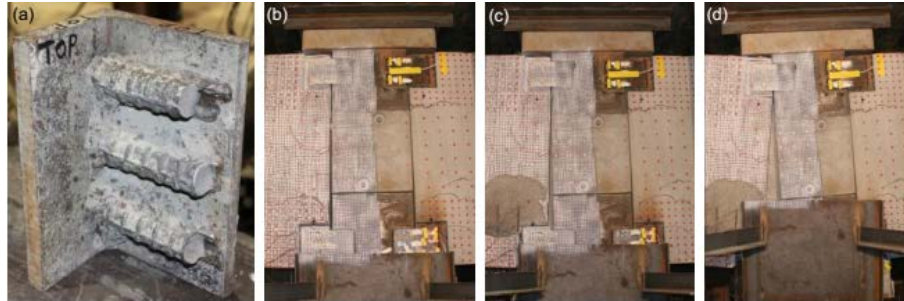


Fig. 2.18: Failure mode at connections to centre column (Main et al. 2014)

2.3.2 Dynamic tests

In addition to experimental tests on isolated structural members under column removal scenarios, on-site dynamic tests were also conducted on reinforced concrete structures to gain insight into the vertical load redistribution and the deformation of the bridging beam under different column loss scenarios (Sasani et al. 2007; Sasani and Sagioglu 2008; Sasani and Sagioglu 2010).

2.3.2.1 Interior column removal scenarios

Sasani and Sagioglu (2010) investigated the resistance and gravity load redistribution of a reinforced concrete structure subject to a dynamic interior column removal. **Fig. 2.19** shows the plan of the building in which the circled column on the first floor was removed by explosion. **Fig. 2.20** shows the vertical displacement of joint C3 above the removed column on the second and seventh floors. It was observed that the vertical displacement of the joint on the seventh floor was substantially smaller than that on the second floor, due to a reduction in the axial compression force in column C3. Following the removal of the supporting column, the axial compression forces in the columns above were reduced significantly, as shown in **Fig. 2.21**. Thus, the columns elongated due to reduced axial forces. Eventually, the axial compression forces in column C3 attained its minimum value on the second floor and maximum value on the eighth floor. Experimental results indicated that the reinforced concrete structure was able to resist the loss of one interior column on the first floor

without progressive collapse. Furthermore, contribution of the bridging beams to the resistance of the structure decreased towards the top of the structure due to elongation of columns above the local damage.

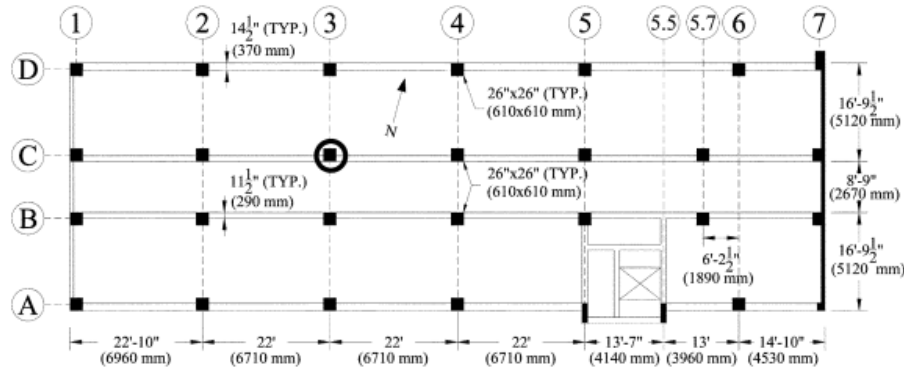


Fig. 2.19: Location of column removal (circled) (Sasani and Sagirolgu 2010)

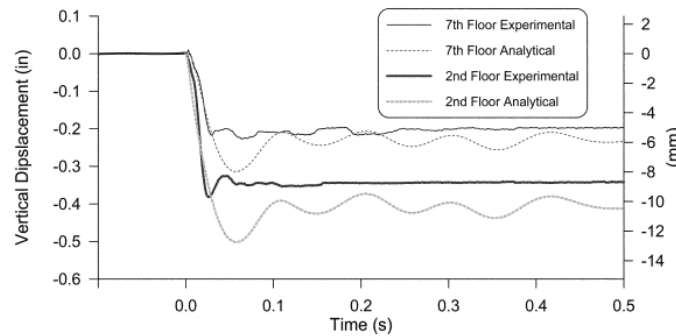


Fig. 2.20: Vertical displacements of second and seventh floor joints above removed column (Sasani and Sagirolgu 2010)

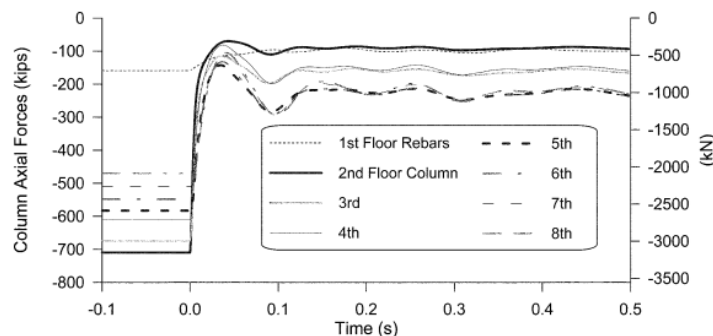


Fig. 2.21: Axial compressive force in column C3 on different floors (Sasani and Sagirolgu 2010)

2.3.2.2 Exterior column removal scenarios

Besides instantaneous loss of an interior column, the progressive collapse resistance of a reinforced concrete structure was evaluated under an exterior column removal

scenario (Sasani et al. 2007). **Fig. 2.22** shows the plan of the building and the location of a removed exterior column. It is notable that the effect of direct air blast on the structure was not included in the experimental study.

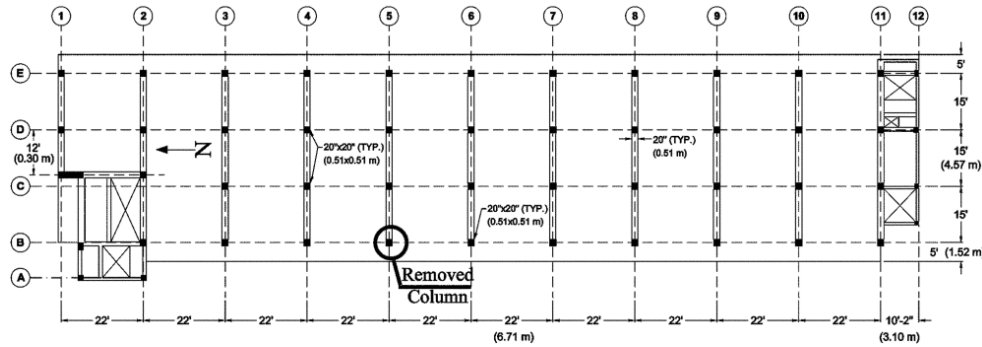


Fig. 2.22: Typical plan of the building and location of column removal (Sasani et al. 2007)

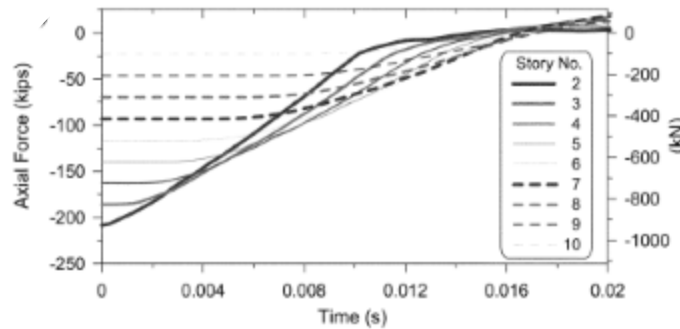


Fig. 2.23: Variations of axial forces in column B5 (Sasani et al. 2007)

Following the removal of the first storey column, the second storey column elongated due to vertical movement of the joint, which reduced the axial force in the column. A similar reduction of column axial force was also observed on the storeys above. **Fig. 2.23** shows the reduction of axial compression force in the columns with time. The compression forces on the lower storeys reduced faster than those on the storeys above. Finally, the compression forces in the columns were substantially reduced in comparison with those at the original state. However, progressive collapse did not occur in the structure as a result of development of Vierendeel action in the beams and columns, as shown in **Fig. 2.24**. Vierendeel action is characterised by relative displacement between the beam ends and double-curvature deformation of the beams and columns (Sasani et al. 2007). It reversed the bending moment in the vicinity of the removed column. Therefore, pull-out failure of bottom reinforcement in the beam has to be prevented above the removed exterior column.

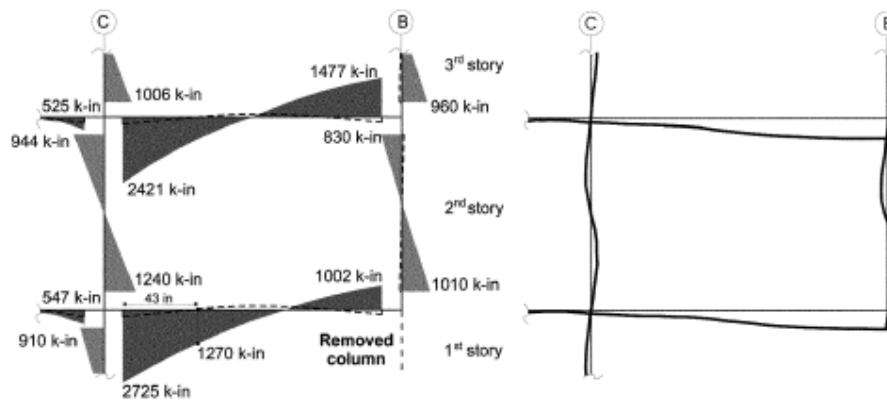


Fig. 2.24: Bending moment diagram and deformed shape of axis 5 (Sasani et al. 2007)

So far, only Main et al. (2014) tested the behaviour of welded joints in precast concrete structures under quasi-static column removal scenarios. The joints are vulnerable to mitigate progressive collapse due to the reduced ductility of steel reinforcement in the heat-affected zones. However, other types of precast concrete joints, such as those recommended by FIB (2002), have not yet been experimentally investigated. Thus, experimental programme on the resistance and deformation capacity of precast concrete beam-column joints has to be conducted. Besides the joint detailing, the effect of boundary conditions on structural performance of precast concrete joints needs to be considered in the experimental programme. Furthermore, development of CAA and subsequent catenary action in the bridging beams imposes additional horizontal forces to adjacent columns, which may lead to premature flexural or shear failure of the columns before the beams attain the catenary action capacity (Lew et al. 2011; Yu 2012). Therefore, to prevent progressive collapse of precast concrete structures, special attention has to be paid to flexural and shear resistances of adjacent columns.

2.4 Engineered Cementitious Composites (ECC)

2.4.1 Material properties

ECC is a high-performance fibre-reinforced cementitious composite which features strain-hardening behaviour and superior strain capacity in tension (Li 2003). **Fig. 2.25** shows a typical tensile stress-strain curve of ECC. A tensile strain capacity up to 5%

in uniaxial tension can be achieved with only 2% fibres by volume. Extensive research studies have been conducted on the material properties of ECC. Its matrix toughness and fibre bridging strength have been optimised by means of the mechanical model so as to achieve high tensile strength and ductility under quasi-static loadings (Li et al. 2001; Li et al. 2002; Yang and Li 2010). Fibre, matrix and fibre/matrix interface have been designed for impact resistance under higher loading rates (Yang and Li 2012).

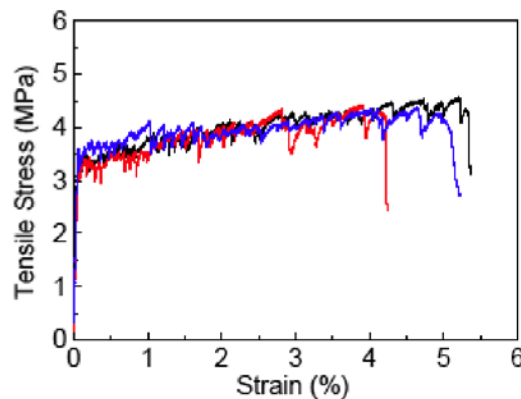


Fig. 2.25: Uniaxial tensile stress-strain curves of ECC with 2% PVA fibres (Li 2003)

2.4.2 Structural performance under various loading conditions

Regarding the performance of structural members and joints made of ECC, experimental programmes have been conducted under various loading conditions. **Fig. 2.26** shows the load-displacement response of columns under reversed cyclic load. Reinforced ECC columns exhibited greater load-carrying capacity and ductility in comparison with reinforced concrete members (Fischer and Li 2002b). Compatible deformations between reinforcement and ECC postponed localisation of cracks in the plastic hinge region beyond yielding of steel bars, thereby resulting in higher energy absorption under large deformations. In terms of shear design, transverse steel reinforcement could be eliminated in the reinforced ECC column, as ECC was able to provide sufficient shear resistance for the column. Furthermore, confinement effect of ECC prevented buckling of steel reinforcement when subjected to compression. Reinforced ECC beams exhibited similar hysteretic behaviour to the columns when subjected to cyclic load reversal (Fukuyama et al. 2000). Moreover, brittle shear

failure and bond splitting failure in the beam were prevented by using ECC in place of conventional concrete, as shown in **Fig. 2.27**.

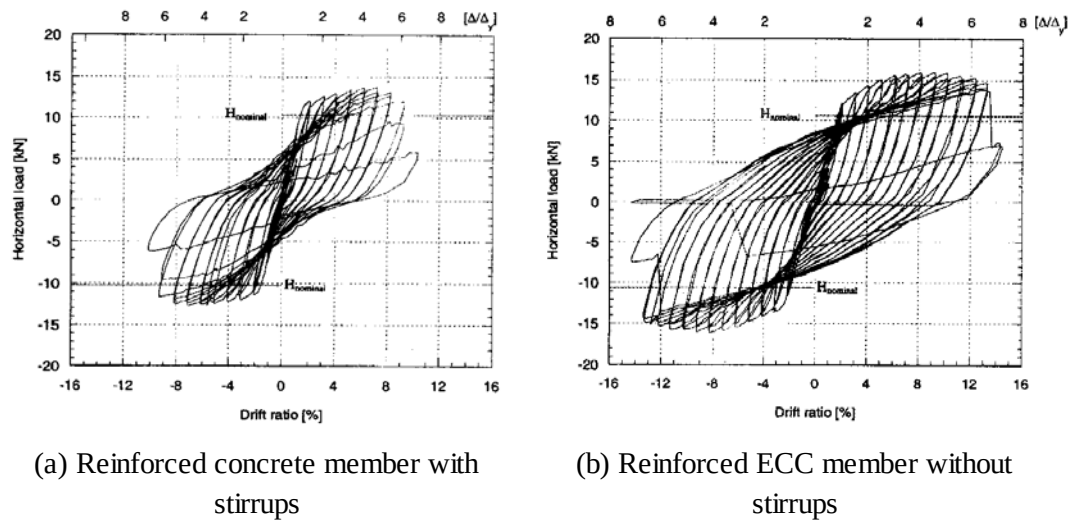
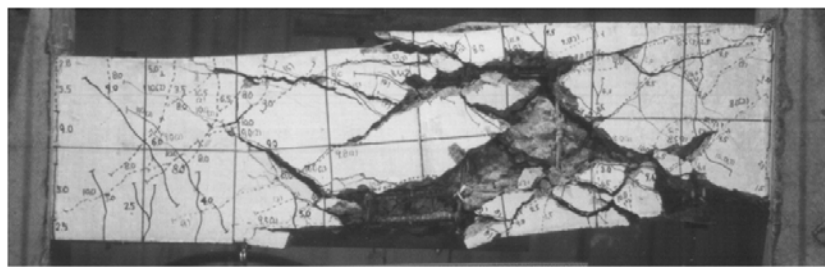
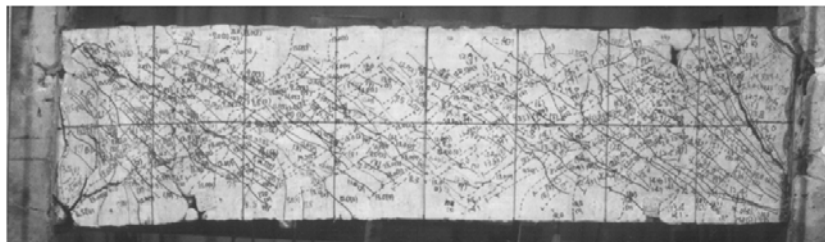


Fig. 2.26: Load-deformation responses of columns subjected to reversed cyclic loading (Fischer and Li 2002a)



(a) Reinforced concrete beam



(b) PVA-ECC beam

Fig. 2.27: Damage properties of beams (Fukuyama et al. 2000)

In addition to ECC members subjected to cyclic loading reversals, flexural behaviour of ECC beams was also experimentally investigated by Yuan et al. (2013). Compared to concrete specimens, ECC beams developed higher load-carrying capacity, shear resistance and ductility. To achieve economy, ECC was only applied in the compression (BREC-C) and tension (BREC-T) zones of concrete/ECC composite

beams. **Fig. 2.28** depicts the failure modes and the load-deflection curves. It indicates that ECC was more effective in resisting flexural loads when applied in the tension zone of BREC-T compared to in the compression zone of BREC-C. Nevertheless, by increasing the depth of ECC layer, final failure was shifted from rupture of fibre-reinforced polymer reinforcement to crushing of concrete in the compression zone.

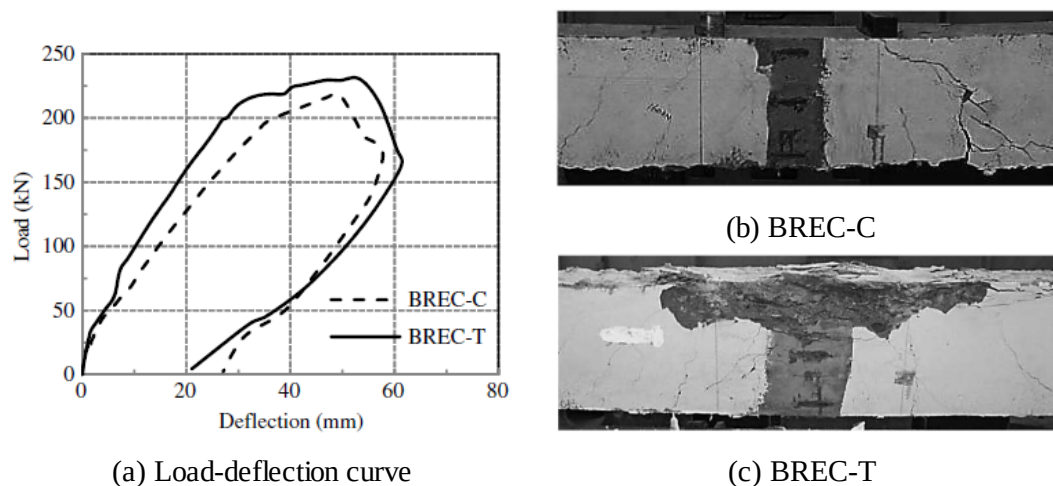


Fig. 2.28: Load-deflection curves and failure modes of concrete/ECC composite beams (Yuan et al. 2013)

2.4.3 Interactions between ECC and reinforcement

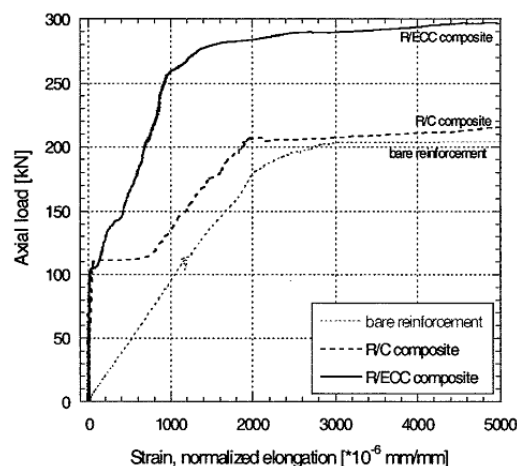


Fig. 2.29: Load-deformation responses of specimens in uniaxial tension (Fischer and Li 2002b)

To investigate the interactions between ECC and steel reinforcing bars, uniaxial tension tests were conducted on reinforced ECC members (Fischer and Li 2002b; Moreno et al. 2014; Moreno et al. 2012). Prior to first cracking, both concrete matrix

and ECC exhibited similar tension-stiffening behaviour in terms of load-deformation response, as shown in **Fig. 2.29**. At post-cracking stage, the multi-cracking behaviour of ECC allowed compatible deformations between ECC and reinforcement, and tensile stress could be transferred across the crack by bridging fibres. Thus, the contribution of ECC to the total load could be maintained after yielding of steel reinforcement (Fischer and Li 2002b).

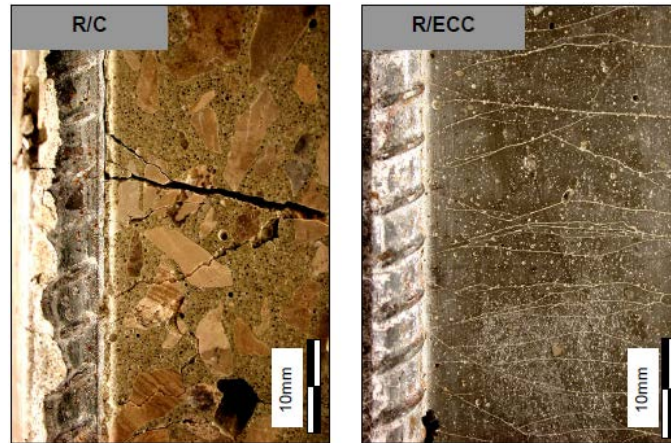


Fig. 2.30: Interface condition in reinforced concrete and ECC (Fischer and Li 2002b)

Fig. 2.30 shows the interface condition between reinforcement, concrete and ECC. Other than inclined cracking in concrete matrix, multiple cracks surrounding the embedded reinforcement prevented debonding between reinforcement and ECC at the post-yield stage of reinforcement (Li 2003), which provided significant tension-stiffening behaviour to reinforced ECC in uniaxial tension. However, beyond the multi-cracking stage of ECC, a substantial reduction in the average strain of reinforced ECC was obtained as compared with bare steel bar and reinforced concrete specimens (Moreno et al. 2014), as shown in **Fig. 2.31**. Multi-cracking and strain-hardening behaviour of ECC led to localisation of a major crack, as shown in **Fig. 2.32**. It considerably reduced the ductility of the reinforced ECC specimen. The tension-stiffening behaviour of reinforced ECC specimens possibly resulted from higher bond stress between ECC and steel reinforcement.

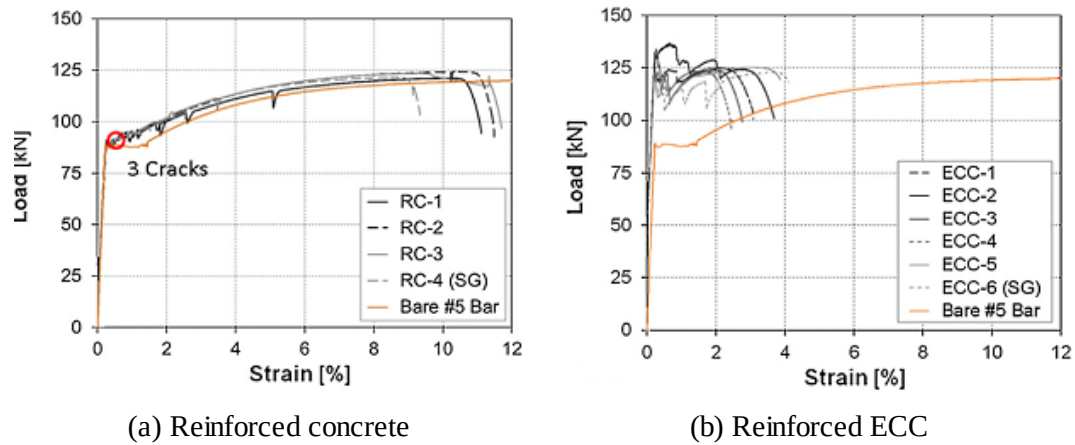


Fig. 2.31: Total load in specimens versus average strain (Moreno et al. 2014)



(a) Reinforced concrete member



b) Reinforced ECC member

Fig. 2.32: Cracks in specimens prior to fracture of reinforcement (Moreno et al. 2014)

2.4.4 Bond stress of reinforcement embedded in ECC

Bandelt and Billington (2014) tested a series of beam specimens under four-point bending to investigate the bond-slip behaviour of steel reinforcement embedded in conventional concrete, ECC, self-consolidating high performance fibre reinforced concrete (SC-HPFRC) and self-consolidating hybrid fibre reinforced concrete (SC-HyFRC). All beams exhibited splitting bond cracks due to insufficient concrete cover for steel reinforcement. Comparisons of normalised bond stress-slip curves indicate that when the same concrete cover (equal to the diameter of beam longitudinal bars) and no stirrup were used in beams, reinforcement embedded in ECC developed the

greatest bond stress among the four concrete materials, as shown in **Fig. 2.33(a)**. The maximum value was around 39% higher as compared to that in concrete. Additional confinement provided by stirrups exhibited little effect on the bond stress of reinforcement in ECC (see **Fig. 2.33(b)**), whereas bond strength of steel bars in concrete, SC-HPFRC and SC-HyFRC was increased. It indicates that ECC could provide better confinement condition than conventional concrete, SC-HPFRC and SC-HyFRC with the given cover thickness. Besides, bond-splitting strength of reinforced ECC elements was also studied through pull-out bond tests on embedded reinforcement with various cover thickness (Asano and Kanakubo 2012; Kunakubu and Hosoya 2015). Empirical equations were also derived for predicting the bond-splitting strength of ECC.

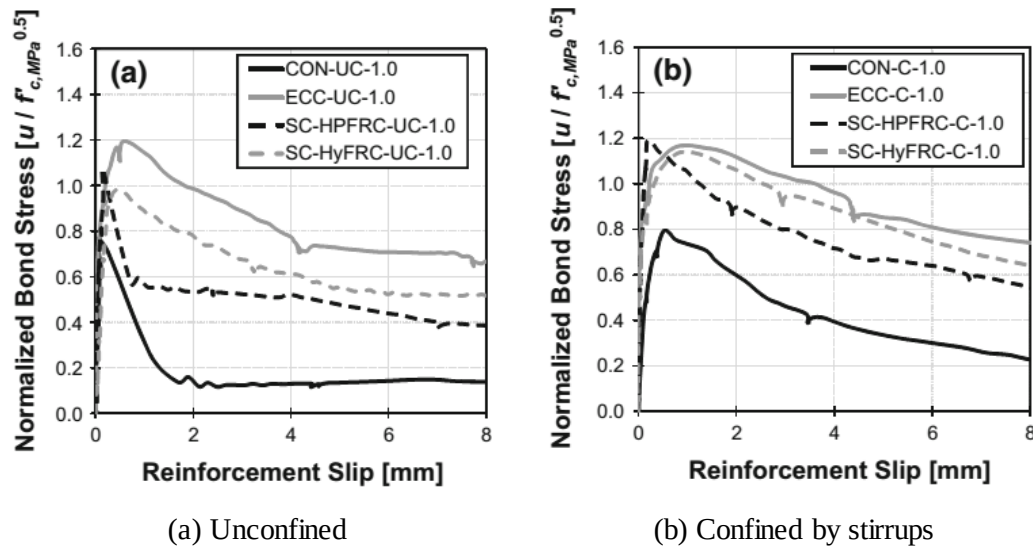


Fig. 2.33: Bond stress-reinforcement slip response (Bandelt and Billington 2014)

Compatible deformations and greater bond strength between ECC and steel reinforcement are likely to reduce the required embedment length of steel bars in ECC, thereby facilitating the design and construction of precast concrete beam-column joints. Moreover, tensile strength of ECC can also be taken into consideration (JSCE 2008), which could increase the flexural resistance of reinforced ECC members. When applied to progressive collapse design, localised cracks at large deformation stage reduce the ductility of reinforced ECC members, as reported by Bandelt and Billington (2014). Therefore, further experimental tests are necessary to investigate the behaviour of ECC sub-assemblages under column removal scenarios.

2.5 Component-Based Joint Models

The concept of component method is to idealise the force transfer at the perimeter of a typical joint as a series of basic components to explicitly represent the joint behaviour (Zoetemeijer 1983). It has been incorporated in BS EN 1993-1-8: 2005 (BSI 2005) and recommended for the design of steel and composite beam-column joints. Nowadays, a unified characterisation procedure for structural joints has been developed (Jaspart 2000). Application of component-based model to reinforced concrete structural joints under cyclic loading conditions has also been proposed (Lowes and Altoontash 2003; Mitra and Lowes 2007). These research works form the common basis for future seismic design codes. However, for beam-column joints under progressive collapse scenarios, the transformation of resisting mechanisms at different stages necessitates special considerations of component characterisation.

2.5.1 Procedure of joint characterisation

In the characterisation process, a joint is considered as a set of individual components, including relevant components in the compression zone, tension zone, interface and joint panel. Each of these components possesses its own constitutive model, and various combinations of these components allow a wide range of joint configurations to be incorporated in the joint model. Although coexistence of different components is likely to affect the strength and stiffness of each fundamental component (Guisse and Jaspart 1995), the principle of component method is still valid for beam-column joints (Jaspart 2000).

Application of component method to the beam-column joint requires the following steps (Jaspart 2000): 1) identification of active components for the joint; 2) evaluation of the response of each basic component; 3) assembly of the components to assess the mechanical properties of the whole joint. All the three steps require a sufficient knowledge on distribution and transfer of internal forces within the joint. Generality of the framework of component method allows for adoption of various techniques of component characterisation and joint assemblies. The stiffness and strength characteristics of the components can be obtained from component tests in laboratory,

numerical simulations by means of finite element programmes, and analytical models (Jaspart 2000).

2.5.2 Joint models under cyclic loading

Youssef and Ghobarah (2001) developed a joint model to consider bond slip of embedded reinforcement or shear failure in the beam-column joint when subjected to earthquake loading conditions. The joint zone is modelled by four pinned rigid members, with shear springs connecting the diagonals, as shown in **Fig. 2.34(a)**. At the joint interface, three concrete springs and three steel springs are used, by means of which bond slip of reinforcement and crushing of concrete can be considered (Youssef and Ghobarah 1999). Beam and column elements are modelled by elastic elements.

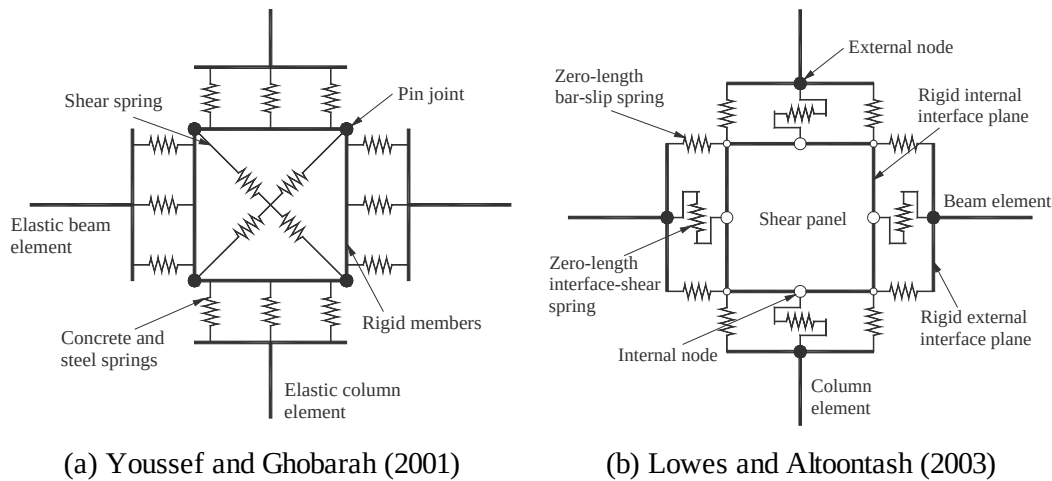


Fig. 2.34: Reinforced concrete beam-column joint models under cyclic loads

To improve the general applicability of joint modelling, Lowes and Altoontash (2003) formulated a four-node, 12 degrees-of-freedom beam-column joint model, as shown in **Fig. 2.34(b)**. This model incorporates one shear-panel component that allows for the shear failure of joint core, eight bar-slip components that simulate the bond strength deterioration for beam and column longitudinal reinforcement, and four interface-shear components that consider the loss of shear-transfer capacity at the beam-joint and column-joint interfaces. In calibrating the load-deformation response, the modified compression field theory proposed by Vecchio and Collins (1986) was used to define the response of the shear panel. To derive the constitutive model for

tensile springs at the joint interface, it was assumed that the embedment length of reinforcement is adequate in the joint. Pull-out force is applied at one end of reinforcement, whereas strain and slip are zero at the other end. A piecewise constant bond stress distribution was assumed along the embedment length of steel reinforcement anchored in the joint, as shown in **Fig. 2.35**. Accordingly, the force-slip relationship of steel reinforcement in tension could be determined. For compressive springs, plane-section assumption was utilised to quantify the total compression force sustained by the spring, whereas corresponding slip could be calculated from the bond-slip model in **Fig. 2.35**. An elastic response was postulated for interface-shear components due to a lack of test data in defining their constitutive relationships. A further study by Mitra and Lowes (2007) recommended ways to improve the accuracy of joint behaviour prediction and to eliminate numerical instability problems: 1) bar-slip springs are located at the centroid of beam and column flexural tension and compression zone; 2) a diagonal compressive strut mechanism is assumed in the joint-panel component; 3) a new bond-slip model is proposed to simulate the frictional resistance for bars in tension and compression. However, the models are not appropriate for modelling of beam-column joints with no transverse reinforcement (Sharma et al. 2011).

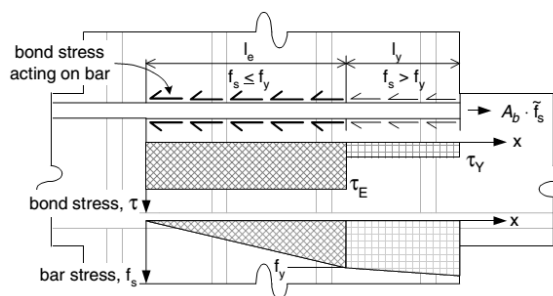


Fig. 2.35: Bond and bar stress distribution along a reinforcing bar embedded under pull-out force (Lowes et al. 2004)

2.5.3 Joint models under progressive collapse

Unlike seismic loading conditions, axial force develops in the bridging beam when subject to column removal (Lew et al. 2014; Yu and Tan 2014), of which the magnitude depends on the stiffness of horizontal restraints to the beam. Thus, based on the joint model formulated by Lowes and Altoontash (2003), Bao et al. (2008) and

Yu and Tan (2010b) constructed their macro-models for reinforced concrete beam-column joints under column removal scenarios, as shown in **Figs. 2.36(a and b)**. In these models, a series of nonlinear springs is assembled at the joint interface, through which the force transfer between structural members and beam-column joints is fulfilled. These springs comprise a shear spring, a compression spring and a tension spring. The difference between the two models lies in the formulation of the joint panel. In Bao's model (Bao et al. 2008), the joint panel is constructed by four rigid elements connected by four pin nodes and two rotational springs are used at the pin nodes to represent the shear distortion of the joint core. Yu and Tan (2010b) assumed a rigid joint panel in the analysis of reinforced concrete beam-column sub-assemblages under column removal scenarios. In the later study, the joint core was represented by four pinned rigid members, with two diagonal springs in the joint panel (Yu 2012). However, under progressive collapse scenarios, the joint panel is less significant in the behaviour of sub-assemblages due to limited shear force in the middle joint.

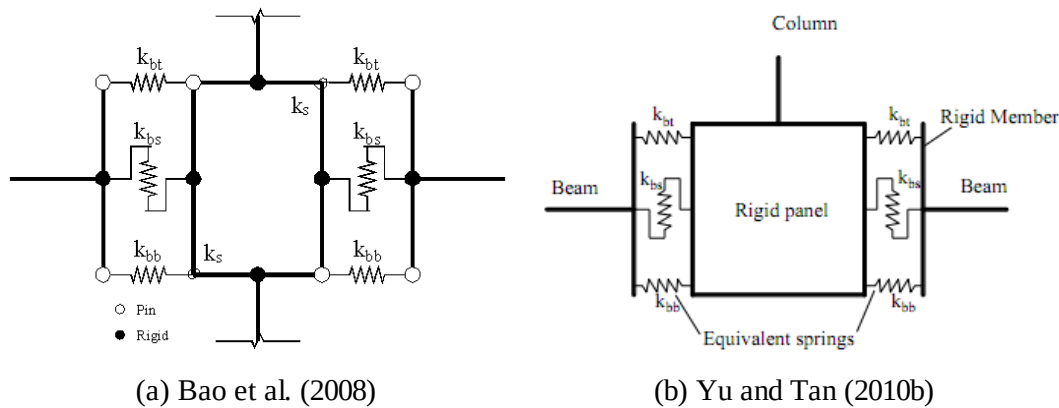


Fig. 2.36: Beam-column joint models under column removal scenarios

Under middle column removal scenarios, bottom reinforcement in the middle joint is subjected to axial tension at two ends. Thus, the bond-slip model proposed by Lowes et al. (2004) is not suitable if the reinforcement passing through the middle joint is insufficiently long to ensure zero strain in the middle of its embedment length. Instead, a bond-slip model for embedded reinforcement under axial tension was derived by Yu and Tan (2010b), as shown in **Fig. 2.37**. In the mode, non-zero tensile strain in the middle of the embedment length of reinforcement was considered and a stepwise bond stress profile was still employed to derive the force-slip relationship of steel

bars. Similar steel strain and bond stress profiles were also assumed by Bao et al. (2014) for reinforced concrete frames subject to column removal scenarios. In deriving the force-slip relationship of compressive springs, equivalent concrete compressive stress block was used and additional compression force in the beam was considered to determine the neutral axis depth at the beam end (Yu 2012). Total force in the compression zone was correlated to the slip of compressive reinforcement so as to obtain the force-slip relationship of compressive springs.

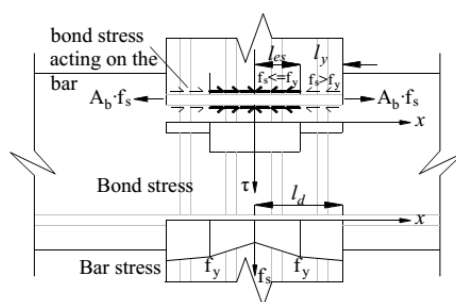


Fig. 2.37: Bond and bar stress distribution along a reinforcing bar under axial tension (Yu and Tan 2010b)

However, in those models, bond stressed at elastic and post-yield stages of steel reinforcement are quite different from one another and need to be re-evaluated by test data. Besides, pull-out failure of reinforcement, which is common for reinforcement with insufficient embedment length in the joint (Orton et al. 2009), has not been incorporated in the component-based models. For the properties of compressive spring, equivalent concrete stress block is not valid at large deformation scenarios as a result of crushing of concrete in compression. Instead, constitutive model for concrete can be more reasonable for calculating the compression force sustained by concrete. Therefore, further improvements are necessary for the component-based joint model under column removal scenarios.

2.6 Summary

In recent years, the increasing risk of terrorist attack necessitates experimental and analytical studies on the progressive collapse resistance of building structures. Meanwhile, design approaches against progressive collapse have been proposed and incorporated in the design guidelines, such as UFC 4-023-03 (DOD 2013). In the

design of building structures against progressive collapse, special focus is placed on the ability to develop alternate load paths under column removal scenarios, which requires beam-column joints to exhibit adequate ductility and robustness when local damage occurs. Experimental tests on reinforced concrete structures at different levels and on static and dynamic loading conditions indicate that CAA and catenary action develop sequentially under middle column removal to resist progressive collapse. However, when it comes to precast concrete structures subject to column removal scenarios, there are very limited results on the resistance and deformation capacity of precast beam-column joints (Main et al. 2014). Therefore, experimental tests on precast concrete beam-column sub-assemblages and frames are necessary in order to assess the resistance, deformation capacity and failure mode under different boundary conditions.

Although utilisation of ECC in structural members is suggested in the design recommendations (JSCE 2008), behaviour of beam-column sub-assemblages with ECC joints and structural topping has not yet been explored under column removal scenarios. Similar to fibre-reinforced concrete (FIB 2013), ductility of ECC under compression is improved by the bridging fibres. Besides, tensile strength of ECC can be considered at strain-hardening stage. Thus, the CAA capacity of beam-column sub-assemblages is expected to be enhanced if ECC is utilised in the beam in place of conventional concrete. However, the behaviour of beam-column sub-assemblages cast with ECC topping remains unknown at large deformations. Therefore, the applicability of ECC with strain-hardening behaviour and superior strain capacity in tension to progressive collapse scenarios needs to be explored through experimental tests.

Finally, based on experimental results, component-based joint models need to be proposed for precast concrete beam-column joints and calibrated to facilitate structural analysis under column removal scenarios. In the models, different failure modes of steel reinforcement embedded in beam-column joints have to be incorporated. Besides, crushing of concrete at large deformation scenarios should also be considered in deriving the force-slip relationship of compressive springs.

CHAPTER 3 EXPERIMENTAL TESTS OF PRECAST CONCRETE BEAM-COLUMN SUB-ASSEMBLAGES

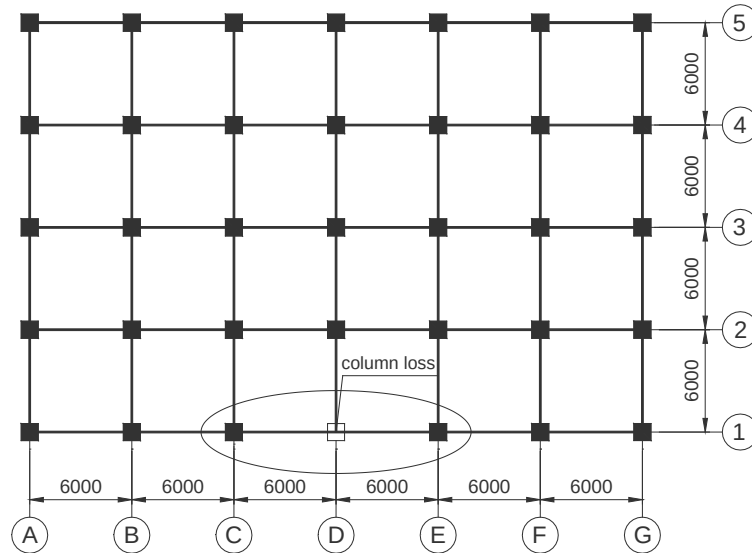
3.1 Introduction

In precast concrete structures, precast beam units with cast-in-situ structural topping and beam-column joints have been used as ductile moment-resisting frames, as reported by FIB bulletins (FIB 2002; FIB 2003) and other documents (CAE 1999; Shiohara and Watanabe 2000; Van Acker 2013). Its design is compatible with design codes for monolithic reinforced concrete structures, but minor modifications are made in the reinforcement detailing to achieve high productivity (Shiohara and Watanabe 2000). With proper reinforcement detailing in beam-column joints, frames can exhibit equivalent behaviour to monolithic reinforced concrete structures under flexure condition (CAE 1999). Detailing practice of structural joints in precast concrete structures have also been recommended under progressive collapse scenarios, which requires utilisation of continuous beam top longitudinal reinforcement passing through the joint (Van Acker 2013). The pertinent question is whether this type of precast beam-column sub-assemblages can exhibit catenary action under column removal scenarios. Thus, experimental studies are necessary to evaluate the resistance and deformation capacity of precast concrete beam-column sub-assemblages under column removal scenarios.

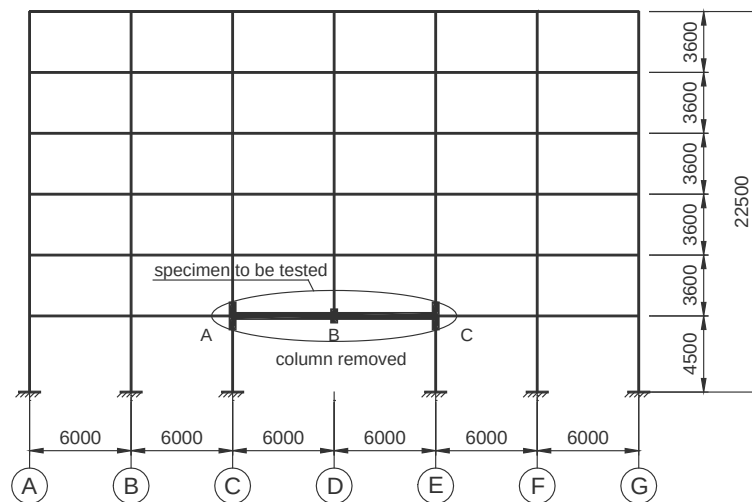
This chapter describes the behaviour of precast concrete beam-column sub-assemblages under middle column removal scenarios. The effects of reinforcement detailing in the joint and longitudinal reinforcement ratio in the beam on compressive arch action (CAA) and catenary action were investigated, and recommendations were made for the design of precast concrete structures against progressive collapse. These findings are relevant to any form of precast concrete construction that does not require special embedded metal inserts or mechanical couplers for the bottom reinforcement in the joint region.

3.2 Test Programme

3.2.1 Prototype structure



(a) Plan view



(b) Elevation view

Fig. 3.1: The prototype precast concrete structure

A six-storey precast concrete frame building was designed under gravity loads in accordance with Eurocode 2 (BSI 2004). **Figs. 3.1(a and b)** show the plan and elevation views of the structure. The height of a typical storey was 3.6 m, except the 4.5 m high first floor. The centre-to-centre spacing of columns in two orthogonal directions was 6 m. The cross sections of a prototype beam and a column were 300 mm by 600 mm and 500 mm by 500 mm, respectively. Under column removal

scenarios, one middle column at the ground floor of the peripheral frame was assumed to be forcibly removed, as shown in **Fig. 3.1(b)**. A beam-column sub-assembly, incorporating the two-span beam and the middle column over the column removal, was extracted from the precast concrete frame.

To fit the extracted sub-assembly within the physical constraints of the Protective Engineering laboratory in Nanyang Technological University, the beams and columns in the prototype building were scaled down to one-half, but the beam reinforcement ratio remained unchanged. Thus, the spacing of precast columns in two orthogonal directions was 3 m, and the dimensions of beams and columns were scaled down to 150 mm by 300 mm and 250 mm square, respectively. Two enlarged column stubs were erected on both sides of the sub-assembly to simulate horizontal restraints from adjacent columns.

3.2.2 Specimen design

Welded connections in precast concrete structures exhibit limited capacity to develop alternate load paths due to the reduced ductility of steel reinforcement in the heat-affected zone (Main et al. 2014). Cast-in-situ concrete provides better robustness for precast structural elements, as reported by FIB bulletin 19 (FIB 2002). Therefore, to enhance the structural performance of precast concrete structures under column removal scenarios, precast beam units mixed with cast-in-situ concrete topping and beam-column joint were used for the ductile moment resisting frame in the experimental programme. Two stages of casting were adopted in fabricating the beam-column sub-assemblies. Firstly, the 225 mm deep precast beam unit, as depicted by the hatched zones in the beam-column sub-assemblies in **Fig. 3.2**, was fabricated. Thereafter, the two beam units were assembled and continuous top reinforcement was placed inside the projecting stirrups. Finally, 75 mm deep concrete topping, the middle beam-column joint and the end column stubs were cast to form an integral sub-assembly. It is noteworthy that cast-in-situ middle column and end column stubs were used in the sub-assemblies, as reinforcement details in the column do not significantly affect the development of alternate load paths in the bridging beam when adequate horizontal restraints are provided by adjacent structural members.

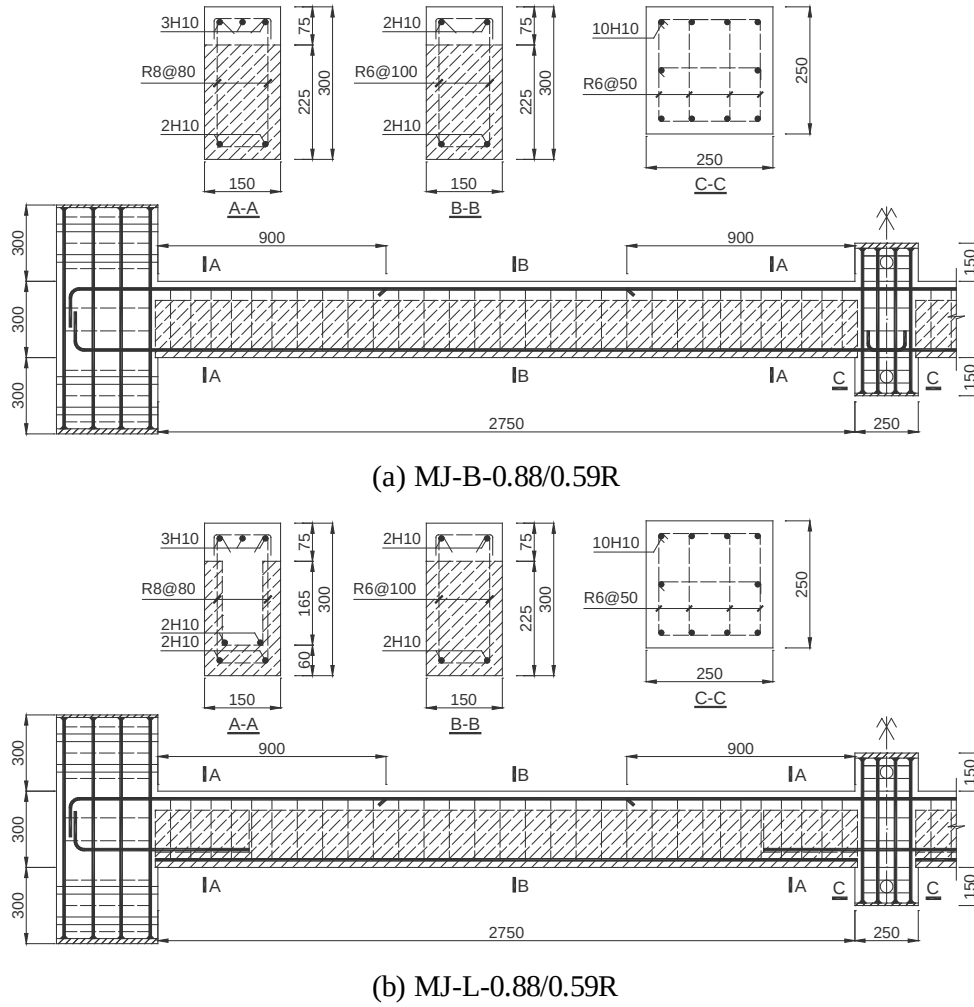


Fig. 3.2: Reinforcement detailing of precast concrete beam-column sub-assemblages

A total number of six precast concrete beam-column sub-assemblages were tested under column removal scenarios. **Table 3.1** lists the three investigated parameters that dominate the behaviour of beam-column sub-assemblages, namely, bottom bar detailing in the middle joint, top and bottom longitudinal reinforcement ratios in the beam and surface preparation of horizontal interface for precast beam units. In the notations of specimens, “MJ” denotes beam-column sub-assemblages incorporating a middle joint and a two-span beam, and “B” and “L” stand for 90° bend and lap-splice of bottom bars in the middle joint, respectively. The first and second numerals denote the respective percentages of top and bottom longitudinal reinforcement in the middle joint. “S” and “R” indicate smooth and rough horizontal surfaces of precast beam units, respectively. For instance, MJ-B-0.88/0.59R represents a specimen with

bottom reinforcement of 90° bend in the middle joint, a top reinforcement ratio of 0.88%, a bottom reinforcement ratio of 0.59%, and a rough horizontal concrete surface for the two precast beam units.

Table 3.1: Geometric property of beam-column sub-assemblages

Specimen	Clear span (m)	Length of curtailed top bar (mm)	Bottom bars at middle joints and length ^{&} (mm)	Longitudinal reinforcement				Surface treatment
				A-A section		B-B section		
				Top	Bottom	Top	Bottom	
MJ-B-0.52/0.35S	2.75	900	90° bend (190+70)	3H10	2H10	2H10	2H10	Smooth
MJ-L-0.52/0.35S		900	Lap-spliced (360)	3H10	2H10	2H10	2H10	Smooth
MJ-B-0.88/0.59R		1000	90° bend (190+90)	3H13	2H13	2H13	2H13	Rough
MJ-L-0.88/0.59R		1000	Lap-spliced (470)	3H13	2H13	2H13	2H13	Rough
MJ-B-1.19/0.59R		1000	90° bend (190+90)	2H16+H13	2H13	2H16	2H13	Rough
MJ-L-1.19/0.59R		1000	Lap-spliced (470)	2H16+H13	2H13	2H16	2H13	Rough

[&]: The anchorage length is calculated from the face of the middle column, and the length (190+90) denotes the horizontal portion of the 90° bend bar is 190 mm and the vertical portion is 90 mm.

For discontinuous bottom reinforcing bars in precast concrete beams, two widely-used joint detailing in local practice were studied. The first joint detailing features protruded bottom longitudinal bars terminating with a 90° bend in the joint region (see **Fig. 3.2(a)**). This type of joint detailing performs well even under earthquake loading conditions and has been recommended for construction in New Zealand (CAE 1999). However, this detailing may lead to congestion of reinforcement in the joint region, and the column size should be sufficiently wide to accommodate the required embedment length (FIB 2003). The second joint detailing is characterised by U-shaped beam trough sections at the beam ends connecting to the joint (**Fig. 3.2(b)**). Unlike the precast concrete beam shells tested by Park and Bull (1986), the U-shaped trough section was only located at the two ends of the precast beams and its length depends on the required embedment length of bottom reinforcement passing through the middle joint. In-situ concrete was cast in the trough, the joint region and the structural topping consisting of continuous top reinforcement. Composite action between precast beam units and concrete topping relies on the roughness of horizontal interface, the amount of protruded stirrups and the concrete strength (Patnaik 2000). In the study, across the horizontal interface between the precast beam units and cast-

in-situ concrete topping, two types of surface preparation were employed to examine the effectiveness of horizontal shear transfer at large deformation stage. “Smooth surface” refers to one without any treatment after vibration, whereas “rough surface” represents an interface that is intentionally roughened to approximately 3 mm roughness complying with Eurocode 2 (BSI 2004). In all specimens, mild steel stirrups of 8 mm diameter at 80 mm spacing were placed at the beam end sections, whereas stirrups of 6 mm diameter at 100 mm spacing were used at the middle sections.

3.2.3 Test setup

The boundary conditions of beam-column sub-assemblages in the frame structure were simplified as two horizontal restraints and one vertical restraint on the column stub at each end, as shown in **Fig. 3.3**. Load cells were used in the horizontal direction to record reaction forces, as shown in **Fig. 3.4(a)**. At the bottom of each column stub, a pin support was seated on steel rollers (see **Fig. 3.4(b)**), and load cells were placed under the rollers to measure the vertical reaction force. The fairly rigid horizontal restraints from the A-frame and the reaction wall provided the upper bound values of structural resistances of the beam-column sub-assemblage under column removal scenarios.

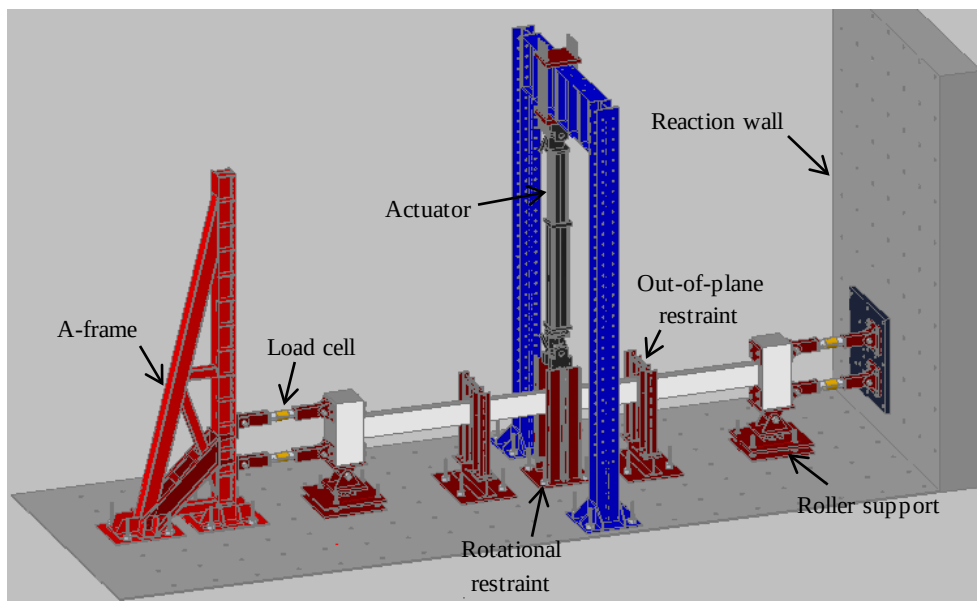


Fig. 3.3: Test setup for beam-column sub-assemblages

In addition to the boundary conditions at the end column stubs, two pairs of universal steel columns with steel rollers were erected on each side of precast beams to prevent out-of-plane bending of the beams as load was applied onto the middle joint, as shown in **Fig. 3.4(c)**. This was to simulate the full restraint from the slab, so that the sub-assembly could only deflect vertically. In the vicinity of the middle joint, two sets of short columns were employed in front of and behind the sub-assembly. Steel rods were placed in the PVC pipes embedded in the middle column, as shown in **Fig. 3.4(d)**. Hence, rotation of the middle joint could be prevented if reinforcing bars only fractured at one vertical face of the joint. A displacement-controlled point load at a rate of 6 mm/min was applied vertically on the middle column through a servo-hydraulic actuator.

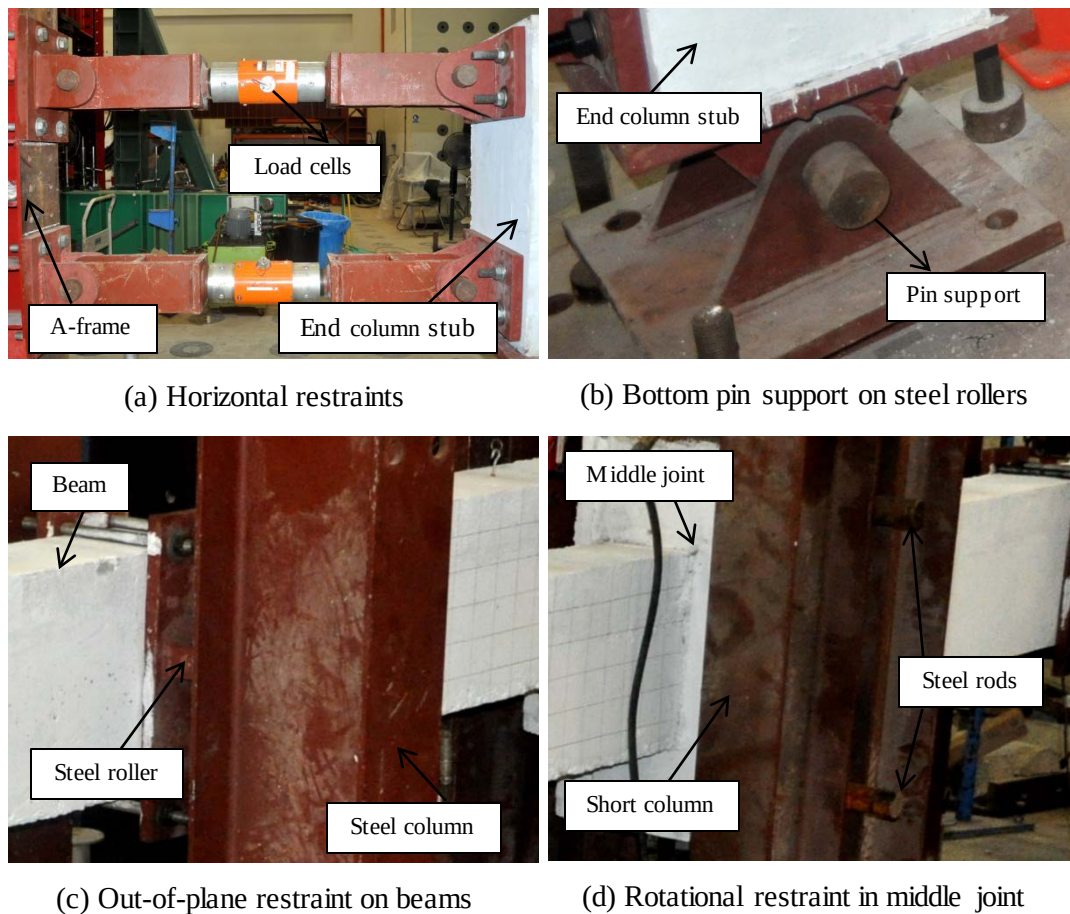


Fig. 3.4: Restraints on beam-column sub-assemblages

3.2.4 Instrumentations

Besides the reaction forces measured by the horizontal and vertical load cells, the deformed geometry of the beam-column sub-assemblages was monitored through linear variable differential transducers (LVDTs) placed along the beam length at regular intervals. Additionally, plastic hinges at the beam ends played a crucial role in the deformation capacity of sub-assemblages, and they were measured by a group of LVDTs mounted onto the beam. **Fig. 3.5** shows the LVDT arrangement to measure the vertical deflections of the beam and the rotations of plastic hinges at the beam ends. The plastic hinge length for a typical beam was taken as $0.5h$, where h is the full depth of the beam cross section (Paulay and Priestley 1992). Thus, the first row of LVDTs measured the plastic hinge rotations over a length of 150 mm. Since the development of catenary action could extend the plastic hinge length, another row of LVDTs was installed at 120 mm away from the first row of LVDTs at both beam ends. As CAA is sensitive to any connection gaps between sub-assemblages and restraints (Yu 2012), horizontal movements of end column stub were monitored through LVDTs LS-1 and LS-2 at the horizontal restraints to account for the effect of connection gaps on the behaviour of sub-assemblages.

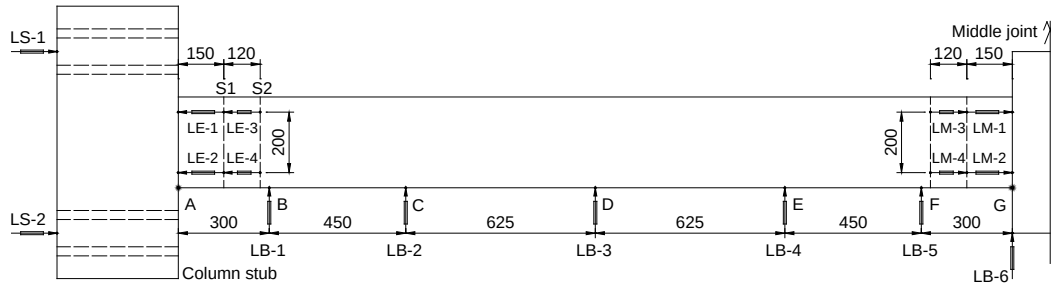


Fig. 3.5: Schematic of hinge rotation and beam deformation measurement

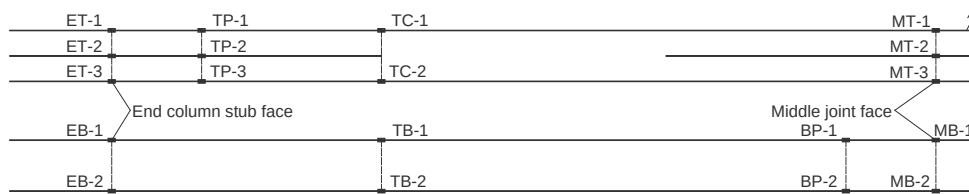


Fig. 3.6: Layout of strain gauges on longitudinal reinforcement

Strain gauges were mounted onto the beam longitudinal reinforcement at the interfaces with the middle joint and end column stub to measure steel strains at

different loading stages. Besides, strains of longitudinal reinforcement were also traced at a section 300 mm away from the faces of the middle joint and end column stub and at the curtailment point of beam top reinforcement. **Fig. 3.6** shows the layout of steel strain gauges on the top and bottom longitudinal bars in the beam. It should be noted that for beam-column sub-assemblages with lap-spliced bottom bars, strain gauges were mounted on the bottom bars passing through the joint.

3.3 Material Properties

Hot-rolled deformed steel bars with diameters of 10, 13 and 16 mm were used for longitudinal reinforcement in the beam, and round bars with 6 and 8 mm diameters were used for stirrups. Concrete with the maximum coarse aggregate size of 10 mm was mixed for the precast beam units, the cast-in-situ concrete topping and the joint itself. Prior to testing, material properties of reinforcement and concrete were obtained, as listed in **Table 3.2** and **Table 3.3**. **Fig. 3.7** shows the typical stress-strain curves of concrete and steel reinforcement.

Table 3.2: Material properties of reinforcing bars

Material		Diameter (mm)	Yield Strength (MPa)	Modulus of elasticity (GPa)	Ultimate strength (MPa)	Fracture strain* (%)
Longitudinal bars	H10	10	462	187.3	553	11.9
	H13	13	471	186.5	568	12.2
	H16	16	527	196.3	618	11.9
Stirrups	R6	6	264	217.9	351	7.9
	R8	8	353	209.6	460	14.9

*: Fracture strain refers to the average strain over a gauge length of 300 mm when steel reinforcement ruptures.

Table 3.3: Compressive strength of concrete

Concrete	Compressive strength (MPa)		Secant modulus (GPa)
Precast beam unit	27.9		24.7
Concrete topping and beam-column joint	MJ-B-0.52/0.35S	35.8	27.8
	MJ-L-0.52/0.35S		
	MJ-B-0.88/0.59R	20.3	20.5
	MJ-L-0.88/0.59R		
	MJ-B-1.19/0.59R		
	MJ-L-1.19/0.59R		

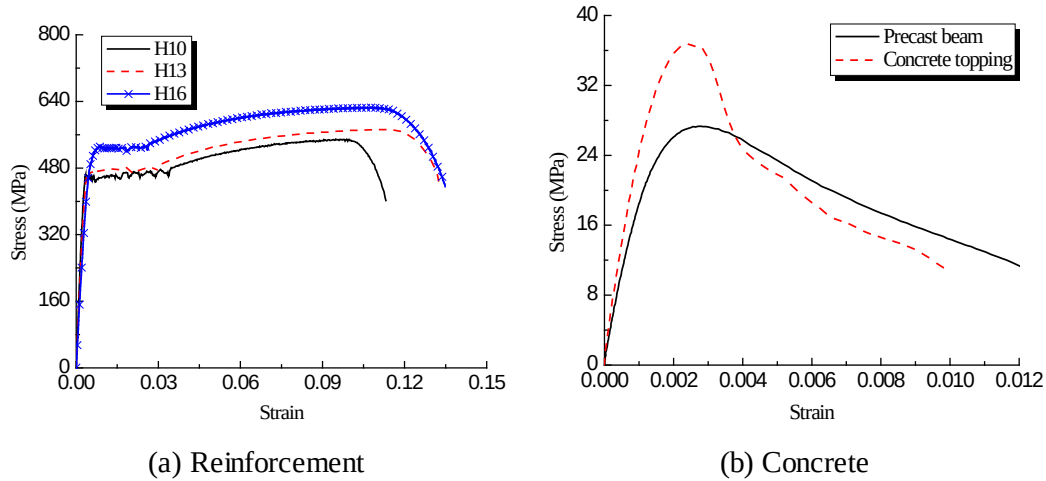


Fig. 3.7: Stress-strain curves of reinforcement and concrete

3.4 Experimental Results of Sub-Assemblages

In the experimental tests, the resistance and deformation capacity of precast concrete beam-column sub-assemblages were obtained under middle column removal scenarios. In addition, crack patterns of the bridging beams and failure modes of the beam-column sub-assemblages were observed. Strains of beam longitudinal reinforcement were also measured by means of strain gauges to shed light on the behaviour of sub-assemblages subjected to progressive collapse.

3.4.1 Load-displacement history of beam-column sub-assemblages

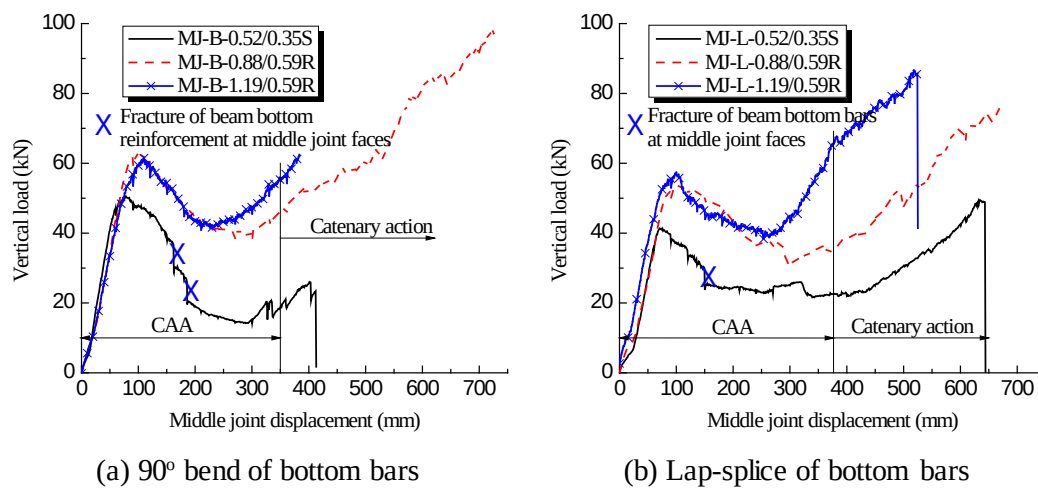


Fig. 3.8: Vertical load-middle joint displacement curves of beam-column sub-assemblages

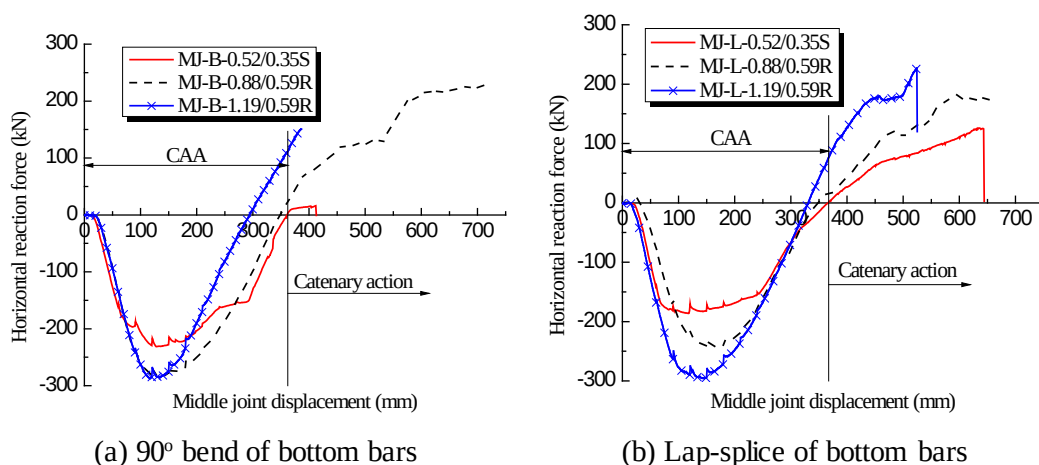


Fig. 3.9: Horizontal reaction-middle joint displacement curves of beam-column sub-assemblages

Fig. 3.8 and **Fig. 3.9** show the vertical load and horizontal reaction force of beam-column sub-assemblages versus the middle joint displacement. Two mechanisms, namely, CAA and catenary action, were sequentially developed in the beam-column sub-assemblages. CAA is termed as the force-transfer mechanism in which significant axial compression force develops in the bridging beam, whereas catenary action represents the stage when axial tension force is initiated in the beam (Su et al. 2009; Yu and Tan 2010a). At CAA stage, horizontal compression force increased with vertical load, but vertical load attained its peak value earlier than the maximum horizontal compression. Due to crushing of concrete in the compression zone at the top beam surface next to the middle joint and the bottom beam surface near the end column stub, both vertical load and horizontal compression force started decreasing with increasing middle joint displacement. In the descending branch, a sudden drop of vertical load marked by crosses as shown in **Figs. 3.8(a and b)** resulted from fracture of bottom reinforcing bars at the middle joint interfaces. However, the fracture of bottom bars imposed a minor effect on the horizontal compression force. When the displacement was larger than one beam depth of 300 mm, catenary action kicked in and the applied load was sustained by the tensile strength of the beam until final failure occurred. Significant catenary action developed in beam-column sub-assemblages except MJ-B-0.52/0.35S, as shown in **Fig. 3.8(a)**. The catenary action capacity surpassed the value of CAA, and therefore catenary action was effective to mitigate progressive collapse under column removal scenarios. It should be noted that

sub-assembly MJ-B-1.19/0.59R sustained much smaller maximum vertical load and horizontal tension force at catenary action stage in comparison with MJ-L-1.19/0.59R, as shown in **Fig. 3.8(a)** and **Fig. 3.9(a)**, due to accidental breakdown of the controlling system. Its catenary action capacity was not attained at that moment.

3.4.2 Resistances of beam-column sub-assemblages

The resistance of beam-column sub-assemblages under column removal scenarios is characterised by the CAA and catenary action capacities. **Table 3.4** lists the load capacities and maximum horizontal compression and tension forces at different loading stages. It is assumed that all the bottom bars in the middle joint are able to develop their yield strength under flexural action, and the flexural capacity of sub-assemblages is calculated based on the plastic hinge mechanism. Accordingly, the enhancement factors of CAA and catenary action to flexural action are calculated, as shown in **Table 3.4**.

Table 3.4: Test results of beam-column sub-assemblages

Specimen	Flexural capacity P_f (kN)	CAA capacity P_c (kN)	Peak horizontal compression (kN)	P_c/P_f	Catenary action capacity P_t (kN)	Peak tension (kN)	P_t/P_f
MJ-B-0.52/0.35S	33.89	50.52	-231.26	1.49	26.05	16.22	0.77
MJ-L-0.52/0.35S	31.23	41.36	-186.10	1.32	49.50	127.74	1.59
MJ-B-0.88/0.59R	55.00	63.28	-282.40	1.15	98.55	229.85	1.79
MJ-L-0.88/0.59R	51.29	53.85	-242.00	1.05	77.24	182.05	1.51
MJ-B-1.19/0.59R	67.62	65.23	-287.25	0.96	--	--	--
MJ-L-1.19/0.59R	61.71	57.37	-290.30	0.93	86.60	227.10	1.40

3.4.2.1 Effect of reinforcement detailing

Although the same reinforcement ratio was used in MJ-B-0.52/0.35S and MJ-L-0.52/0.35S, significantly different CAA and catenary action capacities were obtained for those two specimens, as shown in **Table 3.4**. The CAA capacity of MJ-B-0.52/0.35S was 50.5 kN, 22.1% greater than that of MJ-L-0.52/0.35S. Sub-assembly MJ-B-0.52/0.35S was able to develop 24.3% larger horizontal

compression force than MJ-L-0.52/0.35S at CAA stage. Similar results were obtained for the other four sub-assemblages. MJ-B-0.88/0.59R developed 17.5% larger CAA capacity and 16.7% greater horizontal compression force than MJ-L-0.88/0.59R. A comparison between MJ-B-1.19/0.59R and MJ-L-1.19/0.59R indicates that MJ-B-1.19/0.59R sustained 9.6% greater CAA capacity than MJ-L-1.19/0.59R, but its maximum horizontal compression force was slightly lower than MJ-L-1.19/0.59R. These differences between the CAA capacities were mainly due to reinforcement detailing of beam bottom longitudinal bars in the joint. In comparison with lap-spliced reinforcement, 90° bend of bottom longitudinal reinforcement in the joint provided a larger distance between the compression and tension reinforcement at the middle joint and end column stub faces. Thus, sub-assemblages with 90° bend of bottom bars developed larger moment capacities at the beam end sections than those with lap-spliced bottom reinforcement. Correspondingly, greater CAA capacity and horizontal compression force were attained in sub-assemblages with 90° bend of bottom bars.

At catenary action stage, MJ-L-0.52/0.35S developed 90.0% greater catenary action capacity than MJ-B-0.52/0.35S, indicating the beneficial effect of lap-spliced bottom reinforcement on development of catenary action. Nonetheless, in MJ-L-0.88/0.59R, pull-out failure of bottom beam reinforcement near the end column stub hindered the development of tension force in the beam, as shown in **Fig. 3.9(b)**. Hence, its catenary action capacity was 21.6% lower than that of MJ-B-0.88/0.59R. Therefore, due to different failure modes of sub-assemblages, no apparent effect of reinforcement detailing was observed on the catenary action capacity.

Reinforcement detailing in the beam-column joint also affected the enhancement factor of CAA to flexural action. **Table 3.4** indicates that development of CAA enhanced the flexural capacities of sub-assemblages with 90° bend of bottom longitudinal reinforcement in the joint more than those with lap-spliced bottom bars, due to a larger distance between the compression and tension reinforcement at the joint interface.

3.4.2.2 Effect of reinforcement ratio

By increasing the top and bottom reinforcement ratios in MJ-B-0.88/0.59R and MJ-L-0.88/0.59R, the sagging and hogging moments of beam end sections were substantially increased compared to sub-assemblages MJ-B-0.52/0.35S and MJ-L-0.52/0.35S. Therefore, both the CAA capacity and horizontal compression force were increased in sub-assemblages MJ-B-0.88/0.59R and MJ-L-0.88/0.59R, as shown in **Fig. 3.8** and **Fig. 3.9**. It should be noted that the compressive strength of cast-in-situ concrete topping in MJ-B-0.88/0.59R and MJ-L-0.88/0.59R was lower than that in MJ-B-0.52/0.35S and MJ-L-0.52/0.35S. If concrete topping with the same compressive strength as that in MJ-B-0.52/0.35S and MJ-L-0.52/0.35S were used in MJ-B-0.88/0.59R and MJ-L-0.88/0.59R, the CAA capacities would have been even greater than those obtained in the tests. A further increase in the beam top reinforcement ratio in MJ-B-1.19/0.59R and MJ-L-1.19/0.59R did not significantly increase the CAA capacities as a result of severe shear cracking across the horizontal interface. Although the CAA capacity of beam-column sub-assemblages was increased with increasing reinforcement ratios, as shown in **Figs. 3.8(a and b)**, the enhancement factor of CAA relative to flexural action was reduced, as listed in **Table 3.4**, since the flexural capacity of sub-assemblages was also substantially increased. Thus, the enhancement of CAA to flexural action was more effective when the beam longitudinal reinforcement ratio was relatively low.

Compared with MJ-B-0.52/0.35S and MJ-L-0.52/0.35S, catenary action capacities of MJ-B-0.88/0.59R and MJ-L-0.88/0.59R were increased with greater top and bottom reinforcement ratios in the beam, as shown in **Figs. 3.8(a and b)**. The increase in the catenary action capacities mainly came from the increased top reinforcement ratio in the bridging beam, as bottom longitudinal reinforcement in the middle joint had been pulled out prior to initiation of catenary action. Only the top reinforcement ratio in the beam of MJ-L-1.19/0.59R was increased in comparison with MJ-L-0.88/0.59R. Correspondingly, the catenary action capacity of MJ-L-1.19/0.59R was increased by 12% and the horizontal tension force was increased by 25%, as shown in **Fig. 3.9(b)**. It indicates that a greater top reinforcement ratio favoured the development of catenary action in beam-column sub-assemblages.

3.4.3 Components of vertical load

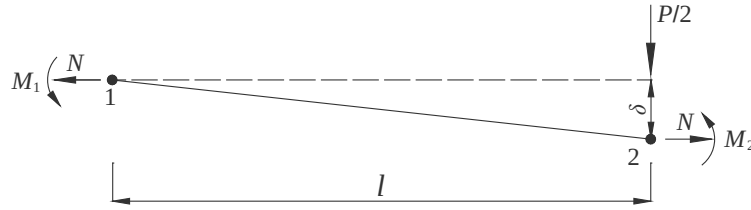
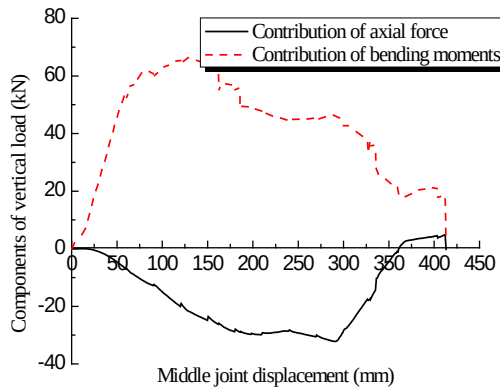


Fig. 3.10: Free body diagram of the single-span beam

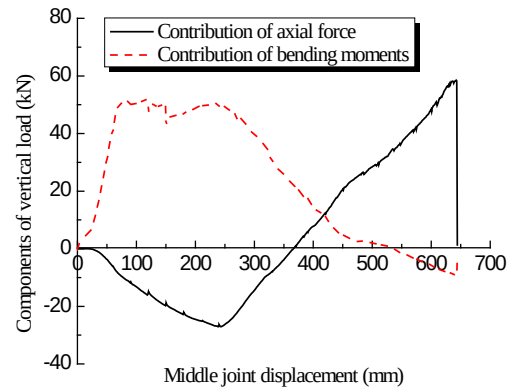
Under column removal scenarios, axial force develops in the bridging beam with horizontal restraints. Therefore, total vertical load on the middle joint can be decomposed into two components, viz. contributions of bending moments at the beam ends and axial force in the beam. Based on the deformation configuration and the force equilibrium of the single-span beam, as shown in **Fig. 3.10**, the components are calculated from Eq. (3-1), in which the first term represents the contribution of bending moments to the vertical load and the second term denotes the contribution of axial force in the beam.

$$\frac{P}{2} = \frac{M_1 + M_2 + N\delta}{l} = \frac{M_1 + M_2}{l} + \frac{N\delta}{l} \quad (3-1)$$

where P is the vertical load applied on the middle joint; and M_2 are the bending moments at the interface of the end column stub and middle joint, respectively; N is the axial force in the beam, equal to the horizontal reaction force; δ is the middle joint displacement; and l is the length of the single-span beam.



(a) MJ-B-0.52/0.35S



(b) MJ-L-0.52/0.35S

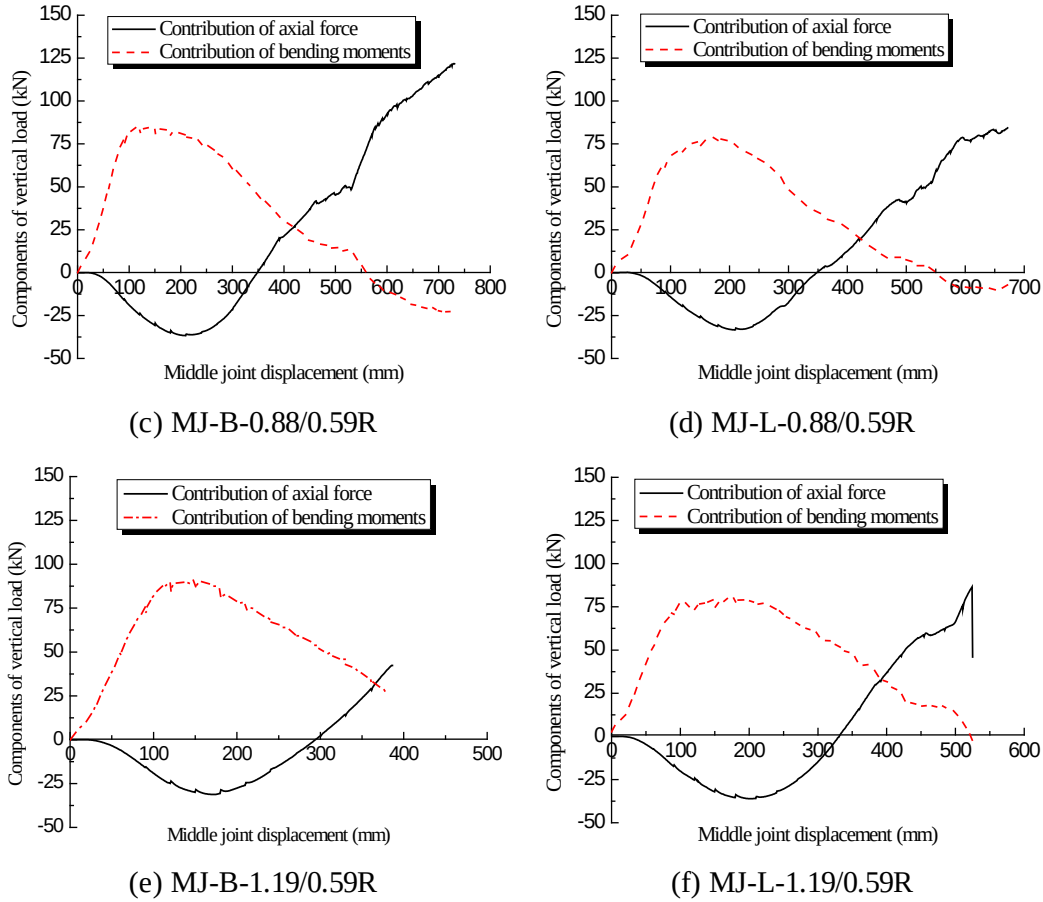


Fig. 3.11: Contributions of axial force and bending moments to vertical load of sub-assemblages

Fig. 3.11 shows the contributions of bending moments and axial force in the beam to the total vertical load. At CAA stage, vertical load was mainly contributed by moment resistances of the beam, whereas horizontal compression force contributed a negative portion to the vertical load, as shown in **Figs. 3.11(a-f)**. However, moment resistances of the beam were increased by axial compression force in the beam according to axial force-bending moment interaction diagram. Bending moments in the beam with 90° bend of beam bottom reinforcement provided more contributions to the vertical load than that with lap-spliced reinforcement in the joint, as included in **Table 3.5**, due to a relatively larger distance between the compression and tension reinforcement. Nevertheless, after the commencement of catenary action, horizontal tension force took up a major portion of the vertical load, whereas bending moment made limited contribution to the vertical load at the ultimate stage. Special attention has to be paid to sub-assemblage MJ-L-0.88/0.59R, in which the horizontal tension force decreased

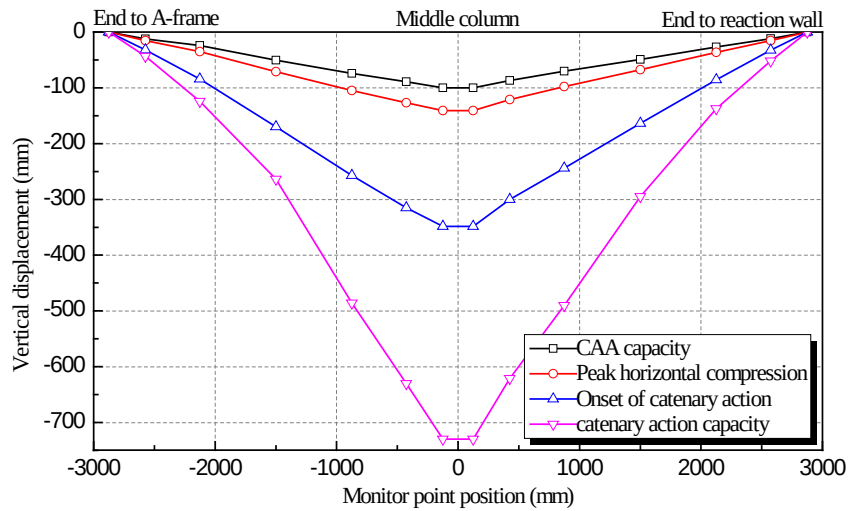
(see **Fig. 3.9(b)**) due to pull-out failure of beam bottom reinforcement near the end column stub prior to failure. However, the contribution of axial tension force in the beam could still increase with increasing middle joint displacement, as shown in **Fig. 3.11(d)**, as a result of a greater deformation capacity.

Table 3.5: Components of vertical load sustained by sub-assemblages

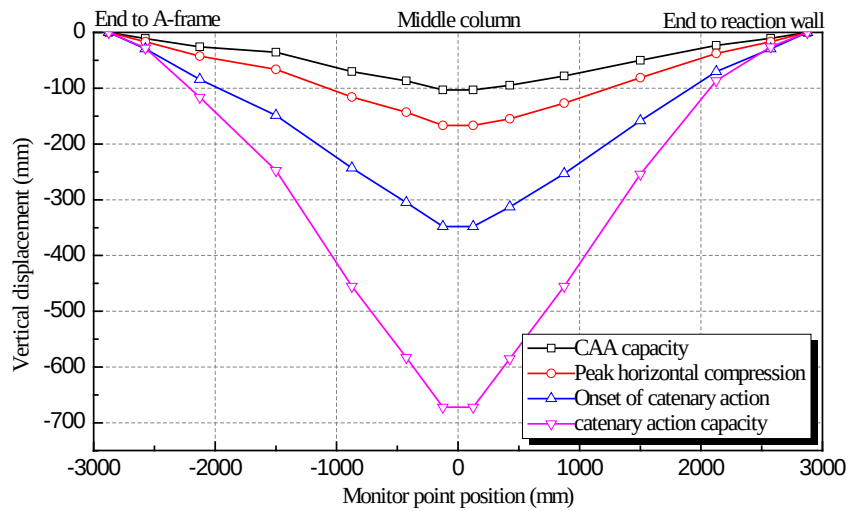
Specimen	Components at CAA stage		Components at catenary action stage	
	Min. contribution of Axial force (kN)	Max. contribution of bending moment (kN)	Max. contribution of Axial force (kN)	Min. contribution of bending moment (kN)
MJ-B-0.52/0.35S	-32.16	66.49	4.86	18.57
MJ-L-0.52/0.35S	-27.08	51.85	58.52	9.56
MJ-B-0.88/0.59R	-36.71	85.67	121.77	-24.16
MJ-L-0.88/0.59R	-33.38	78.91	84.34	-10.19
MJ-B-1.19/0.59R	-31.24	91.00	--	--
MJ-L-1.19/0.59R	-36.06	80.54	86.58	-4.50

3.4.4 Rotational capacities of beam-column sub-assemblages

Fig. 3.12 shows the overall deformed profiles of sub-assemblages MJ-B-0.88/0.59R and MJ-L-0.88/0.59R at different stages, measured by a series of LVDTs along the beam length (see **Fig. 3.5**). It is apparent that when the catenary action capacity was attained, rotation of the middle joint was substantially larger than that of plastic hinges at the end column stubs, due to significant flexural deformations of the beam at catenary action stage. To evaluate the deformation capacity of sub-assemblages, rotation of sub-assemblages is calculated as a ratio of the middle joint displacement to the clear span 2.75 m of the beam when sub-assemblages attained their catenary action capacities. Rotation of plastic hinges near the end column stub was measured by the LVDTs mounted at the beam ends. **Table 3.6** lists the rotations of the beam-column sub-assemblages and the plastic hinges at the end column stub.



(a) MJ-B-0.88/0.59R



(b) MJ-L-0.88/0.59R

Fig. 3.12: Deformed profiles of beam-column sub-assemblages**Table 3.6: Rotations of plastic hinges and beam-column sub-assemblages**

Specimen	At the CAA capacity		At the catenary action capacity		
	Plastic hinge rotation (°)	Rotation of sub-assemblage (°)	Plastic hinge rotation (°)	Rotation of sub-assemblage (°)	Rotation of partial hinge (°)
MJ-B-0.52/0.35S	0.9	1.6	7.7	8.4	0.5
MJ-L-0.52/0.35S	0.5	1.5	6.9	13.2	5.7
MJ-B-0.88/0.59R	0.5	2.1	9.5	15.2	5.6
MJ-L-0.88/0.59R	0.7	2.1	8.2	14.0	8.0
MJ-B-1.19/0.59R	0.6	2.3	--	--	--
MJ-L-1.19/0.59R	0.7	2.1	8.4	10.8	1.5

Table 3.6 indicates that under column removal scenarios precast concrete beam-column sub-assemblages were able to develop comparable rotations to reinforced concrete sub-assemblages (Yu and Tan 2013c). The calculated rotation of sub-assemblages MJ-L-0.52/0.35S, MJ-B-0.88/0.59R and MJ-L-0.88/0.59R was significantly larger than the plastic hinge rotation at catenary action stage. The difference between the plastic hinge rotation and rotation of sub-assemblages was mainly attributable to the formation of a partial hinge in the vicinity of the curtailment point of beam top bars, as shown in **Fig. 3.13**. In accordance with the LVDT measurements along the beam length, the partial hinge rotation is approximately quantified as the difference between rotational angles of beam segments CD and BC (see **Fig. 3.5**), as expressed in Eq. (3-2). **Fig. 3.14** shows the calculated rotations of partial hinges in MJ-L-0.52/0.35S, MJ-B-0.88/0.59R and MJ-L-0.88/0.59R. It is observed that the rotation angle increased rapidly after the onset of catenary action, indicating the localisation of beam rotation at the partial hinge. Simultaneously, the rotation of plastic hinges near the end column stub levelled off, as shown in **Figs. 3.15(a and b)**. Eventually, the rotation of plastic hinges increased once again and led to failure of sub-assemblages. For sub-assemblage MJ-B-0.88/0.59R, the maximum rotation angle of partial hinge was 5.6°, as listed in **Table 3.6**. Similar results were obtained for MJ-L-0.52/0.35S and MJ-L-0.88/0.59R. Thus, the partial hinge rotation significantly increased the deformation capacity of beam-column sub-assemblages under column removal scenarios.

$$\theta_{PH} = \frac{\delta_{LB-3} - \delta_{LB-2}}{l_{CD}} - \frac{\delta_{LB-2} - \delta_{LB-1}}{l_{BC}} \quad (3-2)$$

where θ_{PH} is the rotation angle of the partial hinge at the curtailment point of beam top reinforcement; δ_{LB-1} , δ_{LB-2} and δ_{LB-3} are the readings of LVDTs LB-1, LB-2 and LB-3, respectively, as shown in **Fig. 3.5**; and l_{BC} and l_{CD} are the distances between sections B and C, and sections C and D, i.e. 450 mm and 625 mm, respectively.

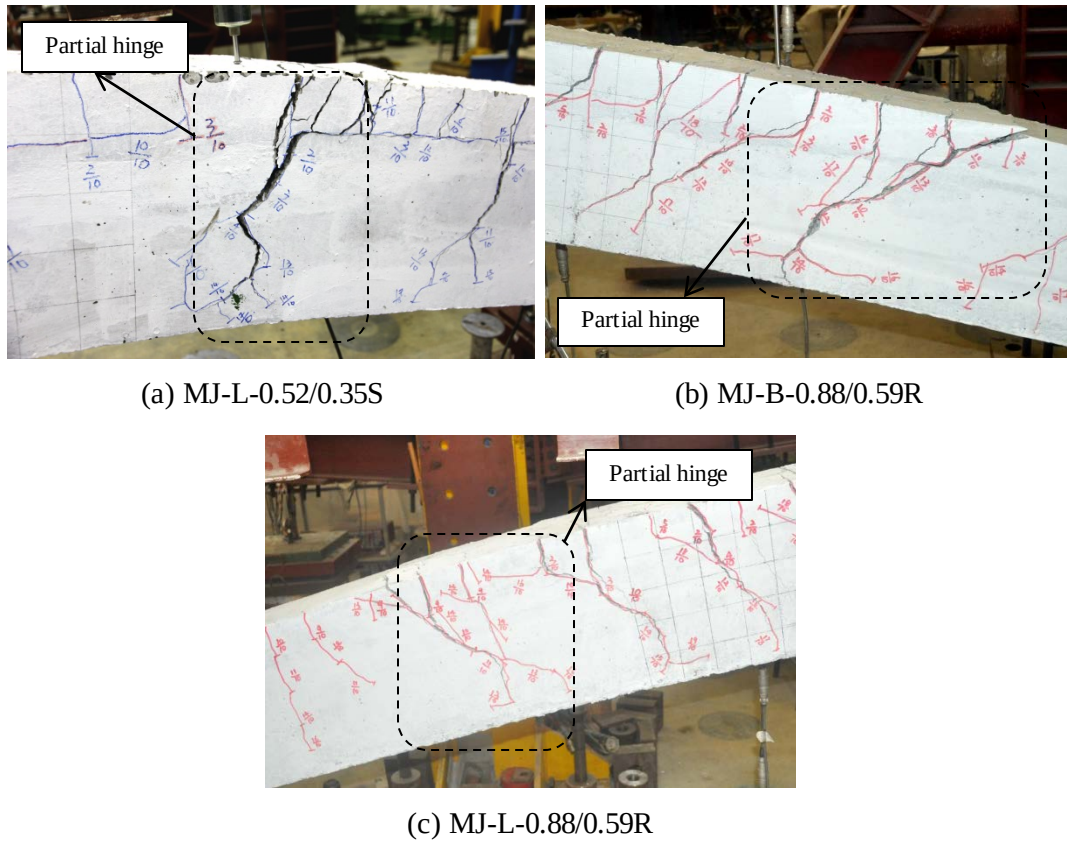


Fig. 3.13: Partial hinge at the curtailment point of top bars

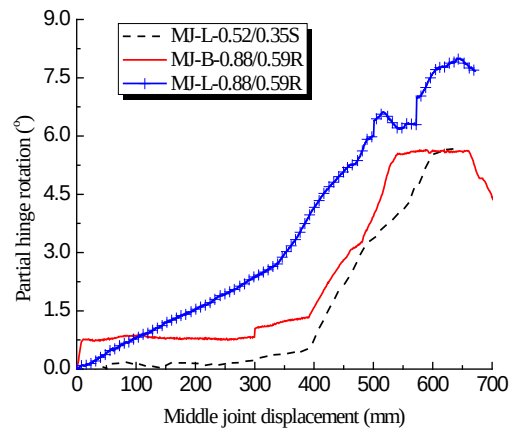


Fig. 3.14: Rotations of partial hinges at the curtailment point

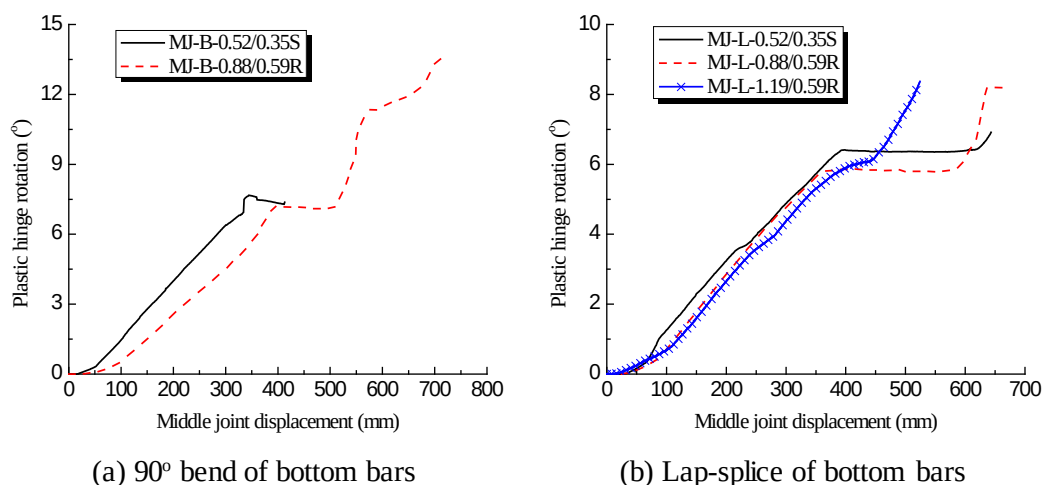


Fig. 3.15: Rotations of plastic hinges at the end column stub of sub-assemblages

3.4.5 Crack patterns and failure modes of precast beams

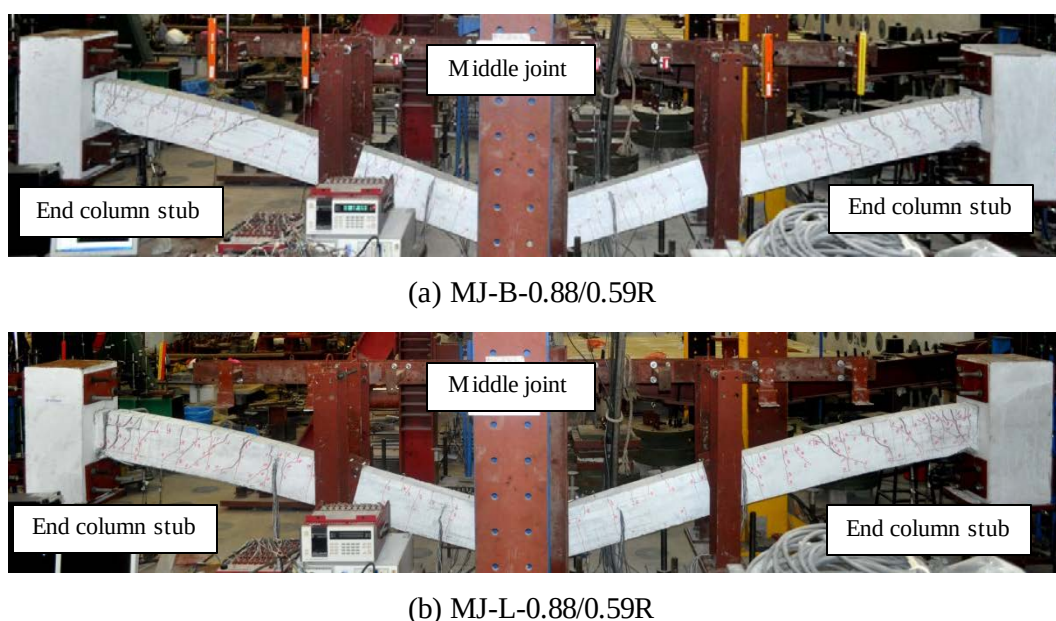


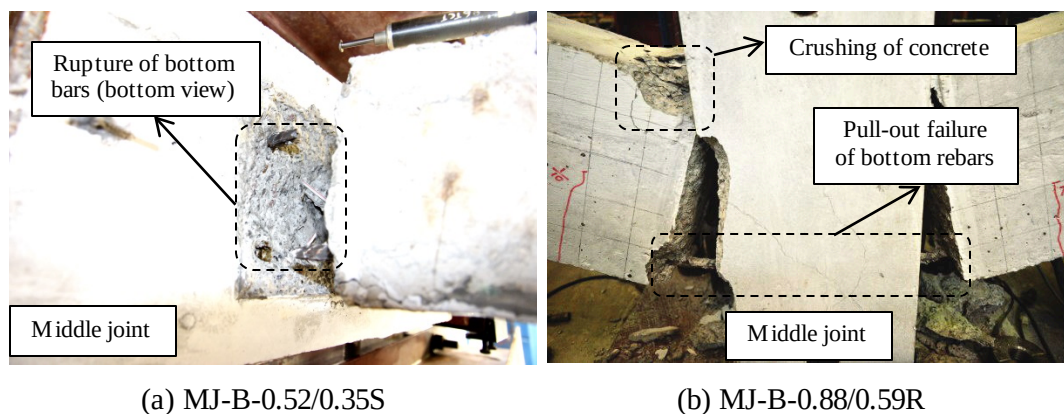
Fig. 3.16: Crack patterns of beam-column sub-assemblages

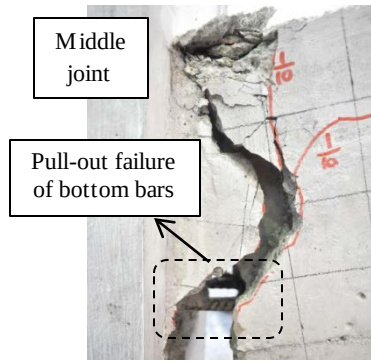
Fig. 3.16 shows the crack patterns of beam-column sub-assemblages MJ-B-0.88/0.59R and MJ-L-0.88/0.59R when their catenary action capacities were attained. Severe cracking was observed along the beam length. In the vicinity of the middle joint, axial tension force in the beam generated more full-depth tension cracks. Diagonal cracks were formed near the end column stub, due to the combined effect of axial tension and shear forces in the beam.

Table 3.7: Failure modes of beam-column sub-assemblages

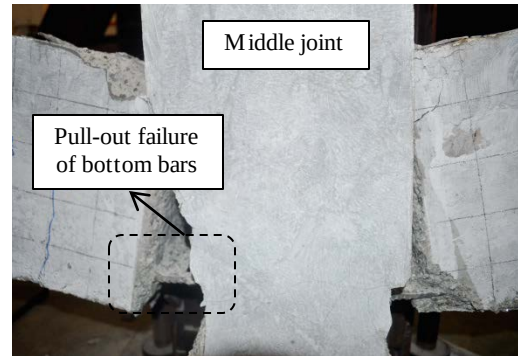
Specimen	In the middle joint	At the end column stub
MJ-B-0.52/0.35S	Fracture of bottom bars	Fracture of top bars
MJ-L-0.52/0.35S	Fracture of one bottom bar, pull-out of the other bar	Fracture of top bars
MJ-B-0.88/0.59R	Pull-out of bottom bars	Fracture of top bars
MJ-L-0.88/0.59R	Pull-out of bottom bars	Pull-out of bottom bars
MJ-L-1.19/0.59R	Pull-out of bottom bars	Fracture of top bars, pull-out of bottom bars

Table 3.7 summarises the failure modes of beam-column sub-assemblages at the middle joint and end column stub. Sub-assemblage MJ-B-0.52/0.35S exhibited fracture of bottom longitudinal reinforcement at the middle joint face, as shown in **Fig. 3.17(a)**. However, in MJ-B-0.88/0.59R, pull-out failure of beam bottom reinforcement was observed at the interface of the middle joint, as shown in **Fig. 3.17(b)**, due to inadequate embedment length of reinforcement. Similar pull-out failure of beam bottom reinforcement was also observed in the middle joint of MJ-L-0.88/0.59R, MJ-B-1.19/0.59R and MJ-L-1.19/0.59R, as shown in **Figs. 3.17(c-e)**. Final failure of beam-column sub-assemblages was caused by fracture of beam top longitudinal reinforcement near the end column stub (see **Figs. 3.18(a-c)**). Development of tension force in the beam also resulted in pull-out failure of beam bottom longitudinal reinforcement near the end column stub of MJ-L-0.88/0.59R and MJ-L-1.19/0.59R, as shown in **Figs. 3.18(d and f)**. It is notable that final failure of MJ-B-1.19/0.59R did not occur when the test stopped. Only the crack pattern in the plastic hinge region is shown in **Fig. 3.18(e)**.

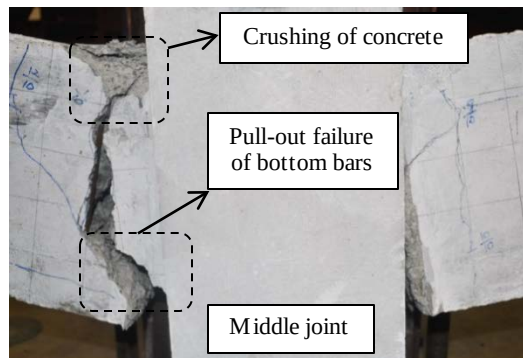




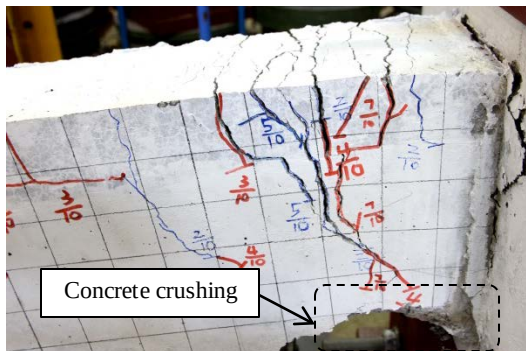
(c) MJ-L-0.88/0.59R



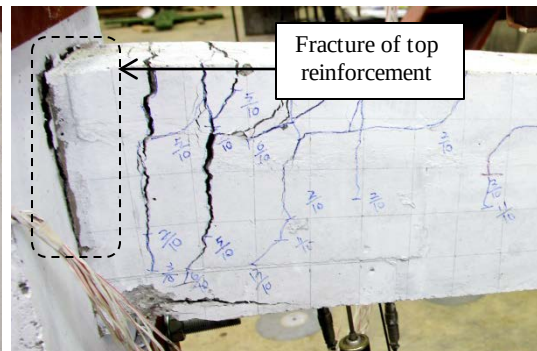
(d) MJ-B-1.19/0.59R



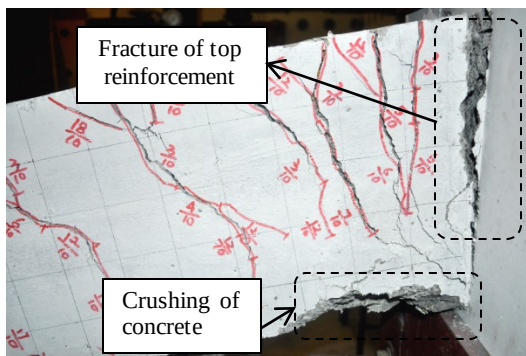
(e) MJ-L-1.19/0.59R

Fig. 3.17: Failure modes of sub-assemblages at the middle joint

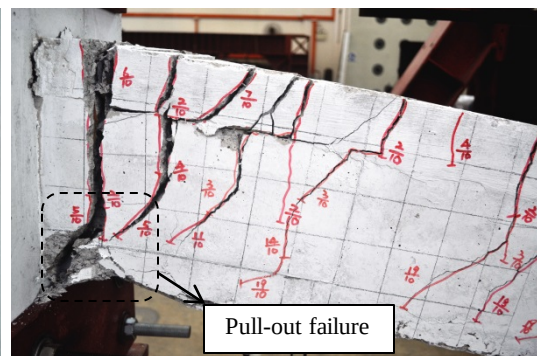
(a) MJ-B-0.52/0.35S



(b) MJ-L-0.52/0.35S



(c) MJ-B-0.88/0.59R



(d) MJ-L-0.88/0.59R

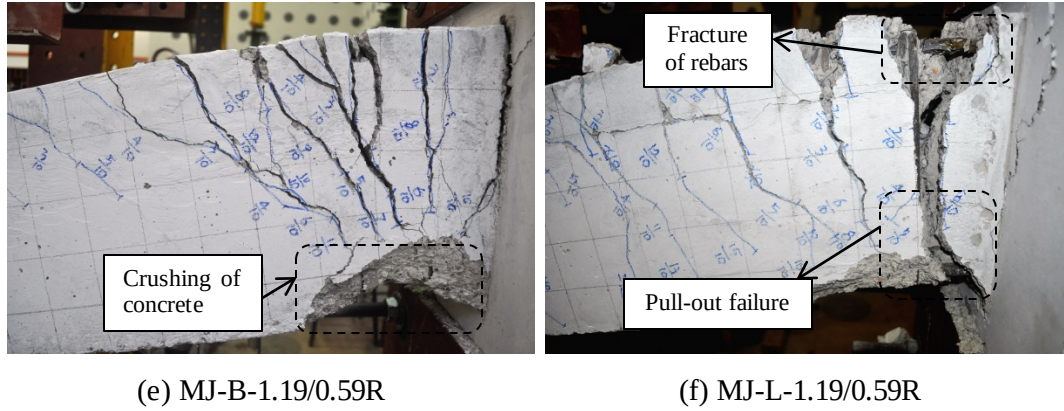


Fig. 3.18: Failure modes of beam-column sub-assemblages at the end column stub

3.4.6 Horizontal shear transfer between precast beam units and cast-in-situ concrete topping

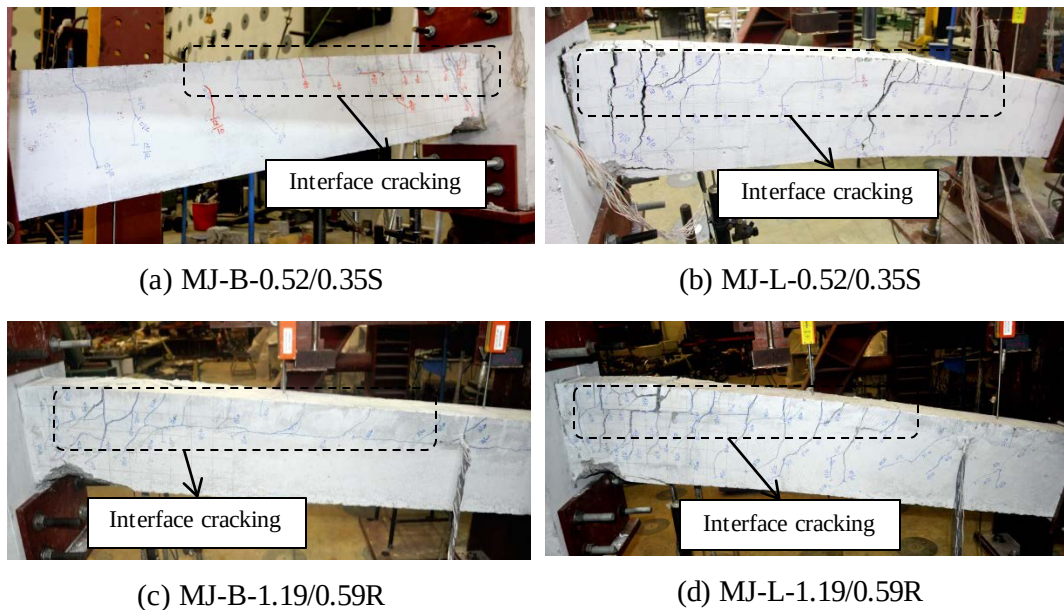


Fig. 3.19: Horizontal cracking across the concrete interface

Depending on the surface preparation of precast beam units, different horizontal interface behaviour was observed between the precast beam units and cast-in-situ concrete topping, as shown in **Fig. 3.19**. In MJ-B-0.52/0.35S and MJ-L-0.52/0.35S, a smooth horizontal interface was prepared between precast beam units and concrete topping, and horizontal shear strength was designed in accordance with Eurocode 2 (BSI 2004). Under column removal scenarios, severe horizontal cracking was observed across the concrete interface at CAA stage, as illustrated in **Figs. 3.19(a**

and b). These cracks were mainly initiated in the region between the cut-off point of top bars and the end column stub. The horizontal interface cracks resulted from axial compression force in the beam at CAA stage. The compression force increased sagging moment at the middle joint face and hogging moment at the end column stub face. Shear force in the beam was increased as well, which induced interface cracking between the precast beam units and cast-in-situ structural topping.

Compared to MJ-B-0.52/0.35S and MJ-L-0.52/0.35S, shear force acting on the horizontal interface of MJ-B-0.88/0.59R and MJ-L-0.88/0.59R was increased due to high moment resistance at the beam ends. However, by intentionally roughening the concrete interface to around 3 mm roughness according to BS EN 1992-1-1:2004 (BSI 2004), only limited interface cracks developed in the plastic hinge region near the end column stub. It indicates that a roughened interface was more effective in preventing horizontal cracking in comparison with a smooth interface.

In MJ-B-1.19/0.59R and MJ-L-1.19/0.59R, the same rough surface preparation and stirrups were utilised for the precast beam units, but the beam top reinforcement ratio was increased to 1.19% compared with MJ-B-0.88/0.59R and MJ-L-0.88/0.59R. Severe interface cracking was observed across the horizontal interface, as shown in **Figs. 3.19(c and d)**. Horizontal cracking weakened the composite action between precast beam units and structural topping, thereby reducing the CAA capacity of beam-column sub-assembly. For instance, the beam top reinforcement ratio in MJ-B-1.19/0.59R and MJ-L-1.19/0.59R was increased by 0.31% in comparison with MJ-B-0.88/0.59R and MJ-L-0.88/0.59R. However, its CAA capacity was only increased by 2.0 kN and 3.5 kN, respectively, as listed in **Table 3.4**. Therefore, effective horizontal shear transfer between the precast beam units and cast-in-situ concrete topping played a crucial role in the resistance of beam-column sub-assemblages at CAA stage.

3.4.7 Strains of beam longitudinal reinforcement

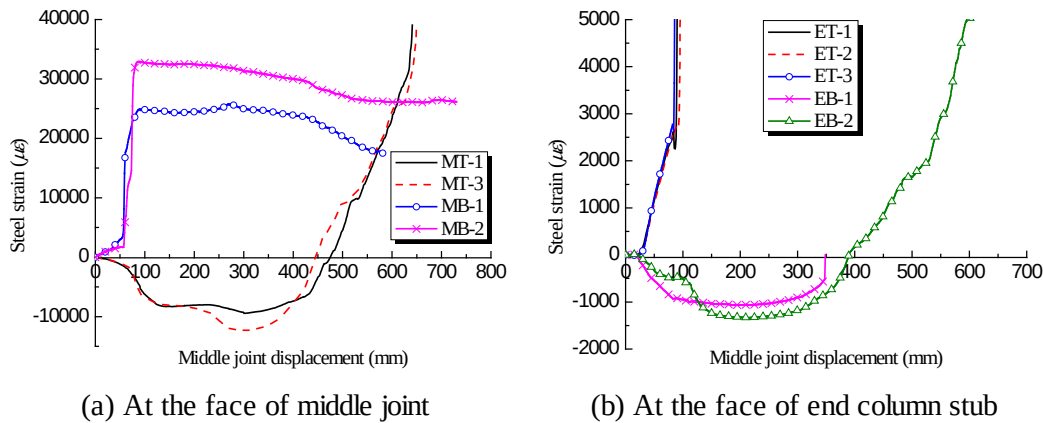


Fig. 3.20: Strains of beam longitudinal reinforcement in MJ-B-0.88/0.59R

To investigate the behaviour of bridging beams at different loading stages, strain gauges were mounted on the longitudinal reinforcement in the beams, as shown in **Fig. 3.6**. The failure mode of MJ-B-0.88/0.59R can also be demonstrated by the measured steel strains of the top and bottom longitudinal reinforcement at the middle joint and end column stub faces, as shown in **Fig. 3.20**. At the middle joint face, strains of bottom longitudinal reinforcement started decreasing gradually after attaining the maximum value at CAA stage, as shown in **Fig. 3.20(a)**, indicating pull-out failure of reinforcing bars. However, at the end column stub face, tensile strains of top longitudinal reinforcement kept increasing until the rebars ruptured at catenary action stage, as shown in **Fig. 3.20(b)**. Top longitudinal bars near the middle joint and bottom bars near the end column stub experienced compression at CAA stage, and then were transformed to tension due to subsequent development of catenary action, as shown in **Figs. 3.20(a and b)**. The measured steel strains agree well with the crack pattern and failure mode of MJ-B-0.88/0.59R (see **Fig. 3.17(b)** and **Fig. 3.18(c)**).

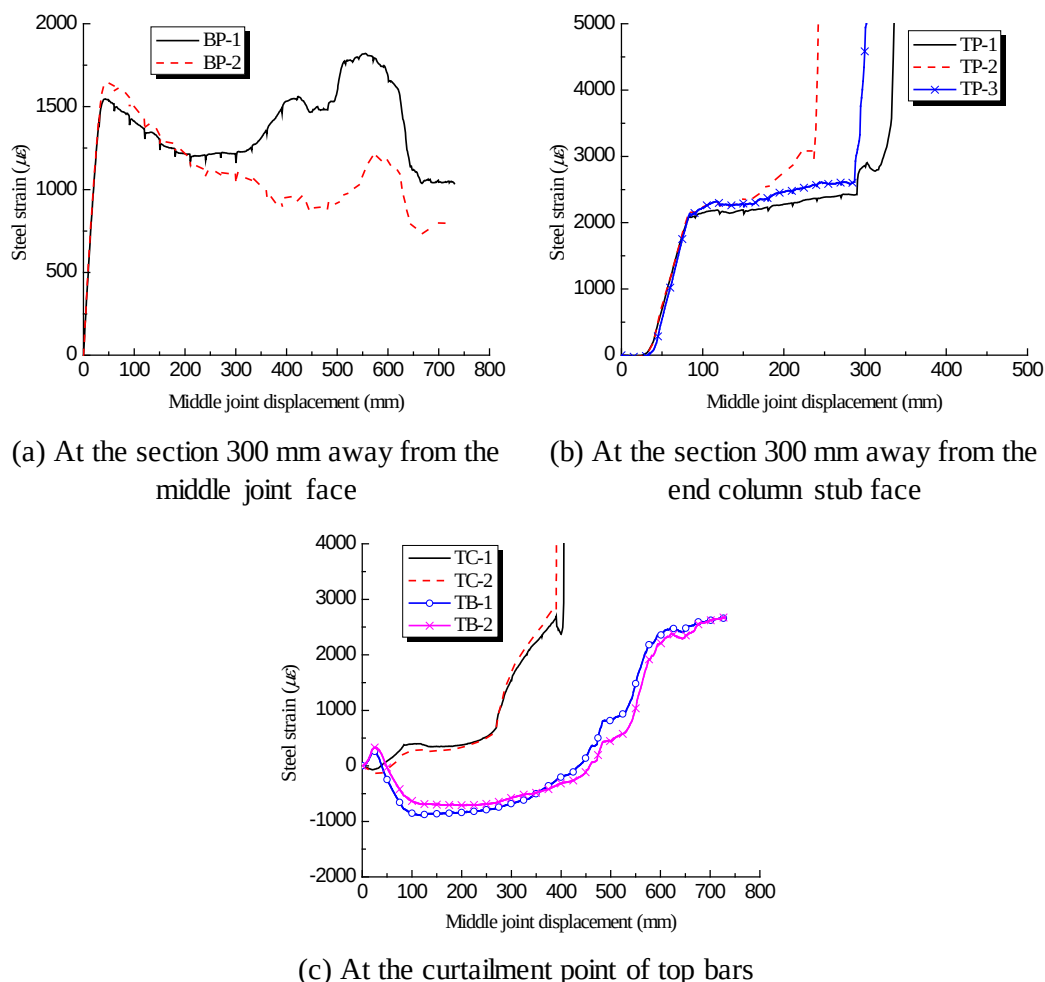


Fig. 3.21: Strains of beam longitudinal reinforcement in the beam of sub-assembly MJ-B-0.88/0.59R

At the beam section 300 mm away from the middle joint face, strains of bottom longitudinal reinforcement in the beam increased to their peak values at CAA stage, and then decreased as a result of pull-out failure of beam bottom reinforcement passing through the middle joint, as shown in **Fig. 3.21(a)**. Following the onset of catenary action, steel strains at the section increased once again due to development of axial tension force in the beam. At the section 300 mm away from the end column stub face, steel strains experienced a plateau stage, with values less than the yield strain of steel reinforcement, and then increased at catenary action stage, as shown in **Fig. 3.21(b)**. It indicates that the length of plastic hinge near the end column stub was shorter than one beam depth (300 mm) at CAA stage. When catenary action kicked in, a partial hinge was formed at the curtailment point of beam top longitudinal reinforcement (see **Fig. 3.13(b)**). Accordingly, the top reinforcement at the section

attained its yield strain, whereas the bottom reinforcement was still in compression, as shown in **Fig. 3.21(c)**. Eventually, bottom reinforcement at the curtailment point of top reinforcement was transformed from compression to tension due to the presence of axial tension force in the beam.

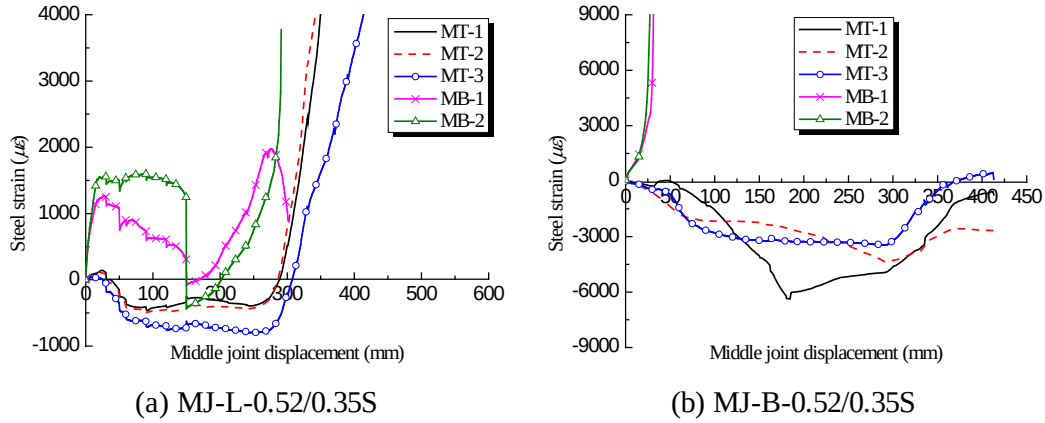


Fig. 3.22: Strains of beam longitudinal reinforcement at the middle joint

Fig. 3.22(a) shows the variations of strains of beam longitudinal reinforcement in the middle joint of MJ-L-0.52/0.35S. Similar to MJ-B-0.88/0.59R, strains MB-1 and MB-2 decreased due to pull-out failure of beam bottom reinforcement in the middle joint. Rupture of one bottom bar in the middle joint led to a sudden reduction of tensile strains of bottom reinforcing bars. Following the rupture of bottom reinforcement, strains of bottom bars could increase with increasing middle joint displacement, as bottom reinforcement at the opposite middle joint face was mobilised by the rotational restraint in the middle joint to resist tension force. Nevertheless, MJ-B-0.52/0.35S exhibited rupture of all bottom bars in the middle joint, as shown in **Fig. 3.17(a)**. Thus, bottom reinforcement at the middle joint interface kept increasing until rupture of reinforcement occurred (see **Fig. 3.22(b)**).

3.5 Discussions and Suggestions

In accordance with UFC 4-023-03 (DOD 2013), the plastic rotation angle for beam-column sub-assembly MJ-B-0.52/0.35S is determined as 1.7° (0.029 radian), whereas the angle for the other five beam-column sub-assemblages is reduced to 0.6° (0.01 radian) due to pull-out failure of beam bottom bars in the middle joint. The acceptance criteria are reasonable if only CAA is taken into account in analysis.

However, when catenary action in the beam-column sub-assemblages is considered, the criteria are too conservative in comparison with the calculated sub-assemblage rotations, as listed in **Table 3.6**. Moreover, pull-out failure of bottom reinforcement in the joint did not significantly reduce the rotation capacity of beam-column sub-assemblages, as long as continuous longitudinal reinforcement was placed in the structural topping and properly embedded in the beam-column joint with adequate anchorage length. Therefore, it is suggested that the acceptance criteria be increased to 11.5° (0.2 radian) to account for catenary action at large deformations, which is consistent with the required rotation for development of tie force in the bridging beam (DOD 2013). It is noteworthy that the revised acceptance criteria are only suitable for fairly rigid boundary conditions. As for inadequate horizontal restraints, more experimental tests are needed to investigate the deformation capacity of beam-column sub-assemblages.

In the design of precast concrete structures against progressive collapse, pull-out failure of embedded reinforcement in the beam-column joint has to be prevented to ensure a more robust structure. Thus, the embedment length of steel reinforcement is suggested to be increased. It may lead to an increase in the cross section of middle columns to accommodate longer embedment length of bottom reinforcement with hooked anchorage, which in turn elevate the difficulties in construction of middle joints; however, only the length of precast trough needs to be adjusted for lap-spliced reinforcement in the middle joint. Besides, in determining the horizontal shear stress across the concrete interface, it is suggested an amplification factor be incorporated to consider the effect of axial compression force in the beam on horizontal shear stress. More stringent requirements on interface treatment have to be employed to ensure full composite action between precast beam units and cast-in-situ concrete topping.

3.6 Conclusions

In this chapter, six experiments were conducted to investigate the behaviour of precast concrete beam-column sub-assemblages under middle column removal scenarios. Two types of middle joint detailing, namely, 90° bend and lap-splice of bottom reinforcement, were studied under quasi-static loading conditions. The following conclusions can be made:

(1) With continuous top reinforcement in the structural topping, CAA and catenary action could be developed in sub-assemblages with 90° bend and lap-splice of bottom longitudinal reinforcement in the joint. A typical failure mode in the middle joint of sub-assemblages was pull-out failure of bottom longitudinal reinforcement, except MJ-B-0.52/0.35S which exhibited fracture of bottom bars. Near the end column stub, fracture of beam top longitudinal reinforcement represented the most common failure mode. However, in MJ-L-0.88/0.59R and MJ-L-1.19/0.59R, bottom beam bars exhibited pull-out failure.

(2) Greater top and bottom reinforcement ratios in MJ-B-0.88/0.59R and MJ-L-0.88/0.59R enhanced CAA and catenary action compared with MJ-B-0.52/0.35S and MJ-L-0.52/0.35S. A further increase in top reinforcement ratio of MJ-B-1.19/0.59R and MJ-L-1.19/0.59R did not impose a considerable beneficial effect on the CAA capacity in comparison with MJ-B-0.88/0.59R and MJ-L-0.88/0.59R, possibly due to severe shear cracking across the horizontal interface between precast beam units and cast-in-situ concrete topping.

(3) Horizontal cracking was observed between the curtailment point of top bars and the end column stub in MJ-B-0.52/0.35S, MJ-L-0.52/0.35S, MJ-B-1.19/0.59R and MJ-L-1.19/0.59R. At CAA stage, development of compression force in the beam increased the horizontal shear stress at the concrete interface. Therefore, more stringent interface preparation has to be implemented to achieve full composite action between precast beam units and cast-in-situ concrete topping.

(4) Except MJ-B-0.52/0.35S, precast concrete beam-column sub-assemblages were able to develop much greater rotations compared to the requirements in UFC 4-023-03 if catenary action in the beam was considered. Thus, it is suggested that the acceptance criteria be revised in accordance with experimental results.

The experimental results represent the resistance of precast beam-column sub-assemblages with relatively rigid boundary conditions. In comparison with realistic horizontal restraints, the CAA and catenary action capacities of the bridging beam are overestimated. Therefore, further experimental tests are necessary to evaluate the influence of restraint boundary conditions on the behaviour of sub-assemblages, which is the topic in the following chapters.

CHAPTER 4 EXPERIMENTAL TESTS OF PRECAST BEAM-COLUMN SUB-ASSEMBLAGES WITH ENGINEERED CEMENTITIOUS COMPOSITES

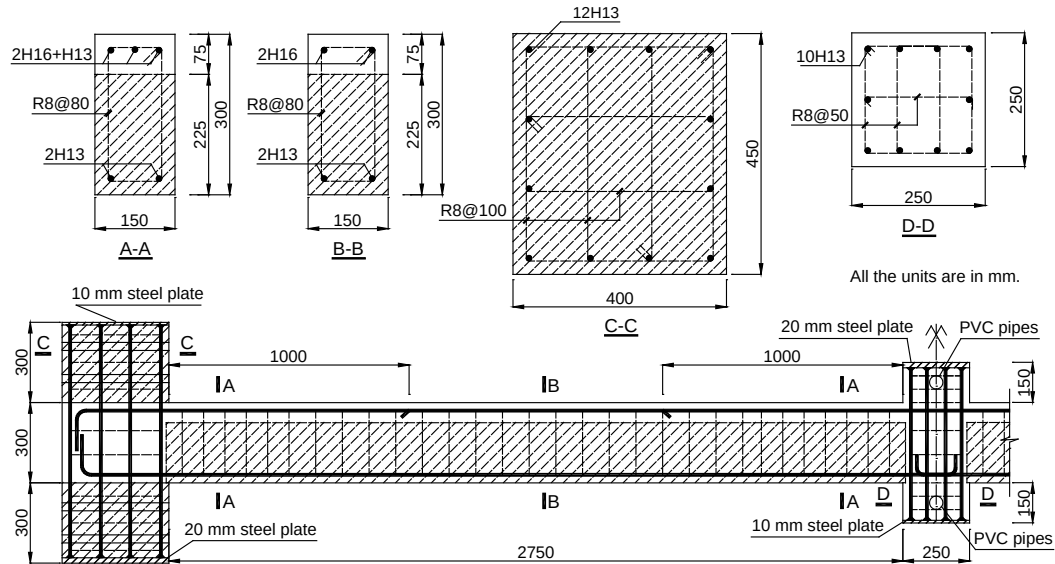
4.1 Introduction

By using either lap-splice or 90° bend of bottom reinforcement in the joint, precast concrete beam-column sub-assemblages exhibited significant compressive arch action (CAA) and catenary action under column removal scenarios, as discussed in Chapter 3. Pull-out failure of bottom reinforcement was observed in the middle joint, indicating inadequate embedment length of reinforcement. Besides special reinforcement detailing in the joint, precast concrete structures also allow innovative materials such as engineered cementitious composites (ECC) to be placed in critical regions so as to enhance structural performance under various loading conditions. One potential advantage of applying ECC lies in its compatible deformations with steel reinforcing bars (Fischer and Li 2002b; Li 2003), which can significantly reduce the required embedment length or lap length between steel bars to develop the full yield strength. Moreover, it exhibits superior strain capacity and damage tolerance in uniaxial tension. However, applications of ECC to mitigate progressive collapse remains a concern due to a high deformation demand on bridging beams to develop CAA and subsequent catenary action under column removal scenarios.

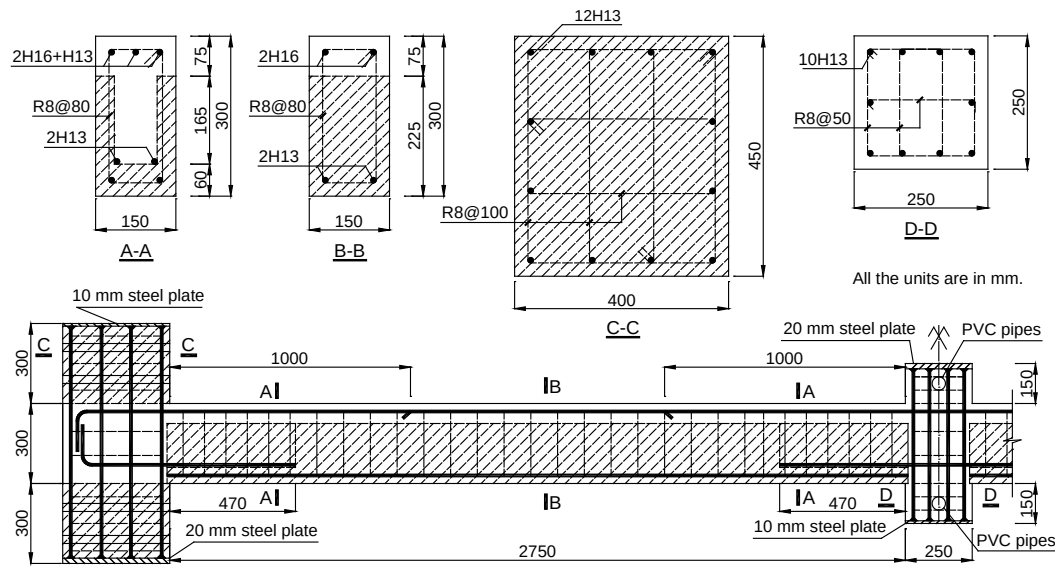
This chapter presents an experimental study on the behaviour of six beam-column sub-assemblages subject to column removal, in which ECC was placed in the structural topping and the beam-column joint. The resistances and failure modes of sub-assemblages were investigated in the experimental programme. Comparisons were also made between the deformation capacities of sub-assemblages made of conventional concrete and ECC. Finally, interactions between steel reinforcement and ECC were studied to gain a deep insight into the effect of ECC on the behaviour of beam-column sub-assemblages under column removal scenarios.

4.2 Experimental Programme on Sub-Assemblages

4.2.1 Specimen design



(a) EMJ-B-1.19/0.59



(b) EMJ-L-1.19/0.59

Fig. 4.1: Geometric properties of precast beam-column sub-assemblages

To develop alternate load paths, one middle supporting column was assumed to be “forcibly removed” without any damage to the beam-column joint (DOD 2013). This is a threat-independent approach and it embodies a number of assumptions. Chiefly among them is the assumption that only one column is removed at one time of an

analysis, which implies that the approach can only be used for a small blast charge. In this regard, the double-span bridging beam with a middle joint above the removed column was extracted from the damaged region and tested under quasi-static push-down loading condition to investigate the resistance and ductility of the joint. Two enlarged concrete stubs were designed at the beam ends to provide horizontal and vertical restraints for the beam-column sub-assembly, as illustrated in **Fig. 4.1**.

To investigate the integrity of precast concrete structures subject to column removal scenarios, precast beam units with cast-in-situ concrete topping were selected in the experimental programme based on local and international construction practices (CAE 1999; FIB 2003). Beam units with bottom longitudinal reinforcement were prefabricated in the casting yard, and then assembled with top reinforcement prior to placement of cast-in-situ concrete topping and beam-column joint. This type of construction technology enables precast concrete structures to perform as well as monolithic reinforced concrete structures, but at the same time seeks to achieve higher productivity through reinforcement detailing (Shiohara and Watanabe 2000). **Fig. 4.1** shows the geometry and reinforcement detailing in beam-column sub-assemblages. It is noteworthy that the hatched zones represent the precast concrete beam units, whereas other parts were made of ECC. The cross section of the beam was 150 mm wide by 300 mm high, in which the depths of the precast beam unit and structural topping were 225 mm and 75 mm, respectively, as shown in **Fig. 4.1**. The clear span of the beam was 2.75 m. The middle column stub was 250 mm square, with a total height of 600 mm.

Table 4.1: Reinforcement details of precast beam-column sub-assemblages

Specimen	Joint detailing	Longitudinal reinforcement*				Stirrups&
		A-A section		B-B section		
		Top	Bottom	Top	Bottom	
CMJ-B-1.19/0.59#	90° bend	2H16+H13 (1.19%)	2H13 (0.59%)	2H16 (0.90%)	2H13 (0.59%)	R8@80
EMJ-B-1.19/0.59	90° bend					
EMJ-L-1.19/0.59	Lap-splice					
EMJ-B-0.88/0.59	90° bend	3H13 (0.88%)	2H13 (0.59%)	2H13 (0.59%)	2H13 (0.59%)	R8@80
EMJ-L-0.88/0.59	Lap-splice					
EMJ-L-0.88/0.88	Lap-splice	3H13 (0.88%)	3H13 (0.88%)	2H13 (0.59%)	2H13 (0.59%)	R8@80

*: H16 and H13 denote high-yield strength deformed reinforcement with nominal diameters of 16 mm and 13 mm, respectively. The value in parenthesis is the geometric reinforcement ratio calculated from $A_s/(bh)$, where $b=150$ mm and $h=300$ mm.

&: R8 represents low-yield strength round bar with nominal diameter of 8 mm. The concrete clear cover was 20 mm, measured from the beam surface to the outmost of stirrups.

#: The beam-column joint and concrete topping of specimen CMJ-B-1.19/0.59 were made of conventional concrete.

In the experimental programme, longitudinal reinforcement was placed in structural topping and passed through the middle beam-column joint of sub-assemblages continuously. Two types of bottom reinforcement detailing, which were identical to those used in precast concrete beam-column sub-assemblages in Chapter 3, were used in the joint of sub-assemblages, as shown in **Fig. 4.1**. The first joint detailing consisted of 90° bend of beam bottom reinforcement protruding from the beam end and anchored in the joint, as shown in **Fig. 4.1(a)**. The second detailing was characterised by lap-spliced bottom reinforcement in the joint, as shown in **Fig. 4.1(b)**. For the second detailing, precast beam units with a trough at each end were cast first, and bottom reinforcement was placed in the middle joint to provide continuity. Based on a concrete cylinder strength of 30 MPa, the anchorage length of bottom steel reinforcing bars was calculated as 470 mm (36 times the rebar diameter). Precast concrete beam-column sub-assemblages exhibited horizontal cracking across the interface between precast beam unit and cast-in-situ concrete topping under column removal scenarios, as discussed in Section 3.4.6. To prevent horizontal cracking and to ensure adequate composite action between the precast beam unit and cast-in-situ concrete topping, sufficient stirrups with a diameter of 8 mm at 80 mm spacing were placed along the beam length. Horizontal interface between precast concrete beam units and ECC was intentionally roughened to 3 mm deep, so as to comply to requirements from Eurocode 2 (BSI 2004).

CMJ-B-1.19/0.59 made from conventional concrete was designed against gravity loads in accordance with Eurocode 2 (BSI 2004). Another five sub-assemblages with different reinforcement detailing and longitudinal reinforcement ratios in the beam were fabricated, in which ECC was used to replace conventional concrete in the structural topping of the double-span beam and beam-column joint. **Table 4.1** lists the reinforcement details of beam-column sub-assemblages. In the notations, the alphabets “CMJ” and “EMJ” represent precast beam-column sub-assemblages with conventional concrete and ECC in structural topping and beam-column joint,

respectively, and “B” and “L” stand for 90° bend and lap-splice of bottom bars in the middle joint. The first and second numerals denote the respective percentages of top and bottom reinforcement at the middle joint. Beam-column sub-assembly CMJ-B-1.19/0.59 with conventional concrete beam and structural topping served as the control specimen, in which 90° bend of beam bottom reinforcement was used in the joint. In EMJ-B-1.19/0.59, concrete topping and beam-column joint were replaced by ECC, whereas the other parameters remained the same as CMJ-B-1.19/0.59, as shown in **Fig. 4.1(a)**, so as to study the effect of ECC on structural resistance. In comparison with EMJ-B-1.19/0.59, lap-spliced beam bottom reinforcement was applied in the middle joint of EMJ-L-1.19/0.59 (see **Fig. 4.1(b)**) to study the effect of reinforcement detailing on the middle joint behaviour. To quantify the effect of beam top reinforcement ratios on progressive collapse resistance, the top reinforcement ratio of EMJ-B-0.88/0.59 and EMJ-L-0.88/0.59 was reduced from 1.19% to 0.88% (see ‘A-A’ section in **Table 4.1**), but the bottom reinforcement ratio was kept the same at 0.59%. Lastly, compared with EMJ-L-0.88/0.59, only the bottom reinforcement ratio was increased from 0.59% to 0.88% in sub-assembly EMJ-L-0.88/0.88 to investigate the influence of bottom reinforcement ratios. Exactly the same test setup as that for precast concrete beam-column sub-assemblages was employed, as shown in **Fig. 3.3** and **Fig. 3.5**.

4.2.3 Material properties

Table 4.2 Mixture proportions of ECC

Ingredient	Cement	Water	Micro-sand	GGBS	PVA fibre
Unit weight (kg/m ³)	430	387	287	1004	26

The key point of the experimental programme lies in the utilisation of ECC materials in precast beam-column sub-assemblages. Thus, the desirable material properties of ECC play a crucial role in potentially enhancing the progressive collapse resistance of the sub-assemblages. Hence, prior to the tests, ECC was designed and tailored made with ordinary Portland cement, ground granulated blast-furnace slag (GGBS), micro-sand, water, and Polyvinyl Alcohol (PVA) fibres, so as to achieve the desired multi-cracking and strain-hardening behaviour in tension. **Table 4.2** shows the mix design of ECC. It is noteworthy that PVA fibres with a diameter of 0.039 mm and a

length of 12 mm were used for ECC. ECC plates of dimensions 75 mm wide by 300 mm long by 12 mm thick were prepared and tested under four-point bending with a 240 mm clear span. **Fig. 4.2** shows a typical load-deflection curve of ECC plates. After the first cracking, applied load increased with significant hardening behaviour, and the deflection corresponding to the maximum load of ECC plates could be up to 13.6 mm. Based on experimental results of ECC plates under four-point bending, tensile stresses and strain capacity of ECC were calculated through inverse methods (Qian and Li 2007; Qian and Li 2008), as shown in **Table 4.3**. The effective tensile strength was estimated as 3.1 MPa and the strain capacity of ECC in tension was around 2.6%. **Table 4.3** also includes the properties of ECC in compression. It should be noted that the compressive strength of ECC was obtained experimentally by testing 50 mm cubes. The equivalent compressive strength of 150 mm diameter by 300 mm high cylinder was around 45.0 MPa.

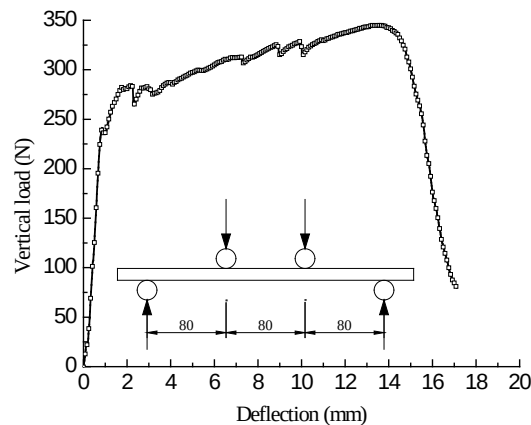


Fig. 4.2: Load-deflection curve of ECC plates under four-point bending

Table 4.3 Strength of ECC in tension and compression

Compressive strength (MPa)		Effective tensile strength (MPa)	Tensile strain capacity
50 mm cube	150 mm by 300 mm cylinder (equivalent)*		
62.7	45.0	3.1	2.6%

*: Compressive strength of 50 mm ECC cubes was converted to that of 150 mm cubes by multiplying a reduction factor of 0.9; thereafter, the equivalent compressive strength of 150 mm diameter by 300 mm long cylinders was calculated by multiplying a modification factor of 0.8.

In addition to ECC plates, material tests were also conducted on concrete cylinders and steel reinforcing bars. **Fig. 4.3** shows a typical stress-strain relationship of a concrete cylinder and a steel bar. It is notable that concrete strain was calculated from the contraction in the middle zone (100 mm long) of 150 mm diameter by 300 mm

long concrete cylinder measured by LVDTs, and it represented the average strain in the concrete zone with little confinement effect. **Table 4.4** summarises the material properties of steel reinforcing bars and concrete, which represent the average values of three coupons for each material.

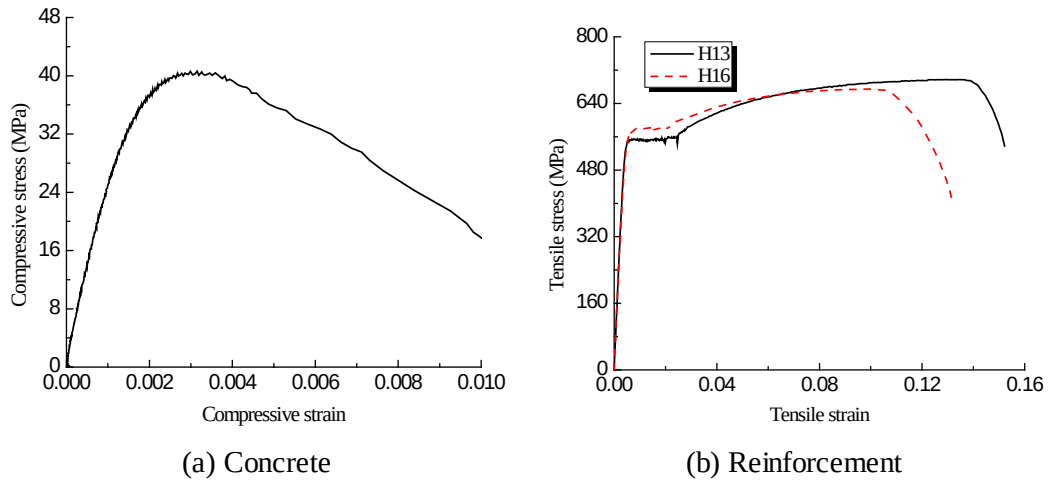


Fig. 4.3: Stress-strain relationships of concrete and steel bar

Table 4.4: Material properties of reinforcing and concrete

Steel reinforcement		Yield Strength (MPa)	Modulus of elasticity (GPa)	Ultimate strength (MPa)	Fracture strain* (%)
Longitudinal reinforcement	H13	549	206.6	698	16.3
	H16	573	211.3	674	12.9
Stirrup	R8	270	202.5	371	27.5
Concrete		Compressive strength (MPa)		Secant modulus (GPa)	
Precast beam unit		40.5		29.2	
Cast-in-situ concrete topping (CMJ-B-1.19/0.59 only)		36.1		31.4	

*: Fracture strain refers to the average strain over a gauge length of 300 mm when steel reinforcement ruptures.

4.3 Resistances of Beam-Column Sub-Assemblages

Under column removal scenarios, CAA and catenary action sequentially developed in the bridging beam. **Table 4.5** summarises the vertical load resistances and horizontal reaction forces of beam-column sub-assemblages subjected to CAA and catenary action. The CAA capacity refers to the maximum vertical load at the stage when horizontal compression force develops in the beam, whereas the catenary action capacity represents the peak load when the beam is subjected to axial tension force.

Horizontal force represents the average value of total horizontal forces acting on the left and right end column stubs. Variations of vertical loads and horizontal reaction forces versus middle joint displacement are shown in **Fig. 4.4** and **Fig. 4.5**. Discussions are made on the effects of ECC, reinforcement detailing in the middle joint and reinforcement ratios in the beam on the behaviour of sub-assemblages.

Table 4.5: Resistances of beam-column sub-assemblages

Specimen	CAA			Catenary action		
	Capacity P_c (kN)	Displacement at P_c (mm)	Max. horizontal compression N_c (kN)	Capacity P_t (kN)	Displacement at P_t (mm)	Max. horizontal tension force N_t (kN)
CMJ-B-1.19/0.59	90.4	105.7	-281.1	108.2	452.0	200.4
EMJ-B-1.19/0.59	91.1	108.9	-274.7	110.3	430.2	199.0
EMJ-L-1.19/0.59	91.1	103.1	-305.8	88.3	431.2	192.2
EMJ-B-0.88/0.59	83.7	101.9	-262.8	55.2	319.3	16.2
EMJ-L-0.88/0.59	82.5	106.9	-317.7	65.3	339.3	97.3
EMJ-L-0.88/0.88	79.2	171.2	-74.4	78.7	386.0	144.1

4.3.1 Effect of ECC

In CMJ-B-1.19/0.59, conventional concrete was used in the structural topping and the beam-column joint. When middle joint displacement was smaller than 25 mm, horizontal reaction force was zero due to gaps in connection between the end column stub and the horizontal restraints, as shown in **Fig. 4.4(b)**. Sub-assemblage CMJ-B-1.19/0.59 was under flexure. With increasing middle joint displacement beyond 25 mm, development of horizontal compression force indicated the commencement of CAA. After attaining the CAA capacity of 90.4 kN and the maximum horizontal compression force of 281.1 kN, both the applied vertical load and horizontal compression force decreased, as shown in **Figs. 4.4(a and b)**, due to progressive crushing of concrete in the compression zones at the middle joint and end column stub. In the descending phase of vertical load, fracture of beam bottom longitudinal reinforcement at the right face of the middle joint led to a sudden drop of the applied load (see **Fig. 4.4(a)**). Thereafter, vertical load started increasing again prior to the onset of catenary action as a result of rotational restraint in the middle joint. When

the middle joint displacement surpassed one beam depth of 300 mm, catenary action was mobilised with increasing axial tension force in the beam, as shown in **Fig. 4.4(b)**. Eventually, fracture of beam top longitudinal reinforcement at the face of the left column stub caused final failure of CMJ-B-1.19/0.59. The sub-assembly was able to develop the catenary action capacity of 108.2 kN and the maximum horizontal tension force of 200.4 kN at the catenary action stage, as included in **Table 4.5**. The ultimate middle joint displacement attained at the catenary action capacity was 452 mm, at about middle joint displacement of 1.5 times the beam depth.

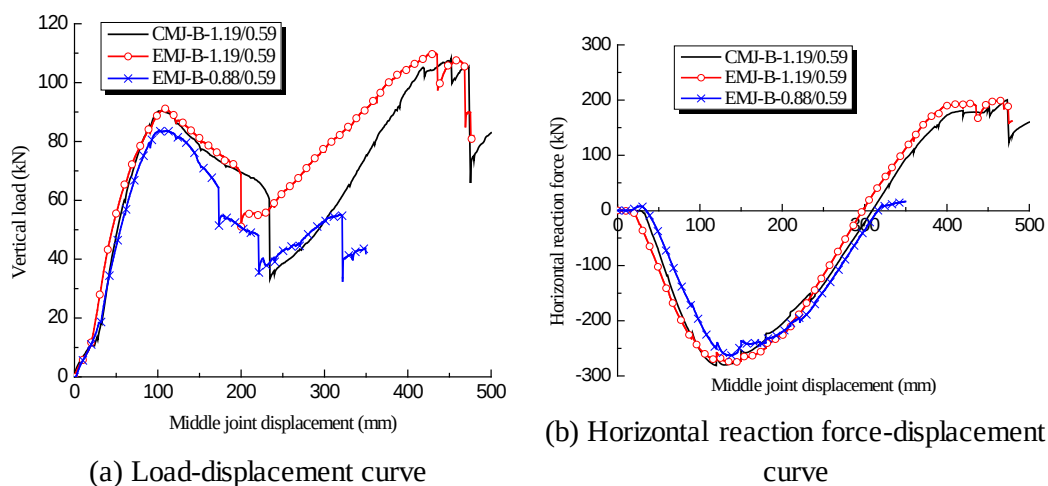


Fig. 4.4: Variations of vertical loads and horizontal reaction forces of sub-assemblages of bottom reinforcement with 90° bend

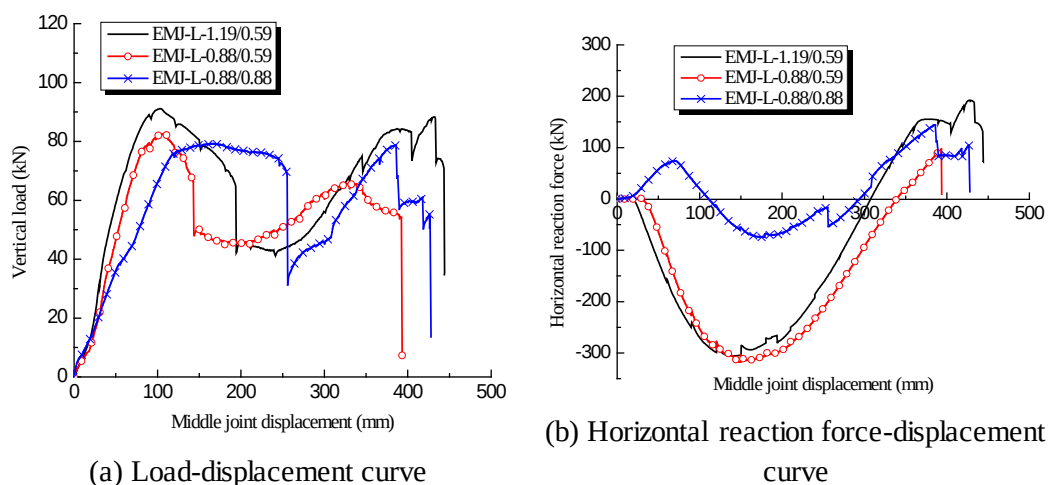


Fig. 4.5: Variations of vertical loads and horizontal reaction forces of sub-assemblages with lap-spliced bottom reinforcement

Compared to CMJ-B-1.19/0.59, sub-assembly EMJ-B-1.19/0.59 possessed the same top and bottom reinforcement ratios in the beam and joint detailing. Although

ECC was used in the structural topping and the beam-column joint in place of conventional concrete, sub-assembly EMJ-B-1.19/0.59 exhibited a similar behaviour to CMJ-B-1.19/0.59 under column removal scenarios, as shown in **Figs. 4.4(a and b)**. At CAA stage, the maximum vertical load sustained by EMJ-B-1.19/0.59 was 91.1 kN and the peak horizontal compression force was 274.7 kN (see **Table 4.5**). Following the fracture of beam bottom reinforcement at the right face of the middle joint, catenary action kicked in as the last line of defence against collapse. At the catenary action stage, vertical load was continually increased with increasing middle joint displacement (see **Fig. 4.4(a)**), whereas a plateau stage with almost constant horizontal tension force was observed in the horizontal reaction force-middle joint displacement curve, as shown in **Fig. 4.4(b)**. At middle joint displacement of 430.2 mm (1.45 times the beam depth), sub-assembly EMJ-B-1.19/0.59 attained its catenary action capacity of 110.3 kN. The maximum horizontal tension force was 199.0 kN at catenary action stage. Compared with CMJ-B-1.19/0.59, EMJ-B-1.19/0.59 developed almost the same CAA capacity and catenary action capacity, as included in **Table 4.5**, although ECC was utilised in the structural topping and the beam-column joint. It seems to indicate that ECC did not significantly enhance the resistance of beam-column sub-assembly at large deformation stage.

4.3.2 Effect of reinforcement detailing

In EMJ-L-1.19/0.59, lap-spliced beam bottom reinforcement was employed in the middle beam-column joint, whereas the top and bottom reinforcement ratios in the beam remained the same as EMJ-B-1.19/0.59. **Figs. 4.5(a and b)** show the vertical load and horizontal reaction force versus the middle joint displacement curves of sub-assembly EMJ-L-1.19/0.59. In comparison with EMJ-B-1.19/0.59, EMJ-L-1.19/0.59 attained the same vertical load capacity of 91.1 kN at CAA stage, but its maximum horizontal compression force was 305.8 kN, 11% greater than EMJ-B-1.19/0.59, as listed in **Table 4.5**. At catenary action stage, vertical load and horizontal tension force reached their maximum values simultaneously after fracture of one beam longitudinal bar occurred at the face of the left column stub. The catenary action capacity of EMJ-L-1.19/0.59 was 88.3 kN, 20% lower than that of EMJ-B-1.19/0.59. The maximum horizontal tension force in EMJ-L-1.19/0.59 was only 4% less than

that in EMJ-B-1.19/0.59. With increasing middle joint displacement, subsequent fracture of all beam top reinforcement at the face of the left column stub led to reductions in vertical load and horizontal tension force, as shown in **Figs. 4.5(a and b)**. Thereafter, a pin joint was formed at the left column stub, indicating failure of beam-column sub-assembly EMJ-L-1.19/0.59. Therefore, by changing the reinforcement detailing in the middle joint from 90° bend to lap-splice, similar CAA capacities were obtained in EMJ-L-1.19/0.59 and EMJ-B-1.19/0.59. However, the catenary action capacity of EMJ-L-1.19/0.59 was significantly lower compared to EMJ-B-1.19/0.59.

4.3.3 Effect of top reinforcement ratios

Top reinforcement ratio in the beam of sub-assembly EMJ-B-0.88/0.59 was reduced from 1.19% to 0.88% in comparison with EMJ-B-1.19/0.59, but reinforcement detailing in the joint and beam bottom reinforcement ratio remained the same, as included in **Table 4.1**. As a result, the CAA capacity of EMJ-B-0.88/0.59 was reduced to 83.7 kN, around 8% lower than that of EMJ-B-1.19/0.59, as shown in **Table 4.5**. Likewise, the maximum horizontal compression force was reduced by 14% to 262.8 kN at CAA stage. In the descending branch of the vertical load, beam bottom reinforcement at the right face of the middle joint fractured sequentially, leading to reductions of vertical load. At the initial stage of catenary action, premature fracture of beam top reinforcement at the face of the right end column stub substantially reduced the vertical load at 320 mm displacement, as shown in **Fig. 4.4(a)**. Sub-assembly EMJ-B-0.88/0.59 attained its catenary action capacity of 55.2 kN (see **Table 4.5**), only 50% of that of EMJ-B-1.19/0.59. With increasing middle joint displacement, beam bottom longitudinal reinforcement at the end column stub could sustain a certain level of tension force. Thus, horizontal tension force increased slowly at the catenary action stage, as shown in **Fig. 4.4(b)**, but vertical load could not surpass the catenary action capacity due to a reduction of hogging moment resistance at the beam end section. Therefore, by reducing the top reinforcement ratio in the beam of EMJ-B-0.88/0.59 compared with EMJ-B-1.19/0.59, the former resistance was substantially reduced, in particular, the catenary action capacity.

Similar conclusions are obtained when a comparison is made between sub-assemblages EMJ-L-0.88/0.59 and EMJ-L-1.19/0.59. The top reinforcement ratio of the former was reduced from 1.19% to 0.88% compared to the latter. **Fig. 4.5(a)** shows the vertical load-middle joint displacement curve of EMJ-L-0.88/0.59. At the CAA stage, the maximum vertical load resisted by EMJ-L-0.88/0.59 was 82.5 kN, as included in **Table 4.5**, 9% lower than that of EMJ-L-1.19/0.59. However, the maximum horizontal compression force in EMJ-L-0.88/0.59 was 274.7 kN, around 5% greater in comparison with EMJ-B-0.88/0.59. At the catenary action stage, the capacity of EMJ-L-0.88/0.59 was 26% lower than that of EMJ-L-1.19/0.59. It is noteworthy that prior to fracture of beam top reinforcement at the end column stub, vertical load of EMJ-L-0.88/0.59 decreased gradually with increasing middle joint displacement, as shown in **Fig. 4.5(a)**.

4.3.4 Effect of bottom reinforcement ratios

To study the effect of beam bottom longitudinal reinforcement on the behaviour of beam-column sub-assemblages, bottom reinforcement ratio in the beam of EMJ-L-0.88/0.88 was increased from 0.59% to 0.88% in comparison with EMJ-L-0.88/0.59. **Fig. 4.5(a)** shows the vertical load-middle joint displacement curve of EMJ-L-0.88/0.88. The sub-assemblage exhibited the CAA capacity of 79.2 kN, 4% lower than EMJ-L-1.19/0.59. No significant descending phase was observed following the CAA capacity. When the middle joint displacement was below 100 mm, horizontal tension force was generated in sub-assemblage EMJ-L-0.88/0.88, as shown in **Fig. 4.5(b)**. This unusual behaviour arose from comparatively large connection gaps between the end column stub and the bottom horizontal restraint. The connection gaps also reduced the horizontal compression force in the beam and hindered the full development of CAA capacity at the initial stage. At catenary action stage, EMJ-L-0.88/0.88 was able to sustain the maximum vertical load of 78.7 kN and the peak horizontal tension force of 144.1 kN. Even though only the bottom reinforcement ratio in the beam was increased in EMJ-L-0.88/0.88 compared to EMJ-L-0.88/0.59, both the catenary action capacity and horizontal tension force of the former were substantially greater than those of the latter, as included in **Table 4.5**.

4.4 Crack Patterns and Failure Modes of Sub-Assemblages

Fig. 4.6 shows the failure mode of sub-assembly CMJ-B-1.19/0.59. At CAA stage, beam bottom longitudinal reinforcement fractured at the right face of the middle joint, as shown in **Fig. 4.6(a)**. When catenary action commenced in the bridging beam, a partial hinge started developing at the curtailment point of top reinforcement in the beam (see **Fig. 4.6(b)**) due to discontinuity in longitudinal reinforcement. The partial hinge at the curtailment point significantly enhanced the deformation capacity of sub-assembly CMJ-B-1.19/0.59. Near the end column stub, fan-shaped cracks were formed in the plastic hinge region, as shown in **Fig. 4.6(c)**. Final failure was induced by rupture of beam top reinforcement in the plastic hinge region.

By replacing conventional concrete in the structural topping and the beam-column joint with ECC, different crack patterns and failure modes were observed in EMJ-B-1.19/0.59, as shown in **Fig. 4.7** and **Fig. 4.8**. After cracking had occurred in the ECC topping, PVA fibres could transfer tensile stresses across the cracks through their bridging strength, and ECC worked compatibly with reinforcing bars due to greater ductility and strain-hardening behaviour (Li 2003). **Fig. 4.7** illustrates the crack pattern of beam-column sub-assembly EMJ-B-1.19/0.59 at the initial loading stage. Closely-spaced hairline cracks spread along the ECC topping due to multi-cracking behaviour of ECC until 150 mm displacement. With a further increase in middle joint displacement, a major crack started propagating near the end column stub and the tensile strain capacity of ECC was exhausted at this section. Nonetheless, away from the major crack, crack width was limited to around 0.1 mm, and structural topping was effective in resisting tensile stresses at larger middle joint displacements. Formation of localised cracks eventually led to fracture of beam top longitudinal bars, as shown in **Fig. 4.8(a)**. It is similar to the results of reinforced ECC components subjected to uniaxial tension (Moreno et al. 2014). Special attention has to be paid to the crack pattern at the curtailment point of beam top reinforcement near the end column stub (see **Fig. 4.1**). In spite of multiple cracking, partial hinge was not formed due to limited crack width in the topping, as shown in **Fig. 4.8(b)**. At the middle joint, a similar failure mode to CMJ-B-1.19/0.59 was observed in EMJ-B-1.19/0.59, as shown in **Fig. 4.8(c)**.

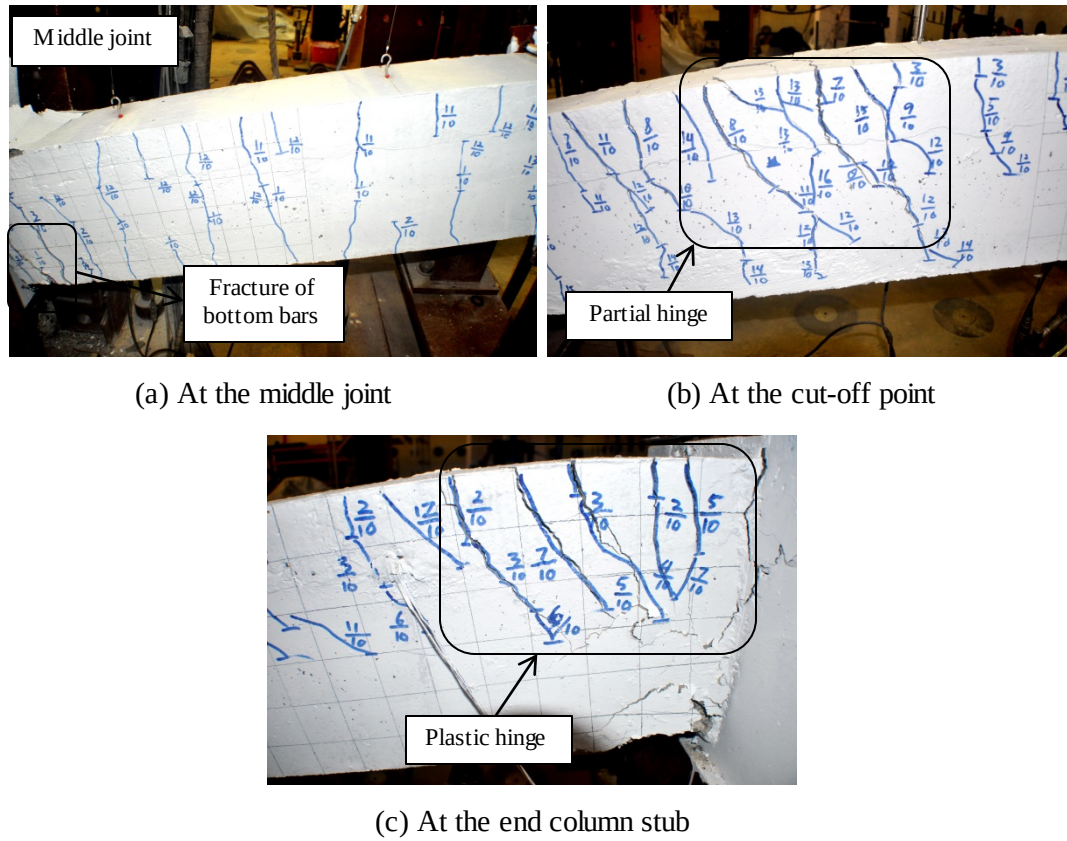


Fig. 4.6: Crack patterns and failure modes of CMJ-B-1.19/0.59

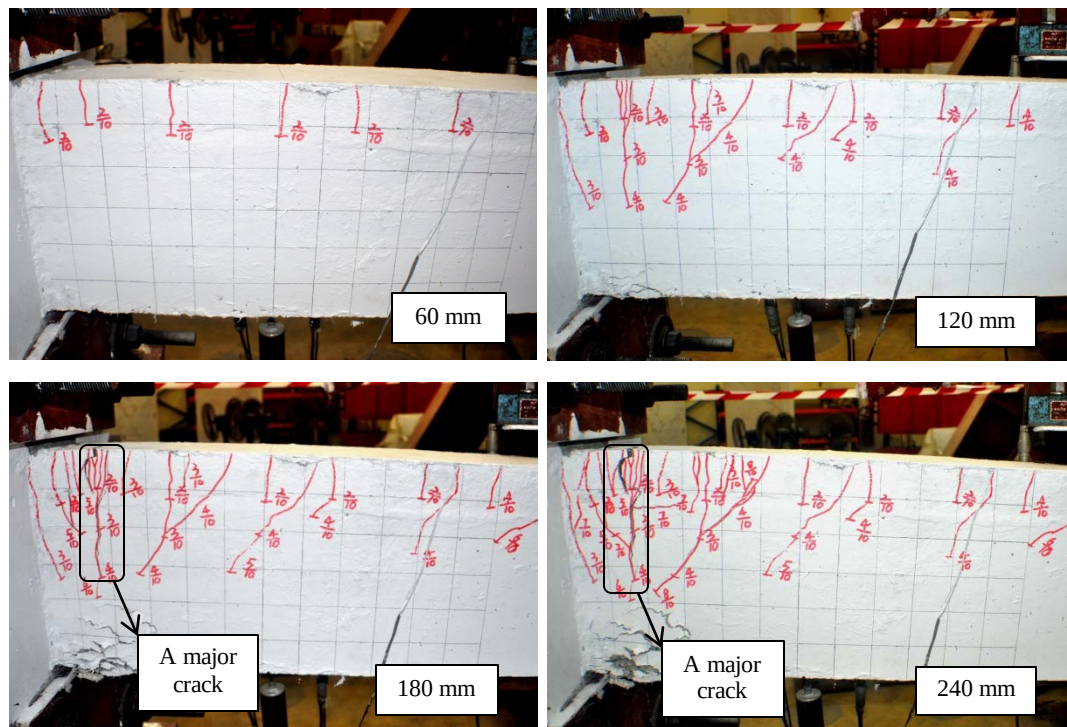


Fig. 4.7: Development of multi-cracking in the structural topping of EMJ-B-1.19/0.59

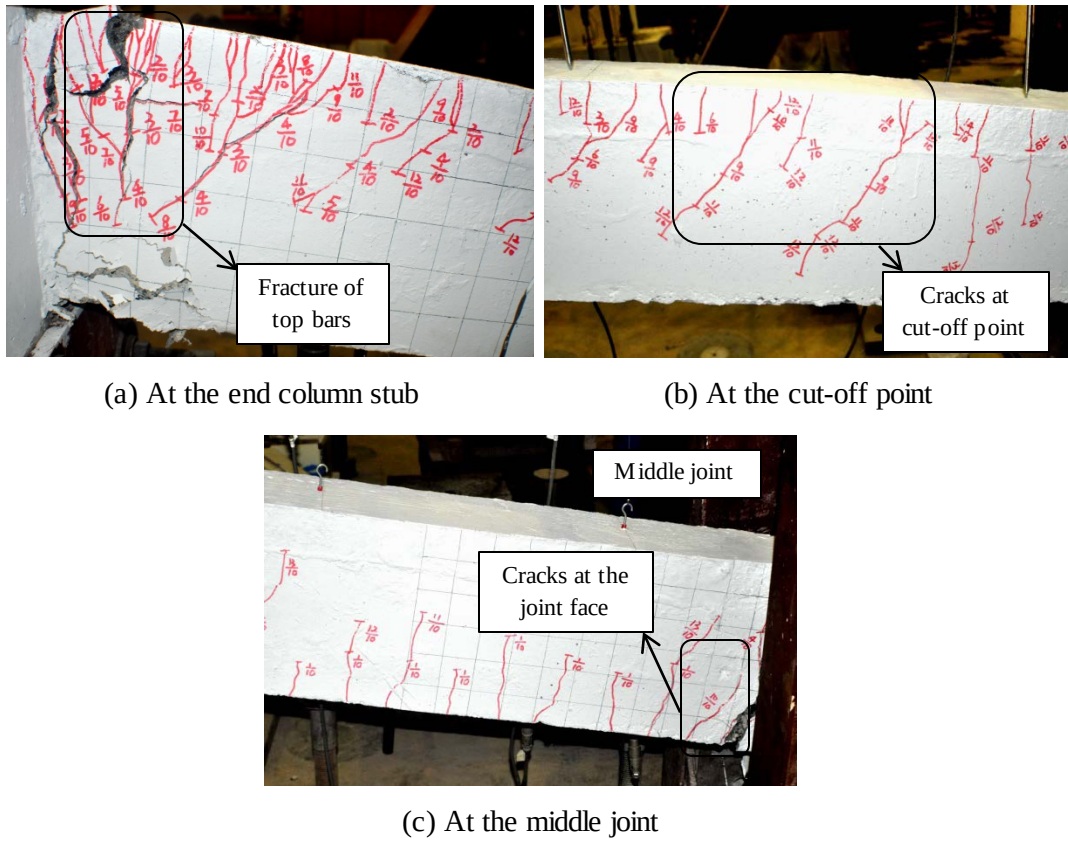


Fig. 4.8: Failure modes of sub-assembly EMJ-B-1.19/0.59

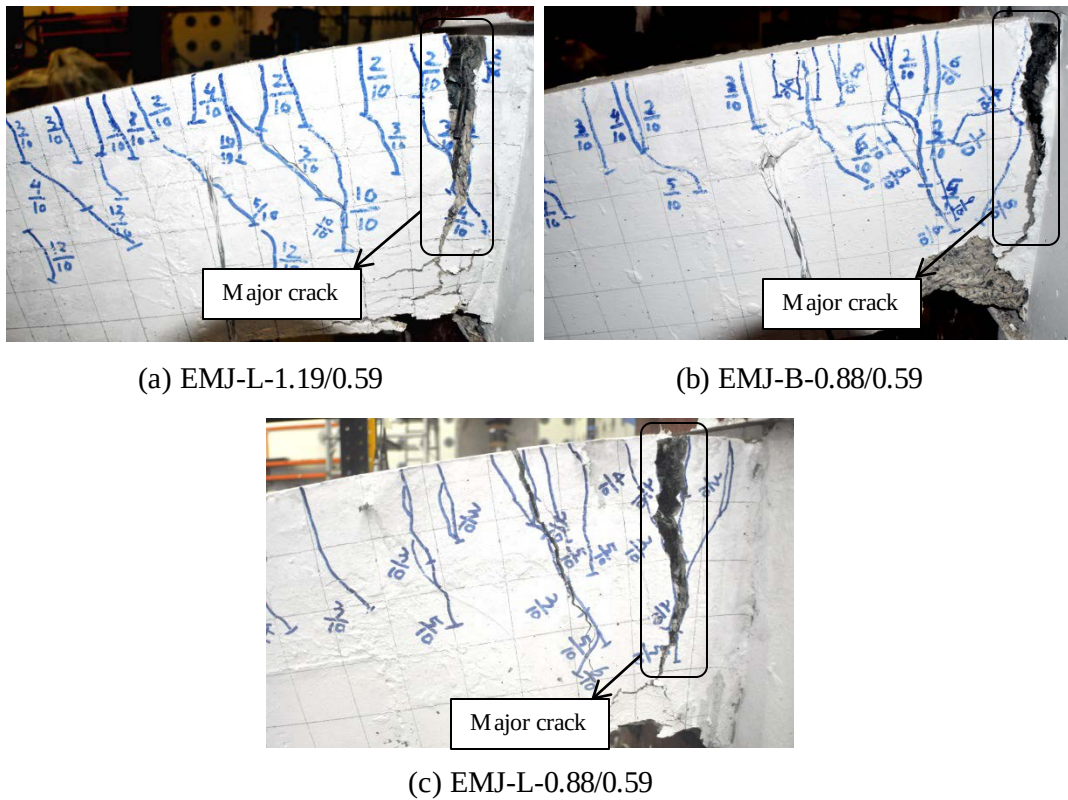


Fig. 4.9: Failure modes at the end column stub of sub-assemblages

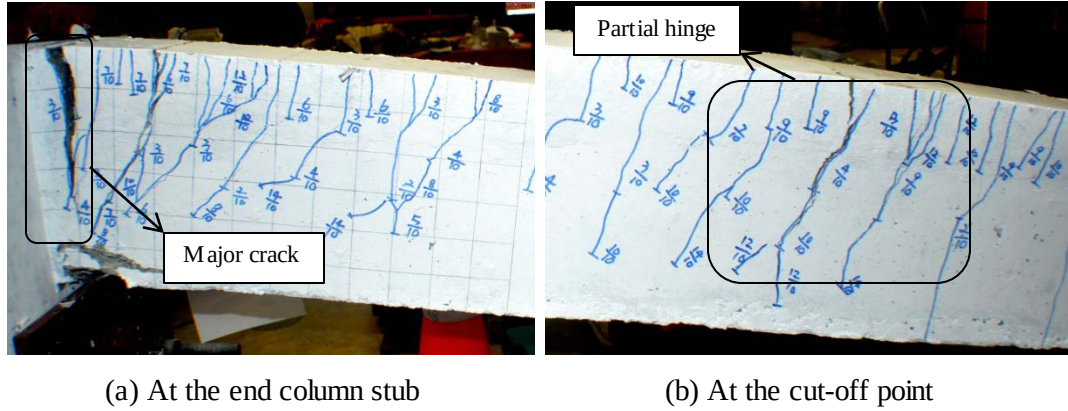


Fig. 4.10: Failure modes of EMJ-L-0.88/0.88

As a result of a greater tensile strain capacity of ECC in tension, a single major crack was also observed at the end column stub of the other four ECC sub-assemblages, as shown in **Figs. 4.9(a-c)** and **Fig. 4.10(a)**. Strain localisation of longitudinal reinforcement at the crack plane eventually caused premature fracture of beam top reinforcement. Only EMJ-L-0.88/0.88 exhibited substantial cracks at the cut-off point of beam top reinforcement, as shown in **Fig. 4.10(b)**, possibly due to connection gaps in the bottom horizontal restraint. As horizontal compression force was significantly reduced by connection gaps, as shown in **Fig. 4.5(b)**, it was more likely to develop flexural cracks at the section where beam top reinforcement was curtailed.

4.5 Horizontal Reaction Forces and Bending Moments

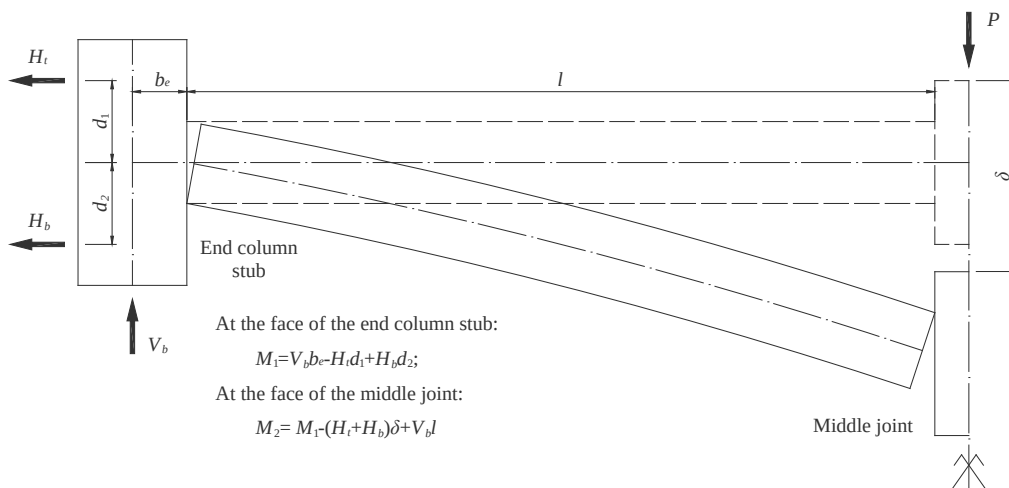


Fig. 4.11: Force equilibrium of deformed sub-assemblage

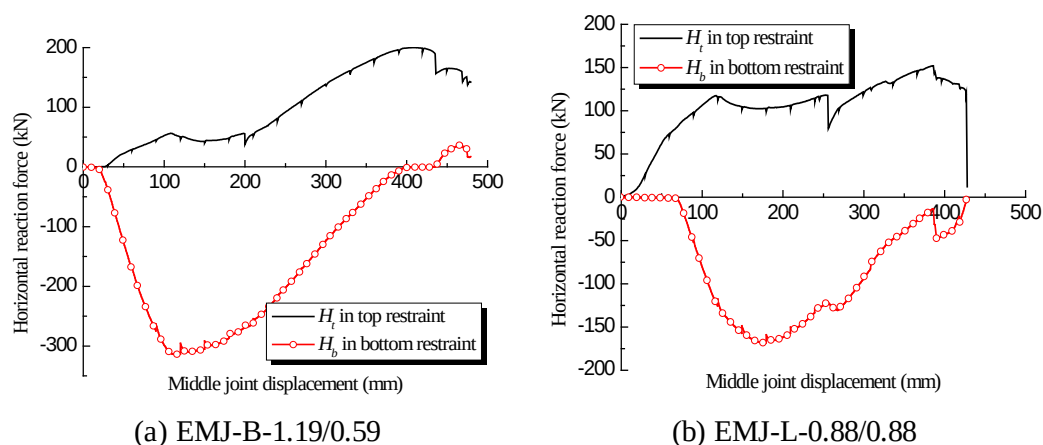


Fig. 4.12: Variations of horizontal reaction forces in sub-assemblages

To investigate the load path of horizontal forces to the support, total horizontal force was decomposed into the reaction forces in the top and bottom restraints, as shown in **Fig. 4.11**. In sub-assemblage EMJ-B-1.19/0.59, the bottom restraint sustained compression force while the top restraint was in tension at CAA stage, as shown in **Fig. 4.12(a)**. The maximum compression force in the bottom restraint was 313.5 kN. Once catenary action commenced, the horizontal tension was primarily transferred to the top restraint. Meanwhile, the reaction force in the bottom restraint was gradually shifted to tension. When the catenary action capacity was attained, the peak tension force in the top restraint was 200 kN, but limited tension force was sustained by the bottom restraint. Eventually, fracture of top reinforcement at the end column stub substantially reduced the tension force in the top restraint. Similar load paths of horizontal forces were also recorded in other sub-assemblages. However, in EMJ-L-0.88/0.88, the presence of connection gaps between the end column stub and the bottom restraint postponed the development of horizontal compression in the bottom restraint, as shown in **Fig. 4.12(b)**. The maximum compression force was only 168.2 kN. Additionally, the tension force in the top restraint developed much earlier in comparison with EMJ-B-1.19/0.59. Therefore, the maximum horizontal compression force at the CAA stage of EMJ-L-0.88/0.88 was significantly smaller than the other sub-assemblages (see **Fig. 4.5(b)**).

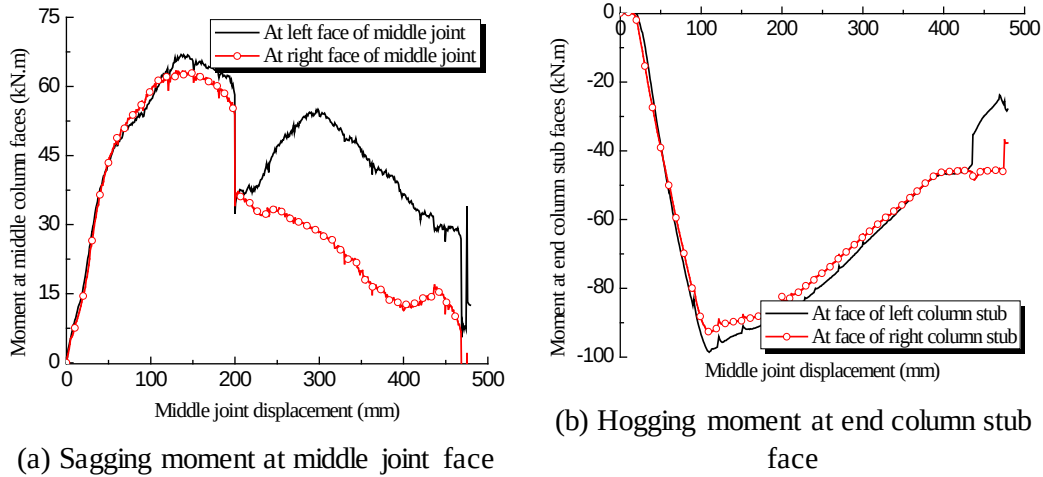


Fig. 4.13: Variations of bending moments in EMJ-B-1.19/0.59

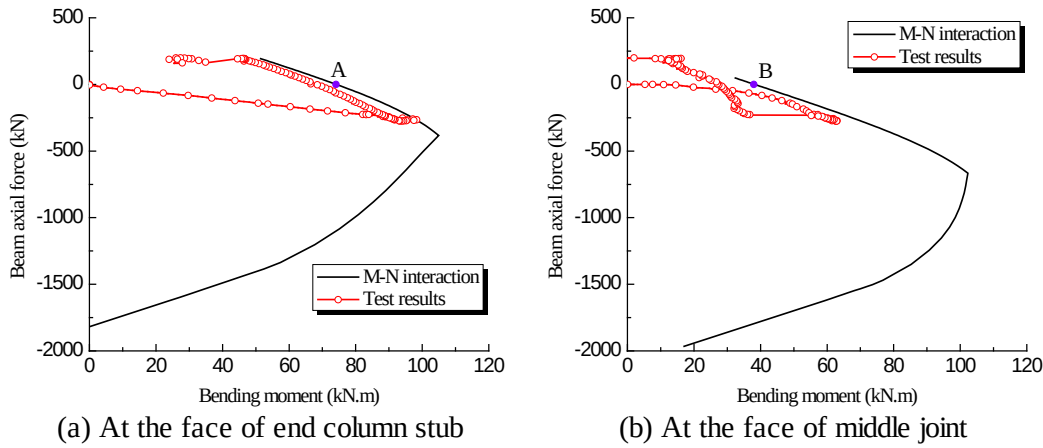


Fig. 4.14: Interaction of bending moment and beam axial force

Bending moments at the faces of the end column stub and middle joint of sub-assembly EMJ-B-1.19/0.59 are calculated based on the force equilibrium of the deformed sub-assemblages (see **Fig. 4.11**). With increasing middle joint displacement, sagging and hogging moments developed at the beam ends and attained their maximum values of 65.1 kN.m and 95.6 kN.m at the CAA stage, as shown in **Figs. 4.13(a and b)**. At about 200 mm middle joint displacement, sagging moments were suddenly reduced as a result of rupture of bottom reinforcement at the left face of the middle joint, as shown in **Fig. 4.13(a)**. Thereafter, rotational restraint on the middle joint allowed for an increase in sagging moment at the right face of the middle joint. Beyond the maximum values, hogging moments near the end column stub decreased gradually until fracture of top reinforcement occurred (see **Fig. 4.13(b)**).

Based on the deformed shape of the beam, axial force in the beam was approximately identical to total horizontal reaction force on each end column stub. Through the axial force-bending moment (N-M) interaction diagram for the beam, correlations between the calculated bending moments at the beam ends and the axial force are established, as shown in **Fig. 4.14**. At the initial stage, hogging moment at the face of the end column stub increased almost linearly with increasing axial compression force in the beam, as shown in **Fig. 4.14(a)**, until the maximum hogging moment was attained. Thereafter, variations of axial compression force and hogging moment followed the theoretical interaction diagram. Similar results were obtained at the face of the middle joint, as shown in **Fig. 4.14(b)**. However, premature fracture of beam bottom reinforcement at the joint interface reduced the sagging moment. Furthermore, comparisons are made between the calculated sagging and hogging moments and the moment capacities of the beam under pure flexure without axial force, as shown in **Fig. 4.14**. At the end column stub, the beam developed a maximum hogging moment of 95.6 kN.m, around 32% greater than the hogging moment capacity of 72.2 kN.m (point A in **Fig. 4.14(a)**) under flexure. Likewise, the calculated sagging moment (65.1 kN.m) was roughly 75% greater than the sagging moment capacity of 37.1 kN.m (point B in **Fig. 4.14(b)**). It indicates that development of horizontal compression force at the CAA stage substantially increased the moment resistances of the bridging beam.

4.6 Deformation Capacities of Beam-Column Sub-Assemblages

In addition to structural resistances of beam-column sub-assemblages subject to column removal scenarios, plastic hinge rotations at the end column stub were also measured through four LVDTs at the beam ends (LE-1 to LE-4 in **Fig. 3.5**). Chord rotation of the beam is calculated as the ratio of the middle joint displacement when the catenary action capacity is attained to the length of the single-span beam, as defined in UFC 4-023-03 (DOD 2013) (see Eq. 4-1). **Table 4.6** includes the plastic hinge rotations and chord rotations of beam-column sub-assemblages. The plastic hinge rotation in a length of 270 mm away from the face of the end column stub was significantly smaller than the chord rotation of each sub-assembly. It implies that the flexural deformation of the bridging beam, in particular, at the curtailment of the

top reinforcement (1000 mm away from the face of the end column stub, as shown in **Fig. 4.1**), contributed a significant portion to the total rotation of the sub-assembly. Thus, in quantifying the deformation capacity of beam-column sub-assemblages, the flexural deformation of the beam has to be considered.

$$\theta_c = \frac{\delta}{l} \quad (4-1)$$

where θ_c is the chord rotation; δ is the vertical displacement of the middle joint when the catenary action capacity is attained; and l is the clear span of the beam.

Table 4.6: Rotation angles of beam-column sub-assemblages

Specimen	Rotation of plastic hinges at end column stub (°)			Chord rotation θ_c (°)
	At A-frame	At reaction wall	Average value θ_p	
CMJ-B-1.19/0.59	7.2	5.7	6.5	9.4
EMJ-B-1.19/0.59	7.7	6.0	6.9	9.0
EMJ-L-1.19/0.59	8.2	7.2	8.7	9.0
EMJ-B-0.88/0.59	4.9	5.3	5.1	6.7
EMJ-L-0.88/0.59	5.7	5.0	5.4	7.1
EMJ-L-0.88/0.88	4.6	5.2	4.9	8.0

With the same top reinforcement ratio in the beam, sub-assemblages CMJ-B-1.19/0.59, EMJ-B-1.19/0.59 and EMJ-L-1.19/0.59 were able to develop nearly the same chord rotations (around 9.0°) under column removal scenarios, as shown in **Table 4.6**. By reducing the beam top reinforcement ratio from 1.19% to 0.88% in sub-assembly EMJ-B-0.88/0.59, the chord rotation was reduced to 6.7°, by around 26% in comparison with EMJ-B-0.88/0.59. A similar reduction in the chord rotation is obtained when a comparison is made between EMJ-L-1.19/0.59 and EMJ-L-0.88/0.59. It indicates that a lower top reinforcement ratio reduced the deformation capacity of sub-assemblages. In EMJ-L-0.88/0.88, only the bottom reinforcement ratio in the beam was increased from 0.59% to 0.88% in comparison with EMJ-L-0.88/0.59. Correspondingly, the chord rotation was increased to 8.0°, by about 13% compared to EMJ-L-0.88/0.59. The increase in the chord rotation was mainly induced by the connection gap in the bottom horizontal restraint. It allowed the end column stub to undergo a rotation of 1.1° and postponed the development of plastic hinge rotations at the beam ends of EMJ-L-0.88/0.88.

4.7 Local Rotations in the Plastic Hinge Region

Precast concrete beam-column sub-assembly CMJ-B-1.19/0.59 developed fan-shaped crack pattern in the plastic hinge region, as shown in **Fig. 4.6(c)**. When ECC was placed in the structural topping and beam-column joint of sub-assemblages, a single major crack occurred in the plastic hinge region near the end column stub at large deformation stage, as shown in **Figs. 4.9(a-c)** and **Fig. 4.10(a)**. Therefore, to quantify the different behaviour of plastic hinges in conventional concrete and ECC sub-assemblages, total rotation of plastic hinges within a length of 270 mm was decomposed into two portions: θ_1 between the end column stub face and section S1 measured by LVDTs LE-1 and LE-2 and θ_2 between sections S1 and S2 measured by LVDTs LE-3 and LE-4 (see **Fig. 3.5**), as expressed in Eqs. (4-2) and (4-3). Ratio γ_θ of θ_1 to $\theta_1 + \theta_2$ is also calculated from Eq. (4-4). If the hogging moment between the end column stub face and section S2 is constant and no localised failure occurs, γ_θ is close to 0.56. When the plastic hinge rotation is fully localised in the beam segment between end column stub face and section S1, γ_θ is equal to 1. Thus, γ_θ can be interpreted as a factor that indicates the degree of localisation of plastic hinge rotation at the beam end. The greater the ratio, the more localised the plastic hinge rotation is at the beam end near the end column stub.

$$\theta_1 = \frac{\delta_{LE-1} - \delta_{LE-2}}{l_{1p}} \quad (4-2)$$

$$\theta_2 = \frac{\delta_{LE-3} - \delta_{LE-4}}{l_{2p}} \quad (4-3)$$

$$\gamma_\theta = \frac{\theta_1}{\theta_1 + \theta_2} \quad (4-4)$$

where θ_1 is the rotation between end column stub face and section S1, measured by LVDTs LE-1 and LE-2, as shown in **Fig. 3.5**; θ_2 is the rotation between sections S1 and S2, measured by LVDTs LE-3 and LE-4; δ_{LE-1} , δ_{LE-2} , δ_{LE-3} and δ_{LE-4} are the readings of LVDTs LE-1, LE-2, LE-3 and LE-4, respectively; l_{1p} and l_{2p} are the distances between end column stub and S1 and between S1 and S2, 150 mm and 120 mm, respectively.

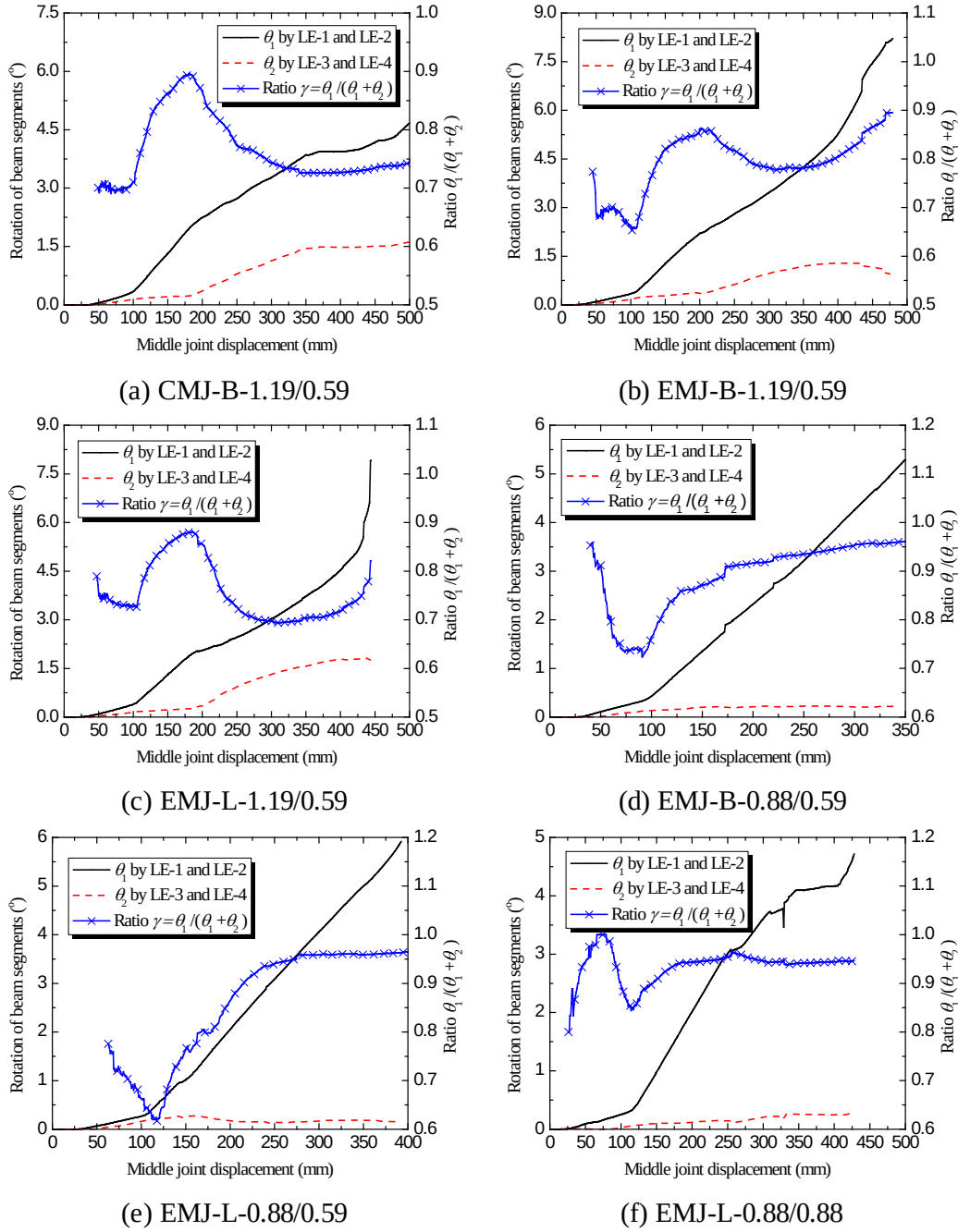


Fig. 4.15: Rotations in plastic hinge regions of sub-assemblages

Fig. 4.15 shows the variations of rotations with middle joint displacement. At the initial stage, rotations measured in the plastic hinge region were zero due to connection gaps between horizontal restraint and end column stub. After around 50 mm middle joint displacement, rotations θ_1 and θ_2 started increasing until beam longitudinal reinforcement ruptured at the face of the end column stub. In sub-assemblage CMJ-B-1.19/0.59, ratio γ_θ increased to its peak value, and then

decreased with increasing middle joint displacement at CAA stage (see **Fig. 4.15(a)**). At catenary action stage, the ratio levelled off at around 0.74 until final failure occurred. The reduction in the ratio was due to penetration of inelastic strain of beam top longitudinal reinforcement into the beam and propagation of fan-shaped cracks in the plastic hinge region, as shown in **Fig. 4.6(c)**. In comparison with CMJ-B-1.19/0.59, a greater ductility of ECC topping in EMJ-B-1.19/0.59 prevented formation of more major cracks in the plastic hinge region, as shown in **Fig. 4.8(a)**. Thus, ratio γ_θ in EMJ-B-1.19/0.59 increased after the onset of catenary action in the bridging beam, as shown in **Fig. 4.15(b)**, indicating localisation of rotation at the plane of major crack. The maximum ratio attained at catenary action stage was 0.90. By reducing the top reinforcement ratio in the beam from 1.19% to 0.88% in EMJ-B-0.88/0.59, more severe localisation of rotation occurred at the plane of the single major crack, as shown in **Fig. 4.9(b)**. Total rotation of plastic hinge was largely contributed by θ_1 , whereas only limited θ_2 was measured between sections S1 and S2, as shown in **Fig. 4.15(d)**. After 100 mm middle joint displacement, γ_θ kept increasing and its maximum value was 0.96. A similar behaviour of plastic hinges was observed in sub-assemblages EMJ-L-1.19/0.59 and EMJ-L-0.88/0.59, as shown in **Figs. 4.15(c and e)**. Thus, application of ECC to structural topping resulted in localisation of rotation at the face of end column stub. A reduction in the beam top reinforcement ratio further led to localisation of rotation near the end column stub. Moreover, a comparison between sub-assemblages EMJ-L-0.88/0.59 and EMJ-L-0.88/0.88 indicates that almost the same ratio γ_θ was obtained in EMJ-L-0.88/0.59 and EMJ-L-0.88/0.88 when beam top reinforcement ratio remained unchanged, as shown in **Figs. 4.15(e and f)**.

4.8 Interactions between Steel Reinforcement and ECC

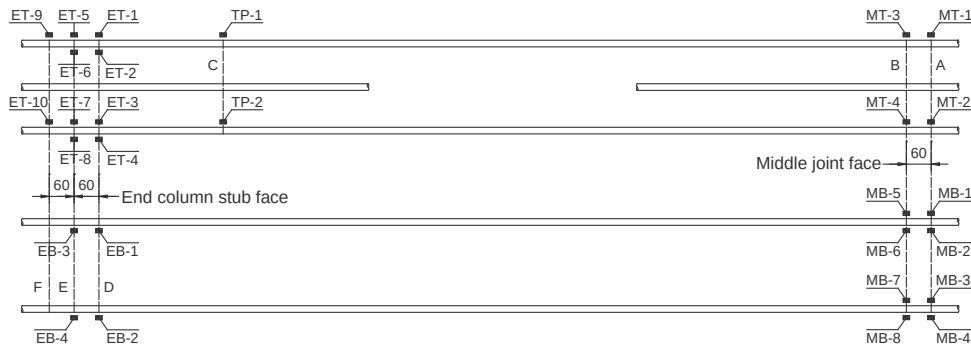


Fig. 4.16: Layout of strain gauges on beam longitudinal reinforcement

To investigate the interactions between ECC and steel reinforcement, strain gauges were mounted onto the top and bottom faces of reinforcing bars at specific sections, as shown in **Fig. 4.16**. Since possible dowel action could generate local bending in the reinforcing bars (Soltani and Maekawa 2008), strain gauges at the two faces of reinforcement provided different readings. **Fig. 4.17** shows the variations of steel strains close to the extreme tension fibre of beam sections. In the middle joint, steel strains attained the maximum value at the joint interface and decreased along the embedment length of reinforcement (see **Fig. 4.17(a)**). At the joint interface (section B), strain gauges MB-6 and MB-8 achieved the yield strain of steel bars at displacement below 40 mm, and developed the post-yield behaviour until rupture of bottom reinforcement occurred, as shown in **Fig. 4.17(a)**. However, at the section 60 mm into the middle joint (section A), steel strains MB-2 and MB-4 experienced a significant plateau stage after the yield strain, and henceforth, attained the peak strain prior to rupture of bottom bars (see **Fig. 4.17(a)**). Although the strains at the middle joint face and inside the joint were significantly different from each other at post-yield stage, difference between steel stresses was limited. As for top reinforcement at the end column stub, a similar variation of steel strains was recorded, as shown in **Fig. 4.17(b)**. Even though strain gauge ET-5 was located in the middle of ET-1 and ET-9, the reading of ET-5 was closer to that of ET-1 than ET-9, possibly due to formation of concrete cone near the column stub face which substantially reduced the bond stress in the region.

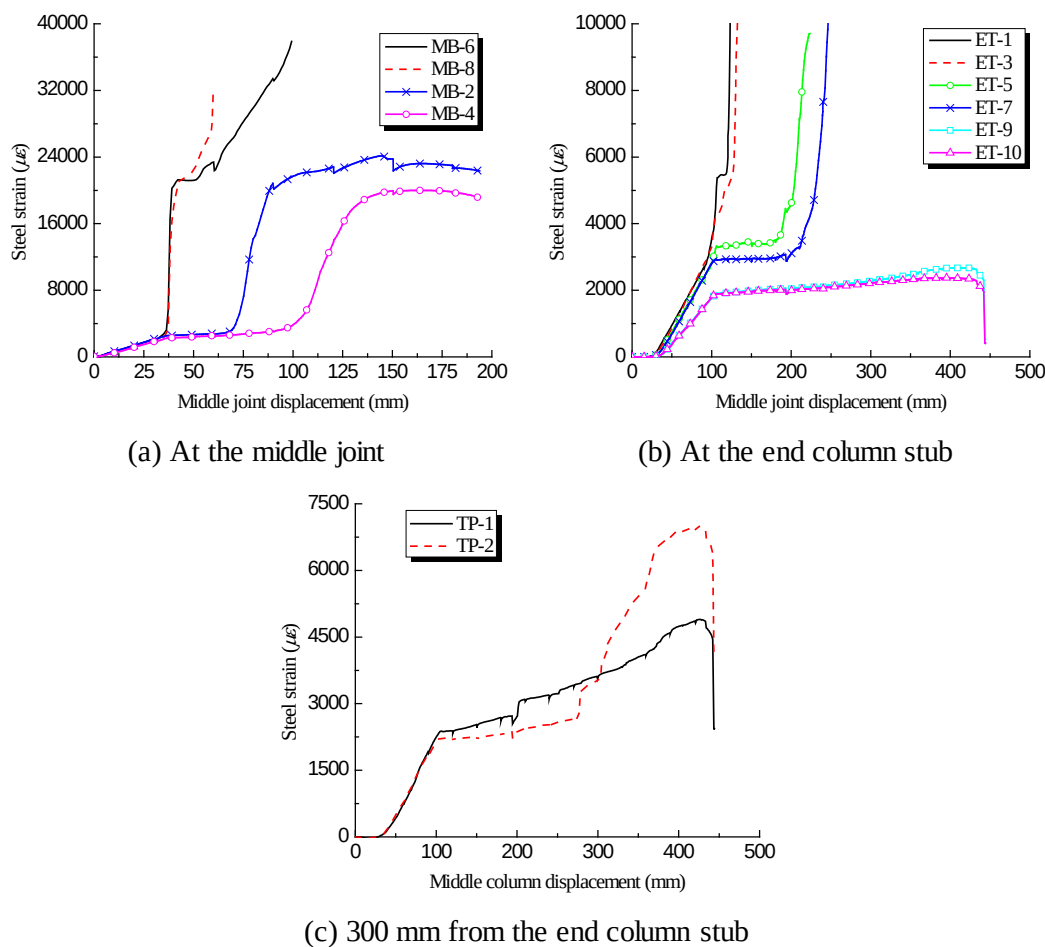


Fig. 4.17: Variations of steel strains in EMJ-L-1.19/0.59

Strain gauges TP-1 and TP-2 were used to measure the strains at a section 300 mm away from the end column stub face (section C), as shown in **Fig. 4.16**. **Fig. 4.17(c)** shows the readings of TP-1 and TP-2. Similar to conventional concrete sub-assemblages discussed in Section 3.4.7, steel strains levelled off with values less than the yield strain of reinforcement after the formation of a major crack at CAA stage. Nevertheless, the strains increased slowly even after the onset of catenary action in EMJ-L-1.19/0.59, indicating that at section C, 300 mm away from the end column stub face, ECC remained at its multi-cracking stage and was effective in sustaining tensile stresses. When final failure occurred, the maximum tensile strain at section C was around 0.7%, as shown in **Fig. 4.17(c)**, much smaller than the calculated tensile strain capacity of ECC in **Table 4.3**.

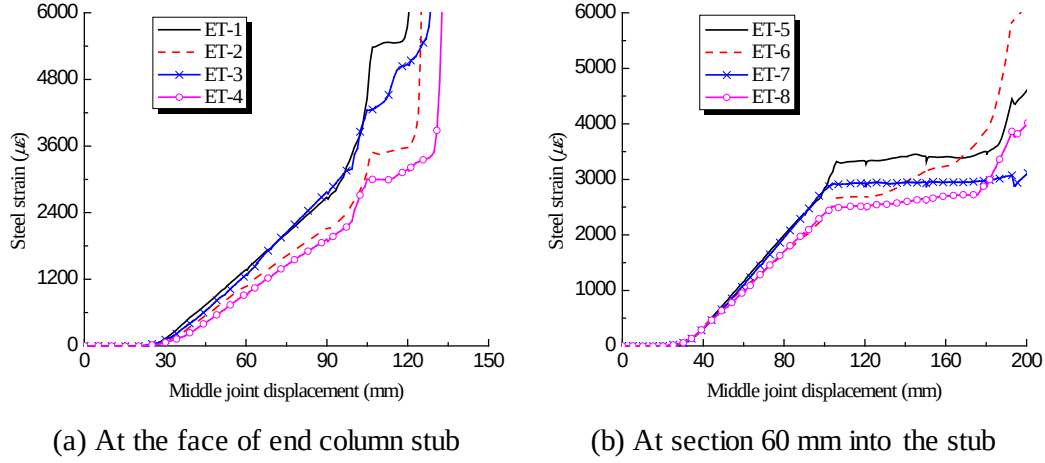


Fig. 4.18: Strains of reinforcement H16 at the end column stub of EMJ-L-1.19/0.59

To study the effect of local bending on steel strains, comparisons are made between strain gauge readings at the top and bottom faces of reinforcing bars, as shown in **Fig. 4.18**. Typically, strain gauges close to the tension face of the beam provided larger tensile strains. At the end column stub, steel strains on the top face was larger than that on the bottom face (**Fig. 4.18(a)**); on the contrary, steel strains on the bottom face of reinforcement were larger at the middle joint (**Fig. 4.18(b)**). In comparison with the end column stub face (section D), difference between strain gauge readings at the top and bottom faces of steel reinforcement was less significant at section E, 60 mm away from section D, as shown in **Fig. 4.18(b)**, due to better confinement provided by surrounding concrete.

$$\kappa_{RC-1/2} = \frac{\varepsilon_{ET-1} - \varepsilon_{ET-2}}{d} \quad (4-5)$$

where ε_{ET-1} and ε_{ET-2} are the readings of strain gauges ET-1 and ET-2, respectively, as shown in **Fig. 4.16**; d is the diameter of longitudinal reinforcement; and $\kappa_{RC-1/2}$ is the curvature of beam top reinforcement at section D.

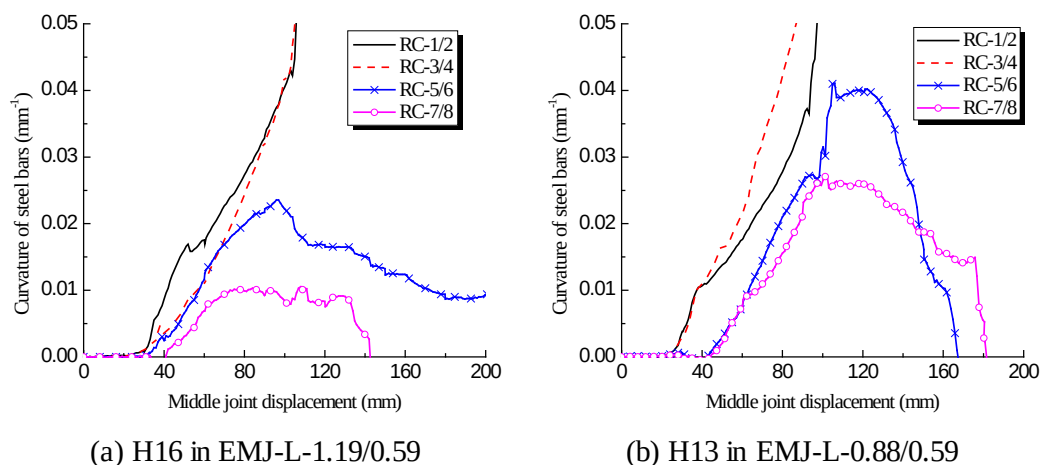


Fig. 4.19: Curvatures of steel bars along the embedment length

The effect of local bending on reinforcement strains can also be demonstrated by the curvature of reinforcement at specific sections, as calculated from Eq. (4-5). **Fig. 4.19** shows the curvatures of rebars at sections D and E for H16 and H13. The notation RC-1/2 represents the curvature of rebar calculated from the readings of strain gauges ET-1 and ET-2. Generated by bending moment at the end column stub, the rebar curvature attained the maximum value at section D and decreased along the embedment length into the end column stub. Thus, it is different from dowel action of steel bars subjected to shear and tension (Soltani and Maekawa 2008), in which the rebar curvature is zero at the crack plane. With increasing middle joint displacement, the curvature of reinforcement at section D kept increasing, whereas the curvature at section E started decreasing after attaining the maximum value, as shown in **Figs. 4.19(a and b)**. The reduction of rebar curvature at section E was due to the increase in strains at the bottom face of steel bars, as shown in **Fig. 4.18(b)**. At middle joint displacements larger than 120 mm, steel strains of ET-6 and ET-8 started surpassing those of ET-5 and ET-7, leading to the reduced curvature at section E. Fundamentally, the variation in curvature might result from inelastic behaviour and local crushing of the supporting concrete under the rebars (Soltani and Maekawa 2008). Comparisons can also be made between the curvatures of reinforcing bars H16 and H13, as shown in **Figs. 4.19(a and b)**. As for reinforcing bars H16 and H13, curvatures at section D were close to each other at the same middle joint displacement. However, curvatures of H13 at section E (60 mm into the end column stub) was substantially larger compared to H16. It indicates that by increasing the diameter of

steel reinforcement, the effect of local bending on the curvature of reinforcement became insignificant along the embedment length of reinforcement when the same middle joint displacement was attained.

4.9 Conclusions

This paper presents the experimental study on six precast beam-column sub-assemblages under column removal scenarios. Conventional concrete and ductile ECC were used in the sub-assemblages. Besides, the effects of reinforcement detailing in the joint and beam longitudinal reinforcement ratios were also investigated in the experimental programme. The following conclusions are drawn based on the test results.

- (1) Significant CAA developed in the bridging beam of sub-assemblages subject to column loss. Horizontal compression force in the beam increased the moment resistance of beam end sections at the CAA stage. However, only CMJ-B-1.19/0.59 and EMJ-B-1.19/0.59 mobilised higher catenary action than CAA to resist progressive collapse.
- (2) Compared to concrete sub-assemblage CMJ-B-1.19/0.59, nearly the same CAA and catenary action capacities were obtained for EMJ-B-1.19/0.59 with ECC in structural topping and the beam-column joints. Thus, ECC did not significantly increase the resistance of EMJ-B-1.19/0.59 under column removal scenarios.
- (3) Approximately the same CAA capacities were obtained for beam-column sub-assemblages with 90° bend and lap-splice of beam bottom reinforcement in the middle beam-column joint (i.e. EMJ-B-1.19/0.59 versus EMJ-L-1.19/0.59, EMJ-B-0.88/0.59 versus EMJ-L-0.88/0.59). However, sub-assemblages EMJ-L-1.19/0.59 and EMJ-L-0.88/0.59 with lap-spliced bottom reinforcement developed greater horizontal compression forces at the CAA stage. At the catenary action stage, the effect of reinforcement detailing on the capacity of sub-assemblages changed with the top reinforcement ratio in the bridging beam.
- (4) With lower top reinforcement ratios in the beam, the CAA capacities of EMJ-B-0.88/0.59 and EMJ-L-0.88/0.59 were reduced by 8% and 9%, respectively, compared to EMJ-B-1.19/0.59 and EMJ-L-1.19/0.59. Moreover, the catenary action capacity of

the sub-assemblages was substantially reduced due to premature fracture of beam top reinforcement near the end column stub.

(5) Interactions between steel reinforcement and ECC were observed with increasing middle joint displacement. At the initial stage, closely-spaced hairline cracks with limited crack width developed along the ECC topping of beam-column sub-assemblages. Beam longitudinal reinforcing bars and ECC sustained tensile stresses and deformed in a compatible manner. Beyond the tensile strain capacity of ECC, a single major crack was formed in the plastic hinge region near the end column stub, which resulted in premature fracture of top reinforcement and hindered the full development of catenary action in the sub-assemblages.

(6) Development of catenary action in the bridging beam imposes a greater demand on the rotation capacity of plastic hinge. However, the formation of a single major crack caused localised rotation of beam-column sub-assemblages in a limited region, especially when the top reinforcement ratio in the beam was relatively low. Besides, a higher toughness of ECC in tension prevented development of flexural deformation in the beam, thereby reducing the deformation capacity of sub-assemblages with ECC topping. Therefore, practical applications of ECC for missing column scenarios are rather limited.

Challenges still exist in the interactions of reinforcement and ECC. Bond stress of steel reinforcement embedded in ECC has not yet been quantified after the formation of a major crack in the plastic hinge region. Thus, pull-out tests on steel bars with short and long embedment lengths are necessary to determine the bond stresses at the elastic and post-yield stages of reinforcing bars. Besides, ECC sustains tensile stresses compatibly with steel reinforcement and exhibits multi-cracking behaviour at relatively small displacement. At the multi-cracking stage, the compatible deformation between ECC and reinforcement may be different from that of embedded reinforcement subjected to pull-out force. Therefore, reinforced ECC members under uniaxial tension need to be tested as well to investigate the bond stress between ECC and reinforcement before the tensile strain capacity of ECC is exhausted.

CHAPTER 5 EXPERIMENTAL STUDY ON PRECAST CONCRETE FRAMES WITH DIFFERENT HORIZONTAL RESTRAINTS

5.1 Introduction

In beam-column sub-assembly tests, the bridging double-span beam over the column removal was extracted from a precast concrete structure and tested under quasi-static loads with enlarged column stubs erected at the two ends. Only rigid horizontal restraints were utilised to arrest the horizontal movement and rotation of the end column stubs. Nonetheless, development of compressive arch action (CAA) and catenary action highly relies on boundary conditions (Park and Gamble 2000; Yu and Tan 2013a). Rigid restraints may lead to significant overestimation of structural resistance of beam-column sub-assemblages. Furthermore, axial compression and tension forces in the bridging beam subjected to CAA and catenary action possibly induce flexural and shear failure to adjacent columns (Choi and Kim 2011; Yi et al. 2008; Yu 2012), which in turn hinders the full mobilisation of CAA and catenary action. Therefore, experimental tests are needed to evaluate the behaviour of precast concrete frames under column removal, in which side columns are designed and erected adjacent to the bridging beam.

This chapter describes experimental tests on four precast concrete frames, in which different horizontal restraints connected to side columns and reinforcement detailing in the beam-column joint were taken into consideration. Structural resistances and deformation capacities of the precast concrete frames were determined in the experimental tests. Dominant factors that affect the behaviour of precast columns and beam-column joints under column removal scenarios were studied. Besides, load paths of horizontal reaction force to the support were analysed at CAA and catenary action stages. Special attention was paid to the behaviour of side columns subjected to CAA and catenary action.

5.2 Test Programme

In accordance with the boundary condition of planar frames, interior and exterior frames are categorised in a building structure. In the interior frame, beams from the adjacent span are connected to side columns, thereby providing a certain level of horizontal restraints against lateral deflections of the columns. By contrast, side columns in the exterior frame are free of horizontal restraints at the beam level. Different behaviour of the side columns in the interior and exterior frames is expected when subjected to CAA and catenary action in the beam. In the experimental programme, both the interior and exterior precast concrete frames were tested so as to investigate the influence of boundary conditions on the frame behaviour.

5.2.1 Frame design and detailing

A prototype precast concrete structure was designed for gravity loads in accordance with Eurocode 2 (BSI 2004), and was scaled down to one-half model, as presented in Section 3.2.1. Precast concrete frames, with 300 mm deep by 150 mm wide beams and 250 mm square columns, were extracted from the perimeter of the structure. The clear span of each beam was 2.75 m, and the column height was 2.35 m. Hogging moment and shear resistances of beam sections were calculated accordingly. As precast beam units were prefabricated ahead of cast-in-situ structural topping and beam-column joints, a horizontal interface existed between the precast units and structural topping. Complying to Eurocode 2 (BSI 2004), the interface was intentionally roughened to 3 mm deep so as to prevent potential delamination across the interface. Closely-spaced stirrups, with 8 mm diameter at 80 mm spacing, were also arranged uniformly along the whole beam length and protruded from the top face of the precast beam units. Top longitudinal reinforcement was enclosed in the stirrups, as specified by Van Acker (2013). As for precast columns, continuous longitudinal reinforcement, confined by stirrups with 8 mm diameter at 100 mm spacing, passed through the beam-column joint and was welded to the steel plate at the end of the column.

Two types of beam bottom reinforcement detailing, namely, 90° bend and lap-splice, were utilised in the beam-column joint region. 90° bend of bottom reinforcement

projected from the end of the beam units (see **Fig. 5.1(a)**) has been widely used in precast concrete structures (FIB 2002). As for lap-spliced bottom reinforcement in the middle joint, similar to the practice recommended by (FIB 2003), a U-shaped trough was cast at each end of the beam unit, and its length depends on the required embedment length of bottom rebars, as shown in **Fig. 5.1(b)**. The inner face of the trough was intentionally roughened to increase the interface shear resistance with cast-in-situ concrete topping. On top of longitudinal bars anchored in the beam-column joint, horizontal hoops were placed in the joint region. The diameter and spacing of horizontal hoops remained identical to those in side columns.

Table 5.1: Details of precast concrete frames

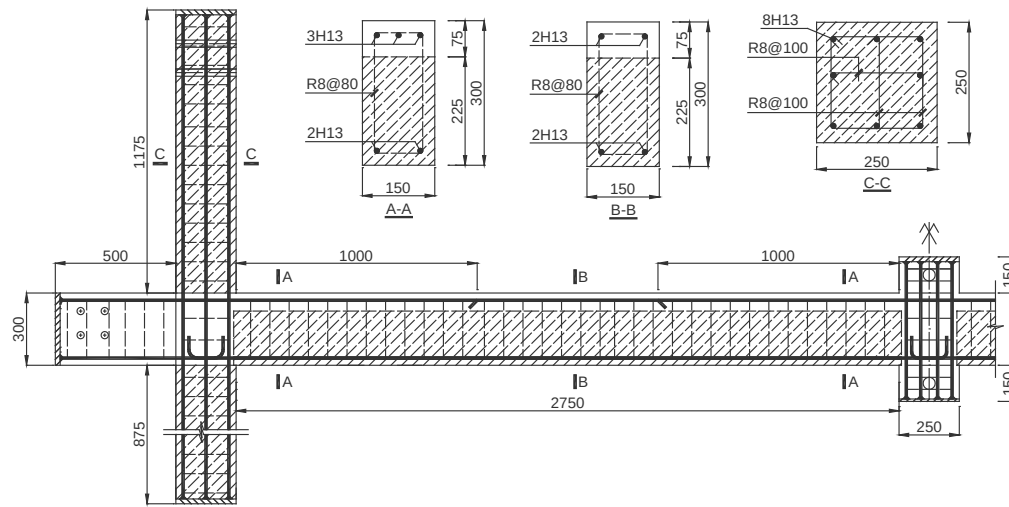
Specimen	Beam				Column		Location
	Joint detailing	TRR*	BRR*	Stirrup	LR [§]	Stirrup	
IF-B-0.88-0.59	90° bend	0.88% (3H13)	0.59% (2H13)	R8@80	1.70% (8H13)	R8@100	Interior
IF-L-0.88-0.59	Lap-splice						Interior
EF-B-0.88-0.59	90° bend						Exterior
EF-L-0.88-0.59	Lap-splice						Exterior

*: TRR and BRR represent respective top and bottom reinforcement ratios in the beam;

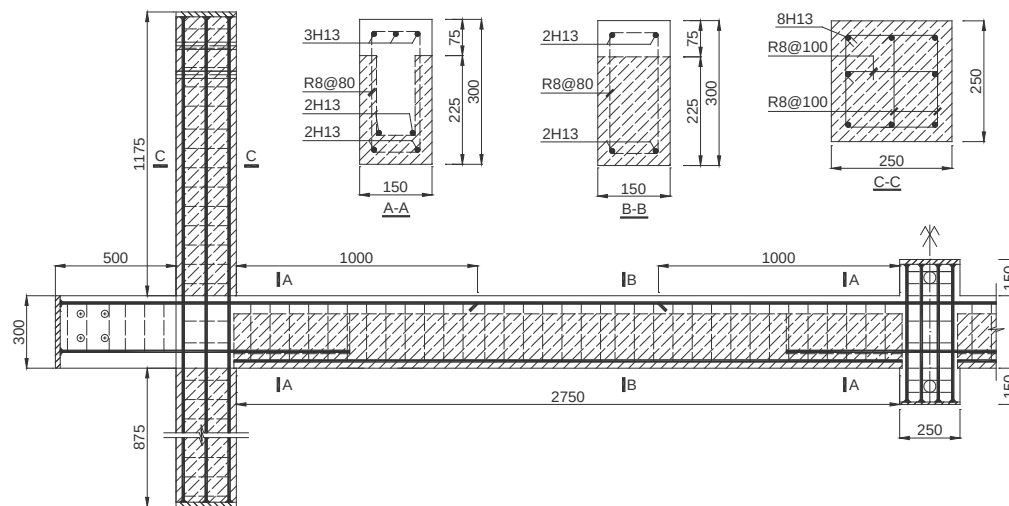
§: LR represents longitudinal reinforcement ratio of the column.

In all the four frames, the cross sections of the double-span beam and precast column remained identical, but reinforcement detailing in the joint and boundary conditions of the columns were changed. **Table 5.1** summarises the joint detailing and the boundary conditions of precast concrete frames. In the notations of specimens, the alphabets “IF” and “EF” denote the respective interior and exterior frames, and “B” and “L” represent the 90° bend and lap-splice of beam bottom reinforcement in the joint. The last two numerals indicate the top and bottom reinforcement ratios of the beam end sections joining the middle column, respectively. **Fig. 5.1** shows the geometry and reinforcement detailing of interior and exterior precast frames. Only half of the frame specimen is shown due to symmetry. In comparison with interior frames IF-B-0.88-0.59 and IF-L-0.88-0.59 (see **Figs. 5.1(a and b)**), the short beam extension protruding beyond the side column was eliminated from exterior frames EF-B-0.88-0.59 and EF-L-0.88-0.59. Both top and bottom reinforcing bars were anchored into the side beam-column joint, but the joint detailing and reinforcement ratio remained identical to those of interior frames, as shown in **Figs. 5.1(c and d)**. It

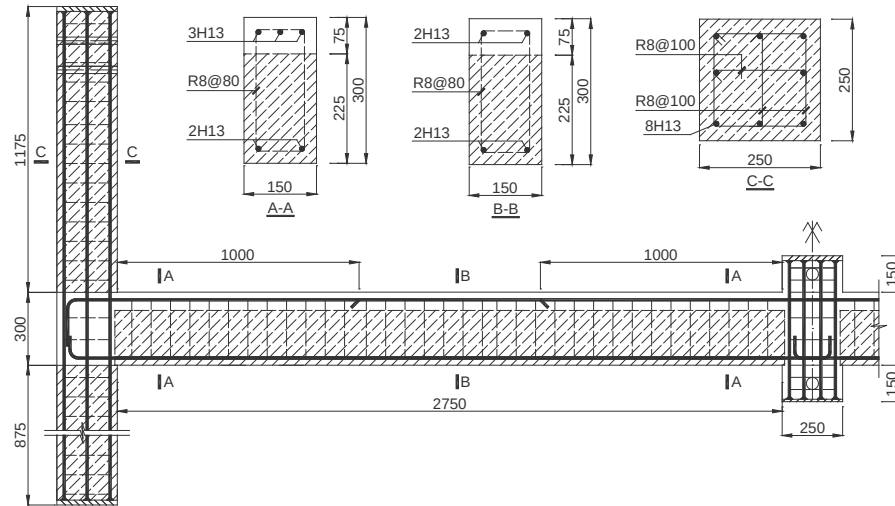
is noteworthy that precast concrete components are hatched to be differentiated from cast-in-situ concrete.



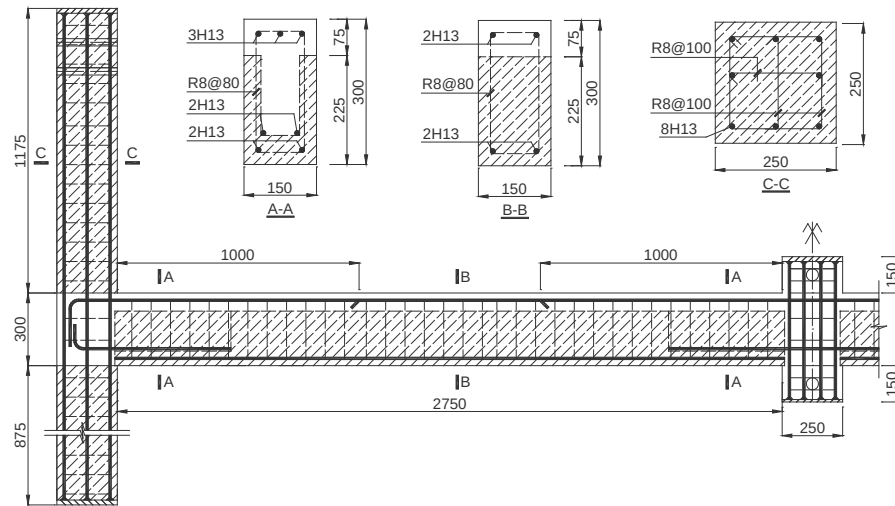
(a) IF-B-0.88-0.59



(b) IF-L-0.88-0.59



(c) EF-B-0.88-0.59



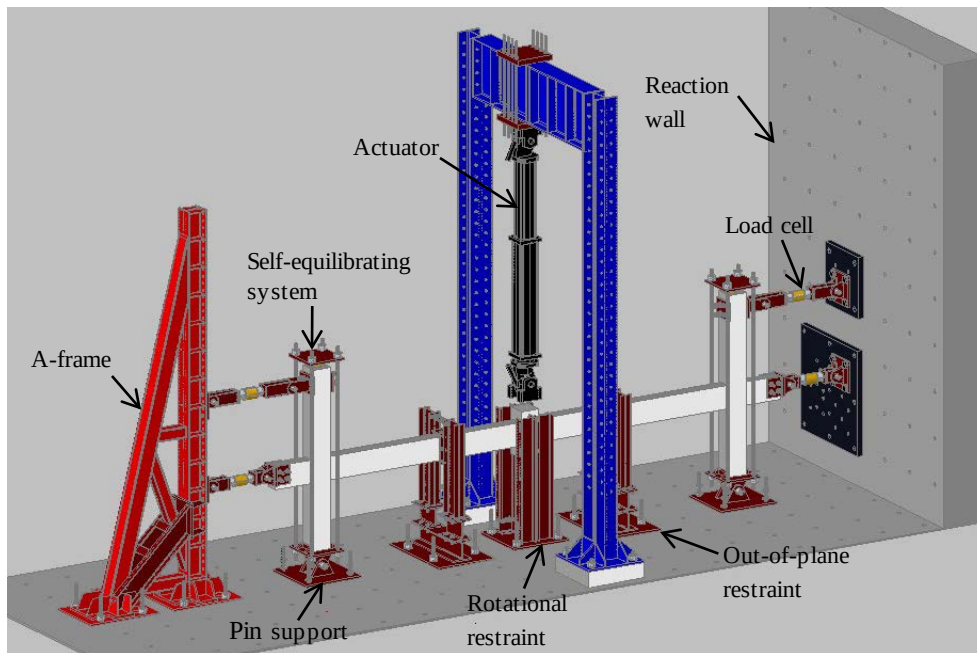
(d) EF-L-0.88-0.59

Fig. 5.1: Geometry and reinforcement detailing of precast concrete frames

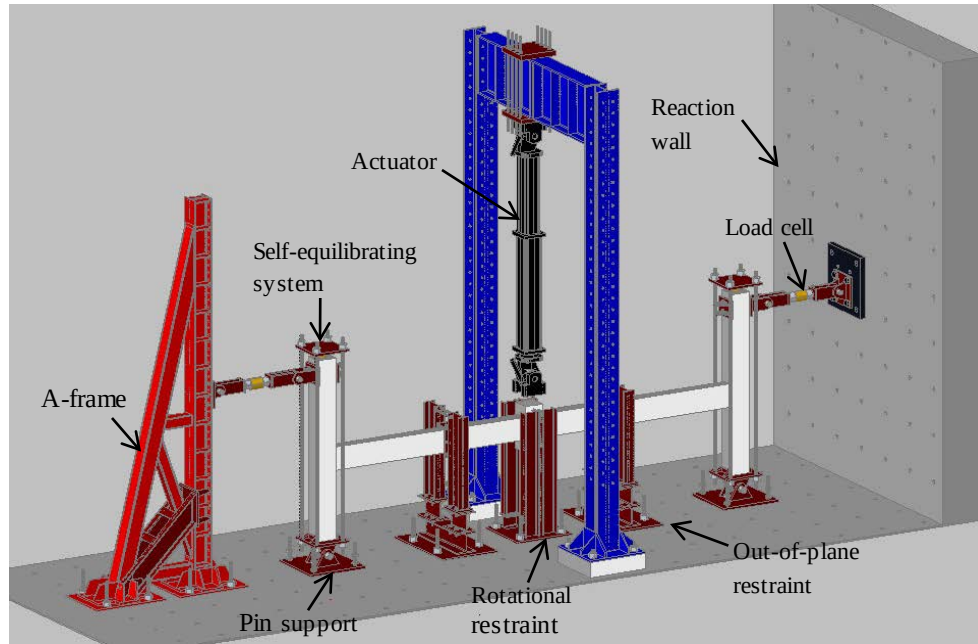
To facilitate installation of horizontal restraints to the beam extension of interior frames IF-B-0.88-0.59 and IF-L-0.88-0.59, four PVC pipes were embedded in the beam extension, as shown in **Figs. 5.1(a and b)**. PVC pipes were also installed on the top of the side columns so as to connect the horizontal restraint. In the middle column, two pipes were placed below and above the joint, as shown in **Figs. 5.1(a-d)**, so that restraints could be provided to prevent the middle joint from rotation after rupture of bottom reinforcement only occurred on one interface of the middle joint.

5.2.2 Test setup

Fig. 5.2(a) shows the test setup for interior precast concrete frames. Similar to the rigs employed by Yu (2012), a horizontal load cell was connected to the precast column, measured 200 mm from its top end, as shown in **Fig. 5.3(a)**. At the bottom, a pin support was designed with a load pin inserted underneath to measure the horizontal reaction force. The distance between the centroid of the load pin and bottom end of the side column was 190 mm, and the effective length of column between the top load cell and bottom pin support was 2.34 m. Another horizontal restraint was applied to the beam extension of interior frames IF-B-0.88-0.59 and IF-L-0.88-0.59, as shown in **Fig. 5.3(b)**. Short steel columns and steel rollers were provided to prevent the out-of-plane deflection of the bridging beams (see **Fig. 5.3(c)**), and a rotational restraint was applied at the middle joint (**Fig. 5.3(d)**) to ensure symmetrical bending if reinforcement fractured on one side only. The same test setup was used for exterior frames EF-B-0.88-0.59 and EF-L-0.88-0.59, but without the beam extensions and associated horizontal load cells, as shown in **Fig. 5.2(b)**.



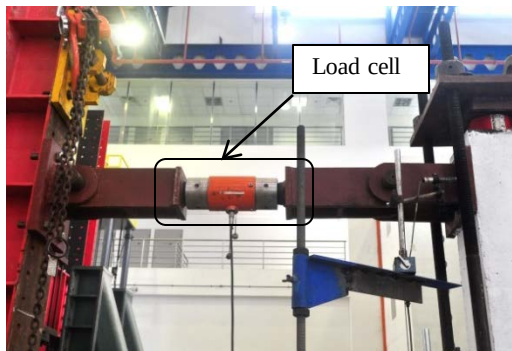
(a) Interior frames



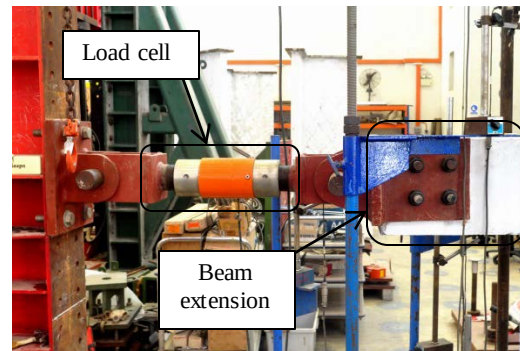
(b) Exterior frames

Fig. 5.2: Test setup for precast concrete frames

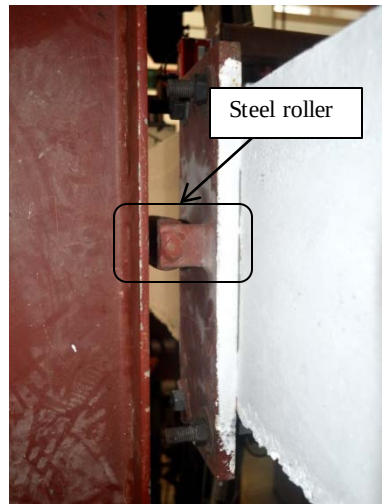
In a realistic building structure subjected to gravity loads, axial compression force exists in the supporting columns, and it affects the behaviour of the columns under progressive collapse scenarios. To simulate the axial compression force, a self-equilibrating system was installed on each side column, through which a hydraulic jack was inserted in between the column and a thick steel plate connected by four steel rods to the bottom pin support, as shown in **Fig. 5.3(e)**. Prior to testing, an axial stress of $0.3f'_c$, where f'_c is the cylinder compressive strength of concrete, was applied to each side column and was kept constant in the course of loading the middle joint.



(a) Horizontal restraint on column top



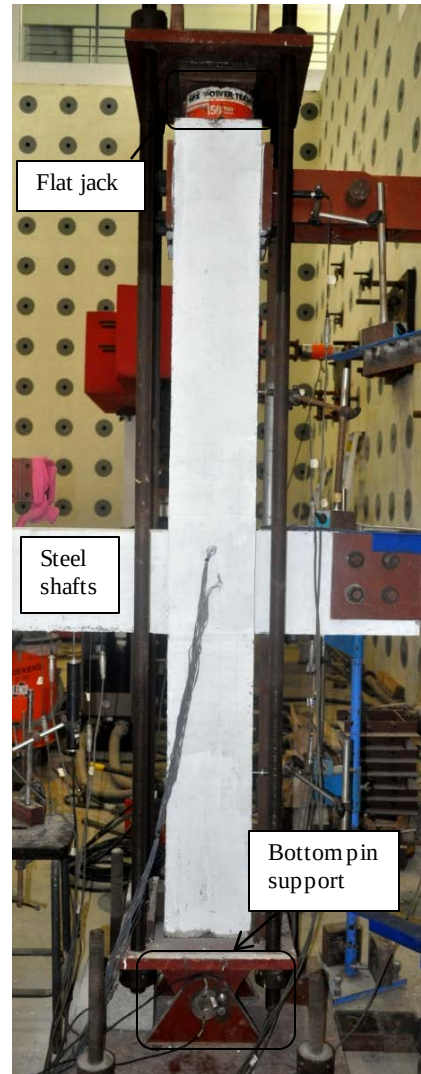
(b) Horizontal restraint on beam extension



(c) Out-of-plane restraint on beam



(d) Rotational restraint in the middle joint



(e) Self-equilibrating system on column

Fig. 5.3: Restraints on precast concrete frames

5.2.3 Instrumentations

In the tests, vertical load and horizontal reaction forces were measured by the corresponding load cells, as shown in **Fig. 5.2**. In addition, deformations of precast concrete frames under column removal scenarios were also measured by means of linear variable differential transducers (LVDTs). **Fig. 5.4** shows the arrangement of LVDTs on precast concrete frames. Similar to the instrumentations in precast concrete beam-column sub-assembly tests as introduced in Section 3.2.4, six vertical LVDTs were installed along the beam length to measure the vertical deflections. Rotations of plastic hinges at the beam ends were also recorded through

four LVDTs in the plastic hinge regions. Under CAA and catenary action, side columns and beam-column joints experienced significant deformations (Yi et al. 2008). Five horizontal LVDTs were installed along the column height to capture the deformed profile of the side columns, and two diagonal LVDTs were mounted in the side beam-column joint to measure the distortion of the joint, as shown in **Fig. 5.4**.

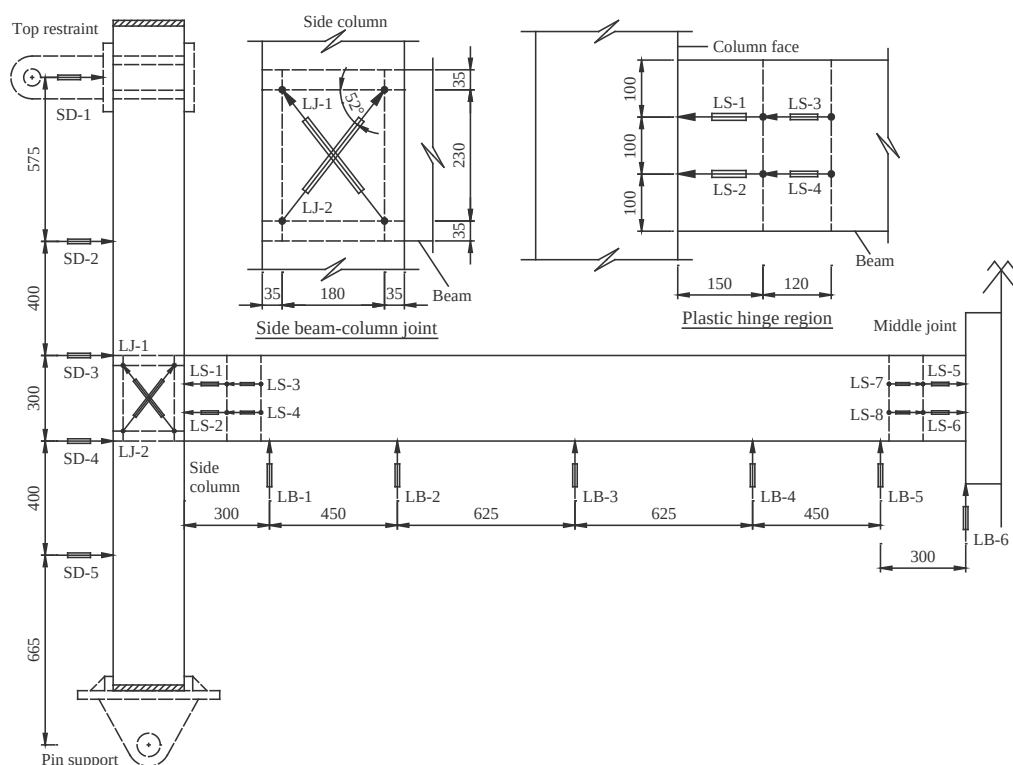


Fig. 5.4: Layout of LVDTs on precast concrete frames

5.3 Material Properties

Hot-rolled deformed steel bars with 13 mm diameter were used as the longitudinal reinforcement in the frames, and mild steel with 8 mm diameter were used as stirrups. For each type of steel reinforcement, tensile tests were conducted on three coupons with 300 mm gauge length to obtain the stress-strain curves, as shown in **Fig. 5.5(a)**. As for concrete materials, three cylinders with 150 mm diameter by 300 mm height were tested. Two aluminium rings at 100 mm spacing were fixed to the middle one-third of each cylinder and three LVDTs were mounted between the rings to measure the average compressive strain in this region. **Fig. 5.5(b)** shows the stress-strain curves of concrete. It is noteworthy that precast beam and column units and the cast-

in-situ concrete (including structural topping and beam-column joints) were cast at different times. **Table 5.2** summarises the material properties of concrete and steel reinforcement.

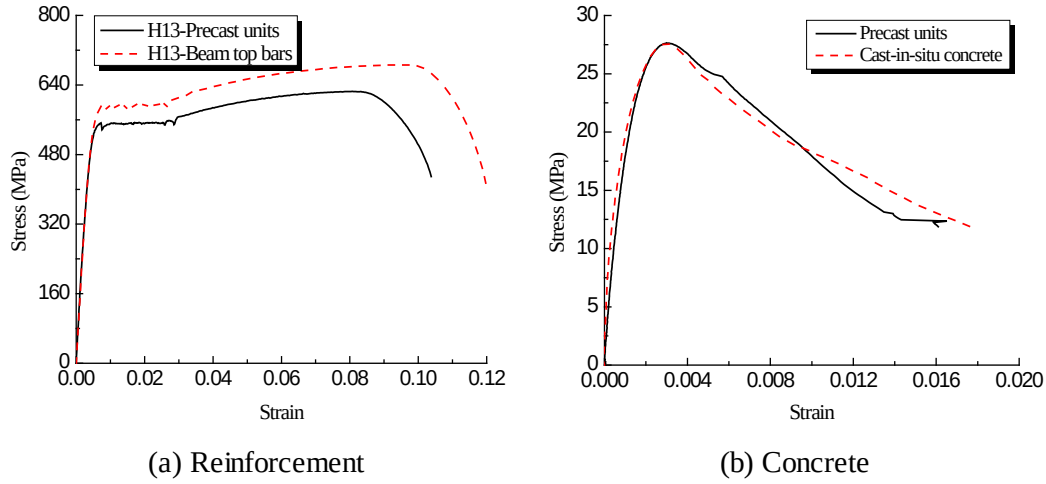


Fig. 5.5: Material stress-strain curves of reinforcement and concrete

Table 5.2: Material properties of concrete and reinforcement

Material		Nominal diameter (mm)	Yield strength (MPa)	Elastic modulus (GPa)	Ultimate strength (MPa)	Fracture strain [#] (%)
Main bars	H13	13	553.2	203.9	630.8	10.8
			593.7*	202.2*	688.4*	12.0*
Stirrups	R8	8	272.4	207.4	359.5	--
Concrete		Compressive strength (MPa)		Modulus of elasticity (GPa)		Splitting tensile strength (MPa)
	Precast units	27.7		22.7		2.0
	Cast-in-situ concrete	26.9		25.8		2.1

*: Yield strength of longitudinal reinforcing bars used in the cast-in-situ concrete topping and beam-column joint of precast frames was 593.7 MPa, whereas yield strength of longitudinal reinforcement in the precast beam and column units was 553.2 MPa.

[#]: Fracture strain refers to the average strain over a gauge length of 300 mm when steel reinforcement ruptures.

5.4 Experimental Results of Precast Concrete Frames

5.4.1 Load-displacement curves

Under displacement-controlled loading condition, vertical load applied onto the middle beam-column joint was recorded by the built-in load cell of the servo-hydraulic actuator. Simultaneously, horizontal reaction forces in the bottom pin

support and horizontal load cells were summed up to calculate the total horizontal reaction force. **Fig. 5.6** and **Fig. 5.7** show the vertical load-middle joint displacement curves and the horizontal reaction force-middle joint displacement curves.

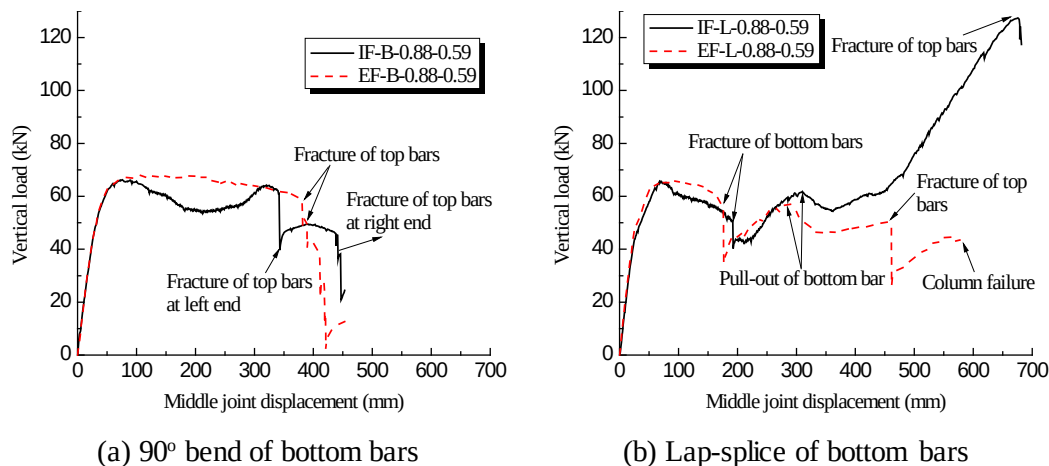


Fig. 5.6: Vertical load-middle joint displacement curves of precast frames

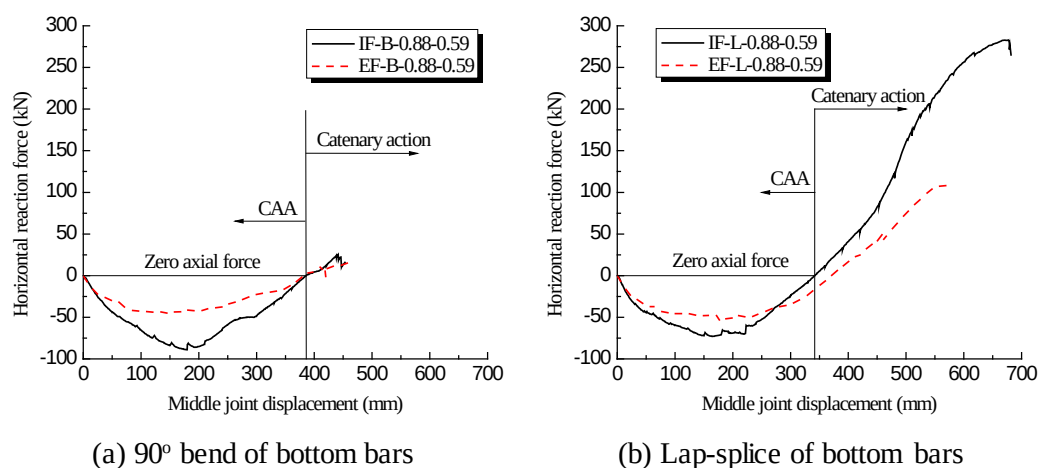


Fig. 5.7: Horizontal reaction force-middle joint displacement curves of precast frames

Under single column removal scenarios, similar CAA developed in the interior and exterior frames, as shown in **Figs. 5.6(a and b)**, when the middle joint displacement was less than 300 mm (one beam depth). With increasing middle joint displacement, precast concrete frames IF-B-0.88-0.59 and EF-B-0.88-0.59 exhibited sequential fracture of beam top longitudinal reinforcement in the vicinity of the side columns, which reduced the hogging moment resistance of beam sections at the side column interface, causing a reduction in the vertical load, as shown in **Fig. 5.6(a)**. Meanwhile, the measured horizontal tension force in the beam was also significantly reduced, as

shown in **Fig. 5.7(a)**. As for exterior frame EF-L-0.88-0.59, beam top bars anchored in the left column ruptured at catenary action stage, resulting in a drop of vertical load, as shown in **Fig. 5.6(b)**. However, tension reinforcement near the right column remained intact until the beam axial tension force attained its maximum value. Eventually, the axial force of beam decreased at a slow rate (see **Fig. 5.7(b)**) due to crushing of concrete in the right column, leading to a gradual decrease of the vertical load as well (**Fig. 5.6(b)**). Among the four precast concrete frames, significant catenary action was only mobilised in interior frame IF-L-0.88-0.59 to redistribute the vertical load through the tensile strength of the bridging beam, as shown in **Fig. 5.6(b)**, with the catenary action capacity of 127.4 kN and the peak tension force of 283.1 kN in the beam.

Table 5.3 summarises the resistances and associated middle joint displacements of precast concrete frames under column removal scenarios. CAA is characterised by compression force in the bridging beam, whereas catenary action commences when the net axial force across a section changes from compression to tension, as defined by Yu and Tan (2010a). Capacities of CAA and catenary action correspond to the maximum vertical loads at CAA and catenary action stages. The maximum horizontal compression force at the CAA stage and tension force at the catenary action stage are also included.

Table 5.3: Resistances and deformations of precast concrete frames

Specimen	Capacity of CAA P_c (kN)	MJD at P_c (mm)*	Max. axial compression (kN)	Capacity of catenary action P_t (kN)	MJD at P_t (mm) *	Peak axial tension (kN)
IF-B-0.88-0.59	66.3	76.1	-89.1	49.5	390.9	25.3
IF-L-0.88-0.59	65.6	69.0	-73.0	127.4	675.8	283.1
EF-B-0.88-0.59	67.9	191.1	-45.0	51.4	383.9	17.1
EF-L-0.88-0.59	65.9	95.0	-53.9	50.3	457.0	108.3

*: MJD represents middle joint displacement.

5.4.2 Effect of reinforcement detailing on frame behaviour

When subject to column removal scenarios, precast concrete frames IF-B-0.88-0.59 and IF-L-0.88-0.59 developed almost the same CAA capacities, as included in **Table 5.3**. Nonetheless, at catenary action stage, IF-L-0.88-0.59 behaved in a different

manner from IF-B-0.88-0.59. IF-L-0.88-0.59 was able to develop 1.57 times greater catenary action capacity compared to IF-B-0.88-0.59 (see **Table 5.3**). The axial tension force developed in the bridging beam of IF-L-0.88-0.59 was ten times greater than that in IF-B-0.88-0.59. Similar results are obtained when a comparison is made between the horizontal tension forces in exterior frames EF-B-0.88-0.59 and EF-L-0.88-0.59.

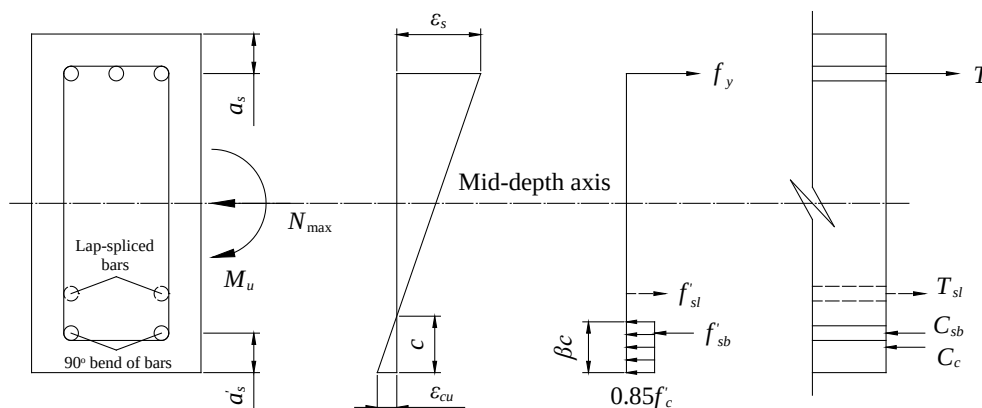


Fig. 5.8: Neutral axis depth of beam sections at the face of the side column

The different behaviour of precast concrete frames with lap-splice and 90° bend of beam bottom reinforcement in the joint was mainly due to the neutral axis depth of the beam sections at the face of the side column which was in hogging moment. To calculate the neutral axis depth, it is assumed that the top reinforcement had attained its yield strength f_y , and the bottom compression fibre had reached its crushing strain ϵ_{cu} , when the maximum horizontal compression forces were obtained at the CAA stage. Based on the plane-section assumption and the force equilibrium, as shown in **Fig. 5.8**, the neutral axis depths for interior frames IF-B-0.88-0.59 and IF-L-0.88-0.59 are calculated as 35 and 39 mm, respectively. In comparison with 90° bend of longitudinal reinforcement, precast beams with lap-spliced bottom reinforcement developed slightly deeper compression zone at the side column face. Correspondingly, the distance between the neutral axis and top face of the beam was smaller in IF-L-0.88-0.59, which delayed fracture of beam top reinforcement at catenary action stage. Moreover, in IF-L-0.88-0.59, the neutral axis depth was smaller than the distance (around 65 mm) between the centroid of bottom reinforcement and beam bottom face, and the lap-spliced bottom reinforcement at the column face

sustained tensile stress at the CAA stage, as shown in **Fig. 5.8**. It was equivalent to additional tension reinforcement in the plastic hinge region. Therefore, the catenary action capacity of IF-L-0.88-0.59 with lap-spliced bottom reinforcement was significantly increased compared with IF-B-0.88-0.59 with 90° bend of bottom reinforcement.

5.4.3 Effect of boundary conditions on frame behaviour

Interior and exterior frames exhibited approximately the same load resistance up to their capacities of CAA, as shown in **Figs. 5.6(a and b)**. Nonetheless, in the descending branch of vertical load, less reduction in the vertical load was observed when comparing exterior frame EF-B-0.88-0.59 with interior frame IF-B-0.88-0.59, as shown in **Fig. 5.6(a)**. The discrepancy in the descending branch of vertical load was attributable to horizontal restraints to the precast concrete frames. With horizontal load cells connected to the beam extensions on both sides of the frame (see **Fig. 5.2(a)**), interior frame IF-B-0.88-0.59 developed larger compression force in the beams than exterior frame EF-B-0.88-0.59, as shown in **Fig. 5.7(a)**. In turn, larger compression force at the CAA stage caused concrete in the compression zone to crush, thereby reducing the moment resistance of beam sections. Consequently, the vertical load on the middle joint of interior frames IF-B-0.88-0.59 were gradually reduced beyond the capacities of CAA, as shown in **Fig. 5.6(a)**.

5.4.4 Pseudo-static resistances of precast concrete frames

To consider dynamic effect under column removal scenarios, the energy balance method proposed by Izzudin et al. (2008) could be used to quantify the pseudo-static resistance of precast concrete frames. In the method, focus is placed on the maximum dynamic response at each load level when the kinetic energy is reduced to zero. Thus, the work done by the external load is equal to the internal energy absorbed by the frames, as expressed in Eq. (5-1). At a vertical displacement u_d , internal energy (i.e. $\int_0^{u_d} P(u)du$) is calculated as the area under the quasi-static load-displacement curve, as shown in **Fig. 5.9(a)**. Correspondingly, the pseudo-static resistance (P_d) at vertical displacement u_d is determined by Eq. (5-1).

$$P_d \cdot u_d = \int_0^{u_d} P(u) du \quad (5-1)$$

Figs. 5.9(a-d) show that the first pseudo-static resistance (point A) was substantially lower compared to the quasi-static CAA capacity of frames. Dynamic increase factor for precast concrete frames, which is defined as the ratio of the CAA capacity to the first peak pseudo-static resistance (point A), fell in the range of 1.10 to 1.23. At catenary action stage, only interior frame IF-L-0.88-0.59 developed significantly greater second peak load (point B) than the first peak load, as shown in **Fig. 5.9(b)**. Development of catenary action in IF-L-0.88-0.59 increased the first pseudo-static resistance by 23.6%.

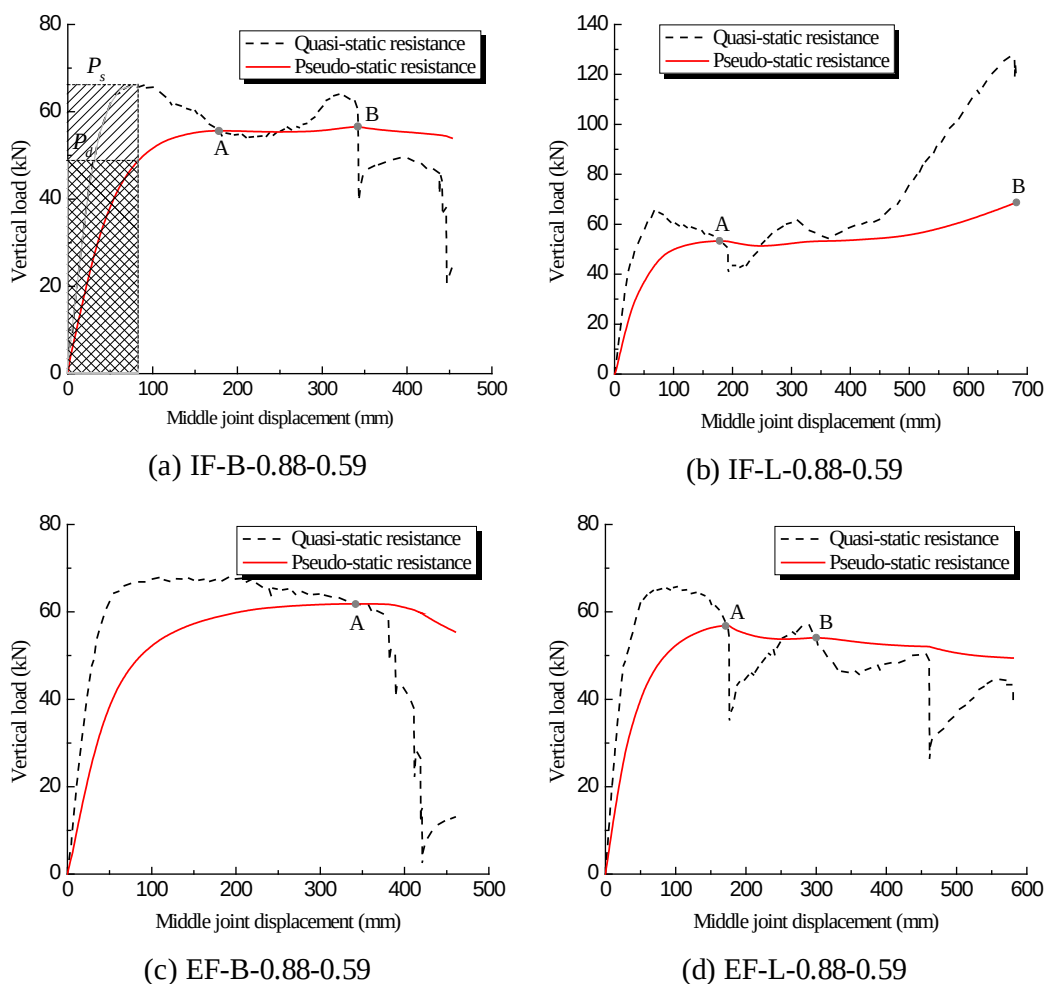


Fig. 5.9: Pseudo-static load-middle joint displacement curves of precast concrete frames

Table 5.4: Pseudo-static resistances of precast concrete frames

Specimen	First peak load P_{dc} at point A (kN)	Middle joint displacement at P_{dc} (mm)	Second peak load P_{dt} at point B (kN)	Middle joint displacement at P_{dt} (mm)	P_{dt}/P_{dc}
IF-B-0.88-0.59	55.6	178.1	56.6	341.9	1.018
IF-L-0.88-0.59	53.3	178.2	68.7	681.7	1.236
EF-B-0.88-0.59	61.8	342.0	--	--	--
EF-L-0.88-0.59	56.8	171.1	54.1	300.0	0.952

Comparisons are also made between the pseudo-static resistances of precast frames, as shown in **Table 5.4**. Although slightly greater CAA capacities of interior frames IF-B-0.88-0.59 and IF-L-0.88-0.59 were obtained under quasi-static loading conditions compared to exterior frames EF-B-0.88-0.59 and EF-L-0.88-0.59 (see **Figs. 5.6(a and b)**), the calculated first peak pseudo-static resistances of exterior frames EF-B-0.88-0.59 and EF-L-0.88-0.59 are greater than those of interior frames IF-B-0.88-0.59 and IF-L-0.88-0.59, as shown in **Table 5.4**. For instance, the capacity of EF-B-0.88-0.59 under pseudo-static loading is 61.8 kN, 1.11 times greater than that of IF-B-0.88-0.59. Similarly, EF-L-0.88-0.59 is able to sustain 1.07 times greater first peak pseudo-static load than IF-L-0.88-0.59. In comparison with interior frames IF-B-0.88-0.59 and IF-L-0.88-0.59, greater first peak pseudo-static resistances of exterior frames EF-B-0.88-0.59 and EF-L-0.88-0.59 result from better energy absorption capacities of exterior frames in the descending branch of quasi-static loads, as shown in **Figs. 5.6(a and b)**.

5.4.5 Load paths of horizontal reaction forces to the support

Horizontal reaction forces as shown in **Fig. 5.7** represent the summation of horizontal forces measured by the horizontal restraint on the column top, the horizontal load cell on the beam extension and bottom pin support (see **Fig. 5.2**). To investigate the load path of horizontal forces to the support, **Fig. 5.10** also shows the individual reading of the load cells and pin support at one side column of the interior and exterior frames. With respect to interior frame IF-B-0.88-0.59, when it was subjected to CAA, the horizontal compression force was mainly sustained by the bottom pin support, and the horizontal load cell connected to the beam extension only took up a small portion of the compression force, as shown in **Fig. 5.10(a)**. Similar results were obtained at

CAA stage of interior frame IF-L-0.88-0.59, as shown in **Fig. 5.10(b)**. Once catenary action kicked in, the horizontal load cell at the beam started sustaining a greater portion of horizontal tension force, and the magnitude of reaction forces in the top load cell and bottom pin support came close to one another (**Fig. 5.10(b)**), as tension force in the beam was dominant over bending moment. Likewise, exterior frame EF-B-0.88-0.59 transmitted the horizontal compression force to the pin support when subjected to CAA (**Fig. 5.10(c)**). Under catenary action, the horizontal tension force developed in the beam was transferred to the support in a different manner from interior frame IF-L-0.88-0.59, as shown in **Fig. 5.10(d)**. The top load cell carried a major fraction of the total horizontal tension force, whereas the horizontal force in the bottom pin support was substantially smaller in comparison with that in the top load cell.

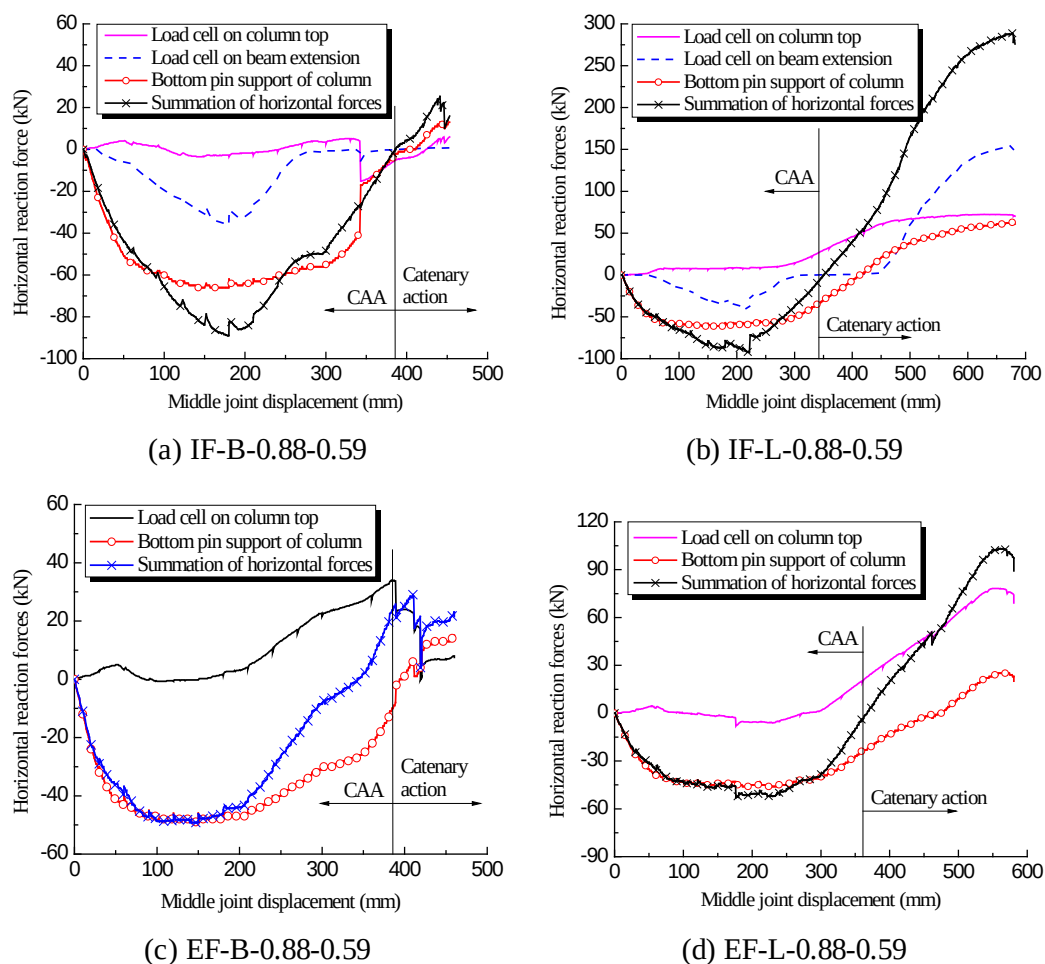


Fig. 5.10: Load paths of horizontal reaction forces to the support

5.4.6 Crack patterns and failure modes of precast beams

Among all four precast concrete frames, only IF-L-0.88-0.59 developed significant catenary action under column removal scenarios, whereas the other frames only mobilised CAA due to premature rupture of beam top reinforcement near the side column. Correspondingly, different crack patterns and failure modes were observed in the precast concrete frames, as summarised in **Table 5.5**.

Table 5.5: Failure modes of precast concrete frames

Specimen	Middle joint	Side column
IF-B-0.88-0.59	Pull-out of all beam bottom bars	Rupture of beam top bars
IF-L-0.88-0.59	Rupture of one bottom bar, pull-out of the other bottom bar	Pull-out failure of bottom bars near right column
EF-B-0.88-0.59	Pull-out of all beam bottom bars	Rupture of beam top bars
EF-L-0.88-0.59	Rupture of one bottom bar, pull-out of the other bottom bar	Rupture of beam top bar near left column, flexural failure of right column

Frames IF-B-0.88-0.59 and EF-B-0.88-0.59, with 90° bend of beam bottom reinforcement in the beam-column joint, developed similar crack patterns and failure modes in bridging beams, as shown in **Fig. 5.11** and **Fig. 5.12**. At the CAA stage, cracks were only concentrated in the flexural tension zones of the beam. Bottom longitudinal reinforcement in the beam was pulled out from the middle joint due to insufficient embedment length. Thus, the applied vertical load varied smoothly at CAA stage, as shown in **Fig. 5.6(a)**. Eventually, top longitudinal reinforcement ruptured at the face of the side column, which caused sudden drops of vertical load. Following the rupture of top reinforcement, a pin was formed at the face of the side column, and the vertical load could not increase any further, as shown in **Fig. 5.6(a)**.

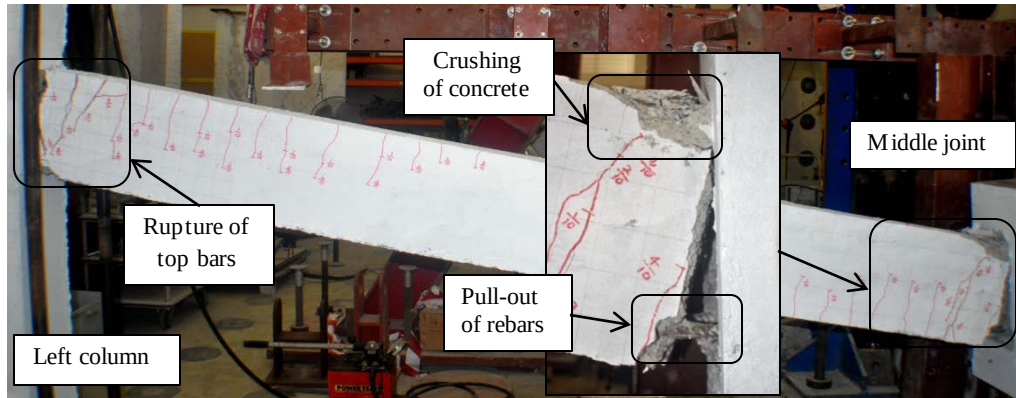


Fig. 5.11: Crack patterns and failure modes of IF-B-0.88-0.59

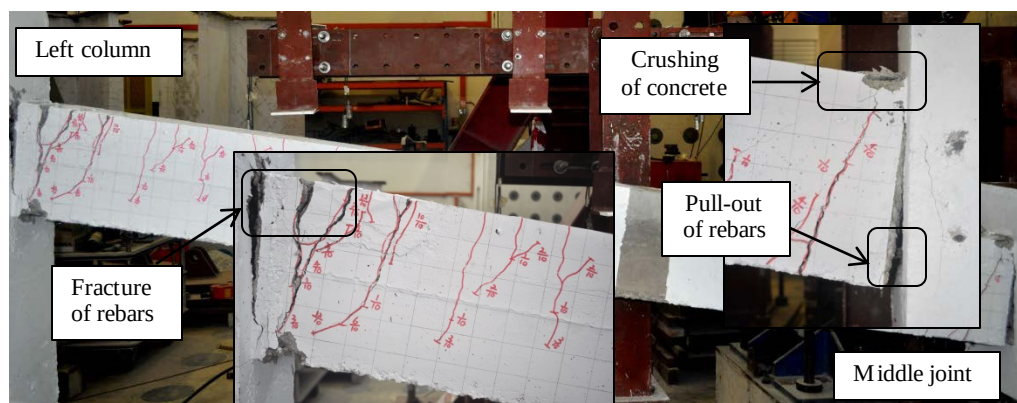


Fig. 5.12: Crack patterns and failure modes of EF-B-0.88-0.59

Precast frames IF-L-0.88-0.59 and EF-L-0.88-0.59 also exhibited similar failure modes of embedded beam bottom reinforcement in the middle joint, as shown in **Fig. 5.13** and **Fig. 5.14**. Only one bottom bar fractured at the middle joint interface, whereas the other bar was pulled out from the joint. Fracture of beam bottom reinforcement in the middle joint led to a sudden drop of vertical load at the CAA stage (see **Fig. 5.6(b)**). Furthermore, after failure of bottom steel bars at one joint interface, moment resistance at the opposite face of could be mobilised due to the presence of rotational restraint in the middle joint. With increasing middle joint displacements, a similar pull-out failure occurred at the opposite face of the middle joint which reduced the vertical load prior to the commencement of catenary action, as shown in **Fig. 5.6(b)**.

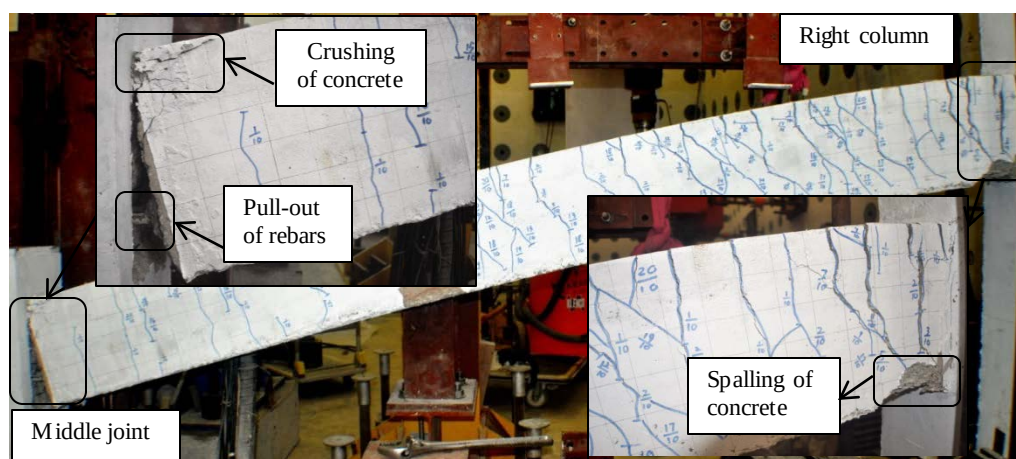


Fig. 5.13: Crack patterns and failure modes of IF-L-0.88-0.59

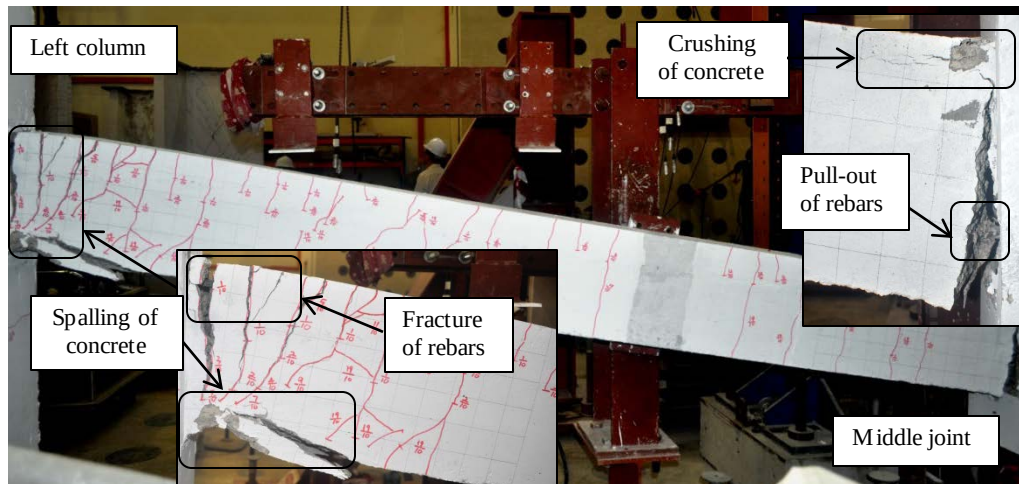


Fig. 5.14: Crack patterns and failure modes of EF-L-0.88-0.59

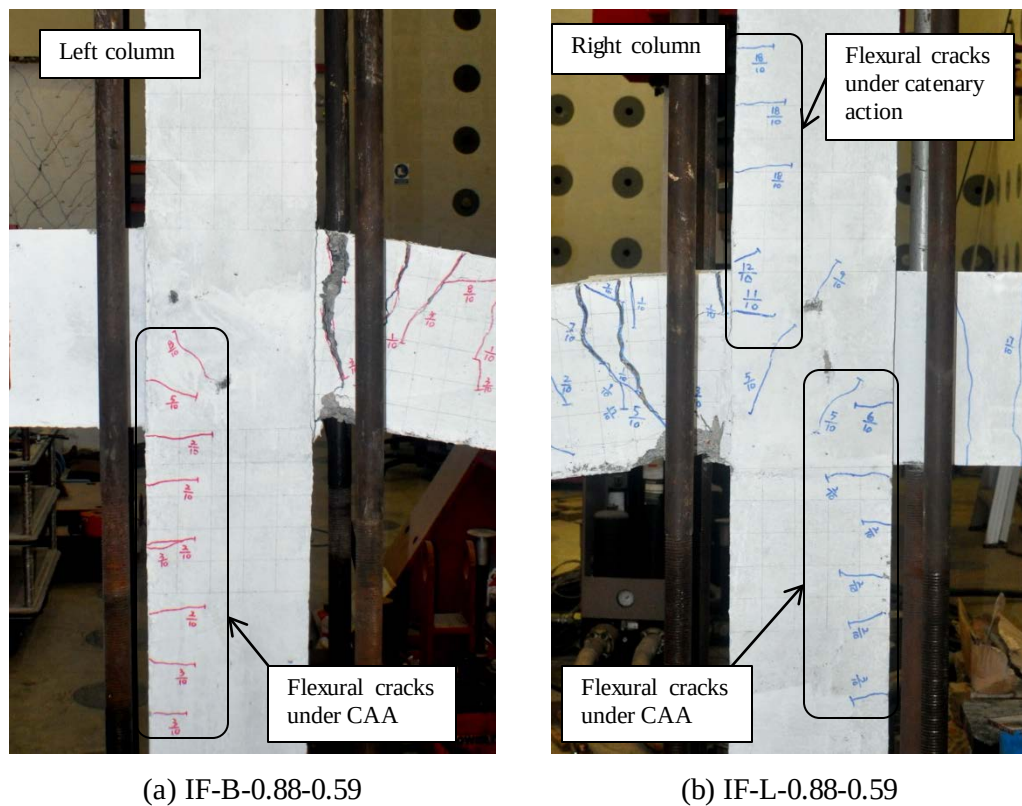
Different crack patterns of the beam and failure modes at the side column interface were observed in IF-L-0.88-0.59 and EF-L-0.88-0.59, as shown in **Fig. 5.13** and **Fig. 5.14**. Interior frame IF-L-0.88-0.59 developed the highest catenary action capacity among the four frames (see **Table 5.3**). Accordingly, remarkable cracking and flexural deformations were observed in the bridging beam, as shown in **Fig. 5.13**. Beam top reinforcement near the right column did not fracture. Instead, the lap-spliced beam bottom reinforcement developed pull-out failure in the plastic hinge region near the side column face. Correspondingly, the tension force in the beam was reduced (see **Fig. 5.7(b)**). In exterior frame EF-L-0.88-0.59, beam top reinforcement fractured at the left column face. Thereafter, axial tension dominated the beam behaviour under catenary action. Full-depth tension cracks ran perpendicular to the beam axis and distributed uniformly along the beam length, as shown in **Fig. 5.14**. Eventually, the right column exhibited crushing of concrete in the region above the right beam-column joint due to the horizontal tension force, and catenary action could not be maintained due to excessive lateral deflections of the right column.

5.4.7 Behaviour of side columns and joints

Fig. 5.15 illustrates the crack patterns of side columns subjected to CAA and catenary action. Under CAA, the horizontal compression force was mainly transferred to the bottom pin support (see **Fig. 5.10**). Column sections below the side joint developed flexural cracks at the outer face, as shown in **Figs. 5.15(a-d)**. Diagonal shear cracks were also observed in the side beam-column joint of EF-B-0.88-0.59, as shown in

Fig. 5.15(c). These cracks were mainly initiated at around 150 mm middle joint displacement, when the horizontal compression force attained its maximum value at the CAA stage. With increasing middle joint displacement, the crack width remained limited and the side beam-column joint did not develop shear failure due to the presence of horizontal hoops in the joint.

At the catenary action stage, significant horizontal tension forces in IF-L-0.88-0.59 and EF-L-0.88-0.59 generated flexural cracks at the column face towards the middle joint, as shown in **Figs. 5.15(b and d)**. In specimens IF-B-0.88-0.59, IF-L-0.88-0.59 and EF-B-0.88-0.59, the side columns did not fail due to premature fracture of beam top reinforcement at the side column face, or presence of horizontal restraints on the beam extension. Only the right column of exterior frame EF-L-0.88-0.59 developed flexural failure, characterised by crushing of concrete above the side beam-column joint, as shown in **Fig. 5.15(d)**. Following the column failure, excessive lateral deflections of the side column hindered development of horizontal tension force, as shown in **Fig. 5.7(b)**.



(a) IF-B-0.88-0.59

(b) IF-L-0.88-0.59

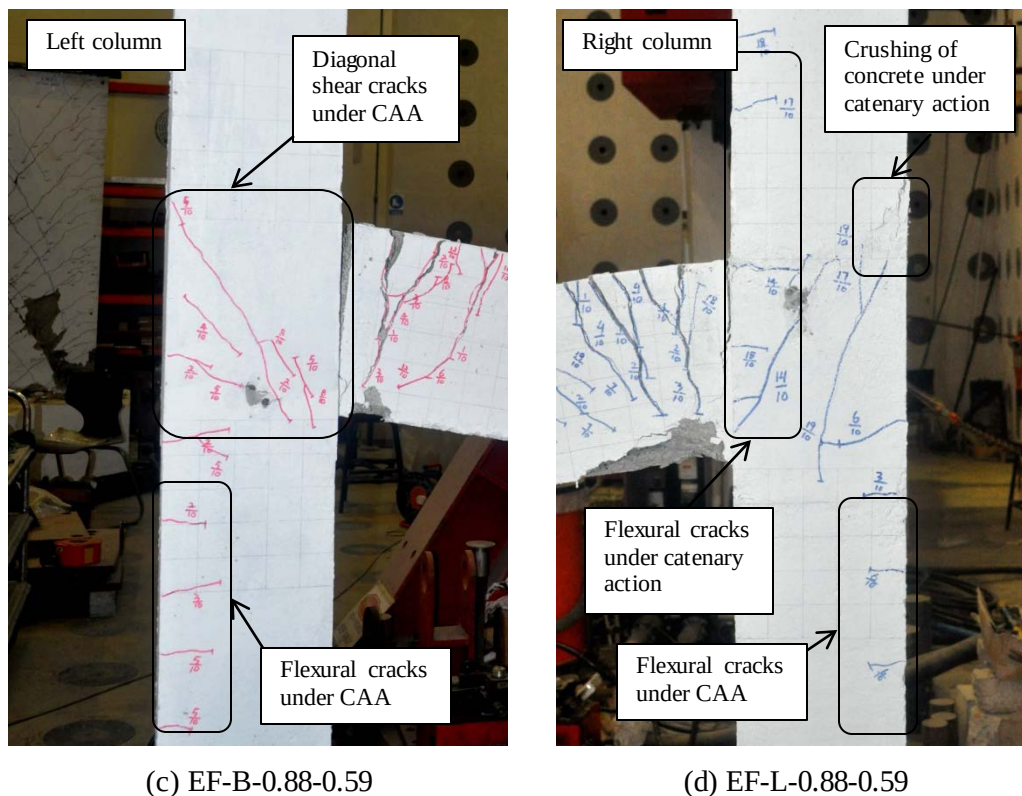


Fig. 5.15: Crack patterns and failure modes of side columns

When subjected to CAA and subsequent catenary action, side columns developed significant lateral deflections. **Fig. 5.16** shows the deformed profiles of side columns. In the sign convention, the negative value denotes outward deflection of the side column relative to the middle joint, and the positive number represents inward deflection towards the middle joint. Similar to the frame behaviour reported by Yi et al. (2008), the side column was initially pushed outwards by the horizontal compression force under CAA, whereas it was pulled towards the middle joint by the tension force under catenary action, as shown in **Figs. 5.16(a-d)**. Interior frames IF-B-0.88-0.59 and IF-L-0.88-0.59 and exterior frames EF-B-0.88-0.59 and EF-L-0.88-0.59 attained almost the same negative deflections at the maximum horizontal compression force. Nonetheless, due to presence of horizontal restraint on the beam extension, interior frame IF-L-0.88-0.59 only exhibited 7.5 mm positive deflections at the peak horizontal tension force, as shown in **Fig. 5.16(b)**. In EF-L-0.88-0.59, the maximum lateral deflection was 15.7 mm due to flexural failure of the right column, as shown in **Fig. 5.16(d)**.

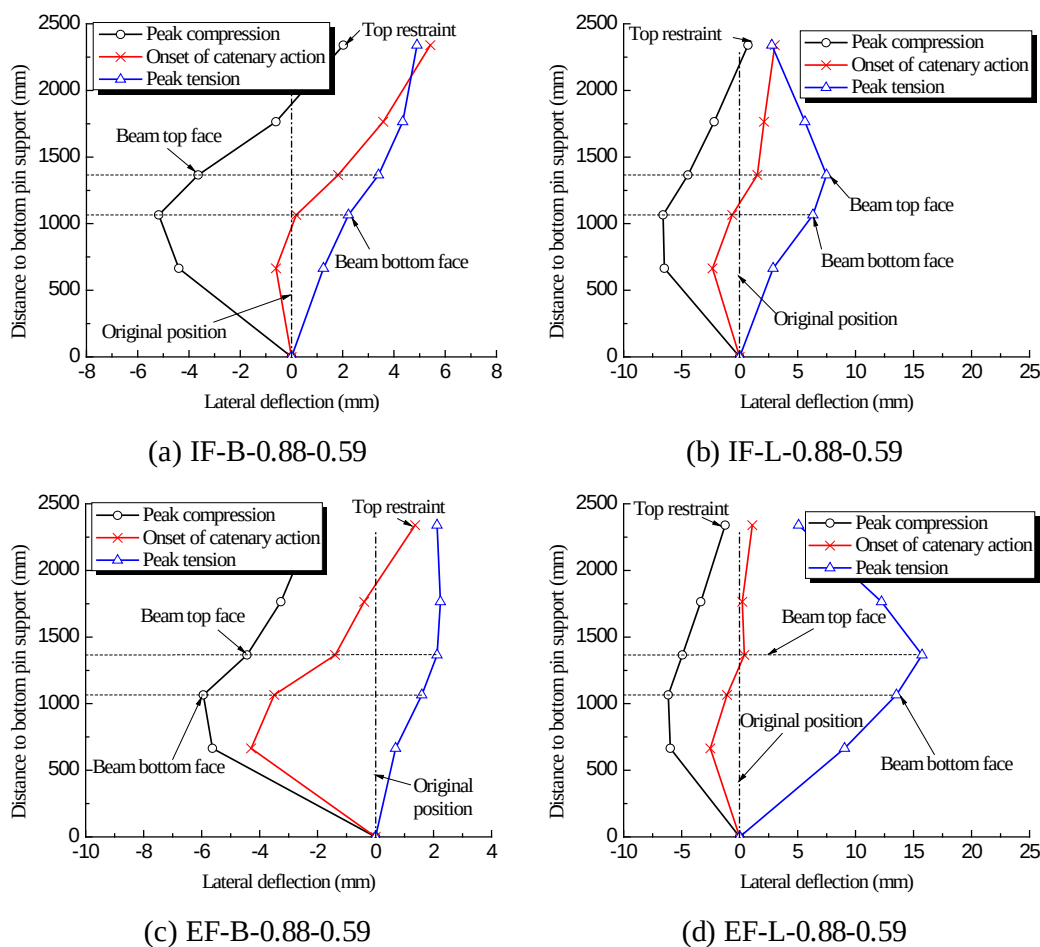


Fig. 5.16: Lateral deflections of side columns

Besides the deflection profiles, measurements of LVDTs SD-3 and SD-4, corresponding to the top and bottom faces of the beam (see Fig. 5.4), on the columns of exterior frames EF-B-0.88-0.59 and EF-L-0.88-0.59 are shown in Fig. 5.17. At CAA stage, SD-4 measured larger negative deflection than SD-3. When catenary action kicked in, the positive value of SD-3 was greater than SD-4. Positive deflections of the right column of EF-L-0.88-0.59 varied at almost a constant rate until the column could not sustain the lateral tension force and crushing of concrete occurred above the joint in the right column (see Fig. 5.15(d)). In the wake of flexural failure of the column, the horizontal tension force was reduced (Fig. 5.7(b)), but lateral deflections of the column was further increased to 15.7 mm, as shown in Fig. 5.17(b).

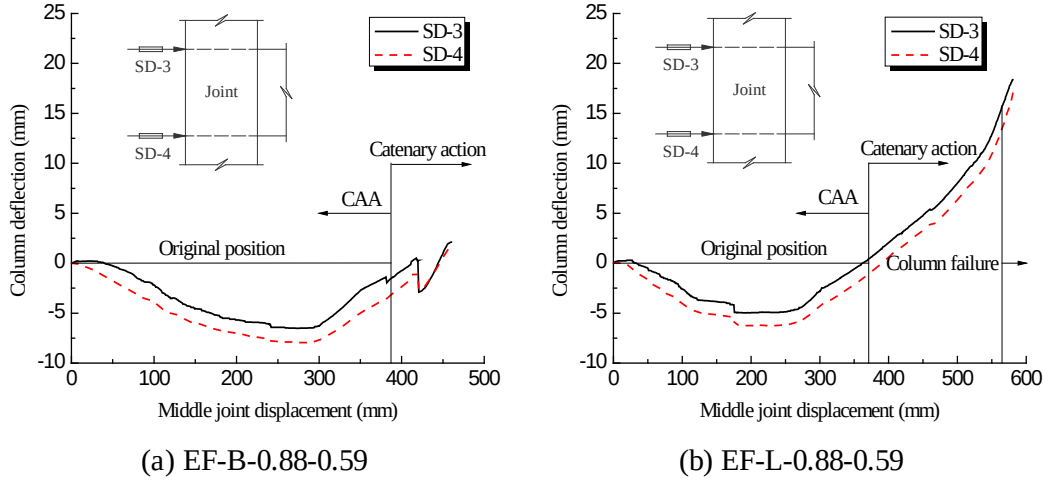


Fig. 5.17: Column deflection-middle joint displacement curves of exterior frames

On top of lateral deflections of the side column, shear behaviour of the side joint may be instrumental to structural resistance of precast concrete frames. To quantify the shear distortion of the side joint subjected to CAA, four steel threads were embedded into the joint panel encased by beam and column reinforcement, on which a pair of diagonal LVDTs LJ-1 and LJ-2 was installed, as shown in **Fig. 5.4**. At CAA stage, a diagonal strut was formed by forces in the compression zones of the beam and side column, as shown in **Fig. 5.18(a)**. LJ-1 was shortened by the diagonal compression force in the joint, whereas LJ-2 was elongated at CAA stage. This observation agrees well with the crack pattern of the side joints, as shown in **Fig. 5.15(c)**. To further calculate the joint distortion from the LVDT measurements, the joint model proposed by Youssef and Ghobarah (2001) is modified, as plotted in **Fig. 5.18(b)**. In accordance with the deformation compatibility condition of the joint panel, total shear distortion γ is computed from Eq. (5-2).

$$\gamma = \gamma_1 + \gamma_2 = \frac{a^2 - b^2}{\sqrt{(a^2 + b^2)^2 - 4a^2b^2 \cos^2(2\theta)}} \quad (5-2)$$

in which $a = \frac{l_j}{2} + \delta_1$, $b = \frac{l_j}{2} - \delta_2$. l_j is the diagonal length of the joint panel; δ_1 and δ_2 are the deformations of the joint panel in the two directions.

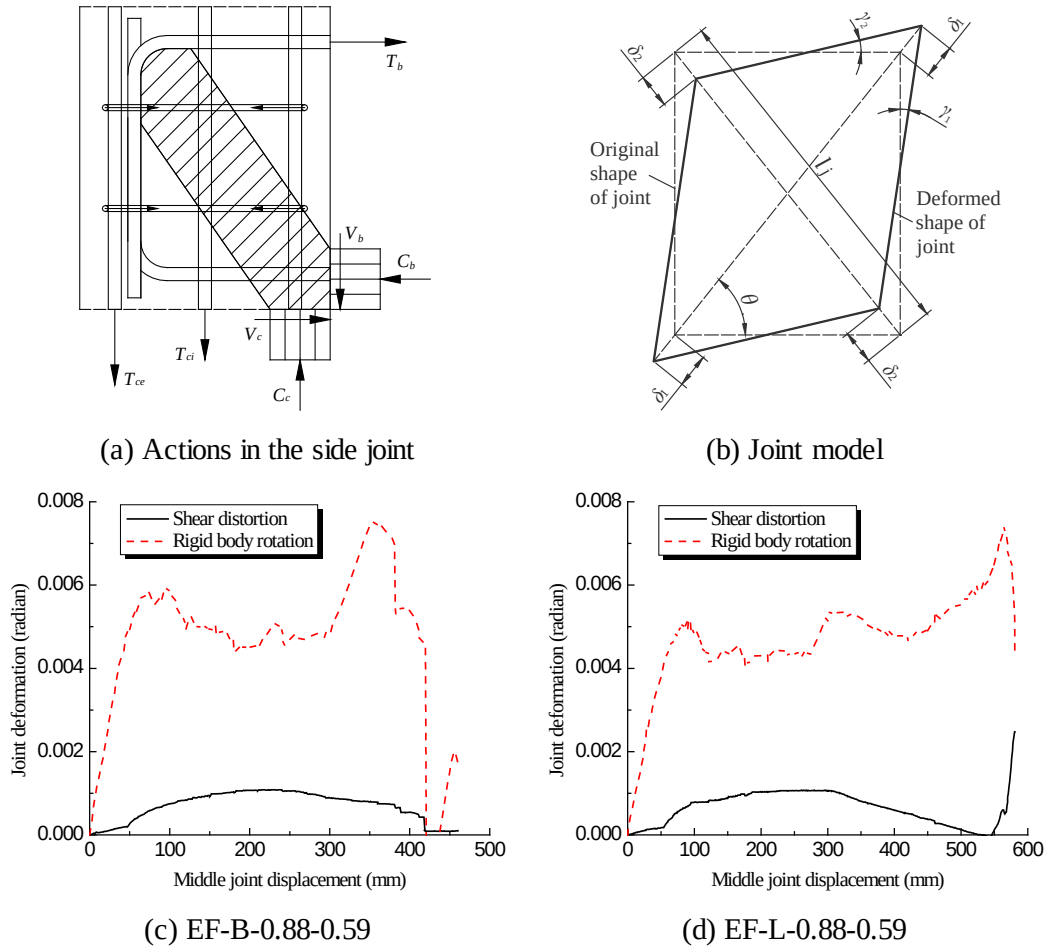


Fig. 5.18: Shear distortion of side beam-column joints

In addition to the shear distortion, the rigid-body rotation of the side joint is calculated as the difference between the readings of SD-3 and SD-4 (see **Fig. 5.4**) divided by their vertical spacing, as expressed in Eq. (5-3). **Figs. 5.18(c and d)** show the comparisons between the shear distortion and rigid-body rotation of the side joint in exterior frames EF-B-0.88-0.59 and EF-L-0.88-0.59. In EF-B-0.88-0.59, the maximum value of the shear distortion was only 0.0012 radian, around 24% of the joint rotation at the same middle joint displacement. Thus, the shear distortion of the side beam-column joint was insignificant compared to the rigid-body rotation under column removal scenarios.

$$\theta_r = \frac{\delta_{SD-3} - \delta_{SD-4}}{d} \quad (5-3)$$

where θ_r is the rigid-body rotation of the side joint, δ_{SD-3} and δ_{SD-4} are the measurements of LVDTs SD-3 and SD-4, respectively; and d is the vertical spacing between SD-3 and SD-4, equal to the full depth of the precast beam.

5.4.8 Variation of steel strains in beams and columns

Fig. 5.19 shows the layout of strain gauges along the bottom longitudinal reinforcement in the beam. In the middle joint, strain gauges were mounted to the middle joint interface and the rebar section 60 mm into the joint. **Fig. 5.20** shows the variations of steel strains with the middle joint displacement of IF-L-0.88-0.59 at CAA stage. At the middle joint faces, steel strains LB-3, RB-3 and RB-4 decreased slowly after attaining the maximum values, as shown in **Fig. 5.20(a)**, indicating pull-out failure of beam bottom reinforcement. However, the strain of LB-4 kept increasing (**Fig. 5.20(a)**) until rebar fractured at the left face. In the middle joint, a similar reduction in steel strains was obtained before 200 mm middle joint displacement was reached, as shown in **Fig. 5.20(b)**. With increasing middle joint displacement, all the steel strains started increasing after fracture of one bottom bar at the left face of the middle joint, as rotational restraint in the middle joint mobilised the moment resistance of the right joint face. At 306 mm middle joint displacement, steel strains decreased again as a result of pull-out failure of rebar at the right joint face.

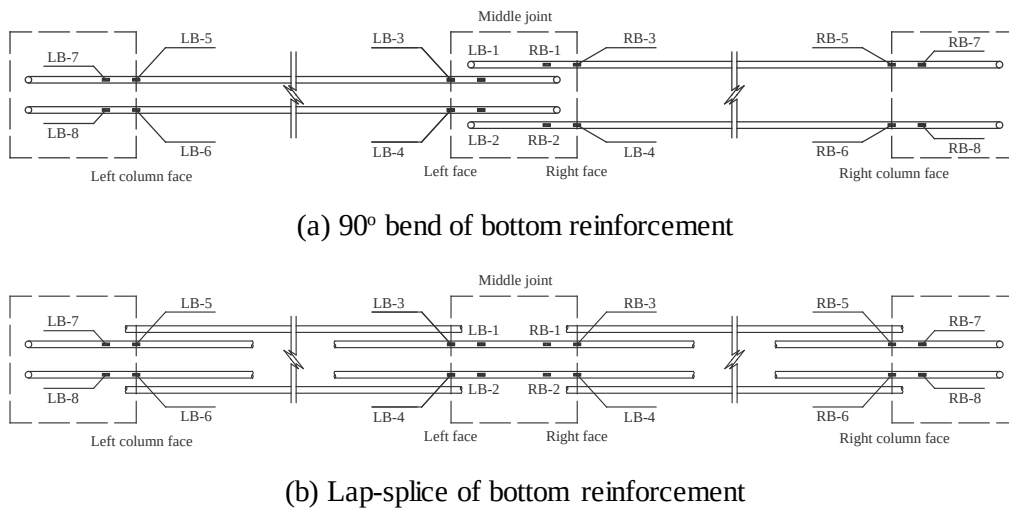


Fig. 5.19: Strain gauge layout along the bottom bars of precast beams

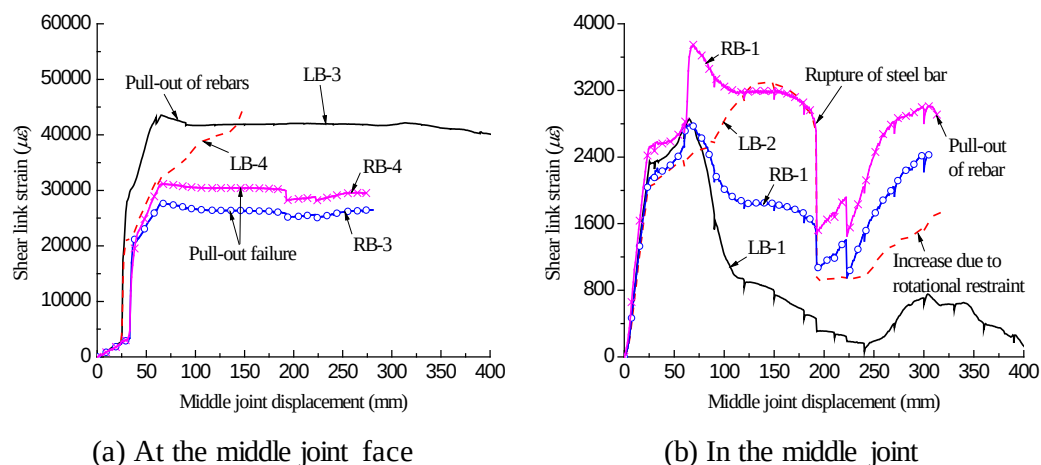


Fig. 5.20: Variations of steel strains in the middle beam-column joint of IF-L-0.88-0.59

Fig. 5.21 shows the strains of beam bottom reinforcement at the face of the side column. Similar to the beam-column sub-assemblages in Section 3.4.7, bottom reinforcement at the column face of IF-B-0.88-0.59 and EF-B-0.88-0.59 sustained compressive stress at CAA stage, as shown in **Figs. 5.21(a and c)**. However, in IF-L-0.88-0.59 and EF-L-0.88-0.59, bottom bars near the side column were subjected to tension at the CAA and catenary action stages, as shown in **Figs. 5.21(b and d)**, due to limited neutral axis depth at the beam end and relatively large distance between the lap-spliced bottom bars and the extreme compression fibre of the beam. Similar to the effect of a middle layer of steel bars as tension reinforcement (Yu and Tan 2014), the two lap-spliced bottom bars helped to improve the rotational capacity of the plastic hinge near the side column, and enhanced the catenary action of precast frame IF-L-0.88-0.59. Prior to failure, steel strains RB-7 and RB-8 in the right column of IF-L-0.88-0.59 decreased dramatically, as shown in **Fig. 5.21(b)**, indicating pull-out failure of the lap-spliced beam bottom bars.

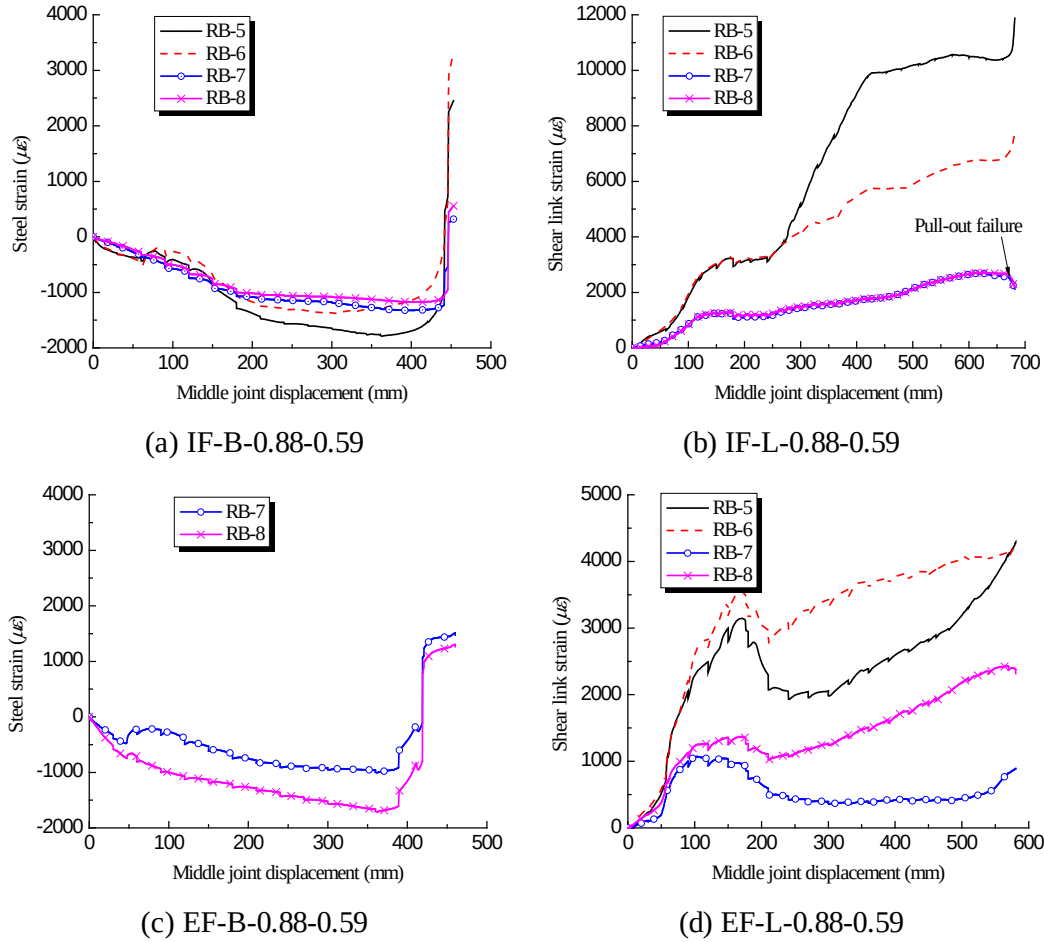


Fig. 5.21: Strains of beam bottom reinforcement embedded in the right column

Fig. 5.22 shows the arrangement of strain gauges on the longitudinal reinforcement and horizontal hoops of the right column. All measurements were initialised to zero prior to testing to eliminate the effect of axial compression force in the side columns, and only steel strains generated by bending moment were recorded. **Fig. 5.23** depicts the variations of steel strains at the selected locations corresponding to the top and bottom faces of the beam. In interior frame IF-L-0.88-0.59, RC-4 monitored the tensile strain of steel reinforcement at CAA stage, with its maximum value smaller than the yield strain, whereas strain gauges RC-1, RC-2 and RC-3 were subjected to compression, as shown in **Fig. 5.23(b)**. The onset of catenary action transformed the strains of RC-1 and RC-3 from compression to tension, but the strain of RC-4 was shifted from tension to compression. Eventually, steel bars RS1 and RS2 were subjected to tension and compression, respectively, due to the axial tension force in the beam. Exterior frame EF-L-0.88-0.59 exhibited similar variation of strains of

column longitudinal reinforcement, as shown in **Fig. 5.23(d)**. However, RC-2 carried tensile stress at CAA stage, due to a lack of horizontal restraint on the side beam-column joint. Similar variations of steel strains to IF-L-0.88-0.59 and EF-L-0.88-0.59 were also recorded in IF-B-0.88-0.59 and EF-B-0.88-0.59, as shown in **Figs. 5.23(a and c)**.

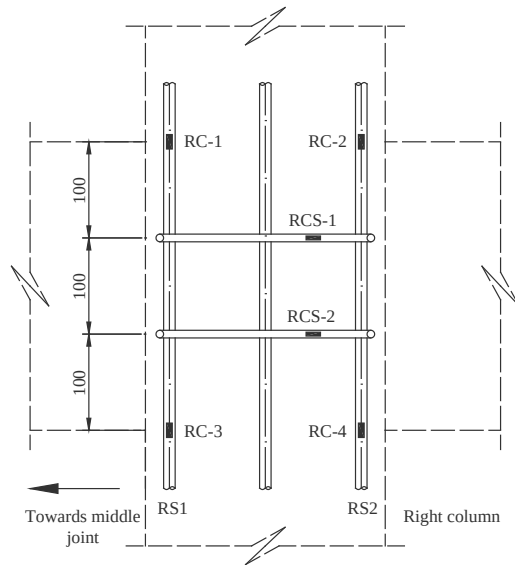
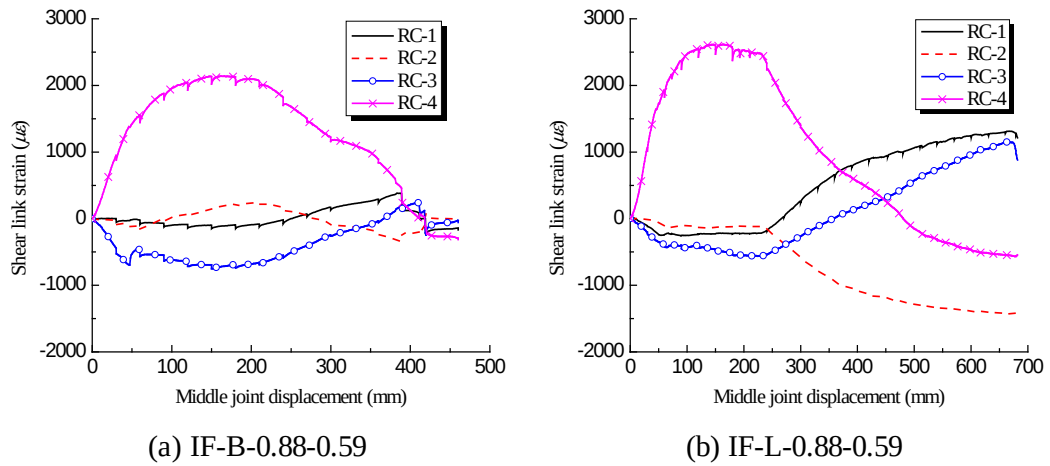


Fig. 5.22: layout of strain gauges in side beam-column joint



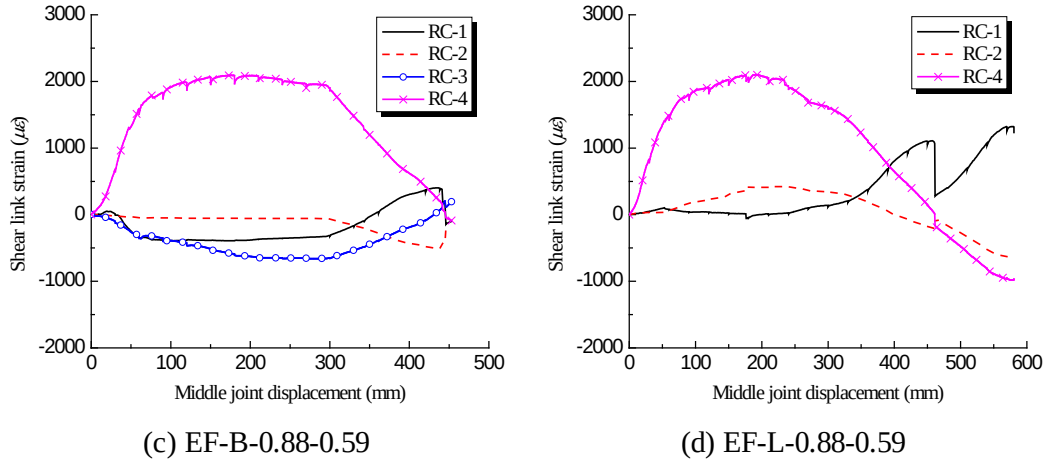


Fig. 5.23: Variations of reinforcement strains in side columns

Strains of horizontal hoops in the side joint were also measured through strain gauges, as shown in **Fig. 5.24**. In the notations, “RCS-1” and “RCS-2” represent strain gauges on the horizontal hoops in the right joint, as depicted in **Fig. 5.22**, and “LCS-1” and “LCS-2” correspond to strain gauges in the left joint. Interior frames IF-B-0.88-0.59 and IF-L-0.88-0.59 exhibited similar variations of strains of horizontal hoops in the joint, as shown in **Figs. 5.24(a and b)**. With minor diagonal cracking in the joint zone (see **Figs. 5.15(a and b)**), horizontal hoops only sustained limited tensile strains under CAA. At catenary action stage, LCS-1 and LCS-2 in IF-L-0.88-0.59 increased simultaneously as a result of horizontal tension force transmitted to the joint, as shown in **Fig. 5.24(b)**. Exterior frame EF-B-0.88-0.59 developed significant shear cracking in the side joint. At about 60 mm middle joint displacement, tensile strains of horizontal hoops started increasing, as shown in **Fig. 5.24(c)**, indicating formation of shear cracks in the joint. Following diagonal cracking in the side joint, strains of horizontal hoops increased slowly. However, the horizontal hoops did not reach their yield strain and remained largely at the elastic stage up to failure. Therefore, in spite of shear cracking, the side joint was able to resist the shear force at CAA stage. In EF-L-0.88-0.59, tension force in the beam also increased the strain of RCS-1, but slightly reduced that of RCS-2, as shown in **Fig. 5.24(d)**.

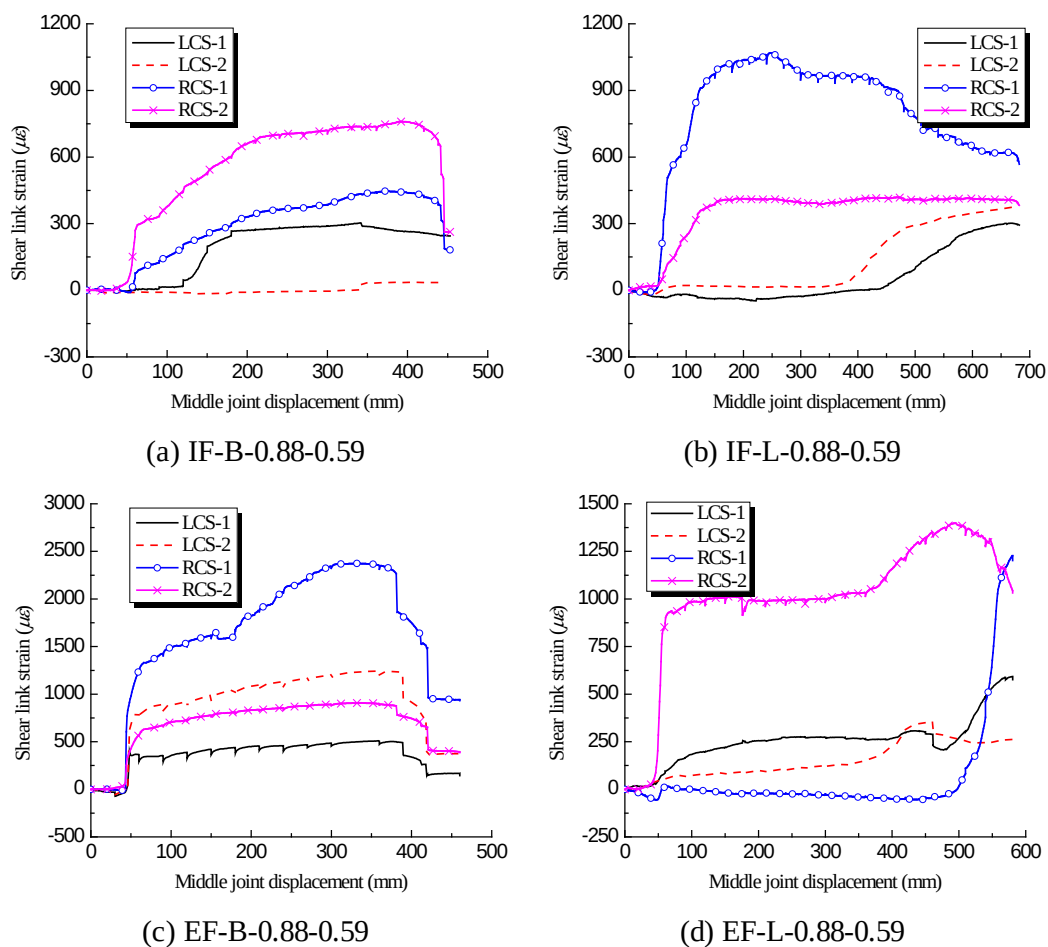


Fig. 5.24: Strains of horizontal hoops in side joint zone

5.5 Summary

In the experimental programme, four precast concrete frames were tested under quasi-static loads to investigate structural resistances and deformation capacities under middle column removal scenarios. 90° bend and lap-splice of beam bottom reinforcement were utilised in the middle and side beam-column joints. Apart from the joint detailing, the effect of boundary conditions on the behaviour of precast concrete frames was studied experimentally. Conclusions are drawn from experimental results as follows:

- (1) Under quasi-static loading conditions, similar behaviour of precast concrete frames was obtained at CAA stage, with approximately the same capacities of CAA for all four frames. However, the catenary action capacities of frames varied greatly as a result of different reinforcement detailing and horizontal restraints.

(2) Compared with 90° bend of beam bottom reinforcement, lap-spliced reinforcement in the joint enabled the development of greater catenary action in precast concrete frames, in particular for IF-L-0.88-0.59. Strain gauge readings of beam longitudinal reinforcement indicate that even the lap-spliced bottom reinforcement at the right column face was subjected to tension due to limited neutral axis depth, which substantially enhanced the rotational capacity of the beam plastic hinge near the side joint.

(3) Exterior frames EF-B-0.88-0.59 and EF-L-0.88-0.59 developed less significant catenary action than interior frames IF-B-0.88-0.59 and IF-L-0.88-0.59 due to either premature fracture of beam top longitudinal reinforcement at the side column face (i.e. EF-B-0.88-0.59), or flexural failure of the side column (i.e. EF-L-0.88-0.59).

(4) Exterior frames EF-B-0.88-0.59 and EF-L-0.88-0.59 develop greater pseudo-static load resistances at CAA stage than interior frames IF-B-0.88-0.59 and IF-L-0.88-0.59, due to better energy absorption capacities in the descending branch of vertical loads at CAA stage.

(5) The side column experienced significant lateral deflections and cracking at CAA and catenary action stages. However, only the right column of exterior frame EF-L-0.88-0.59 developed flexural failure under catenary action. Significant shear cracks were observed in the side beam-column joint of EF-B-0.88-0.59, but horizontal hoops remained in elastic stage until final failure occurred. At the CAA stage, shear distortion of the side joint was insignificant in comparison with the rigid-body rotation of the joint.

In accordance with experimental results, lap-splice of beam bottom reinforcement in the joint is recommended for use in precast concrete structures against progressive collapse. To enable the full development of catenary action in exterior frames, the side column and beam-column joint have to be prevented from potential flexural and shear failures at large deformations. Thus, further experimental tests on exterior precast concrete frames are necessary to study the behaviour of side columns under progressive collapse scenarios.

CHAPTER 6 EXPERIMENTAL STUDY ON EXTERIOR PRECAST CONCRETE FRAMES

6.1 Introduction

For exterior precast concrete frames, horizontal restraints are only provided by adjacent columns. If these columns are slender or with low horizontal stiffness, development of compressive arch action (CAA) and catenary action in the bridging beam over the column removal will be limited. Besides, horizontal compression and tension forces imposed by CAA and catenary action on the side columns may induce shear failure in the side joints and flexural failure of the columns, as introduced in Section 5.4.7. Therefore, the behaviour of the side columns subjected to CAA and catenary action has to be investigated when CAA and subsequent catenary action develop in the bridging beam.

This chapter presents an experimental study on four exterior precast concrete frames under column removal scenarios, in which the horizontal load cell connected to the beam extension was eliminated. In comparison with the frame specimens in Chapter 5, the top reinforcement ratio in the bridging beams was increased from 0.88% to 1.19%, as it has been shown to be an effective way to enhance structural resistance under column removal scenarios (Yu and Tan 2013c). The resistance of exterior frames was quantified under quasi-static loading condition. An attempt was also made to quantify the resistance of side columns and beam-column joints under CAA and catenary action, endeavouring to shed light on the design of columns and beam-column joints against progressive collapse.

6.2 Experimental Programme

6.2.1 Specimen design and detailing

In the experimental programme, four precast concrete frames were designed and fabricated in accordance with Eurocode 2 (BSI 2004). **Table 6.1** includes the geometry and reinforcement details of the beams and columns. In the notations, “EF” represents exterior frames. “B” or “L” denotes 90° bend or lap-splice of bottom layer

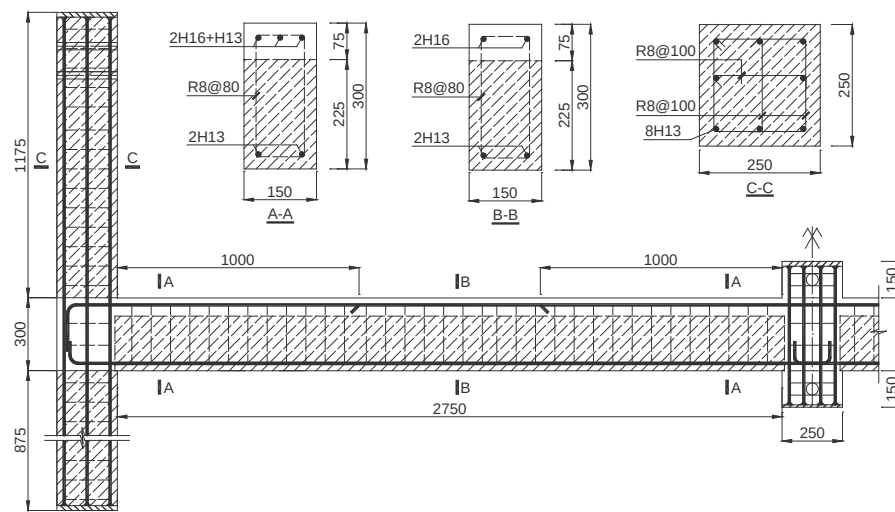
of beam longitudinal reinforcement in the joint. The two numerals separated with slash represent the respective top and bottom reinforcement ratios at beam end sections. The last letter “S” denotes that the cross section of precast columns was enlarged from 250 mm square to 300 mm square. The whole cross section of the beam was 150 mm by 300 mm, with 225 mm deep precast beam unit and 75 mm thick cast-in-situ concrete topping. The centre-to-centre spacing of columns was 3 m. In frames EF-B-1.19/0.59 and EF-L-1.19/0.59, the column section was 250 mm square and the clear span of the beam was 2.75 m. However, due to the enlarged cross section of side columns in frames EF-B-1.19/0.59S and EF-L-1.19/0.59S, the clear span of these two specimens was reduced from 2.75 m to 2.7 m. In all the four specimens, the area of beam and column longitudinal reinforcement was kept constant, as listed in **Table 6.1**.

Table 6.1: Geometry and reinforcement details of precast concrete frames

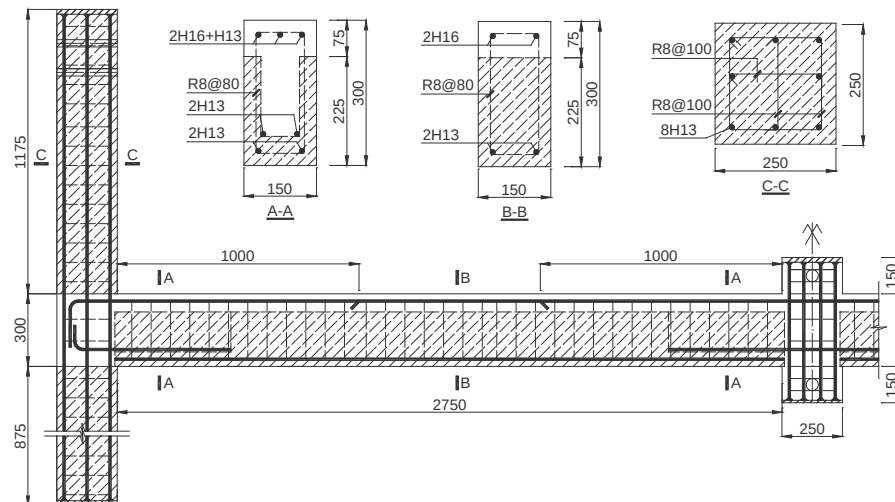
Specimen	Beam					Column		
	Cross section (mm)	Clear span (m)	Longitudinal bars		Stirrups	Cross section (mm)	Main bars	Stirrups
			A-A	B-B				
EF-B-1.19/0.59	150 x 300	2.75	2H16+H13 (top); 2H13 (bottom)	2H16 (top); 2H13 (bottom)	R8@80	250 x 250	8H13	R8@100
EF-L-1.19/0.59								
EF-B-1.19/0.59S		2.70				300 x 300		
EF-L-1.19/0.59S								

In the frame specimens, the beam and column units were prefabricated, as shown by the hatched zones in **Fig. 6.1**. The precast components were assembled into the frame by proper bottom reinforcement detailing in the joint and continuous top longitudinal reinforcement. Finally, cast-in-situ concrete was placed as structural topping to form the integral frames. Two types of reinforcement detailing (i.e. 90° bend and lap-splice), identical to those in precast concrete sub-assemblages in Chapter 3, were used in the beam-column joint, as shown in **Figs. 6.1(a and b)**. In frames EF-B-1.19/0.59 and EF-B-1.19/0.59S, beam bottom reinforcement was projected from the beam end and bent into the joint (see **Fig. 6.1(a)**). However, in EF-L-1.19/0.59 and EF-L-1.19/0.59S, bottom longitudinal bars were curtailed at the end of precast beam units, and two short steel bars were placed in the prefabricated trough in the beam, as shown in **Fig. 6.1(b)**. The horizontal interface between the precast concrete units and the

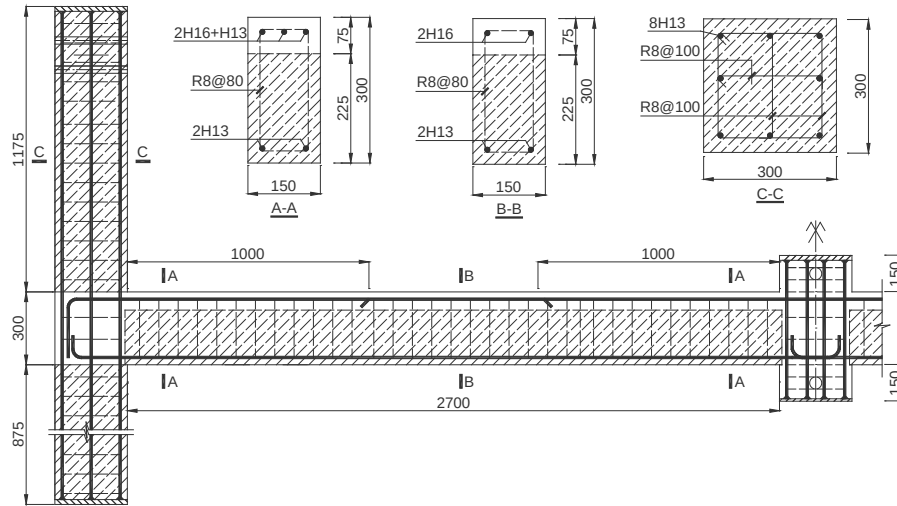
cast-in-situ concrete was intentionally roughened to about 3 mm roughness according to Eurocode 2 (BSI 2004). Stirrups of 8 mm diameter at 80 mm spacing along the beam length were protruded from the precast beam units to prevent delamination between the precast beam units and structural topping. Stirrups with 8 mm diameter at 100 mm spacing were placed in the column and side beam-column joint. The same test setup and instrumentations as those in Section 5.2.2 and Section 5.2.3 were used for the exterior frames. Moreover, an axial compressive stress of $0.3f'_c$, where f'_c is the cylinder compressive strength of concrete, was applied to side columns and kept constant during testing.



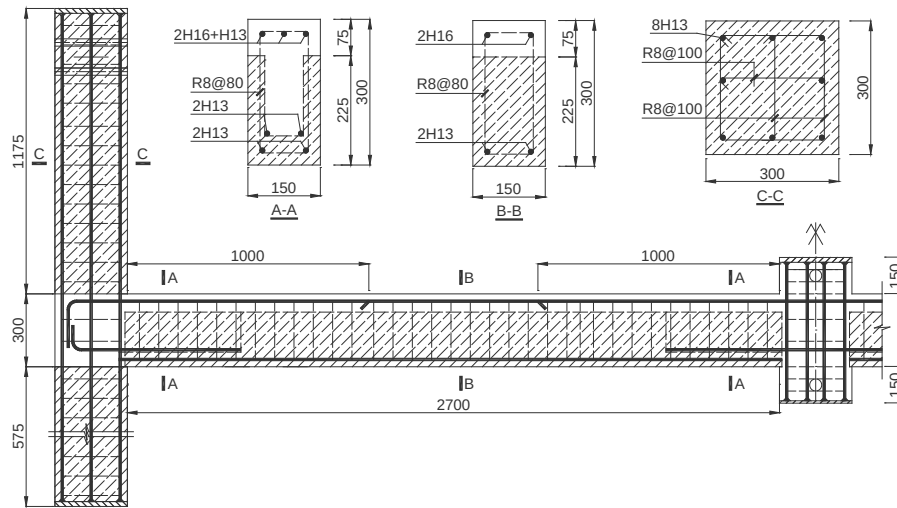
(a) EF-B-1.19/0.59



(b) EF-L-1.19/0.59



(c) EF-B-1.19/0.59S



(d) EF-L-1.19/0.59S

Fig. 6.1: Geometry and reinforcement detailing of precast concrete frames

6.2.2 Material properties

Hot-rolled high strength deformed bars H13 and H16 were used for longitudinal reinforcement, and mild steel bars of 8 mm diameter were used for stirrups in the beam and column. Material properties of steel reinforcement were obtained through testing, as listed in **Table 6.2**. It is noteworthy that the steel bars at the top and bottom layers of beam longitudinal reinforcement were from different batches of reinforcement, and their nominal strengths were different (see **Table 6.2**). As for concrete, the compressive and splitting tensile strengths were obtained through tests on 150 mm diameter and 300 mm long concrete cylinders. Concrete strain gauges

with a gauge length of 60 mm were mounted in the middle of each cylinder to obtain the modulus of elasticity. **Table 6.3** shows the strengths and elastic moduli of concrete cylinders.

Table 6.2: Material properties of steel reinforcement

Material		Nominal diameter (mm)	Yield strength (MPa)	Elastic modulus (GPa)	Ultimate strength (MPa)	Fracture strain* (%)	Remark
Main bars	H13	13	553.2	203.9	630.8	10.8	Beam bottom bars and column bars
			593.7	202.2	688.4	12.0	Beam top bars
	H16	16	493.9	204.0	615.7	16.0	Beam top bars
Stirrups	R8	8	272.4	207.4	359.5	--	Beam and column

*: Fracture strain refers to the average strain over a gauge length of 300 mm when steel reinforcement ruptures.

Table 6.3: Compressive and splitting tensile strengths of concrete

Specimen	Location	Compressive strength (MPa)	Modulus of elasticity (GPa)	Splitting tensile strength (MPa)
EF-B-1.19/0.59 EF-L-1.19/0.59	Precast beam and column units	26.9	25.8	2.1
EF-B-1.19/0.59S EF-L-1.19/0.59S	Cast-in-situ concrete	38.1	26.3	2.8

6.3 Test Results of Exterior Frames

6.3.1 Load-displacement curves

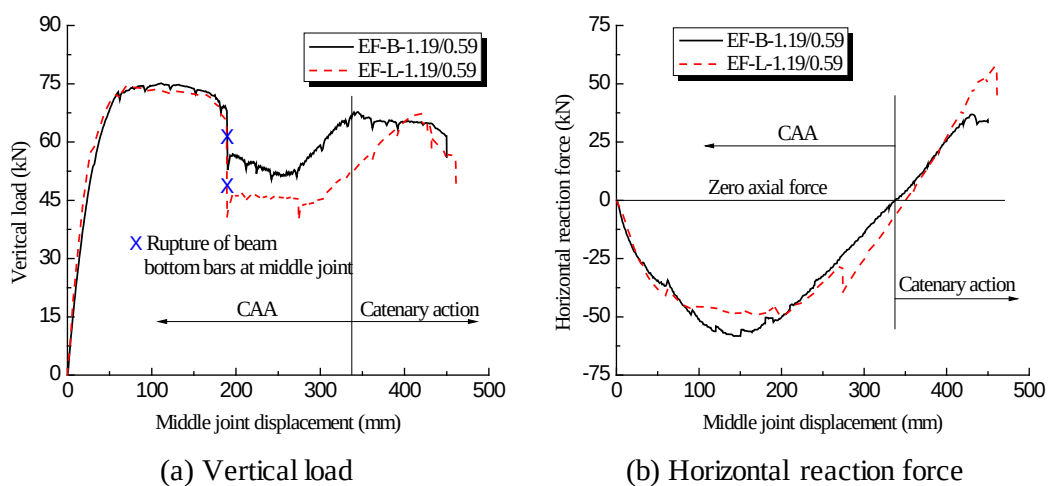


Fig. 6.2: Vertical loads and horizontal reaction forces of exterior frames EF-B-1.19/0.59 and EF-L-1.19/0.59

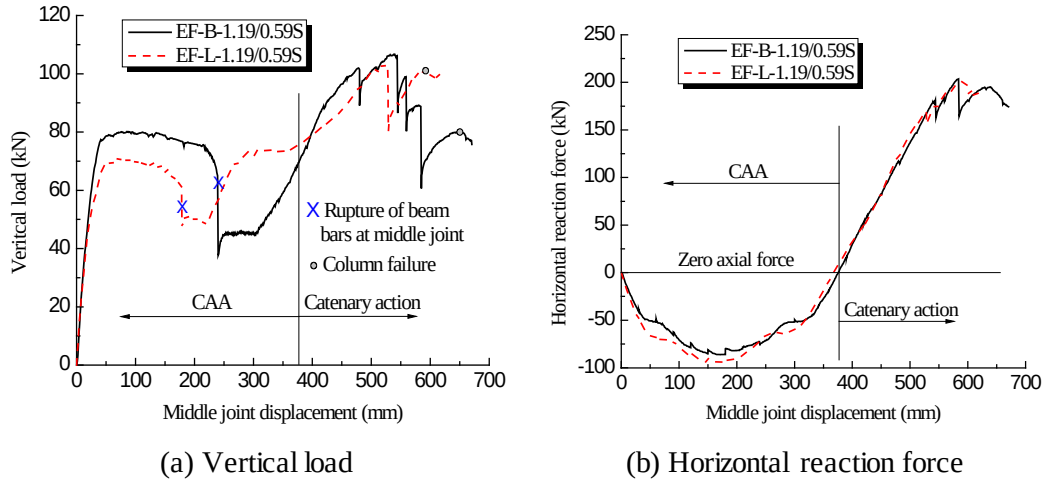


Fig. 6.3: Vertical loads and horizontal reaction forces of exterior frames EF-B-1.19/0.59S and EF-L-1.19/0.59S

Fig. 6.2 and **Fig. 6.3** show the variations of vertical loads and horizontal reaction forces with middle joint displacement. Under column removal scenarios, significant CAA and catenary action sequentially developed in the beam. This was evident from the beam initial axial compression and subsequent tension forces, as defined by Su et al. (2009) and Yu and Tan (2010a). At the CAA stage, vertical load increased with increasing middle joint displacement until a plateau stage was reached, as shown in **Fig. 6.2(a)** and **Fig. 6.3(a)**. In the meantime, axial compression force developed in the bridging beam of precast concrete frames (see **Fig. 6.2(b)** and **Fig. 6.3(b)**), which achieved the maximum value later than the vertical load. Rupture of beam bottom reinforcement occurred at the face of the middle joint, leading to a sudden drop in the vertical load, as shown in **Fig. 6.2(a)** and **Fig. 6.3(a)**. A further increase in the middle joint displacement increased the vertical load due to the presence of rotational restraint at the middle joint. In EF-B-1.19/0.59 and EF-L-1.19/0.59, shear failure in the side beam-column joint gradually reduced the vertical load on the middle joint (**Fig. 6.2(a)**), even though the horizontal tension force could increase with increasing middle joint displacement (**Fig. 6.2(b)**). Frames EF-B-1.19/0.59S and EF-L-1.19/0.59S attained substantially greater catenary action capacities than the CAA capacities, as shown in **Fig. 6.3(a)**. Eventually, flexural failure of side columns hindered the development of tension force in the beam, causing collapse of specimens EF-B-1.19/0.59S and EF-L-1.19/0.59S.

6.3.2 Resistances of precast concrete frames

Behaviour of precast concrete frames is primarily characterised by the maximum vertical load imposed on the middle beam-column joint and the peak horizontal force developed in the bridging beam. **Table 6.4** summarises the vertical load resistance, the horizontal compression force and associated middle joint displacements of precast concrete frames at CAA stage. Although different bottom reinforcement detailing was used in the joint, exterior frames EF-B-1.19/0.59 and EF-L-1.19/0.59 developed almost the same CAA capacities due to relatively weak side columns, as shown in **Table 6.4**. However, significant difference existed between the CAA capacities of EF-B-1.19/0.59S and EF-L-1.19/0.59S with enlarged column sections. EF-B-1.19/0.59S was able to develop 13% greater CAA capacity compared to EF-L-1.19/0.59S, as included in **Table 6.4**. It agrees well with the effect of reinforcement detailing on the CAA capacity of beam-column sub-assemblages, as discussed in Section 3.4.2. Besides the reinforcement detailing in the joint, side columns also affected the CAA capacity of frames. In EF-B-1.19/0.59S and EF-L-1.19/0.59S, the enlarged side columns provided stronger horizontal restraints for “anchoring” the bridging beam, thereby increasing the horizontal compression force by 48% and 90%, respectively, compared to EF-B-1.19/0.59 and EF-L-1.19/0.5. Accordingly, the CAA capacity of EF-B-1.19/0.59S was also increased in comparison with EF-B-1.19/0.59. Nonetheless, frame EF-L-1.19/0.59S only developed a CAA capacity of 71.0 kN, even smaller than EF-L-1.19/0.59.

Table 6.4: Experimental results of precast concrete frames at CAA stage

Specimen	Peak load P_c (kN)	Horizontal reaction at P_c (kN)	MJD at P_c (mm)*	Max. horizontal compression N_c (kN)	Vertical load at N_c (kN)	MJD at N_c (mm)*
EF-B-1.19/0.59	75.1	-54.3	111.2	-58.3	74.2	143.2
EF-L-1.19/0.59	74.4	-41.5	72.1	-49.8	46.5	199.1
EF-B-1.19/0.59S	80.1	-63.6	87.2	-86.2	77.3	171.1
EF-L-1.19/0.59S	71.0	-70.6	78.2	-94.8	47.1	179.1

*: MJD represents middle joint displacement.

Beyond CAA, catenary action in the bridging beam was mobilised to resist vertical load. **Table 6.5** lists the maximum vertical load and horizontal tension force at the catenary action stage. In EF-B-1.19/0.59 and EF-L-1.19/0.59, irrespective of joint

detailing, significantly lower catenary action capacities than CAA capacities were obtained due to shear failure in side beam-column joints, as listed in **Table 6.5**. With the column sizes enlarged from 250 mm to 300 mm square, EF-B-1.19/0.59S and EF-L-1.19/0.59S were capable of sustaining nearly the same catenary action capacities (see **Table 6.5**), substantially greater than the CAA capacities. Therefore, to mobilise catenary action as an effective line of defence against progressive collapse, strong side columns with sufficient flexural and shear strengths have to be provided in reinforced concrete frames.

Table 6.5: Resistances and deformations of precast concrete frames at catenary action stage

Specimen	Peak load P_t (kN)	Horizontal reaction at P_t (kN)	MJD at P_t (mm)	Max. horizontal tension N_t (kN)	Vertical load at N_t (kN)	MJD at N_t (mm)
EF-B-1.19/0.59	67.7	1.8	344.0	36.9	64.8	430.9
EF-L-1.19/0.59	67.2	38.7	419.8	57.3	56.5	457.8
EF-B-1.19/0.59S	106.7	180.0	539.8	203.6	88.8	583.9
EF-L-1.19/0.59S	103.1	160.5	516.9	200.7	100.9	590.9

Comparisons can also be made between specimens EF-B-1.19/0.59 and EF-L-1.19/0.59 and exterior frames EF-B-0.88-0.59 and EF-L-0.88-0.59 in Chapter 5. By increasing the top reinforcement ratio in the beam from 0.88% to 1.19%, EF-B-1.19/0.59 developed around 11% greater CAA capacity and 41% higher axial compression force than EF-B-0.88-0.59. At the catenary action stage, the load capacity was increased by 33%. However, tension force in the beam was limited due to shear failure of side beam-column joints. Similar results are also obtained when comparisons are made between EF-L-1.19/0.59 and EF-L-0.88-0.59.

6.3.3 Failure modes of precast frames

Similar to the failure mode of middle joint in reinforced concrete beam-column sub-assemblages (Lew et al. 2011; Su et al. 2009; Yu and Tan 2013c), one bottom bar in EF-B-1.19/0.59 ruptured at the left face of the middle joint, as shown in **Fig. 6.4(a)**, leading to a drop of the vertical load at the CAA stage, whereas the other steel bar was pulled out at the left face. Precast concrete frames EF-L-1.19/0.59, EF-B-1.19/0.59S and EF-L-1.19/0.59S exhibited similar failure modes at the middle joint,

as shown in **Figs. 6.4(b-d)**. Following the rupture of bottom reinforcement at the middle joint face, sagging moment resistance of beam sections was mobilised at the right face of the middle joint due to rotational restraint, which increased the vertical load prior to the commencement of catenary action. With increasing middle joint displacement, pull-out failure of beam bottom reinforcement was observed at the right face of the middle joint, as shown in **Figs. 6.4(b and d)**.

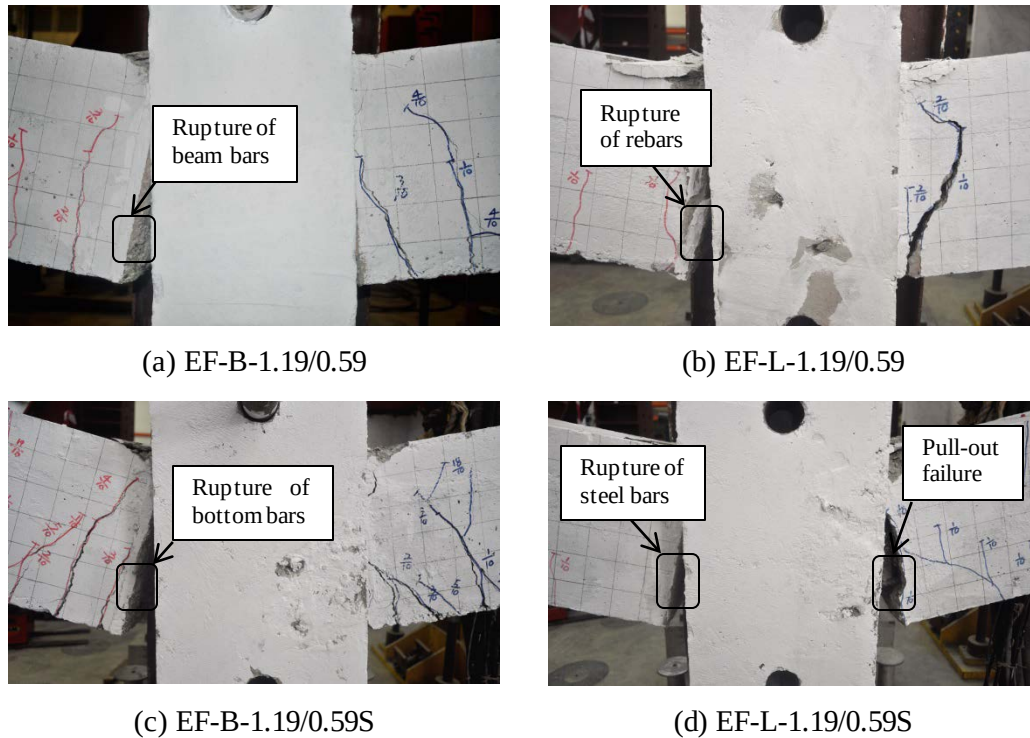
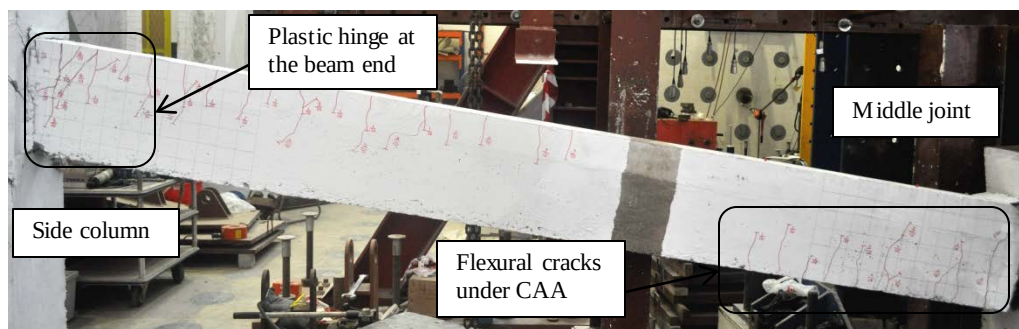
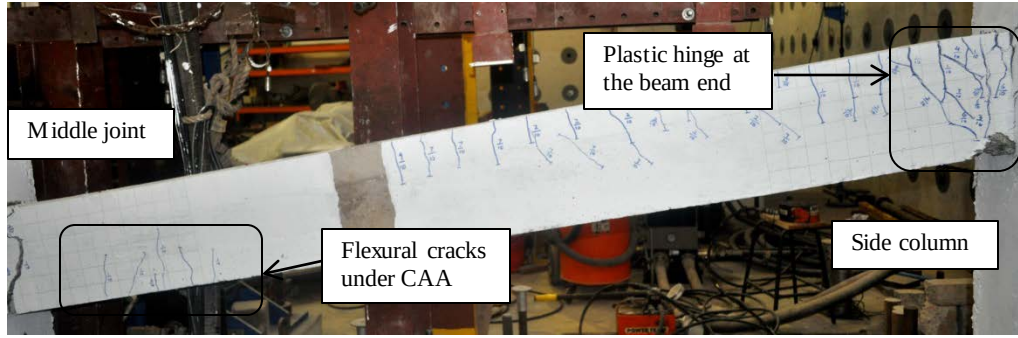
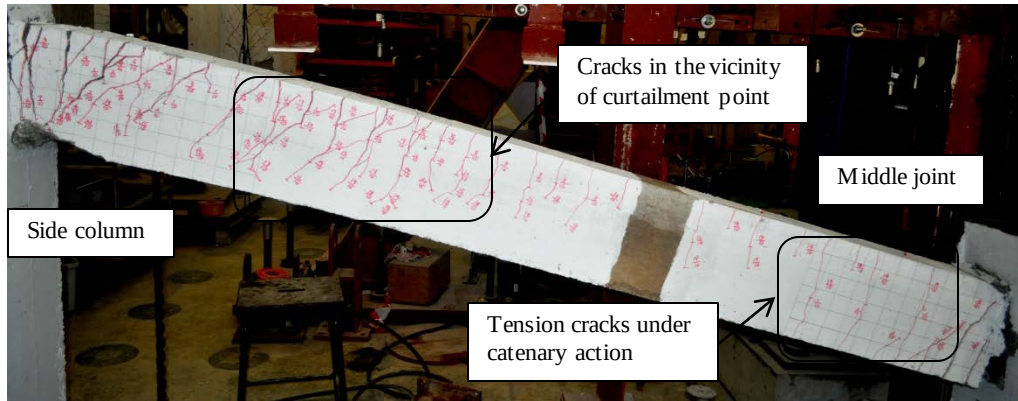


Fig. 6.4: Failure modes of middle beam-column joints

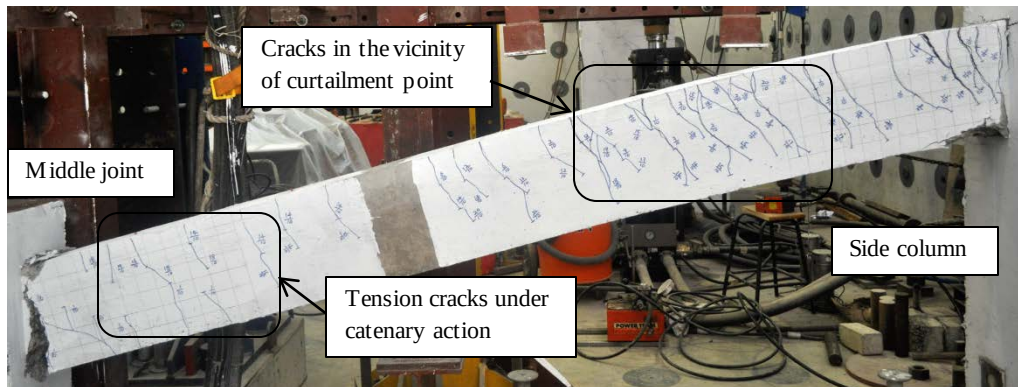




(b) EF-L-1.19/0.59



(c) EF-B-1.19/0.59S



(d) EF-L-1.19/0.59S

Fig. 6.5: Crack patterns of bridging beams

Flexural and tension cracks were observed along the bridging beam, as shown in **Fig. 6.5**. Due to relatively small column size, frames EF-B-1.19/0.59 and EF-L-1.19/0.59 exhibited similar crack patterns along the beam length, as shown in **Figs. 6.5(a and b)**. Under column removal scenarios, beam bottom longitudinal reinforcement near the middle joint was subjected to tension. Flexural cracks were formed at the bottom face of the beam in the vicinity of the middle joint. Similarly, hogging moment at the side column generated tension force in the top longitudinal reinforcement, and cracks

were observed at the top face of the beam. In EF-B-1.19/0.59 and EF-L-1.19/0.59, premature shear failure of the side beam-column joints hindered the full development of catenary action in the beams, and therefore only limited tension cracks were formed along the beam length, as shown in **Figs. 6.5(a and b)**. By enlarging the cross section of side columns in EF-B-1.19/0.59S and EF-L-1.19/0.59S, significant catenary action was mobilised in the bridging beams, as shown in **Fig. 6.3(a)**. Tension cracks developed along the beam length at catenary action stage, in particular, on the beam segment near the middle joint, as shown in **Figs. 6.5(c and d)**. Axial tension force in the beam also generated closely-spaced cracks at the curtailment point of top reinforcement, indicating the formation of a partial hinge. Similar to beam-column sub-assemblages in Section 3.4.4, these cracks substantially contributed to total vertical deformation of the frames. Eventually, instead of shear failure in the side joint, crushing of concrete occurred in the side joints, which lead to flexural failure of the side columns.

Compared to exterior frames EF-B-0.88-0.59 and EF-L-0.88-0.59 in Chapter 5, greater beam top reinforcement ratio in EF-B-1.19/0.59 and EF-L-1.19/0.59 led to severe shear cracking in side beam-column joints, even though the same amount of horizontal hoops was provided in the side joints. **Fig. 6.6** shows the propagation of shear cracks in the side joint of EF-B-1.19/0.59. Before 150 mm middle joint displacement, only one single diagonal crack was observed in the joint (**Fig. 6.6(a)**). With increasing middle joint displacement, more cracks started developing, as shown in **Figs. 6.6(b-d)**. At about 270 mm displacement, crushing of concrete took place near the compression zone of the bottom column segment (**Fig. 6.6(e)**). A further increase in the middle joint displacement did not generate more shear cracks in the joint zone, as shown in **Fig. 6.6(f)**. Instead, width of the diagonal cracks was increased by the tension force in the beam once catenary action commenced. **Fig. 6.7(a)** shows the final failure mode of the side joint in EF-B-1.19/0.59. Spalling of concrete also occurred along the column height above the side joint. **Fig. 6.7(b)** shows similar crack pattern and failure mode in the side joint of EF-L-1.19/0.59. As a result of severe shear cracks in the joint, top longitudinal reinforcement in the beam remained intact, particularly for EF-B-1.19/0.59 in which no significant plastic hinge developed at the beam end, as shown in **Fig. 6.7(a)**. Axial force in the beam also produced flexural

cracks in the column segment below the side joint. At the CAA stage, horizontal compression force in the beam pushed the side columns outwards, thereby generating cracks on the rear face of the side column, as shown in **Figs. 6.7(a and b)**.

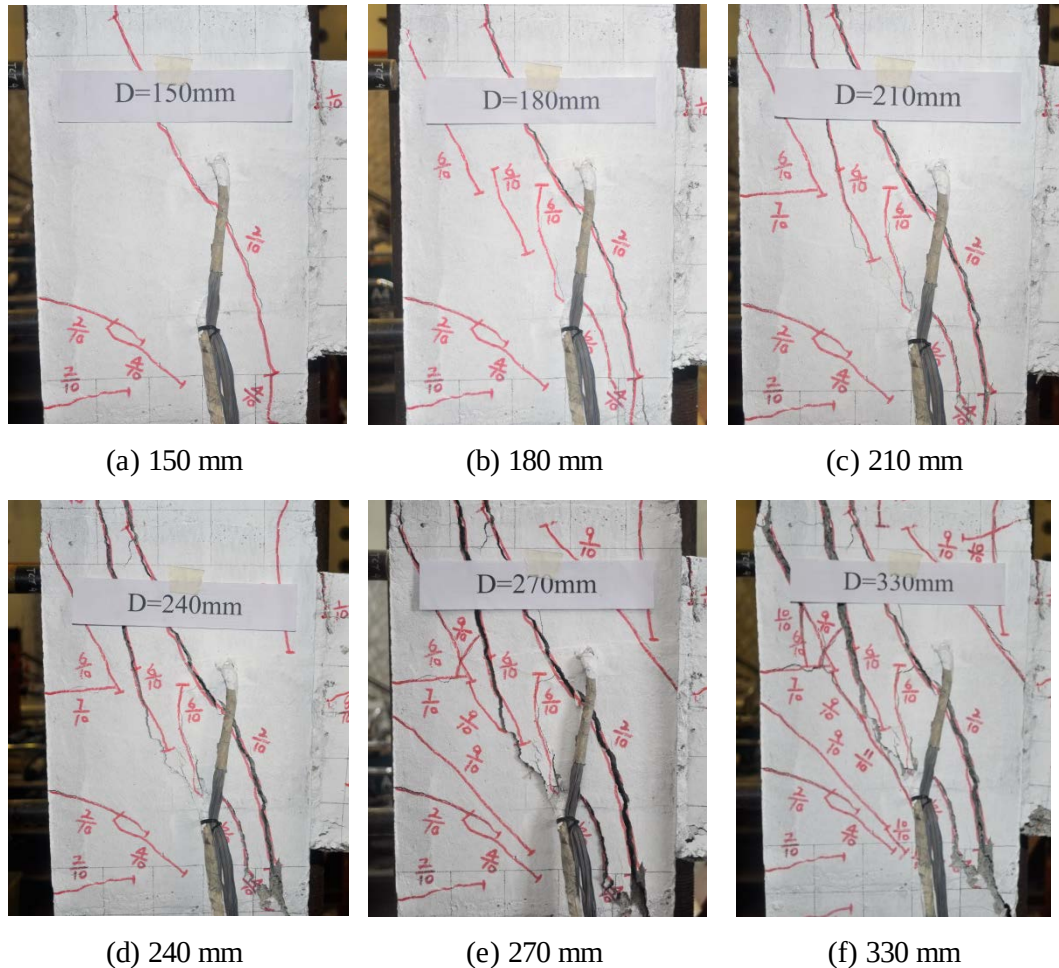
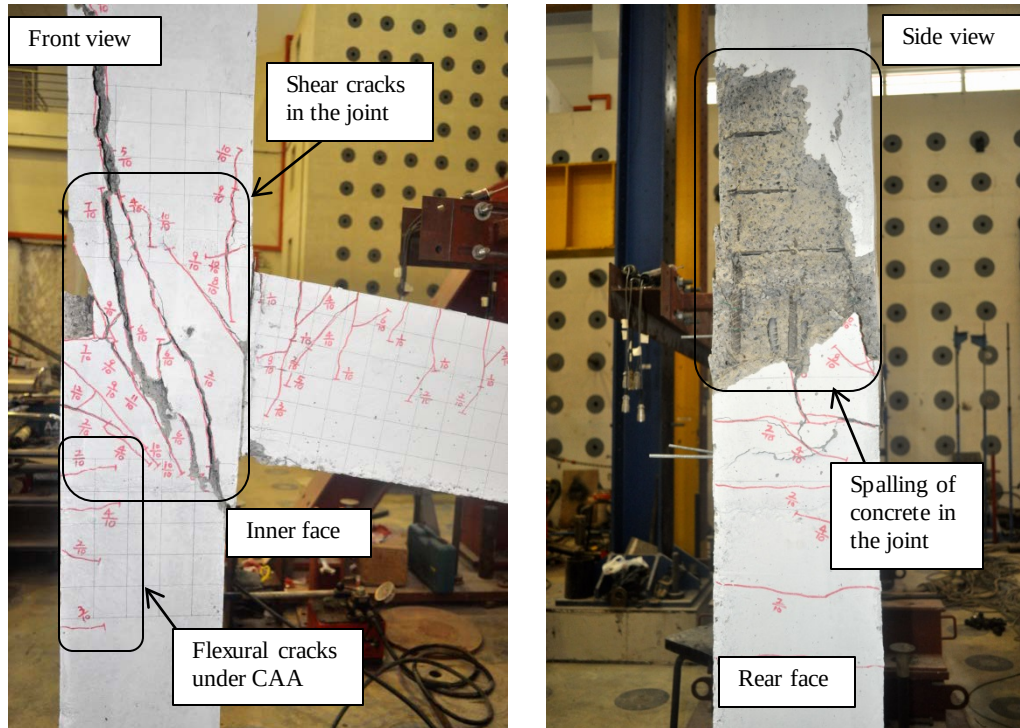


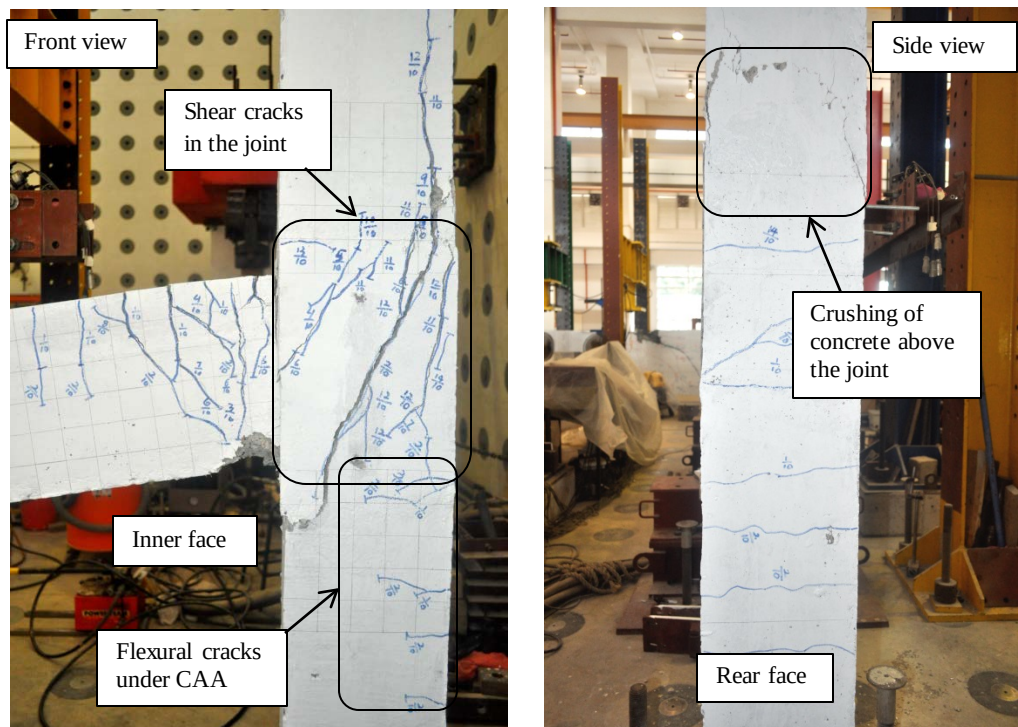
Fig. 6.6: Propagation of shear cracks in the side joint of EF-B-1.19/0.59

By enlarging the size of side columns in EF-B-1.19/0.59S and EF-L-1.19/0.59S, diagonal shear failure in the side beam-column joints was averted when subjected to CAA, as shown in **Figs. 6.7(c and d)**, even though horizontal hoops with the same diameter and spacing were used in the joint. Only limited flexural cracks were formed on the rear face of the columns at CAA stage. At the catenary action stage, significant tension force in the beam created more cracks on the inner column face towards the middle joint, and crushing of concrete eventually occurred in the compression zone of the column immediately above the side joint as shown in **Figs. 6.7(c and d)**, indicating flexural failure of the columns under combined vertical axial compression and horizontal tension forces. Thereafter, the side columns developed substantial

lateral deflections with increasing middle joint displacement, and both the vertical load on the middle joint and horizontal tension force were reduced due to column failure.



(a) EF-B-1.19/0.59



(b) EF-L-1.19/0.59

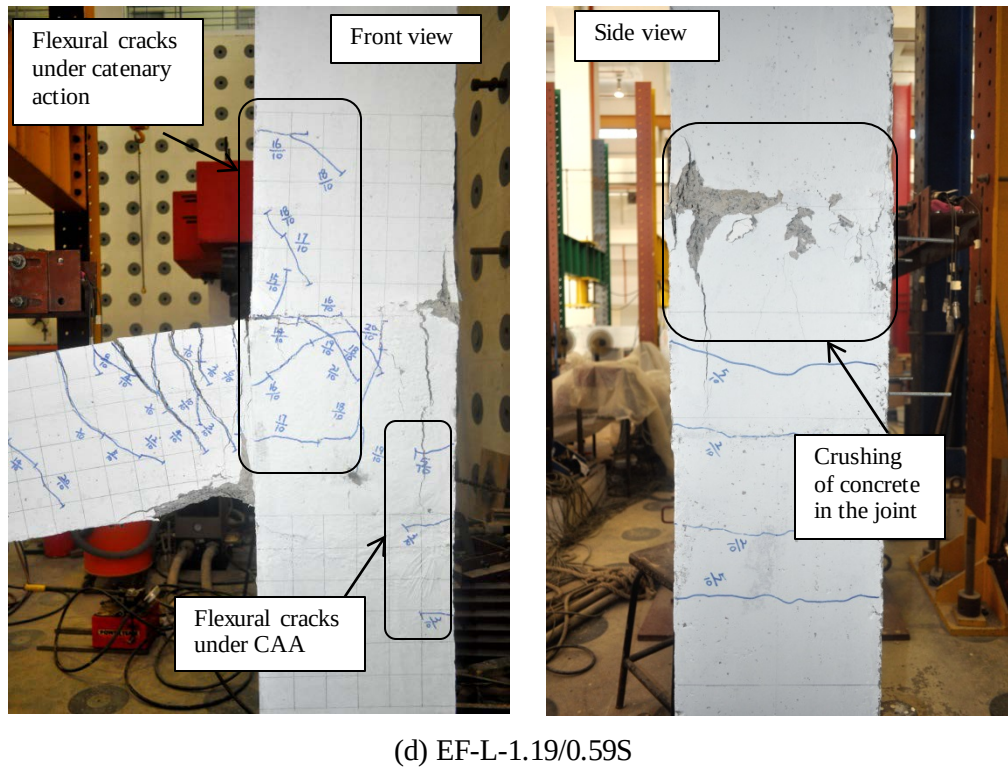
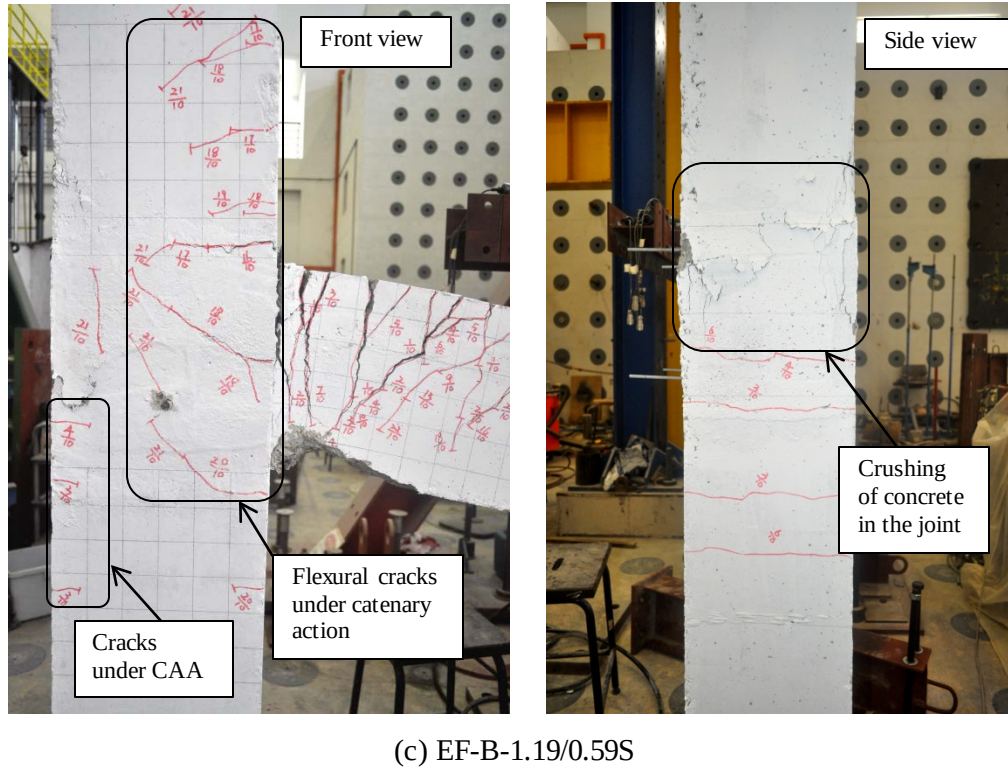


Fig. 6.7: Crack patterns and failure modes of side beam-column joints

6.3.4 Lateral deflections of side columns

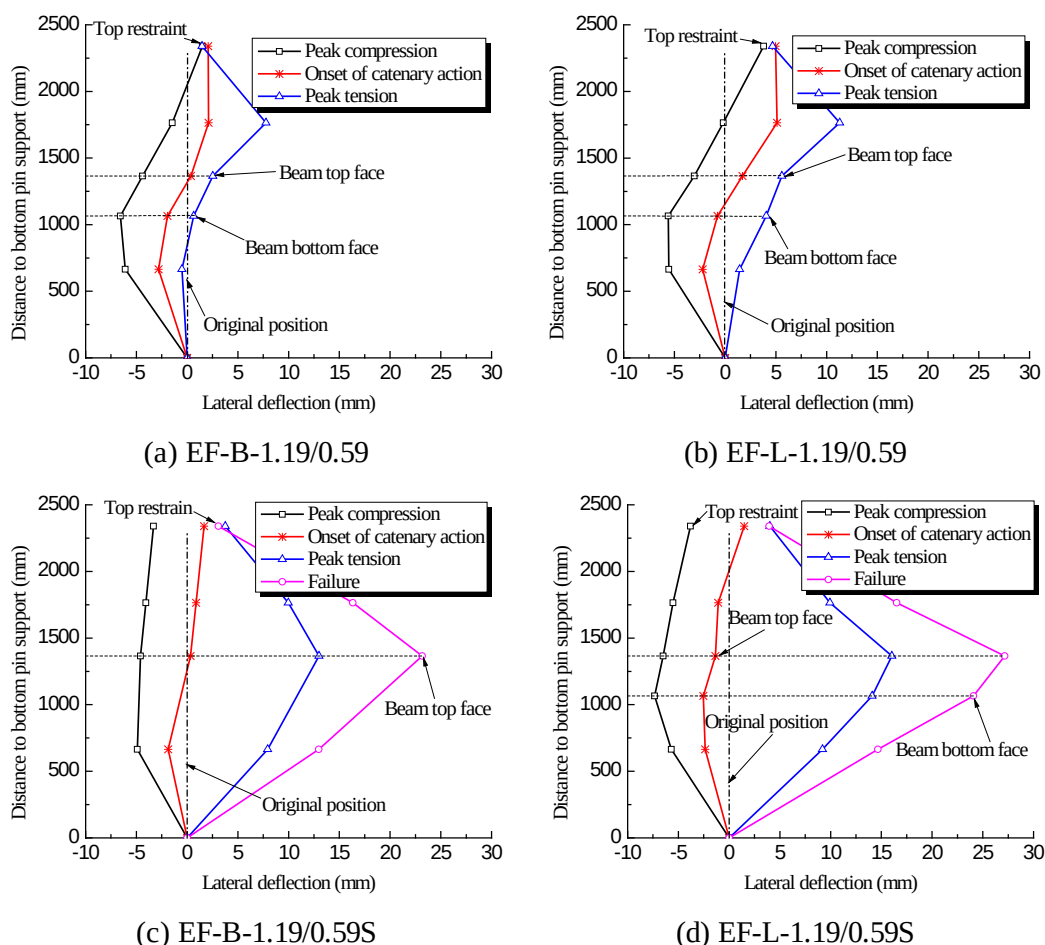


Fig. 6.8: Lateral deflections of side columns

Development of CAA and catenary action in the beam imposed horizontal compression and tension forces on the side column. Correspondingly, the side column developed significant lateral deflections when subjected to CAA and catenary action, as shown in **Fig. 6.8**. Negative value denotes deflection away from the middle joint, and positive value stands for deflection towards the middle joint. Under CAA, similar deformed profiles of the side column were obtained in all four frames (see **Fig. 6.8**). The side column was pushed outwards by the horizontal compression force in the bridging beam. A maximum negative deflection of up to 6 mm was attained at the column section corresponding to the bottom face of the beam. Following the onset of catenary action, inward deflections were induced to the side column by the horizontal tension force. The deformed shape of the side column varied with the failure mode of the frame. In EF-B-1.19/0.59 and EF-L-1.19/0.59, diagonal shear cracks in the side

beam-column joints created a kink at the column section corresponding to the beam top face, as shown in **Figs. 6.8(a and b)**. The upper column segment above the side joint exhibited significant lateral deflections towards the middle joint, whereas the lower column segment developed much less deflections as a result of diagonal shear failure in the joint. Nonetheless, frames EF-B-1.19/0.59S and EF-L-1.19/0.59S showed dramatically different lateral deflection profiles from EF-B-1.19/0.59 and EF-L-1.19/0.59, as shown in **Figs. 6.8(c and d)**. The side columns were pulled inwards by considerable horizontal tension force in the bridging beams, until flexural failure of the column occurred. The maximum positive deflection took place at the column section associated with the top face of the beam, with a value of 27.2 mm in EF-L-1.19/0.59S.

6.3.5 Shear strength of beam-column joints

Precast frames EF-B-1.19/0.59 and EF-L-1.19/0.59 exhibited shear failure in the side beam-column joints, as shown in **Figs. 6.7(a and b)**. Thus, horizontal shear force in the joints has to be determined. It is assumed that tensile stress of the beam top reinforcement at the side column interface increased linearly with increasing middle joint displacement prior to yielding, and then remained at its yield stress until final failure occurred. In accordance with force equilibrium at the side joint, as shown in **Fig. 6.9(a)**, shear force in the side joint is calculated from Eq. (6-1). **Fig. 6.9(b)** shows the shear force diagram along the column height at CAA and catenary action stages. **Fig. 6.10** shows the shear force-middle joint displacement relationships. It is obvious that the joint shear force attained its maximum value at the CAA stage, and then decreased with increasing middle joint displacement before the onset of catenary action. Thereafter, hogging moment at the beam end, equal to the difference of bending moments M_{cd} and M_{ce} (see **Fig. 6.9(c)**), was significantly reduced, which led to a reduction in the joint shear force. Therefore, the side beam-column joint was only likely to exhibit shear failure when subjected to CAA. When the cross section of side columns was enlarged to 300 mm square in frames EF-B-1.19/0.59S and EF-L-1.19/0.59S, the maximum horizontal shear force sustained by the side joint was also increased. However, shear failure did not occur due to increased shear resistance of the side joint.

$$V_{jc} = T + R_t \quad (6-1)$$

where V_{jc} is the horizontal shear force in the side beam-column joint; R_t is the reaction force in the horizontal load cell on the column top; and T is the tension force sustained by the beam top longitudinal reinforcement.

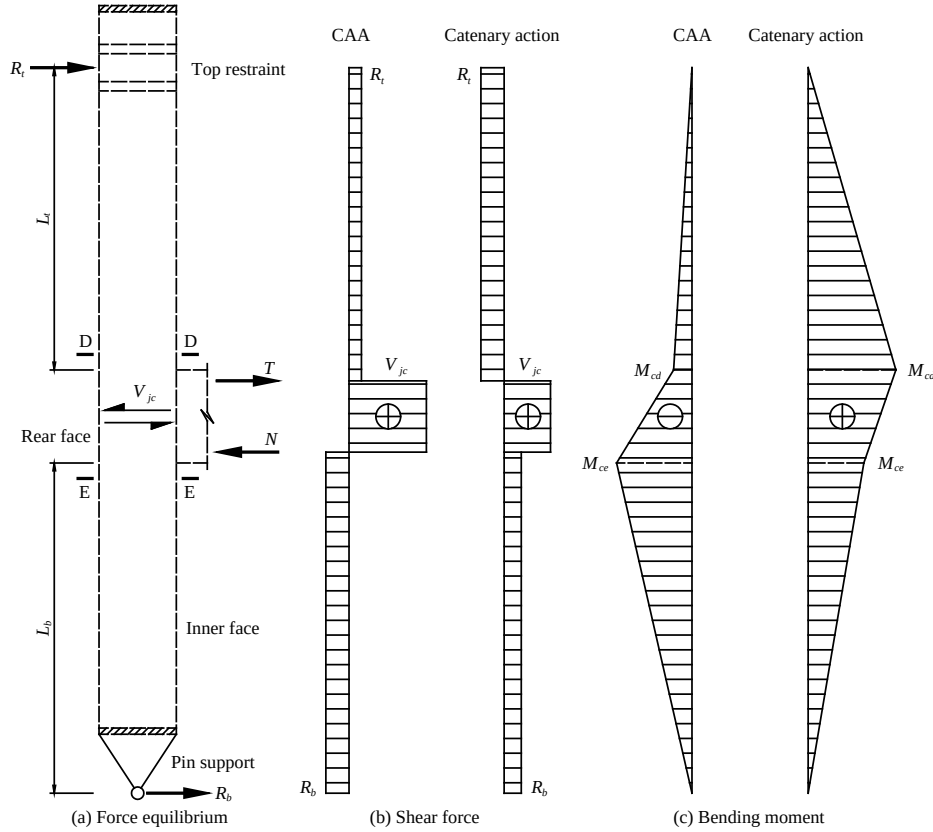


Fig. 6.9: Shear forces and bending moments on side column

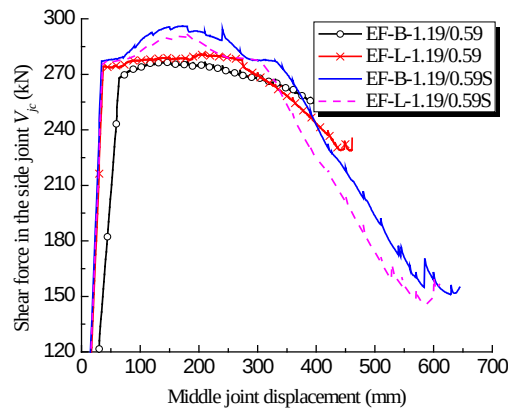


Fig. 6.10: Shear forces in side beam-column joints

Table 6.6 summarises the maximum horizontal shear force V_{jc} in the side beam-column joint at CAA stage. The hogging moment capacity of the beam end section

under pure flexural action is calculated in accordance with Eurocode 2 (BSI 2004). Moreover, the hogging moment is decomposed into a tension force T sustained by the top reinforcement and a compression force N in the compression zone. Based on the force equilibrium of the side column (see **Fig. 6.9(a)**), shear force V_{jf} in the joint is also calculated under flexural action, as expressed in Eq. (6-2). Comparisons between joint shear forces under CAA and flexural action indicate that development of CAA in the bridging beam increased the shear force in the side joint by around 8% for EF-B-1.19/0.59 and EF-L-1.19/0.59. Additionally, by enlarging the size of side columns, the shear force in the side joint of EF-B-1.19/0.59S and EF-L-1.19/0.59S was increased by 15% compared to the calculated shear force under flexural action. Therefore, shear force in the side joint was related to the horizontal compression force developed at the CAA stage. A larger compression force induced by stiffer side columns significantly increased the joint shear force.

$$V_{jf} = T_y - \frac{M_{bb}}{l_t + h + l_b} \quad (6-2)$$

where V_{jf} is the shear force in the side joint under flexural action; T_y is the yield force of tension reinforcement; M_{bb} is the moment resistance of beam end section, acting on the column face; l_t and l_b are the lengths of column segment above and below the side joint; and h is the depth of the beam.

Table 6.6: Maximum shear forces in side beam-column joints

Specimen	Shear force V_{jc} under CAA (kN)			Shear force V_{jf} under flexural action (kN)	V_{jc}/V_{jf}
	Left joint	Right joint	Average		
EF-B-1.19/0.59	276.9	276.9	276.9	255.5	1.08
EF-L-1.19/0.59	281.4	273.2	277.3	256.6	1.08
EF-B-1.19/0.59S	296.1	296.6	296.4	255.5	1.16
EF-L-1.19/0.59S	296.4	292.9	294.7	256.6	1.15

6.3.6 Flexural strength of side columns subjected to horizontal tension

In frames EF-B-1.19/0.59S and EF-L-1.19/0.59S, shear failure of side beam-column joints was averted by enlarging the size of side columns. Instead, crushing of concrete took place on the rear face of the side columns, as shown in **Figs. 6.7(c and d)**.

Bending moments at column sections D-D and E-E (see **Fig. 6.9**) are calculated based on force equilibrium, as expressed in Eqs. (6-3) and (6-4). It is assumed that the bending moment is positive when the column face towards the middle joint is in tension and negative when the rear face is subjected to tension, as shown in **Fig. 6.9(c)**. **Fig. 6.11** shows the variations of bending moments with middle joint displacement. At CAA stage, horizontal compression force on the side column generated negative bending moment along the column length (see **Fig. 6.9(c)**). In all the specimens, bending moment at section E-E was substantially larger than that at section D-D. Thus, flexural cracks were only formed at column sections below the side joint, as shown in **Figs. 6.7(a and b)**. With development of horizontal tension force at catenary action stage, negative bending moments were gradually transformed to positive values at sections D-D and E-E (**Fig. 6.9(c)**). In frames EF-B-1.19/0.59 and EF-L-1.19/0.59, shear failure of the side joints substantially limited the bending moments at sections D-D and E-E, as shown in **Figs. 6.11(a and b)**. However, the side columns of EF-B-1.19/0.59S and EF-L-1.19/0.59S sustained much greater bending moments than EF-B-1.19/0.59 and EF-L-1.19/0.59. It should be noted that section D-D generally sustained greater bending moment than section E-E, as shown in **Fig. 6.11(b and d)**, indicating that flexural failure of the side columns was more likely to be initiated at section D-D. It agrees well with the failure mode of the side columns, as shown in **Figs. 6.7(c and d)**.

$$M_{cd} = -R_t l_t \quad (6-3)$$

$$M_{ce} = -R_b l_b \quad (6-4)$$

where M_{cd} and M_{ce} are the bending moments at sections D-D and E-E, respectively; and R_b is the horizontal reaction force in the pin support at the bottom end of the side column, as shown in **Fig. 6.9**.

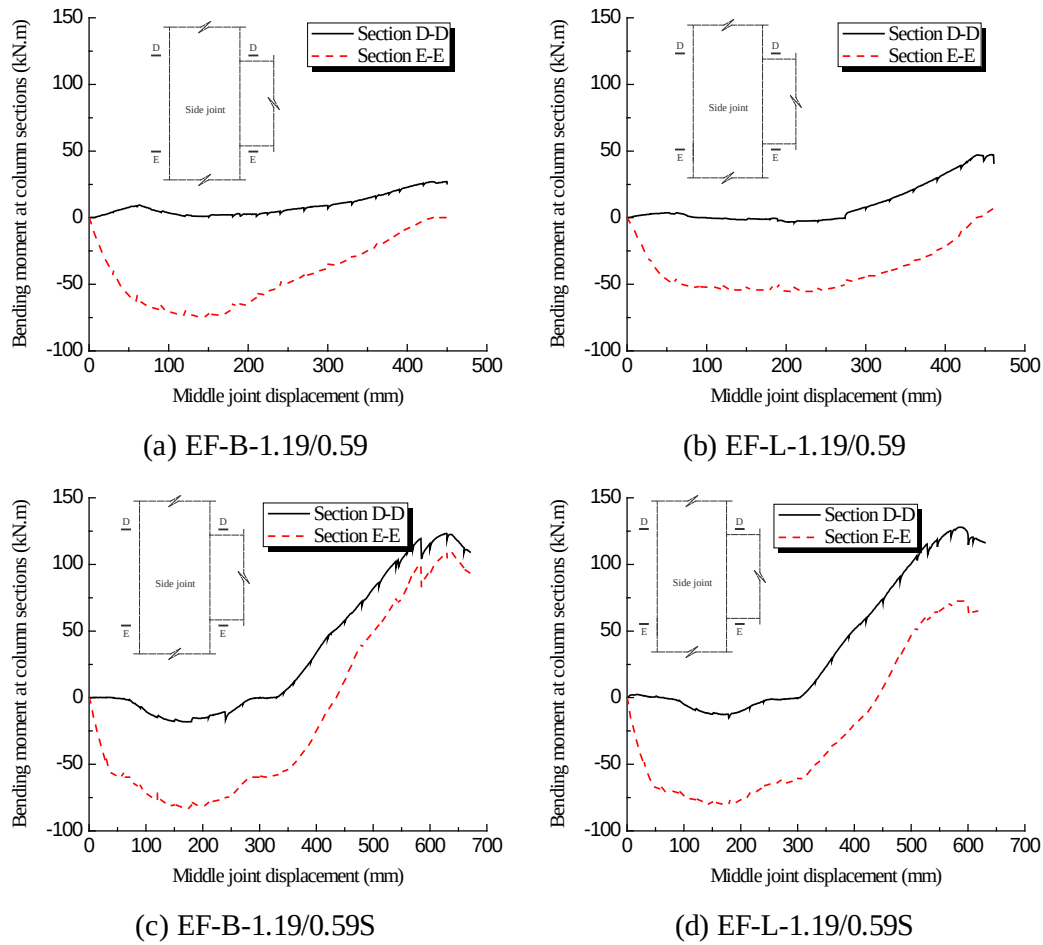


Fig. 6.11: Variations of bending moments at column sections

Table 6.7: Maximum bending moments at column sections

Specimen	Bending moment under catenary action (kN.m)		Moment capacity M_c of column section (kN.m)	M_{cd}/M_c
	M_{cd} (section D-D)	M_{ce} (section E-E)		
EF-B-1.19/0.59	27.0	1.0	75.3	0.36
EF-L-1.19/0.59	47.3	6.4	75.3	0.63
EF-B-1.19/0.59S	123.3	108.6	111.6	1.10
EF-L-1.19/0.59S	127.9	72.4	111.6	1.15

Table 6.7 lists the maximum positive bending moments at column sections D and E at catenary action stage. Moment capacity M_c of the column sections subjected to combined axial compression force and bending moment is also calculated through the axial force-bending moment interaction diagram according to Eurocode 2 (BSI 2004). As for frames EF-B-1.19/0.59S and EF-L-1.19/0.59S, the maximum bending moments sustained by column section D-D were respectively 10% and 15% greater

than the calculated moment capacity, indicating that the moment capacity of column sections was attained at failure. However, column section D-D in EF-B-1.19/0.59 and EF-L-1.19/0.59 sustained much less bending moments than their respective capacity due to premature shear failure in the side joints.

6.3.7 Variation of steel strain in side joints

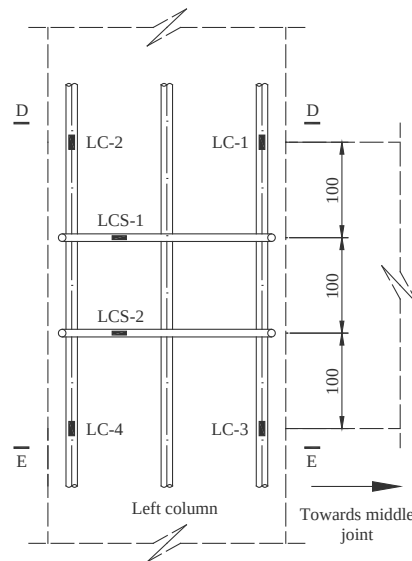


Fig. 6.12: Layout of strain gauges in the side joint

To gain a deeper insight into the behaviour of side beam-column joints under CAA and catenary action, steel strain gauges were mounted on column longitudinal reinforcement and horizontal hoops in the joint, as shown in **Fig. 6.12**. **Fig. 6.13** shows the variations of longitudinal reinforcement strains in the columns. When subjected to CAA, development of horizontal compression force in the beam generated tensile strain at LC-4 and compressive strain at LC-3. The longitudinal reinforcement at LC-1 and LC-2 showed few strains due to limited bending moment at section D-D. Following the commencement of catenary action in the beam, horizontal tension force acting on the column reversed the sign of LC-3 and LC-4. However, strains of LC-1 and LC-2 depended on the failure mode of side beam-column joints. In EF-B-1.19/0.59 and EF-L-1.19/0.59, diagonal shear cracks in the joints enabled the development of tensile strains at LC-2, as shown in **Figs. 6.13(a and b)**, indicating that longitudinal reinforcement in the columns was mobilised to sustain the joint shear force through dowel action. Nevertheless, in EF-B-1.19/0.59S

and EF-L-1.19/0.59S, tensile strain of LC-1 and compressive strain of LC-2 were mobilised (see **Figs. 6.13(c and d)**) until flexural failure of the side column occurred.

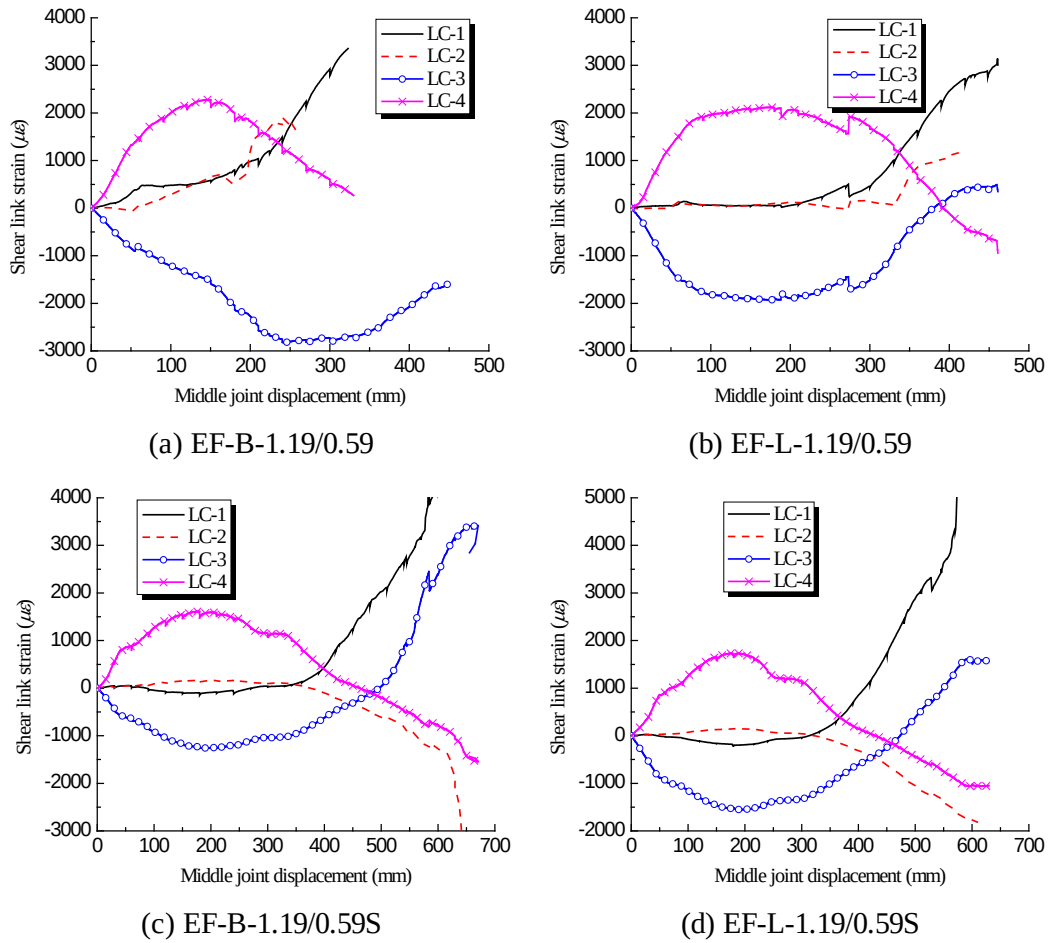


Fig. 6.13: Variations of reinforcement strains in side columns

In addition to the column longitudinal reinforcement, horizontal hoops in the side joint also contributed to the shear resistance of the joint. Strain gauges LCS-1 and LCS-2 were mounted on the hoops in the left joint, as shown in **Fig. 6.12**. Likewise, strains of the horizontal hoops in the right joint were measured by RCS-1 and RCS-2. **Fig. 6.14** shows the strain development of horizontal hoops in the joint zones. Prior to diagonal cracking in the side joints of EF-B-1.19/0.59 and EF-L-1.19/0.59 at CAA stage, horizontal joint hoops were in compression with limited compressive strains, as shown in **Figs. 6.14(a and b)**. With middle joint displacements greater than 50 mm, strains LCS-1 and RCS-1 in EF-B-1.19/0.59 and EF-L-1.19/0.59 started increasing and entered into post-yield stage, indicating the formation of shear cracks along the diagonal line of the joint panel. Comparatively, LCS-2 and RCS-2

developed the post-yield strains much later than LCS-1 and RCS-1 due to greater distances from the diagonal shear cracks. In EF-B-1.19/0.59S and EF-L-1.19/0.59S, the enlarged size of the side columns prevented shear failure in the joint. Most horizontal hoops were in compression at CAA stage, as shown in **Figs. 6.14(c and d)**. The increase in tensile strains of joint hoops prior to failure was induced by horizontal tension force transmitted to the joints at catenary action stage. Therefore, all the strains developed rapidly into post-yield stage.

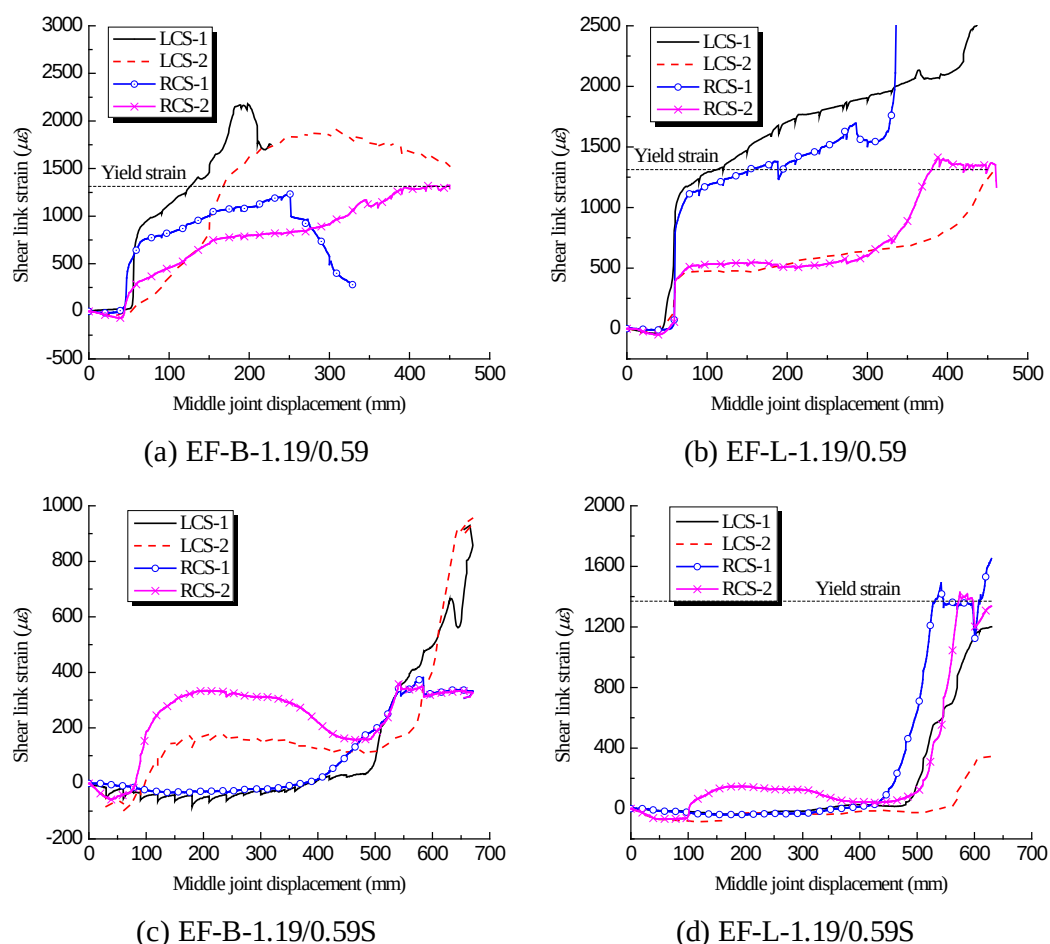


Fig. 6.14: Strains of horizontal hoops in side beam-column joints

Based on the strains of reinforcement, actions in the side joint could be simplified at CAA and catenary action stages, as shown in **Fig. 6.15**. Under CAA, horizontal compression force developed in the bridging beam, but bending moment at cross section E-E was negligible (see **Fig. 6.11**). Thus, a strut mechanism as shown in **Fig. 6.15(a)** was significant. Development of the diagonal compressive strut caused shear cracking in the joints of frames EF-B-1.19/0.59 and EF-L-1.19/0.59 (see **Figs. 6.7(a**

and b)), which mobilised horizontal hoops to resist shear force and to confine concrete in the joint zones (**Figs. 6.14(a and b)**). Eventually, shear failure occurred in the side joints of frames EF-B-1.19/0.59 and EF-L-1.19/0.59. In EF-B-1.19/0.59S and EF-L-1.19/0.59S, joint shear failure was prevented by enlarging the cross sections of the side columns and catenary action developed in the beams to sustain vertical load. At catenary action stage, tension force in the beams was mainly transmitted to the joints through bond stresses between reinforcement and the concrete, as shown in **Fig. 6.15(b)**. Therefore, tensile strains of the horizontal hoops were increased to resist the horizontal tension force in the joints, as shown in **Figs. 6.14(c and d)**.

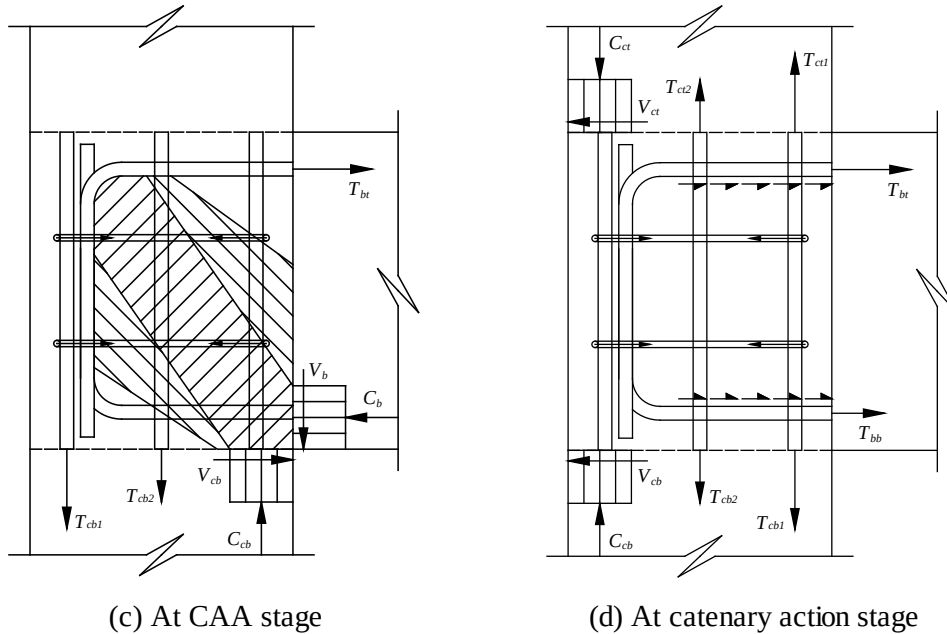


Fig. 6.15: Actions in side beam-column joint

6.4 Summary

This chapter presents an experimental study on the behaviour of four exterior precast concrete frames subject to column removal scenarios. Two types of reinforcement detailing were used in the beam-column joints. The effect of the side column dimensions on the resistance and failure mode was also investigated. Based on force equilibrium, shear force in the joints and bending moments at the column sections corresponding to the top and bottom faces of the bridging beam were calculated.

Strains of longitudinal reinforcement and horizontal hoops in the joints were also measured. According to experimental results, the following conclusions are obtained.

(1) All the precast concrete frames were able to develop CAA in the bridging beam under column removal scenarios. In comparison with frames EF-B-1.19/0.59S and EF-L-1.19/0.59S with enlarged column cross section, relatively slender columns in EF-B-1.19/0.59 and EF-L-1.19/0.59 limited the development of horizontal compression forces at CAA stage.

(2) Little catenary action was mobilised in frames EF-B-1.19/0.59 and EF-L-1.19/0.59 due to premature shear failure in the side beam-column joints. By increasing the size of side columns, frames EF-B-1.19/0.59S and EF-L-1.19/0.59S developed significant catenary action, which even surpassed the CAA capacities.

(3) In EF-B-1.19/0.59 and EF-L-1.19/0.59, development of horizontal compression force at CAA stage increased the shear force in the side joints by 8% compared to the value calculated under flexural action, which induced severe diagonal shear cracking in the joints. Frames EF-B-1.19/0.59S and EF-L-1.19/0.59S provided much greater joint shear resistance due to larger cross section of the side columns. Therefore, shear failure did not occur in the joints, even though the maximum shear force was increased by around 15% than that under pure flexural action.

(4) Mobilisation of significant horizontal tension force in EF-B-1.19/0.59S and EF-L-1.19/0.59S caused flexural failure of the side columns in the vicinity of the side joints at catenary action stage. At failure, the moment capacity of column sections was attained as a result of horizontal tension force acting on the column. With increasing middle joint displacement, horizontal tension force could not increase further.

To prevent shear failure in the beam-column joint and flexural failure of the side column under column removal scenarios, horizontal forces have to be considered in the design of columns and joints in exterior frames. However, the maximum horizontal compression and tension forces developed in the bridging beam vary with the column dimensions. By increasing the cross section of the side column, both are expected to be increased due to stiffer horizontal and rotational restraints provided by the column for the bridging beam, until the upper bound values are obtained.

Therefore, further experimental and analytical studies are necessary to establish the relationship between the column size and horizontal forces that are able to develop in the frame.

CHAPTER 7 ANALYTICAL MODEL FOR COMPRESSIVE ARCH ACTION OF BEAM-COLUMN SUB-ASSEMBLAGES

7.1 Introduction

Compressive arch action (CAA) represents a mechanism through which the resistance of horizontally restrained beams is greatly enhanced due to the development of axial compression force in the beam. A relatively small vertical deflection of the bridging beam will lead to rotation and outward movement of the beam ends against stiff boundaries, which in turn generates compressive thrusts in between the flexural compression zones at the middle and end joints (Welch et al. 1999). These thrusts will result in an enhancement to moment resistance via arching effect (Park and Gamble 2000).

Early researchers (Keenan 1969; Park 1964) proposed rigid-plastic analytical models for predicting the peak capacity and corresponding mid-span deflection of one-way slabs subjected to CAA. The term “rigid-plastic” refers to the assumption that plastic hinges have developed at specified points and the one-way slab typically remains rigid except for the elastic axial shortening and the inelastic rotation of plastic hinges (Welch et al. 1999). These assumptions form the basis of the following mechanical models for predicting the CAA. Nevertheless, Park’s model (Park 1964) does not provide a clear determination of the stress state of the steel bars in the compression zones, thereby hindering the precise prediction of the CAA capacity of reinforced concrete beams and slabs. In analysing the response of reinforced concrete slabs, Guice et al. (1989) assumed that compressive concrete attained its ultimate strain and calculated the compressive steel strain by using the plane-section assumption. This method requires the predetermined peak capacity deflection which leads to inaccuracy in predicting the response of slabs. Yu and Tan (2013a) proposed an analytical procedure to predict the CAA capacity of beam-column sub-assemblages under column removal scenarios and incorporated the rotational stiffness and connection gap of supports into the model. It is capable of predicting the CAA capacity and peak axial compression of sub-assemblages with reasonable accuracy. However, in Yu and Tan’s model, the strain of extreme compression fibre is assumed

to be constant at the crushing strain of concrete. This assumption results in a reduction in tensile strain of reinforcement, which violates the realistic variation of the tensile strain (Yu 2012). Besides, application of the equivalent rectangular concrete block in the model leads to an overestimation of vertical load at the initial stage. Therefore, further improvement of the model is still needed.

In this chapter, in accordance with previous analytical studies, a new analytical model is proposed for beam-column sub-assemblages incorporating the actual stress state of compressive and tensile reinforcing bars. It can also consider tensile strength of engineered cementitious composites (ECC). The mechanical model is calibrated with experimental results of conventional concrete beam-column sub-assemblages, and is utilised for predicting the behaviour of ECC sub-assemblages under column removal scenarios. Finally, a series of parametric studies is conducted to investigate factors that govern the CAA behaviour of beam-column sub-assemblages.

7.2 Development of the Analytical Model

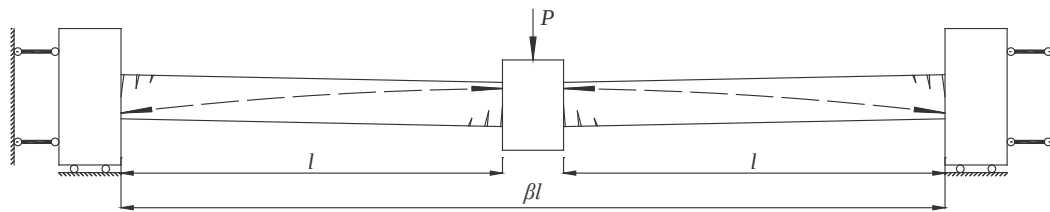


Fig. 7.1: Geometric configuration of horizontally restrained beams

When subject to removal of a supporting middle column, the affected beam-column sub-assemblage is able to develop CAA between the compression zones of the bridging beam when the middle joint displacement is less than one beam depth, as shown in **Fig. 7.1**. Based on rigid-plastic assumption, the system can be idealised as two rigid beam segments and four zero-length hinges concentrated at the faces of the middle joint and column stubs (see **Fig. 7.2**). The hinges at the interfaces sustain all flexural rotations, whereas the beam segments only experience axial shortening generated by net compression force in the beam (Park and Gamble 2000). However, in either the prototype structure or the test setup, horizontal restraints on the beam-column sub-assemblage are far from fully fixed with infinite stiffness, and it is necessary to consider the actual horizontal movements and rotations of the supports

in the model. Comparatively, the vertical support on both beam ends plays a minor role in the CAA capacity of the sub-assemblages. Therefore, only the effect of the horizontal and rotational restraints is incorporated in the analytical model, as shown in Fig. 7.2.

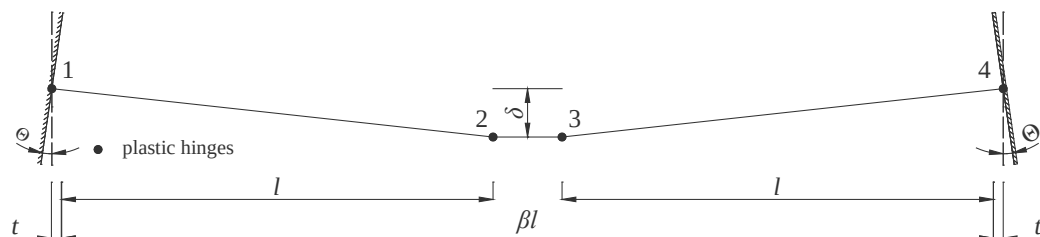


Fig. 7.2: Plastic hinge mechanism of beam-column sub-assemblages

7.2.1 Constitutive models

7.2.1.1 Steel reinforcement

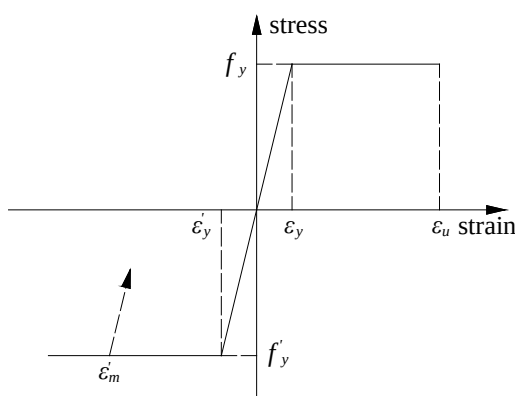


Fig. 7.3: Stress-strain relationship of steel bars

Experimental tests on beam-column sub-assemblages demonstrate that strains of beam bottom longitudinal reinforcement at the middle joint face and top reinforcement at the end column stub face keep increasing under column removal scenarios, whereas top steel bars at the middle joint and bottom bars at the end column stub are subjected to compressive strains at the initial stage, and are gradually shifted to tension with increasing middle joint displacement (Yu and Tan 2010a). Therefore, the constitutive law of reinforcing bars has to be defined prior to the derivation of the analytical model. Fig. 7.3 shows the elastic-perfectly-plastic stress-strain relationship of steel bars, with yield strains of ϵ_y in tension and ϵ'_y in compression. As the strain

of compressive steel reinforcement is reduced at large deformations, a linear unloading phase is defined in the compressive branch of the stress-strain curve, with its stiffness equal to the elastic modulus of steel. Correspondingly, the stress-strain relationship of steel reinforcement in tension and compression is expressed in Eqs. (7-1) and (7-2).

$$\begin{aligned}\sigma_s &= E_s \varepsilon_s, \text{ if } \varepsilon_s < \varepsilon_y \\ \sigma_s &= f_y, \text{ if } \varepsilon_s \geq \varepsilon_y\end{aligned}\quad (7-1)$$

$$\begin{aligned}\sigma'_s &= E_s \varepsilon'_s, \text{ if } \varepsilon'_s < \varepsilon'_y \text{ and } \varepsilon'_m < \varepsilon'_y \\ \sigma'_s &= f'_y, \text{ if } \varepsilon'_s \geq \varepsilon'_y \text{ and } \varepsilon'_m \leq \varepsilon'_s \\ \sigma'_s &= f'_y - (\varepsilon'_m - \varepsilon'_s) E_s, \text{ if } \varepsilon'_m \geq \varepsilon'_y \text{ and } \varepsilon'_m > \varepsilon'_s\end{aligned}\quad (7-2)$$

where σ_s and σ'_s are the tensile and compressive stresses of reinforcement, respectively; E_s is the modulus of elasticity of steel bars; ε_s and ε'_s are the tensile and compressive strains of reinforcement; ε_y and ε'_y are the yield strains of steel in tension and compression, respectively; f_y and f'_y are the tensile and compressive yield strengths of steel reinforcement, respectively; and ε'_m is the maximum compressive strain that reinforcement has attained.

7.2.1.2 Concrete

In previous analytical studies on horizontally-restrained reinforced concrete slabs, equivalent rectangular concrete stress block was used to determine the load-displacement curves of slabs (Guice and Rhomberg 1988; Park and Gamble 2000). A similar approach was used for beam-column sub-assemblages under column removal scenarios (Yu and Tan 2013a). However, severe crushing of concrete occurred in the compression zone of bridging beams at CAA stage, as discussed in Section 3.4.5. Thus, under large deformations, rectangular concrete stress block is not valid. Realistic stress-strain relationship of concrete has to be used to consider the compressive stress of concrete when subjected to large strains. In deriving the analytical model, the constitutive model for unconfined concrete in uniaxial compression proposed by Mander et al. (1988) is employed to calculate the compression force sustained by concrete, as expressed in Eq. (7-3). **Fig. 7.4** shows

the stress-strain curve of unconfined concrete. Unlike rectangular concrete stress block, the model is able to consider the compressive stress and softening behaviour of concrete when compressive strain is greater than 0.0035.

$$\begin{aligned}\sigma &= \frac{f'_c \varepsilon r}{r - 1 + \varepsilon^r} \\ r &= \frac{E_c}{E_c - E_{sec}} \\ E_c &= 5000 \sqrt{f'_c} \\ E_{sec} &= \frac{f'_c}{\varepsilon_c}\end{aligned}\tag{7-3}$$

where σ and ε are the compressive stress and strain of concrete, respectively; E_c and E_{sec} are the tangent and secant moduli of elasticity of concrete, respectively; f'_c is the cylinder compressive strength of concrete; and ε_c is the strain corresponding to f'_c and is generally assumed to be 0.002.

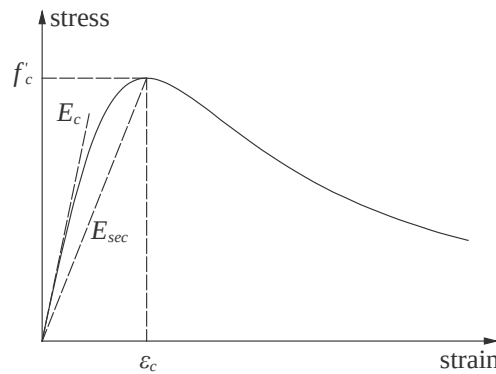


Fig. 7.4: Constitutive model for concrete in compression

7.2.1.3 Engineered cementitious composites

Different from conventional concrete with limited ductility in tension, ECC exhibits tensile strain-hardening behaviour and superior strain capacity (Fischer and Li 2002a; Fischer and Li 2003; Li 2003). The contribution of its tensile strength to the resistance of beam-column sub-assemblages can be considered at CAA stage. When subjected to uniaxial tension, ECC remains in the elastic stage and its tensile stress increases linearly with strain prior to cracking. After the first cracking, a hardening stage develops with multi-cracking along the tensile coupon length. Thus, the stress-strain

relationship of ECC can be simplified as a bilinear curve with hardening behaviour, as shown in **Fig. 7.5(a)**. Eq. (7-4) expresses the tensile stress-strain relationship of ECC (Yuan et al. 2013).

$$\begin{aligned} \sigma &= \frac{\sigma_{tc}}{\varepsilon_{tc}} \varepsilon, \text{ if } \varepsilon \leq \varepsilon_{tc} \\ \sigma &= \sigma_{tc} + (\sigma_{tu} - \sigma_{tc}) \frac{\varepsilon - \varepsilon_{tc}}{\varepsilon_{tu} - \varepsilon_{tc}}, \text{ if } \varepsilon_{tc} < \varepsilon \leq \varepsilon_{tu} \end{aligned} \quad (7-4)$$

where σ_{tc} and ε_{tc} are the first cracking strength and strain of ECC in tension; σ_{tu} is the ultimate tensile strength and ε_{tu} is the associated tensile strain at σ_{tu} .

Under compression, a bilinear stress-strain relationship of ECC is assumed in the ascending branch of compressive stress (Maalej and Li 1994), as shown in **Fig. 7.5(b)**. After attaining the compressive strength, the stress linearly drops to 50% of the compressive strength, and then decreases slowly with increasing middle joint displacement. Thus, a constant compressive stress is assumed with increasing compressive strain. Eq. (7-5) expresses the compressive stress-strain relationship of ECC (Yuan et al. 2013).

$$\begin{aligned} \sigma &= 2 \frac{\sigma_{c0}}{\varepsilon_{c0}} \varepsilon, \text{ if } 0 \leq \varepsilon \leq \frac{1}{3} \varepsilon_{c0} \\ \sigma &= \frac{2}{3} \sigma_{c0} + \frac{\sigma_{c0}}{2 \varepsilon_{c0}} (\varepsilon - \varepsilon_{c0}), \text{ if } \frac{1}{3} \varepsilon_{c0} \leq \varepsilon \leq \varepsilon_{c0} \\ \sigma &= \sigma_{c0} + (\sigma_{cu} - \sigma_{c0}) \frac{\varepsilon - \varepsilon_{c0}}{\varepsilon_{cu} - \varepsilon_{c0}}, \text{ if } \varepsilon_{c0} \leq \varepsilon \leq \varepsilon_{cu} \\ \sigma &= \sigma_{cu}, \text{ if } \varepsilon_{cu} \leq \varepsilon \end{aligned} \quad (7-5)$$

where σ_{c0} is the compressive strength of ECC; ε_{c0} is the compressive strain at σ_{c0} ; σ_{cu} is the ultimate compressive strength in the post-peak branch; ε_{cu} is the ultimate compressive strain. It is assumed that $\sigma_{cu} = 0.5\sigma_{c0}$ and $\varepsilon_{cu} = 1.5\varepsilon_{c0}$.

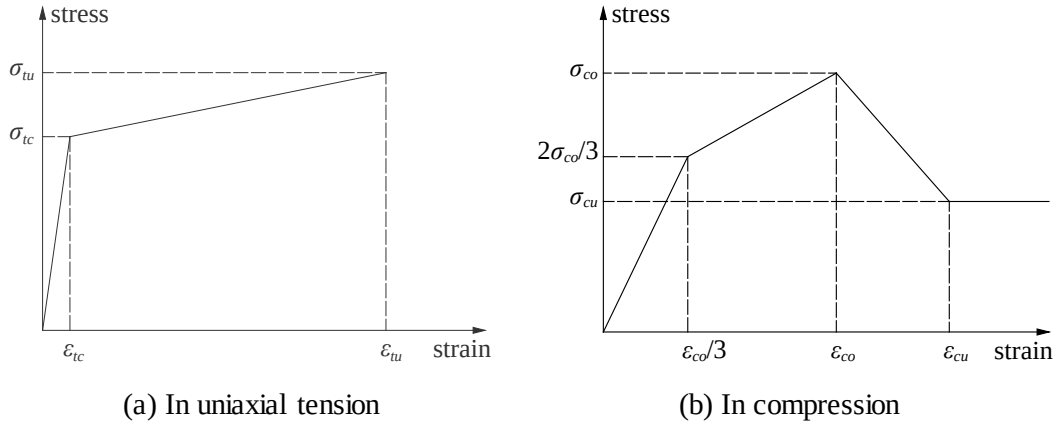


Fig. 7.5: Stress-strain relationship of ECC

7.2.2 Equilibrium condition

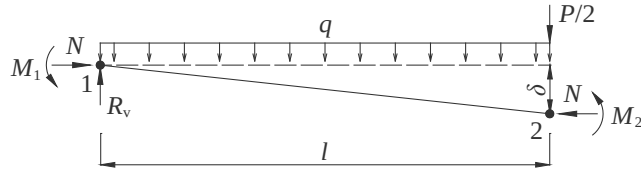


Fig. 7.6: Free-body diagram of the single-span beam

Due to symmetry of geometry and loading, only the single-span beam is extracted from the plastic hinge mechanism, as shown in **Fig. 7.6**. Through equilibrium (Yu and Tan 2013a), the correlation between the applied vertical force on the middle joint and corresponding vertical deflection can be established as:

$$P = \frac{2(M_1 + M_2 - N\delta - ql^2/2)}{l} \quad (7-6)$$

where M_1 and M_2 are the bending moments at sections 1 (end support face) and 2 (middle joint face), respectively; N is the beam axial compression force; q is the self-weight of the affected beam; l is the clear span of the beam; P is the concentrated point force on the middle beam-column joint; and δ is the deflection of the middle joint.

To derive the load-displacement curve of the beam, axial compression force N and bending moments M_1 and M_2 have to be quantified based on the deformed geometry shown in **Fig. 7.6**. Therefore, cross-sectional analysis at sections 1 and 2 are necessary. **Fig. 7.7** shows the strain profile and internal force equilibrium at beam

sections 1 and 2. It is assumed that the plane-section assumption is valid at large deformation stage. Thus, a linear variation of strains is obtained across the section. Correspondingly, force equilibrium at the section can be established through the strain profile across the section and the respective material models for reinforcing bars, conventional concrete and ECC.

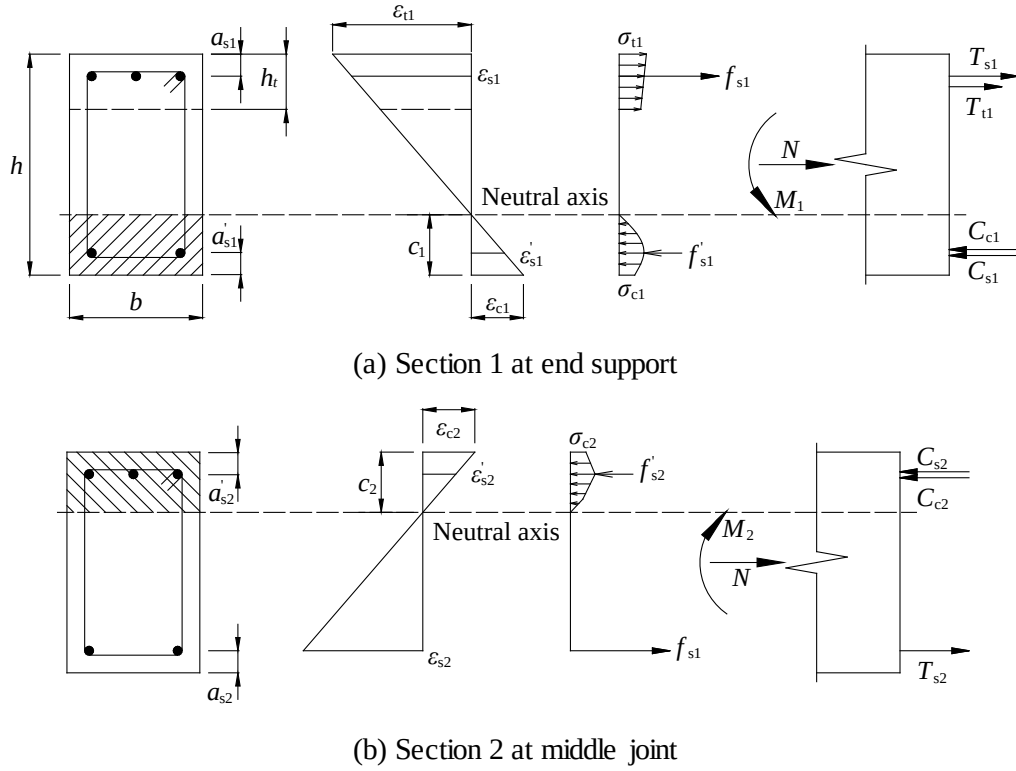


Fig. 7.7: Force equilibrium of beam sections

As ECC is utilised in the structural topping of beam-column sub-assemblages, its tensile strength has to be considered at section 1 (within the depth of h_t) subjected to hogging moment. Compared to conventional concrete sub-assemblages, an additional term is added to the equilibrium condition of internal forces at section 1 of ECC sub-assemblages, as expressed in Eq. (7-7).

$$T_{t1} = b \int_{h-c_1-h_t}^{h-c_1} \sigma_{t1} dx \quad (7-7)$$

where T_{t1} is the tension force sustained by the ECC topping; b is the width of the beam section; h is the depth of the beam; h_t is the thickness of the ECC topping; c_1 is the neutral axis depth at section 1; and σ_{t1} is the tensile stress of the ECC topping.

In the compression zone, the unconfined stress-strain model proposed by Mander et al. (1988) is used to calculate the compression force instead of the equivalent rectangular concrete stress block. Based on the plane-section assumption, compressive strains of concrete fibres can be determined and associated compressive stresses can be calculated using the constitutive model for concrete. Integration of compressive stress across the compression zone is necessary at section 1 to quantify the compression force sustained by the concrete, as expressed in Eq. (7-8).

$$C_{c1} = b \int_0^{c_1} \sigma_{c1} dx \quad (7-8)$$

$$C_{c2} = b \int_0^{c_2} \sigma_{c2} dx \quad (7-9)$$

where C_{c1} and C_{c2} are the compression forces in concrete; c_1 and c_2 are the respective neutral axis depths at sections 1 and 2; σ_{c1} and σ_{c2} are the respective compressive stresses of concrete.

A similar procedure can be used to calculate the internal forces at section 2. It is notable that at section 2 only the beam longitudinal reinforcement sustains tension force, whereas ECC topping is subjected to compression. The material model for ECC is employed in Eq. (7-9) to compute the compression force.

In the analytical model, axial compression force in the beam is assumed to be constant along the beam length (Park and Gamble 2000; Yu and Tan 2013a). From the internal force equilibrium illustrated in **Fig. 7.7**, axial compression force at section 1 is expressed in Eq. (7-10). Bending moment at section 1 can be calculated. For ECC sub-assemblages, total hogging moment is a summation of four terms, namely, contributions of compressive concrete, compressive reinforcement, tensile reinforcement and tensile ECC, as expressed in Eq. (7-11). Sagging moment at section 2 is only contributed by three terms, as expressed in Eq. (7-12). Contribution of tensile ECC is eliminated, as the ECC topping is in compression at section 2. The material model for ECC in compression has to be used in Eq. (7-12) for ECC sub-assemblages.

$$N = C_{c1} + C_{s1} - T_{s1} - T_{t1} = C_{c2} + C_{s2} - T_{s2} \quad (7-10)$$

$$M_1 = b \int_0^{c_1} (0.5h - c_1 + x) \sigma_{c1} dx + C_{s1} (0.5h - a'_{s1}) + T_{s1} (0.5h - a_{s1}) + b \int_{h-c_1-h_t}^{h-c_1} (x + c_1 - 0.5h) \sigma_{c1} dx \quad (7-11)$$

$$M_2 = b \int_0^{c_2} (0.5h - c_2 + x) \sigma_{c2} dx + C_{s2} (0.5h - a'_{s2}) + T_{s2} (0.5h - a_{s2}) \quad (7-12)$$

in which N is the net compression force in the beam; C_{s1} and C_{s2} are the compression forces in the compressive reinforcement; T_{s1} and T_{s2} are the tension forces sustained by reinforcement; a_{s1} and a_{s2} are the distances from the centroid of tension reinforcement to the extreme tension fibre; a'_{s1} and a'_{s2} are the distances from the centroid of compression reinforcement to the extreme compression fibre, at sections 1 and 2, respectively.

The single-span beam is assumed to sustain elastic compressive strain under CAA due to the axial compression force. The compression force in the beam can be quantified by its compressive strain (Yu 2012), as expressed in Eq. (7-13).

$$N = bhE_c \varepsilon_b \quad (7-13)$$

in which ε_b is the axial compressive strain of the beam; E_c is the equivalent modulus of elasticity of the section, approximately equal to $4700\sqrt{f'_c}$ (ACI 2005). For sections with several concrete layers of different strengths, the elastic modulus can be calculated by $E_c = \Sigma(E_{ci}b_ih_i)/bh$.

Through internal force equilibrium of the single-span beam, the relationship between the axial compression force and bending moment at the end section is established. Nonetheless, in order to calculate the internal forces, neutral axis depths c_1 and c_2 , strains of compressive reinforcement ε'_{s1} and ε'_{s2} , strains of tension steel ε_{s1} and ε_{s2} , concrete compressive strains ε_{c1} and ε_{c2} , and axial strain ε_b of the beam have to be quantified at a certain middle joint displacement δ by means of compatibility condition.

7.2.3 Compatibility condition

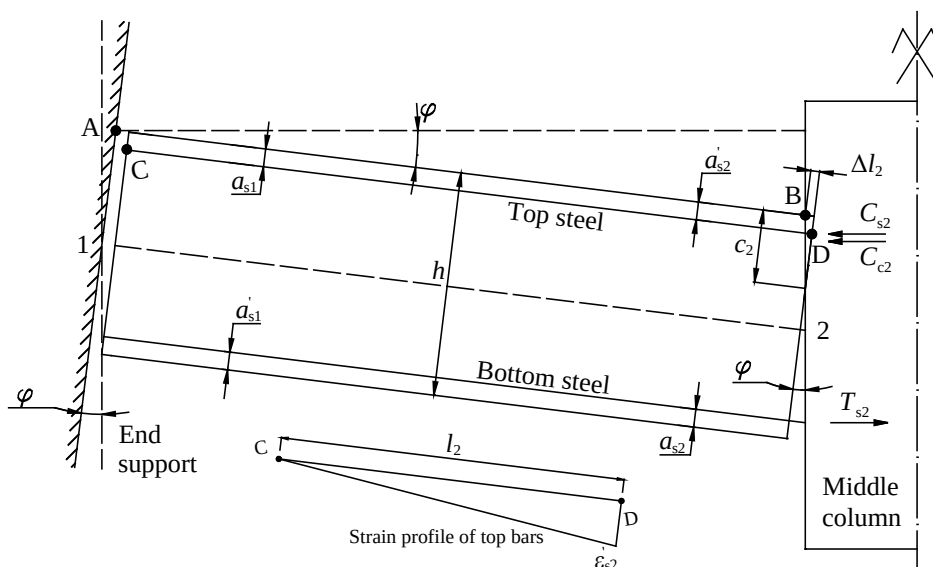


Fig. 7.8: Configuration of beam at small deformation stage

As assumed in previous analytical studies (Park and Gamble 2000; Welch et al. 1999), all the flexural rotations are concentrated at the hinges at the beam ends, but the beam sustains axial deformation induced by the constant compression force along the beam length. Compatibility condition is established based on the rigid-plastic assumption. In experimental tests, connection gaps between end column stubs and horizontal restraints significantly reduce the CAA capacities of sub-assemblages (Yu and Tan 2013a). When the middle joint displacement is relatively small, the beam is only subjected to flexural action and no axial compression force develops due to the connection gap between the beam and the horizontal support. The connection gap also allows free rotation of the end support. Therefore, the connection gap and free rotation of the end support have to be considered in the analytical model.

Fig. 7.8 shows the deformed configuration of the beam at the free rotation stage of the end support. The beam only sustains sagging moment in the middle joint and axial compression force has not been mobilised in the beam. Thus, the resultant force at section 2 is zero. The initial connection gap is reduced by the vertical deflection of the beam, as calculated from Eq. (7-14). However, the compatibility condition of the beam cannot be invoked until the connection gap is closed up.

$$t_1 = l(1 - \cos \varphi) + t_0 - (0.5h - c_2) \tan \varphi \approx t_0 - (0.5h - c_2) \tan \varphi \quad (7-14)$$

where t_0 is the connection gap prior to loading, measured in the experimental test from the measured horizontal force versus horizontal displacement plot; φ is the rotation angle of the beam.

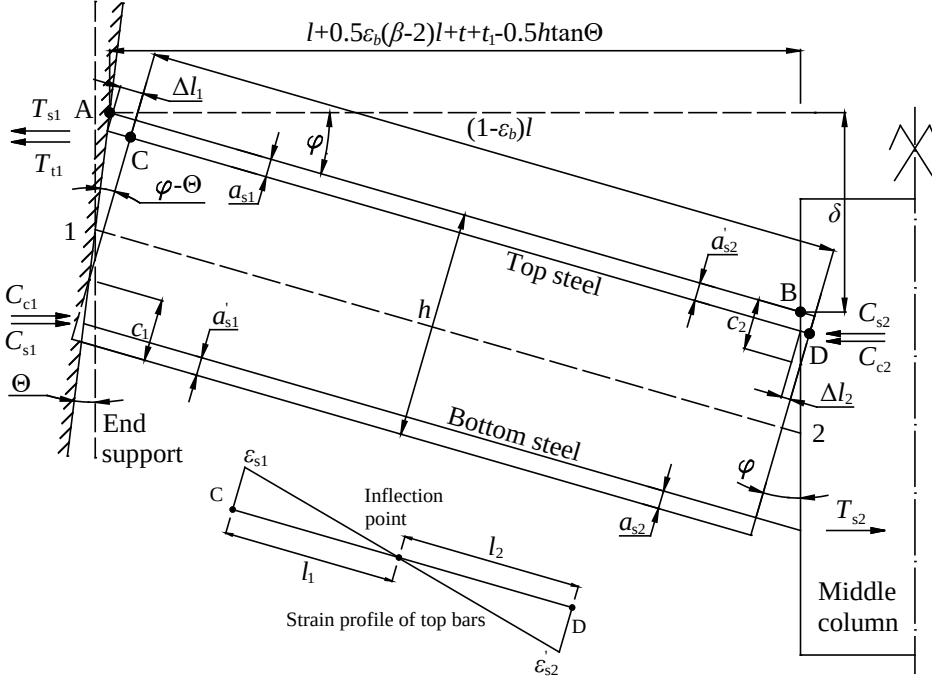


Fig. 7.9: Compatibility condition of beam at large deformation stage

When free rotation angle Θ_0 of the end support is attained, axial compression force starts developing in the beam. Compatibility condition of the beam can be applied. **Fig. 7.9** shows the geometric condition of the beam. Due to inadequate stiffness of the rotational restraint, the support of the single-span beam experiences a rotation angle of Θ when the beam rotates at an angle of φ , and it compresses the top layer of the beam by $0.5h \tan \Theta$. The axial strain of the beam and middle joint is assumed at a constant value of ε_b along the beam segment in **Fig. 7.9**. Thus, the middle joint is shortened by $0.5\varepsilon_b(\beta-2)l$. The beam compressive force imposes a leftward movement of t to the end support (Park and Gamble 2000). In addition, connection gap t_0 between the beam end and the support is taken into consideration as well (Yu and Tan 2013a). The horizontal distance l_{AB} between points A at the end support and B at the middle joint can be calculated as:

$$(l_{AB})_1 = l + 0.5\varepsilon_b(\beta - 2)l + t + t_0 - 0.5h \tan \Theta \quad (7-15)$$

where t and Θ are the horizontal translation and the rotation of the end support, respectively.

Support movement t and rotation Θ can be determined by Eqs. (7-16) and (7-17), respectively. In calculating K_a and K_r from experimental results, the approach recommended by Yu and Tan (2013a) is employed.

$$t = N/K_a \quad (7-16)$$

$$\Theta = \Theta_0 + M_1/K_r \quad (7-17)$$

where K_a and K_r are the respective stiffnesses of horizontal and rotational restraints, and Θ_0 is the free rotation angle of the end support induced by the connection gap.

From the geometric compatibility condition of the beam (Yu and Tan 2013a), the spacing l_{AB} between points A and B can also be expressed in Eq. (7-18). Accordingly, vertical displacement of the middle joint is calculated from Eq. (7-19).

$$(l_{AB})_2 = (l(1 - \varepsilon_b) + (h - c_1) \tan(\varphi - \Theta) - c_2 \tan \varphi) \cos \varphi \quad (7-18)$$

$$\delta = (l + 0.5\varepsilon_b(1 - 2\beta)l + t + t_0 - 0.5h \tan \Theta) \tan \varphi \quad (7-19)$$

In order to determine the strain profiles at sections 1 and 2, it is assumed that the strain of beam top longitudinal reinforcement is zero at the inflection point of the beam and varies linearly between the inflection point and beam end sections, as shown in **Fig. 7.9**. Accordingly, at the centroid of reinforcement, total elongation of the top reinforcement between the inflection point and section 1 is equal to the distance between the left end of the beam and support, and the shortening of the bar between section 2 and inflection point is equal to the distance between the right end of the beam and middle joint interface, as expressed in Eqs. (7-20) and (7-21).

$$\Delta l_1 = (h - c_1 - a_{s1}) \tan(\varphi - \Theta) = 0.5\varepsilon_{s1}l_1 \quad (7-20)$$

$$\Delta l_2 = (c_2 - a'_{s2}) \tan \varphi = 0.5\varepsilon'_{s2}l_2 \quad (7-21)$$

where l_1 and l_2 are the respective distances from the inflection point of the beam to its left and right ends; ε_{s1} is the tensile strain of top reinforcement at section 1; and ε'_{s2} is the compressive strain of top reinforcement at section 2.

Due to different ratios of top and bottom longitudinal reinforcement in the beam, the inflection point is not located at the mid-span of the beam between the end support and middle joint. It shifts with increasing middle joint displacement at large deformation stage. The position of the inflection point is determined by the bending moments at sections 1 and 2, as expressed in Eqs. (7-22) and (7-23). The presence of axial compression force in the beam will reduce the length of beam segment between the inflection point and end support (section 1). However, in comparison with the maximum strain of tension reinforcement generated by bending moment, the beam compressive strain is much smaller, and can be neglected in the model. Thus, the original beam length is used in place of its deformed length.

$$l_1 = \frac{lM_1}{M_1 + M_2} \quad (7-22)$$

$$l_2 = \frac{lM_2}{M_1 + M_2} \quad (7-23)$$

where M_1 is the hogging moment at section 1; M_2 is the sagging moment at section 2. It is notable that M_1 is zero at free rotation stage of the end support.

After determining the neutral axis depth and steel strain at each section, the strain profile can be determined in accordance with the plane-section assumption, as expressed in Eq. (7-24) to Eq. (7-28).

$$\varepsilon'_{s1} = \frac{c_1 - a'_{s1}}{h - c_1 - a_{s1}} \varepsilon_{s1} \quad (7-24)$$

$$\varepsilon_{s2} = \frac{h - c_2 - a_{s2}}{c_2 - a'_{s2}} \varepsilon'_{s2} \quad (7-25)$$

$$\varepsilon_{c1} = \frac{c_1}{c_1 - a_{s1}} \varepsilon'_{s1} \quad (7-26)$$

$$\varepsilon_{c2} = \frac{c_2}{c_2 - a_{s2}} \varepsilon'_{s2} \quad (7-27)$$

$$\varepsilon_{t1} = \frac{h - c_1}{h - c_1 - a_{s1}} \varepsilon_{s1} \quad (7-28)$$

where ε'_{s1} is the compressive strain of bottom reinforcement at section 1; ε_{s2} is the tensile strain of bottom reinforcement at section 2; ε_{c1} and ε_{c2} are the strains of extreme compression fibres at sections 1 and 2, respectively; and ε_{t1} is the strain of extreme tension fibre.

7.2.4 Solution procedure

Due to the presence of the connection gap between the beam and end support, compatibility condition of the beam is not satisfied at the initial stage. Only force equilibrium is considered when rotation φ of the beam is smaller than free rotation angle Θ_0 of the end support. Based on the deformed geometry of the beam, as shown in **Fig. 7.8**, neutral axis depth c_2 at section 2 is assumed. Due to zero hogging moment at section 1, l_2 is equal to l at this stage, and compressive strain ε'_{s2} of the top longitudinal reinforcement at section 2 is calculated from Eq. (7-21). Accordingly, tensile strain ε_{s2} of the bottom reinforcement and strain ε_{c2} of the extreme compression concrete fibre are determined via Eqs. (7-25) and (7-27). Based on the material models for steel reinforcement and concrete, force equilibrium at section 2 is established through Eq. (7-10). It has to be mentioned that ε_b , t , M_1 and N are zero, and Θ is equal to φ at this stage. Once force equilibrium is satisfied, bending moment M_2 at section 2 is computed from Eq. (7-12). Finally, vertical displacement of the middle joint and applied load are calculated from Eqs. (7-19) and (7-6), respectively.

When rotation φ of the beam surpasses free rotation angle Θ_0 of the end support, compatibility condition in **Fig. 7.9** has to be satisfied in calculating the vertical load and middle joint displacement. A set of solution procedure is proposed for calculating the CAA capacity of beam-column sub-assemblages under middle column removal scenarios, as illustrated in **Fig. 7.10**. In each step, rotation angle $\varphi - \Theta$ and neutral axis depth c_1 are assumed at section 1. Initially hogging moment M_{10} at section 1 has to be assumed so as to determine l_1 and l_2 from Eqs. (7-22) and (7-23). M_2

calculated in the previous step is used to locate the inflection point of the beam. Tensile strain ε_{s1} of the top reinforcement at section 1 is obtained through Eq. (7-20), and compressive strain ε'_{s1} of the bottom reinforcement and concrete strain ε_{c1} are determined based on the plane-section assumption, as expressed in Eqs. (7-24) and (7-26). Correspondingly, axial force N_1 and bending moment M_1 are calculated through Eqs. (7-10) and (7-11). The analytical procedure continues if M_1 is approximately equal to the initially assumed value within a limited tolerance ΔM_{tol} . At section 2, neutral axis depth c_2 is assumed, and a similar procedure as abovementioned is repeated to determine axial force N_2 and bending moment M_2 . When N_1 and N_2 are close to each other within a small tolerance ΔN_{tol} , geometric compatibility condition of the beam is examined via Eqs. (7-15) and (7-18). Once the compatibility condition is satisfied, the vertical displacement and applied load can be calculated from Eqs. (7-19) and (7-6).

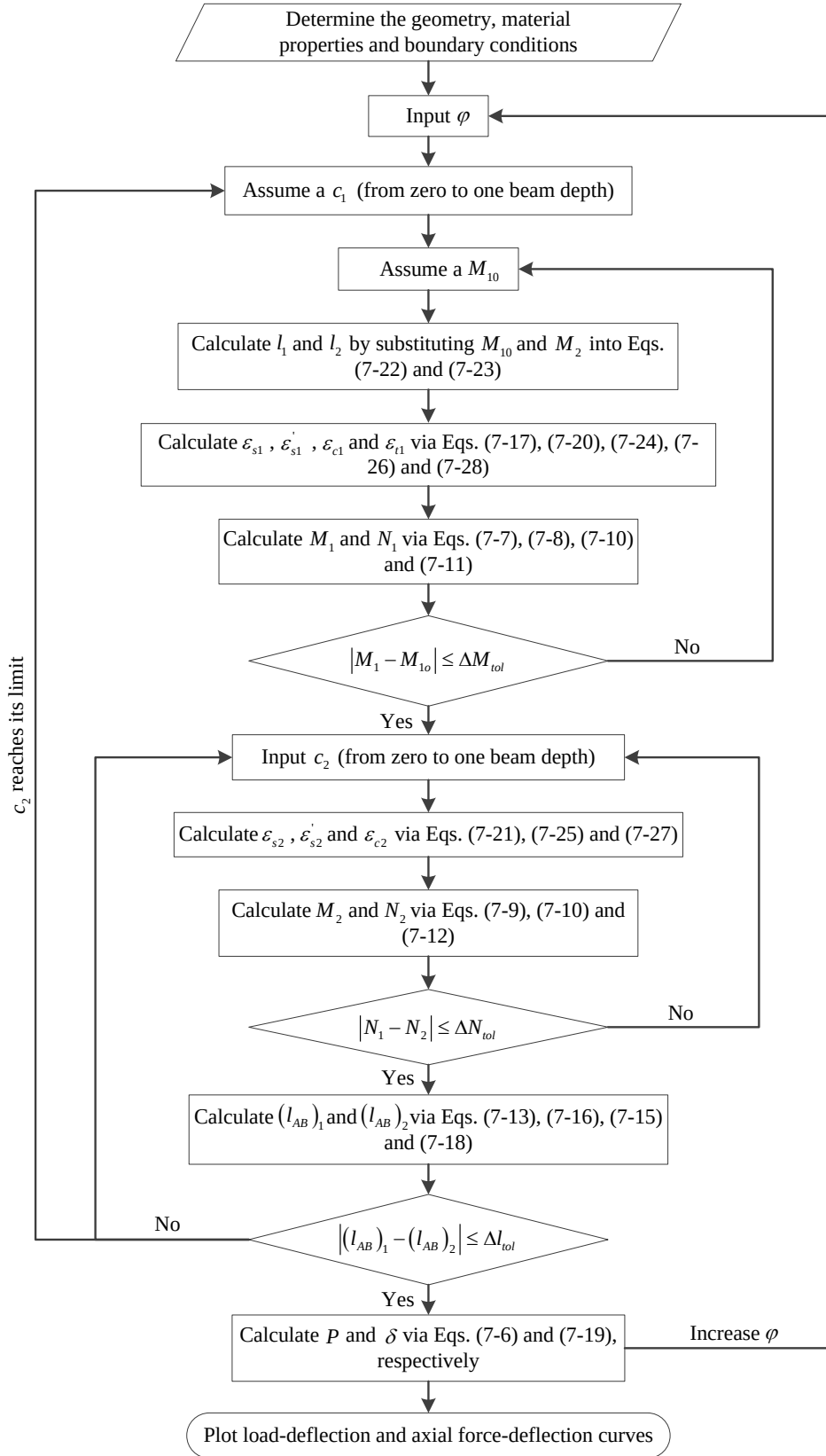


Fig. 7.10: Solution procedure for the analytical model

7.3 Validation of the Analytical Model

The proposed analytical model is validated by experimental results of reinforced concrete sub-assemblages under column removal scenarios (FarhangVesali et al. 2013; Yu and Tan 2013a). FarhangVesali et al. (2013) did not measure the stiffness of horizontal restraints in the experimental tests. Thus, referring to Yu and Tan's analytical study (Yu and Tan 2013a), horizontal stiffness of each sub-assemblage is determined to be 1.0×10^6 kN/m. Free rotation of the end column stub is not considered for reinforced concrete sub-assemblages tested by Yu (2012). Besides, precast concrete sub-assemblages with conventional concrete and ECC are also simulated in the model. It is noteworthy that pull-out failure of longitudinal reinforcement embedded in the joint is not considered in the model, and thus only conventional concrete sub-assemblage MJ-B-0.52/0.35S is analysed through the model. **Table 7.1** lists the boundary conditions of the specimens used in the verification studies. In the following sections, comparisons are made between the experimental and analytical results. Moreover, variations of strains of steel reinforcement and concrete fibres are also shown to gain a deeper insight into the behaviour of sub-assemblages subjected to CAA.

Table 7.1: Boundary conditions of beam-column sub-assemblages

Specimen		Cross section (mm)	Clear span (mm)	Boundary conditions			
				Axial stiffness (kN/m)	Connection gap (mm)	Rotational stiffness (kN.m/rad)	Free rotation angle (radian)
Farhang Vesali et al. (2013)	F1	180x180	2200	1.0x10 ⁶	0	1.45x10 ⁴	0
	F2					1.35x10 ⁴	
	F3					1.85x10 ⁴	
	F4					1.80x10 ⁴	
	F5					1.60x10 ⁴	
	F6					1.45x10 ⁴	
Yu and Tan (2013a)	S1	150x250	2750	1.06x10 ⁵	0.5	1.0x10 ⁴	0
	S2				1.2		
	S3			4.29x10 ⁵	1.2	3.0x10 ⁴	
	S4				1.0		
	S5				0.8		
	S6				0.8		

	S7		2150		1.2		
	S8		1550		0.8		
MJ-B-0.52/0.35S		150x300	2750	1.51×10^5	1.1	2.14×10^4	0.005
CMJ-B-1.19/0.59				2.05×10^5	0.9	1.99×10^4	0.01
EMJ-B-1.19/0.59*				1.83×10^5	0.7	1.49×10^4	0.006
EMJ-B-0.88/0.59*				1.49×10^5	0.8	2.30×10^4	0.008
EMJ-L-1.19/0.59*				1.77×10^5	0.2	1.72×10^4	0.01
EMJ-L-0.88/0.59*				1.59×10^5	0.4	2.44×10^4	0.008

*: “EMJ” represents beam-column sub-assemblages with ECC in the structural topping and the beam-column joint.

7.3.1 Prediction of CAA capacity and horizontal reaction force

Table 7.2 shows the predicted CAA capacities and horizontal compression forces. The mean ratio of the analytical and experimental CAA capacities is 0.99, with a coefficient of variation of 0.05. It indicates that the analytical model gives very good predictions of the CAA capacity of beam-column sub-assemblages. The horizontal compression force in the beams is slightly overestimated, with an average ratio of 1.04 and a coefficient of variation of 0.09, as it is more sensitive to the boundary conditions of sub-assemblages than the CAA capacity. It is noteworthy that due to constraints of test set-up, horizontal compression force could not be quantified in reinforced concrete beam-column sub-assemblages tested by FarhangVesali et al. (2013).

Table 7.2: Comparisons of experimental and analytical results

Author and Specimen		Capacity of CAA			Maximum axial compression		
		Experimental results P_c (kN)	Analytical results P_a (kN)	$\frac{P_a}{P_c}$	Experimental results N_c (kN)	Analytical results N_a (kN)	$\frac{N_a}{N_c}$
Farhang Vesali et al. (2013)	F1	40.5	37.5	0.93	--	211.1	--
	F2	35.7	35.7	1.00	--	193.0	--
	F3	41.4	37.5	0.91	--	210.6	--
	F4	40.1	39.3	0.98	--	177.9	--
	F5	41.6	39.3	0.94	--	189.3	--
	F6	39.4	39.5	1.00	--	191.7	--
Yu and Tan (2013a)	S1	41.6	41.9	1.01	177.9	186.7	1.05
	S2	38.4	36.8	0.96	155.9	169.2	1.09
	S3	54.5	51.8	0.95	221.0	235.3	1.06
	S4	63.2	59.1	0.94	212.7	251.8	1.18

	S5	70.3	68.4	0.97	238.4	267.5	1.12
	S6	70.3	71.0	1.01	218.1	221.7	1.02
	S7	82.8	82.9	1.00	233.1	272.0	1.17
	S8	121.3	129.5	1.07	272.5	293.8	1.08
MJ-B-0.52/0.35S		50.5	48.5	0.96	231.3	239.0	1.03
CMJ-B-1.19/0.59		90.4	94.5	1.05	281.1	288.2	1.03
EMJ-B-1.19/0.59		91.1	99.0	1.09	274.7	271.6	0.99
EMJ-B-0.88/0.59		83.7	86.4	1.03	262.8	276.6	1.05
EMJ-L-1.19/0.59		91.1	93.1	1.02	305.8	271.1	0.89
EMJ-L-0.88/0.59		82.5	82.1	1.00	317.7	273.9	0.86
Mean value				0.99			1.04
Coefficient of variation				0.05			0.09

7.3.2 Prediction of load-displacement curve

Fig. 7.11 and **Fig. 7.12** show the comparisons between the analytical and experimental vertical load-middle joint displacement curves. It indicates that the analytical model is capable of predicting the vertical load-middle joint displacement curve with reasonable accuracy. However, rupture of beam bottom reinforcement at the middle joint interface cannot be predicted, as it involves localised strain concentration of reinforcement at the face of middle joint. Thus, beyond the CAA capacity, the calculated vertical load decreases slowly with increasing middle joint displacement.

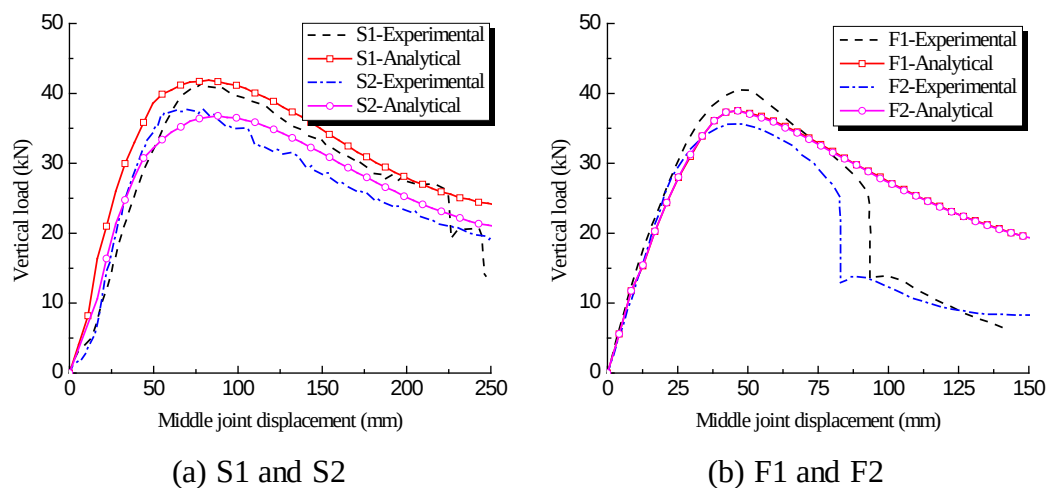


Fig. 7.11: Comparisons of analytical and experimental vertical load-middle joint displacement curves of reinforced concrete sub-assemblages

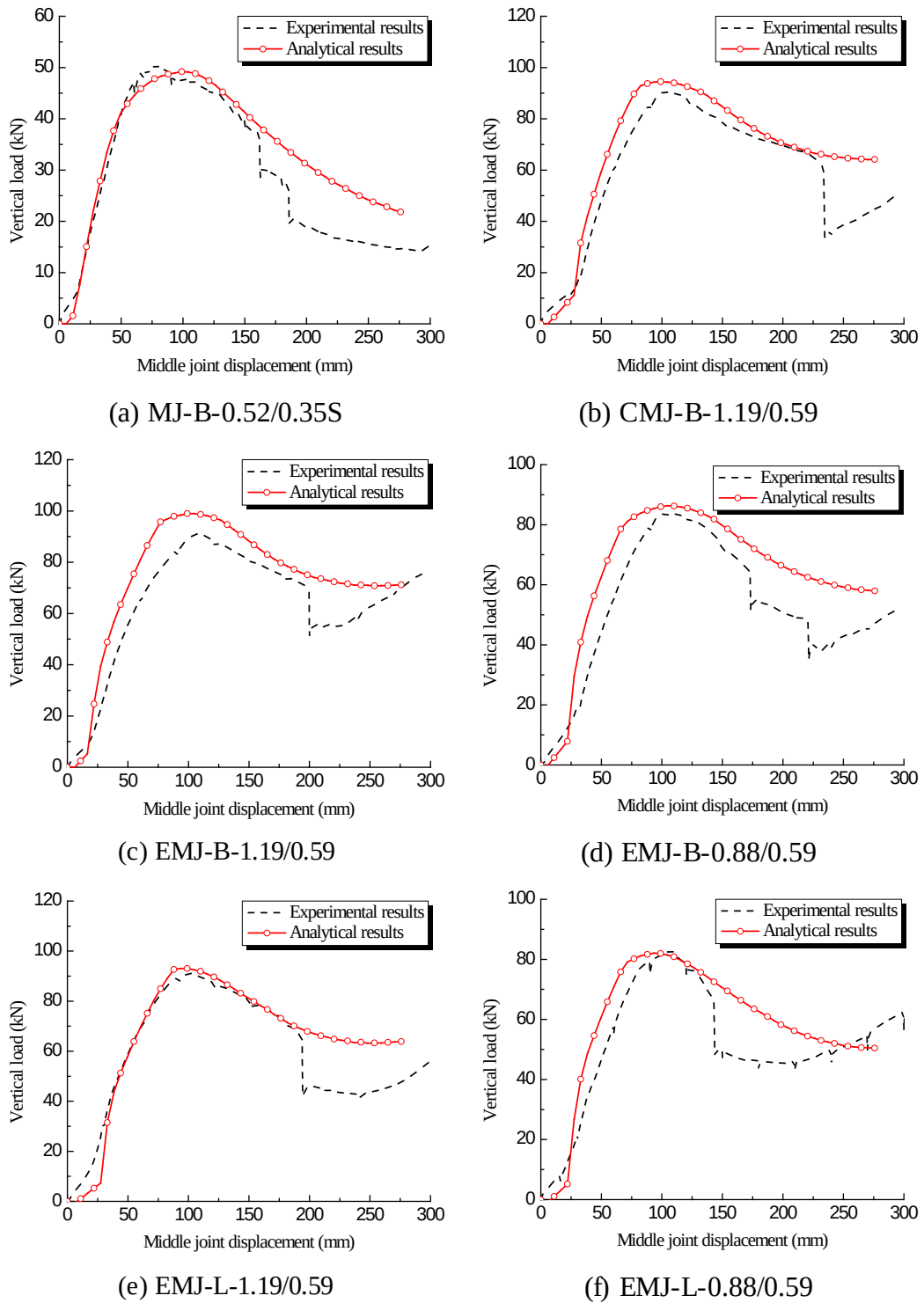


Fig. 7.12: Comparisons of analytical and experimental vertical load-middle joint displacement curves of precast beam-column sub-assemblages

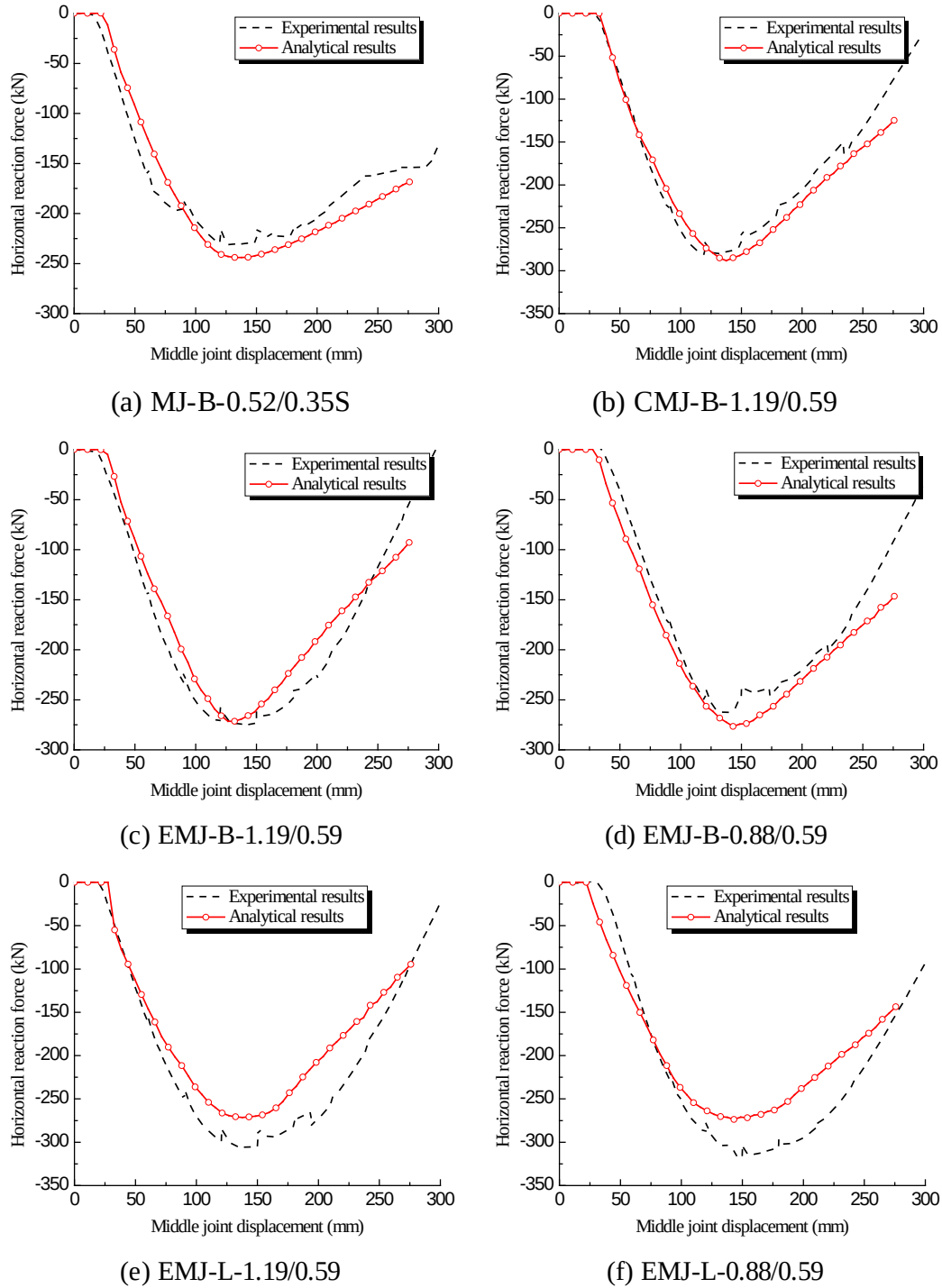


Fig. 7.13: Comparisons of analytical and experimental horizontal reaction force-middle joint displacement curves of precast beam-column sub-assemblages

With respect to the horizontal reaction force-middle joint displacement curve, good agreement is reached between the analytical and experimental results, as shown in **Fig. 7.13**. At the initial stage, horizontal force is zero due to free rotation of the end

column stub. Once the connection gap is closed up, horizontal compression force in the bridging beam starts increasing until it attains the maximum value. With increasing middle joint displacement, the horizontal compression force decreases due to crushing of concrete in the compression zones of the beam. The analytical model cannot predict well this part of the curve, as spalling of concrete is not considered in Mander's concrete model.

7.3.3 Variation of bending moments

Sagging moment at the middle joint and hogging moment at the end support can also be obtained using the model. **Fig. 7.14** shows the variations of bending moments with middle joint displacement of ECC sub-assembly EMJ-B-1.19/0.59. The predicted sagging moment at the middle joint agrees well with the experimental results. In the descending branch of sagging moment, rupture of bottom reinforcement at the middle joint interface led to a drop of sagging moment, as shown in **Fig. 7.14(a)**. However, this phenomenon cannot be captured by the model. At the end support, the maximum hogging moment is predicted earlier than the experimental curve (see **Fig. 7.14(b)**).

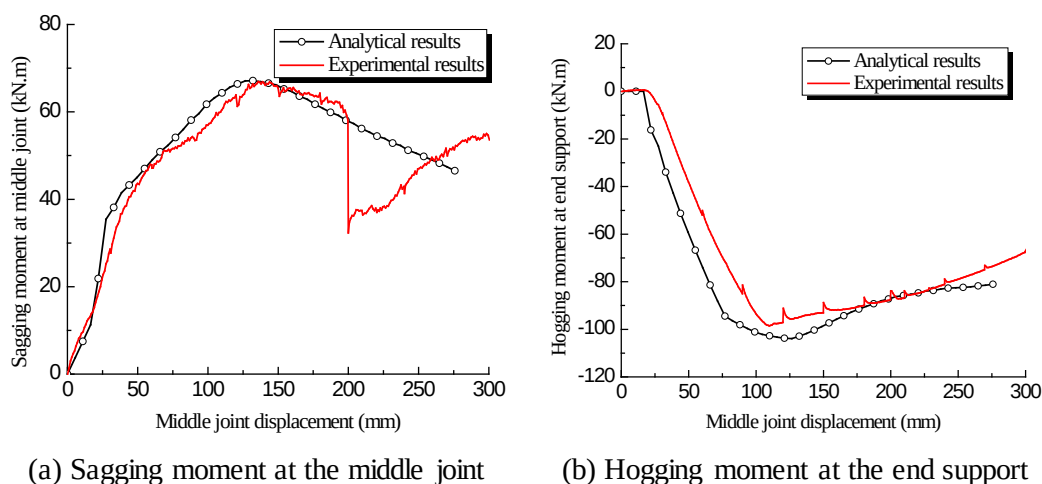


Fig. 7.14: Variations of bending moments at the middle joint and end support

7.3.4 Estimate of neutral axis depth and reinforcement strain

To investigate the behaviour of beam-column sub-assemblages at the fibre level, **Fig. 7.15** shows the strains of reinforcing bars and concrete fibre at each end of the beam in sub-assembly EMJ-B-1.19/0.59. Even though the absolute value of the steel

strain is substantially underestimated in the model, the overall trend agrees well with the experimental results. Strains of tensile reinforcement keep increasing at the middle joint and end support, as shown in **Fig. 7.15(a)**. Tensile strain of the bottom reinforcement at the middle joint is several times larger than that of the top reinforcement at the end support. Thus, rupture of bottom reinforcement at the middle joint interface occurs earlier than top reinforcement at the end support. A similar variation of extreme compression concrete fibres is obtained, as shown in **Fig. 7.15(b)**. However, strains of compressive reinforcement increase at CAA stage and then decrease with increasing middle joint displacement (see **Fig. 7.15(c)**), in particular, at the middle joint. Moreover, compressive strain of the top reinforcement at the middle joint is considerably smaller than that of the bottom reinforcement at the end support, due to a greater compressive reinforcement ratio and a smaller neutral axis depth at the middle joint.

Neutral axis depth at the beam end sections can also be obtained in the analytical model. **Fig. 7.16** shows the variations of neutral axis depths with middle joint displacement. Compared to the end support, compressive reinforcement ratio is greater but tensile reinforcement ratio is smaller at the middle joint. Correspondingly, the neutral axis depth at the middle joint is significantly smaller than that at the end support. It continues to decrease with increasing middle joint displacement at CAA stage. However, different variation of the neutral axis depth is obtained at the end support. At the initial stage, neutral axis depth at the end support is zero due to free rotation of the end column stub, and it attains a peak value when the beam top reinforcement yields. Following the yielding of top tensile reinforcement at the end section, the neutral axis depth decreases slightly with increasing middle joint displacement, and then crushing of compressive concrete increases the neutral axis depth at the end support. A further increase in the middle joint displacement decreases the neutral axis depth at the end support.

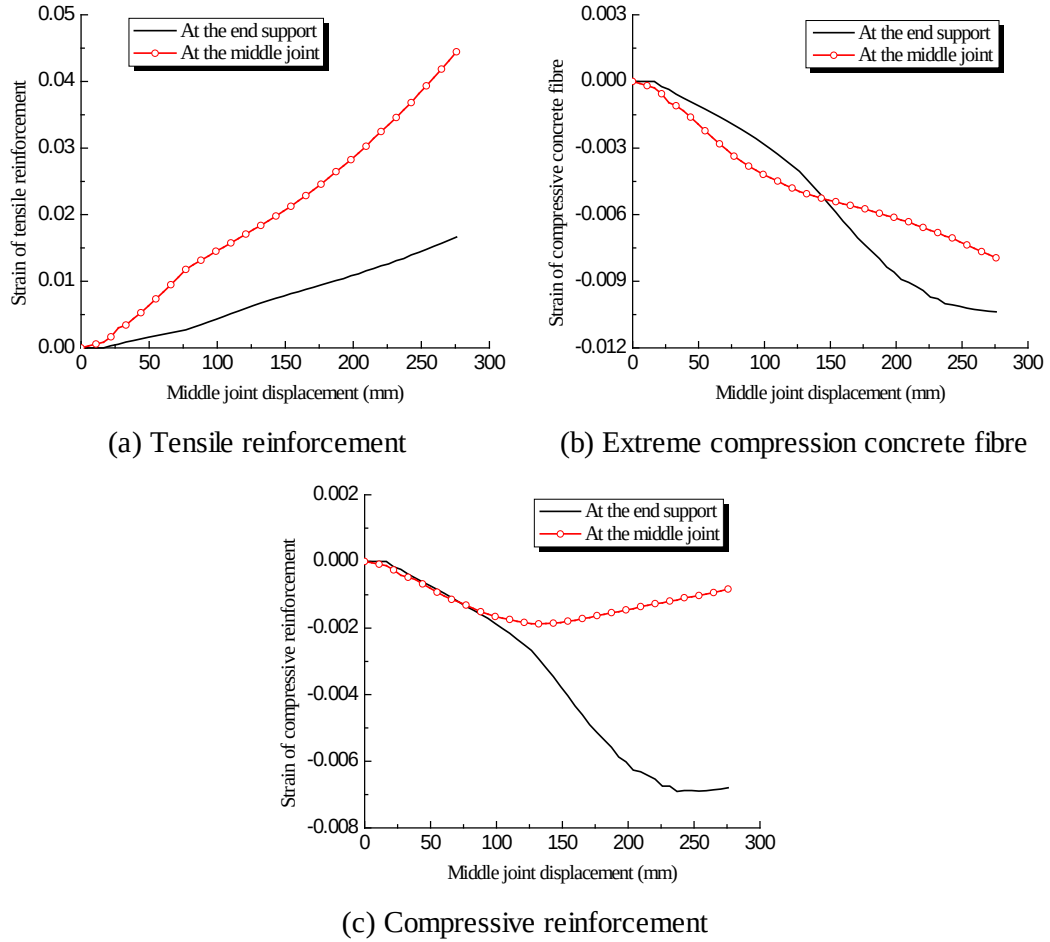


Fig. 7.15: Variations of numerical strains of steel reinforcement and concrete with middle joint displacement

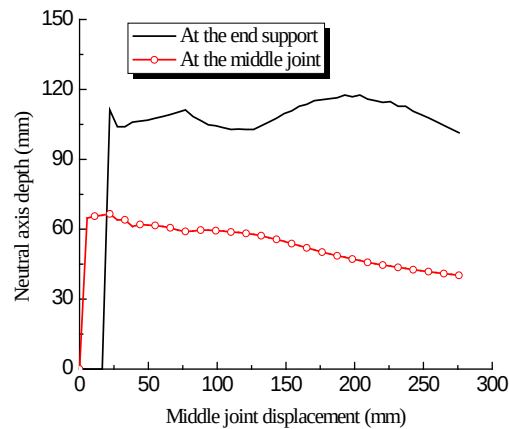


Fig. 7.16: Variations of neutral axis depths with middle joint displacement

7.4 Limitations of the Analytical Model

Several limitations are identified in the analytical model as follows:

(1) The analytical model is not applicable if horizontal shear cracking occurs between the precast beam units and cast-in-situ concrete topping, as full composite action between the precast beam unit and concrete topping is assumed in the model.

(2) The strain profile of top longitudinal reinforcement is assumed to be linear between the inflection point and beam end sections and flexural deformations of the beam are totally concentrated at the beam ends. However, after yielding of top reinforcement near the end support, tensile strain of longitudinal reinforcement in the plastic hinge region is substantially greater than that in the middle portion of the beam, as reported by Yu and Tan (2013c). A similar variation of compressive strain is also observed at the face of the middle joint. Therefore, the assumption significantly underestimates the maximum strains of steel reinforcement at the middle joint and end column stub. Accordingly, rupture of longitudinal reinforcement cannot be predicted in the analytical model.

(3) Due to underestimation of strains at the end support, the contribution of ECC to the resistance of beam-column sub-assemblages is exaggerated at large deformation stage, as its tensile strain capacity is attained later in the analytical model than in the experimental test.

(4) Spalling of concrete in the compression zone is not considered, which results in increasing discrepancy between the experimental and analytical results in the descending phase of the vertical load and horizontal reaction force.

7.5 Parametric Studies

Through the proposed analytical model, a series of parametric studies is conducted to investigate the effects of different concrete models, tensile strength and strain capacity of ECC, stiffness of horizontal restraint, and longitudinal reinforcement ratios on the CAA of beam-column sub-assemblages. Beam-column sub-assemblage EMJ-B-1.19/0.59 is selected as a benchmark in the parametric studies.

7.5.1 Effect of concrete models

In the flexural design of reinforced concrete members, equivalent rectangular concrete stress block is generally used to calculate the compression force sustained

by concrete. However, at large deformation stage, the rectangular stress block is no longer appropriate due to crushing and spalling of concrete in the compression zone. Stress-strain models for concrete have to be employed in place of the equivalent rectangular concrete stress block. In the analytical model, the concrete model proposed by Mander et al. (1988) is used to predict the CAA capacity of beam-column sub-assemblages and reasonably accurate results are obtained. Furthermore, the behaviour of beam-column sub-assemblages may vary greatly through different stress-strain models for concrete, in particular, the descending branch due to the assumption of linear strain variation along the beam length. Therefore, it is necessary to evaluate the effect of concrete models on the behaviour of beam-column sub-assemblages at CAA stage.

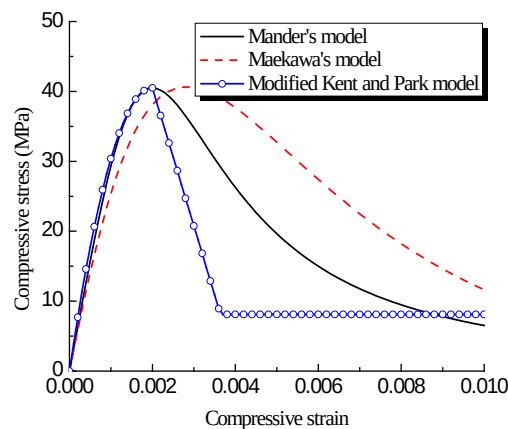
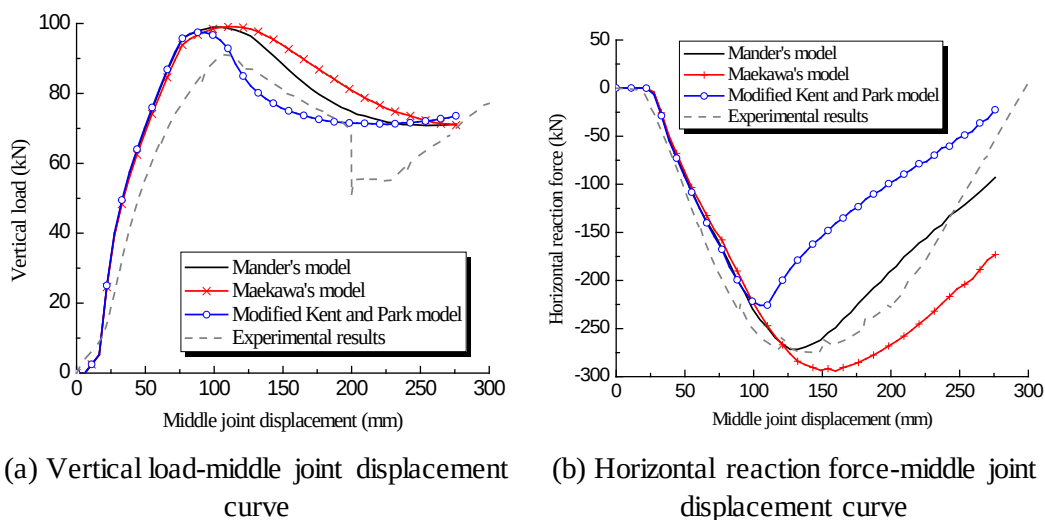


Fig. 7.17: Comparison of stress-strain models for concrete



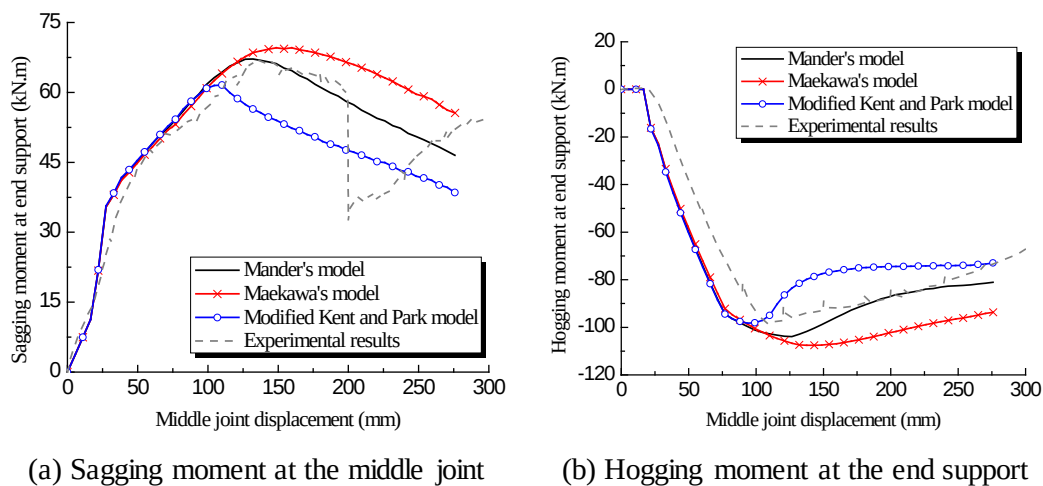
(a) Vertical load-middle joint displacement curve

(b) Horizontal reaction force-middle joint displacement curve

Fig. 7.18: Comparisons of load-displacement curves with different concrete models

Besides Mander's model, stress-strain model proposed by Maekawa et al. (2003) and modified Kent and Park model (Scott et al. 1982) are also incorporated in the analytical model. **Fig. 7.17** shows the comparisons among the three concrete models. Limited differences exist in the ascending branch of the stress-strain curves. However, the concrete models differ greatly from one another in the descending branch, which may affect the behaviour of beam-column sub-assemblages at large deformation stage.

Fig. 7.18 shows the predicted vertical loads and horizontal compression forces with different concrete models. Comparisons among the analytical results indicate that almost the same CAA capacities of the sub-assemblage are estimated with different concrete models, as shown in **Fig. 7.18(a)**. However, significant differences exist in the descending phase of the vertical load. Maekawa's model provides the least descending vertical load beyond the CAA capacity, whereas modified Kent and Park model predicts the most significant descending phase. The predicted horizontal compression forces also vary from each other, as shown in **Fig. 7.18(b)**. Maekawa's model overestimates the maximum compression force due to its more ductile stress-strain relationship, but modified Kent and Park model substantially underestimates the peak compression force due to its brittle stress-strain model. Therefore, the concrete model plays a significant role in the development of compression force in the beam.



(a) Sagging moment at the middle joint

(b) Hogging moment at the end support

Fig. 7.19: Comparisons of bending moments at the middle joint and end support

Fig. 7.19 shows the comparisons of sagging moment at the middle joint and hogging moment at the end support. It is observed that the three concrete models predict almost the same ascending branch of bending moments at the middle joint and end support. With regard to the maximum sagging and hogging moments, Maekawa's model provides the greatest value and modified Kent and Park model gives the smallest value. Similar results are obtained in the descending branch of bending moments, as shown in **Figs. 7.19(a and b)**.

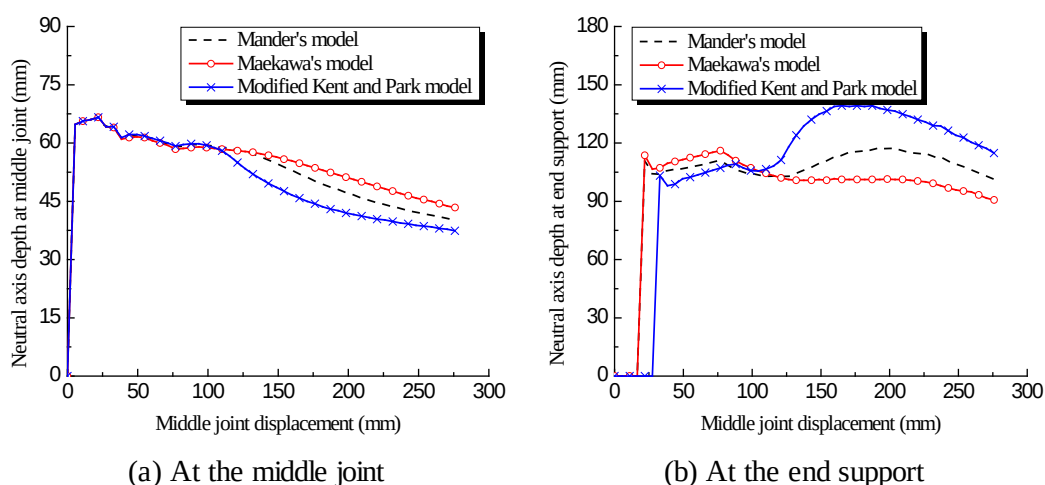


Fig. 7.20: Comparisons of neutral axis depths with different concrete models

Variations of neutral axis depths at the middle joint and end support are also estimated using the analytical model. **Fig. 7.20** shows the comparisons of the neutral axis depths at the end support and middle joint. When the middle joint displacement is smaller than 100 mm, similar neutral axis depths are obtained at the end support and middle joint through different concrete models, as shown in **Figs. 7.20(a and b)**. With middle joint displacement greater than 100 mm, the neutral axis depths start deviating from one another, in particular, at the end support (see **Fig. 7.20(b)**). It is due to different descending branches of the stress-strain models for concrete. At the middle joint, there is less deviation of neutral axis depth as a result of a greater compressive reinforcement ratio and relatively less contribution of concrete to total force in the compression zone.

7.5.2 Effect of tensile strength of ECC

The CAA capacity of beam-column sub-assemblages is sensitive to boundary conditions and connection gaps (Yu and Tan 2013a). In the experimental tests, different stiffness values of horizontal restraints and connection gaps were obtained, as summarised in **Table 7.1**. Thus, a direct comparison cannot be made between conventional concrete and ECC specimens. Instead, parametric studies are conducted to investigate the effect of tensile strength of ECC on the ultimate resistance of sub-assemblages.

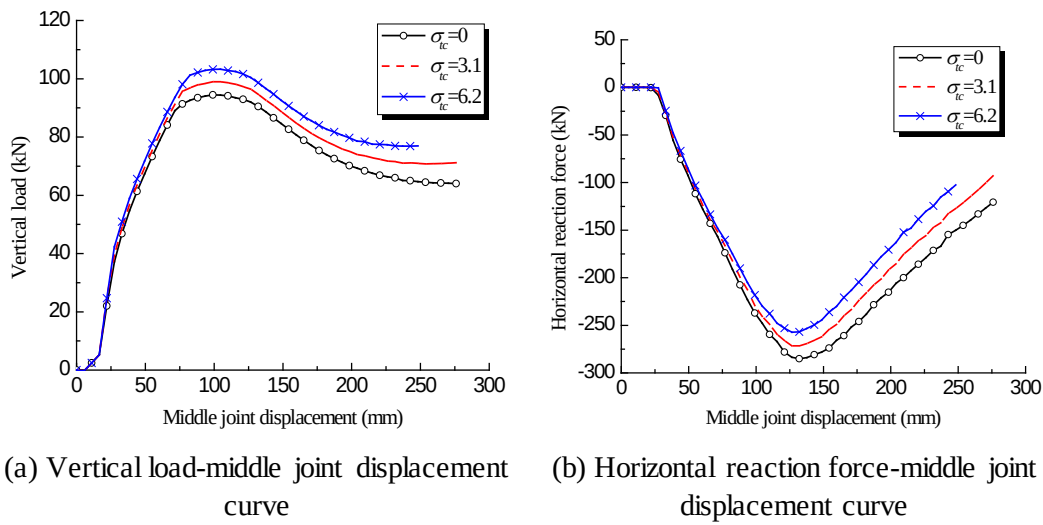


Fig. 7.21: Comparisons of load-displacement curves with different tensile strengths of ECC

In sub-assemblage EMJ-B-1.19/0.59, the effective tensile strength of ECC is calculated as 3.1 MPa. To study the enhancement of ECC to CAA capacity, the tensile strength is reduced to zero and increased to 6.2 MPa, respectively. **Fig. 7.21** shows the variations of vertical loads and horizontal reaction forces with middle joint displacement. In comparison with conventional concrete sub-assemblage in which the tensile strength of concrete is not considered (i.e. $\sigma_{tc} = 0$), ECC topping with a tensile strength of 3.1 MPa increases the CAA capacity of the sub-assemblage by 4.5 kN, around 4.8% of the CAA capacity. Increasing the tensile strength of ECC from zero to 6.2 MPa also increases the CAA capacity by 9%, as shown in **Fig. 7.21(a)**. However, the maximum compression force in the beam decreases with increasing tensile strength of ECC, as shown in **Fig. 7.21(b)**. When σ_{tc} is 3.1 MPa, the

maximum compression force in the beam is reduced by 4.8% compared to that when the tensile strength is zero.

Fig. 7.22 shows the comparisons of neutral axis depths with different tensile strengths of ECC. Similar variations of the neutral axis depths are obtained with increasing middle joint displacement. The tensile strength of ECC has opposite effects on the neutral axis depths at the middle joint and end support. A higher tensile strength of ECC reduces the neutral axis depth at the middle joint (see **Fig. 7.22(a)**), as the net compression force in the beam is slightly reduced but the force in the tension zone remains almost the same. However, the neutral axis depth at the end support is increased (see **Fig. 7.22(b)**), since total force in the tension zone of the beam is increased at the end support while axial compression force in the beam is slightly reduced. The differences among the neutral axis depths are limited for the range of tensile strengths investigated.

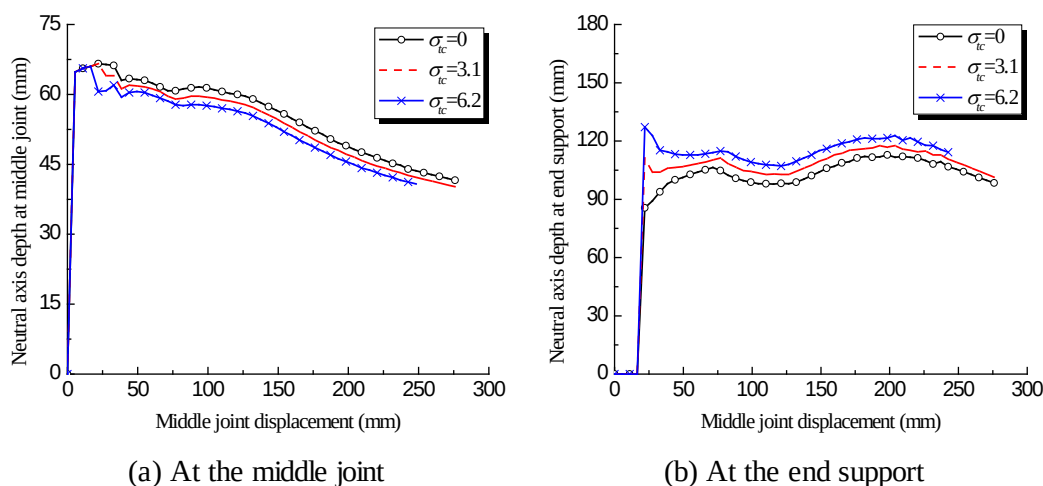


Fig. 7.22: Comparisons of neutral axis depths with different tensile strengths of ECC

7.5.3 Effect of tensile strain capacity of ECC

The effect of tensile strain capacity on the resistance of the sub-assembly is also investigated in the analytical model. In sub-assembly EMJ-B-1.19/0.59, tensile strain capacity of ECC in the structural topping is around 2.6%. The strain capacity is reduced to zero and doubled to 5%, respectively, to study the effect of tensile strain capacity on CAA behaviour.

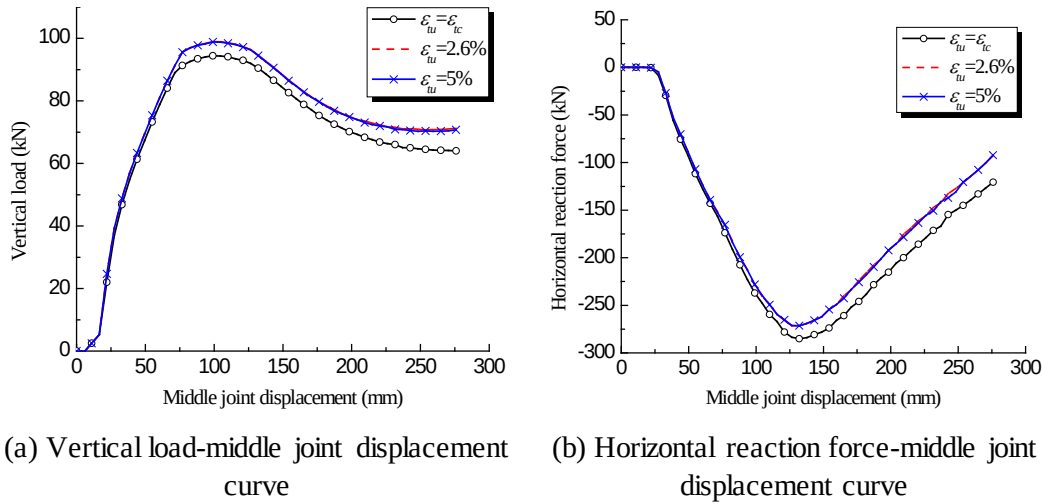


Fig. 7.23: Comparisons of load-displacement curves with different tensile strain capacities of ECC

Fig. 7.23 shows the calculated vertical loads and horizontal reaction forces. It is found that if the tensile strain capacity of ECC is increased from zero to 2.6%, the CAA capacity of the sub-assembly is enhanced by 4.7%, but the maximum compression force in the beam is reduced by 5%. A further increase in the strain capacity of ECC from 2.6% to 5% does not significantly change the CAA capacity and horizontal compression force of the sub-assembly. Analytical results indicate that an upper bound value of the strain capacity exists, beyond which the CAA capacity cannot be increased by a greater tensile strain capacity. As the CAA capacity of sub-assembly EMJ-B-1.19/0.59 is attained at around 100 mm middle joint displacement, the upper bound value of the strain capacity is associated with the strain of extreme ECC fibre at this displacement. In the analytical model, the value is estimated as 0.6% for sub-assembly EMJ-B-1.19/0.59. If ECC with a greater tensile strain capacity is used, CAA capacity is not increased and only the resistance in the descending branch of vertical load is enhanced.

7.5.4 Effect of stiffness of horizontal restraint

To investigate the effect of horizontal restraints on the behaviour of beam-column sub-assembly at CAA stage, restraint stiffness is normalised by the measured value of sub-assembly EMJ-B-1.19/0.59 (see **Table 7.1**). γ_a represents the ratio of the assumed and measured stiffness. γ_a is equal to 1 for EMJ-B-1.19/0.59. γ_a is

increased to 10 and reduced to 0.1 for comparison purpose. **Fig. 7.24** shows the vertical loads and horizontal reaction forces versus the middle joint displacement. By increasing γ_a from 1 to 10, the CAA capacity of the sub-assembly is increased by 7%. However, the middle joint displacement corresponding to the CAA capacity is reduced, as shown in **Fig. 7.24(a)**. When γ_a is reduced from 1 to 0.1, the CAA capacity is reduced by 12.7%, but the associated vertical displacement is substantially increased. A more significant effect of horizontal restraint is observed on horizontal reaction forces, as shown in **Fig. 7.24(b)**. By reducing γ_a from 1 to 0.1, the maximum compression force in the beam is reduced from 271.6 kN to 88.2 kN. Therefore, a stiffer horizontal restraint is necessary to mobilise CAA in the beam-column sub-assembly.

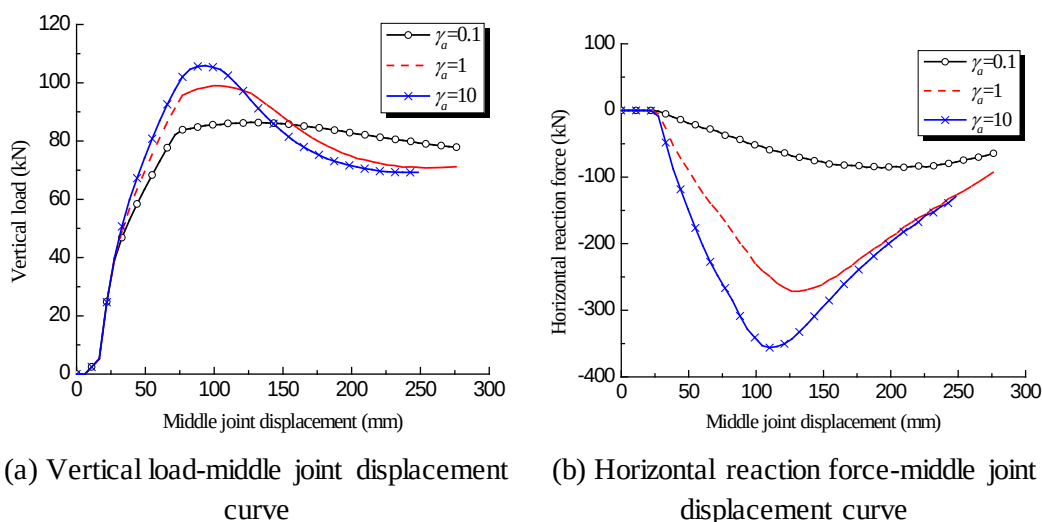


Fig. 7.24: Comparisons of load-displacement curves with different horizontal restraints

In addition to quasi-static resistance of the sub-assembly subject to column removal, pseudo-static resistance is also calculated in accordance with the energy balance method proposed by Izzudin et al. (2008), as discussed in Section 5.4.4. **Fig. 7.25** shows the pseudo-static resistance of the sub-assembly. When γ_a is increased from 1 to 10, the pseudo-static resistance of the sub-assembly is only increased by 0.9 kN. Similarly, the pseudo-static resistance is reduced by 1.6 kN if γ_a is reduced from 1 to 0.1. Thus, the stiffness of horizontal restraints does not have a significant influence on pseudo-static resistance of the sub-assembly. However, by increasing

the stiffness of horizontal restraint, the vertical displacement corresponding to the pseudo-static resistance is substantially reduced, as shown in **Fig. 7.25**.

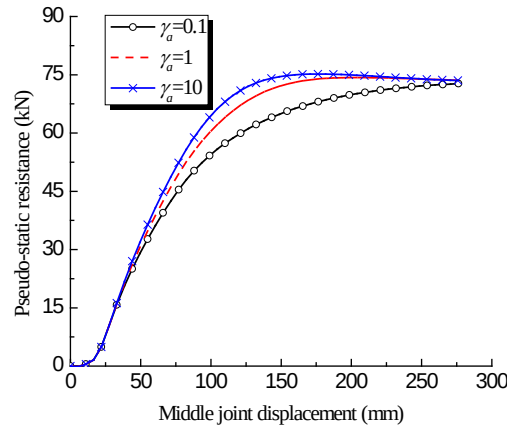


Fig. 7.25: Pseudo-static resistances of sub-assemblages with different horizontal restraints

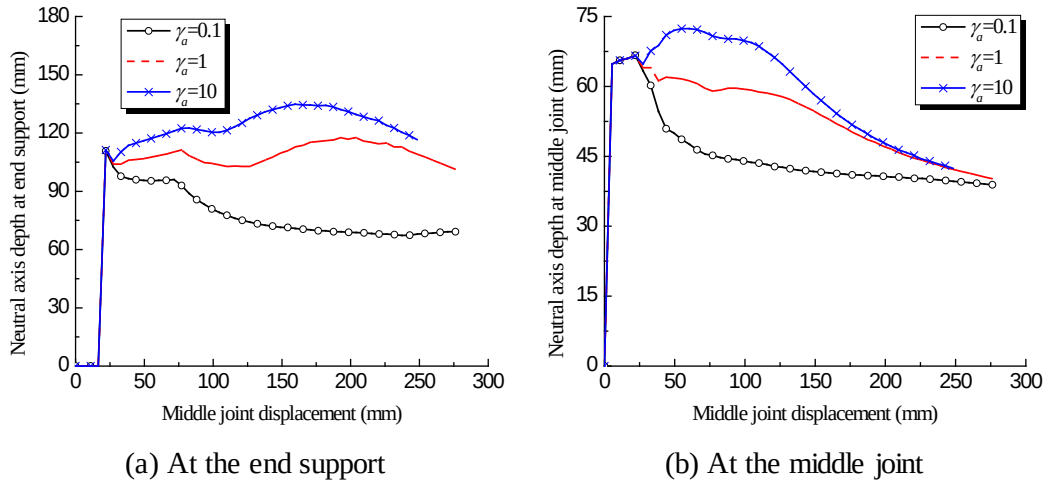


Fig. 7.26: Comparisons of neutral axis depths with different horizontal restraints

Under column removal scenarios, the pseudo-static resistance of the sub-assemblage is related to its quasi-static resistance and ductility (Izzudin et al. 2008). By increasing the stiffness of horizontal restraint, quasi-static resistance is significantly increased, as shown in **Fig. 7.24(a)**. However, its ductility is reduced due to deeper neutral axis depths at the middle joint and end support, as shown in **Fig. 7.26**. Thus, a limited increase is obtained in the pseudo-static resistance of the sub-assemblage. When a weak horizontal restraint is provided for the sub-assemblage, say $\gamma_a = 0.1$, CAA capacity is reduced, but ductility is improved due to lower neutral axis depths at the beam ends, as shown in **Fig. 7.26**. Therefore, pseudo-static resistance of the sub-

assembly is slightly reduced, but vertical displacement corresponding to the pseudo-static resistance is increased.

7.5.5 Effect of reinforcement ratio

To investigate the effect of the top and bottom reinforcement ratios on the CAA capacity of the sub-assembly, two hypothetic beam-column sub-assemblies EMJ-B-1.48/0.59 and EMJ-B-1.19/0.89, with identical geometry and boundary conditions to EMJ-B-1.19/0.59, are simulated using the analytical model, as listed in **Table 7.3**. In comparison with EMJ-B-1.19/0.59, the top reinforcement ratio is increased from 1.19% to 1.48% in EMJ-B-1.48/0.59, and the bottom reinforcement ratio is increased from 0.59% to 0.89% in EMJ-B-1.19/0.89. Total reinforcement area in EMJ-B-1.48/0.59 and EMJ-B-1.19/0.89 remains the same.

Table 7.3: Reinforcement ratios in beam-column sub-assemblages

Specimen	Top reinforcement		Bottom reinforcement	
	Area (mm ²)	Ratio	Area (mm ²)	Ratio
EMJ-B-1.19/0.59	534.9 (2H16+H13)	1.19%	265.5 (2H13)	0.59%
EMJ-B-1.48/0.59	667.6 (2H16+2H13)	1.48%	265.5 (2H13)	0.59%
EMJ-B-1.19/0.89	534.9 (2H16+H13)	1.19%	402.1 (2H16)	0.89%

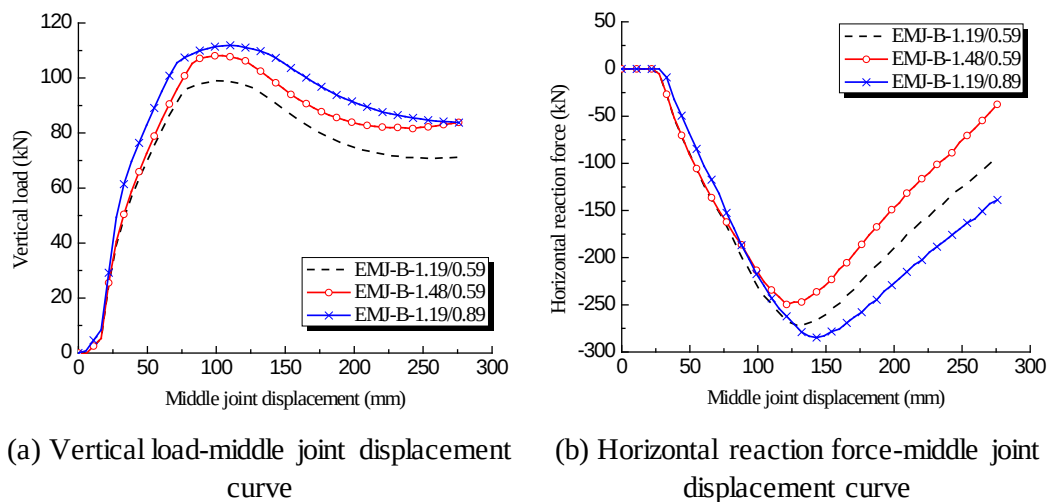


Fig. 7.27: Comparisons of load-displacement curves with different reinforcement ratios

Fig. 7.27 shows the predicted vertical loads and horizontal compression forces. EMJ-B-1.48/0.59 and EMJ-B-1.19/0.89 exhibit greater CAA capacities in comparison with

EMJ-B-1.19/0.59, as shown in **Fig. 7.27(a)**. The CAA capacity of EMJ-B-1.19/0.89 is 3.5% greater than that of EMJ-B-1.48/0.59, indicating that a greater bottom reinforcement ratio is more efficient to enhance the CAA capacity of the sub-assembly when the same amount of longitudinal reinforcement is placed in the beam. Furthermore, horizontal compression force is 14% greater in EMJ-B-1.19/0.89 than that in EMJ-B-1.48/0.59, as shown in **Fig. 7.27(b)**. At the section level, a greater top reinforcement ratio in the beam of EMJ-B-1.48/0.59 increases the neutral axis depth at the end support but decreases the depth at the middle joint compared to EMJ-B-1.19/0.59, as shown in **Figs. 7.28(a and b)**. However, in EMJ-B-1.19/0.89, the neutral axis depth at the end support is reduced but the depth at the middle joint is increased compared to EMJ-B-1.19/0.59, as shown in **Figs. 7.28(a and b)**, due to a greater bottom reinforcement ratio in the beam of EMJ-B-1.19/0.89.

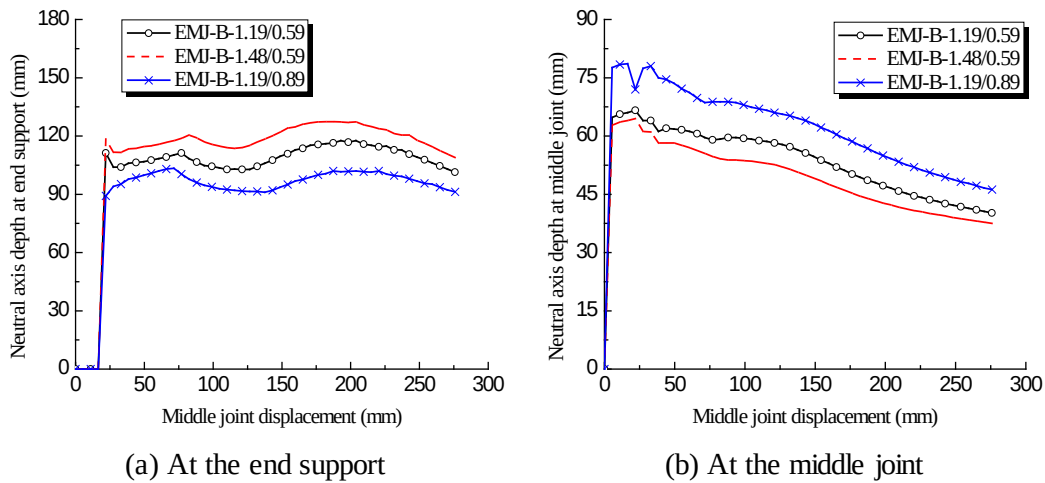


Fig. 7.28: Comparisons of neutral axis depths with different reinforcement ratios

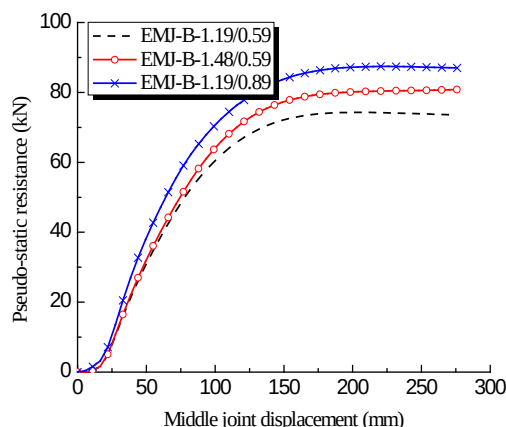


Fig. 7.29: Pseudo-static resistances of sub-assemblages with different reinforcement ratios

More significant differences exist between the pseudo-static resistances of sub-assemblages EMJ-B-1.48/0.59 and EMJ-B-1.19/0.89, as shown in **Fig. 7.29**, even though the same amount of longitudinal reinforcement is placed in the beam. The pseudo-static resistance of EMJ-B-1.19/0.89 is 8.2% greater than that of EMJ-B-1.48/0.59, since both the CAA capacity and ductility are enhanced in EMJ-B-1.19/0.89 due to a greater bottom reinforcement ratio in the beam. Thus, in order to enhance the pseudo-static resistance of beam-column sub-assemblages under column removal scenarios, more bottom reinforcement is suggested to be placed in the beam, which reduces the neutral axis depth and improves the ductility of the bridging beam at the hogging region.

7.6 Conclusion

Based on the rigid-plastic assumption, an analytical model is proposed to predict the CAA capacity of beam-column sub-assemblages under column removal scenarios. A linear variation of reinforcement strain along the beam length is assumed in the model. Instead of employing equivalent rectangular concrete stress block, Mander's stress-strain model for concrete is incorporated to take account of the softening branch of concrete in compression. Tensile strength and strain capacity of ECC in tension can also be considered if needed. The model is calibrated by experimental results on reinforced concrete and ECC beam-column sub-assemblages and is able to estimate the CAA capacity with reasonably good accuracy.

A series of parametric studies is conducted through the analytical model, in which the effects of different concrete models, tensile strength and strain capacity of ECC, stiffness of horizontal restraints, and top and bottom reinforcement ratios are investigated. The following conclusions are drawn from the parametric studies.

(1) Concrete model does not have a significant influence on the CAA capacity of sub-assemblages, but it affects the descending phase of vertical load and maximum compression force due to dramatically different softening branches of the concrete stress-strain models. Thus, concrete model with a moderate softening branch has to be selected. Analytical results indicate that Mander's model (Mander et al. 1988) provides the best agreement with experimental results.

(2) CAA capacity of sub-assemblages increases with tensile strength of ECC if the strain capacity of ECC is kept constant. When the strain capacity is 2.6%, the maximum enhancement of ECC to the CAA capacity of sub-assemblages is 9% for a practical range of tensile strength from 3.1 MPa to 6.2 MPa. However, the maximum horizontal compression force in the beam decreases with increasing tensile strength of ECC.

(3) Increasing the tensile strain capacity of ECC also increases the CAA capacity of sub-assemblages when the strain capacity is low. When the tensile strain capacity is 2.6%, the CAA capacity of the sub-assemblage is increased by 4.8% compared to the sub-assemblage with conventional concrete of the same strength as ECC. However, an upper bound value of the strain capacity exists, beyond which the CAA capacity cannot be enhanced by a greater strain capacity of ECC. For sub-assemblage EMJ-B-1.19/0.59, the value is determined as 0.6% through the proposed model. Thus, a tensile strain capacity of 2.6% obtained in the experimental tests is adequate to ensure the maximum enhancement of ECC to the CAA capacity of sub-assemblage EMJ-B-1.19/0.59.

(4) Horizontal restraint with a higher stiffness increases CAA capacity but reduces ductility of sub-assemblages. Therefore, when the quasi-static resistance of sub-assemblages is converted to the pseudo-static resistance through the energy balance method, the effect of a stiffer horizontal restraint on the resistance of sub-assemblages

becomes limited. Nonetheless, the vertical displacement corresponding to the pseudo-static resistance is substantially reduced by a stiffer horizontal restraint.

(5) A greater top or bottom reinforcement ratio increases the CAA capacity of sub-assemblages. However, increasing the bottom reinforcement ratio in the beam is more efficient to enhancing the CAA capacity of sub-assemblages. Analytical results also suggest that more bottom reinforcement should be provided at the hogging region of the beam to improve ductility and to increase the pseudo-static resistance. Nonetheless, it may hinder the development of catenary action at large deformation stage.

Limitations exist in the analytical model. The assumption of linear strain profile between the inflection point and beam end sections substantially underestimate the strain of top and bottom steel reinforcement at locations where fracturing of reinforcement occurs. Rupture of tension reinforcement cannot be accurately predicted in the model. Additionally, spalling of concrete is neglected in the model, which may lead to increasing discrepancy between the measured and predicted horizontal compression forces at large deformation stage.

CHAPTER 8 COMPONENT-BASED JOINT MODEL FOR PRECAST CONCRETE BEAM-COLUMN SUB-ASSEMBLAGES

8.1 Introduction

The component-based joint model can be used for analysis of reinforced concrete structures subjected to various loading conditions (Bao et al. 2008; Lowes and Altoontash 2003; Mitra and Lowes 2007; Yu and Tan 2013b). In the model, interactions between the structural member and beam-column joint are simplified as a group of nonlinear zero-length springs, namely, tensile, compressive and shear springs (Lowes et al. 2004). The force-slip relationships of tensile and compressive springs can be determined by the bond-slip behaviour of embedded reinforcement in the joint.

Tension force in reinforced concrete members is assumed to be sustained by steel reinforcement. Transmission of tension force from beams or columns to structural joints depends on the bond-slip behaviour of the embedded reinforcement. According to its anchorage length, two types of boundary conditions have been considered for embedded reinforcement in axial tension (Yu and Tan 2010b). If the embedment length of reinforcement is sufficiently long to ensure zero strain in the middle, the bond-slip model developed by Lowes et al. (2004) can be used; otherwise, non-zero strain in the middle of the embedment length has to be considered. However, Bond stresses of steel reinforcement at elastic and post-yield stages are substantially different from experimental results by Lehman and Moehle (2000). Thus, bond stresses have to be calibrated against test data of steel bars under pull-out force. Besides, pull-out failure of reinforcement anchored in the middle joint, as discussed in Chapter 3, cannot be taken into account in the model. As a result, further improvement is necessary in the bond-slip behaviour of reinforcement with various embedment lengths.

In the compression zone, total compression force is contributed by both compressive concrete and reinforcement and it varies with neutral axis depth and the amount of compressive reinforcement. The contribution of concrete is generally determined from the equivalent concrete stress block (Lowes et al. 2004; Yu 2012). It seems

reasonable to evaluate the seismic resistance of beam-column joints subjected to cyclic loadings. However, under column removal scenarios, severe crushing and spalling of concrete occurred in the compression zone. Thus, constitutive model for concrete should be used in calculating the compressive force instead of the equivalent concrete stress block.

This chapter presents a component-based model for precast concrete beam-column joints. In the model, bond stresses of tensile steel reinforcement at elastic and post-yield stages are re-evaluated and calibrated against experimental results. Besides, pull-out failure is also considered for reinforcing bars with inadequate embedment length. Various failure modes of reinforcement can be considered through defining the force-slip relationship of tensile springs. As for compressive springs, softening branch of the force-slip curve caused by crushing of compressive concrete is taken into account. The model is calibrated by experimental results of precast and reinforced concrete sub-assemblages under column removal scenarios.

8.2 Beam-Column Joint Model

When interface delamination between precast beam units and cast-in-situ concrete topping is prevented, the component-based joint model is applicable to precast concrete beam-column sub-assemblages. **Fig. 8.1** shows the component model for precast concrete beam-column joints in Engineers' Studio (Forum8 2008). In the sub-assemblage, the precast beam is modelled with 2-node fibre elements, in which a layered cross section with different compressive strengths of concrete is used and delamination of horizontal interfaces is assumed not to occur. The end column stub is simulated by elastic elements. Concrete model proposed by Mander et al. (1988) is employed for concrete in the beam and end column stub. Trilinear stress-strain model with yield plateau is utilised for longitudinal reinforcement in the sub-assemblage.

At the interface of the middle joint, three zero-length springs are used to connect the beam to the joint, as shown in **Fig. 8.1**. Bottom longitudinal reinforcement passing through the middle joint sustains tension force under column removal scenarios. Thus, a tensile spring is defined and located at the centroid of the bottom bars in the beam. In the compression zone, concrete and compressive reinforcement carry compressive

stresses and a compressive spring is placed at the centroid of the compression zone. The spring can also sustain tension force after the fracture of bottom springs at large deformation stage. A linear-elastic shear spring is defined at the interface to transmit shear force between the beam and middle joint. In the middle joint, four rigid elements are connected by pin nodes to form the joint panel, and two diagonal springs with infinite stiffness are defined, since shear distortions of the middle joint are negligible under column removal scenarios. Similar springs are also provided at the interface of the end column stub. Prior to analysis, constitutive properties of the springs have to be defined in accordance with material and geometric properties.

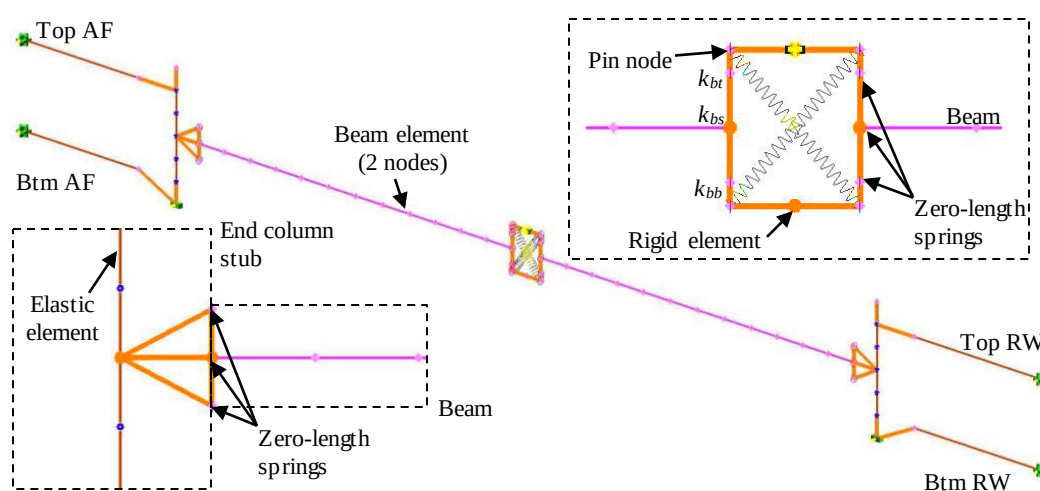


Fig. 8.1: Component-based joint model for beam-column sub-assemblages

8.3 Properties of Tensile Spring

When subjected to tension force from one end, embedded reinforcement with various anchorage lengths exhibits different failure modes, as summarised in **Table 8.1**. A “sufficiently long” embedment length enables a reinforcing bar to rupture and the free end slip is zero even at the ultimate strength of the bar. This can be found in column longitudinal reinforcement anchored in a reinforced concrete footing. Although reinforcement with a “long” embedment length mobilises slip at the free end, it can still rupture in tension. “Short” reinforcement develops inelastic behaviour at the loaded end and eventually exhibits pull-out failure in tension. With a further reduction in embedment length to “extremely short”, embedded reinforcement shows pull-out failure at the elastic stage. In addition to pull-out force, continuous beam

bottom longitudinal reinforcement passing through the middle joint sustains axial tension force under column removal scenarios (Yu and Tan 2010b). Under such a loading condition, reinforcement is able to rupture however long the embedment length is.

Table 8.1: Failure modes of embedded bars subjected to pull-out force

Embedment length	Free end slip	Stress state at the loaded end	Failure mode
Sufficiently long	Zero	Post-yield	Rupture
Long	Non-zero	Post-yield	Rupture
Short	Non-zero	Post-yield	Pull-out
Extremely short	Non-zero	Elastic	Pull-out

In accordance with different anchorage lengths and loading conditions, Shima et al. (1987) categorised three boundary conditions, namely, zero strain with zero slip, zero strain with non-zero slip and non-zero strain with zero slip, for embedded steel reinforcement in concrete. Zero strain with zero slip represents an embedded reinforcement with a “sufficiently long” embedment length to ensure zero slip and zero strain at the free end. Zero strain with non-zero slip refers to steel reinforcement with non-zero slip at the free end due to a “long” or “short” or “extremely short” embedment length. Non-zero strain with zero slip represents a reinforcing bar in axial tension and with zero slip and non-zero strain at the mid-point of the embedment length. It is noteworthy that the foregoing categories apply to straight bars. In case of reinforcing bars with hooked anchorage, an equivalent length of $l_s + 5d$ is suggested (Filippou et al. 1983), where l_s is the length of the straight portion in front of the hook and d is the diameter of steel reinforcement.

8.3.1 Zero strain with zero slip

Two categories of bond-slip model, viz. micro-model and macro-model, have been classified by Sezen and Setzler (2008) for studies on bond stress. A micro-model always demands a nested iteration procedure to obtain the slip of an embedded reinforcing bar with respect to concrete through various bond stress-slip models (Eligehausen et al. 1983; Shima et al. 1987), whereas stepped or piecewise bond stress distribution along the embedment length can be assumed in a macro-model so as to substantially reduce computational cost but optimise the accuracy (Alsiwat and

Saatcioglu 1992; Lehman and Moehle 2000; Sezen and Setzler 2008). Therefore, macro-models have been widely applied to structural analysis under earthquake loadings (Lowes et al. 2004; Mitra and Lowes 2007) and progressive collapse scenarios (Bao et al. 2008; Lew et al. 2011; Yu and Tan 2010b). In the macro-model-based simulation of reinforced concrete joints, predicted behaviour is very sensitive to the postulated average bond stresses along the embedment length of steel bars. Hence, the average bond stress has to be elaborately derived based on micro-models and calibrated by experimental results.

8.3.1.1 Average bond stress at elastic stage

In calculating the average bond stress at the elastic stage, Lowes et al. (2004) utilised the bond-slip model proposed by Eligehausen et al. (1983) and postulated that zero and the maximum bond stress in the model are developed at two ends of an embedded reinforcement. Average bond stress at the elastic stage was determined as $1.8\sqrt{f'_c}$, where f'_c is the cylinder compressive strength of concrete. Recently, Yu (2012) used the same value in deriving the joint model for beam-column joints under column removal scenarios. It is noteworthy that the bar yields far before the maximum bond stress is attained in Eligehausen's bond-slip model. Thus, the average bond stress is considerably overestimated. Further analytical studies are needed to quantify the average bond stress at the elastic stage.

In this chapter, the bond stress-strain-slip model proposed and calibrated by Shima et al. (1987) is used to calculate the average bond stress along the embedment length of reinforcement. For each reinforcing bar, it is assumed that the yield strength is attained at the loaded end and the embedment length of reinforcement is adequate to ensure zero strain and zero slip at the free end. A nested iteration procedure is employed to determine the required embedment length of reinforcement. **Table 8.2** lists the properties of steel reinforcement and concrete and the calculated average elastic bond stresses. The mean value is 4.97 MPa, with a coefficient of variation of 18%. Typically, average bond stress is expressed as a function of $\sqrt{f'_c}$ to consider the influence of concrete compressive strength (Eligehausen et al. 1983; Lehman and

Moehle 2000). Accordingly, the value becomes $1.0\sqrt{f'_c}$, with a coefficient of variation of 9%.

Table 8.2: Average bond stress predicted by Shima's model

Author	Item	Steel properties			Concrete compressive strength (MPa)	Average bond stress		
		Diameter (mm)	Yield strength (MPa)	Elastic modulus (GPa)		MPa	In $\sqrt{f'_c}$	In $\sqrt{f'_c f_y}$
Shima et al. (1987)	SD30	19.5	350	190.0	19.6	3.70	0.84	0.045
	SD50		610			4.55	1.03	0.042
	SD70		820			4.95	1.12	0.039
Bigaj (1995)	B16	16	539.67	128.5	27.62	5.21	0.99	0.043
	B20	20	526.24	150.3		5.05	0.96	0.042
Yu (2012)	T13	13	494	185.9	38.2	6.35	1.03	0.046
Mean value						4.97	1.00	0.043
Coefficient of variation						18%	9%	6%

Besides the average bond stress when the yield strength of reinforcement is attained at the loaded end, variation of bond stresses along the embedment length of reinforcement is obtained through the bond-slip model, as shown in **Fig. 8.2**. For steel reinforcement SD-30, SD-50 and SD-70 with the same diameter but different yield strengths, nonlinear bond stress distribution along the embedment length is mobilised to transfer steel stress to the surrounding concrete. The calculated average bond stresses vary significantly from one another for steel reinforcement SD-30, SD-50 and SD-70 (see **Table 8.2**). In order to capture the effect of reinforcement yield strength, the average bond stress is expressed as a function of $\sqrt{f'_c f_y}$, as listed in **Table 8.2**. Therefore, the average bond stress is determined as $0.043\sqrt{f'_c f_y}$, with a coefficient of variation of 6%, when reinforcement is loaded to yield.

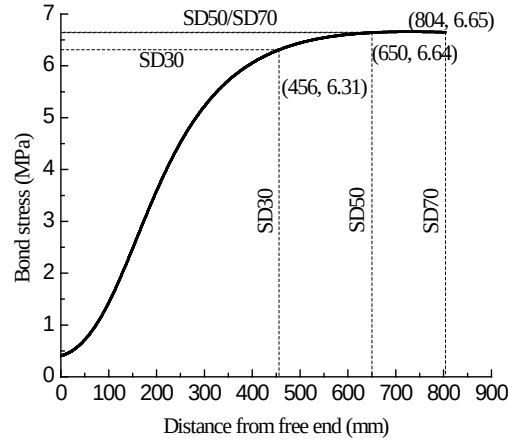


Fig. 8.2: Variation of bond stresses along embedment length of a reinforcing bar

A similar method can be used to determine the average bond stress of steel bars when tensile stress at the loaded end is less than the yield strength. **Fig. 8.3(a)** shows the average bond stress along the embedment length when different tensile stresses are applied at the loaded end of reinforcement SD-70. When the tensile stress is 350 MPa, the associated average bond stress is 3.71 MPa. If the tensile stress is increased to 820 MPa, the average bond stress becomes 4.95 MPa. **Fig. 8.3(b)** shows the variation of normalised average bond stress by $\sqrt{f_s}$, where f_s is the tensile stress at the loaded end of reinforcing bars. It is observed that the normalised average bond stress increases first, and then slightly decreases with increasing steel stress from zero to 820 MPa. Therefore, a mean value of 0.19 can be taken in the range of steel stresses from zero to 820 MPa. Correspondingly, for concrete with a compressive strength of 19.6 kN, the average bond stress can be expressed as $0.043\sqrt{f'_c f_s}$. When steel reinforcement yields at the loaded end, the average bond stress is $0.043\sqrt{f'_c f_y}$.

$$\tau_e = 0.043\sqrt{f'_c f_s} \quad (8-1)$$

$$s = \frac{f_s^2 d}{8\tau_e E_s} \quad (8-2)$$

where τ_e is the bond stress at the elastic stage of steel reinforcement; f'_c is the cylinder compressive strength of concrete; f_s is the tensile stress at the loaded end of

reinforcement; d is the diameter of reinforcement; s is the slip at the loaded end; and E_s is the modulus of elasticity of reinforcement.

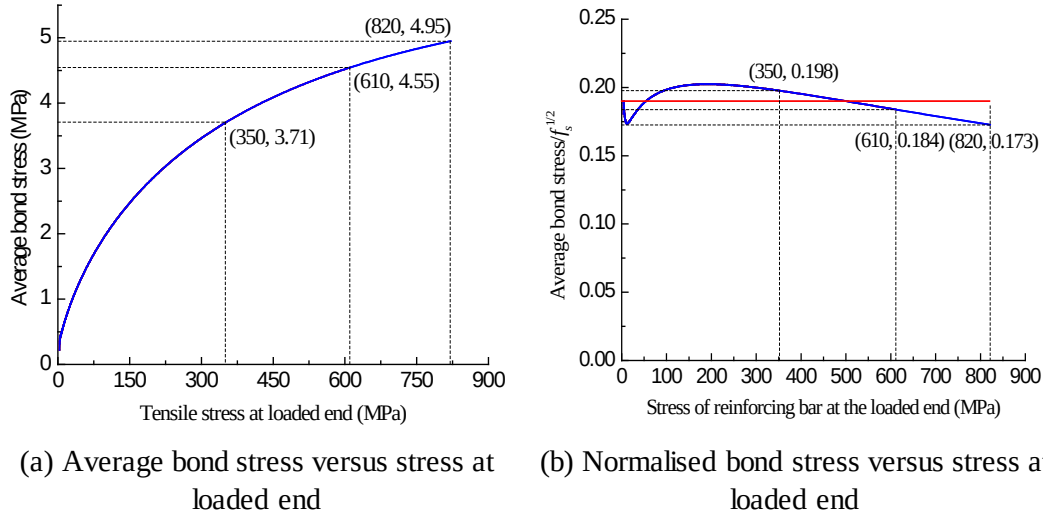


Fig. 8.3: Relationship of average bond stress with force and stress at the loaded end of reinforcing bars

After determining the average bond stress from Eq. (8-1), reinforcing bars SD30, SD50 and SD70 tested by Shima et al. (1987) are simulated using the macro-model (Lowes et al. 2004), as expressed in Eq. (8-2). Besides, comparisons are made among the force-slip curves predicted by different models (Lowes et al. 2004; Shima et al. 1987; Soltani and Maekawa 2008). **Fig. 8.4** suggests that Lowes's model provides the stiffest force-slip curve among all the models, as it greatly overestimates the average bond stress (i.e. $1.8\sqrt{f'_c}$) at elastic stage. By using the same bond-slip model as Lowes et al. (2004) but a lower average bond stress (i.e. $0.043\sqrt{f'_c f'_s}$), a much softer force-slip relationship is obtained at elastic stage. Reasonably good agreement is reached between the force-slip curves by the proposed bond stress and the analytical model by Soltani and Maekawa (2008), if bond deterioration zone is not considered in the analysis. It is noteworthy that Shima's micro bond stress-strain-slip model gives stiffer force-slip response in comparison with the proposed bond stress, even though the average bond stress is derived based on Shima's model. It results from the fact that only force equilibrium along the whole embedment length is considered and compatibility between finite steel segments is neglected in macro-models.

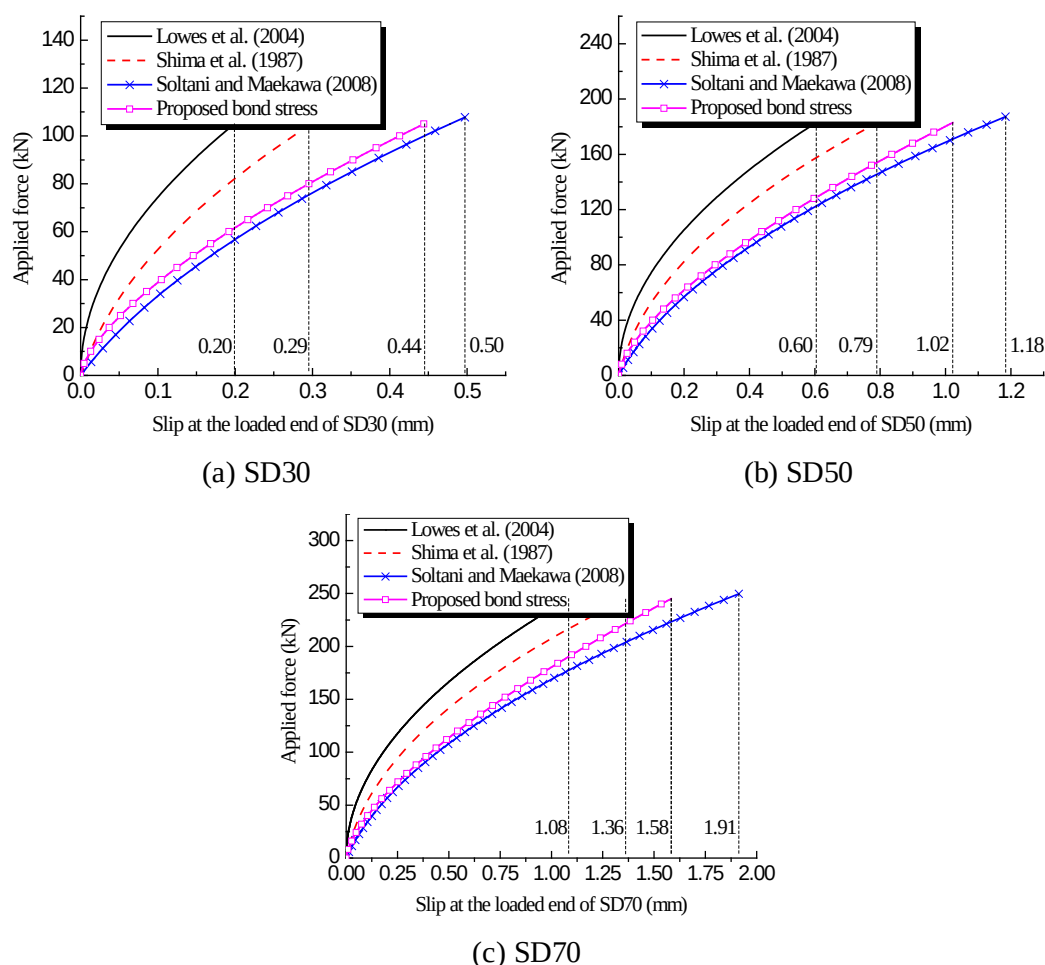


Fig. 8.4: Relationships of applied force and loaded end slip for steel bars

8.3.1.2 Bond stress at post-yield stage

Once steel reinforcement enters into its post-yield stage, bond stress is suddenly reduced due to stress redistribution at the interface of the steel bar and surrounding concrete (Bigaj 1995). Shima et al. (1987) quantified the post-yield bond stress as $0.40\sqrt{f'_c}$ in experimental tests. At the post-yield stage of steel reinforcement, concrete keys between steel lugs are sheared off due to inelastic elongation of the bars. It is similar to the pull-out phase of short reinforcement controlled by frictional bond stress (Alsiwat and Saatcioglu 1992; Pochanart and Harmon 1989), except the Poisson effect on the steel bars, namely, contraction of steel cross section due to inelastic tensile elongation. Therefore, frictional bond stress given by Eligehausen et al. (1983) can be used to determine the bond stress at post-yield stage of reinforcement if the Poisson effect is considered.

It is reported that the Poisson effect reduces the bond stress by 20 to 30% when steel bars are subjected to tension (Eligehausen et al. 1983; Viwathanatepa et al. 1979). Hence, bond stress at post-yield stage can be taken as 70 to 80% of the frictional bond stress of reinforcement with a short embedment length. In order to calculate the bond stress at post-yield stage, the frictional bond can be multiplied by a factor of 0.75 to consider the Poisson effect. For short steel bars embedded in well-confined concrete of 30 MPa compressive strength, the frictional bond stress is quantified as 5 MPa (Eligehausen et al. 1983). The value becomes $0.91\sqrt{f'_c}$ when it is normalised by the concrete compressive strength. Thus, the post-yield bond stress can be determined as $0.68\sqrt{f'_c}$ for the given confining condition. However, the value is much greater than that proposed by Shima et al. (1987), possibly due to different confining conditions provided by both concrete cover and stirrups.

To eliminate the enhancement of good confinement provided by stirrups for seismic design, it is assumed that the post-yield bond stress is proportional to the maximum bond stress obtained at elastic stage of the embedded reinforcement. The ratio of the calculated post-yield bond stress (i.e. $0.68\sqrt{f'_c}$) to the maximum elastic bond stress (i.e. $2.46\sqrt{f'_c}$) obtained by Eligehausen et al. (1983) is 0.28. Shima et al. (1987) implies that the maximum bond stress is around 6.65 MPa for 19.6 MPa compressive strength of concrete. The normalised value by concrete compressive strength is $1.50\sqrt{f'_c}$. Accordingly, the post-yield bond stress can be calculated as $0.41\sqrt{f'_c}$. This value agrees well with the experimental results by Shima et al. (1987). In this chapter, a value of $0.4\sqrt{f'_c}$ will be used in the analysis, as expressed in Eq. (8-3), which may slightly overestimate the ultimate slip of embedded reinforcement.

$$\tau_y = 0.4\sqrt{f'_c} \quad (8-3)$$

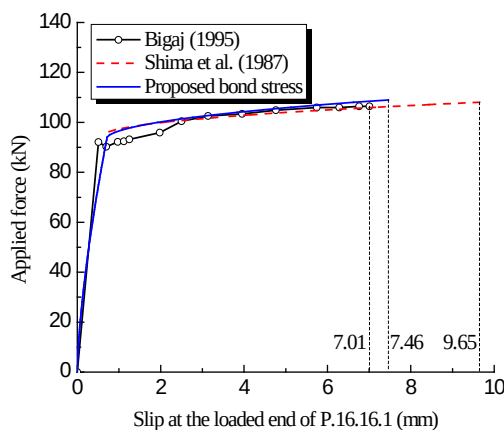
$$s = \frac{f_y^2 d}{8\tau_e E_s} + \frac{(f_s - f_y) f_y d}{4\tau_y E_s} + \frac{(f_s - f_y)^2 d}{8\tau_y E_h} \quad (8-4)$$

where τ_y is the bond stress at post-yield stage of reinforcement; f_y is the yield strength of steel reinforcement; and E_s is the hardening modulus of steel bars.

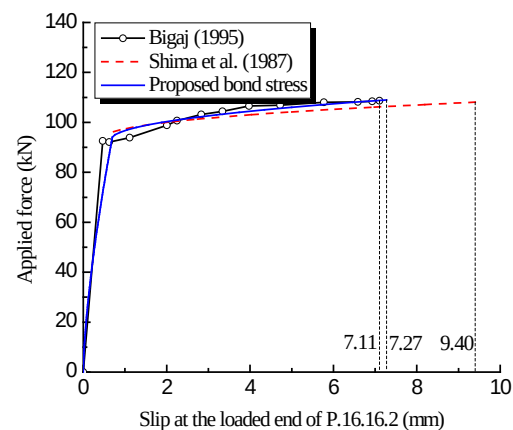
After determining the bond stresses at elastic and post-yield stages, the pull-out tests conducted by Bigaj (1995) are simulated through the macro-model proposed by Lowes et al. (2004), as expressed in Eqs. (8-2) and (8-4). The bond stress-strain-slip model proposed by Shima et al. (1987) is also employed for comparison. A bilinear stress-strain relationship is used for steel reinforcement. **Table 8.3** includes the material properties of steel reinforcement and concrete. **Fig. 8.5** shows the force-slip curves of reinforcement. It should be noted that the equivalent bar diameter is calculated from realistic contact area due to grooving. Slip represents the value at the starting point of the embedment length, measured 10 times the bar diameter from the specimen surface. Through comparisons, it is observed that force-slip response of the embedded reinforcing bars predicted by the proposed bond stresses agrees well with the experimental results. However, Shima's model substantially overestimates the ultimate slip at rupture of steel bars.

Table 8.3: Material properties of embedded reinforcement (Bigaj 1995)

Specimen	Steel properties						Concrete comp. strength (MPa)
	Diameter (mm)	Area (mm ²)	Yield strength (MPa)	Elastic modulus (GPa)	Ultimate tensile strength (MPa)	Hardening modulus (MPa)	
P.16.16.1	16	174.2	539.67	128.5	624.35	945	26.98
P.16.16.2							28.36
P.20.16.1	20	280.9	526.24	150.4	612.87	952	28.36
P.20.16.2							26.78



(a) P.16.16.1



(b) P.16.16.2

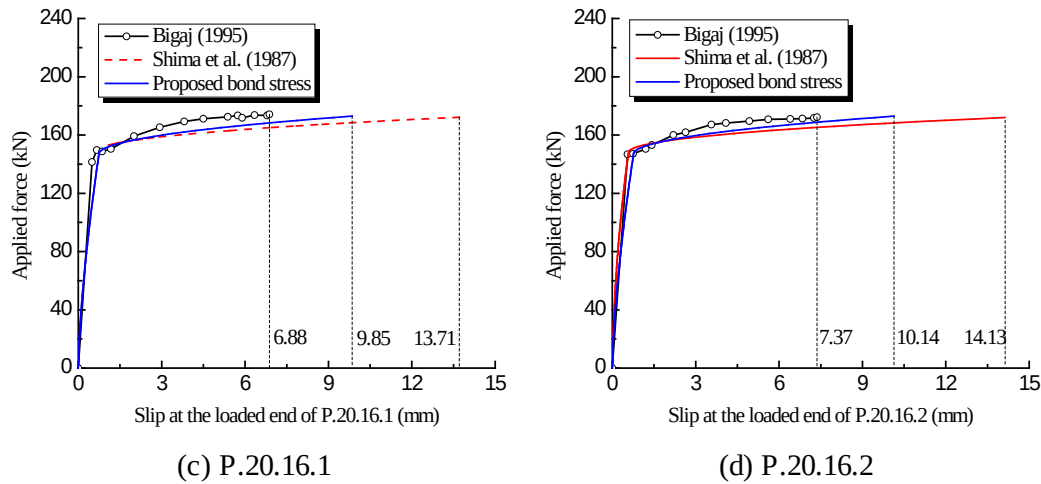


Fig. 8.5: Comparisons of experimental and analytical force-slip relationships

8.3.2 Zero strain with non-zero slip

In reinforced concrete structures designed against seismic loading conditions, sufficient embedment length is provided for tension reinforcement and failure is defined by rupture of steel bars (Sadek et al. 2011; Yu and Tan 2010a). However, in precast concrete structures with non-seismic design, bottom reinforcement in the middle joint may not be able to develop rupture due to inadequate embedment length. Therefore, it is essential to take account of pull-out failure in analysing the behaviour of beam-column sub-assemblages subject to progressive collapse.

Elgehausen et al. (1983) tested short embedded reinforcement in beam-column joints under monotonic loadings and proposed a bond-slip model in accordance with the experimental results. The model is valid for steel bars with pull-out failure at elastic stage. When it is applied for reinforcement with inelastic pull-out failure, the ultimate capacity is greatly overestimated (Monti et al. 1993). Viathanatapa et al. (1979) investigated the inelastic bond-slip behaviour and pull-out failure of ribbed bars embedded in reinforced concrete columns. Nonetheless, little attention was paid to the post-yield bond stress in the proposed bond-slip model. Indeed, bond stress decreases dramatically at post-yield stage of embedded reinforcement (Shima et al. 1987), due to contraction of rebar cross section and shearing-off of concrete keys between steel lugs (Bigaj 1995). Huang et al. (1996) defined a bilinear bond stress-slip relationship to take account of post-yield bond stress, in which four sets of parameters are provided in terms of bond conditions and concrete strength. This

model provides a possible solution for assessing the pull-out behaviour of steel bars at post-yield stage. Nevertheless, compared with the experimental results by Shima et al. (1987) and Bigaj (1995), the model significantly overestimates the post-yield bond stress.

To accurately evaluate the bond-slip behaviour and potential failure mode of short steel reinforcing bars embedded in concrete, a new analytical approach is proposed, in which special attention is paid to the reduction of bond stress and pull-out failure at post-yield stage. Additionally, this approach is further simplified and calibrated through published experimental results.

8.3.2.1 Analytical approach

In deriving the load-slip relationship of embedded reinforcing bars, the bond-slip relationship proposed by Huang et al. (1996) is used for the elastic segments, as expressed in Eq. (8-5). Compared with Eligehausen's mode, it is capable of capturing the decreasing frictional bond stress at large slips. At post-yield stage, bond stress is approximately assumed to be uniform over the yielded steel segments, as postulated in the beam-column joint model by Lowes et al. (2004).

$$\begin{aligned}
 \tau &= \tau_1 \left(\frac{s}{s_1} \right)^{0.4} \quad \text{for } s \leq s_1, \quad \varepsilon \leq \varepsilon_y \\
 \tau &= \tau_1 \quad \text{for } s_1 < s \leq s_2, \quad \varepsilon \leq \varepsilon_y \\
 \tau &= \tau_2 + (\tau_1 - \tau_2)(s_3 - s)/(s_3 - s_2) \quad \text{for } s_2 < s \leq s_3, \quad \varepsilon \leq \varepsilon_y \\
 \tau &= \tau_2 (s_4 - s)/(s_4 - s_3) \quad \text{for } s_3 < s \leq s_4, \quad \varepsilon \leq \varepsilon_y \\
 \tau &= \tau_y \quad \text{for } \varepsilon > \varepsilon_y
 \end{aligned} \tag{8-5}$$

where τ_1 , τ_2 and τ_y are the maximum bond stress, onset of frictional bond and post-yield bond stress, respectively, which are quantified through experimental results; s_1 and s_2 are slips, with $s_1 = 1$ mm and $s_2 = 3$ mm; s_3 is the clear spacing of steel lugs and taken as 10.5 mm; s_4 is the slip when bond stress between concrete and reinforcement is zero and it is taken as $3s_3$ (FIB 2000; Huang et al. 1996); ε is the strain of steel segment; and ε_y is the yield strain of steel bar.

As for a reinforcing bar with an “extremely short” embedment length, the bond-slip model shown in **Fig. 8.6** can be employed in the whole loading process. Nonetheless, reinforcement with a “short” embedment length, as defined in **Table 8.1**, exhibits post-yield behaviour when subjected to pull-out force. The post-yield bond stress has to be determined for the yielded steel segments, whereas the model shown in **Fig. 8.6** is still valid for the elastic steel segments. Beyond its ultimate load capacity, a descending branch exists and unloading of yielded steel segments occurs (Engström et al. 1998). Bond stress along yielded steel segments is assumed to be identical to the post-yield bond stress, as concrete keys between steel lugs have been sheared off and bond stress cannot be restored even when steel strains decrease with increasing loaded end slip.

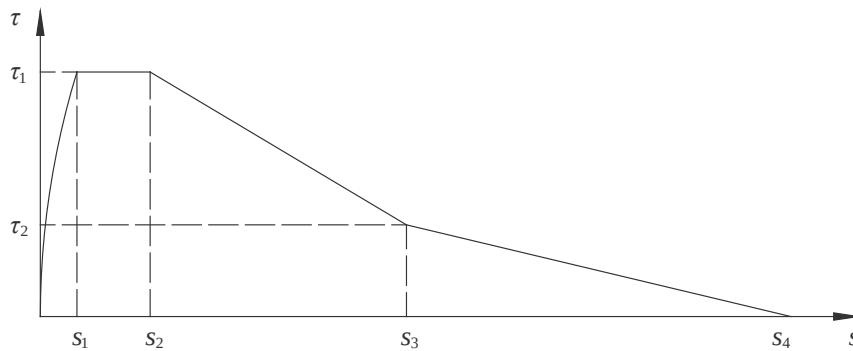


Fig. 8.6: Bond-slip model for embedded reinforcing bars at elastic stage

The aforementioned bond-slip relationship holds for well-confined concrete, namely, thick concrete cover or adequate stirrups are provided in the concrete specimens such that tension splitting cracks can be arrested.

8.3.2.2 Determination of bond stresses

In previous studies, Elgehausen et al. (1983) recommended the maximum elastic bond stress and the frictional bond through pull-out tests on embedded reinforcement. Once reinforcing bars enter into inelastic stage under pull-out loads, bond stress at the post-yield stage has to be determined in order to evaluate the bond-slip behaviour and failure mode of the embedded reinforcement. As discussed in Section 8.3.1, the frictional bond stress given by Elgehausen et al. (1983) can be used to calculate the post-yield bond stress if Poisson effect is considered. It is reported that the Poisson effect contributes to 20 to 30% increase in the bond stress of steel bars in compression

(Elgehausen et al. 1983; Viwathanatepa et al. 1979). A value of 25% is selected and the post-yield bond stress can be quantified as $0.68\sqrt{f'_c}$.

The post-yield bond stress can also be estimated from the embedded bars featuring pull-out failure at post-yield stage (Engström et al. 1998; Ueda et al. 1986; Viwathanatepa et al. 1979). When the maximum load is applied to reinforcement, a stepwise uniform bond stress profile is assumed over the embedment length, as shown in **Fig. 8.7**. The maximum elastic bond stress τ_1 is attained along the elastic steel segments and post-yield bond stress τ_y is over the yielded segments. As the ratio of τ_y to τ_1 is 0.28 as introduced in Section 8.3.1, bond stresses τ_1 and τ_y can be determined in accordance with the force equilibrium of the elastic and yielded steel segments. **Table 8.4** shows the bond stresses of embedded reinforcement tested by Viwathanatepa et al. (1979), Ueda et al. (1986) and Engström et al. (1998). It is noteworthy that failure cone close to the loaded end is considered by deducting its length from the total embedment length, and therefore the effective embedment length is used in calculating the bond stresses. The average post-yield bond stress under pull-out forces can be quantified as $0.66\sqrt{f'_c}$, close to the value calculated from Elgehausen's model by considering the Poisson effect.

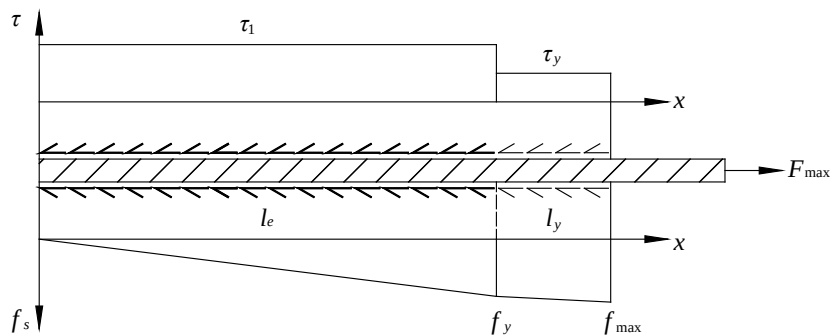


Fig. 8.7: Bond stress distribution along a reinforcing bar at the peak pull-out force

Table 8.4: Bond stress of embedded steel bars under pull loading condition

Author	Specimen	Effective embedment length (mm)	Bond stress (with f'_c in MPa)		
			$\tau_1/\sqrt{f'_c}$	$\tau_2/\sqrt{f'_c}$	$\tau_y/\sqrt{f'_c}$
Viwathanatepa et al. (1979)	#3	546	2.93	1.08	0.82

Ueda et al. (1986)	S61	330	2.48	0.91	0.69
	S62	330	2.66	0.96	0.74
	S63	330	1.88	0.69	0.53
	S101	532	3.13	1.16	0.88
	S102	532	2.58	0.95	0.72
	S104	532	1.62	0.59	0.45
	S105	532	2.15	0.79	0.60
	S106	532	2.43	0.90	0.68
	S107	532	1.87	0.69	0.52
Engström et al. (1998)	N290b	260	2.19	0.80	0.61
Mean value			2.36	0.87	0.66

After determining the bond stresses, a nested iteration procedure is employed to calculate the force-slip relationship of embedded bars. The proposed analytical approach is calibrated against published relevant experimental results (Engström et al. 1998; Ueda et al. 1986; Viwathanatepa et al. 1979).

8.3.2.3 Calibration of analytical approach

In calibrating the analytical approach, a bilinear stress-strain relationship of steel reinforcement is adopted. **Table 8.5** includes the material properties of reinforcement. Both the force-slip curves and bond stress distribution along the embedment length are obtained analytically. As bond stresses are directly computed from experimental results, the maximum loads sustained by embedded reinforcing bars are approximately identical to the experimental values, as shown in **Fig. 8.8**. Comparisons between the experimental and analytical force-slip relationships demonstrate that the proposed analytical approach yields reasonably accurate predictions of the ascending phase of load-slip curves. However, for the descending phase due to pull-out of embedded reinforcement from surrounding concrete, only the experimental result of N290b by Engström et al. (1998) is provided and the analytical result agrees well with it.

Table 8.5: Material properties of embedded bars

Author	Steel bar	Diameter (mm)	Elastic modulus (GPa)	Yield strength (MPa)	Hardening modulus (MPa)	Ultimate strength (MPa)
Viwathanatepa et al. (1979)	#3	25.4	201.3	468.5	2275	737.8

Engström et al. (1998)	N290b	16	200.0	569.0	921	648.0
Ueda et al. (1986)	S61	19.1	199.8	438.2	5925	775
	S101	32.3	203.9	414.1	5822	660.8

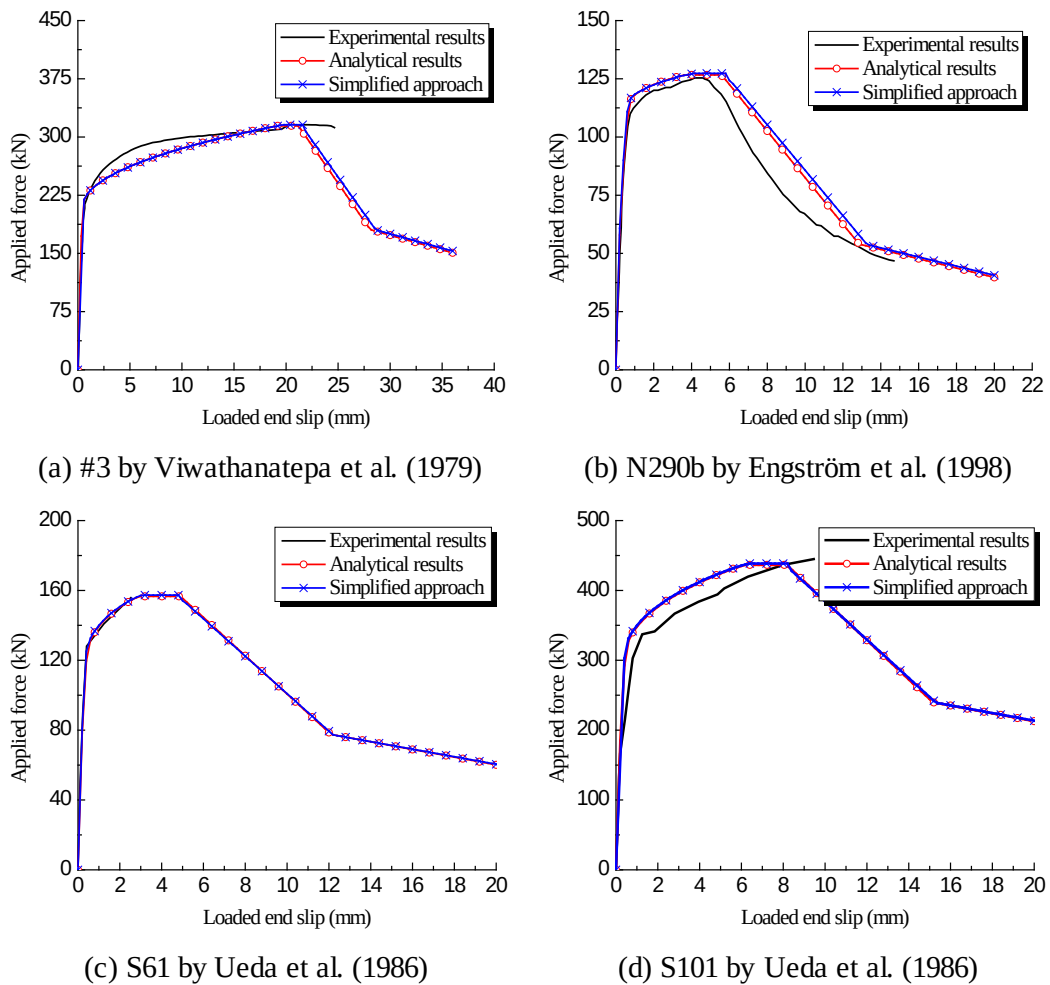


Fig. 8.8: Comparisons between experimental and analytical results under pull-out loads

Fig. 8.9 shows the bond stress profiles along the reinforcing bar of N290b. At elastic stage, bond stress varies almost linearly along the embedment length and attains the minimum value at the free end and the maximum value at the loaded end, as shown in **Fig. 8.9(a)**. Bond stress drops suddenly once plasticity kicks in near the loaded end of the steel bar. At the ultimate load capacity of reinforcement, the maximum elastic bond stress is reached along the elastic steel segments. Thereafter, unloading of reinforcement occurs and the bond stress at the free end is slightly greater than that at the section where the steel bar yields, as shown in **Fig. 8.9(b)**.

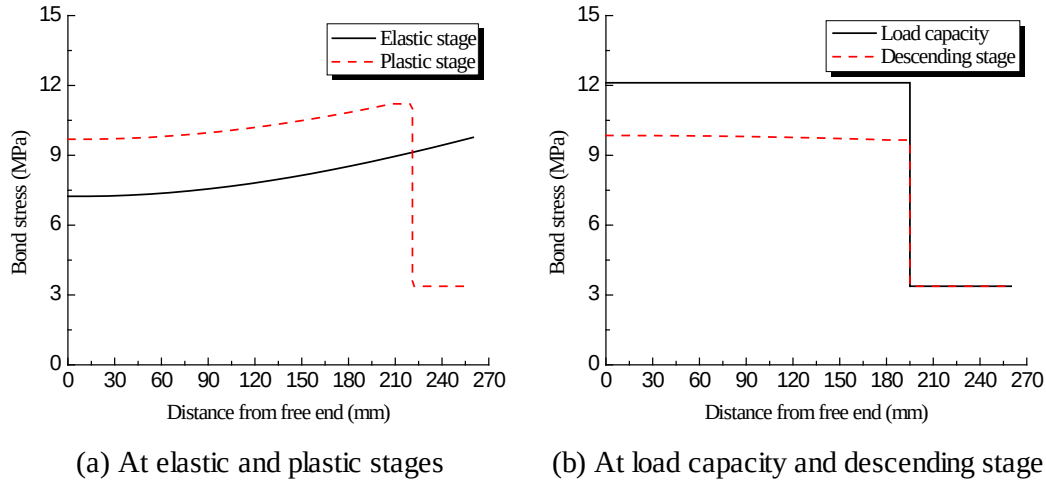


Fig. 8.9: Variations of bond stress along embedment length at different loading stages

8.3.2.4 Simplified approach

Bond stress distribution along the embedment length of reinforcement varies with applied load to the steel bar, as shown in **Fig. 8.9**. Hence, three stages, namely, elastic ascending stage, post-yield ascending stage and descending stage are classified so as to simplify the analytical approach in accordance with bond stress distribution.

(a) Elastic ascending stage

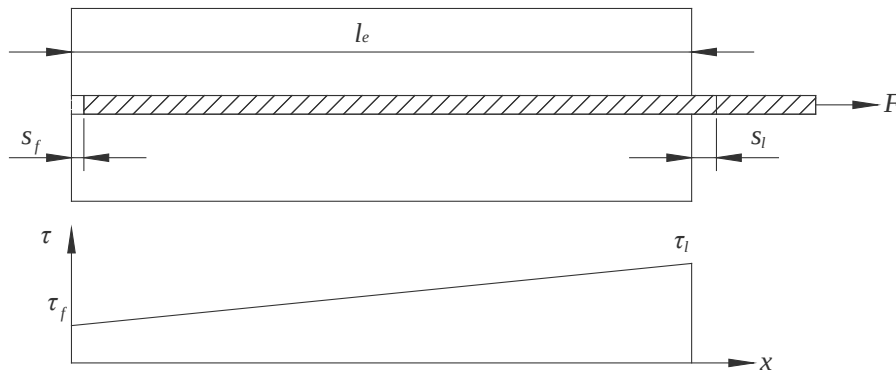


Fig. 8.10: Bond stresses and slips of an embedded reinforcing bar at elastic ascending stage

At elastic ascending stage, a linear bond stress profile is assumed along the whole embedment length of a steel bar, as shown in **Fig. 8.10**. For a given slip at the loaded end, a slip at the free end is assumed and force sustained by the steel bar can be obtained through equilibrium, as expressed in Eq. (8-6). Bond stresses at the two ends

of the steel bar can be determined through the bond-slip relationship. Correspondingly, steel strain at the loaded end can be determined through the constitutive model for the steel bar.

$$F = \frac{(\tau_f + \tau_l)l_e \pi d}{2} \quad (8-6)$$

$$s_l = s_f + \frac{2(\tau_f + 2\tau_l)l_e^2}{3E_s d} \quad (8-7)$$

where s_f and s_l are the slips at the free end and loaded end, respectively; E_s is the elastic modulus of the steel bar; d is the diameter of the bar; τ_f and τ_l are the bond stresses at the free and loaded ends, respectively; and l_e is the embedment length of the steel bar.

As bond stress varies linearly along the embedment length, distribution of steel strain is a parabolic function, with zero strain at the free end and maximum strain at the loaded end. Accordingly, the loaded end slip can be taken as a summation of the free end slip and the integration of steel strains along the embedment length (Shima et al. 1987). Thus, slip at the loaded end can be calculated from Eq. (8-7). Once compatibility of the embedded reinforcement is satisfied, namely, the calculated loaded end slip is equal to the initially assumed value, slips at the free and loaded ends are obtained.

(b) Post-yield ascending stage

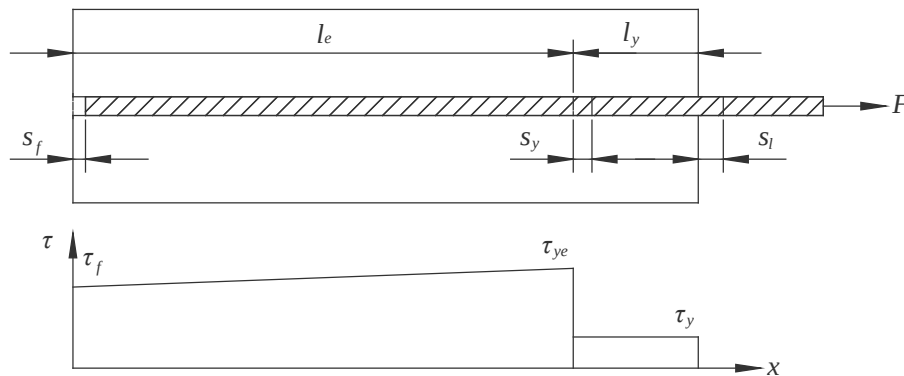


Fig. 8.11: Bond stresses and slips of an embedded reinforcing bar at plastic ascending stage

Once plasticity is initiated at the loaded end, the bond stress substantially reduces at post-yield stage of the steel bar, as depicted in **Fig. 8.9(a)**. Hence, force equilibrium and compatibility have to be appropriately modified to take account of the post-yield bond stress. In addition to the free end and loaded end slips, the length of the yielded steel segment has to be assumed at post-yield ascending stage, as shown in **Fig. 8.11**. Therefore, the force at the loaded end can be calculated from the force equilibrium of the yielded steel segment, as expressed in Eq. (8-8). Through a bilinear constitutive model of reinforcement, the steel strain at the loaded end is calculated accordingly and the slip at the yielded section can be determined from Eq. (8-9). Thereafter, the analytical bond-slip relationship is used to determine the bond stress at the yielded section. With respect to the elastic steel segment, the same procedure as that used for elastic ascending stage is followed. As force at the section where the steel bar yields is known, the calculated force from Eq. (8-10) must be equal to the yield force of the reinforcement. Moreover, slips computed from Eqs. (8-9) and (8-11) must be equal to one another so as to satisfy compatibility.

$$F = F_y + \pi d \tau_y l_y \quad (8-8)$$

$$s_y = s_l - \frac{(\varepsilon_y + \varepsilon_l) l_y}{2} \quad (8-9)$$

$$F_y = \frac{(\tau_f + \tau_{ye}) l_e \pi d}{2} \quad (8-10)$$

$$s_y = s_f + \frac{2(\tau_f + 2\tau_{ye}) l_e^2}{3E_s d} \quad (8-11)$$

Where F_y , ε_y , s_y and τ_{ye} are the force, strain, slip and bond stress at the section where the steel bar attains its yield strength, respectively; τ_f is the post-yield bond stress; l_y is the length of yielded steel segment; and ε_l is the steel strain at the loaded end.

(c) Descending stage

Beyond the maximum load that the embedded bar is able to sustain, the applied force starts decreasing with increasing loaded end slip, as shown in **Fig. 8.8**. Hence, a different analytical procedure is proposed to take account of the descending stage.

Once the descending stage commences, the strain of the embedded bar decreases with increasing loaded end slip. The maximum strain at each section should be used to determine the steel stress at descending stage. Based on stress state at post-yield ascending stage, the whole embedment length of reinforcement is divided into two segments, namely, elastic segment and debonded segment. The length of the debonded segment can be treated as the length of yielded steel segment at post-yield ascending stage, over which bond stress remains the same as the post-yield bond stress. Nonetheless, a linear bond stress distribution can still be assumed along the elastic segment, as shown in **Fig. 8.12**.

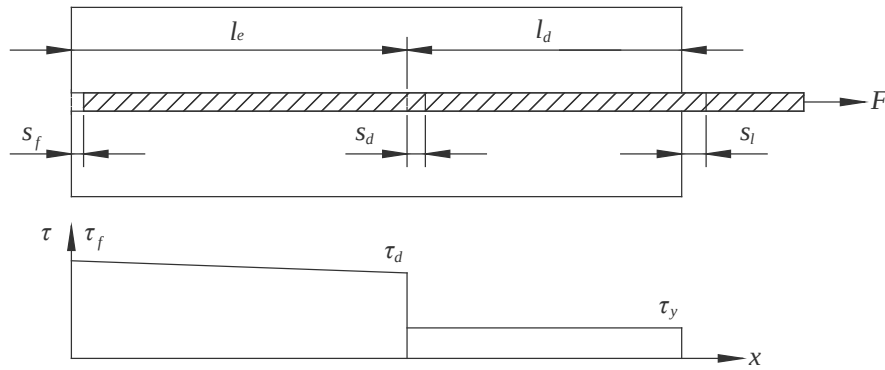


Fig. 8.12: Bond stresses and slips of an embedded reinforcing bar at descending stage

As for the elastic steel segment, the analytical procedure at elastic ascending stage is applied and slips at the free end and transition section between the elastic and debonded segments are correlated by Eq. (8-12). It is notable that the force at the transition section, which can be calculated from equilibrium in Eq. (8-13), should be smaller than the yield force of the steel bar. The procedure used for the yielded steel segment at post-yield ascending stage can be employed for the debonded segment, as expressed in Eqs. (8-14) and (8-15).

$$s_d = s_f + \frac{2(\tau_f + 2\tau_d)l_d^2}{3E_s d} \quad (8-12)$$

$$F_d = \frac{(\tau_f + \tau_d)l_e \pi d}{2} \quad (8-13)$$

$$s_l = s_d + \frac{(\varepsilon_d + \varepsilon_l)l_d}{2} \quad (8-14)$$

$$F = F_d + \pi d \tau_y l_d \quad (8-15)$$

where F_d , s_d and ε_d are the force, slip and strain at the transition section, respectively; l_d is the length of debonded segment which is the same as the yielded length of steel bar at the load capacity; and τ_d is the bond stress at the transition section.

8.3.2.5 Verification of simplified approach

Experimental results listed in **Table 8.4** are simulated through the simplified approach. Comparisons between the experimental and analytical results indicate that the simplified approach yields reasonably good predictions of the overall bond-slip behaviour of embedded steel reinforcement, as shown in **Fig. 8.8**. However, the calculated free end slip is slightly larger than that estimated by the nested iteration procedure due to the assumption of linear bond stress distribution along the elastic steel segment.

8.3.3 Non-zero strain with zero slip

The average bond stresses at elastic and post-yield stages of embedded steel reinforcement with sufficient anchorage length have been quantified and calibrated by experimental results in Section 8.3.1. Nonetheless, under axial tension loading condition, the steel stress at the centre of the embedment length may not be zero and its value increases with decreasing embedment length and increasing load acting on the reinforcement. With a tensile stress f_s at two ends of an embedded reinforcing bar, only if the embedment length is sufficient to ensure zero strain at the centre point, the average elastic bond stress is $0.043\sqrt{f_c' f_s}$. If the embedment length is insufficient, the force-slip relationship of the steel bar subjected to axial tension has to be derived in accordance with the stress state of the loaded end and mid-point of the embedment length (Yu 2012).

Fig. 8.13 shows a steel bar under axial tension when tensile stresses at the loaded end and mid-point of the embedment length are below the yield strength, namely, $f_s \leq f_y$ and $0 < f_{sc} < f_y$. Due to symmetry, the steel segment between points A and B is

extracted from the embedded reinforcing bar. For a given tensile stress f_s at section B, steel stress at section A is f_{sc} . Thus, steel segment AB is equivalent to the difference of segments CD and EF, as shown in **Fig. 8.13**. At sections C and E, both steel strains and slips are zero. In accordance with the proposed elastic bond stress under the boundary condition of zero strain with zero slip, bond stress τ_{CD} acting on segment CD is $0.043\sqrt{f'_c f_s}$. Length l_{CD} can be determined from the force equilibrium of steel segment CD, as expressed in Eq. (8-16). Accordingly, slip s_D at section D can be calculated from Eq. (8-17).

$$\frac{1}{4}\pi d^2 f_s = \pi d \tau_{CD} l_{CD} \quad (8-16)$$

$$s_D = \frac{f_s^2 d}{8E_s \tau_{CD}} \quad (8-17)$$

where d is the diameter of reinforcement; E_s is the elastic modulus of steel bars.

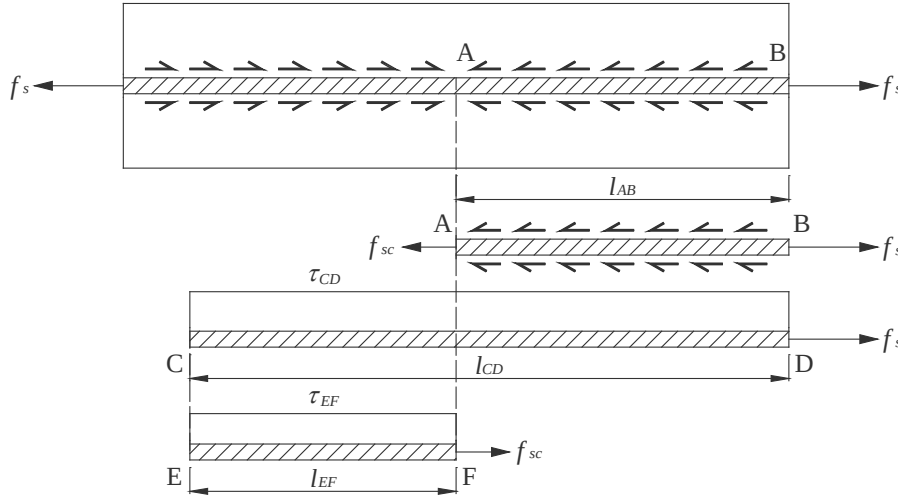


Fig. 8.13: Bond stress distribution for an elastic steel bar under axial tension

The length of segment EF is taken as the difference between l_{AB} and l_{CD} , as expressed in Eq. (8-18).

$$l_{EF} = l_{AB} - l_{CD} \quad (8-18)$$

With a tensile stress f_{sc} acting on section F, bond stress τ_{EF} along steel segment EF is $0.043\sqrt{f'_c f_{sc}}$. Based on the force equilibrium of steel segment EF, tensile stress

$$\frac{1}{4}\pi d^2 f_{sc} = \pi d \tau_{EF} l_{EF} \quad (8-19)$$

$$S_F = \frac{f_{sc}^2 d}{8E_s \tau_{EF}} \quad (8-20)$$

$$S_B = S_D - S_F \quad (8-21)$$

$$l_{GD} = \frac{(f_s - f_y)d}{4\tau_v} \quad (8-22)$$

At section G, the embedded reinforcement attains its yield strength. Based on the proposed elastic bond stress, τ_{CG} along steel segment CG can be determined as

$0.043\sqrt{f'_c f_y}$, and the length of steel segment CG can be calculated from Eq. (8-23).

For steel segment CD, tensile stress at section C is zero, and slip at section D can be computed from Eq. (8-24). Based on compatibility, the length of steel segment EF can be quantified from Eq. (8-25). Force equilibrium expressed in Eq. (8-19) can be used to determine tensile stress f_{sc} at section F. Thus, slips at sections F and B can be calculated from Eqs. (8-20) and (8-21), respectively.

$$l_{CG} = \frac{f_y d}{4\tau_{CG}} \quad (8-23)$$

$$s_D = \frac{f_y^2 d}{8\tau_{CG} E_s} + \frac{(f_s - f_y) f_y d}{4\tau_y E_s} + \frac{(f_s - f_y)^2 d}{8\tau_y E_h} \quad (8-24)$$

$$l_{EF} = l_{CG} + l_{GD} - l_{AB} \quad (8-25)$$

Furthermore, the mid-point of the embedded reinforcement may develop inelastic strain due to a short embedment length, namely, $f_s > f_y$ and $f_{sc} > f_y$, as shown in **Fig. 8.15**. At that stage, the post-yield bond stress (i.e. $0.4\sqrt{f'_c}$) is uniformed distributed along steel segment AB. Tensile stress f_{sc} at section A can be determined from the force equilibrium of segment AB, as expressed in Eq. (8-26). Slip at section B can be calculated from Eq. (8-27).

$$f_{sc} = f_s - \frac{4\tau_y l_{AB}}{d} \quad (8-26)$$

$$s_B = \frac{(f_s - f_{sc}) f_{sc} d}{4\tau_y E_s} + \frac{(f_s - f_{sc})^2 d}{8\tau_y E_h} \quad (8-27)$$

where E_h is the hardening modulus of steel reinforcement.

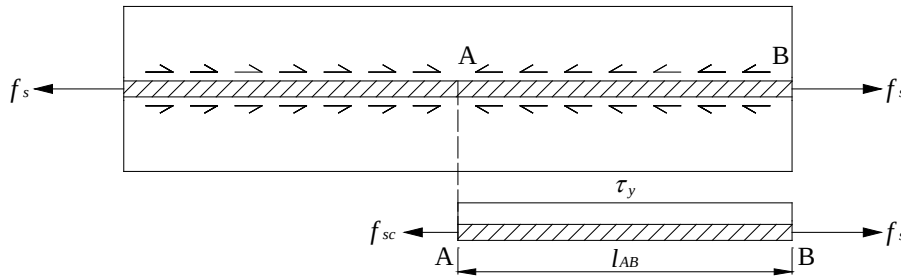


Fig. 8.15: Bond stress distribution for a yielded steel bar at mid-point of embedment length

Material properties and embedment length of T13 rebar given by Yu (2012) are used for simulations through the proposed model, as listed in **Table 8.6**. **Fig. 8.16(a)** shows the predicted force-slip curves at the elastic stage. Similar to the boundary condition of zero strain with zero slip, the proposed model predicts considerably softer force-slip response in comparison with Yu's model (Yu and Tan 2010b) and Shima's model (Shima et al. 1987). At the post-yield stage, the proposed model gives the smallest ultimate slip at the loaded end, as shown in **Fig. 8.16(b)**, due to a greater post-yield bond stress used in the model. However, limited published experimental results on axial tension tests prevent further calibration of the analytical model.

Table 8.6: Material properties and embedment length for T13 rebar (Yu 2012)

Item	Steel properties			Concrete strength (MPa)	Embedment length (mm)
	Diameter (mm)	Yield strength (MPa)	Elastic modulus (GPa)		
T13	13	494	185.9	38.2	125

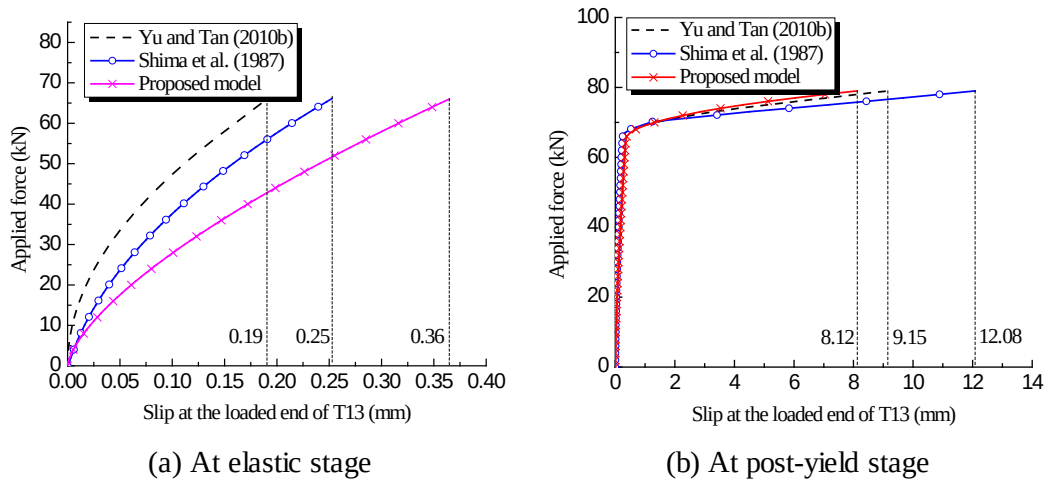


Fig. 8.16: Relationship of applied force and loaded end slip for reinforcing bar T13 under axial tension

8.4 Properties of Compressive Spring

Generally, tensile strength of conventional concrete is neglected in determining the properties of tensile spring. The force-slip relationship of bare steel bars in tension can be used to represent the tensile spring at the joint interface, as proposed by Lowes et al. (2004) and Yu and Tan (2010b). However, in the compressive spring, the contribution of compressive concrete to total compression force has to be properly

considered. In order to derive the force-slip relationship of the compressive spring, compression force sustained by concrete is related to the compressive strain of longitudinal reinforcement (Lowes et al. 2004; Yu 2012). Lowes et al. (2004) expressed the total force in the compression zone as a function of the compressive force in reinforcement and calculated the ratio of the total compression force to the reinforcement force when the extreme compression fibre attained the crushing strain of concrete. In determining the ratio, no axial compression force was considered in reinforced concrete beams. Thus, it is not suitable for column removal scenarios which are characterised by axial compression force in the bridging beam at compressive arch action (CAA) stage. Yu (2012) considered a compression force in the beam and assumed the compression force sustained by concrete to vary linearly with the compressive strain of reinforcement before the crushing strain of concrete was attained at the extreme compression fibre. Equivalent rectangular concrete stress block was employed to calculate the maximum compression force in concrete. However, at large deformation stage, the equivalent concrete stress block is not valid due to severe crushing of concrete in the compression zones of the beam. Thus, the force in the compression zone has to be re-examined.

8.4.1 Determination of compression force

Under column removal scenarios, compression force develops in the bridging beam at CAA stage and varies with increasing middle joint displacement. Therefore, in determining the total force in the compression zone, axial compression force in the beam has to be considered. Due to changes in the compression force, the centroid of compressive stress sustained by concrete also varies at CAA stage. In order to derive the force-slip relationship of compressive spring, neutral axis depths at the middle joint and end column stub have to be determined.

For simplicity, it is assumed that the neutral axis depth at the beam end is kept constant at CAA stage (Yu 2012). It is calculated when the beam end section attains its moment capacity with a compression force of $0.5N_{cr}$ acting on it, as shown in **Fig.**

8.17. N_{cr} is the axial compression force in the beam at a critical state when tensile reinforcement attains its yield strain and compressive concrete reaches its crushing

strain simultaneously. Thus, force equilibrium at the beam end is expressed in Eq. (8-28).

$$0.5N_{cr} = f'_{scr}A'_s + 0.85f'_c b \beta c - f_y A_s \quad (8-28)$$

in which f_y is the yield strength of reinforcement in tension; f'_{scr} is the compressive stress of reinforcement at the critical state; A_s and A'_s are the areas of reinforcement in the tension and compression zones, respectively; b is the width of the beam; and c is the neutral axis depth.

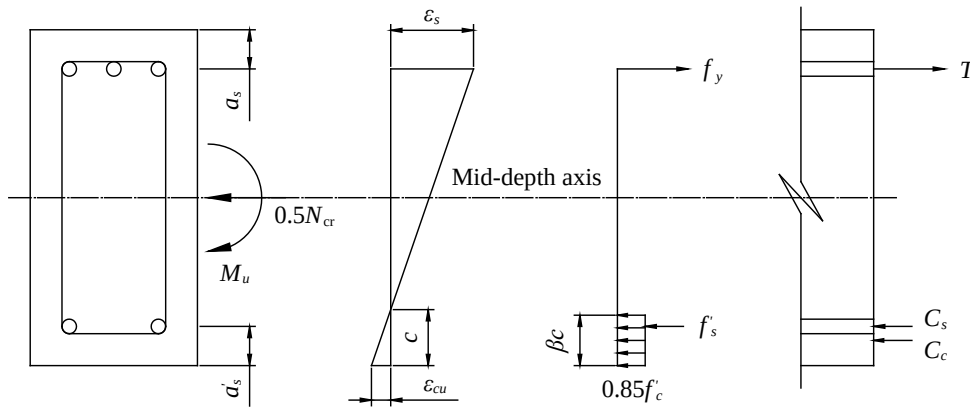


Fig. 8.17: Neutral axis depth at beam end

After determining the neutral axis depth at the beam ends, relationship between the compression force in concrete and the strain of compressive reinforcement has to be established in accordance with the plane-section assumption. Instead of the equivalent rectangular concrete stress block in the compression zone, the constitutive model for concrete proposed by Mander et al. (1988) is used to calculate the compression forces in concrete. At each strain of the compressive reinforcement, the strain profile of concrete in the compression zone is determined and the associated compressive stress is obtained through Mander's concrete stress-strain model. The compression force sustained by concrete can be calculated through integration of the compressive stress across the compression zone. Similar to the method proposed by Lowes et al. (2004), the ratio of total compression force to the force sustained by the compressive reinforcement is calculated from Eq. (8-29).

$$\gamma_c = \frac{C_c + C_s}{C_s} \quad (8-29)$$

in which C_c and C_s are the compression forces sustained by the concrete and the compressive reinforcement, respectively.

As for beam-column sub-assemblages designed against gravity loads, the top longitudinal reinforcement ratio in the beam is normally greater than the bottom reinforcement ratio. Under column removal scenarios, more tensile reinforcement is provided at the face of the end column stub in comparison with compressive reinforcement. However, due to reversal of bending moment at the middle joint, the compressive reinforcement ratio is significantly greater than the tensile reinforcement ratio at the face of the middle joint. Therefore, beam sections at the faces of the middle joint and end column stub have to be analysed separately by using the foregoing procedure. **Table 8.7** lists the material and geometric properties of beam sections given by Yu (2012).

Table 8.7: Material and geometric properties of beam sections

Section	Cross section (mm)	Concrete strength (MPa)	Tensile reinforcement		Compressive reinforcement	
			Area (mm ²)	Strength (MPa)	Area (mm ²)	Strength (MPa)
(I) Column stub face	150x250	38.1	398.2	494	265.5	494
(II) Middle joint face			265.5	494	398.2	494

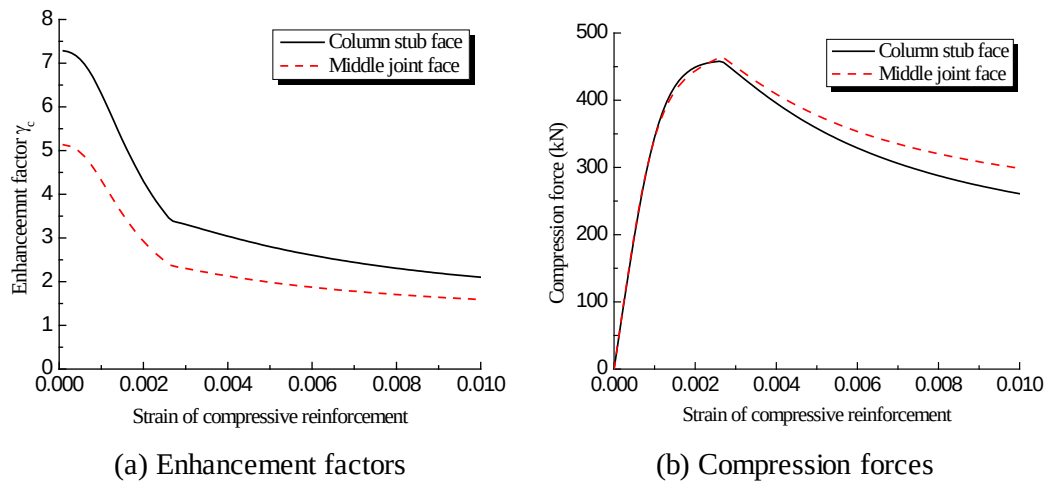


Fig. 8.18: Enhancement factors and compression forces at middle joint and end column stub

Fig. 8.18 shows the variations of enhancement factor and total compression force with the strain of compressive reinforcement. It is apparent that the enhancement factor decreases with increasing strain of reinforcement in the compression zones, as

shown in **Fig. 8.18(a)**. Besides, the enhancement factor at the face of the end column stub is substantially greater than the value at the face of the middle joint due to a lower compressive reinforcement ratio at the end column stub. However, almost the same maximum compression forces are obtained at the two faces, as shown in **Fig. 8.18(b)**. The compression force starts decreasing after attaining its maximum value due to softening of compressive concrete. With more compressive reinforcement at the face of the middle joint, total compression force decreases more slowly compared to that at the face of the end column stub.

Through the proposed method, force in the compression zone of beam sections can be quantified. It is assumed that compressive spring is located at the centroid of the compression zone. Thus, the total compression force has to be transformed to the equivalent compression force in the compressive spring so that bending moment at the beam section remains the same as the actual value. Furthermore, in order to determine the force-slip relationship of the compressive spring, the bond stress of reinforcement subjected to compression has to be determined and the slip of compressive reinforcement at each load level has to be calculated in accordance with bond-slip model.

8.4.2 Bond stress in compression

In the analytical model proposed by Eligehausen et al. (1983), it is indicated that the same bond stress can be used for reinforcement in tension and compression. However, it only holds at elastic stage. Once a steel bar yields, Poisson effect comes into effect and affects the post-yield bond stress of reinforcement in tension and compression. Therefore, the Poisson effect has to be considered in determining the bond stress of compressive reinforcement at post-yield stage.

The post-yield bond stress of a reinforcing bar with an adequate embedment length in tension can be determined as $0.41\sqrt{f'_c}$ (see Section 8.3.1), when the steel cross section contracts due to the Poisson effect, resulting in a reduction of frictional bond stress by 25% at inelastic stage. Likewise, expansion of the steel cross section in compression would increase the bond stress by 25%, as discussed by Eligehausen et al. (1983) and Viwathanatapa et al. (1979). Hence, the bond stress for yielded

reinforcing bars in compression can be estimated as $0.68\sqrt{f'_c}$, 1.67 times that of yielded reinforcement in tension. If the embedment length of a steel bar is insufficient to ensure zero slip at the free end, the post-yield bond stress of the bar in tension can be calculated as $0.68\sqrt{f'_c}$, as discussed in Section 8.3.2. The value becomes $1.14\sqrt{f'_c}$ if the steel reinforcement yields in compression.

8.4.3 Force-slip relationship of compressive spring

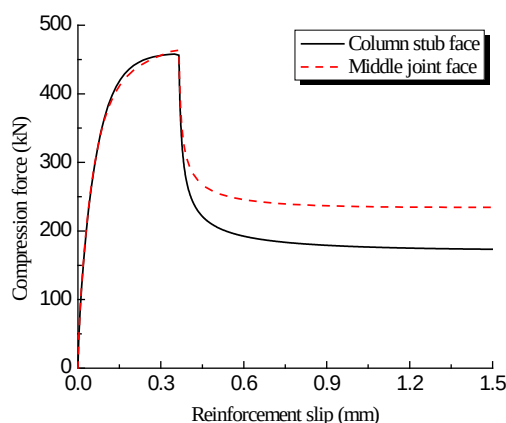


Fig. 8.19: Force-slip relationships of compressive springs

The same boundary conditions as those in tension, namely, zero strain with zero slip, zero strain with non-zero slip and non-zero strain with zero slip, are defined for embedded reinforcement subjected to compression. Force-slip relationship of reinforcement can be determined according to the proposed bond stresses in compression. The slip of reinforcement is assumed to be identical to that of compressive spring. To derive the force-slip relationship of compressive spring, the force in the compressive spring and associated slip are correlated through the strain of compressive reinforcement. **Fig. 8.19** shows the force-slip relationships of the compressive springs at the end column stub and middle joint. For each spring, compression force increases with increasing slip of the compressive reinforcement until it attains the yield strain. Following yielding of the compressive reinforcement, total compression force drops rapidly due to crushing of compressive concrete, and then levels off as a result of compression force sustained by steel reinforcement. Compared to the end column stub, a greater compressive reinforcement ratio at the

face of the middle joint enables the compressive spring to sustain a greater residual force at the post-yield stage of the compressive reinforcement.

8.5 Shear Panel Spring

In previous numerical studies, modified compression field theory was used to define the constitutive law of shear-panel component in reinforced concrete beam-column joints under cyclic loading conditions (Lowes and Altoontash 2003). Under column removal scenarios, Bao et al. (2008) defined two rotational springs at the pin nodes of the joint zone to capture distortion of joints. Similarly, Yu (2012) proposed two diagonal springs in the joint panel to represent shear distortions of beam-column joints. However, it is observed that the deformation of the middle beam-column joint is insignificant under column removal scenarios due to limited shear force in the joint. For the sake of brevity, a rigid joint panel is assumed in the model. In other words, the diagonal springs are assumed to be at linear elastic stage with infinite stiffness values when subjected to CAA and catenary action.

8.6 Validation of Joint Model

8.6.1 Parameters of springs

Properties of the tensile and compressive springs can be determined based on the proposed approaches. In the component-based joint model, the constitutive model of each spring is simplified as a trilinear curve. **Table 8.8** summarises the properties of zero-length springs at the joint interfaces of sub-assemblages MJ-B-0.88/0.59R and CMJ-B-1.19/0.59.

Bottom spring at the middle joint and top spring at the end column stub sustain tension forces under column removal scenarios. Thus, only their tensile branch is defined, as included in **Table 8.8**. MJ-B-0.88/0.59R exhibited pull-out failure of bottom reinforcement in the middle joint. Based on the boundary condition of zero strain with zero slip at the free end, the ultimate slip of the tensile spring at the middle joint interface is 42.2 mm and associated load is nearly zero. In CMJ-B-1.19/0.59, rupture of bottom bars occurred at the interface of the middle joint and the respective slips are calculated as 17.199 mm. At the face of the end column stub, all three sub-

assemblages developed rupture of beam top reinforcement. Therefore, force-slip relationships of the top reinforcement embedded in the end column stub are determined in accordance with the boundary condition of zero strain with zero slip at the free end.

Table 8.8: Parameters of springs in joint model

Specimen	At the middle joint interface*					
	k_{bb}		k_{bt}			
	Tensile branch		Tensile branch		Compressive branch	
	s_t (mm)	F_t (kN)	S_t (mm)	F_t (kN)	s_c (mm)	F_c (kN)
MJ-B-0.88/0.59R	0.800	119.65	0.454	186.00	0.300	359.00
	6.400	123.25	3.431	207.00	0.462	400.00
	42.200	1.00	12.600	228.00	25.000	317.00
CMJ-B-1.19/0.59	0.392	145.74	0.495	303.25	0.342	555.00
	4.731	165.52	3.643	332.50	0.503	594.00
	17.199	185.29	13.486	364.42	25.000	417.00
Specimen	At the end column stub interface*					
	k_{bt}		k_{bb}			
	Tensile branch		Tensile branch		Compressive branch	
	s_t (mm)	F_t (kN)	s_t (mm)	F_t (kN)	s_c (mm)	F_c (kN)
MJ-B-0.88/0.59R	0.460	187.55	0.460	125.03	0.300	425.00
	3.365	206.86	3.365	137.91	0.462	448.00
	11.629	226.18	11.629	150.78	25.000	290.00
CMJ-B-1.19/0.59	0.495	303.25	0.392	145.74	0.237	524.00
	3.822	333.46	4.731	165.52	0.379	545
	13.285	363.68	17.199	185.29	25.000	297

*: k_{bt} and k_{bb} are the top and bottom springs at the face of the middle joint and end column stub.

Top spring at the middle joint and bottom spring at the end column stub are in compression at the CAA stage and are shifted to tension with increasing vertical displacement at the catenary action stage. Therefore, both tensile and compressive branches are defined, as shown in **Table 8.8**. In the middle joint, the boundary condition of non-zero strain with zero slip is used to determine the force-slip relationship of the top spring. However, at the end column stub, the embedment length of bottom reinforcement is adequate to ensure zero strain and zero slip at the free end. Thus, properties of the bottom spring are determined in accordance with the boundary condition of zero strain with zero slip. Once the compressive spring at CAA

stage is shifted to tension at catenary action stage, the same boundary condition are used for the tensile branch, whereas the average bond stress has to be changed accordingly.

8.6.2 Comparisons with experimental results

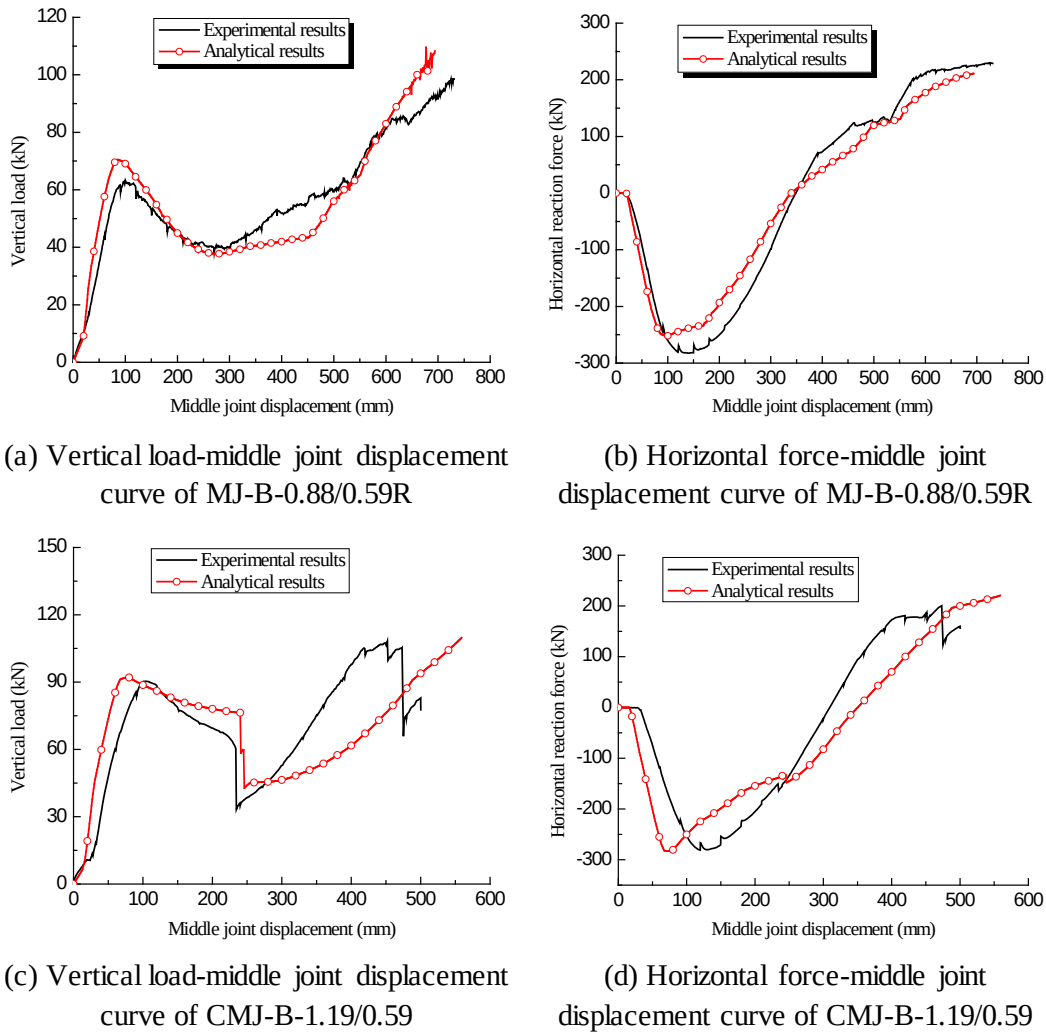


Fig. 8.20: Comparisons of experimental and numerical results of precast concrete beam-column sub-assemblages

The component model for precast concrete joints is validated by the experimental results of precast concrete beam-column sub-assemblages. **Fig. 8.20** shows the comparisons between the experimental and numerical results. It indicates that the model is able to estimate the vertical loads and horizontal forces of precast concrete sub-assemblages MJ-B-0.88/0.59R and CMJ-B-1.19/0.59 with reasonable accuracy. Pull-out failure and rupture of bottom reinforcement in the middle joint are

successfully captured in MJ-B-0.88/0.59R and CMJ-B-1.19/0.59, respectively. Besides, behaviour of reinforced concrete beam-column sub-assemblages tested by Yu (2012) is also simulated under column removal scenarios. Reasonably good agreement is also achieved between the experimental and numerical results, as shown in Fig. 8.21.

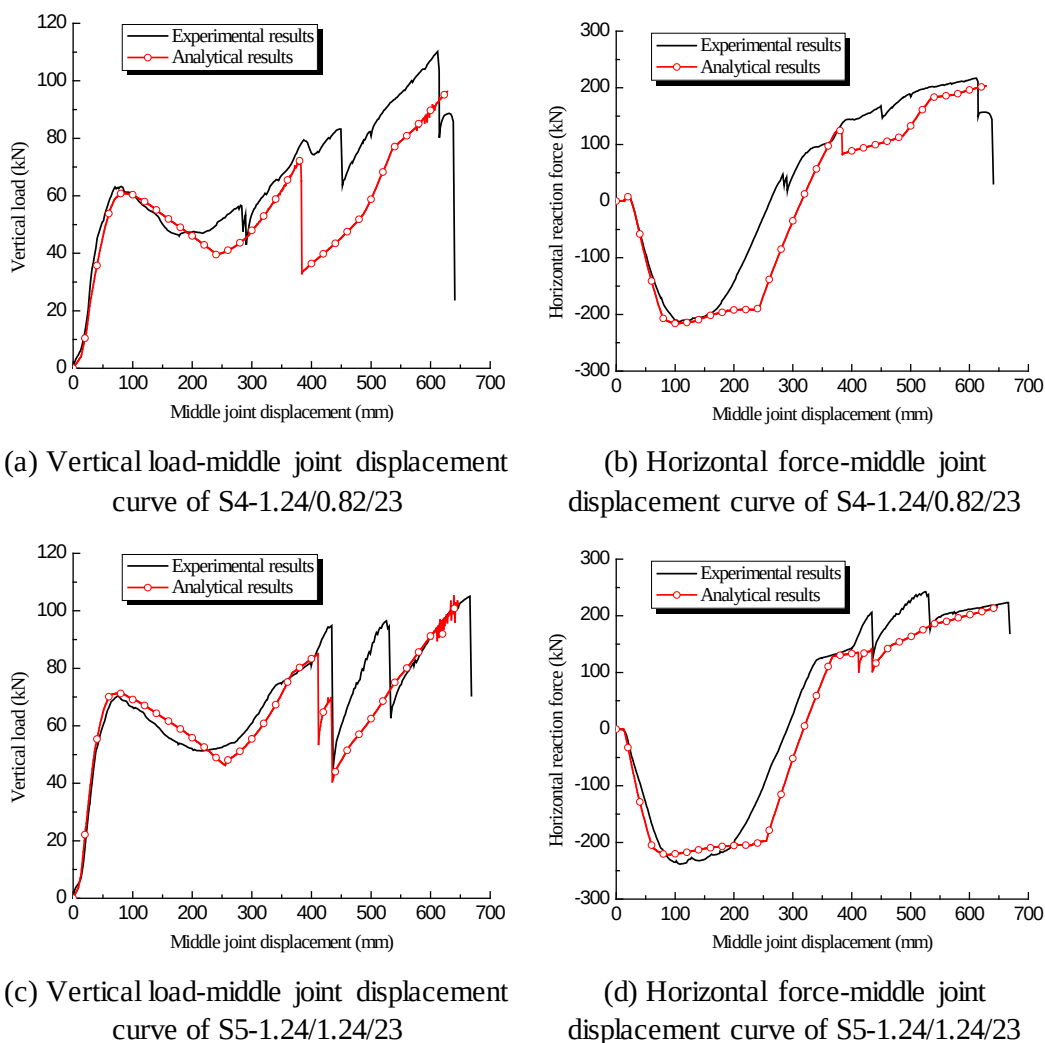


Fig. 8.21: Comparisons of experimental and numerical results of reinforced concrete sub-assemblages

8.7 Discussions

The component model for beam-column joints cannot be used for short beams featuring shear failure, as linear elastic shear springs are assumed at the joint interface. Besides, in deriving the force-slip relationship of tensile springs in the joint model, different bond stresses are utilised for steel reinforcing bars with different embedment

lengths. If the embedment length of reinforcement is insufficient to ensure zero slip at the free end, bond stresses at the elastic and post-yield stages are significantly greater than that of steel bars with adequately long length and zero slip at the free end. This phenomenon is also reported by FIB (2000) for short reinforcement embedded in concrete. However, limited emphasis is placed on the fundamental mechanism for bond-slip behaviour of short and long steel bars. Hence, experimental tests on reinforcement with various embedment lengths are needed so that direct comparison can be made between bond stresses at the elastic and post-yield stages of reinforcement.

Under cyclic loading conditions, no axial force develops in the reinforced concrete beam. The force in the compression zone of the beam can be simply assumed to be equal to the tension force sustained by longitudinal reinforcement. However, under column removal scenarios, the bridging beam over the damaged column develops axial compression force when vertical displacement is less than one beam depth. In order to achieve reasonably accurate estimation of the joint behaviour under column removal scenarios, the compression force in the beam has to be quantified in determining the properties of compressive spring (Yu 2012). The magnitude of the compression force depends on the span-depth ratio and boundary condition of the bridging beam, which increases the difficulty in deriving the force-slip relationship of the compressive spring. In this chapter, a constant axial compression force is considered in calculating the neutral axis depth at beam ends and it is reasonable for beam-column sub-assemblages with moderate span-depth ratio and rigid boundary condition. However, the value may not be suitable for sub-assemblages with weaker restraints and smaller span-depth ratios. Therefore, further experimental and numerical investigations are necessary to establish the relationships of axial compression force, span-depth ratio and boundary condition under column removal scenarios.

8.8 Conclusions

In this chapter, a component-based joint model is developed for precast concrete beam-column joints under column removal scenarios. In the model, interactions between the beam and joint are represented by tensile, compressive and shear springs.

The tensile spring can be defined by the force-slip relationship of embedded reinforcement. In accordance with the embedment length and loading conditions, boundary conditions of embedded reinforcement, namely, zero strain with zero slip, zero strain with non-zero slip and non-zero strain with zero slip, are considered in deriving the properties of the spring. Macro-models are employed to calculate the slip of reinforcement, in which average bond stresses of reinforcement at the elastic and post-yield stages are re-evaluated based on experimental results. Besides fracture of reinforcement with an adequate embedment length, pull-out failure of short embedded reinforcement can also be predicted in the model. As for the compressive spring, its slip is represented by the slip of compressive reinforcement, whereas total compression force in the spring is contributed by concrete and the compressive reinforcement. In calculating the compression force sustained by concrete, Mander's concrete model is used instead of equivalent concrete stress block. Therefore, descending branch of the compression force is captured. The shear spring is assumed to be at linear elastic stage with infinite stiffness. Finally, the joint model is calibrated by experimental results of precast and reinforced concrete beam-column sub-assemblages under column removal scenarios. Comparisons between the experimental and numerical results indicate that the joint model is able to estimate the vertical load capacity and horizontal reaction force with reasonable accuracy.

CHAPTER 9 CONCLUSIONS AND FUTURE WORK

9.1 Conclusions

The research programme investigated the behaviour of precast concrete beam-column sub-assemblages and frames subject to middle column removal scenarios. Besides, engineered cementitious composites (ECC), a strain-hardening fibre-reinforcement concrete in tension, was utilised in cast-in-situ concrete topping and beam-column joints to study the potential enhancement of ductile concrete to structural resistance and deformation capacity of sub-assemblages. Thereafter, an analytical model was proposed for estimating the compressive arch action (CAA) of sub-assemblages and a component-based joint model was developed for precast concrete beam-column joints under column removal scenarios.

Experimental tests on precast concrete beam-column sub-assemblages

Precast concrete structures feature weak beam-column joints and discontinuity of longitudinal reinforcement in the beam. Under column removal scenarios, the ability of precast concrete joints to develop alternate load paths remains questionable due to a lack of integrity and robustness. Thus, an experimental programme was conducted to investigate the resistance and deformation capacity of precast concrete beam-column sub-assemblages subject to column removal scenarios.

With fairly rigid boundary conditions, precast concrete sub-assemblages developed significant CAA and subsequent catenary action under middle column removal scenarios, due to the presence of continuous top reinforcement in the cast-in-situ structural topping. A higher top reinforcement ratio in the beam favoured the development of catenary action at large deformation stage. Similar to reinforced concrete specimens, the deformation capacity of precast concrete sub-assemblages was considerably greater than that specified in UFC 4-023-03. In addition to plastic hinge rotations at the beam ends, flexural deformations of the bridging beam, in particular, formation of a partial hinge at the curtailment point of top reinforcement, contributed a significant portion to total deformation of sub-assemblages.

At CAA stage, flexural cracks were mainly concentrated on the tension side of the beam and bottom reinforcement in the middle joint exhibited pull-out failure. Severe crushing of concrete occurred in the compression zones of the beam. Horizontal cracking was observed across the interface between precast beam units and cast-in-situ concrete topping at CAA stage, as compression force in the beam increased the horizontal shear stress acting on the concrete interface. Following the onset of catenary action, full-depth tension cracks were generated along the beam length, with nearly equal spacing. Discontinuity of top reinforcement enabled the development of a partial hinge at the curtailment point of beam top reinforcement. Final failure was caused by rupture of top longitudinal reinforcement near the end column stub.

In the experimental programme on beam-column sub-assemblages, relatively rigid boundary conditions were provided for the bridging beam by enlarged end column stubs. Accordingly, the resistance of sub-assemblages was significantly overestimated in comparison with those with realistic boundary conditions. Moreover, development of CAA and subsequent catenary action in the bridging beam may induce shear failure to the joint and flexural failure to the side column. Therefore, experimental tests were conducted on precast concrete frames with slender side columns to evaluate the effect of boundary conditions on the behaviour of beam-column joints.

Effect of ECC on structural resistances of sub-assemblages

Experimental results of precast concrete beam-column sub-assemblages indicate that the embedment length of bottom steel reinforcement in the middle joint has to be increased to prevent pull-out failure when subjected to sagging moment. An alternative is to utilise ECC in the joint zone which increases the bond strength between reinforcement and surrounding concrete and considerably reduces the required embedment length. Besides, the tensile strength of ECC can be considered in the design due to its strain-hardening behaviour and ultra-high strain capacity in tension. However, potential applications of ECC to progressive collapse scenarios have not been explored yet. Therefore, experimental tests were conducted on precast ECC beam-column sub-assemblages under column removal scenarios.

Under quasi-static loading conditions, sub-assembly EMJ-B-1.19/0.59, with ECC in structural topping and beam-column joints, was capable of developing nearly the same CAA and catenary action as precast concrete specimen CMJ-B-1.19/0.59. Its catenary action capacity was 21.1% greater than the CAA capacity, indicating the effective enhancement of catenary action to structural resistance. By reducing the top reinforcement ratio in the beam, limited catenary action was obtained as a result of premature fracture of top reinforcement near the end column stub. The fracture of reinforcement might result from a greater bond strength between steel reinforcement and ECC.

In terms of the crack pattern, multiple hairline cracks developed in the ECC topping at the initial stage. Steel reinforcement and ECC sustained tensile stresses in a compatible manner. At large deformation stage, formation of a major crack near the end column stub localised the action of sub-assemblages in a limited region, which eventually expedited fracture of top reinforcement at the crack. By reducing the reinforcement ratio in the structural topping, more severe localisation of flexural deformations was recorded at the end column stub. Therefore, compared to conventional concrete, ECC significantly increased the demand on the deformation capacity of plastic hinges near the end column stub under progressive collapse scenarios.

Resistance and deformation capacity of precast concrete frames

Precast concrete frames exhibited different behaviour from beam-column sub-assemblages. Due to insufficient horizontal restraints, limited compression force developed in the bridging beam. The enhancement of CAA to flexural action was considerably lower than that in the beam-column sub-assemblages. When vertical displacement was larger than one beam depth, frames IF-B-0.88-0.59 and EF-B-0.88-0.59 exhibited premature rupture of top reinforcement as a result of the reduced neutral axis depth at the column face. Only interior frame IF-L-0.88-0.59 was able to mobilise significant catenary action at large deformation stage, as lap-spliced bottom reinforcement in the beam sustained tensile stress and postponed rupture of top bars. In exterior frame EF-L-0.88-0.59, horizontal tension force at catenary action stage

resulted in flexural failure of the side column, which hindered the full development of catenary action.

By increasing the top reinforcement ratio in the beam of exterior frames, severe diagonal shear cracking developed in the side beam-column joint and propagated into the column, when compression force in the beam attained the maximum value at CAA stage. Force equilibrium of the side column indicates that at CAA stage shear force in the joint was increased by the horizontal compression force in the beam. At catenary action stage, the shear force in the joint decreased with increasing middle joint displacement. Therefore, in the design of precast concrete beam-column joints against shear failure under column removal scenarios, compression force in the beam has to be considered at CAA stage.

Further experimental studies on exterior frames demonstrate that shear failure of the side joint was prevented by enlarging the cross section of side columns. CAA and subsequent catenary action developed in the bridging beam when adequate horizontal restraints were provided by the side columns. Eventually, flexural failure of the side columns occurred due to substantial horizontal tension force acting on the columns. To protect the side columns from flexural failure under column removal scenarios, the magnitude of horizontal tension force in the beam has to be quantified and considered in the flexural design of the side columns.

Analytical model for CAA of beam-column sub-assemblages

To investigate the potential enhancement of ECC to structural resistance, the analytical model proposed by Park and Gamble (2000) and later modified by Yu and Tan (2013a) was used to estimate the CAA capacity of sub-assemblages. In the model, a new method was proposed to determine the strain of steel reinforcement and concrete fibres, so that tensile strength and strain capacity of ECC could be considered. Constitutive model for concrete proposed by Mander et al. (1988) was employed instead of equivalent rectangular concrete stress block. Comparisons with experimental results indicate that the model is able to predict the CAA capacity and horizontal compression force with reasonably good accuracy.

A series of parametric studies was conducted to study the effect of concrete models, tensile strength and strain capacity of ECC, stiffness of horizontal restraint and reinforcement ratios on the CAA of sub-assemblages. Analytical results indicate that application of ECC in precast beam-column sub-assemblages enhances the CAA capacity to a limited extent. Meanwhile, the maximum horizontal compression force is reduced. A stiffer horizontal restraint provides a greater CAA capacity, but the ductility of sub-assemblages is reduced. In accordance with the energy-balance method, pseudo-static resistance of sub-assemblages was also calculated. By increasing the stiffness of horizontal restraint, the pseudo-static resistance of sub-assemblages does not increase significantly, whereas the associated vertical displacement of middle joint is substantially reduced. Furthermore, more longitudinal reinforcement has to be provided in the compression zone in order to effectively enhance the pseudo-static resistance of sub-assemblages under column removal scenarios.

Several limitations exist in the analytical model. Due to the assumption of linear strain profile along the beam length, strains of tensile reinforcement are substantially underestimated at the beam ends. Correspondingly, fracture of steel reinforcement cannot be successfully predicted in the model. Thus, component-based models need to be developed for precast concrete beam-column joints under column removal scenarios.

Component-based joint model for precast concrete beam-column joints

A component-based joint model was constructed for precast concrete beam-column joints to investigate the catenary action capacity of beam-column sub-assemblages at large deformation stage. Three zero-length springs were modelled at the interface of beam-column joint to transfer tension, compression and shear forces between the joint and the beam. In deriving the force-slip relationship of tensile spring, a new method was proposed to determine the bond stress at the elastic and post-yield stages and pull-out failure of short embedded reinforcement was considered. The constitutive model for concrete was used in the compression zone of the beam instead of rectangular concrete stress block to quantify the total force sustained by compressive spring. Shear spring was assumed to be at linear elastic stage with

infinite stiffness. The joint model was calibrated against experimental results of precast and reinforced concrete beam-column sub-assemblages. Comparisons between analytical and experimental results suggest that the joint model is capable of estimating the catenary action of sub-assemblages with reasonable accuracy. However, under column removal scenarios, development of CAA in the beam requires predetermination of compression force in the bridging beam. Further experimental and analytical studies are needed to establish the relationship of compression force and boundary condition.

9.2 Future Works

By using ECC in the cast-in-situ structural topping and beam-column joints, the deformation capacity of beam-column sub-assemblages was substantially reduced compared to concrete specimens. Two possible reasons have been identified, namely, greater bond stress between reinforcement and ECC in the joint and tension-stiffening behaviour of the bridging beam at large deformation stage. It is not possible to figure out the bond stress in the experimental tests on beam-column sub-assemblages, even though strain gauges were mounted on the longitudinal reinforcement embedded in the joint. Therefore, further pull-out tests on steel reinforcement embedded in ECC joints have to be conducted to assess the bond-slip behaviour of steel bars. Uniaxial tension tests are also necessary to study the tension-stiffening behaviour of reinforced ECC beams.

In the experimental programme on exterior precast concrete frames, severe shear failure of the side beam-column joint indicates that horizontal compression force in the beam increased the shear force in the side joint at CAA stage. At catenary action stage, horizontal tension force resulted in flexural failure of the side column. Thus, in the design of side columns against progressive collapse, the horizontal compression and tension forces developed in the bridging beam have to be considered at the CAA and catenary action stages. However, the horizontal force varies with boundary conditions, geometries of the beam and reinforcement ratios. Therefore, it is necessary to estimate the magnitude of the horizontal force in various circumstances through experimental and numerical studies.

Present study only focuses on the behaviour of planar precast concrete specimens under column removal scenarios. In precast concrete structures, lateral beams connected to the removed column may develop CAA or catenary action as effective alternate load paths to mitigate progressive collapse, depending on the locations of column removal. Moreover, reinforced concrete slabs may redistribute vertical loads by means of membrane action if sufficient horizontal restraints can be provided by adjacent structural members. Thus, three-dimensional effect of lateral beams and reinforced concrete slabs has to be incorporated in future study.

In addition to the quasi-static resistance, dynamic resistance of precast concrete structures needs to be investigated experimentally to determine the dynamic increase factor at various levels of vertical loads. In the tests, the influence of CAA on the resistance and ductility of bridging beams has to be studied when horizontal restraints with different stiffnesses are provided at the ends. Meanwhile, the calculated pseudo-static resistance from the proposed analytical model and the energy balance method can be verified by experimental results under dynamic column removal scenarios.

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APPENDIX A QUANTIFICATION OF BOUNDARY CONDITIONS

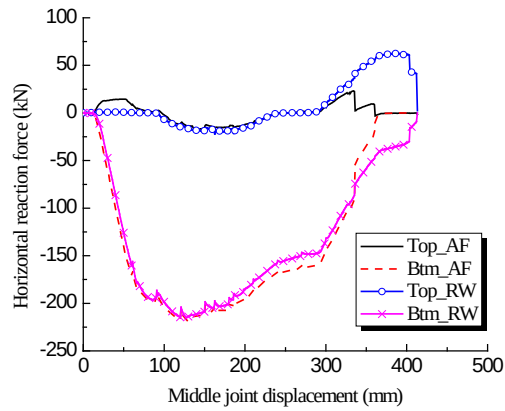
In the experimental tests, horizontal reaction forces on precast concrete sub-assemblages and frames were calculated by summing up the forces in the top and bottom horizontal load cells connected to each end column. The forces represent the average value of horizontal forces acting on the two end column stubs. To quantify the boundary conditions of precast concrete specimens, reaction forces in each horizontal load cell are presented in this chapter. Besides, corresponding horizontal displacements were also monitored through linear variable differential transducers (LVDTs), as shown in **Fig. 3.5** and **Fig. 5.4**. Thus, stiffness of horizontal restraints and connection gaps can be quantified by correlating the reaction force to the displacement. Meanwhile, connection gaps between the end column stub and the horizontal restraint can also be quantified from the load-displacement curve.

A.1 Precast Concrete Beam-Column Sub-Assemblages

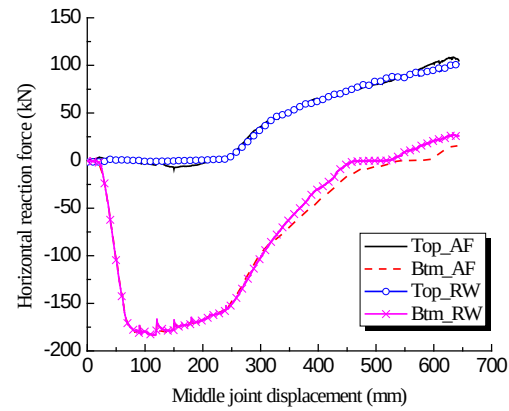
A.1.1 Horizontal reaction forces

On each precast concrete beam-column sub-assemblage, four horizontal forces were measured through load cells embedded in horizontal restraints, as shown in **Fig. 3.3**. “Top_AF” and “Btm_AF” represent respective top and bottom load cells near the A-frame, and “Top_RW” and “Btm_RW” are the load cells near the reaction wall. **Fig. A.1** shows the reaction forces in the horizontal load cells. It is apparent that horizontal compression forces were mainly transferred to the bottom restraints near the A-frame and reaction wall at the compressive arch action (CAA) stage, whereas forces in the top restraints were very limited. However, at the catenary action stage, top load cells contributed a significant portion to total horizontal tension force. Eventually, sequential fracture of beam top reinforcement near the end column stubs led to sudden drops of tension forces in the top restraints, as shown in **Fig. A.1(a)**. Thus, the bottom restraints played an important role in developing CAA in the bridging beams, but the top restraints was more critical at the catenary action stage. Besides, limited

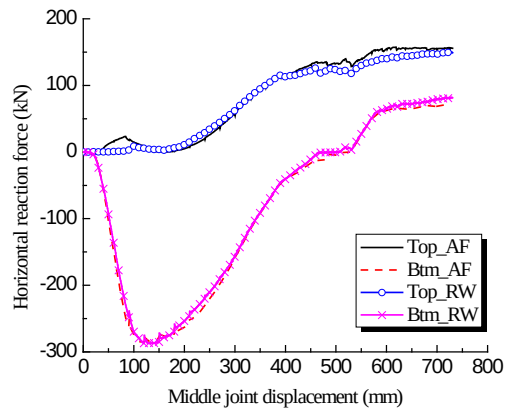
differences exist between horizontal reaction forces at the two end column stubs prior to fracture of beam top reinforcement.



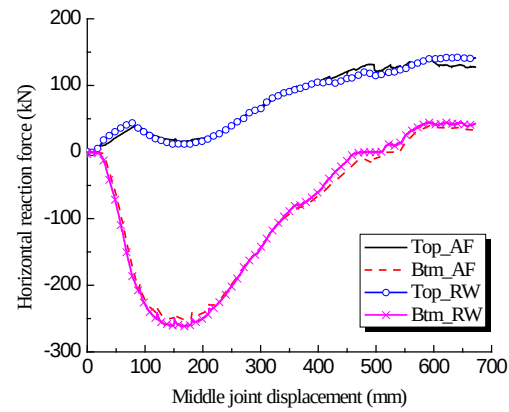
(a) MJ-B-0.52/0.35S



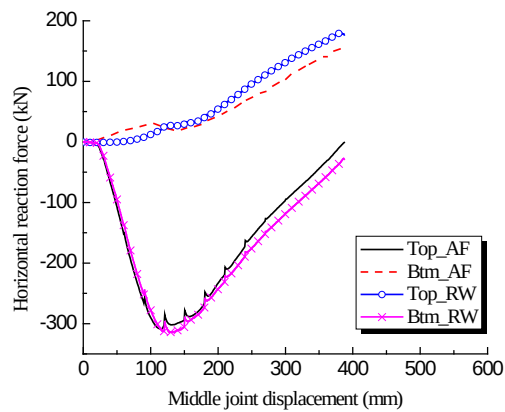
(b) MJ-L-0.52/0.35S



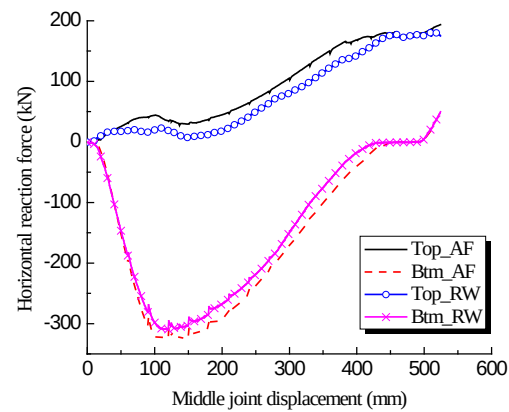
(c) MJ-B-0.88/0.59R



(d) MJ-L-0.88/0.59R



(e) MJ-B-1.19/0.59R



(f) MJ-L-1.19/0.59R

Fig. A.1 Horizontal reaction forces of precast concrete beam-column sub-assemblages

A.1.2 Stiffness of horizontal restraints

To determine the stiffness of horizontal restraints connected to the end column stubs, relationships of horizontal reaction force and corresponding displacement are shown in **Fig. A.2** to **Fig. A.7**. It is found that horizontal reaction forces were nearly zero when horizontal displacements were small (see **Figs. A.2(a and b)**), as a result of connection gaps between the end column stubs and horizontal restraints. Similar results are also obtained for other sub-assemblages, as shown in **Fig. A.3** to **Fig. A.7**. Thus, connection gaps have to be quantified for beam-column sub-assemblages.

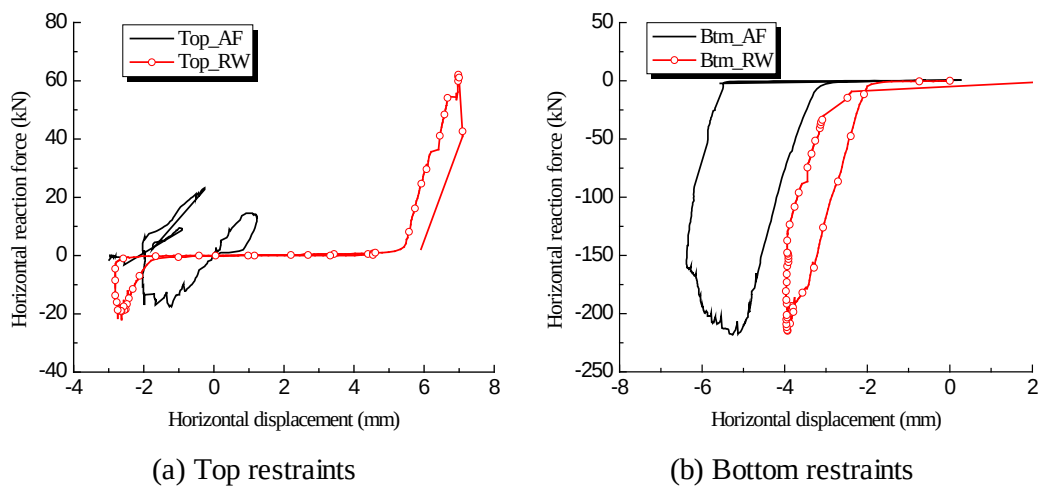


Fig. A.2 Horizontal force-displacement relationships of MJ-B-0.52/0.35S

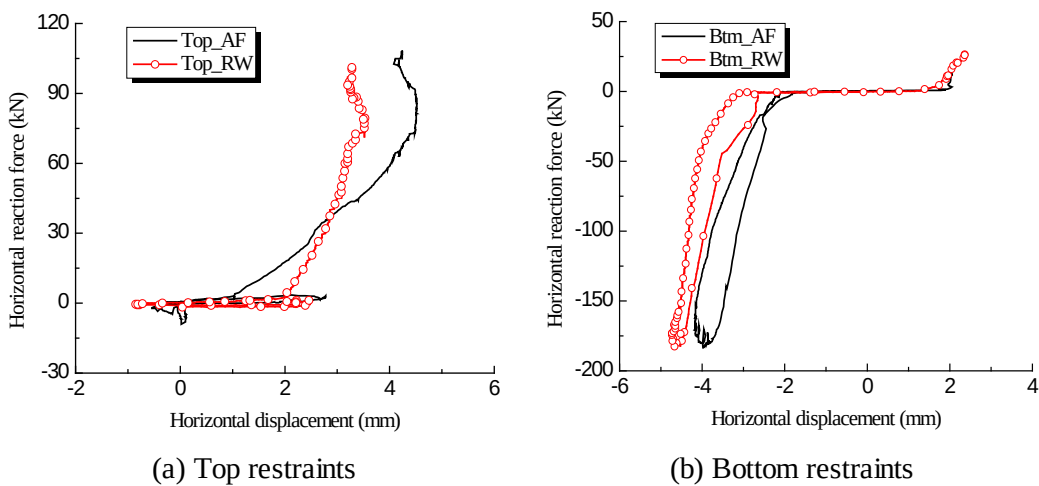
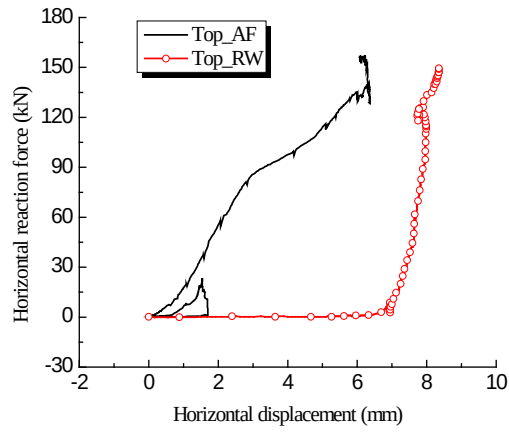
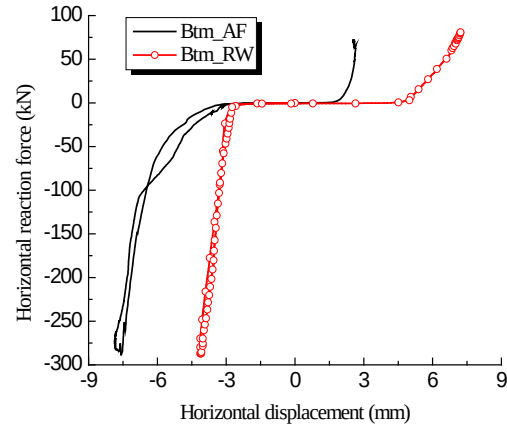


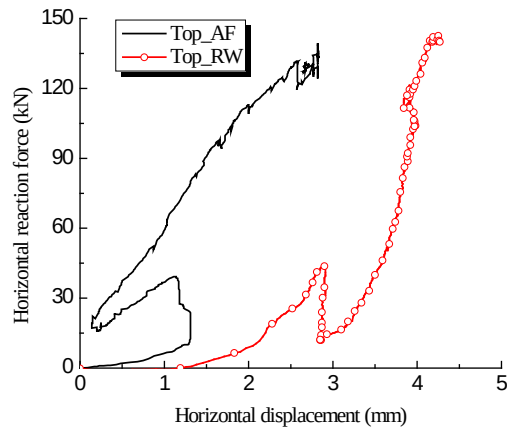
Fig. A.3 Horizontal force-displacement relationships of MJ-L-0.52/0.35S



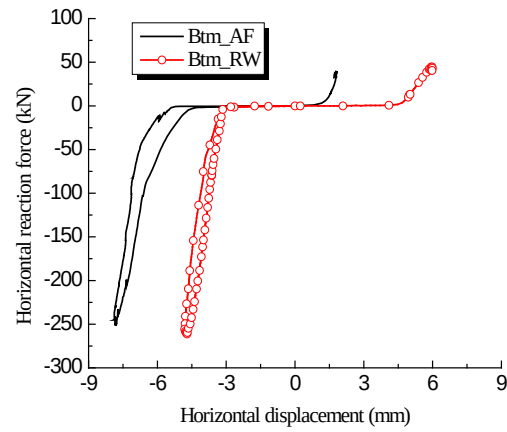
(a) Top restraints



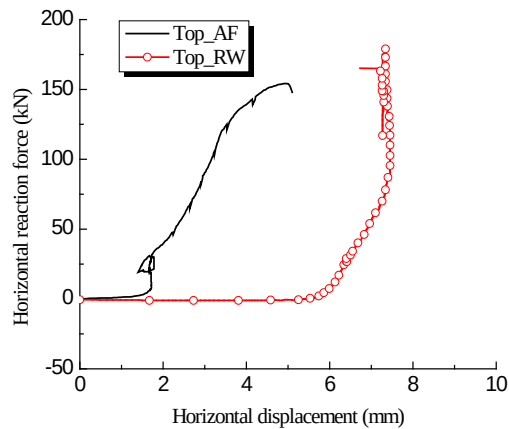
(b) Bottom restraints

Fig. A.4 Horizontal force-displacement relationships of MJ-B-0.88/0.59R

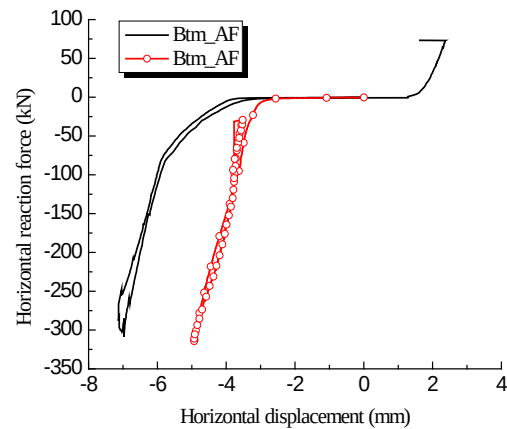
(a) Top restraints



(b) Bottom restraints

Fig. A.5 Horizontal force-displacement relationships of MJ-L-0.88/0.59R

(a) Top restraints



(b) Bottom restraints

Fig. A.6 Horizontal force-displacement relationships of MJ-B-1.19/0.59R

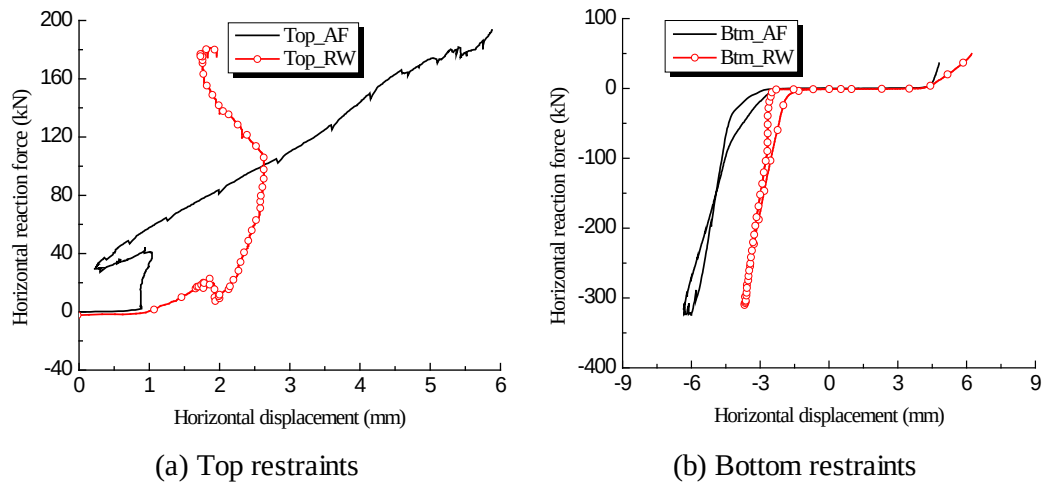


Fig. A.7 Horizontal force-displacement relationships of MJ-L-1.19/0.59R

Based on the horizontal force-displacement curves, stiffness of horizontal restraints and connection gaps between the end column stubs and horizontal restraints can be calculated through linear regression. **Table A.1** summarises the boundary conditions of precast concrete beam-column sub-assemblages. Only tension stiffnesses and associated connection gaps can be determined for top restraints Top_AF and Top_RW, as the restraints were mainly in tension under column removal scenarios. However, both tension and compression stiffnesses of bottom restraints (i.e. Btm_AF and Btm_RW) are quantified for most of the sub-assemblages. It is notable that the connections gaps are the maximum values in the top and bottom restraints and may not be attained simultaneously during testing.

Table A.1 Horizontal stiffness of precast concrete beam-column sub-assemblages

Specimen	Horizontal restraint	Tension stiffness (N/mm)	Compression stiffness (N/mm)	Tension gap (mm)	Compression gap (mm)
MJ-B-0.52/0.35S	Top_AF	10485	--	0.8	--
	Btm_AF	--	124907	--	-3.1
	Top_RW	36101	--	4.5	--
	Btm_RW	--	107122	--	-1.9
MJ-L-0.52/0.35S	Top_AF	20420	--	2.8	--
	Btm_AF	34473	110037	1.9	-2.2
	Top_RW	54160	--	2.5	--
	Btm_RW	34958	122812	1.6	-3.1
	Top_AF	23685	--	1.7	--

MJ-B- 0.88/0.59R	Btm_AF	100401	165251	2.1	-5.9
	Top_RW	100198	--	7.0	--
	Btm_RW	34182	216810	5.0	-2.9
MJ-L- 0.88/0.59R	Top_AF	42458	--	1.3	--
	Btm_AF	73505	118501	1.3	-5.7
	Top_RW	120697	--	1.9	--
	Btm_RW	30676	156666	4.5	-3.1
MJ-B- 1.19/0.59R	Top_AF	47375	--	1.7	--
	Btm_AF	85565	194781	1.6	-5.4
	Top_RW	47821	--	5.4	--
	Btm_RW	--	167576	--	-2.9
MJ-L- 1.19/0.59R	Top_AF	28227	--	0.9	--
	Btm_AF	90280	174168	4.4	-4.1
	Top_RW	137660	--	1.0	--
	Btm_RW	21887	170410	4.2	-1.9

A.2 Precast Beam-Column Sub-Assemblages with ECC

A.2.1 Horizontal reaction forces

Similar to precast concrete beam-column sub-assemblages, reaction forces in each horizontal load cell was measured for sub-assemblages with cast-in-situ ECC topping and beam-column joint, as shown in **Fig. A.8**. Special attention has to be paid on sub-assemblage MJ-L-0.88/0.88, in which compression forces in the bottom restraints (i.e. Btm_AF and Btm_RW) developed much later than tension force in the top restraints, as shown in **Fig. A.8(f)**. Besides, the maximum compression forces in the bottom load cells were substantially smaller than other sub-assemblages. In the top restraints, tension forces were considerably greater at the CAA stage compared to other sub-assemblages. It was due to larger connection gaps between the end column stubs and bottom horizontal restraints.

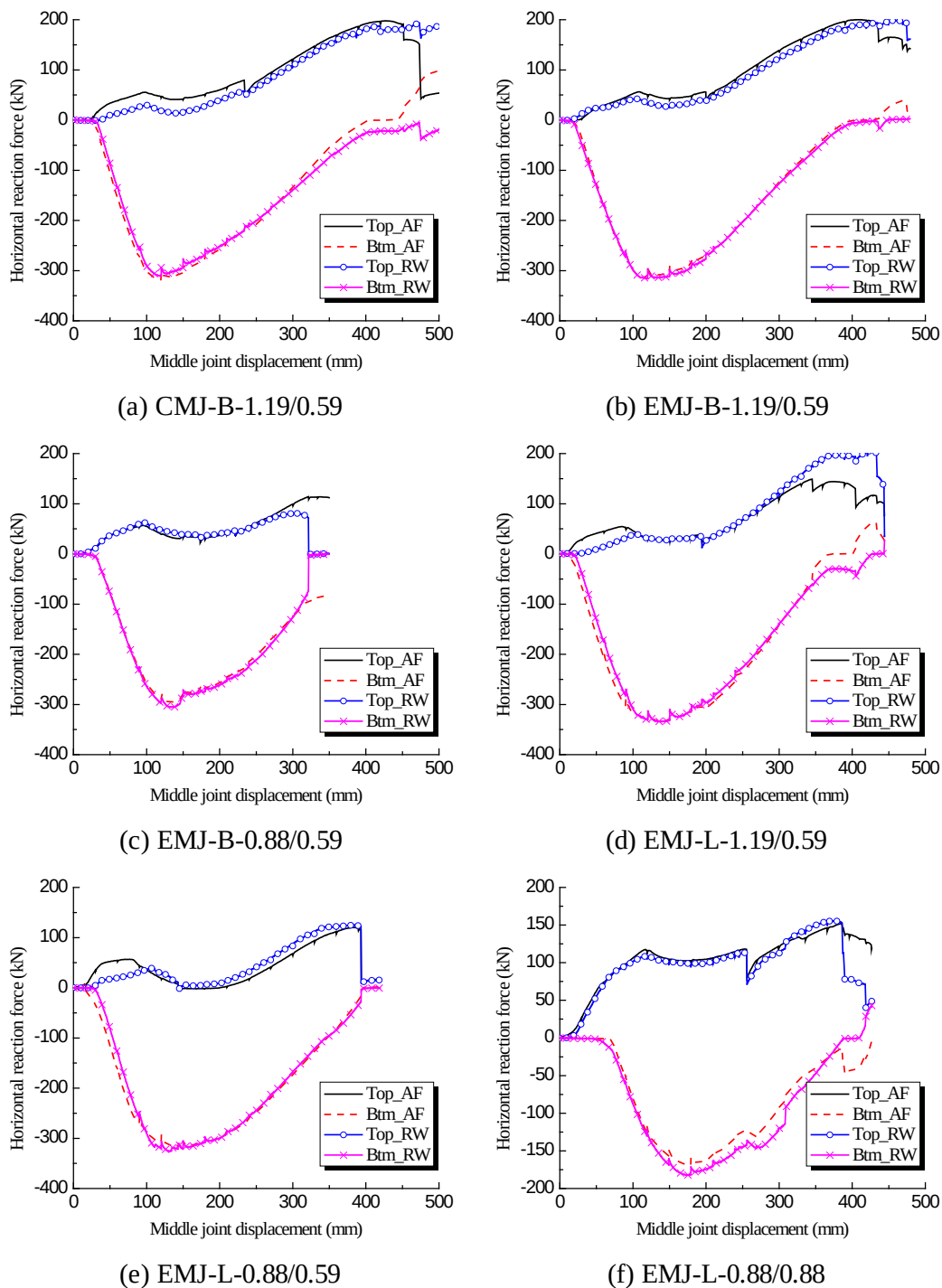


Fig. A.8 Horizontal reaction forces of precast beam-column sub-assemblages with ECC

A.2.2 Stiffness of horizontal restraints

Fig. A.9 to Fig. A.14 shows the horizontal force-displacement curves of ECC sub-assemblages. Similar horizontal reaction force-displacement curves are obtained for

ECC sub-assemblages. However, compared to other sub-assemblages, EMJ-L-0.88/0.88 developed substantially lower horizontal compression force at the CAA stage, as a result of larger connection gaps between the end column stubs and bottom restraints, as shown in **Fig. A.14**. Furthermore, stiffness of horizontal restraints and connection gaps are also determined. **Table A.2** summarises the boundary conditions of beam-column sub-assemblages with cast-in-situ ECC topping and beam-column joint. As mentioned before, connection gaps in the top and bottom restraints might not be attained at the same time.

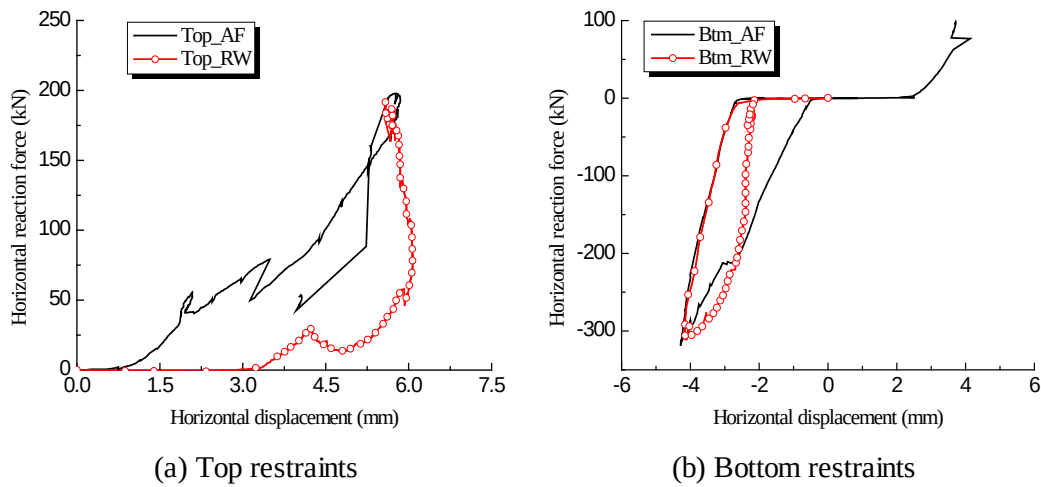


Fig. A.9 Horizontal force-displacement relationships of CMJ-B-1.19/0.59

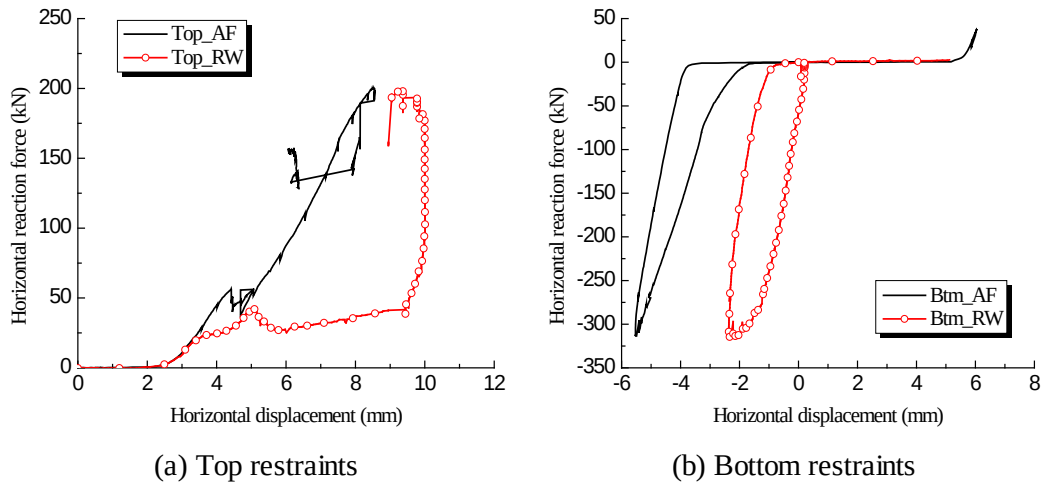


Fig. A.10 Horizontal force-displacement relationships of EMJ-B-1.19/0.59

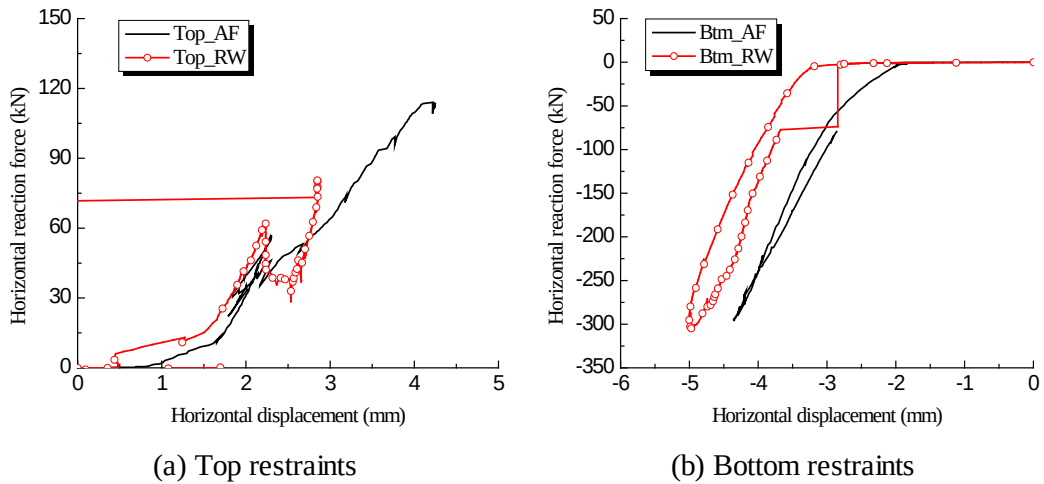


Fig. A.11 Horizontal force-displacement relationships of EMJ-B-0.88/0.59

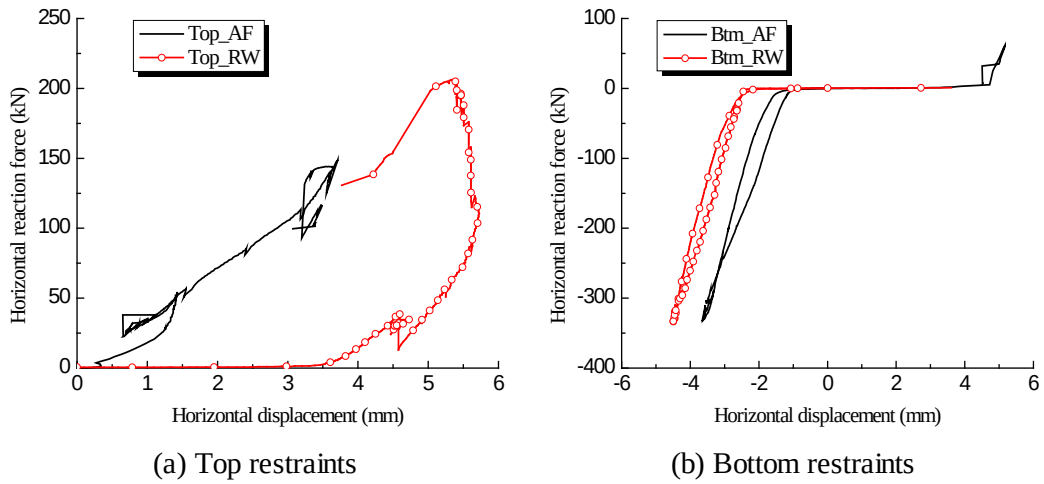


Fig. A.12 Horizontal force-displacement relationships of EMJ-L-1.19/0.59

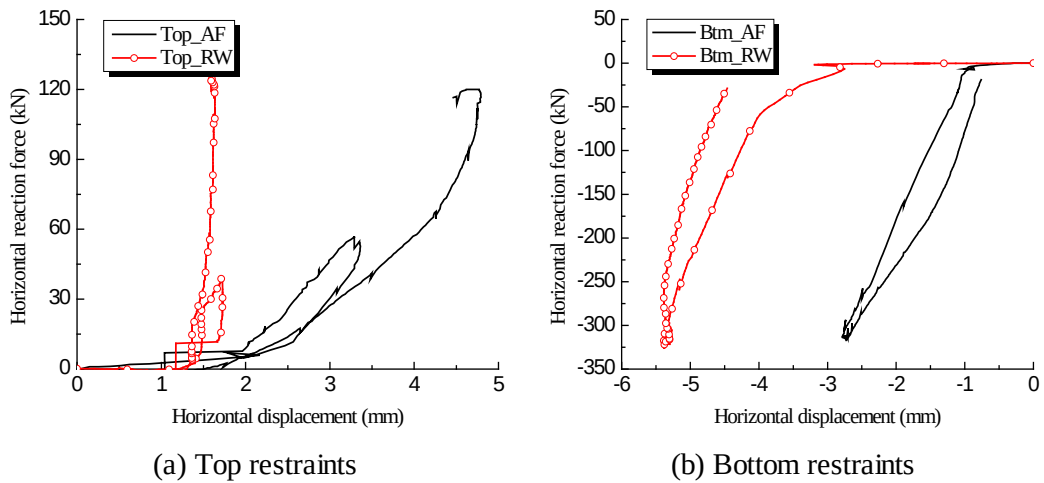


Fig. A.13 Horizontal force-displacement relationships of EMJ-L-0.88/0.59

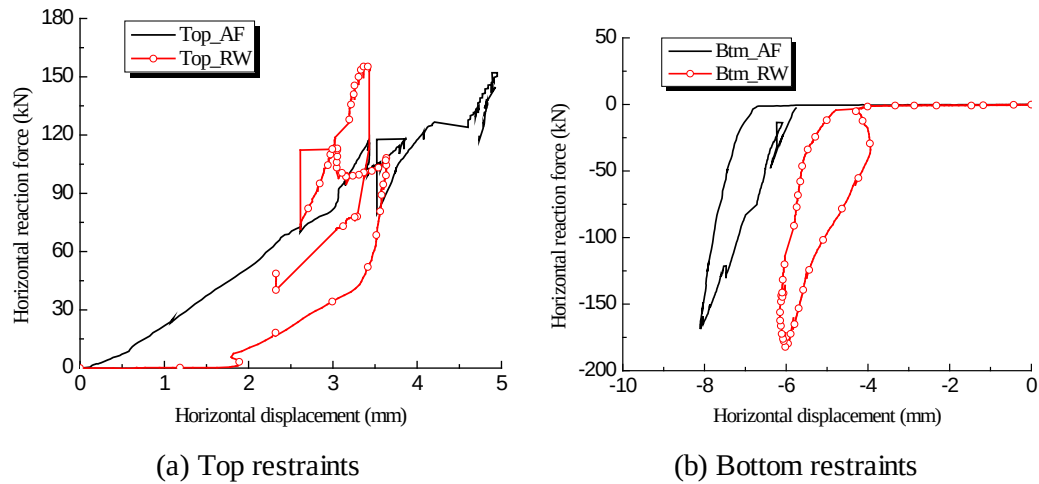


Fig. A.14 Horizontal force-displacement relationships of EMJ-L-0.88/0.88

Table A.2 Horizontal stiffness of beam-column sub-assemblages with ECC

Specimen	Horizontal restraint	Tension stiffness (N/mm)	Compression stiffness (N/mm)	Tension gap (mm)	Compression gap (mm)
CMJ-B-1.19/0.59	Top_AF	35365	--	1.0	--
	Btm_AF	74044	202709	2.6	-2.8
	Top_RW	32101	--	3.3	--
	Btm_RW	--	206435	--	-2.8
EMJ-B-1.19/0.59	Top_AF	34843	--	3.0	--
	Btm_AF	73587	182843	5.5	-3.9
	Top_RW	13993	--	2.2	--
	Btm_RW	--	243320	--	-1.2
EMJ-B-0.88/0.59	Top_AF	35135	--	1.0	--
	Btm_AF	--	149253	--	-2.4
	Top_RW	70782	--	1.4	--
	Btm_RW	--	171223	--	-3.2
EMJ-L-1.19/0.59	Top_AF	40071	--	0.2	--
	Btm_AF	109881	176716	4.6	-1.7
	Top_RW	38560	--	3.6	--
	Btm_RW	--	184997	--	-2.7
EMJ-L-0.88/0.59	Top_AF	49276	--	2.3	--
	Btm_AF	--	170891	--	-0.9
	Top_RW	96984	--	1.3	--
	Btm_RW	--	159386	--	-3.5
EMJ-L-0.88/0.88	Top_AF	33642	--	0.3	--
	Btm_AF	--	150977	--	-7.0

	Top_RW	29917	--	1.7	--
	Btm_RW	--	75900	--	-3.6

In deriving the analytical model for CAA of beam-column sub-assemblages, equivalent gaps for the bridging beam have to be quantified. Moreover, as a result of connection gaps in the top and bottom restraints, end column stubs also experienced free rotation at the initial stage. **Fig. A.15** shows the free rotation of end column stubs and equivalent gap at the centroid of the bridging beam. The equivalent gap for the bridging beam can be computed from Eq. (A-1).

$$\delta_e = \frac{\delta_t + \delta_b}{2} \quad (\text{A-1})$$

where δ_e is the equivalent connection gap at the centroid of the beam; δ_t is the gap in the top restraint; and δ_b is the gap in the bottom restraint. It is noteworthy that negative values are for compression and positive values are for tension.

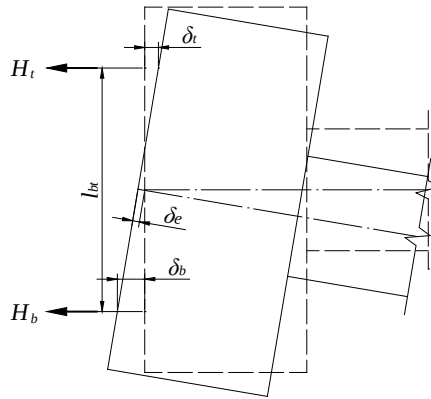


Fig. A.15 Equivalent connection gap at the beam centroid

Besides the horizontal stiffness and corresponding connection gap, rotational stiffness of the bridging beam also needs to be quantified. Bending moment acting on the end column stubs can be calculated based on the force equilibrium, as expressed in **Fig. 4.11**. Meanwhile, rotation of the column stubs can be determined from the measurements of LVDTs (see **Fig. 3.5**), as expressed in Eq. (A-2).

$$\theta_f = \frac{\delta_t - \delta_b}{l_{bt}} \quad (\text{A-2})$$

where θ_f is the rotation angle of the end column stubs and l_{bt} is the distance between the top and bottom horizontal load restraints.

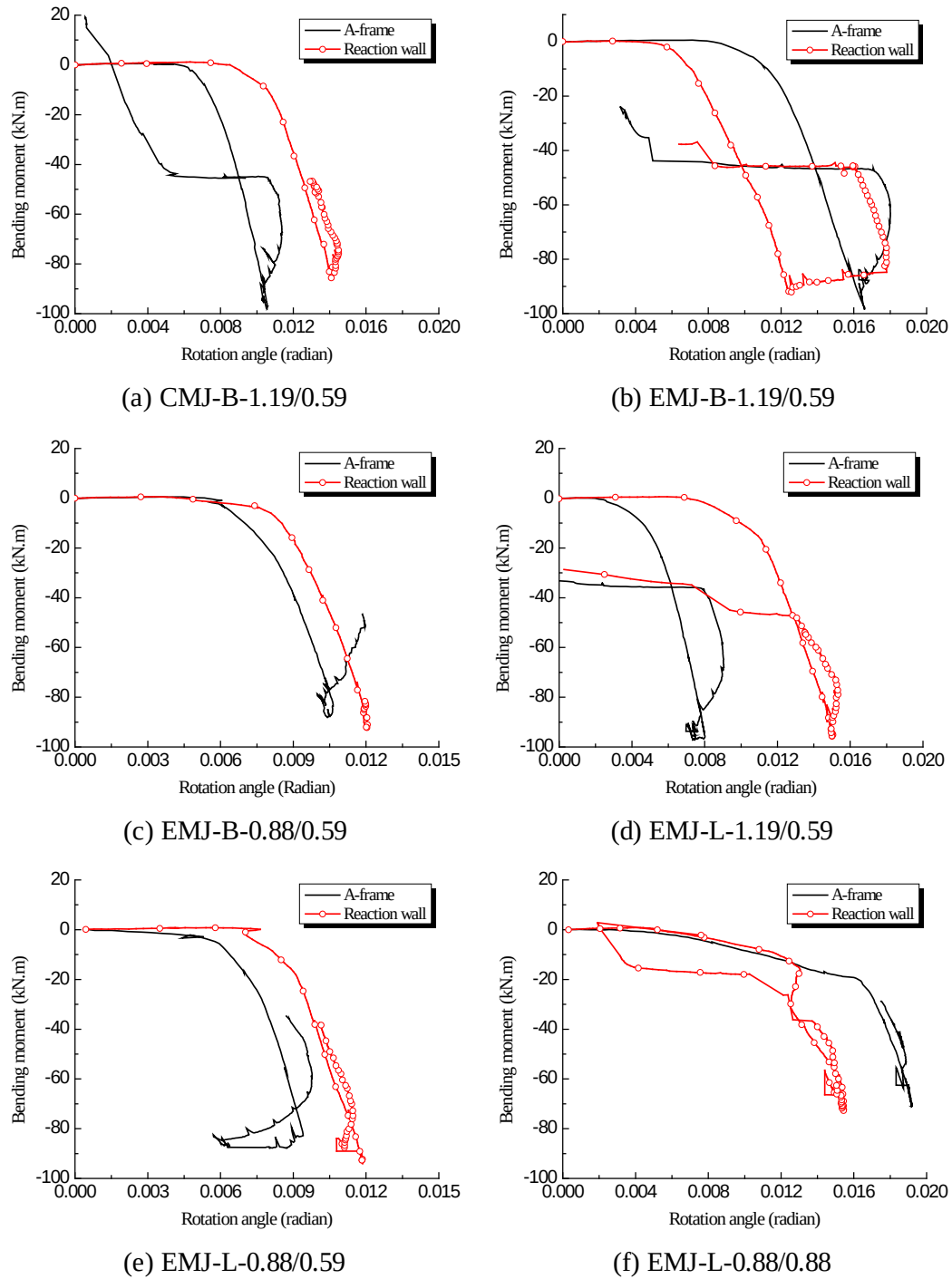


Fig. A.16 Bending moment-rotation relationships of end column stubs

Fig. A.16 shows the bending moment-rotation relationships of end column stubs. At initial stage, free rotation of the end column stubs was allowed as a result of connection gaps in the top and bottom restraints. Thereafter, bending moments increased almost linearly with increasing measured rotations of the end column stubs.

The free rotation angle and rotation stiffness of end column stubs can be quantified by linear regression, as listed in **Table A.3**.

Table A.3 Rotational stiffness of beam-column sub-assemblages with ECC

Specimen	A-frame		Reaction wall	
	Rotational stiffness (kN.m/rad)	Free rotation angle (radian)	Rotational stiffness (kN.m/rad)	Free rotation angle (radian)
CMJ-B-1.19/0.59	25850	0.007	19923	0.01
EMJ-B-1.19/0.59	17564	0.011	14854	0.006
EMJ-B-0.88/0.59	20307	0.007	23004	0.008
EMJ-L-1.19/0.59	29330	0.005	17238	0.01
EMJ-L-0.88/0.59	22572	0.006	24380	0.008
EMJ-L-0.88/0.88	19433	0.015	16059	0.011

A.3 Precast Concrete Frames

As for interior precast concrete frames, horizontal restraints were connected to the top and bottom of the side columns and beam extension, as shown in **Fig. 5.2(a)**. Under column removal scenarios, horizontal compression forces were transmitted to the bottom restraint at the CAA stage, whereas tension force was sustained by the load cell connected to the beam extension at the catenary action stage (see **Figs. 5.10(a and b)**). Exterior frames were only restrained by horizontal load cells at the top and bottom ends of the side columns, as shown in **Fig. 5.2(b)**. At the CAA stage, similar load paths of horizontal compression forces were measured. However, tension forces were mainly sustained by load cells at the top of the side columns following the commencement of catenary action, as shown in **Figs. 5.10(c and d)**.

A.3.1 Horizontal reaction forces

By increasing the top reinforcement ratio in the beams, shear failure was initiated in the side beam-column joints of EF-B-1.19/0.59 and EF-L-1.19/0.59 at the CAA stage, as shown in **Figs. 6.7(a and b)**. Hence, with increasing middle joint displacement, tension forces in the top restraints could not increase, even though beam top reinforcement remained intact at the catenary action stage, as shown in **Figs. A.17(a and b)**. Furthermore, cross section of the side columns in EF-B-1.19/0.59S and EF-L-1.19/0.59S was enlarged to prevent premature shear failure of the side beam-

column joints. Compared to EF-B-1.19/0.59 and EF-L-1.19/0.59, horizontal compression forces in the bottom restraints of EF-B-1.19/0.59S and EF-L-1.19/0.59S did not significantly increased at the CAA stage. However, tension forces sustained by the top and bottom restraints were substantially greater as a result of stiffer side columns, as shown in **Figs. A.17(c and d)**.

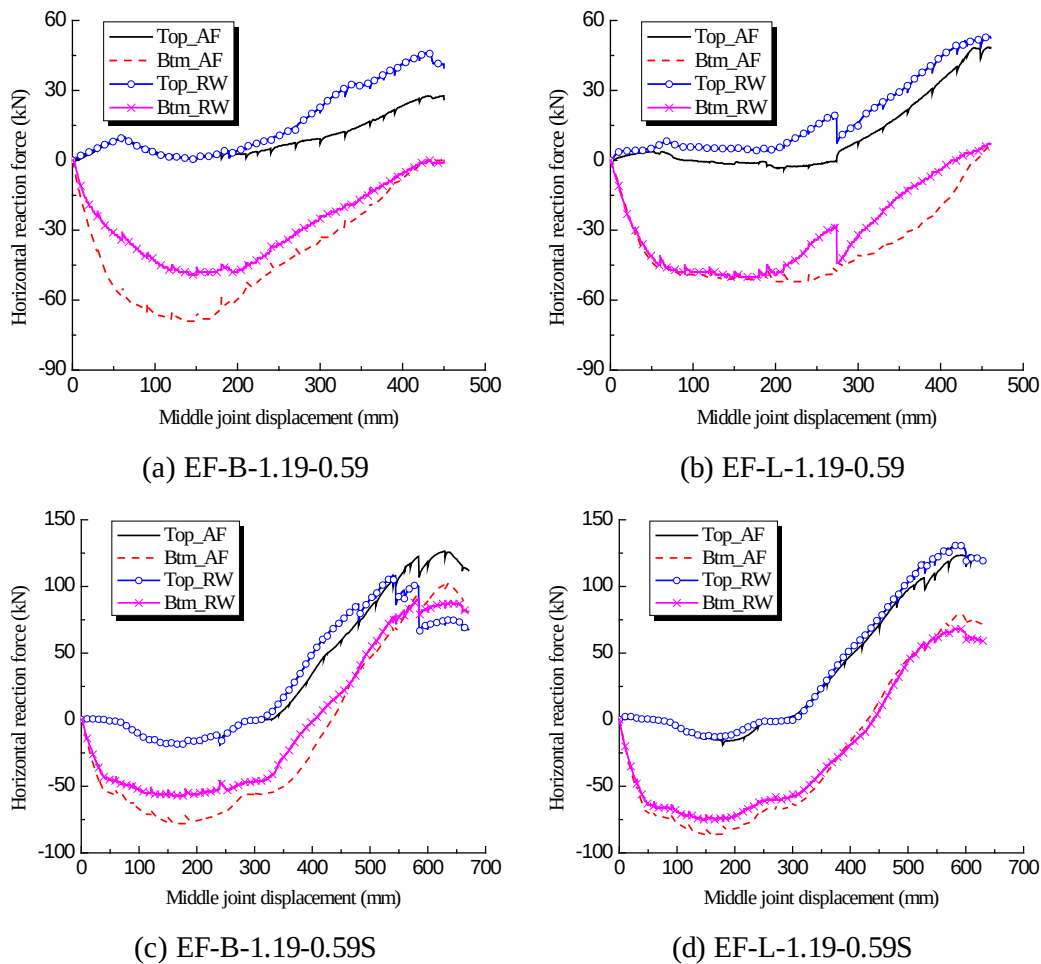


Fig. A.17 Horizontal reaction forces of precast concrete frames

A.3.2 Stiffness of horizontal restraints

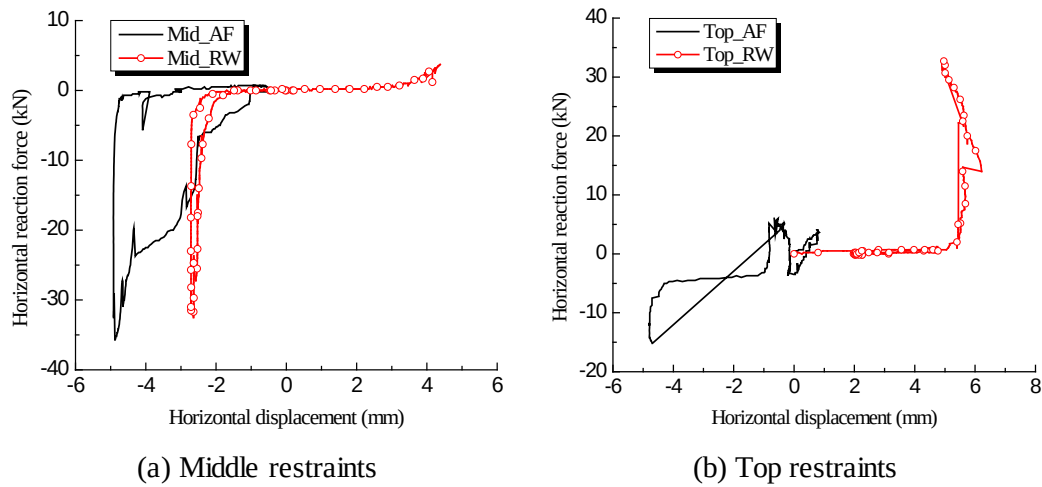


Fig. A.18 Horizontal force-displacement relationships of IF-B-0.88-0.59

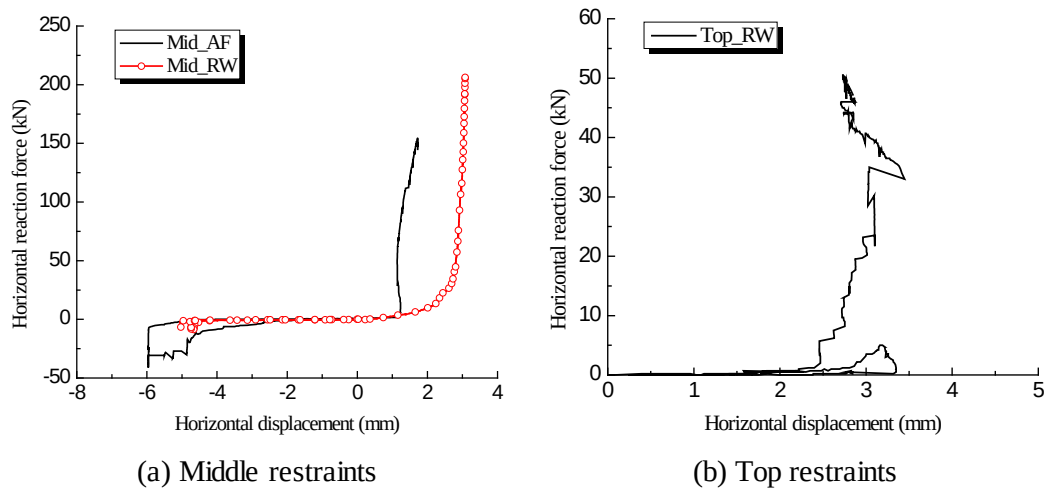


Fig. A.19 Horizontal force-displacement relationships of IF-L-0.88-0.59

For interior precast concrete frames, pin supports were designed at the bottom of the side columns and only horizontal reaction forces were measured. However, both reaction forces and displacements were captured on the column top and at the beam extensions. **Fig. A.18** and **Fig. A.19** show the horizontal force-displacement curves of interior frames. In IF-L-0.88-0.59, LVDT placed at the top load cell near the reaction wall failed to measure the horizontal displacement, and thus only the horizontal force-displacement curve of the top restraint near the A-frame is shown in **Fig. A.19(b)**. **Table A.4** summarises the horizontal stiffness and connection gap of restraints calculated from linear regression.

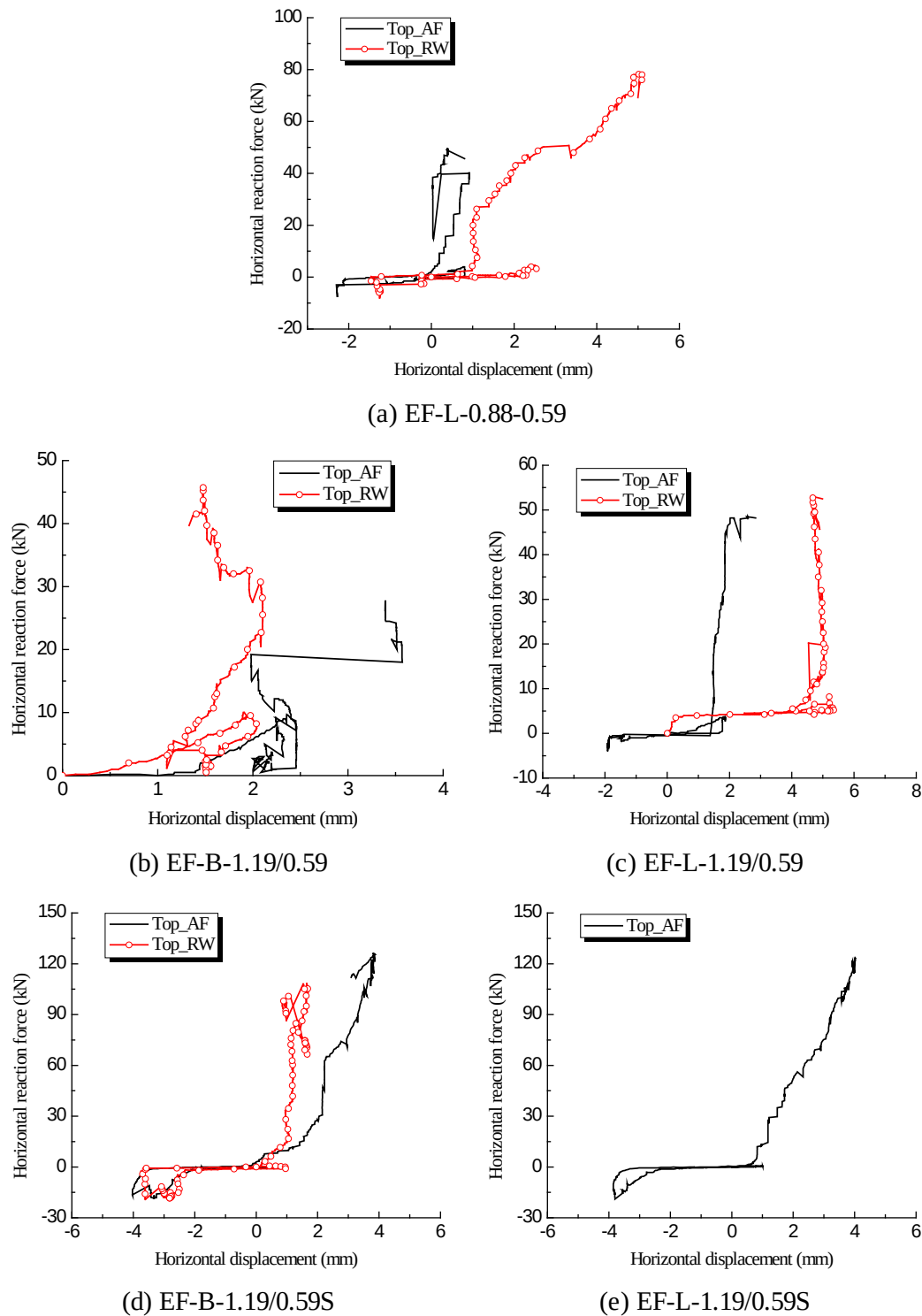


Fig. A.20 Horizontal force-displacement relationships of exterior frames

In exterior frames, only horizontal reaction force and corresponding displacement at the top restraints were recorded, as shown in **Fig. A.20**. As limited compression forces were sustained by the top restraints at the CAA stage, only the stiffness of restraints in tension and associated gap are quantified, as listed in **Table A.4**. Special

attention has to be paid to exterior frame EF-L-0.88-0.59 (see **Fig. 5.10(a)**). Premature fracture of beam top reinforcement hindered the development of tension force in the top restraints. As a result, the horizontal stiffness and connection gap cannot be quantified.

Table A.4 Horizontal stiffness of precast concrete frames

Specimen	A-frame		Reaction wall		Remark*
	Horizontal stiffness (N/mm)	Connection gap (mm)	Horizontal stiffness (N/mm)	Connection gap (mm)	
IF-B-0.88-0.59	9055 [#]	-1.3 [#]	65420 [#]	-2.2 [#]	Middle
	--	--	34024	5.3	Top
IF-L-0.88-0.59	205816	0.9	44466	2.7	Middle
	--	--	45983	2.4	Top
EF-L-0.88-0.59	43206	0.8	14045	1.0	Top
EF-B-1.19/0.59	8382	1.3	26884	1.0	Top
EF-L-1.19/0.59	72291	1.3	--	--	Top
EF-B-1.19/0.59S	44725	1.1	113730	0.7	Top
EF-L-1.19/0.59S	34152	0.6	--	--	Top

*: "Middle" and "Top" represent the horizontal restraints connected to the beam extension and column top, respectively.

[#]: Negative connection gaps and associated stiffness are for compression.