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Department of Structural, Geotechnical and Building Engineering

Master's degree in building engineering

Master Thesis

**Probabilistic evaluation of the robustness of reinforced concrete systems
considering the effects of side face rebars**

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Contents

SUMMARY	11
PREFACE.....	14
CHAPTER 1: STRUCTURAL ROBUSTNESS.....	16
1.1 Introduction.....	16
1.2 Definition of Structural Robustness	16
1.3 Progressive and Disproportionate Collapse.....	17
1.4 Historical Background.....	18
1.5 Design Approaches for Structural Robustness.....	20
1.5.1 Indirect Design Methods	20
1.5.2 Direct Design Methods.....	21
1.6 Alternate Load Path Method	21
1.7 Structural Mechanisms in RC Systems after Column Loss	21
1.7.1 Flexural Action.....	21
1.7.2 Compressive Arch Action.....	22
1.7.3 Catenary Action	22
1.8 Governing Parameters of Structural Response.....	22
1.9 Failure Mechanisms.....	23
1.10 Role of Experimental Studies.....	23
1.11 Role of Numerical Modeling	24
1.12 Scope of the Thesis	24
CHAPTER 2: Experimental Database.....	25
2.1 Purpose and organization of the experimental database.....	25
2.2 Overview of the selected experimental campaigns.....	26
2.3 Common interpretation framework: from flexural action to catenary action	27
2.4 Yu and Tan (2011): baseline beam-column sub-assemblages under middle-column removal	28
2.5 Yu and Tan (2013): effect of reinforcement ratio and span-to-depth ratio.....	32
2.6 Qian et al. (2022): influence of loading regime on column-loss behavior	36
2.7 Farhang Vesali et al. (2013): arching action in longitudinally restrained RC beams.....	40
2.8 Sasani et al. (2011): reinforcement fracture and post-fracture catenary response.....	47
2.9 Clement Huilie report: ATENA 2D reconstruction of selected experimental specimens.....	53
2.10 Lim et al. (2017): rotational capacity and horizontal restraint.....	54
2.11 Ren et al. (2016): B-series continuous RC beams and the role of beam height	59
2.12 NIST experimental study: IMF and SMF beam-column sub-assemblages under column-loss scenario	62



2.13 Cross-study comparison of mechanisms	66
2.14 Implications for later nonlinear finite element modeling	68
2.15 Chapter Conclusion	69
2.16 Database limitations and rules for later use	70
CHAPTER 3: Non-Linear Finite Element Analysis	83
3.1 Introduction.....	83
3.2 Numerical Modeling Strategy	83
3.3 Material Modeling and Calibration Procedure.....	85
3.4 MATLAB–ATENA Integrated Workflow	87
3.5 Nonlinear Analysis Procedure and Structural Response	91
3.6 Concluding Remarks	94
CHAPTER 4: Probabilistic Analysis Framework	95
4.1 Introduction.....	95
4.2 Definition of the Probabilistic Framework.....	95
4.3 Latin Hypercube Sampling and Automated Model Generation.....	96
4.4 Extraction of Capacity Curves and Structural Parameters.....	96
4.5 Statistical Interpretation and Lognormal Fitting	98
4.6 Merging of Probabilistic Results and Global Comparison	99
4.7 Concluding Remarks	99
CHAPTER 5: Improved Numerical Models and Reinforcement Enhancement Strategy	101
5.1 Introduction.....	101
5.2 Reinforcement Enhancement Methodology	101
5.3 Geometric Definition of the Additional Reinforcement	104
5.4 Modification of the Numerical Models.....	105
5.5 Repetition of the Probabilistic Workflow	106
5.6 Structural Behavior of the Improved Models	107
5.7 Concluding Remarks	108
CHAPTER 6: Comparative Analysis between Original and Improved Models.....	109
6.1 Introduction and purpose of the comparative analysis	109
6.2 Comparison indicators and evaluation logic	109
6.3 Comparison of P_o and P_u values	110
6.4 Probability of catenary activation	113
6.5 Classification of improvement trends	116
6.6 Detailed interpretation of the improvement groups	129
6.7 Mechanism-based interpretation of the comparative results.....	131



6.8 Comparison between +25% and +30% reinforcement	131
6.9 Chapter conclusions.....	132
CHAPTER 7: Conclusions and Future Developments	133
7.1 General conclusions	133
7.2 Main methodological outcomes.....	133
7.3 Conclusions from the improved-model comparison	134
7.4 Future developments	135
References.....	136

Index of tables

TABLE 1: EXPERIMENTAL DATABASE OVERVIEW	27
TABLE 2: GEOMETRIC PROPERTIES OF PROTOTYPE FRAMES AND SPECIMENS [6]	30
TABLE 3: MATERIAL PROPERTIES OF REINFORCEMENT AND CONCRETE [6]	30
TABLE 4: FORCE AND DISPLACEMENT AT CRITICAL POINTS OF LOAD-MJD CURVES [6]	31
TABLE 5: YU & TAN (2011) SAMPLES OVERVIEW	32
TABLE 6: GEOMETRIC PROPERTIES OF SUB-ASSEMBLAGE SPECIMENS [7]	33
TABLE 7: MATERIAL PROPERTIES OF STEEL REINFORCEMENT [7]	33
TABLE 8: EXPERIMENTAL RESULTS AT THE FLEXURAL AND CAA STAGES [7]	34
TABLE 9: EXPERIMENTAL RESULTS AT THE CATENARY ACTION STAGE [7]	35
TABLE 10: YU & TAN (2013) SAMPLES OVERVIEW	36
TABLE 11: SPECIMENS PROPERTIES [11]	37
TABLE 12: MATERIAL PROPERTIES OF REINFORCEMENTS [11]	38
TABLE 13: TEST RESULTS [11]	39
TABLE 14: QIAN ET AL. SAMPLES OVERVIEW	40
TABLE 15: DETAILS OF THE LONGITUDINAL REINFORCEMENT, SPACING OF STIRRUPS, CONCRETE COVER ON TOP AND BOTTOM REINFORCEMENT AND ADJUSTED ULTIMATE STRAIN OF EMBEDDED STEEL BARS. [12]	42
TABLE 16: ROTATIONAL STIFFNESS OF END SUPPORTS AND PEAK LOADING CAPACITY OF THE SPECIMENS [12]	45
TABLE 17: FARHANG VESALI ET AL. SAMPLES OVERVIEW	46
TABLE 18: CLEMENT HUILIE SAMPLES OVERVIEW	54
TABLE 19: REINFORCEMENT DETAILS AND GEOMETRIC PROPERTIES OF SPECIMENS. [10]	55
TABLE 20: RATIO OF FLEXURAL AND CATENARY RESISTANCES OF FR-R TO FR BASED ON EC2. [10]	55
TABLE 21: MATERIAL PROPERTIES [10]	55
TABLE 22: ENHANCEMENT RATIO OF CAA AND CA [10]	56
TABLE 23: LIM ET AL. SAMPLES OVERVIEW	59
TABLE 24: SECTIONAL SIZES OF THE PROTOTYPE STRUCTURE AND THE CONTROL SPECIMEN. [9]	60
TABLE 25: CROSS-SECTIONAL DIMENSIONS AND REINFORCEMENT DETAILS (UNITS: MM). [9]	60
TABLE 26: PROGRESSIVE COLLAPSE RESISTANCES OF THE TESTED SPECIMENS. [9]	61
TABLE 27: REN ET AL. SAMPLES OVERVIEW	62
TABLE 28: AVERAGE COMPRESSIVE AND TENSILE STRENGTHS OF 6X12 CONCRETE CYLINDERS AT THE TIME OF TESTING [2]	62
TABLE 29: OVERALL SUMMARY OF THE EXPERIMENTAL DATABASE	82
TABLE 30: CALIBRATION PROCESS SUMMARY	85
TABLE 31: MATLAB SCRIPTS DIVISION	88
TABLE 32: ATENA CCT FILE CONTENTS	90
TABLE 33: SUMMARY OF THE NUMERICAL WORKFLOW ADOPTED IN THE THESIS, FROM THE PREPARATION AND CALIBRATION OF THE ORIGINAL ATENA MODEL TO MATERIAL SAMPLING, CCT REGENERATION, AND ATENA 2D ANALYSES FOR OBTAINING PROBABILISTIC CAPACITY CURVES.	90
TABLE 34: MAIN ATENA MODELLING COMPONENTS USED IN THE NONLINEAR FINITE ELEMENT ANALYSIS AND THEIR INFLUENCE ON THE CAPACITY-CURVE RESPONSE, HIGHLIGHTING THEIR RELEVANCE TO FLEXURAL BEHAVIOUR, CONCRETE CRUSHING, SHEAR TRANSFER, REINFORCEMENT YIELDING, ...	91

TABLE 35: COMPARISON OF INITIAL AND ENHANCED REINFORCEMENT RATIOS FOR THE INVESTIGATED SAMPLES, SHOWING THE BASELINE REINFORCEMENT RATIO AND THE MODIFIED VALUES OBTAINED BY APPLYING 25% AND 30% REINFORCEMENT INCREASES.	102
TABLE 36: ADDITIONAL REINFORCEMENTS' COORDINATES FOR EACH SAMPLE	106
TABLE 37: GLOBAL COMPARISON INDICATORS ACROSS THE 26 SPECIMENS.	110
TABLE 38: REPRESENTATIVE PU CHANGES: STRONGEST INCREASES AND REDUCTIONS.	111
TABLE 39: FULL COMPARISON OF MEAN PO VALUES FOR ALL SPECIMENS.	112
TABLE 40: FULL COMPARISON OF MEAN PU VALUES FOR ALL SPECIMENS.	113
TABLE 41: CATENARY ACTIVATION PROBABILITY FOR ALL SPECIMENS.	115

Index of figures

FIGURE 1: PROGRESSIVE COLLAPSE SCHEMATIC VIEW[3]	17
FIGURE 2: ALTERNATE LOAD PATH SCHEMATIC VIEW[3]	18
FIGURE 3: STRUCTURAL CONTINUITY SCHEMATIC REPRESENTATION[3]	20
FIGURE 4: PLAN VIEW OF PROTOTYPE BUILDING, FRONT VIEW OF THE PROTOTYPE BUILDING[6]	29
FIGURE 5: DETAILING OF SIMPLIFIED BEAM-COLUMN SUB-ASSEMBLAGES[6]	29
FIGURE 6: TEST SET-UP, DETAILS OF TEST SET-UP [6]	30
FIGURE 7: DETAILING AND BOUNDARY CONDITIONS OF THE SPECIMENS (UNIT: MM) [7]	32
FIGURE 8: LOADING REGIMES FOR DISPROPORTIONATE COLLAPSE STUDIES: A) DYNAMIC LOADING REGIME B) CONCENTRATED LOADING REGIME C) SEQUENTIAL LOADING REGIME [11]	37
FIGURE 9: DIMENSIONS AND REINFORCEMENT DETAILING OF SPECIMENS A) CL/SL-13 B) CL/SL-11 C) CL/SL-8 [11]	38
FIGURE 10: OUTLINE OF FLOOR PLAN [12]	41
FIGURE 11: OUTLINE OF GEOMETRY. CROSS-SECTION AND REINFORCING DETAILS FOR THE BEAM ASSEMBLAGES. [12]	41
FIGURE 12: EXPERIMENTAL SET UP FOR THE REINFORCED CONCRETE ASSEMBLAGES. [12]	42
FIGURE 13: OUTLINE OF THE CRACKS DEVELOPED AT SECTIONS ADJACENT TO THE CENTER STUB. [12]	43
FIGURE 14: OUTLINE OF THE (A) GENERIC MODEL ADOPTED IN THE NUMERICAL STUDY [1], (B) BENDING MOMENT-ROTATION RELATIONSHIP FOR NODAL SPRINGS CAPTURING STRAIN PENETRATION AND (C) PROCEDURE FOR CALCULATING THE STIFFNESS OF ROTATIONAL SPRINGS. [12]	44
FIGURE 15: OUTLINE OF THE ITERATIVE PROCEDURE ADOPTED FOR CALIBRATING THE STIFFNESS OF ROTATIONAL SPRINGS WHICH REPRESENT END SUPPORTS. [12]	45
FIGURE 16: TYPICAL FLOOR PLAN & REMOVAL OF COLUMN DUE TO EXPLOSION/IMPACT [5]	47
FIGURE 17: SPECIMEN AND LOADING SETUP [5]	47
FIGURE 18: REINFORCEMENT DETAILING OF TWO-SPAN EXTERIOR BEAM BETWEEN AXES C AND E IN SECOND FLOOR [5]	48
FIGURE 19: BEAM ROTATIONS OVER 7.5 IN. (190MM) FROM FACE OF REMOVED COLUMN AND SIDE SUPPORT (COLUMN) [5]	48
FIGURE 20: FINITE ELEMENT MODEL CHART[3]	49
FIGURE 21: FINITE ELEMENT MODEL OF A QUARTER OF TWO-SPAN BEAM & CROSS SECTION DISCRETIZATION [5]	50
FIGURE 22: BEAM ROTATIONS OVER 7.5 IN. (190MM) FROM FACE OF REMOVED COLUMN AND SIDE SUPPORT (COLUMN) [5]	50
FIGURE 23: BOTTOM AND TOP STEEL BAR TENSILE STRAINS IN VICINITY OF REMOVED COLUMN AND SIDE SUPPORT (COLUMN) [5]	51
FIGURE 24: LONGITUDINAL STRAINS IN BEAM CENTER PLANE (A) AT PEAK RESISTING FORCE, AND (B) SOON AFTER BAR FRACTURE; AND IN BEAM SECTIONS (C) AT FACE OF CENTER BLOCK (LEFT) AT PEAK RESISTING FORCE, AND (RIGHT) SOON AFTER BAR FRACTURE; AND IN BEAM SECTIONS (D) AT FACE [5]	51
FIGURE 25: BEAM'S DEFORMED SHAPE RIGHT AFTER BAR FRACTURE [5]	52
FIGURE 26: 2-D RC FRAME SPECIMENS & DIFFERENT SCENARIOS REPRESENTED BY SPECIMENS.[10]	54
FIGURE 27: TYPICAL TEST SETUP [10]	56
FIGURE 28: CRACK PATTERNS IN BEAMS AND COLUMNS OF FR, FR-S AND FR-R. [10]	57

FIGURE 29: FAILURE MODES OF IR-1 AND IR-2. [10]	58
FIGURE 30: THE PROTOTYPE STRUCTURE (UNITS: M). [9]	59
FIGURE 31: TEST SETUP & DETAILS OF TEST SETUP AND ARRANGEMENT OF INSTRUMENTATION (UNITS: MM) [9]	60
FIGURE 32: DETAILS OF IMF SPECIMEN [2]	63
FIGURE 33: DETAILS OF SMF SPECIMEN [2]	63
FIGURE 34: SMF SPECIMEN [2]	64
FIGURE 35: TEST CONFIGURATION FOR IMF AND SMF SPECIMENS [2]	64
FIGURE 36: CRACK PATTERNS OF IMF AND SMF SPECIMEN AT CENTER COLUMN DEFLECTIONS OF (A) 24 IN (610 MM) AND (B) 43 IN (1092 MM) [2]	65
FIGURE 37: ATENA 2D WORKING SPACE	84
FIGURE 38: SOME GRAPHS (CAPACITY CURVES) RELATED TO CALIBRATION OF THE MODEL WITH RESPECT TO IT'S EXPERIMENTAL DATA	86
FIGURE 39: EXAMPLES OF EXCEL-BASED TEMPLATES FOR CALIBRATION OF THE SAMPLES (GRAY ROWS WHERE THE PARAMETERS THAT WOULD BE CALIBRATED AND BE PROBABILISTICALLY CHANGED AFTER RUNNING THE MATLAB CODE (LHS))	87
FIGURE 40: A VIEW OF THE LHS MATLAB CODE	88
FIGURE 41: EXCEL-BASED TEMPLATES BEFORE AND AFTER RUNNING THE LHS MATLAB CODE	89
FIGURE 42: PART OF ATENA CCT FILE CONFIGURATION	89
FIGURE 43: INDICATION OF SOLUTION PARAMETERS FOR THE SAMPLE B3	92
FIGURE 44: PRE-DEFINED MONITORING POINTS FOR THE SAMPLE B3	93
FIGURE 45: INDICATION OF PO AND PU	93
FIGURE 46: CAPACITY CURVE GENERATED AFTER THE ANALYSIS INSIDE ATENA 2D	97
FIGURE 47: INDICATION OF SELECTED PO (RED CIRCLE) AND PU (BLACK CIRCLE) WITH AUTOMATIC MATLAB CODE	97
FIGURE 48: PDF GRAPHS GENERATED BASED ON LOGNORMAL FOR SAMPLES A) SPECIMEN-02, B) S2	98
FIGURE 49: VARIATION OF REINFORCEMENT RATIO ACROSS THE ANALYZED SPECIMENS FOR THE BASELINE CONFIGURATION AND THE TWO STRENGTHENED CASES, CORRESPONDING TO 25% AND 30% INCREASES IN REINFORCEMENT.	103
FIGURE 50: REINFORCEMENT ARRANGEMENTS A) BEFORE ADDING ADDITIONAL REINFORCEMENTS, B) AFTER ADDING ADDITIONAL REINFORCEMENTS	103
FIGURE 51: ATENA 2D REINFORCEMENT MODELLING INTERFACE FOR THE INVESTIGATED SPECIMEN, ILLUSTRATING THE DISCRETE BAR REINFORCEMENT ARRANGEMENT AND THE ASSIGNED REINFORCEMENT PARAMETERS, INCLUDING SEGMENT GEOMETRY, STEEL MATERIAL MODEL, CROSS-SECTIONAL AREA.	104
FIGURE 52: AVERAGE PO AND PU FOR ORIGINAL, +25%, AND +30% CONFIGURATIONS.	110
FIGURE 53: PERCENTAGE CHANGE IN MEAN PU RELATIVE TO THE ORIGINAL CONFIGURATION.	113
FIGURE 54: CATENARY ACTIVATION PROBABILITY FOR THE 26 SPECIMENS.	116
FIGURE 55: PROBABILISTIC EVALUATION OF THE MEHRDAD SPECIMEN FOR DIFFERENT REINFORCEMENT ARRANGEMENTS, THE FITTED LOGNORMAL PROBABILITY DENSITY FUNCTIONS, AND THE PROBABILITY OF CATENARY ACTIVATION	117
FIGURE 56: PROBABILISTIC EVALUATION OF THE IR-1 SPECIMEN FOR DIFFERENT REINFORCEMENT ARRANGEMENTS, THE FITTED LOGNORMAL PROBABILITY DENSITY FUNCTIONS, AND THE PROBABILITY OF CATENARY ACTIVATION	117
FIGURE 57: PROBABILISTIC EVALUATION OF THE B3 SPECIMEN FOR DIFFERENT REINFORCEMENT ARRANGEMENTS, THE FITTED LOGNORMAL PROBABILITY DENSITY FUNCTIONS, AND THE PROBABILITY OF CATENARY ACTIVATION	118

FIGURE 58: PROBABILISTIC EVALUATION OF THE S7 SPECIMEN FOR DIFFERENT REINFORCEMENT ARRANGEMENTS, THE FITTED LOGNORMAL PROBABILITY DENSITY FUNCTIONS, AND THE PROBABILITY OF CATENARY ACTIVATION	118
FIGURE 59: PROBABILISTIC EVALUATION OF THE FR SPECIMEN FOR DIFFERENT REINFORCEMENT ARRANGEMENTS, THE FITTED LOGNORMAL PROBABILITY DENSITY FUNCTIONS, AND THE PROBABILITY OF CATENARY ACTIVATION	119
FIGURE 60: PROBABILISTIC EVALUATION OF THE B2 SPECIMEN FOR DIFFERENT REINFORCEMENT ARRANGEMENTS, THE FITTED LOGNORMAL PROBABILITY DENSITY FUNCTIONS, AND THE PROBABILITY OF CATENARY ACTIVATION	119
FIGURE 61: PROBABILISTIC EVALUATION OF THE IMF SPECIMEN FOR DIFFERENT REINFORCEMENT ARRANGEMENTS, THE FITTED LOGNORMAL PROBABILITY DENSITY FUNCTIONS, AND THE PROBABILITY OF CATENARY ACTIVATION	120
FIGURE 62: PROBABILISTIC EVALUATION OF THE SMF SPECIMEN FOR DIFFERENT REINFORCEMENT ARRANGEMENTS, THE FITTED LOGNORMAL PROBABILITY DENSITY FUNCTIONS, AND THE PROBABILITY OF CATENARY ACTIVATION	120
FIGURE 63: PROBABILISTIC EVALUATION OF THE CL-11 SPECIMEN FOR DIFFERENT REINFORCEMENT ARRANGEMENTS, THE FITTED LOGNORMAL PROBABILITY DENSITY FUNCTIONS, AND THE PROBABILITY OF CATENARY ACTIVATION	121
FIGURE 64: PROBABILISTIC EVALUATION OF THE CL-13 SPECIMEN FOR DIFFERENT REINFORCEMENT ARRANGEMENTS, THE FITTED LOGNORMAL PROBABILITY DENSITY FUNCTIONS, AND THE PROBABILITY OF CATENARY ACTIVATION	121
FIGURE 65: PROBABILISTIC EVALUATION OF THE FR-R SPECIMEN FOR DIFFERENT REINFORCEMENT ARRANGEMENTS, THE FITTED LOGNORMAL PROBABILITY DENSITY FUNCTIONS, AND THE PROBABILITY OF CATENARY ACTIVATION	122
FIGURE 66: PROBABILISTIC EVALUATION OF THE FR-S SPECIMEN FOR DIFFERENT REINFORCEMENT ARRANGEMENTS, THE FITTED LOGNORMAL PROBABILITY DENSITY FUNCTIONS, AND THE PROBABILITY OF CATENARY ACTIVATION	122
FIGURE 67: PROBABILISTIC EVALUATION OF THE S1 SPECIMEN FOR DIFFERENT REINFORCEMENT ARRANGEMENTS, THE FITTED LOGNORMAL PROBABILITY DENSITY FUNCTIONS, AND THE PROBABILITY OF CATENARY ACTIVATION	123
FIGURE 68: PROBABILISTIC EVALUATION OF THE S2 SPECIMEN FOR DIFFERENT REINFORCEMENT ARRANGEMENTS, THE FITTED LOGNORMAL PROBABILITY DENSITY FUNCTIONS, AND THE PROBABILITY OF CATENARY ACTIVATION	123
FIGURE 69: PROBABILISTIC EVALUATION OF THE S3 SPECIMEN FOR DIFFERENT REINFORCEMENT ARRANGEMENTS, THE FITTED LOGNORMAL PROBABILITY DENSITY FUNCTIONS, AND THE PROBABILITY OF CATENARY ACTIVATION	124
FIGURE 70: PROBABILISTIC EVALUATION OF THE S4 SPECIMEN FOR DIFFERENT REINFORCEMENT ARRANGEMENTS, THE FITTED LOGNORMAL PROBABILITY DENSITY FUNCTIONS, AND THE PROBABILITY OF CATENARY ACTIVATION	124
FIGURE 71: PROBABILISTIC EVALUATION OF THE S5 SPECIMEN FOR DIFFERENT REINFORCEMENT ARRANGEMENTS, THE FITTED LOGNORMAL PROBABILITY DENSITY FUNCTIONS, AND THE PROBABILITY OF CATENARY ACTIVATION	125
FIGURE 72: PROBABILISTIC EVALUATION OF THE S6 SPECIMEN FOR DIFFERENT REINFORCEMENT ARRANGEMENTS, THE FITTED LOGNORMAL PROBABILITY DENSITY FUNCTIONS, AND THE PROBABILITY OF CATENARY ACTIVATION	125
FIGURE 73: PROBABILISTIC EVALUATION OF THE SPECIMEN-01 SPECIMEN FOR DIFFERENT REINFORCEMENT ARRANGEMENTS, THE FITTED LOGNORMAL PROBABILITY DENSITY FUNCTIONS, AND THE PROBABILITY OF CATENARY ACTIVATION	126

FIGURE 74: PROBABILISTIC EVALUATION OF THE SPECIMEN-02 SPECIMEN FOR DIFFERENT REINFORCEMENT ARRANGEMENTS, THE FITTED LOGNORMAL PROBABILITY DENSITY FUNCTIONS, AND THE PROBABILITY OF CATENARY ACTIVATION	126
FIGURE 75: PROBABILISTIC EVALUATION OF THE SPECIMEN-03 SPECIMEN FOR DIFFERENT REINFORCEMENT ARRANGEMENTS, THE FITTED LOGNORMAL PROBABILITY DENSITY FUNCTIONS, AND THE PROBABILITY OF CATENARY ACTIVATION	127
FIGURE 76: PROBABILISTIC EVALUATION OF THE SPECIMEN-04 SPECIMEN FOR DIFFERENT REINFORCEMENT ARRANGEMENTS, THE FITTED LOGNORMAL PROBABILITY DENSITY FUNCTIONS, AND THE PROBABILITY OF CATENARY ACTIVATION	127
FIGURE 77: PROBABILISTIC EVALUATION OF THE SPECIMEN-05 SPECIMEN FOR DIFFERENT REINFORCEMENT ARRANGEMENTS, THE FITTED LOGNORMAL PROBABILITY DENSITY FUNCTIONS, AND THE PROBABILITY OF CATENARY ACTIVATION	128
FIGURE 78: PROBABILISTIC EVALUATION OF THE SPECIMEN-06 SPECIMEN FOR DIFFERENT REINFORCEMENT ARRANGEMENTS, THE FITTED LOGNORMAL PROBABILITY DENSITY FUNCTIONS, AND THE PROBABILITY OF CATENARY ACTIVATION	128

SUMMARY

This thesis investigates the robustness of reinforced concrete (RC) structures subjected to local damage scenarios, with particular attention to the sudden loss of a supporting column. The study is motivated by the need to ensure that structural systems can sustain accidental or exceptional events without developing collapse mechanisms that are disproportionate to the initial damage. In this context, robustness is interpreted as the ability of a structure to redistribute internal forces, activate alternative load paths, and preserve load-carrying capacity after the interruption of a primary vertical support.

The research focuses on the progressive collapse response of RC beam-column sub-assemblages and related structural systems. The structural behavior after column loss is not governed by a single mechanism, but by a progressive sequence of resisting actions. At small displacements, the response is mainly controlled by flexural action, where bending resistance and plastic hinge formation dominate. As displacement increases and axial restraint becomes significant, compressive arch action may develop, increasing the load-carrying capacity through axial compression in the beams. At large displacements, after flexural and arching mechanisms degrade, catenary action may become the dominant resisting mechanism, with tensile forces in the longitudinal reinforcement providing residual resistance.

A detailed experimental database was first established from selected studies in the literature. The database includes beam-column sub-assemblages, restrained beams, beam-only systems, frame specimens, and full-scale assemblies subjected to column-loss conditions. The selected studies were interpreted using a consistent mechanism-based framework, with attention to geometry, reinforcement detailing, material properties, boundary conditions, loading protocols, observed damage, and failure modes. This database showed that the development of catenary action depends strongly on reinforcement continuity, anchorage conditions, deformation capacity, horizontal restraint, and the ability of critical regions to avoid premature brittle failure.

The experimental evidence was then used as the foundation for nonlinear finite element analysis. The selected specimens were modeled and calibrated in ATENA 2D in order to reproduce the main features of their load-displacement response, including stiffness degradation, cracking, compressive arch action, and catenary action. The calibrated models were not evaluated only through their maximum load, but also through their ability to reproduce the sequence of structural mechanisms observed experimentally. This ensured that the numerical framework remained connected to the physical behavior of the investigated RC systems.

After calibration, the numerical models were integrated with a MATLAB-based probabilistic workflow. The material parameters of each specimen were sampled using Latin Hypercube Sampling Technique by generating thirty probabilistic sub-samples for each original model. These material realizations were then converted into ATENA-compatible CCT files and analyzed to obtain capacity curves. The characteristic points P_o and P_u were extracted from the curves, where P_o represents the first significant resistance peak and P_u represents the ultimate resistance associated with the catenary stage.

The thesis also investigated improved numerical models obtained by increasing the amount of longitudinal reinforcement by 25% and 30% with respect to the original configurations. The additional reinforcements were positioned as side face rebars, approximately at one-third and two-thirds of the beam depth, in order to enhance tensile force redistribution during large deformation. The same probabilistic workflow was then repeated for the improved models, producing a consistent basis for comparing the original, +25%, and +30% reinforcement configurations.

The comparative analysis was carried out in probabilistic terms by assuming lognormal distributions for both P_o and P_u . Based on this probabilistic comparison, the reinforcement enhancement generally increased the average P_o and P_u values, with a more meaningful effect on P_u than on P_o . This indicates that the added reinforcement mainly influenced the catenary-stage response rather than only the early flexural or arching resistance. However, the results also showed that additional reinforcement does not automatically guarantee proportional improvement in robustness. Some specimens exhibited strong activation improvement, while others showed limited benefit because their response was governed by concrete crushing, shear localization, restraint limitations, arching-dominated behavior, or already saturated catenary activation.

The comparison between the +25% and +30% configurations demonstrated that the additional 5% reinforcement increase did not always produce a significant additional benefit. Although the +30% models generally provided slightly higher ultimate resistance, the activation probability did not increase substantially in many cases. This result confirms that the +25% configuration represents a good trade off between robustness improvements and sustainability principles.

Overall, the thesis demonstrates that robustness in reinforced concrete structures is a mechanism-dependent and uncertainty-sensitive property. A reliable robustness assessment must consider not only initial strength, but also deformation capacity, reinforcement continuity, anchorage, boundary restraint, and the probability of catenary action activation. The proposed framework, combining experimental interpretation, nonlinear finite element

modeling, probabilistic analysis, and improved reinforcement configurations, provides a structured approach for evaluating and enhancing the progressive collapse resistance of RC systems under column-loss scenarios.

PREFACE

In recent decades, the concept of structural robustness has gained increasing importance in the field of structural engineering. While traditional design approaches have focused primarily on ensuring safety under predefined loading conditions, a growing body of research has highlighted the need to consider the response of structures under exceptional and unforeseen events. These events, although characterized by low probability, may have severe consequences, including progressive or disproportionate collapse.

Reinforced concrete structures, which constitute a significant portion of the built environment, are particularly relevant in this context due to their widespread use and inherent nonlinear behavior. The complexity of RC systems arises from the interaction between concrete and steel, the presence of cracking and bond-slip phenomena, and the sensitivity of structural performance to detailing and construction practices. These characteristics make the assessment of robustness a challenging but essential task.

One of the most commonly studied scenarios in robustness analysis is the sudden loss of a supporting column. This scenario provides a simplified yet effective representation of local damage and allows researchers to investigate how the remaining structure responds to the redistribution of loads. The ability of the structure to activate alternative load paths and to sustain the applied loads without collapse is a key indicator of its robustness.

The present thesis focuses on this problem and aims to contribute to the understanding of the mechanisms that govern the response of RC structures under column-loss conditions. The work is based on a detailed analysis of experimental studies available in the literature, which provide valuable insights into the behavior of structural components under large deformation and damage conditions.

A fundamental aspect of this research is the recognition that experimental results must be interpreted within a consistent and structured framework. Individual studies often differ in terms of specimen configuration, loading protocols, and measurement techniques, making direct comparison difficult. To address this issue, the thesis adopts a database-oriented approach, in which each experimental study is reformulated using a common set of descriptors. This approach facilitates comparison and enables the identification of general trends and governing parameters.

Another important consideration is the role of different load-resisting mechanisms in the overall structural response. As demonstrated in numerous experimental investigations, the behavior of RC subassemblies under column removal is characterized by a progressive transition between different mechanisms, each associated with specific deformation levels

and structural conditions. Understanding this transition is essential for both the interpretation of experimental results and the development of reliable numerical models.

The scope of the present work is deliberately limited to the initial stages of the research process. The primary objective is to establish a clear conceptual framework and to develop a comprehensive experimental database that can serve as a reference for subsequent analyses. Later stages of the thesis will build upon this foundation to explore numerical modeling techniques, probabilistic approaches, and potential strategies for improving structural robustness.

This phased approach ensures that each component of the research is developed on a solid and well-documented basis. By first focusing on experimental evidence and mechanism identification, the thesis avoids premature assumptions and provides a reliable benchmark for the validation of future models.

In summary, the work presented here represents the first step in a broader investigation into the robustness of reinforced concrete structures. It provides the necessary background, data, and conceptual tools required to address more advanced topics in structural analysis and design. The insights gained from this study are expected to contribute to the development of more resilient structural systems and to support the ongoing evolution of design methodologies in the field of structural engineering.

CHAPTER 1: STRUCTURAL ROBUSTNESS

1.1 Introduction

Modern structural engineering design is traditionally based on the assumption that structures are subjected to predefined actions whose magnitude and probability of occurrence are established through design codes and standards. These include permanent loads, variable loads, and environmental actions such as wind and seismic effects. However, real structures may also be exposed to exceptional or accidental actions, which are not always explicitly considered within conventional design frameworks. These may include impact loads, explosions, construction errors, or localized failures of structural components.

Although such events are characterized by low probability, their consequences can be severe, particularly when they trigger a chain of failures leading to progressive collapse. Historical evidence has shown that structures designed according to standard codes may still be vulnerable to these scenarios, especially when load redistribution mechanisms are insufficient or when structural redundancy is limited (Miceli & Castaldo, 2024).

This has led to the development of the concept of structural robustness, which addresses the ability of a structure to sustain local damage without experiencing collapse that is disproportionate to the initiating event. In reinforced concrete (RC) structures, this problem is particularly complex due to the nonlinear interaction between concrete and steel, the presence of cracking, and the sensitivity of structural behavior to detailing and boundary conditions.

The study of robustness in RC systems therefore requires a comprehensive approach that integrates experimental evidence, theoretical interpretation, and numerical modeling. In this context, the present chapter introduces the fundamental principles of structural robustness, outlines the main design approaches, and presents the key structural mechanisms governing the response of RC systems subjected to column-loss scenarios.

1.2 Definition of Structural Robustness

Structural robustness is commonly defined as the ability of a structure to avoid damage that is disproportionate to the cause of the initiating event. This concept has been formalized in several design guidelines, including the CNR-DT 214/2018 document, which defines robustness as the capacity of a structure to withstand exceptional actions without undergoing collapse that exceeds acceptable limits

This definition emphasizes that robustness is not intended to prevent damage entirely, but rather to control its propagation. In this sense, the focus shifts from the prevention of failure to the mitigation of its consequences.

From an engineering perspective, robustness involves:

- the initiation phase, where local damage occurs
- the propagation phase, where damage may spread
- the final structural response, which must remain limited

These stages highlight the importance of ensuring that structures possess sufficient continuity, redundancy, and ductility, allowing them to redistribute loads after local damage. The absence of such properties can lead to unstable structural behavior and rapid collapse.

Furthermore, robustness is closely related to the concept of structural integrity, which refers to the ability of a structure to remain whole and functional despite damage. In RC systems, integrity is largely governed by reinforcement detailing, bond behavior, and the performance of critical regions such as beam-column joints.

1.3 Progressive and Disproportionate Collapse

The concepts of progressive collapse and disproportionate collapse are fundamental to the understanding of structural robustness and must be clearly distinguished.

Progressive collapse refers to a mechanism of failure propagation, in which an initial local damage triggers a sequence of failures in adjacent structural elements. This process may develop gradually or rapidly, depending on the structural configuration and loading conditions [2].

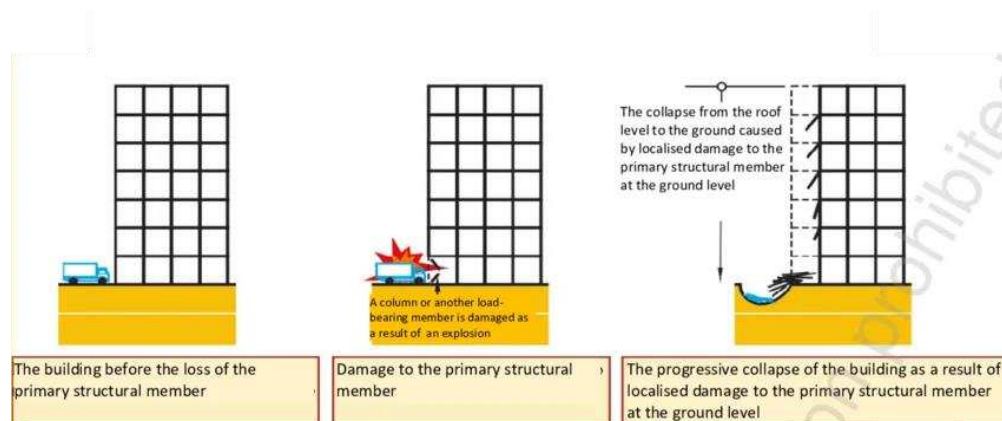


Figure 1: Progressive collapse schematic view[3]

Disproportionate collapse, on the other hand, refers to the extent of damage, where the final collapse is significantly larger than the initiating event. In this context, disproportionate collapse is considered unacceptable from a design perspective, as it indicates a lack of robustness in the structural system. [4]

The relationship between these two concepts can be summarized as follows:

- progressive collapse describes the process
- disproportionate collapse describes the outcome

Experimental studies have shown that the transition from local damage to global collapse is strongly influenced by structural redundancy and the availability of alternative load paths [5], [6].

In RC systems, progressive collapse is often associated with the loss of a column, which disrupts the load path and forces the structure to redistribute loads through adjacent members. The success of this redistribution determines whether the structure survives or collapses.

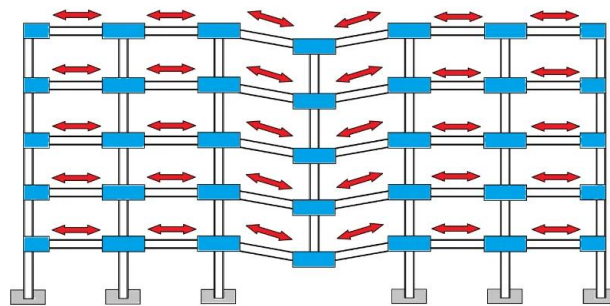


Figure 2: Alternate load path schematic view[3]

1.4 Historical Background

The importance of structural robustness has been highlighted by several catastrophic failures that occurred in the past decades. These events demonstrated that local damage can lead to widespread structural collapse, even when the structure satisfies conventional design requirements.

Among the most significant examples are:

- Ronan Point (1968) [3]



- Alfred P. Murrah Federal Building (1995) [3]



- World Trade Center collapse (2001) [3]



These events revealed critical deficiencies in structural design, including:

- lack of redundancy
- insufficient continuity
- inadequate detailing for extreme conditions

As a result, structural design codes began to incorporate robustness requirements, focusing on the prevention of progressive collapse and the enhancement of structural resilience [1]

These developments have led to a shift in design philosophy, from purely strength-based approaches to more comprehensive strategies that consider structural behavior under abnormal conditions.

1.5 Design Approaches for Structural Robustness

The design of robust structures can be approached through two main strategies: indirect and direct methods. These approaches differ in their level of complexity and the degree to which they explicitly consider damage scenarios.

1.5.1 Indirect Design Methods

Indirect methods aim to improve robustness by enhancing general structural properties without explicitly modeling specific damage scenarios. These methods are widely adopted in design codes due to their simplicity and practicality.

Typical measures include:

- increasing structural continuity
- providing horizontal and vertical ties
- improving ductility
- ensuring redundancy

While these measures contribute to robustness, they do not guarantee adequate performance under specific scenarios such as column removal. Their effectiveness depends on how well they translate into actual load redistribution capacity.

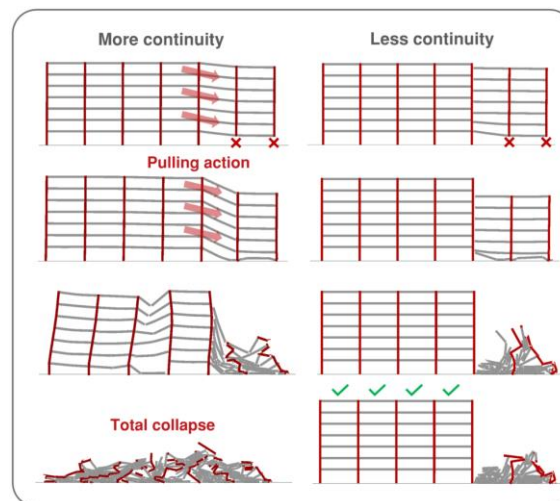


Figure 3: Structural continuity schematic representation[3]

1.5.2 Direct Design Methods

Direct methods explicitly analyze the structural response after local damage. These methods provide a more accurate assessment of robustness but require more detailed modeling.

The main approaches include:

- Alternate Load Path Method (ALPM)
- Key Element Design
- Compartmentalization

Among these, the Alternate Load Path Method is the most widely used for RC structures, as it directly evaluates the ability of the structure to redistribute loads after element removal. [2]

1.6 Alternate Load Path Method

The Alternate Load Path Method is based on the simulation of a local damage scenario, typically the removal of a structural element such as a column. The structure is then analyzed to determine whether it can maintain equilibrium through alternative load paths.

This method allows engineers to directly evaluate the robustness of a structure by examining its ability to:

- redistribute internal forces
- develop new load paths
- sustain applied loads without collapse

Experimental studies have extensively used this approach to investigate the behavior of RC subassemblies under column-loss scenarios. [7], [8]

The method is particularly suitable for research purposes because it provides a clear and controlled framework for studying structural response under extreme conditions.

1.7 Structural Mechanisms in RC Systems after Column Loss

The structural response of RC systems after column removal is characterized by a sequence of load-resisting mechanisms that develop as deformation increases.

1.7.1 Flexural Action

At small deformations, the structural response is dominated by flexural behavior. The beams resist loads through bending, and plastic hinges form at critical sections.

This stage is characterized by:

- linear or mild nonlinear behavior
- development of flexural cracks
- limited contribution of axial forces

Flexural action corresponds to the conventional design behavior of RC members. [6]

1.7.2 Compressive Arch Action

As deformation increases, axial compression develops in the beams due to boundary restraints. This leads to compressive arch action, which enhances load-carrying capacity.

This mechanism is influenced by:

- axial restraint stiffness
- geometric configuration
- concrete strength

Experimental studies have shown that compressive arch action can significantly increase the peak load compared to pure flexural capacity. [9], [10]

1.7.3 Catenary Action

At large deformations, the structural response transitions to catenary action, in which the load is carried primarily by axial tension in the reinforcement.

This stage requires:

- large deformation capacity
- reinforcement continuity
- sufficient ductility

Catenary action provides significant reserve capacity but is highly sensitive to failure mechanisms such as bar fracture and bond-slip. [2], [5]

1.8 Governing Parameters of Structural Response

The ability of a structure to develop these mechanisms depends on several interacting parameters.

These include:

- Geometry, which affects stiffness and deformation capacity

- Reinforcement detailing, which controls ductility and failure modes
- Boundary conditions, which influence load redistribution
- Material properties, which determine strength and deformation limits
- Slab participation, which enhances both stiffness and strength

These parameters must be considered together, as their interaction determines the overall structural response.

1.9 Failure Mechanisms

Failure in RC structures under column removal is typically governed by localized phenomena.

Common failure modes include:

- reinforcement fracture
- concrete crushing
- bond-slip failure
- joint failure

Experimental studies have shown that these local failures often control global structural behavior. [2], [6]

1.10 Role of Experimental Studies

Experimental research is essential for understanding structural robustness, as it provides direct observation of structural behavior under realistic conditions.

It allows engineers to:

- identify mechanism transitions
- observe crack development
- determine failure modes
- validate numerical models

The studies analyzed in this thesis form the basis for the experimental database developed in Chapter 2.

1.11 Role of Numerical Modeling

Numerical modeling complements experimental studies by enabling simulation of complex structural behavior.

Advanced models can capture:

- material nonlinearity
- bond-slip effects
- large deformation behavior
- failure mechanisms

However, their accuracy depends on proper calibration using experimental data.

1.12 Scope of the Thesis

This thesis is structured to progressively develop the analysis of structural robustness in reinforced concrete systems. The present chapter establishes the conceptual framework, while the following chapters build upon this foundation.

The main structure of the thesis is as follows:

- Chapter 1: Structural robustness (conceptual framework)
- Chapter 2: Experimental database
- Chapter 3: Nonlinear finite element analysis
- Chapter 4: Probabilistic analysis
- Chapter 5: Improved models
- Chapter 6: Comparative analysis
- Chapter 7: Conclusions

CHAPTER 2: Experimental Database

2.1 Purpose and organization of the experimental database

The objective of this chapter is to establish a coherent experimental foundation for the study of structural robustness in reinforced concrete (RC) systems subjected to column-loss scenarios. The chapter is not intended to be a simple list of separate papers. Instead, the selected experimental campaigns are treated as a cumulative database, in which each study contributes a specific piece of evidence about how RC members and subassemblies redistribute loads after local damage. The database therefore supports the later numerical and probabilistic parts of the thesis by identifying the mechanisms, parameters, and failure modes that must be reproduced by any reliable nonlinear finite element model.

The selected studies cover different levels of structural representation. Some tests are performed on two-span beam-column sub-assemblages, where the removed column is represented by a central joint or column stub. Other studies isolate restrained beams in order to clarify compressive membrane or arching action under well-controlled boundary conditions. Additional works include beam-slab systems, full-scale RC assemblies, and numerical calibrations based on experimental specimens. The variety of configurations is important because progressive collapse resistance is not controlled by a single parameter. It depends on the interaction between geometry, reinforcement arrangement, boundary restraints, material ductility, bond behavior, slab participation, and the sequence of loading.

The common scenario behind the database is the loss of a vertical support. In an actual building, this local damage may be caused by impact, blast, construction error, foundation failure, or other accidental actions. In experimental research, the event is usually idealized by removing the middle column or by applying a vertical displacement at the location where the support is assumed to be lost. This idealization is consistent with the Alternate Load Path approach because it does not require direct modeling of the initiating hazard; instead, it evaluates whether the remaining structure can develop alternative load paths after the original load path is interrupted.

A central issue throughout the chapter is the evolution of load-resisting mechanisms. At small displacements, RC beams normally resist the imposed load through flexural action and the development of plastic hinges. At moderate displacements, if horizontal restraint is present, beam elongation is restrained and compressive axial force develops, producing compressive arch action. At larger displacements, once flexural capacity and compressive arching degrade, the axial force may change from compression to tension and catenary action may become the dominant mechanism. This sequence is repeatedly documented in

the experimental literature, but the extent to which each mechanism develops depends strongly on the test configuration and on local failure conditions.

For this reason, the database is interpreted using a mechanism-based logic rather than a purely chronological one. Each experimental campaign is discussed with attention to four levels of information: the specimen configuration, the boundary and loading system, the observed structural mechanisms, and the failure process. Quantitative results are reported where they are necessary for later comparison, particularly loads, displacements, reinforcement ratios, span-to-depth ratios, material strengths, and characteristic failure points. At the end of the chapter, the evidence is synthesized to identify the key implications for the subsequent ATENA modeling and for the interpretation of improved numerical models.

2.2 Overview of the selected experimental campaigns

Study	Experimental representation	Main variable or focus	Database relevance
Yu and Tan (2011)	Two half-scale RC beam–column sub-assemblages (S1–S2)	Seismic vs non-seismic detailing; transition from flexural action to compressive arch action (CAA) and catenary action (CA)	Fundamental benchmark for beam–column joint behavior and catenary activation
Yu and Tan (2013)	Series of half-scale RC beam–column sub-assemblages (S3–S8) with varying reinforcement layouts	Influence of top/bottom reinforcement ratio and span-to-depth ratio	Parametric effects on CAA, CA, shear failure, and deformation capacity
Qian et al. (2022)	Beam–column sub-assemblages tested under column removal scenarios	Loading protocol and span-to-depth ratio	Effect of realistic loading conditions on critical sections and catenary capacity
Farhang Vesali et al. (2013)	Axially restrained RC beams subjected to large deformations	Compressive membrane (arching) action under axial restraint	Contribution of axial restraint to strength enhancement and robustness modeling
Sasani et al. (2011)	Full-scale RC frame tests under sudden column removal	Large deformation response and progressive collapse mechanisms	Explicit modeling of bar fracture and post-fracture catenary action

Huilie / ATENA report	Numerical simulations of selected experimental specimens	ATENA 2D calibration and modeling strategies	Model sensitivity, reinforcement continuity, and robustness assessment
Kim and Choi	Analytical and numerical studies on RC frame sub- assemblages	Seismic vs gravity detailing and catenary activation	Role of anchorage, joint integrity, and global frame behavior
Lim et al. (2017)	2D RC frame specimens under column loss scenarios	Rotational capacity and horizontal restraint	Influence of restraint conditions on failure modes
Ren et al. (2016) (<i>B-series emphasized</i>)	Beam-only and beam–slab specimens	Beam height and slab participation	Catenary action, slab membrane contribution, and bar fracture behavior

Table 1: Experimental database overview

The table shows that the selected database is deliberately heterogeneous. It contains sub-assemblages, restrained beams, frame tests, beam-slab systems, and numerical reproductions of experiments. This variety is useful because the thesis is not only concerned with one specimen type; it is concerned with the mechanisms that must be reproduced when an RC structural system is analyzed after column loss. The database therefore gives a broader basis for identifying which parameters are essential and which assumptions may be simplified in later modeling.

2.3 Common interpretation framework: from flexural action to catenary action

The first stage of the response is flexural action. In this stage, the applied vertical displacement or load is mainly resisted by bending moments in the beam spans adjacent to the removed column. Crack formation begins in the tensile regions and plastic hinges tend to form near the beam-column interfaces or other critical sections defined by reinforcement curtailment and boundary conditions. The flexural peak is often used as the first reference point in the load-displacement curve because it represents the capacity that would be expected from ordinary beam behavior before substantial axial effects become dominant.

The second stage is compressive arch action, frequently abbreviated as CAA. This mechanism develops when the beam tries to elongate due to cracking and curvature but the surrounding structure restrains that elongation. The restraint produces axial compression in the beam. This compression increases the effective flexural capacity of the critical sections through axial-flexural interaction and can lead to a load level greater than the pure flexural resistance. The effectiveness of CAA depends strongly on boundary stiffness,

span-to-depth ratio, concrete compressive strength, and the ability of the member to avoid premature crushing or shear failure.

The third stage is catenary action, frequently abbreviated as CA. At large displacements, the beam axial force may change from compression to tension. Once this happens, the vertical load is increasingly carried by the tensile force in the longitudinal reinforcement. Catenary action can provide a large reserve capacity, but it requires the beam-column joint, anchorage zones, and reinforcement to remain sufficiently intact. It is therefore governed by large rotation capacity, bond-slip behavior, bar fracture strain, and reinforcement continuity rather than by conventional flexural strength alone.

The same sequence does not always develop completely. Some specimens terminate in shear failure before meaningful catenary action is mobilized. Others develop catenary action only after local bar fracture has modified the deformation shape. Some tests show that a loading regime based only on concentrated loading can overestimate the ultimate catenary capacity because it shifts the critical section toward the middle column, whereas more realistic sequential loading may produce failure near the side column. These differences are essential for database interpretation and for the calibration of later numerical models.

The interpretation framework used in this thesis therefore does not simply ask whether the maximum load is high or low. It asks which mechanism produced the load, which local region controlled the response, whether the axial force was compressive or tensile, whether the failure was ductile or brittle, and whether the experimental boundary conditions were representative of an actual frame. This is the reason why the following sections discuss each campaign not only in terms of results but also in terms of test philosophy and modeling implications.

2.4 Yu and Tan (2011): baseline beam-column sub-assemblages under middle-column removal

Yu and Tan (2011) represent one of the fundamental experimental references for understanding the progressive collapse resistance of RC beam-column sub-assemblages under a middle-column removal scenario. The study is important because it combines controlled quasi-static testing with numerical modeling, and because it separates the early flexural and compressive arching stages from the large-displacement catenary stage. In the context of this thesis, the paper provides a benchmark for the interpretation of mechanism transition and for the numerical representation of joint and bond-slip effects.

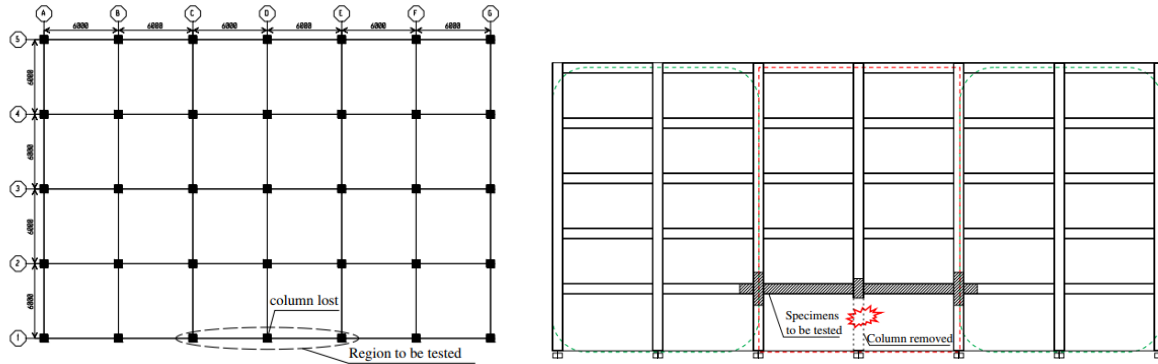


Figure 4: Plan view of prototype building, Front view of the prototype building[6]

The experimental program consisted of two half-scale RC beam-column sub-assemblages, identified as S1 and S2. Each specimen represented a simplified portion of a frame after the loss of a middle column. The two specimens had a similar global geometry, but differed in reinforcement detailing. Specimen S1 was detailed according to seismic principles, with improved confinement and reinforcement continuity, whereas S2 was designed with non-seismic detailing. This comparison was intended to show whether seismic detailing improves progressive collapse resistance under a column-loss scenario.

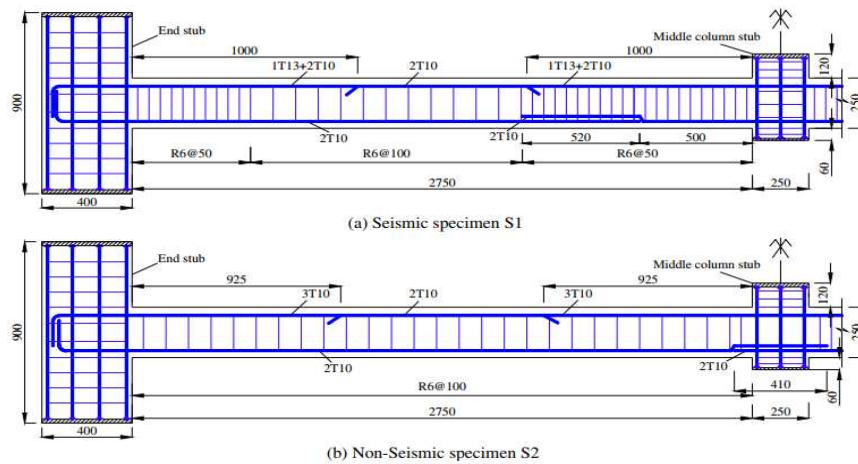


Figure 5: Detailing of simplified beam-column sub-assemblages[6]

Prototype and specimen	Beam net span (mm)	Beam size (mm)		Middle column size		Reinforcement ratio at the middle joint ^a		Detailing
		Depth	Width	Depth	Width	Top	Bottom	
Prototype 1	5500	500	300	500	500	0.85% (2T20 + 1T25)	0.48% (2T20)	Seismic
Specimen S1	2750	250	150	250	250	0.90% (1T13 + 2T10)	0.49% (2T10)	Seismic
Prototype 2	5500	500	300	500	500	0.71% (3T20)	0.48% (2T20)	Non-seismic
Specimen S2	2750	250	150	250	250	0.73% (3T10)	0.49% (2T10)	Non-seismic

^a Concrete covers of prototypes and specimens are 40 mm and 20 mm, respectively; reinforcement ratio is calculated by $\rho = A_s/bd$, in which b and d are the width and the effective depth of beam sections.

Table 2: Geometric properties of prototype frames and specimens[6]

Tested items		Nominal diameter (mm)	Elastic modulus (MPa)	Yield strength (MPa)	Ultimate strength (MPa)	Fracture strain (%) ^a	Bar type
Longitudinal reinforcing bars	T10	10	182,611	511	731	12.32	Deformed
	T13	13	185,763	527	640	10.76	Deformed
Stirrups	R6	6	178,500	310	422	14.00	Smooth
Concrete (150 mm (dia.) × 300 mm (height))			Compressive strength: 31.2 MPa Splitting tensile strength: 3.2 MPa Initial modulus of elasticity: 27,663 MPa				

^a All reinforcement strength is based on the nominal diameter of reinforcement and the gage length of reinforcement is 200 mm.

Table 3: Material properties of reinforcement and concrete [6]

The specimens were loaded quasi-statically by imposing vertical displacement at the middle joint, which represented the point where the column had been removed. The boundary conditions were selected to allow the development of axial forces in the beams, which is essential for observing compressive arch action and catenary action. The instrumentation included displacement transducers, load cells, and strain gauges on reinforcement, allowing the authors to monitor not only the global load-displacement response but also the development of horizontal reactions and reinforcement strain localization.

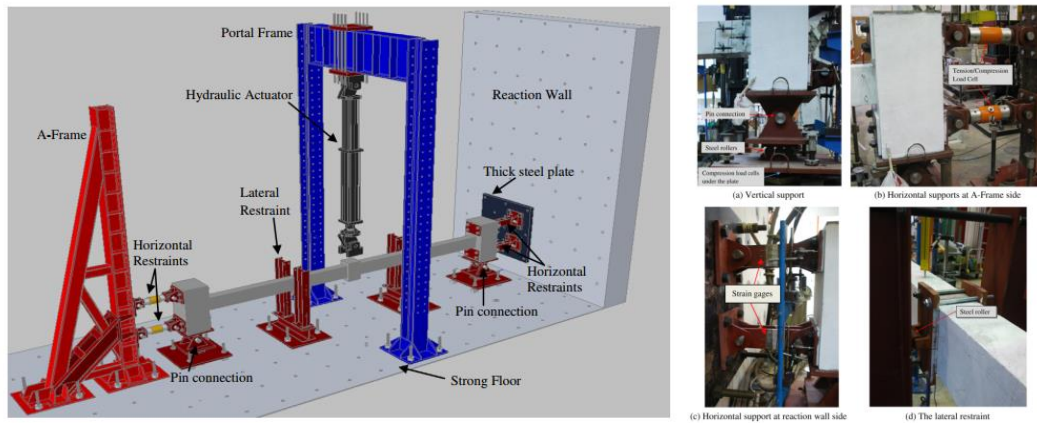


Figure 6: Test set-up, Details of test set-up [6]

Both specimens developed the typical sequence of mechanisms. At the beginning of the test, the response was governed by flexural behavior. As the middle joint displacement increased, the beams attempted to elongate, and the end restraints generated axial compression. This produced compressive arch action, increasing the load above the pure flexural capacity. At still larger displacements, the axial force changed from compression to tension and the longitudinal reinforcement became the main load-carrying component, producing catenary action.

Specimen	Critical displacement (mm)					Critical capacity (kN)		
	At the capacity of CAA ^a	Resistance reversing to rise	1st Rebar fracture ^b	2nd Rebar fracture ^b	Top rebar fracture at beam end	Calculated capacity of FA ^c	Capacity of CAA	Catenary action capacity
S1	78	246	227	246	573	33.08	41.64	68.91
S2	73	255	379	N.A.	612	29.02	38.38	67.63

^a CAA means compressive arch action.

^b The rebars mentioned are located at the bottom layer of the middle joint.

^c FA denotes flexural action.

Table 4: Force and displacement at critical pints of load-MJD curves [6]

The reported quantitative results are particularly useful for later comparison. Specimen S1 reached approximately 41.64 kN during the compressive arching stage and approximately 68.91 kN in the catenary stage. Specimen S2 reached approximately 38.38 kN during compressive arching and approximately 67.63 kN in catenary action. The similarity of the final catenary resistance is significant because it shows that the global response was not dominated only by nominal seismic detailing. Instead, continuity, anchorage, bond behavior, and local bar fracture controlled the large-deformation response.

An important observation is that damage localized primarily near the beam-joint interfaces and beam ends. This localization was associated with bond-slip, anchorage degradation, crack widening, and reinforcement fracture. The tests stopped when reinforcement fracture became critical. The results therefore demonstrate that the global load-displacement curve must be interpreted together with the local damage pattern. A high global load capacity is meaningful only if the local joint and anchorage regions can sustain the strain demand required to activate catenary action.

The numerical part of the study is also relevant for this thesis because Yu and Tan introduced a component-based joint model with springs representing load transfer and bond-slip effects. This approach showed that conventional beam elements alone are insufficient if the objective is to reproduce the large tensile demand and local slip at the joint. For later ATENA modeling, this point is important: the model must be able to represent not only concrete cracking and reinforcement yielding but also the influence of bond, anchorage, and strain localization.

For the experimental database, Yu and Tan (2011) should therefore be understood as a baseline study. It confirms the basic mechanism sequence and provides quantitative values for flexural, CAA, and CA stages. At the same time, it warns that robustness cannot be evaluated through flexural strength alone. The catenary stage is controlled by a different set of parameters, especially reinforcement continuity and joint behavior.

Specimen	Configuration	Main detailing feature	Key reported resistance	Database interpretation
S1	Half-scale beam-column sub-assembly	Seismic detailing	CAA about 41.64 kN; CA about 68.91 kN	Damage near beam-joint interfaces; reinforcement fracture controlled final stage
S2	Half-scale beam-column sub-assembly	Non-seismic detailing	CAA about 38.38 kN; CA about 67.63 kN	Similar final CA resistance; local anchorage and bond behavior critical

Table 5: Yu & Tan (2011) samples overview

2.5 Yu and Tan (2013): effect of reinforcement ratio and span-to-depth ratio

Yu and Tan (2013) extended the previous work by testing six additional half-scale RC beam-column sub-assemblages, designated S3 to S8, under a middle-column removal scenario. This study is central to the database because it provides a controlled parametric investigation of top reinforcement ratio, bottom reinforcement ratio, and span-to-depth ratio. Unlike the 2011 work, which mainly compared seismic and non-seismic detailing, the 2013 study clarifies how geometry and reinforcement quantity affect the relative contributions of flexural action, compressive arch action, and catenary action.

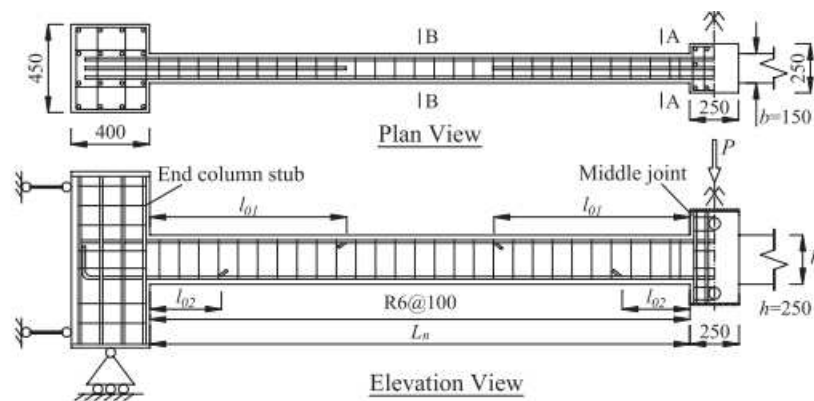


Figure 7: Detailing and boundary conditions of the specimens (unit: mm) [7]

The specimens maintained the same general sub-assembly concept: two beam spans framing into a central joint representing the removed column. The beam section was kept constant at 150 mm by 250 mm, with an effective depth of approximately 215 mm and a concrete cover of 20 mm. The net span was 2750 mm for specimens S3 to S6, 2150 mm for S7, and 1550 mm for S8. This produced span-to-depth ratios of approximately 23, 18.2,

and 13.4. The different span-to-depth ratios are important because they directly affect axial stiffness and deformation capacity.

Test specimen ^a	L_n (mm)	L/h	Position of curtailment (mm)		Longitudinal reinforcement ^c				Bottom bars at the middle joints ^d
					A-A section		B-B section		
			l_{01}	l_{02} ^b	Top	Bottom	Top	Bottom	
S1-0.90/0.49/23S	2,750	23	1,000	NA	1T13 + 2T10 (0.90%)	2T10 (0.49%)	2T10 (0.49%)	2T10 (0.49%)	Continuous
S2-0.73/0.49/23	2,750	23	925	NA	3T10 (0.73%)	2T10 (0.49%)	2T10 (0.49%)	2T10 (0.49%)	Lap splice
S3-1.24/0.49/23	2,750	23	1,000	345	3T13 (1.24%)	2T10 (0.49%)	2T13 (0.82%)	2T10 + 1T13 (0.90%)	Lap splice
S4-1.24/0.82/23	2,750	23	1,000	NA	3T13 (1.24%)	2T13 (0.82%)	2T13 (0.82%)	2T13 (0.82%)	Continuous
S5-1.24/1.24/23	2,750	23	1,000	NA	3T13 (1.24%)	3T13 (1.24%)	2T13 (0.82%)	3T13 (1.24%)	Continuous
S6-1.87/0.82/23	2,750	23	1,000	345	3T16 (1.87%)	2T13 (0.82%)	2T16 (1.25%)	2T13 (0.82%)	Continuous
S7-1.24/0.82/18.2	2,150	18.2	780	NA	3T13 (1.24%)	2T13 (0.82%)	2T13 (0.82%)	2T13 (0.82%)	Continuous
S8-1.24/0.82/13.4	1,550	13.4	560	NA	3T13 (1.24%)	2T13 (0.82%)	2T13 (0.82%)	2T13 (0.82%)	Continuous

Note: The beam sections are 150-mm wide and 250-mm deep for all specimens (i.e., $b = 150$ mm and $h = 250$ mm); the concrete cover thickness is 20 mm for all specimens; total net span $L = 2L_n + 250$ (mm); and test results of Specimens S1 and S2 are shown in Yu and Tan (2011).

^aThe seismic detailing in terms of the arrangement of the stirrups is indicated by S.

^bNA = not available.

^cReinforcement ratio in brackets is calculated by A_s/bd , where $b = 150$ mm and $d = 215$ mm; the A-A and B-B sections are given in Fig. 1.

^dThe lap-splice lengths for the bottom bars of Specimens S3 and S6 are 410 and 530 mm, respectively. The lap-spliced position is through the middle joint.

Table 6: Geometric properties of sub-assembly specimens [7]

The notation used for the specimens is linked to the reinforcement ratios and geometry. For example, specimens S3 to S6 were mainly used to investigate reinforcement ratio effects, whereas S7 and S8 were used to study the influence of span-to-depth ratio. S3 had a top reinforcement ratio of 1.24% and a bottom reinforcement ratio of 0.49%; S4 had 1.24% top and 0.82% bottom; S5 had 1.24% top and 1.24% bottom; S6 had 1.87% top and 0.82% bottom; S7 and S8 both had 1.24% top and 0.82% bottom but different span-to-depth ratios.

Bar type ^a	Yield strength f_y (MPa)	Elastic modulus E_s (MPa)	Strain at		Tensile strength f_u (MPa)	Ultimate strain ϵ_u (%)
			the start of hardening ϵ_{sh} (%)			
R6	349	199,177	—		459	—
T10	511	211,020	2.51		622	11.00
T13	494	185,873	2.66		593	10.92
T16	513	184,423	2.87		612	13.43

^aFor the bar type, R and T = low-yield and high-yield strength reinforcements, with nominal yield strengths of 250 and 460 MPa, respectively; the number following R or T represents the nominal diameter of a bar.

Table 7: Material properties of steel reinforcement [7]

The material properties reported in the study provide useful values for later modeling. These values are important because catenary action depends on reinforcement strength and ductility.

The experimental results showed that compressive arch action exceeded flexural capacity in all specimens that did not fail prematurely in shear. This means that the presence of axial

restraint can significantly increase the peak load even before catenary action becomes active. However, the effectiveness of CAA was not uniform. It was more pronounced in specimens with smaller span-to-depth ratios because shorter beams have greater axial stiffness and are more able to mobilize compression under restrained elongation. Conversely, larger span-to-depth ratios allowed greater deformation and therefore favored the development of catenary action at later stages.

The flexural capacities reported by the authors show clear parameter effects. The flexural resistance increased from 40.89 kN in S3 to 61.97 kN in S6, reflecting the influence of reinforcement ratio, particularly top reinforcement near critical sections. The CAA peak loads were also substantial, reaching values such as 54.47 kN for S3, 63.22 kN for S4, 70.33 kN for S5 and S6, 82.82 kN for S7, and 121.34 kN for S8. The high value for S8 reflects the strong arching effect associated with the short span-to-depth ratio, although this specimen also demonstrated the risk of shear-governed behavior.

Specimen	Flexural action	Compressive arch action					
	[calculated capacity P_f^a (kN)]	Peak load P_{CAA}^b (kN)	MJD at P_{CAA} (mm)	Horizontal reaction at P_{CAA} (kN)	Max. compressive horizontal reaction N_{max} (kN)	MJD at N_{max} (mm)	Enhancement factor α (%) ^c
S1	33.08	41.64	78	-165.3	177.90	114.3	25.9
S2	29.02	38.38	73	-145.7	155.90	128.3	32.2
S3	40.89	54.47	74.4	-196.40	221.00	100.1	33.2
S4	47.76	63.22	81.0	-183.10	212.65	107.2	32.4
S5	56.61	70.33	74.5	-207.00	238.35	108.7	24.2
S6	61.97	70.33	114.4	-218.10	218.10	114.4	13.5
S7	61.09	82.82	74.4	-220.00	233.10	87.0	35.6
S8	84.74	121.34	45.9	-264.50	272.50	55.0	43.2

^a P_f is calculated based on the conventional plastic hinge mechanism.

^b P_{CAA} corresponds to the first peak load in the load-MJD curves; i.e., the CAA capacity.

^cEnhancement factor $\alpha = (P_{CAA} - P_f)/P_f$.

Table 8: Experimental results at the flexural and CAA stages [7]

The catenary stage provided an even more important insight. In most specimens, catenary action exceeded the CAA peak significantly, but this was not true for all cases. Specimen S8, despite having a high CAA peak, did not develop a favorable catenary response because shear failure limited the structural capacity. This observation is critical for design interpretation. Catenary action cannot be assumed to guarantee robustness if the member does not first satisfy adequate shear capacity and local ductility requirements.

Specimen	MJD at the start of catenary action (mm)	Load at first fracture of bottom bars, P_{f1} (kN)	MJD at P_{f1} (mm)	Load at second fracture of bottom bars, P_{f2} (kN)	MJD at P_{f2} (mm)	Load at fracture of top bars, P_{f3} (kN) ^a	MJD at P_{f3} (mm)	P_{f3}/P_{CAA} ^b
S1	307.1	25.91	227.0	—	—	68.91	573.0	1.65
S2	294.4	39.1	378.7	—	—	67.63	612.0	1.76
S3	254.1	41.26	140.1	50.35	367.2	124.37	729.3	2.28
S4	261.6	59.18	283.0	83.24	449.6	103.68	614.3	1.64
S5	290.3	94.86	433.9	94.58	530.3	105.07	665.9	1.49
S6	248.7	93.03	337.1	96.82	438.4	139.90	675.3	1.99
S7	276.7	65.43	254.1	68.43	378.2	105.99	628.5	1.28
S8	136.0	83.53	256.6	70.62	351.0	59.58	504.1	0.49

^aExcept for Specimens S6 and S8, P_{f3} of the other specimens represents the capacity of catenary action; the catenary action capacities of Specimens S6 and S8 are 143.28 and 91.83 kN, respectively.

^b P_{CAA} denotes the structural capacity of CAA.

Table 9: Experimental results at the Catenary action stage [7]

The authors proposed a useful deformation-based interpretation, noting that catenary action becomes relevant when the middle joint displacement approaches the beam depth and that a displacement of about 10% of the total span can be used as a conservative reference for catenary capacity. This criterion is not a universal design rule, but it provides an experimentally grounded benchmark. For this thesis, it is useful when interpreting later load-displacement curves and when distinguishing between pre-catenary and catenary-dominated response.

In database terms, Yu and Tan (2013) provides the most systematic evidence on how reinforcement ratio and span-to-depth ratio influence progressive collapse resistance. The study demonstrates that compressive arch action and catenary action are complementary but not interchangeable. Shorter beams and stronger axial restraint improve arching, whereas large deformation capacity and reinforcement ductility govern catenary action. The failure of S8 also proves that shear capacity and local failure control must be included in any reliable robustness assessment.

Specimen	TRR/BRR/L:h	Main variable	Key reported values	Interpretation
S3	1.24/0.49/23	Baseline high span-to-depth ratio with low bottom reinforcement	Flexural 40.89 kN; CAA 54.47 kN	Useful for low BRR behavior
S4	1.24/0.82/23	Increased bottom reinforcement	Flexural 47.76 kN; CAA 63.22 kN	Improved flexural and arching stage
S5	1.24/1.24/23	High bottom reinforcement	Flexural 56.61 kN; CAA 70.33 kN	Higher beam mechanism resistance
S6	1.87/0.82/23	High top reinforcement	Flexural 61.97 kN; CAA 70.33 kN	Strong catenary contribution

			kN; CA about 143.28 kN	
S7	1.24/0.82/18.2	Reduced span-to- depth ratio	Flexural 61.09 kN; CAA 82.82 kN	Stronger arching with adequate deformation
S8	1.24/0.82/13.4	Short span-to- depth ratio	Flexural 84.74 kN; CAA 121.34 kN; CA about 91.83 kN	High CAA but shear-sensitive; CA limited

Table 10: Yu & Tan (2013) samples overview

2.6 Qian et al. (2022): influence of loading regime on column-loss behavior

Qian et al. (2022) address a problem that is especially important for interpreting quasi-static column-removal tests: the loading regime. Many experimental studies simulate column removal by applying a concentrated vertical load or displacement at the position of the removed column. This concentrated loading regime is convenient and allows the complete load-displacement curve to be obtained, but it does not fully reproduce the real load history of a building. In an actual structure, beams carry gravity loads before the column is lost, and those loads influence moment distribution, crack formation, and the location of critical sections.

To investigate this issue, Qian et al. compared two loading regimes. In the concentrated loading regime, designated CL, the middle column support was removed first and the specimen was then loaded vertically at the middle column location. In the sequential loading regime, designated SL, distributed gravity loads were first applied along the beams using suspended weights; then the middle column support was gradually removed; if the specimen did not collapse, additional concentrated loading was applied to reach the ultimate state. This sequential procedure was intended to approximate the real condition in which gravity loads exist before local support loss.

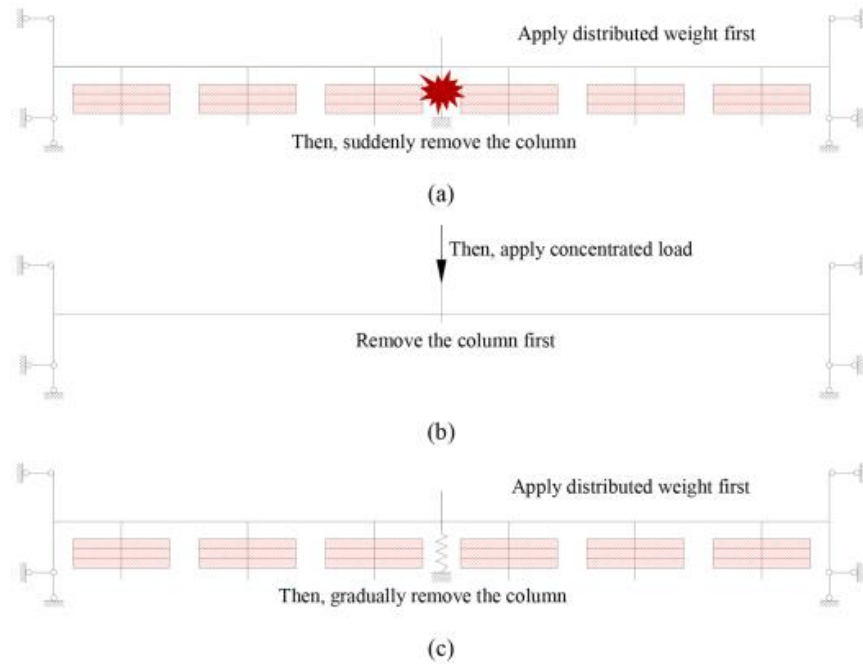


Figure 8: Loading regimes for disproportionate collapse studies: a) dynamic loading regime b) Concentrated loading regime c) Sequential loading regime [11]

The experimental program consisted of two series of half-scale RC beam-column sub-assemblages. The specimens were named CL-13, CL-11, CL-8 and SL-13, SL-11, SL-8, where the number indicates the span-to-depth ratio. All specimens had the same beam section of 250 mm by 150 mm and the same middle column section of 250 mm by 250 mm. The longitudinal reinforcement was based on T12 deformed bars and the transverse reinforcement on R6 plain bars. The specimens were extracted conceptually from a prototype eight-storey non-seismically designed RC frame and included non-seismic reinforcement curtailment, which allowed realistic local failure development.

Test ID	Beam clear span (mm)	Beam longitudinal reinforcement				Loading regime
		A-A section		B-B section		
		Top	Bottom	Top	Bottom	
CL-13	3250	3T12	2T12	2T12	2T12	Concentrated loading
SL-13	3250	3T12	2T12	2T12	2T12	Sequential loading
CL-11	2750	3T12	2T12	2T12	2T12	Concentrated loading
SL-11	2750	3T12	2T12	2T12	2T12	Sequential loading
CL-8	2000	3T12	2T12	2T12	2T12	Concentrated loading
SL-8	2000	3T12	2T12	2T12	2T12	Sequential loading

Table 11: Specimens properties [11]

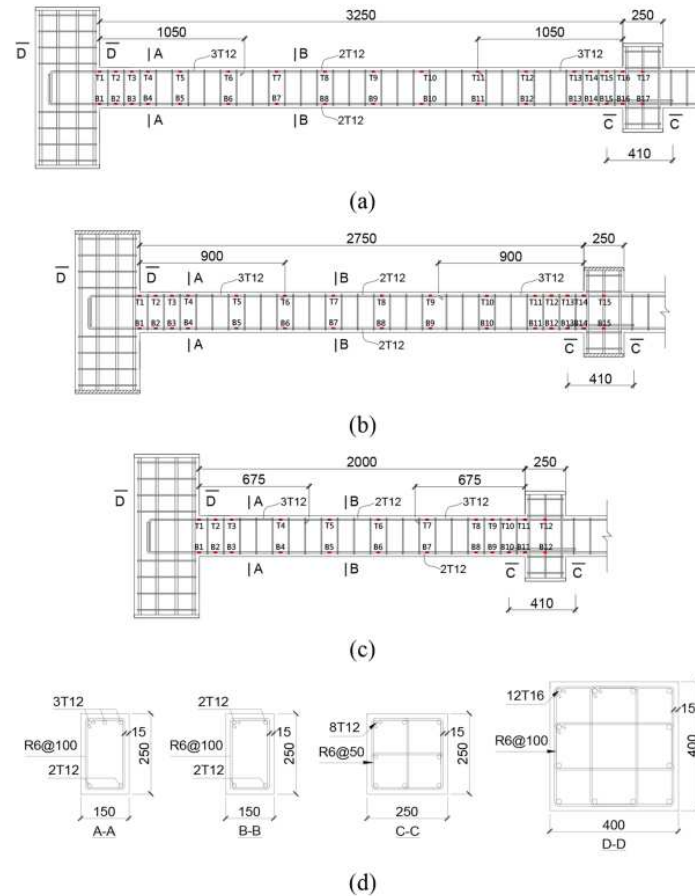


Figure 9: Dimensions and reinforcement detailing of specimens a) CL/SL-13 b) CL/SL-11 c) CL/SL-8
[11]

Items		Nominal diameter (mm)	Yield strength (MPa)	Ultimate strength (MPa)	Elongation (%)
Longitudinal rebar	T12	12	438	577	16.6
Transverse links	R6	6	348	486	25.4

Note: R6 represents plain bar with diameter of 6 mm; T12 represents deformed rebar with diameter of 12 mm.

Table 12: Material properties of reinforcements [11]

The most important result is that the CL and SL regimes produced similar yield strengths and first peak strengths for specimens with the same span-to-depth ratio, but they produced different ultimate strengths, deformation capacities, critical sections, and failure modes. This means that the early flexural and compressive arching stages may be reasonably captured by concentrated loading, but the catenary stage can be misrepresented if the actual gravity load history is ignored.

Test ID	Critical displacements (mm)			Critical loads (kN)			MHCF (kN)	MHTF (kN)	UR (rad)	Lateral stiffness (kN/m)
	YS	FPS	US	YS	FPS	US				
CL-13	45	108	731	33	43	81+	-153	148	0.23	1.0×10^5
SL-13-Shift	43	100	550	31	39	63	-154	111	0.17	
CL-11	36	90	712	37	52	94	-178	154	0.26	
SL-11-Shift	35	90	593	36	49	85	-167	167	0.22	
CL-8	25	79	551	53	77	88	-202	147	0.28	
SL-8-Shift	23	80	357	54	74	63	-224	110	0.18	

Note: YS means yield strength; FPS represents first peak strength; US represents ultimate strength; MHCF means maximum horizontal compressive force; MHTF means maximum horizontal tensile force; and UR indicates ultimate rotation.

Table 13: Test results [11]

The quantitative differences are clear. For CL-13 and SL-13, the yield strengths were approximately 33 kN and 31 kN, while the first peak strengths were approximately 43 kN and 39 kN. However, the ultimate strength reached at least 81 kN for CL-13 but only about 63 kN for SL-13. For CL-11 and SL-11, the ultimate strengths were about 94 kN and 85 kN, respectively. For CL-8 and SL-8, the ultimate strengths were about 88 kN and 63 kN. These results show that concentrated loading can significantly overestimate deformation and ultimate catenary resistance, especially for the shorter specimens.

The failure location is even more important than the peak value. Under the CL regime, the first reinforcement fracture occurred at the beam end near the middle column (BENM). Under the SL regime, the critical fracture shifted to the beam end near the side column (BENS). This shift is caused by the different bending moment distribution produced by the initial gravity loading. Therefore, a model calibrated only against CL tests may identify the wrong critical region when applied to a real building with pre-existing gravity loads.

The horizontal reaction results show that both regimes were able to develop compressive force during the arching stage and tensile force during the catenary stage. However, comparable horizontal forces did not imply comparable vertical load capacity. In the SL tests, early fracture and unfavorable rotation geometry reduced the vertical component of the tensile force. This confirms that catenary resistance depends not only on axial tension but also on the geometry of the deformed shape, the fracture location, and the load history that produces the final configuration.

For the database, Qian et al. (2022) provides a necessary correction to the interpretation of older quasi-static tests. The paper does not invalidate concentrated loading tests, but it shows their limitation. CL tests are useful for identifying the basic response sequence and for obtaining complete load-displacement curves, while SL tests better represent the internal force history of real structural systems. Later numerical models should therefore consider whether the loading protocol used for calibration is consistent with the structural scenario being simulated.

Pair	Span-to-depth ratio	YS CL/SL (kN)	FPS CL/SL (kN)	US CL/SL (kN)	Main interpretation
CL-13 / SL-13	13	33 / 31	43 / 39	81+ / 63	CL overestimates CA capacity and deformation; SL shifts damage toward side column
CL-11 / SL-11	11	37 / 36	52 / 49	94 / 85	Similar early response; lower SL catenary capacity
CL-8 / SL-8	8	53 / 54	77 / 74	88 / 63	Short span gives high early capacity but SL ultimate is strongly reduced

Table 14: Qian et al. samples overview

2.7 Farhang Vesali et al. (2013): arching action in longitudinally restrained RC beams

The study focuses on compressive membrane or arching action in longitudinally restrained RC beams. Although the specimens are not full beam-column sub-assemblages, the paper is highly relevant because it isolates one of the main mechanisms that increases resistance after column loss. In a frame, when a beam deflects after support loss, cracking and curvature tend to produce axial elongation. If surrounding structural elements restrain this elongation, a compressive thrust develops in the beam. This thrust can substantially increase load capacity before the large-deformation catenary stage begins.

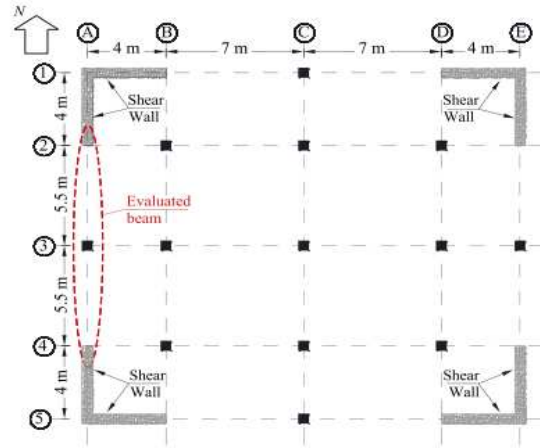


Figure 10: Outline of floor plan [12]

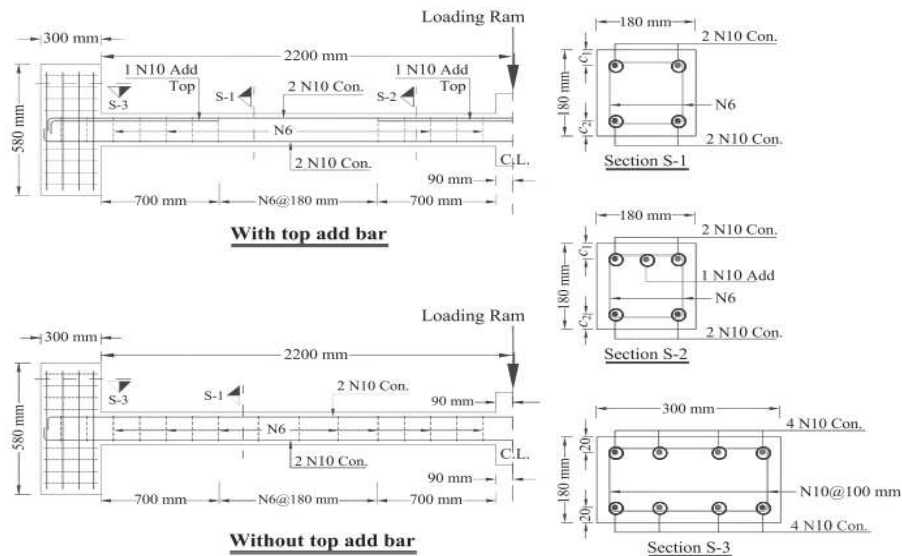


Figure 11: Outline of geometry. Cross-section and reinforcing details for the beam assemblages. [12]

The experimental program used six 2/5-scale beam assemblages with end restraints. The specimens represented an exterior beam of a braced RC frame after loss of a middle column at ground floor level. The tests were conducted under monotonically increasing mid-span displacement at a loading rate of 0.1 mm/s. The support system was designed to provide strong axial restraint through grouted gaps and Macalloy bars (20 mm), while rotational restraint was provided by bolted double C-channels and end supporting blocks.

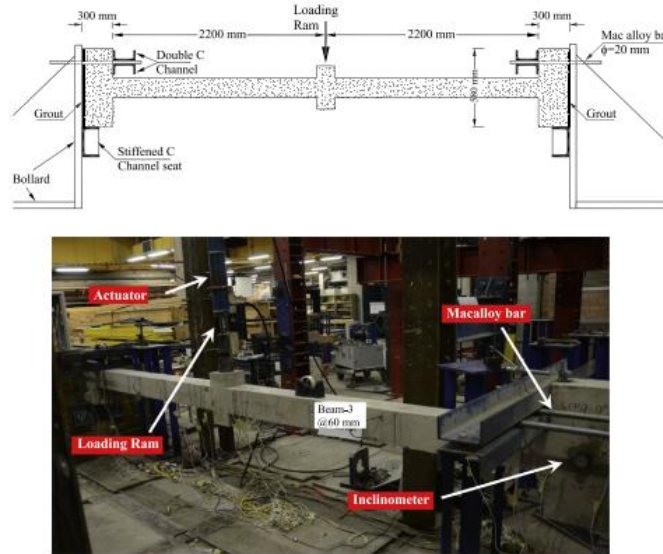


Figure 12: Experimental set up for the reinforced concrete assemblages. [12]

The main variables were the longitudinal reinforcement configuration and the stirrup spacing over the end third of the span. Some specimens had an additional top bar, while others did not. The stirrup spacing over the end region varied between 180 mm, 120 mm, and 60 mm, while the middle third of the span had a constant spacing of 180 mm. This variable selection was intended to determine whether increased longitudinal reinforcement or improved passive confinement would substantially modify the arching response.

Specimen No.	f_{cm} (MPa) ^a	Top add bar	Spacing of stirrups ^b (mm)		Concrete cover		Adjusted ultimate strain of embedded steel bars $\epsilon_{u,sl}$ (viz. Fig. 14b)
			End 1/3 span	Middle 1/3 span	c_1 (mm)	c_2 (mm)	
1	30.5	No	180	180	31	31	0.034
2	27.0	No	120	180	35	35	0.033
3	30.0	No	60	180	31	31	0.034
4	26.0	Yes	180	180	31	31	0.036
5	29.5	Yes	120	180	35	35	0.032
6	30.0	Yes	60	180	35	35	0.032

^a Average compressive strength of concrete at the time of testing (average of three 300 mm by 150 mm diameter cylinders). Further, the cracking (tensile) strength of concrete is obtained from $f_t = 0.6\sqrt{f_{cm}}$.

^b Diameter of the stirrups is $d_b = 6$ mm.

Table 15: Details of the longitudinal reinforcement, spacing of stirrups, concrete cover on top and bottom reinforcement and adjusted ultimate strain of embedded steel bars. [12]

The material properties and test observations show that concrete behavior was decisive. The reported concrete compressive strengths at testing ranged from approximately 26.0 MPa to 30.5 MPa. The longitudinal reinforcement was made of 10 mm cold-rolled ribbed bars with nominal grade 500 MPa, while the stirrups were 6 mm round bars with nominal grade 250 MPa.

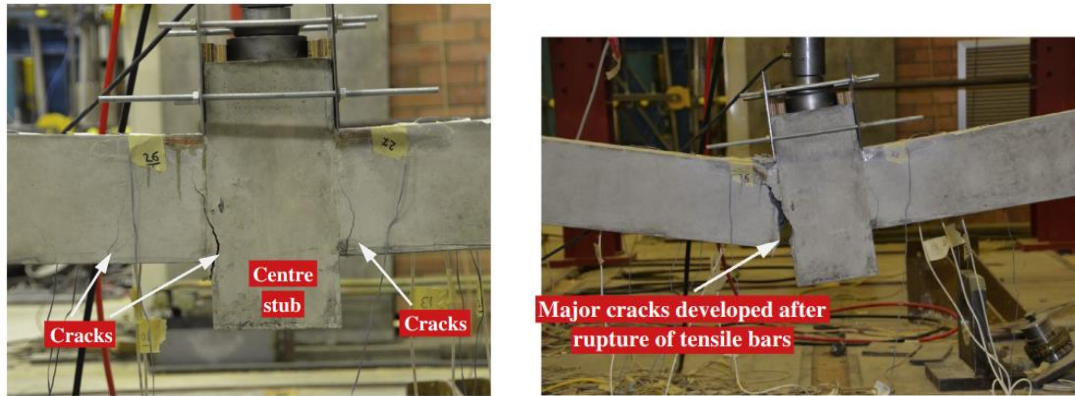


Figure 13: Outline of the cracks developed at sections adjacent to the center stub. [12]

The specimens exhibited first major cracks at sections adjacent to the center stub, and first rupture of tensile bars occurred at similar locations. The first bar rupture typically occurred at middle-stub displacements of about 74 to 91 mm.

The load-displacement curves showed peak loads generally within the mid-30 kN to low-40 kN range. More importantly, the authors concluded that the longitudinal reinforcement ratio and the stirrup configuration had only minor influence on the peak arching capacity when the member was under-reinforced and brittle shear failure was avoided. In contrast, concrete compressive strength and strain penetration had a stronger influence on both peak load and post-peak behavior. This finding is important because it separates arching capacity from catenary capacity. More reinforcement does not automatically mean a higher CAA peak if the dominant mechanism is controlled by concrete compression and restraint conditions.

Importantly, strain gauges within the anchored length show tensile strains exceeding yield strain, and the authors interpret this as evidence of **slip along anchored length** and **strain penetration** into the beam end support.

Concrete strain results indicate crushing occurs over an approximate length of ~200 mm, consistent with plastic hinge size. Confinement effects from end supporting blocks and center stub influence strain readings and ductility locally.

A major contribution of the study is the treatment of support flexibility and strain penetration. The authors emphasized that experimental fixed supports are rarely ideal. Connection looseness, reaction frame compliance, and anchorage slip can modify the actual boundary condition. For this reason, end support rotations were measured using inclinometers and then used to calibrate a modified 1D finite element model. The model included rotational springs representing both end-support rotation and strain penetration at the beam-to-support connection.

The model assigns two rotational springs at each node:

1. one for strain penetration effects (fixed-end rotation),
2. one for end-support rotation measured in tests.

A bilinear moment–rotation relationship is used for springs representing strain penetration (Fig. x b). Yield rotation is calculated using Eq. (1):

Equation 1: Yield Rotation

$$\theta_y = \frac{S_y}{d - x}$$

where S_y is bar slip at yield, and $d - x$ is effective lever arm (definitions per text). Bar slip S_y is computed using Eq. (2), which depends on bar diameter, steel yield strength, concrete compressive strength, and a bond parameter a (taken as 0.4 per CEB-FIP 1990).

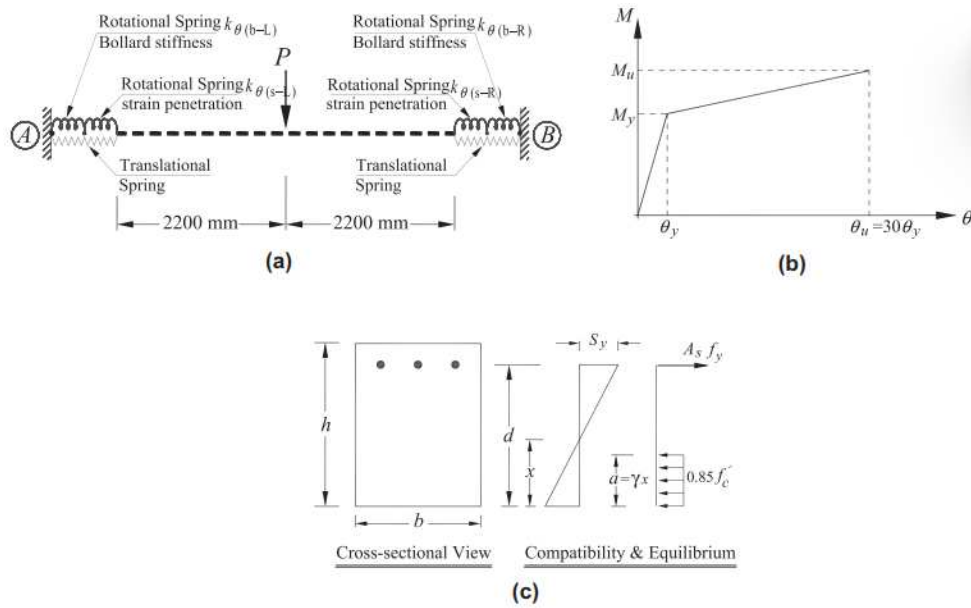


Figure 14: Outline of the (a) generic model adopted in the numerical study [1], (b) bending moment-rotation relationship for nodal springs capturing strain penetration and (c) procedure for calculating the stiffness of rotational springs. [12]

End-support rotational stiffness is calibrated using the measured rotation curves through an iterative procedure. The resulting average rotational stiffness values are reported in Table 16.

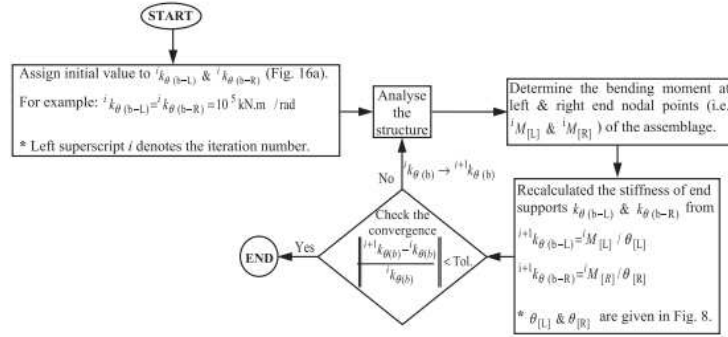


Figure 15: Outline of the iterative procedure adopted for calibrating the stiffness of rotational springs which represent end supports. [12]

Specimen No.	Rotational stiffness of nodal springs (kN m/rad)			Peak Load (kN)			
	From strain penetration model		$k_{\theta(y)}$ of end supports calibrated against measured rotations of end blocks given in Fig. 8 ^a	Test	1D FE model		
	M_y (kN m)	θ_y (rad)			W/strain penetration	W/O strain penetration	Ideally fully fixed support
1	13.0	0.0031	1.45×10^4	40.5	40.1	44.2	45.2
2	12.5	0.0033	1.35×10^4	35.7	36.5	39.9	40.8
3	13.1	0.0031	1.85×10^4	41.4	41.1	44.3	45.1
4	18.5	0.0037	1.80×10^4	40.1	40.5	44.6	44.9
5	18.8	0.0035	1.60×10^4	41.6	41.5	44.9	45.1
6	18.2	0.0037	1.45×10^4	39.4	40.7	44.7	45.1

Table 16: Rotational stiffness of end supports and peak loading capacity of the specimens [12]

The modified 1D FE model reproduces the load–displacement response well.

Table 16 shows peak load comparisons, where the calibrated model generally matches test peaks closely. For example:

- Specimen 1: Test peak 40.5 kN, 1D FE (with strain penetration) 40.1 kN.
- Specimen 2: Test 35.7 kN, 1D FE 36.5 kN.
- Specimen 3: Test 41.4 kN, 1D FE 41.1 kN.

When strain penetration is neglected, the model overestimates initial stiffness and can overpredict peak capacity by up to 11%. The “ideally fully fixed support” assumption also slightly increases peak predictions relative to the calibrated support case.

The numerical model reproduced the experimental load-displacement response with good accuracy when strain penetration and measured support rotation were included. When strain penetration was neglected, the model overestimated initial stiffness and could overpredict peak load by up to about 11%. This observation is directly relevant to later ATENA simulations. If support flexibility, anchorage slip, or strain penetration are ignored, the model may appear stiffer and stronger than the real specimen, especially during the arching stage.

For the database, FarhangVesali et al. provide an important mechanism-level reference. The study does not primarily address catenary action, but it clarifies the conditions under which compressive arch action develops. It also shows that the boundary system is not a secondary detail; it is part of the structural mechanism. Therefore, any comparison between experimental tests or numerical models must carefully distinguish between nominal boundary conditions and measured effective boundary conditions.

Specimen	Top Add Bar	Stirrup Spacing (End 1/3 Span)	Measured Concrete Strength f_{cm} (MPa)	Peak Load (kN)	First Bar Rupture Displacement (mm)	Main Observations
1	No	180 mm	30.5	40.5	~80–90	Reference specimen without additional top reinforcement. Typical arching behavior observed with first tensile bar rupture near center stub.
2	No	120 mm	27.0	35.7	~74–85	Reduced concrete strength led to lower peak load. Deformation pattern similar to Specimen 1.
3	No	60 mm	30.0	41.4	~80–90	Closely spaced stirrups increased confinement but had little effect on arching capacity.
4	Yes	180 mm	26.0	34.5	~75–85	Additional top bar did not significantly increase peak load due to arching-controlled behavior.
5	Yes	120 mm	29.5	37.0	~80–90	Slight increase in peak capacity relative to Specimen 4 due to higher concrete strength.
6	Yes	60 mm	30.0	39.5	~85–91	Closest stirrup spacing produced slightly improved ductility but minimal change in arching strength.

Table 17: Farhang Vesali et al. samples overview

2.8 Sasani et al. (2011): reinforcement fracture and post-fracture catenary response

The study addresses a critical numerical and experimental issue: the explicit modeling of reinforcement fracture under large deformation. This issue is essential for progressive collapse analysis because bar fracture is a sudden local event that changes the resisting mechanism. In RC beams after column loss, fracture of bottom reinforcement may reduce flexural capacity, but it does not necessarily mean immediate collapse. If the structure is sufficiently continuous, the remaining reinforcement may still develop catenary action and provide additional resistance.

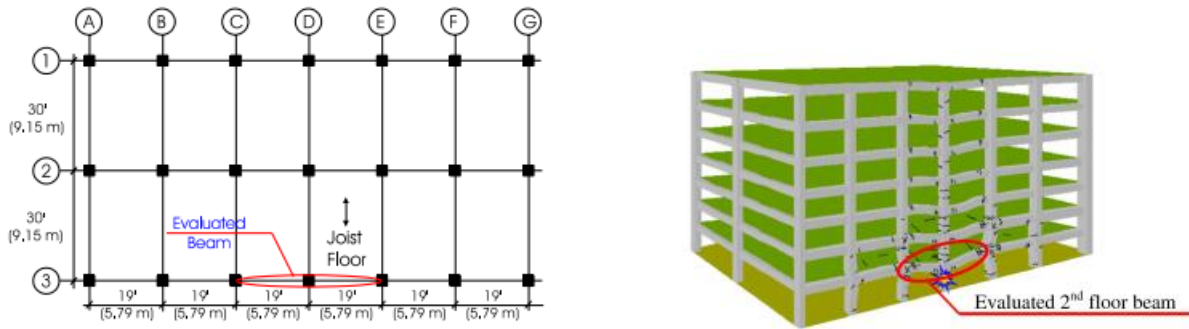


Figure 16: Typical floor plan & Removal of column due to explosion/impact [5]

The experimental specimen was derived from a seven-storey RC building designed according to ACI 318 requirements, including integrity reinforcement provisions. The specific member studied was the second-floor exterior beam spanning over a removed first-floor column. A 3/8-scale model of the two-span beam was constructed and tested under fixed boundary conditions. The vertical displacement was applied monotonically at the location representing the removed column. The displacement was applied slowly at first and then increased at a constant rate after initial deformation was established.

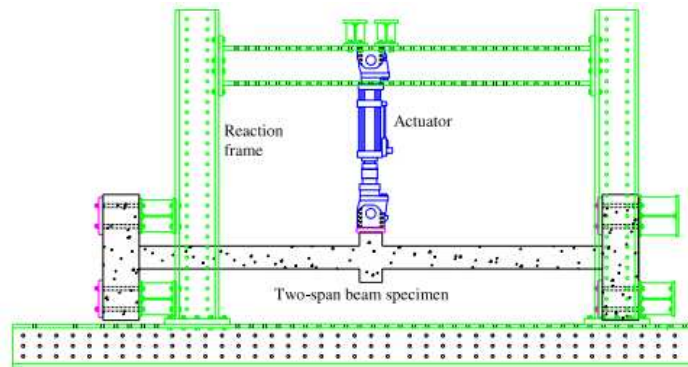


Figure 17: Specimen and loading setup [5]

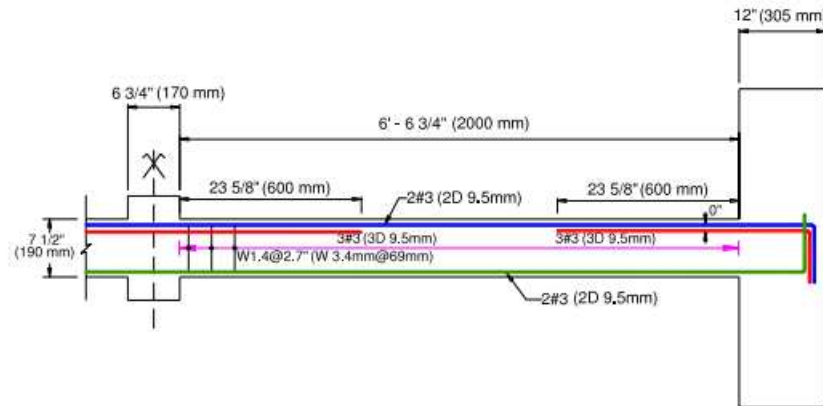


Figure 18: Reinforcement detailing of two-span exterior beam between axes C and E in second floor [5]

The material properties were experimentally determined. The longitudinal reinforcement had a yield stress of approximately 75 ksi, corresponding to about 516 MPa, and an ultimate stress of approximately 103 ksi, corresponding to about 709 MPa. The concrete compressive strength was approximately 6 ksi, or about 41 MPa, with a peak compressive strain close to 0.002. These values are relevant because the test focused on the interaction between concrete crushing, axial force development, and reinforcement fracture.

The force-displacement response showed an initial peak, a reduction in resistance, and then a later recovery associated with catenary action. The bottom reinforcement fractured at vertical displacements of approximately 6.1 in. (155 mm) and 7.5 in. (190 mm), near one side of the removed column. After bottom-bar fracture, the beam did not collapse immediately. Instead, top reinforcement began to carry tensile force and the structural response developed toward catenary action. This observation is fundamental because it shows that fracture of one set of bars is not necessarily the final limit state of the system.

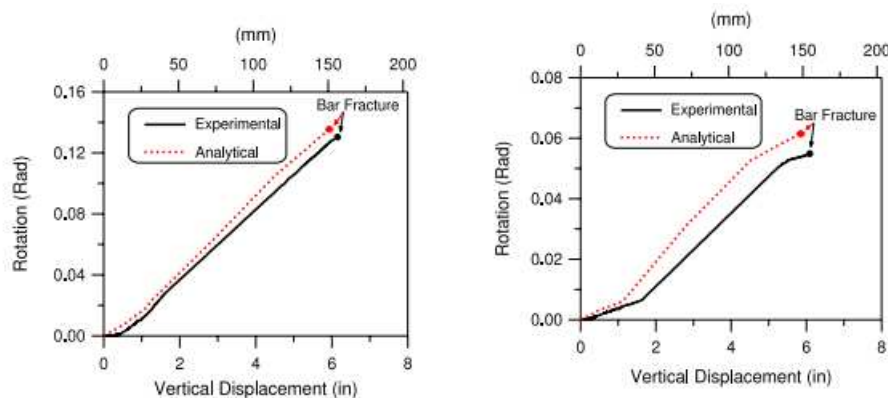


Figure 19: Beam rotations over 7.5 in. (190mm) from face of removed column and side support (column) [5]

The interpretation of the pre-fracture response is particularly useful. Initially, restrained beam elongation generated axial compression, which increased the flexural resistance through compressive arch action. As deformation increased, concrete crushing and mid-height strain changes reduced beam growth and decreased axial compression. This reduction in compression, combined with P-delta effects, led to the observed decrease in vertical resisting force. Therefore, the post-peak drop before bar fracture was not caused only by reinforcement behavior; it was the result of the interaction between material degradation, axial force evolution, and geometric nonlinearity.

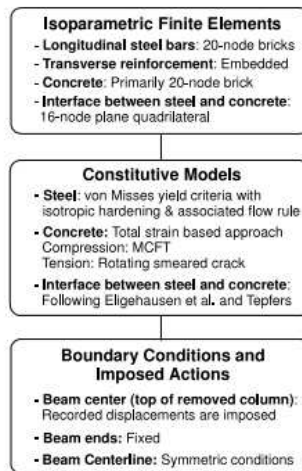


Figure 20: Finite element model chart[3]

The finite element model was developed in DIANA and included geometric nonlinearity through a total Lagrangian formulation. Concrete was modeled primarily using 20-node isoperimetric brick elements, while reinforcement was represented through compatible reinforcement elements. Steel-concrete interaction was explicitly modeled with zero-thickness interface elements in order to capture bond-slip. This level of detail was necessary because the study aimed to simulate local fracture and post-fracture redistribution.

Steel reinforcement is modeled using the von Mises yield criterion with isotropic hardening. A tri-linear stress-strain relationship is adopted for longitudinal bars based on experimental data, while transverse reinforcement is modeled using a bilinear idealization.

Concrete behavior is modeled using a total strain-based approach, consistent with the Modified Compression Field Theory (MCFT). Tensile cracking is represented using a rotating smeared crack model, which avoids spurious tensile stresses and improves numerical stability under large deformations. Concrete tensile strength is taken as 0.47 ksi (3.2 MPa).

Bond behavior between steel and concrete is explicitly modeled, accounting for chemical adhesion, mechanical interlock, and friction. A nonlinear bond stress–slip relationship is adopted, with post-peak softening to represent concrete crushing around reinforcement ribs.

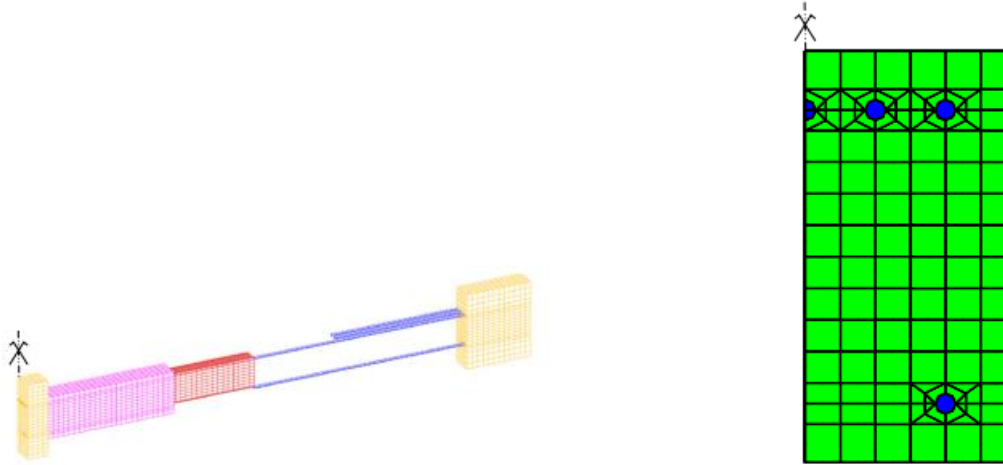


Figure 21: Finite element model of a quarter of two-span beam & Cross section discretization [5]

The beam is first analyzed under increasing displacement until the steel strain reaches the fracture limit. The analytical model predicts first bar fracture at a vertical displacement of approximately 5.9 in. (150 mm), which closely matches the experimental value.

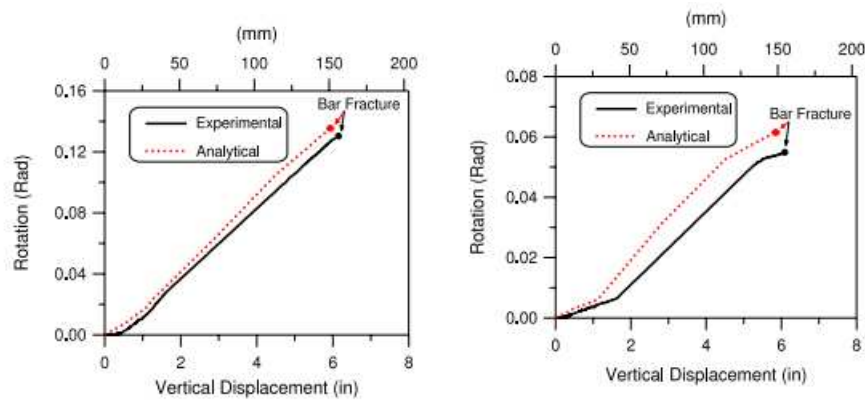


Figure 22: Beam rotations over 7.5 in. (190mm) from face of removed column and side support (column) [5]

Beam rotations are evaluated over a length equal to the beam depth (7.5 in. or 190 mm), and analytical results show good agreement with experimental measurements at both the removed column and the side support. Steel strain histories in both top and bottom reinforcement also correlate well with experimental observations.

The analytical model slightly overestimates initial stiffness and underestimates peak load by approximately 7–10%, but overall agreement with the experimental force–displacement curve is good.

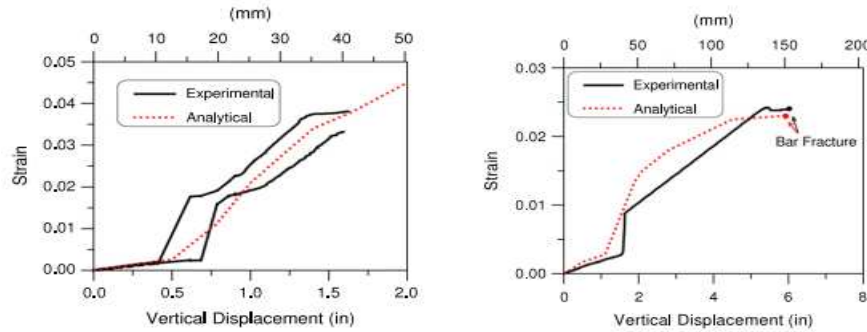


Figure 23: Bottom and Top steel bar tensile strains in vicinity of removed column and side support (column) [5]

The reduction in vertical resisting force after the peak is explained through the interaction of axial force development, beam growth, concrete crushing, and $P-\Delta$ effects. Initially, constrained beam elongation leads to axial compression, which enhances flexural strength. As deformation increases, concrete crushing reduces tensile strain at mid-height, limiting beam growth and causing axial compression to decrease. The combined reduction in $P-M$ interaction and adverse $P-\Delta$ effects leads to the observed drop in vertical resistance.

A major contribution of the paper is the proposed procedure for modeling bar fracture. The analysis proceeds up to the initiation of fracture, nodal forces at the fracture location are extracted, the steel and interface elements are separated at that location, and equal and opposite forces are applied to simulate the sudden release associated with fracture.

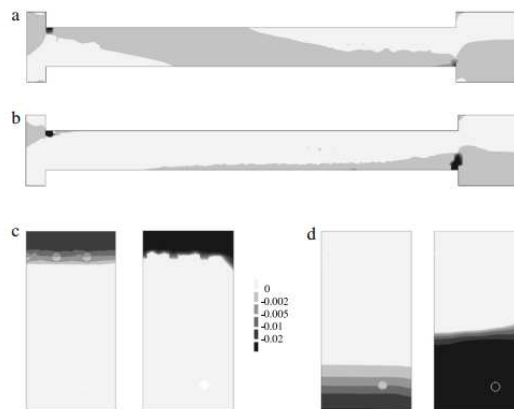


Figure 24: Longitudinal strains in beam center plane (a) at peak resisting force, and (b) soon after bar fracture; and in beam sections (c) at face of center block (left) at peak resisting force, and (right) soon after bar fracture; and in beam sections (d) at face [5]



Figure 25: Beam's deformed shape right after bar fracture [5]

This approach allows the numerical model to capture the abrupt loss of flexural strength while maintaining global equilibrium. After bar fracture, the beam transitions from double curvature to a single-curvature (cantilever-type) deformation, consistent with experimental observations.

After the simulated fracture, the numerical model reproduced the change from double-curvature response toward a cantilever-type deformation pattern and captured the continued load resistance through catenary action. For this thesis, the paper provides a key modeling lesson: if reinforcement fracture is expected in the experimental response, a numerical model must either simulate fracture explicitly or use a calibrated equivalent approach. Otherwise, the post-peak and catenary-stage response may be unreliable.

In database terms, Sasani et al. complement the beam-column sub-assembly tests by focusing on one difficult local phenomenon. The study shows that progressive collapse resistance is not only a question of peak load. It is also a question of what happens after local fracture, whether the remaining load path survives, and whether the model can reproduce the sudden discontinuity introduced by bar rupture.

Aspect	Reported information	Database significance
Prototype basis	Seven-storey RC building; exterior two-span beam over removed column	Column-loss mechanism in frame-derived beam
Scale	3/8-scale beam model	Allows detailed large- deformation test under fixed boundaries
Key fracture event	Bottom bars fractured at about 155 mm and 190 mm displacement	Fracture reduces flexural strength but does not necessarily cause collapse
Post-fracture mechanism	Top reinforcement developed axial tension	Catenary action provided continued resistance
Modeling method	DIANA continuum FE with explicit bond-slip and fracture procedure	Important for later NLFE modeling strategy

2.9 Clement Huilie report: ATENA 2D reconstruction of selected experimental specimens

The report is not a primary experimental campaign in the same sense as the other papers, but it is still relevant to this thesis because it links experimental evidence to ATENA 2D numerical modeling. The report focuses on progressive collapse of RC beam-column systems and attempts to reproduce the response of selected experimentally tested specimens using nonlinear finite element analysis. In the structure of this thesis, this material is best treated as a bridge between the experimental database and the later chapter on nonlinear finite element analysis.

The report studies specimens previously tested under column-removal scenarios, with particular attention to specimens S7 and S8 from the Yu and Tan experimental series. These specimens are relevant because S7 and S8 have different span-to-depth ratios and therefore show different balances between compressive arch action and catenary action. S7 represents a specimen in which reduced span-to-depth ratio improves arching while maintaining meaningful deformation capacity, whereas S8 illustrates how very short geometry can produce high early arching resistance but may also become sensitive to shear-dominated behavior.

The main value of the report is the recognition that ATENA can reproduce the global load-displacement behavior only when the model parameters and boundary conditions are adjusted carefully. The calibrated curves can capture the general stiffness evolution and resistance stages, but they depend strongly on softening parameters, mesh discretization, and support representation. This is especially important for later stages of the thesis, where numerical models must not be treated as automatic truth. They must be checked against experimental evidence.

The report also examines robustness-oriented design modifications, particularly symmetry of sections and continuity of longitudinal bars. These modifications are consistent with the broader literature, which shows that reinforcement continuity is essential for catenary action. Removing or interrupting longitudinal continuity weakens the ability of the beam to develop axial tension after large displacement, while symmetric or better-balanced reinforcement can improve the redistribution of forces across critical regions.

For the present experimental database chapter, the Clement Huilie report is therefore not used as a source of new physical test data. Instead, it is used as an interpretation layer. It demonstrates how the experimental database can be converted into numerical input, how model calibration affects the predicted response, and why later ATENA modeling must carefully represent concrete cracking, reinforcement yielding, bond behavior, boundary restraint, and reinforcement continuity.

Item	Modeling role	Reported calibration detail	Interpretation
S7	ATENA 2D model calibrated against experimental curve	Mesh 0.1 m; spring stiffness 400 MPa; tensile softening 4.70E-04; crack reduction 0.7; compression softening 0.05	Calibration shows sensitivity of global response to softening and support parameters
S8	ATENA 2D model calibrated against experimental curve	Mesh 0.07 m; spring stiffness 200 MPa; tensile softening 4.40E-03; crack reduction 0.6; compression softening 0.14	Different parameter set needed because geometry and failure behavior differ
Design modifications	Symmetry and longitudinal continuity investigated	Continuity modification directly affects catenary action	Useful link to improved-model chapter

Table 18: Clement Huilie samples overview

2.10 Lim et al. (2017): rotational capacity and horizontal restraint

Lim et al. (2017) provide one of the clearest experimental investigations of two parameters that are essential for catenary action: rotational capacity and horizontal restraint. The study consists of five 2D RC frame specimens, each representing a different combination of detailing and boundary condition. The main importance of the paper is that it shows catenary action is not guaranteed by the beam alone. It requires both sufficient rotation capacity at critical sections and sufficient horizontal restraint from the surrounding structure.

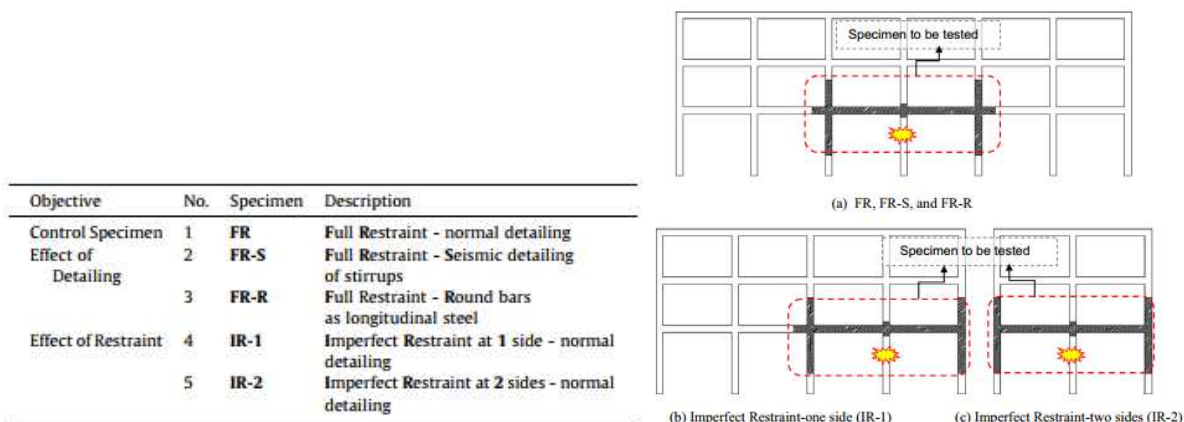


Figure 26: 2-D RC frame specimens & Different scenarios represented by specimens.[10]

The specimens were scaled to 2/5 of a six-storey prototype frame. The beam cross-section was approximately 180 mm by 100 mm, the span approximately 2220 mm, and the columns approximately 180 mm by 180 mm with a height of about 1530 mm. A vertical displacement was applied at the middle joint at a loading rate of 6 mm/min. Axial load was applied to the columns to represent upper-storey gravity effects. The boundary conditions were then varied to represent full restraint or imperfect restraint conditions.

Beam		Column		Concrete cover	For all specimens
Length	Cross-section (depth × width)	Length	Cross-section (depth × width)		
2400 mm	180 × 100 mm ²	1530 mm	180 × 180 mm ²	15 mm	
Specimen	Reinforcement in beams ^a	Bottom		Reinforcement in columns ^a	
FR, FR-S, IR-1, IR-2	Top 1.52% (3T10)	1.01% (2T10)		1.78% (4T10+4T6)	
FR-R	2.02% (4R10)	1.33% (2R10+1R8)		1.78% (4T10+4T6)	

^a Reinforcement ratio is calculated by $\rho = A_s/bd$, in which b and d are the width and the effective depth of beam sections, respectively.

Table 19: Reinforcement details and geometric properties of specimens. [10]

Mechanism		FR	FR-R	Ratio
Flexural resistance	Positive moment (M_p) at mid-joint	11.0 kNm	11.1 kNm	1.01
	Negative moment (M_a) at beam-end	16.5 kNm	14.2 kNm	0.86
Catenary resistance	Ultimate force at bottom bar	94.2 kN	97.4 kN	1.03
	Ultimate force at top bar	141.3 kN	157.0 kN	1.11

Table 20: Ratio of flexural and catenary resistances of FR-R to FR based on EC2. [10]

Material	Size	Bar Type	Elastic modulus (GPa)	Yield strength (MPa)	Ultimate strength (MPa)	Fracture strain (%)
Longitudinal Reinforcement	T10	Deformed	200	507	609	11
	R10	Round	200	400	500	30
	R8	Round	200	300	395	25
Stirrup	R6	Round	200	400	583	25
Concrete [150 mm (dia.) × 300 mm (height)]			Compressive strength (MPa)			32
			Splitting tensile strength (MPa)			3.7
			Elastic modulus (GPa)			26.6

Table 21: Material properties [10]

The fully restrained specimen FR served as the control specimen. FR-S used closer stirrup spacing near the joints to improve rotational capacity through seismic-type detailing. FR-R used smooth round bars with higher deformation capacity and slip, intended to improve rotation and delay fracture. IR-1 and IR-2 represented imperfect horizontal restraint conditions, with one side or both sides of beam-level horizontal restraint removed. This specimen design allows the effect of detailing to be separated from the effect of restraint.

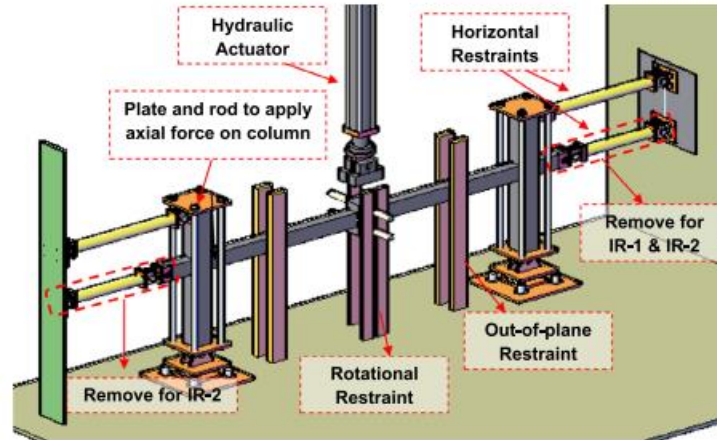


Figure 27: Typical test setup [10]

The fully restrained specimens developed the expected sequence of flexural behavior, compressive arch action, and catenary action. At small displacements, the beam pushed outward and horizontal restraints generated compressive axial force, increasing the load capacity beyond pure bending. At larger displacements, concrete crushing and reinforcement fracture reduced flexural contribution, the axial force changed from compression to tension, and catenary action became dominant. The transition to catenary action was observed around a middle joint displacement of approximately 250 mm, with catenary behavior becoming dominant at larger displacements.

Specimen	Flexure		CAA		CA		P_{CA}/P_{BIF}	P_{CA}/P_{BIF}
	P_{BIF}	Δ_{BIF}	P_{CAA}	Δ_{CAA}	P_{CA}	Δ_{CA}		
FR	24.5 kN	46 mm	27.5 kN	84 mm	70.9 kN	540 mm	1.12	2.89
FR-S	24.5 kN	45 mm	26.9 kN	95 mm	68.5 kN	522 mm	1.10	2.80
FR-R	22.8 kN	34 mm	24.0 kN	70 mm	76.0 kN	560 mm	1.05	3.33
IR-1	24.5 kN	46 mm	27.2 kN	89 mm	34.1 kN	320 mm	1.11	1.39
IR-2	24.5 kN	53 mm	25.8 kN	70 mm	29.4 kN	513 mm	1.04	1.20

P_{BIF} , P_{CAA} , P_{CA} are the capacities of flexural mechanism, peak CAA, and peak CA, respectively.

Δ_{BIF} , Δ_{CAA} , Δ_{CA} are the MJD corresponding to P_{BIF} , P_{CAA} , P_{CA} respectively.

Table 22: Enhancement ratio of CAA and CA [10]

The quantitative results show the scale of the catenary contribution. The flexural capacities were approximately 24.5 kN for FR, FR-S, IR-1, and IR-2, and 22.8 kN for FR-R. The CAA peak loads were in the range of about 24.0 to 27.5 kN, showing a relatively modest increase over flexural capacity. The CA peak loads, however, were much larger for fully restrained specimens: 70.9 kN for FR, 68.5 kN for FR-S, and 76.0 kN for FR-R. Thus, catenary action increased resistance to roughly 2.8 to 3.3 times the flexural capacity under full restraint.

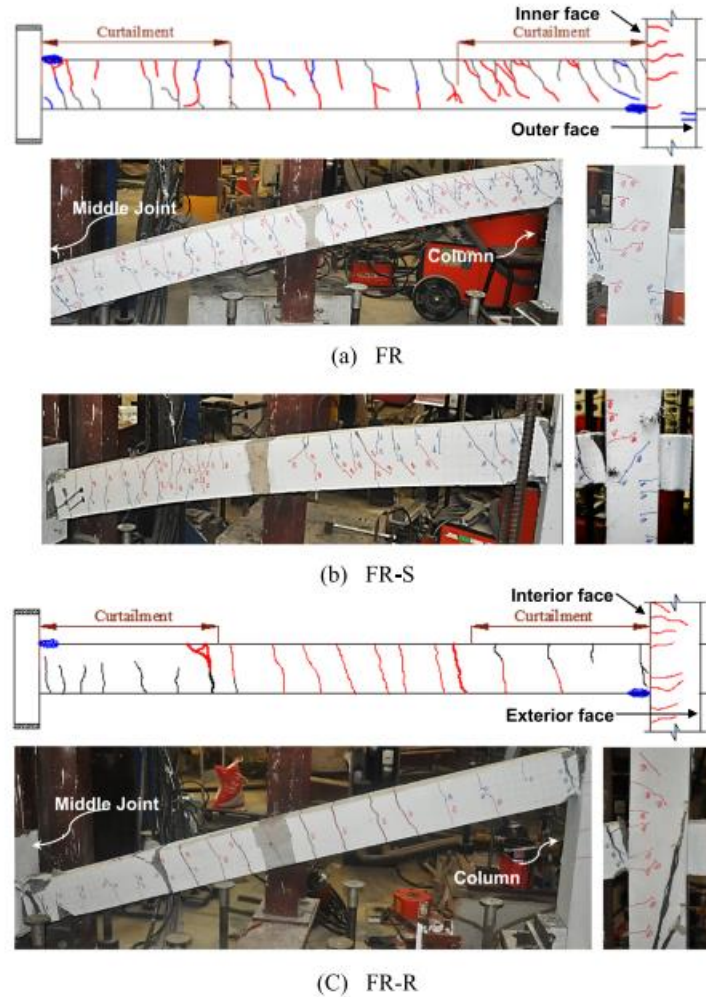


Figure 28: Crack patterns in beams and columns of FR, FR-S and FR-R. [10]

The rotational capacity results are also important. FR reached approximately 0.23 rad, FR-S also about 0.23 rad, and FR-R approximately 0.30 rad. The round-bar specimen therefore achieved a higher rotation capacity, attributed to greater bar ductility and slip. The seismic stirrup detailing improved local confinement and reduced joint shear distortion but did not significantly increase the ultimate catenary capacity. This indicates that catenary action is more sensitive to tensile reinforcement ductility and rotation capacity than to confinement alone, provided shear failure is already prevented.

The imperfect restraint specimens provided the strongest lesson. IR-1 and IR-2 did not develop catenary action to the same level as the fully restrained specimen. Their CA peak loads were only about 34.1 kN and 29.4 kN, respectively. Failure shifted from beam-controlled reinforcement fracture to column-controlled instability. The exterior columns were pulled inward by beam tension during the catenary stage, and once the column lateral resistance was exceeded, the system became unstable. This shows that even if the beam

has adequate reinforcement, the surrounding frame must be able to restrain the tensile forces required for catenary action.

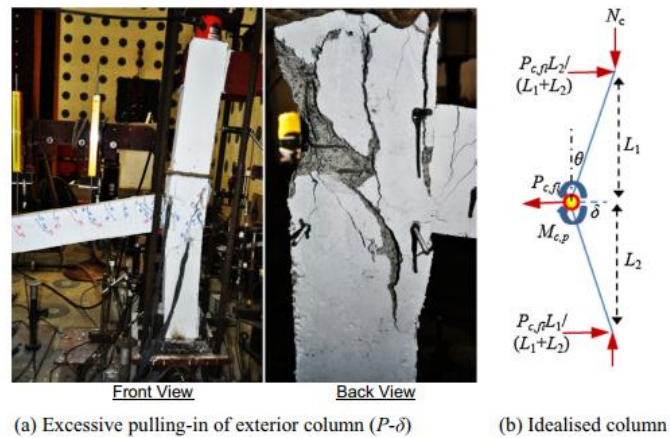


Figure 29: Failure modes of IR-1 and IR-2. [10]

For the database, Lim et al. (2017) are essential because they connect local deformation capacity with global restraint. The study proves that catenary action is a system mechanism, not a member mechanism. The beam, joints, columns, and surrounding restraints must all participate. This conclusion is directly relevant to the numerical modeling of 2D frames and to any later attempt to include orthogonal frame effects or equivalent springs. The study confirms that catenary action is highly dependent on both rotational capacity and horizontal restraint.

Main findings:

- Fully restrained frames can develop strong catenary action ($\approx 2.9 \times$ flexural capacity)
- Rotational capacity improvements:
 - have limited effect with stirrups
 - but significant effect with ductile reinforcement (round bars)
- Horizontal restraint is critical:
 - without it, CA cannot fully develop
 - failure occurs in columns instead of beams

The study highlights that design for progressive collapse must ensure both ductility and adequate restraint, otherwise catenary action cannot be relied upon.

Specimen	Configuration	Flexural capacity (kN)	CAA peak (kN)	CA peak (kN)	Main conclusion
FR	Full restraint, standard detailing	24.5	27.5	70.9	Full CA development; beam-controlled failure
FR-S	Full restraint, closer stirrup detailing	24.5	26.9	68.5	Better local confinement; limited global CA increase
FR-R	Full restraint, round bars	22.8	24.0	76.0	Higher rotation capacity and improved CA
IR-1	Imperfect restraint on one side	24.5	27.2	34.1	CA limited by column instability
IR-2	Imperfect restraint on both sides	24.5	25.8	29.4	Insufficient restraint; column-controlled failure

Table 23: Lim et al. samples overview

2.11 Ren et al. (2016): B-series continuous RC beams and the role of beam height

Ren et al. (2016) investigate progressive collapse resistance in one-way RC beam-slab substructures. For the purpose of the present thesis, the B-series specimens are particularly important because they isolate the behavior of beam-only systems without slab participation, while the broader study also clarifies how slab action modifies the response. Since the user specifically requested emphasis on the B-series rather than the S-series, the discussion here focuses mainly on B2 and B3, using the slab specimens only for contextual interpretation.

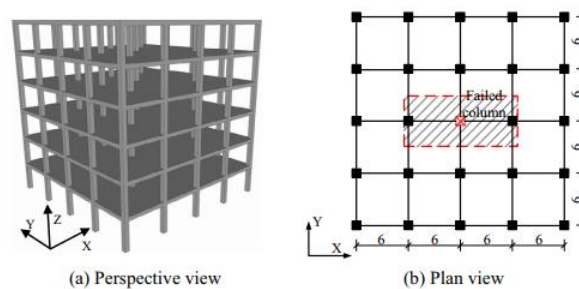


Figure 30: The prototype structure (units: m). [9]

Type	Column	Beam	Boundary beam
Prototype structure	600 mm × 600 mm	250 mm × 500 mm	–
Control specimen	200 mm × 200 mm	85 mm × 170 mm	500 mm × 370 mm

Table 24: Sectional sizes of the prototype structure and the control specimen. [9]

The prototype structure was a six-storey RC frame with 6 m spans in both directions, a first-storey height of 4.2 m, and upper-storey heights of 3.6 m. The tested specimens were scaled to approximately one-third of the prototype. The B-series consisted of continuous beam substructures without slab contribution. B2 and B3 had the same general reinforcement concept and boundary conditions, but B3 had increased beam height compared with B2. This makes the pair useful for isolating the influence of beam height on beam action and catenary action.

Specimen	Beam height h_b	Beam width w_b	Slab thickness t_s	Slab width w_s	Beam top		Beam bottom	Slab bottom Longitudinal/transverse	Slab top Longitudinal/transverse
					Beam end	Mid-span			
B2	170	85	–	–	2 ϕ 8 + 1 ϕ 6	2 ϕ 8	2 ϕ 8	–	–
B3	200	85	–	–	2 ϕ 8 + 1 ϕ 6	2 ϕ 8	2 ϕ 8	–	–

Table 25: Cross-sectional dimensions and reinforcement details (units: mm). [9]

The test simulated middle-column removal by applying vertical displacement at the column stub location. The ends were fixed through welded and bolted steel-plate connections, and hydraulic actuators applied the displacement-controlled loading. The loading rate was 2 mm/min during the beam-dominated stage and 4 mm/min during the catenary stage. Strain gauges and displacement transducers were installed to monitor deformation, concrete strain, and reinforcement response at critical sections.

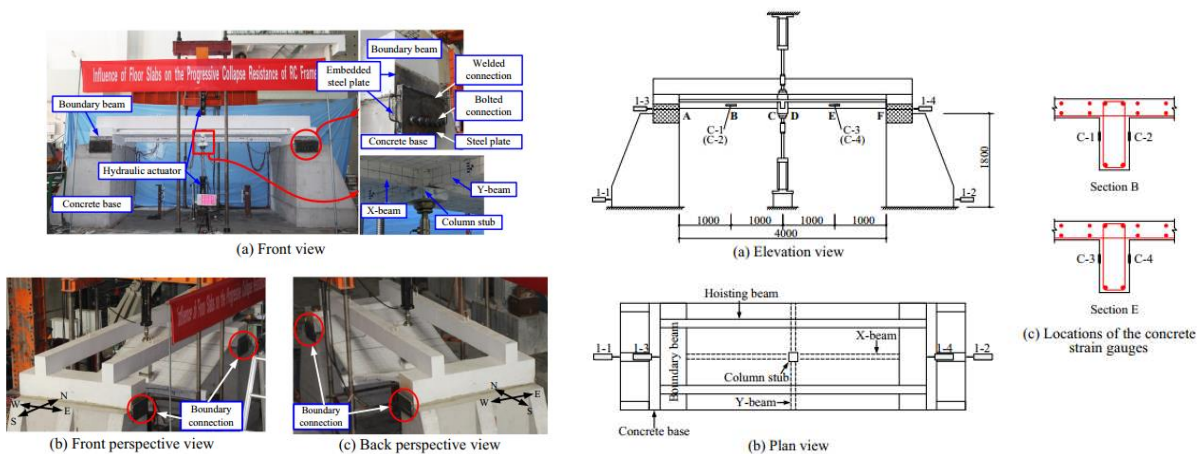


Figure 31: Test Setup & Details of test setup and arrangement of instrumentation (units: mm) [9]

The global response of B2 and B3 can be interpreted through two main stages: beam action and catenary action. In the beam-action stage, flexural resistance and compressive arch action contribute to the vertical load capacity. As displacement increases, concrete crushing

and crack propagation degrade the flexural mechanism. The axial force then changes toward tension and catenary action begins to develop, allowing the beam to carry additional load through longitudinal reinforcement.

The quantitative results clearly show the difference between the two specimens. For B2, the beam mechanism peak F_b was about 34 kN, the transition load F_t about 17 kN, and the catenary peak F_c about 58 kN. For B3, the corresponding values were about 41 kN, 21 kN, and 42 kN. Thus, increasing beam height improved the beam mechanism capacity but did not proportionally improve catenary capacity. In fact, the catenary peak of B3 was lower than that of B2, which shows that catenary resistance is not controlled simply by beam depth.

	B2	B3
F_b (kN)	34	41
F_t (kN)	17	21
F_c (kN)	58	42
F_c/F_b	1.71	1.02

Table 26: Progressive collapse resistances of the tested specimens. [9]

The failure modes explain this result. In B2, failure was governed by rupture of bottom reinforcement in the positive-moment region, after the specimen developed a significant catenary-stage load increase. In B3, failure occurred due to rupture of top reinforcement near the beam ends, associated with larger rotations at the supports. The deeper beam was better in flexure, but the large-deformation tensile mechanism depended on reinforcement strain capacity and rotation demand, not only on flexural depth.

This distinction is extremely important for robustness design. Increasing member depth may improve flexural capacity and delay mechanism transition, but it does not automatically increase catenary capacity. At large deformations, the critical parameters are the total tensile reinforcement able to bridge the damaged region, its anchorage and continuity, and its ability to sustain large strain without fracture. Therefore, geometry and reinforcement detailing must be interpreted separately for the beam and catenary stages.

Although the present emphasis is on the B-series, the comparison with slab specimens in Ren et al. confirms that slab participation can increase both the early resistance and the large-deformation resistance. The slab contributes reinforcement area and membrane action, but also changes the compression zone and may introduce different failure modes. This contextual evidence is useful for later chapters, especially if improved numerical models or 3D effects are considered.

For the database, Ren et al. provide a clear example of why the maximum load alone is insufficient. B3 has a higher beam mechanism load than B2, but B2 develops a stronger

catenary peak. The thesis must therefore compare specimens according to mechanism stage, not only according to final capacity. This approach will be useful when comparing experimental results with improved models in later chapters.

Specimen	Configuration	Main variable	F_b (kN)	F_t (kN)	F_c (kN)	Failure interpretation
B2	Beam-only continuous substructure	Control beam height	34	17	58	Stronger catenary development; bottom reinforcement rupture
B3	Beam-only continuous substructure	Increased beam height	41	21	42	Higher beam capacity but lower CA; top reinforcement rupture at beam ends

Table 27: Ren et al. samples overview

2.12 NIST experimental study: IMF and SMF beam–column sub-assemblages under column-loss scenario

Following the discussion of analytical and numerical investigations on frame sub-assemblages, it is necessary to consider experimental studies that explicitly address the influence of structural detailing level on progressive collapse resistance. In this context, the experimental program conducted by the National Institute of Standards and Technology (NIST) provides a fundamental contribution, as it enables a direct comparison between different design philosophies under a consistent column-loss scenario.

The NIST study focuses on beam–column sub-assemblages designed according to two distinct categories of seismic provisions: Intermediate Moment Frame (IMF) and Special Moment Frame (SMF). These two systems represent different levels of ductility and detailing rigor commonly adopted in engineering practice. IMF systems are characterized by moderate confinement and anchorage provisions, whereas SMF systems incorporate enhanced detailing requirements intended to ensure large deformation capacity and reliable energy dissipation under extreme loading conditions.

Specimen	Compressive Strength psi (MPa)		Split-Cylinder Tensile Strength psi (MPa)
	Footings	Beams and Columns	Beams and Columns
IMF	5700 (39)	4700 (32)	450 (3.1)
SMF	5300 (37)	5200 (36)	420 (2.9)

Table 28: Average compressive and tensile strengths of 6x12 concrete cylinders at the time of testing[2]

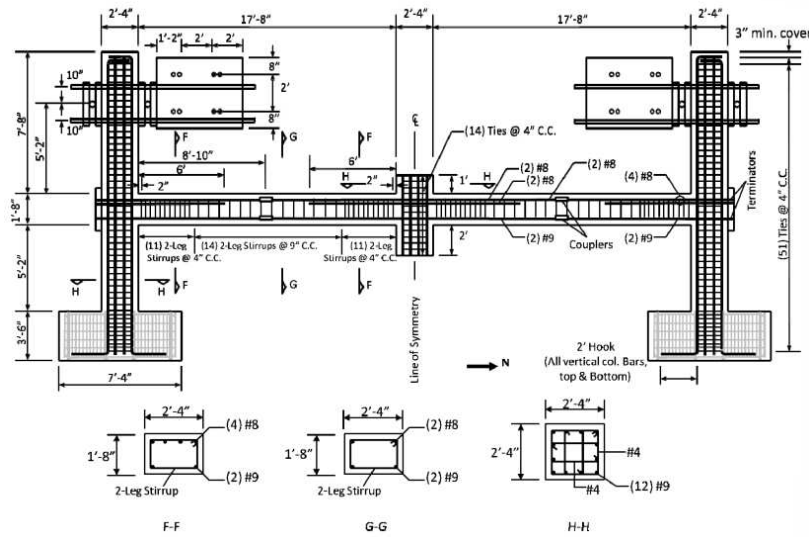


Figure 32: Details of IMF specimen [2]

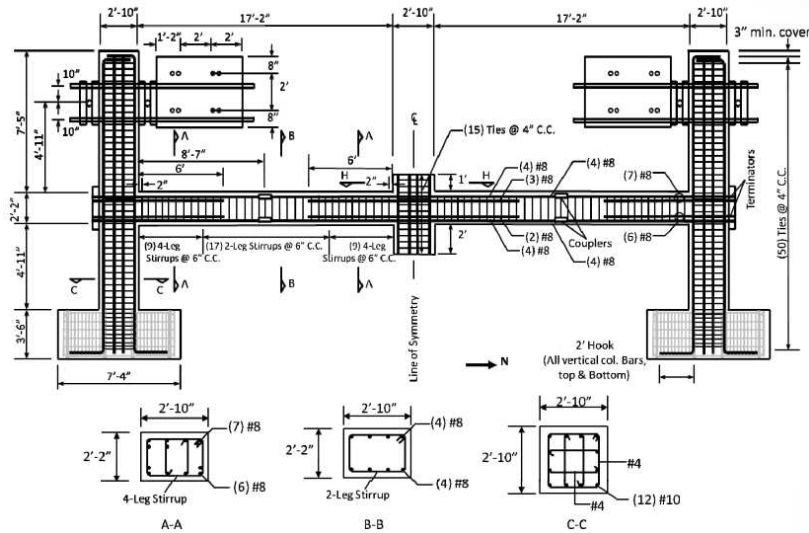


Figure 33: Details of SMF specimen [2]

The experimental setup follows the Alternate Load Path (ALP) methodology, consistent with the majority of studies presented in this chapter. In this configuration, the central column of a two-span beam-column assembly is effectively removed, and a vertical displacement is imposed at the location of the lost support. This approach allows the structure to respond through load redistribution mechanisms, thereby revealing its capacity to activate alternative load paths and sustain the applied load after local damage.



Figure 34: SMF specimen [2]

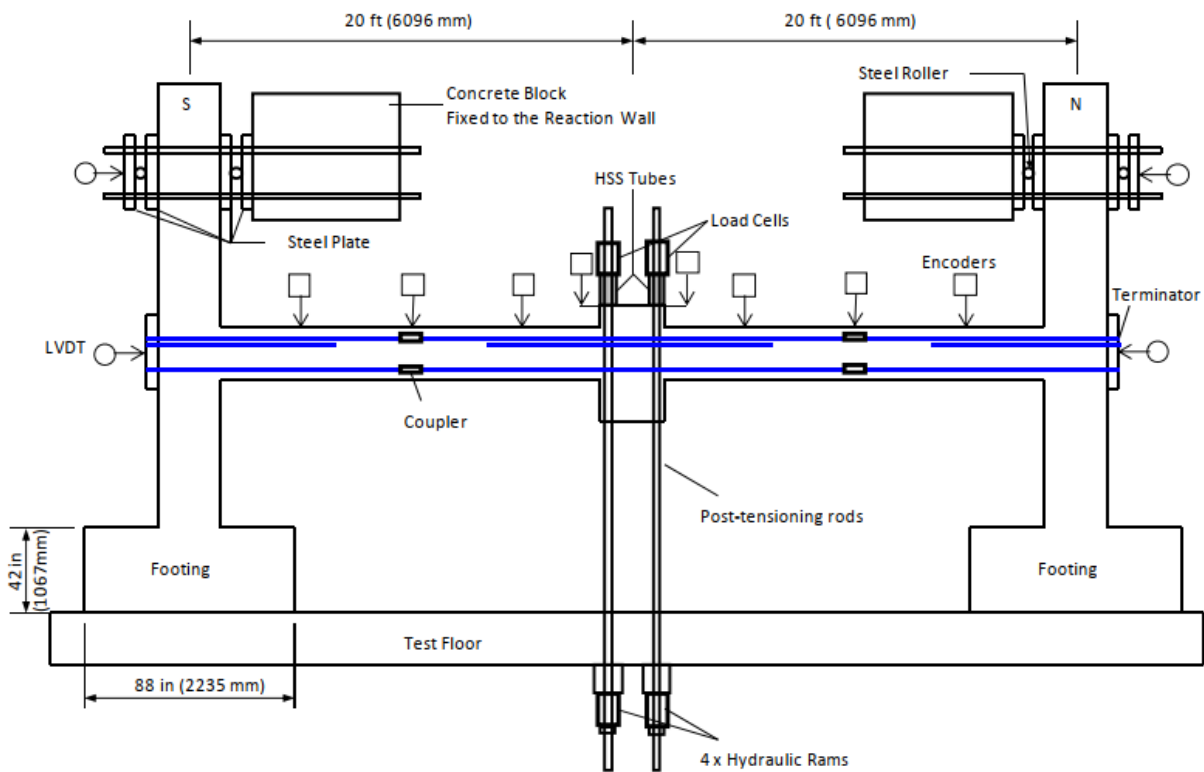


Figure 35: Test configuration for IMF and SMF specimens [2]

At the initial stage of loading, the response of both IMF and SMF specimens is governed by flexural behavior, in line with the observations reported for the Yu and Tan sub-assemblages. Cracking develops in the tensile regions of the beams, and plastic hinges begin to form near the beam–column joints. At this stage, the influence of detailing level is relatively limited, as the response is primarily controlled by bending resistance and sectional capacity.

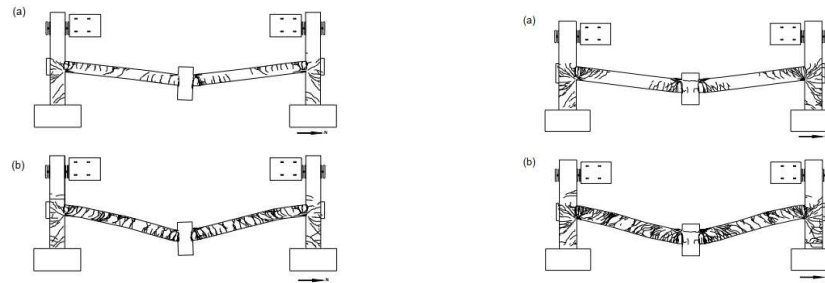


Figure 36: Crack patterns of IMF and SMF specimen at center column deflections of (a) 24 in (610 mm) and (b) 43 in (1092 mm) [2]

As the displacement increases, the structural response evolves toward compressive arch action (CAA), driven by the axial restraint provided by the boundary conditions. This mechanism results in an increase in load-carrying capacity beyond the flexural limit, reflecting the contribution of compressive forces in the beam. While both IMF and SMF specimens exhibit this strengthening phase, differences begin to emerge in terms of stability and damage progression. In particular, SMF specimens tend to maintain a more stable arching response, whereas IMF specimens show earlier degradation due to localized damage at the beam–column interfaces.

The most significant divergence between the two systems occurs at large deformation levels, where the structure attempts to transition into catenary action (CA). This stage represents the final load-resisting mechanism, in which axial tension in the longitudinal reinforcement becomes the dominant force transfer mechanism. The successful development of catenary action requires not only sufficient reinforcement but also the preservation of load continuity through effective anchorage and joint integrity.

In SMF specimens, the enhanced detailing proves to be highly effective in facilitating this transition. The improved confinement and anchorage conditions allow the reinforcement to sustain large tensile strains without premature failure, enabling the development of a stable and sustained catenary mechanism. This behavior is often characterized by a secondary increase in load capacity after the initial peak, reflecting the activation of tensile resistance in the reinforcement.

In contrast, IMF specimens exhibit a more limited capacity to mobilize catenary action. Although the transition toward tensile behavior may initiate, it is frequently interrupted by damage localized at the beam–column joints. Bond degradation and insufficient anchorage reduce the effectiveness of the tensile load path, preventing the full development of the catenary mechanism. As a result, the load-carrying capacity decreases more rapidly, and failure is often governed by joint-related phenomena rather than by reinforcement fracture.

These observations highlight a critical aspect of structural robustness that complements the findings of previous sections: the development of alternative load paths is strongly dependent on detailing quality and local structural integrity. While compressive arch action is primarily governed by boundary conditions and global geometry, the activation of catenary action requires the preservation of local load-transfer mechanisms, particularly at beam–column joints.

From the perspective of the experimental database developed in this thesis, the NIST study introduces an essential parameter that is not explicitly captured in purely geometric or material variations, namely the level of structural detailing. The IMF and SMF specimens therefore represent two distinct categories within the dataset:

- IMF specimens → systems with moderate robustness, where catenary action is limited or unstable
- SMF specimens → systems with high robustness, capable of developing sustained catenary behavior

This distinction is particularly relevant for the comparative analysis presented in the following section, where different experimental campaigns are evaluated within a unified framework. The inclusion of IMF and SMF specimens enables the identification of lower and upper bounds of catenary capacity, thereby strengthening the interpretation of variability across the selected 26-sample dataset.

Furthermore, the NIST results provide important guidance for the subsequent numerical modeling phase. In order to accurately reproduce the observed behavior, the numerical model must capture not only material and geometric nonlinearities but also bond-slip effects, anchorage performance, and joint degradation mechanisms. The contrast between IMF and SMF responses thus serves as a benchmark for evaluating the reliability of the modeling approach.

2.13 Cross-study comparison of mechanisms

When the selected studies are considered together, the most important conclusion is that RC robustness under column loss is a staged mechanism rather than a single resistance value. The first stage, flexural action, is strongly influenced by ordinary beam design parameters such as reinforcement ratio, effective depth, and moment capacity. Yu and Tan (2013) and Ren et al. (2016) both show that changes in reinforcement or beam height can significantly modify the first peak or beam mechanism resistance. However, these same parameters do not necessarily produce a proportional increase in catenary resistance.

The second stage, compressive arch action, is fundamentally restraint-dependent. Yu and Tan showed that CAA develops when the beam elongation is restrained by the test system, while Farhang Vesali et al. isolated this mechanism in longitudinally restrained beams. The latter study is particularly useful because it demonstrates that support stiffness, concrete strength, and strain penetration strongly influence the peak arching load. Therefore, a numerical model that assumes ideal fixed ends may overestimate arching stiffness and capacity unless the real restraint condition is measured or calibrated.

The third stage, catenary action, is reinforcement- and system-dependent. Sasani et al., Lim et al., Yu and Tan, and Kim-Choi all show that catenary action requires reinforcement continuity and sufficient anchorage. However, Lim et al. demonstrates that even a ductile beam cannot fully develop catenary action if the surrounding frame cannot provide horizontal restraint. In the imperfectly restrained specimens, failure shifted to column instability. This proves that catenary action is not a local beam feature; it is a system-level mechanism involving beam, joint, column, and surrounding restraint.

The studies also show that local failure controls global capacity. Reinforcement fracture, bond-slip, joint failure, concrete crushing, and shear failure are repeatedly observed as the critical limit states. Yu and Tan (2011) emphasize bond-slip and joint interface failure; Yu and Tan (2013) show the limiting role of shear failure in S8; Sasani et al. focus on bar fracture and post-fracture redistribution; Kim-Choi show that joint failure prevents catenary action; Lim et al. show that column instability can limit catenary action under imperfect restraint. These observations mean that a robust structure must avoid premature brittle failure before the desired alternative load path can be activated.

The loading protocol is another cross-cutting issue. Qian et al. show that concentrated loading can capture early-stage yield and CAA reasonably, but it may overestimate catenary capacity and identify the wrong critical section. This finding is important for the interpretation of many quasi-static tests. A model calibrated against concentrated loading may match the experimental curve but still be unconservative for a real building if distributed gravity load history shifts damage to another region.

The experimental database therefore supports a hierarchy of modeling requirements. At minimum, a later nonlinear finite element model should reproduce the load-displacement curve and the mechanism transition. At a higher level, it should reproduce the critical damage region, the axial force transition from compression to tension, and the key failure mode. At the most rigorous level, it should represent bond-slip, strain penetration, reinforcement fracture, support flexibility, and slab or orthogonal frame contribution where these mechanisms control the response.

2.14 Implications for later nonlinear finite element modeling

The experimental database gives direct guidance for the development of nonlinear finite element models in later chapters. First, the model must be capable of reproducing material nonlinearity in both concrete and steel. Concrete cracking, compression softening, crushing, and tension stiffening affect the stiffness degradation and peak response. Steel yielding, strain hardening, and fracture strain affect both flexural and catenary stages. A model that only captures yielding but not large tensile strain demand cannot reliably predict catenary action.

Second, geometric nonlinearity must be included. Catenary action is a large-displacement mechanism, and the vertical component of axial tensile force depends on the deformed shape. Qian et al. show that the same axial force level can produce different vertical resistance depending on the rotation and failure location. Therefore, later pushdown analyses must not treat the geometry as unchanged when large displacements are expected.

Third, boundary conditions must be carefully represented. The experimental studies repeatedly show that support stiffness and horizontal restraint are not secondary assumptions. In Farhang Vesali et al., support rotation and strain penetration altered stiffness and peak load. In Lim et al., imperfect horizontal restraint changed the controlling failure mode from beam fracture to column instability. In ATENA modeling, equivalent supports or springs must therefore be calibrated or justified rather than assumed arbitrarily.

Fourth, local damage and failure criteria must be considered. Yu and Tan, Sasani, Kim-Choi, and Ren all show that reinforcement fracture, joint degradation, and concrete crushing control the ultimate response. If the model cannot represent these mechanisms explicitly, then the interpretation of ultimate capacity must be conservative and tied to experimentally observed failure thresholds. This is particularly important when comparing improved numerical models with experimental curves.

Fifth, reinforcement continuity must be modeled consistently with the experimental specimen and with the intended design improvement. The catenary stage is fundamentally dependent on continuous tensile reinforcement crossing the damaged region. If bars are curtailed, lapped, or anchored differently, the model must reflect this. Otherwise, the predicted catenary force may be physically unrealistic.

Finally, the database suggests that validation should not be based only on matching the maximum load. A good model should reproduce the sequence of mechanisms, approximate stiffness evolution, critical displacement ranges, axial force transition, and failure localization. Only then can it be used as a credible basis for the improved models and probabilistic analysis planned in later chapters.

2.15 Chapter Conclusion

The experimental database assembled in this chapter provides the technical basis for the subsequent parts of the thesis. The selected studies collectively show that the robustness of RC systems after column loss is governed by a sequence of mechanisms: flexural action, compressive arch action, and catenary action. These mechanisms are not independent; each stage depends on the structural condition created by the previous stage and on the ability of the specimen to avoid premature local failure.

The early-stage response is governed mainly by flexural capacity and compressive arching. Reinforcement ratios, beam depth, span-to-depth ratio, and concrete strength influence this stage. However, the later catenary stage depends more strongly on reinforcement continuity, anchorage, ductility, rotation capacity, and horizontal restraint. Therefore, improving robustness requires more than increasing flexural strength. It requires details and boundary conditions that allow the structure to reach large deformation without losing the alternative load path.

The database also shows that local failure mechanisms dominate global behavior. Reinforcement fracture, bond-slip, concrete crushing, joint failure, shear failure, and column instability all appear in the selected studies as controlling events. A design or model that ignores these mechanisms may overestimate the structural reserve after column loss. This is particularly important for catenary action because the mechanism requires large strain and rotation, which are precisely the conditions under which local failures are likely to occur.

The comparison of loading regimes demonstrates that experimental methodology strongly influences the conclusions drawn from a test. Concentrated loading is useful for controlled investigation, but it may overestimate catenary capacity when compared with sequential loading that includes pre-existing gravity loads. This finding must be kept in mind when later experimental curves are used for model calibration or validation.

For the later chapters of this thesis, the experimental database defines the target behavior that numerical models must reproduce. The ATENA models should not only match the shape of load-displacement curves but also capture the mechanism transition, damage localization, and relevant failure modes. The improved models and probabilistic analyses should be interpreted against this experimental evidence. In this way, the chapter acts as the bridge between literature review and numerical investigation, transforming individual experimental papers into a unified technical foundation for the thesis.

2.16 Database limitations and rules for later use

Although the experimental database is broad, it must be used with clear limitations. The specimens were tested at different scales, with different boundary systems, different reinforcement details, and different loading protocols. Therefore, direct numerical comparison between all peak loads is not appropriate unless the comparison is normalized by geometry, reinforcement ratio, and loading condition. For example, a high CAA peak in a short-span specimen should not be interpreted as superior global robustness if the same specimen cannot sustain the deformation required for catenary action. Similarly, a specimen with lower first peak but larger deformation capacity may be more relevant for evaluating residual resistance after local damage.

A second limitation concerns scale effects. Many of the selected experiments are half-scale, 2/5-scale, 3/8-scale, or one-third-scale specimens. Scaling affects reinforcement bond, crack spacing, aggregate size, confinement, and bar fracture behavior. The experimental results are still valuable because they reproduce the qualitative mechanism transition, but the numerical model should not blindly transfer all local fracture or bond parameters to full-scale structures without checking material laws and size-dependent assumptions. This is especially important in ATENA modeling, where fracture energy, tension stiffening, crack band width, mesh size, and reinforcement bond behavior may influence the predicted response.

A third limitation concerns the idealization of boundary conditions. Several tests were designed with strong axial and rotational restraints in order to force the development of arching and catenary action. Such restraints are useful for understanding the mechanisms, but they may not fully represent an actual building where the surrounding frame, slab, columns, and foundations provide finite stiffness. Lim et al. demonstrate this issue directly: when horizontal restraint is imperfect, the system may fail by column instability before full catenary action develops. Farhang Vesali et al. also show that even nominally fixed supports exhibit finite rotations and strain penetration. Therefore, later numerical models should represent realistic restraint conditions, or the results should be clearly interpreted as upper-bound mechanism studies.

A fourth limitation is related to loading history. Concentrated quasi-static loading is widely used because it gives a clean load-displacement curve, but Qian et al. demonstrate that it can overestimate catenary capacity and misidentify the critical section. This does not mean that concentrated loading tests should be discarded. They remain very valuable for comparing flexural, arching, and catenary stages under controlled conditions. However, when the objective is to simulate a real building under gravity load before column loss, sequential loading or an equivalent numerical procedure should be considered. In later

chapters, this issue should be explicitly discussed when selecting the load application method for ATENA analyses.

A fifth limitation concerns failure criteria. Experimental tests can observe bar fracture, joint failure, concrete crushing, and column instability directly, while numerical models often require these phenomena to be represented through constitutive laws or failure thresholds. If a model does not include explicit bar fracture, the predicted catenary capacity may extend beyond the experimental limit. If bond-slip is ignored, the stiffness and strain localization may be overestimated. If compression softening or confinement is poorly calibrated, the arching peak may be inaccurate. The database should therefore be used not only to calibrate curves but also to define realistic termination criteria for numerical simulations.

For these reasons, the experimental database should be used according to a hierarchy. The first level is qualitative validation: the model should reproduce the mechanism sequence observed experimentally. The second level is global quantitative validation: the model should approximately reproduce characteristic loads and displacements. The third level is local validation: the model should reproduce crack localization, reinforcement yielding, bar fracture location, or joint damage where such data are available. The fourth level is predictive use: only after the previous levels are satisfied should the model be used for improved designs or probabilistic sampling. This hierarchy protects the thesis from treating numerical results as independent of experimental evidence.

In practical terms, the most reliable use of the database in this thesis is comparative rather than absolute. Yu and Tan (2013) can be used to compare the influence of reinforcement ratio and span-to-depth ratio. Qian et al. can be used to compare loading regimes. Lim et al. can be used to compare full and imperfect horizontal restraint. Ren et al. can be used to compare beam height and, with caution, slab participation. Farhang Vesali et al. can be used to isolate support rotation and strain penetration. Sasani et al. can be used to interpret reinforcement fracture and post-fracture catenary action. This comparative logic is more technically defensible than simply combining all maximum loads into a single ranking.

The final rule for later chapters is that each numerical or improved model should be linked back to a specific experimental lesson. If a model improves reinforcement continuity, it should be connected to the catenary evidence from Yu and Tan, Kim-Choi, Lim, and Ren. If a model modifies boundary stiffness or introduces equivalent springs, it should be connected to Farhang Vesali and Lim. If a model predicts large post-peak response, it should be checked against Sasani and the bar fracture observations. If the model uses concentrated loading, its limitations should be discussed in light of Qian et al. This source-to-model traceability will make the thesis more coherent and technically defensible.

Sample	Related paper / role	Geometry and scale	Reinforcement / detailing	Concrete and steel data	Loading and boundary conditions	Main results	Failure mode / technical interpretation
S1	Yu and Tan (2011); seismic detailing reference specimen.	Half-scale RC beam-column subassembly; two beams framing into a middle joint; beam 150 x 250 mm; middle column 250 x 250 mm; net beam span 2750 mm; total span 5750 mm.	Top reinforcement at middle joint: 1T13 + 2T10 (approx. 0.90%); bottom: 2T10 (approx. 0.49%); bottom reinforcement continuous; Class B splice; stronger joint confinement with R6 stirrups, including three-legged R6 @ 40 mm at middle joint.	Concrete: f_c approx. 31.2 MPa; splitting tensile strength approx. 3.2 MPa; E_c approx. 27.7 GPa. Steel: T10 f_y approx. 511 MPa, f_u approx. 731 MPa; T13 f_y approx. 527 MPa, f_u approx. 640 MPa; R6 f_y approx. 310 MPa.	Quasi-static displacement-controlled vertical loading applied at the middle joint after simulating middle-column removal. End conditions provided axial and rotational restraint; instrumentation included LVDTs, load cells, and reinforcement strain gauges.	Pure flexural resistance approx. 37.01 kN. Progressive collapse resistance: CAA approx. 38.2 kN; catenary action approx. 65.8 kN. Dynamic increase factor approx. 1.09-1.10 at CAA and approx. 1.05 at catenary action.	Clear sequence of flexural action -> CAA -> catenary action. Damage localized near beam-joint interfaces. Bottom bar fracture occurred near the middle joint; final failure governed by reinforcement fracture. Seismic detailing improved local confinement but did not radically change global capacity compared with S2.
S2	Yu and Tan (2011); non-seismic detailing comparison specimen.	Half-scale RC beam-column subassembly with the same global geometry as S1; beam 150 x 250 mm; middle column 250 x 250 mm; net beam span 2750 mm; total span 5750 mm.	Top reinforcement at middle joint: 3T10 (approx. 0.73%); bottom: 2T10 (approx. 0.49%); bottom reinforcement lap-spliced; Class A splice; less favorable joint confinement and anchorage detailing compared with S1.	Concrete and reinforcement were from the same material batch as S1: f_c approx. 31.2 MPa; T10 f_y approx. 511 MPa; R6 f_y approx. 310 MPa.	Same test rig and loading procedure as S1; quasi-static vertical displacement at middle joint with axial and rotational restraints monitored by LVDTs and load cells.	Pure flexural resistance approx. 34.02 kN. Progressive collapse resistance: CAA approx. 34.9 kN; catenary action approx. 64.6 kN. Dynamic increase factors similar to S1.	The specimen developed the same mechanism sequence as S1. No lap-splice failure was reported. The test indicated that, for these slender specimens where shear failure was avoided, global catenary capacity was governed more by reinforcement continuity and large-deformation response than by seismic confinement alone.
S3	Yu and Tan (2013);	Half-scale RC beam-	Top reinforcement	Concrete: f_c approx.	Same middle-column-	Flexural resistance	Demonstrated higher CAA and

	influence of top/bottom reinforcement ratio.	column subassembly; beam 150 x 250 mm; effective depth approx. 215 mm; net span 2750 mm; span-to-depth ratio approx. 23.	t ratio approx. 1.24%; bottom reinforcement t ratio approx. 0.49%; lap splice through middle joint reported.	38.2 MPa; tensile strength approx. 3.5 MPa; E _c approx. 29.6 GPa. Steel: T10/T13/T16 properties reported in source; T13 f _y approx. 494 MPa, f _u approx. 593 MPa.	removal quasi-static displacement-controlled loading scheme; added rotational restraint at middle joint to suppress asymmetric rotation.	approx. 40.89 kN; CAA peak approx. 54.47 kN; catenary peak reported approx. 124.37 kN at top-bar fracture stage.	CA than earlier lower-reinforced references. Catenary action became significant after axial force changed from compression to tension.
S4	Yu and Tan (2013); bottom reinforcement effect.	Half-scale subassembly; beam 150 x 250 mm; net span 2750 mm; span-to-depth ratio approx. 23.	Top reinforcement t ratio approx. 1.24%; bottom reinforcement t ratio approx. 0.82%.	Concrete: f _c approx. 38.2 MPa. Steel properties as reported for corresponding deformed bars.	Quasi-static vertical loading at middle joint under maintained axial restraint.	Flexural resistance approx. 47.76 kN; CAA peak approx. 63.22 kN; catenary-stage peak approx. 103.68 kN.	Increasing bottom reinforcement improved flexural/CAA response but catenary capacity depended on deformation capacity and reinforcement fracture sequence.
S5	Yu and Tan (2013); high bottom reinforcement comparison.	Half-scale subassembly; beam 150 x 250 mm; net span 2750 mm; span-to-depth ratio approx. 23.	Top reinforcement t ratio approx. 1.24%; bottom reinforcement t ratio approx. 1.24%.	Concrete: f _c approx. 38.2 MPa; reinforcement material properties as tested.	Same ALP middle-column-removal procedure.	Flexural resistance approx. 56.61 kN; CAA peak approx. 70.33 kN; catenary-stage peak approx. 105.07 kN.	Higher bottom reinforcement increased early resistance but did not proportionally increase final catenary capacity because the latter was controlled by fracture and rotation capacity.
S6	Yu and Tan (2013); high top reinforcement ratio.	Half-scale subassembly; beam 150 x 250 mm; net span 2750 mm; span-to-depth ratio approx. 23.	Top reinforcement t ratio approx. 1.87%; bottom reinforcement t ratio approx. 0.82%; lap splice through middle joint reported.	Concrete: f _c approx. 38.2 MPa. T16 bars: f _y approx. 513 MPa, f _u approx. 612 MPa; ultimate strain approx. 13.43%.	Same quasi-static displacement-controlled middle-joint loading.	Flexural resistance approx. 61.97 kN; CAA peak approx. 70.33 kN; catenary capacity approx. 143.28 kN.	Higher top reinforcement strongly enhanced catenary resistance, confirming that tensile reinforcement continuity and capacity are decisive in the large-deformation stage.
S7	Yu and Tan (2013); span-to-	Half-scale subassembly; beam	Top reinforcement t ratio	Concrete: f _c approx. 38.2 MPa.	Same loading rig; middle-joint vertical	Flexural resistance approx.	Lower span-to-depth ratio improved CAA

	depth ratio effect; also later modeled in ATENA report.	150 x 250 mm; net span 2150 mm; span-to-depth ratio approx. 18.2.	approx. 1.24%; bottom reinforcement ratio approx. 0.82%.	Steel properties as in source material table.	displacement applied quasi-statically.	61.09 kN; CAA peak approx. 82.82 kN; catenary-stage peak approx. 105.99 kN.	due to higher axial stiffness and arching efficiency. Catenary still developed but with less relative dominance than long-span specimens.
S8	Yu and Tan (2013); short-span/shear-sensitive specimen; also later modeled in ATENA report.	Half-scale subassembly; beam 150 x 250 mm; net span 1550 mm; span-to-depth ratio approx. 13.4.	Top reinforcement ratio approx. 1.24%; bottom reinforcement ratio approx. 0.82%.	Concrete: f_c approx. 38.2 MPa. Steel properties as in source material table.	Same quasi-static ALP loading procedure.	Flexural resistance approx. 84.74 kN; CAA peak approx. 121.34 kN; catenary capacity approx. 91.83 kN; catenary onset occurred at smaller displacement than longer spans.	Short span strongly increased CAA, but premature shear-dominated behavior limited catenary benefit. This specimen is important because it shows that catenary action cannot compensate for inadequate shear capacity.
CL-13	Qian et al. (2022); concentrated loading; $L/h = 13$.	Half-scale double-span beam-column subassembly; beam 250 x 150 mm; middle column 250 x 250 mm; span-to-depth ratio 13.	Beam longitudinal reinforcement: A-A top 3T12, bottom 2T12; B-B top 2T12, bottom 2T12; non-seismic curtailment; R6 transverse links.	T12: f_y approx. 438 MPa, f_u approx. 577 MPa, elongation 16.6%; R6: f_y approx. 348 MPa, f_u approx. 486 MPa; concrete f_c approx. 30.1-32.5 MPa across specimens.	Concentrated loading regime: middle support removed first, then displacement-controlled load applied at middle column stub.	YS/FPS/US approx. 33/43/81+ kN; critical displacements approx. 45/108/731 mm; max compression approx. -153 kN; max tension approx. 148 kN; ultimate rotation approx. 0.23 rad.	Critical fracture near beam end at middle column (BENM). CL regime produced high deformation and catenary capacity but may overestimate realistic response.
CL-11	Qian et al. (2022); concentrated loading; $L/h = 11$.	Half-scale subassembly; beam 250 x 150 mm; span-to-depth ratio 11.	Same reinforcement layout as CL-13.	Same material family as Qian specimens.	Concentrated middle-joint displacement after prior support removal.	YS/FPS/US approx. 37/52/94 kN; critical displacements approx. 36/90/712	Higher early strength than $L/h = 13$ due to lower span-to-depth ratio. CA still developed

						mm; max compression approx. - 178 kN; max tension approx. 154 kN; ultimate rotation approx. 0.26 rad.	strongly under CL regime.
CL-8	Qian et al. (2022); concentrated loading; $L/h = 8$.	Half-scale subassembly; beam 250 x 150 mm; span-to-depth ratio 8.	Same reinforcement layout as other Qian specimens.	Same material family as Qian specimens.	Concentrated displacement-controlled loading at removed-column location.	YS/FPS/US approx. 53/77/88 kN; critical displacements approx. 25/79/551 mm; max compression approx. - 202 kN; max tension approx. 147 kN; ultimate rotation approx. 0.28 rad.	Shorter span increased early-stage strength; final CA capacity did not rise proportionally because failure was still governed by local reinforcement/fracture sequence.
Specimen 1	FarhangVesali et al. (2013) / Nima section; reference without top add bar.	2/5-scale longitudinally restrained RC beam assemblage representing exterior beam after middle-column loss; exact dimensions from figure; support system provided axial and rotational restraint.	No top additional bar; N10 longitudinal reinforcement; R6 stirrups; stirrup spacing over end third = 180 mm; middle third = 180 mm.	Concrete strength at testing approx. 30.5 MPa; longitudinal bars nominal 500 MPa cold-rolled ribbed; stirrups nominal 250 MPa round bars; grout compressive strength approx. 45 MPa.	Monotonic mid-span displacement at 0.1 mm/s; fully restrained ends using grouted gaps, 20 mm Macalloy bars and bolted double C-channels; rotations measured by inclinometers.	Peak load approx. 40.5 kN; first tensile-bar rupture displacement approx. 80-90 mm. FE with strain penetration predicted approx. 40.1 kN.	Typical arching behavior; first major cracks and first tensile-bar rupture near centre stub. Used as baseline for arching action.
Specimen 2	FarhangVesali et al. (2013); stirrup	Same scaled restrained-beam assemblage	No top additional bar; end-third stirrup	Concrete strength approx. 27.0 MPa; same	Same displacement-controlled	Peak load approx. 35.7 kN; first rupture	Lower concrete strength reduced peak load, confirming

	spacing reduced to 120 mm.	geometry as Specimen 1.	spacing = 120 mm.	nominal reinforcement grades.	restrained-beam test.	approx. 74-85 mm; FE with strain penetration approx. 36.5 kN.	concrete compressive strength is a major parameter for arching capacity.
Specimen 3	FarhangVesali et al. (2013); closest stirrup spacing without add bar.	Same scaled restrained-beam assemblage geometry.	No top additional bar; end-third stirrup spacing = 60 mm.	Concrete strength approx. 30.0 MPa; same nominal reinforcement grades.	Same loading and support conditions.	Peak load approx. 41.4 kN; first rupture approx. 80-90 mm; FE with strain penetration approx. 41.1 kN.	Closer stirrup spacing improved confinement locally but had only minor influence on arching capacity when brittle shear failure was prevented.
Specimen 4	FarhangVesali et al. (2013); top add bar with wide stirrup spacing.	Same scaled restrained-beam assemblage geometry.	Top additional N10 bar present; end-third stirrup spacing = 180 mm.	Concrete strength approx. 26.0 MPa; same nominal reinforcement grades.	Same displacement-controlled pushdown test.	Peak load approx. 34.5 kN; first rupture approx. 75-85 mm.	Additional top reinforcement did not compensate for lower concrete strength because the response was arching-controlled.
Specimen 5	FarhangVesali et al. (2013); top add bar with 120 mm stirrups.	Same scaled restrained-beam assemblage geometry.	Top additional bar present; end-third stirrup spacing = 120 mm.	Concrete strength approx. 29.5 MPa.	Same restrained-beam setup.	Peak load approx. 37.0 kN; first rupture approx. 80-90 mm.	Moderate increase relative to Specimen 4 mainly associated with higher concrete strength; reinforcement ratio effect remained secondary.
Specimen 6	FarhangVesali et al. (2013); top add bar with 60 mm stirrups.	Same scaled restrained-beam assemblage geometry.	Top additional bar present; end-third stirrup spacing = 60 mm.	Concrete strength approx. 30.0 MPa.	Same setup and loading.	Peak load approx. 39.5 kN; first rupture approx. 85-91 mm.	Closest stirrup spacing slightly improved deformation/ductility, but peak arching strength was still governed mainly by concrete strength and strain penetration.
S7 - ATENA model	Clement Huilie report; numerical reproduction of Yu and Tan S7.	RC beam-column substructure corresponding to Yu and Tan S7; half-scale; shorter span case with span-to-	Reinforcement based on experimental S7: top ratio approx. 1.24%, bottom ratio approx. 0.82%; later	Material properties calibrated in ATENA rather than independently tested in the report. Model	ATENA 2D nonlinear displacement-controlled simulation at removed-column location; supports and	Numerical curve Test B16 selected as calibrated response for S7; reproduced global	Used to demonstrate that nonlinear FE modeling can reproduce flexural, arching and catenary stages. Reinforcement symmetry and

		depth ratio approx. 18.2.	modified to study symmetry and continuity effects.	parameters from best calibration: mesh 0.1 m; spring stiffness 400 MPa; tensile softening 4.70E-04; crack reduction 0.7; compression softening 0.05.	springs calibrated to reproduce experimental load-displacement curve.	load-displacement response and mechanism transition.	continuity were later studied as robustness improvements.
S8 - ATENA model	Clement Huilie report; numerical reproduction of Yu and Tan S8.	RC beam-column substructure corresponding to Yu and Tan S8; half-scale; shortest span case with span-to-depth ratio approx. 13.4.	Reinforcement based on experimental S8: top ratio approx. 1.24%, bottom ratio approx. 0.82%.	Best calibration parameters: mesh 0.07 m; spring stiffness 200 MPa; tensile softening 4.40E-03; crack reduction 0.6; compression softening 0.14.	ATENA 2D displacement-controlled simulation after central support removal.	Numerical curve Test B18 selected as calibrated response for S8; aimed to reproduce peak load, deformation capacity and failure mechanism.	Important for short-span behavior where CAA is strong but shear/failure localization limits catenary benefit.
3/8-scale exterior two-span beam model	Sasani et al. (2011) / Mehrdad section; experimental and FE bar-fracture study.	Model of second-floor exterior beam from a seven-story RC building; 3/8-scale; beam depth prototype 20 in. (0.51 m); one-way joist floor system; exact specimen dimensions shown in source figures.	Longitudinal reinforcement based on ACI 318 integrity provisions; measured longitudinal bars: f_y approx. 75 ksi (516 MPa), f_u approx. 103 ksi (709 MPa).	Concrete compressive strength approx. 6 ksi (41 MPa); peak compressive strain approx. 0.002; E_c approx. 4400 ksi (30 GPa); tensile strength in model approx. 0.47 ksi (3.2 MPa).	Monotonic displacement-controlled loading at removed-column location; slow initial loading to 0.4 in. (10 mm), then displacement rate approx. 2 in./s (51 mm/s). FE model in DIANA with explicit bond-slip and bar-fracture procedure.	Bottom bar fractures at approx. 6.1 in. (155 mm) and 7.5 in. (190 mm); top bars yielded at approx. 8.5 in. (216 mm); test stopped at 16 in. (406 mm). FE predicted first fracture at approx. 5.9 in. (150 mm).	Bar fracture caused local loss of flexural strength but not immediate collapse. Post-fracture resistance increased through catenary action in top reinforcement. The study is important for modeling sudden discontinuous bar fracture in continuum FE analysis.
FR	Lim et al. (2017); full horizontal	2/5-scale 2-D RC frame; beam 180 x	Standard deformed-bar detailing;	Concrete f_c approx. 32 MPa; T10	Middle-joint displacement control at 6	Flexural resistance approx.	Full restraint allowed strong catenary action.

	restraint reference.	100 mm; beam span approx. 2220 mm; columns 180 x 180 mm, height approx. 1530 mm.	beam top ratio approx. 1.52% (3T10), bottom approx. 1.01% (2T10); column reinforcement approx. 1.78% (4T10 + 4T6).	f_y approx. 507 MPa, f_u approx. 609 MPa, fracture strain approx. 11%; R6 stirrups f_y approx. 400 MPa.	mm/min; axial load approx. 0.3 f_c applied to columns; full horizontal restraint at beam level.	24.5 kN; CAA peak approx. 27.5 kN; CA peak approx. 70.9 kN; CA/Flexural ratio approx. 2.89; MJD at CA peak approx. 540 mm; rotation approx. 0.23 rad.	Failure governed by beam reinforcement fracture rather than column instability.
FR-S	Lim et al. (2017); full restraint with seismic stirrup detailing.	Same global geometry as FR.	Closer stirrup spacing in support/joint regions following Eurocode 8 DCM concept; same main beam reinforcement as FR.	Same concrete and steel family as FR.	Same full-restraint setup and displacement-controlled loading.	Flexural resistance approx. 24.5 kN; CAA peak approx. 26.9 kN; CA peak approx. 68.5 kN; CA/Flexural ratio approx. 2.80; rotation approx. 0.23 rad.	Seismic stirrups reduced joint/shear distortion and improved local behavior, but did not significantly increase global catenary capacity.
FR-R	Lim et al. (2017); round-bar ductility specimen.	Same global geometry as FR.	Round bars used to improve deformation capacity; beam top approx. 2.02% (4R10), bottom approx. 1.33% (2R10 + 1R8), adjusted to maintain comparable strength.	Round bars: R10 f_y approx. 400 MPa, f_u approx. 500 MPa, fracture strain approx. 30%; R8 f_y approx. 300 MPa, f_u approx. 395 MPa, fracture strain approx. 25%; concrete	Same full-restraint loading setup.	Flexural resistance approx. 22.8 kN; CAA peak approx. 24.0 kN; CA peak approx. 76.0 kN; CA/Flexural ratio approx. 3.33; rotation approx. 0.30 rad.	Higher ductility and greater slip delayed fracture and improved deformation capacity, giving the strongest CA enhancement.

				approx. 32 MPa.			
FR-R	Lim et al. (2017); round-bar ductility specimen.	Same global geometry as FR.	Round bars used to improve deformation capacity; beam top approx. 2.02% (4R10), bottom approx. 1.33% (2R10 + 1R8), adjusted to maintain comparable strength.	Round bars: R10 f_y approx. 400 MPa, f_u approx. 500 MPa, fracture strain approx. 30%; R8 f_y approx. 300 MPa, f_u approx. 395 MPa, fracture strain approx. 25%; concrete approx. 32 MPa.	Same full-restraint loading setup.	Flexural resistance approx. 22.8 kN; CAA peak approx. 24.0 kN; CA peak approx. 76.0 kN; CA/Flexural ratio approx. 3.33; rotation approx. 0.30 rad.	Higher ductility and greater slip delayed fracture and improved deformation capacity, giving the strongest CA enhancement.
IR-1	Lim et al. (2017); imperfect restraint on one side.	Same scaled 2-D frame geometry as FR.	Standard deformed-bar detailing as FR.	Same concrete/steel properties as FR.	One side horizontal restraint removed to represent penultimate-column scenario; axial column load applied.	Flexural resistance approx. 24.5 kN; CAA peak approx. 27.2 kN; CA peak approx. 34.1 kN; CA/Flexural ratio approx. 1.39; MJD at CA peak approx. 320 mm.	Catenary action limited by exterior-column pull-in and instability. Failure shifted from beam-controlled to column-controlled.
IR-2	Lim et al. (2017); imperfect restraint on both sides.	Same scaled 2-D frame geometry as FR.	Standard deformed-bar detailing as FR.	Same concrete/steel properties as FR.	Both horizontal restraints removed/limited, representing reduced surrounding-frame restraint.	Flexural resistance approx. 24.5 kN; CAA peak approx. 25.8 kN; CA peak approx. 29.4 kN; CA/Flexural ratio approx. 1.20; MJD	Inadequate restraint prevented full catenary development. Column instability and joint distortion governed collapse resistance.

						at CA peak approx. 513 mm.	
B2	Ren et al. (2016); beam-only control specimen.	1/3-scale continuous beam substructure without slab; beam section approx. 85 x 170 mm; derived from six-storey prototype with 6 m spans.	Beam reinforcement generally 2/8 + 1/6 top at beam end and 2/8 at mid-span/bottom; no slab reinforcement contribution.	Concrete C30; average compressive strength approx. 44 MPa. Rebar yield strengths: $\phi 6$ approx. 387 MPa; $\phi 8$ approx. 390 MPa; $\phi 10$ approx. 370 MPa; ultimate strengths approx. 475, 468 and 560 MPa respectively.	Middle-column-removal simulated by vertical displacement at column stub; beam ends restrained through welded/bolted steel plates; displacement rate 2 mm/min during beam mechanism, 4 mm/min during catenary stage.	Progressive collapse resistances: F_b approx. 34 kN, F_t approx. 17 kN, F_c approx. 58 kN; F_c/F_b approx. 1.71. Cracks initiated at approx. 6 mm; transition around 150 mm; bottom bars ruptured at approx. 421 mm; test ended at approx. 447 mm.	Two-stage beam/catenary response. Final failure governed by simultaneous rupture of bottom rebars at positive-moment region.
B3	Ren et al. (2016); beam-only specimen with increased beam height.	1/3-scale continuous beam substructure without slab; beam width approx. 85 mm; beam height approx. 200 mm.	Same reinforcement philosophy as B2, allowing effect of beam height to be isolated.	Same concrete and reinforcement family as B2.	Same middle-column-removal displacement-controlled test and boundary conditions.	Progressive collapse resistances: F_b approx. 41 kN, F_t approx. 21 kN, F_c approx. 42 kN; F_c/F_b approx. 1.02.	Greater beam height increased beam/flexural mechanism but did not improve catenary capacity proportionally. Failure occurred by top-bar rupture at beam ends due to larger support rotations.
IMF	NIST Technical Note 1720 / Lew et al. (2011). Full-scale Intermediate Moment Frame assembly, extracted from a ten-story RC moment-	Full-scale RC beam-column assembly with three columns and two beams. Each side beam span: 240 in. (6096 mm) centerline-to-centerline. Assembly	Beam top and bottom longitudinal bars were spliced at mid-span using threaded couplers to avoid premature lap-splice failure. Beam	Nominal design materials: 4000 psi (27.6 MPa) concrete and ASTM A706 Grade 60 reinforcement. Measured concrete strength: footings 5700 psi (39	Monotonically increasing vertical displacement was imposed at the center column to simulate support removal. Footings were clamped to the strong floor by ten 1	Initial peak load: 66.5 kip (296 kN) at about 5 in. (127 mm) center-column displacement. Ultimate load: 123 kip (547 kN) at 43.0 in. (1092	The response showed three clear phases: arching action, plastic-hinging/softening, and catenary action. Failure involved crushing/spalling of concrete near the center-column region, major flexural crack widening, and

	frame building designed for Seismic Design Category C.	taken from the third-floor framing system of the prototype building. End-column footings: 88 x 64 x 42 in. (2235 x 1626 x 1067 mm). Buttressing slab between footings: 28 x 10 in. cross-section.	longitudinal bars were anchored at exterior joints using threaded mechanical anchorage devices and 3/4 in. (19 mm) steel plates. Concrete around anchorage devices was confined with spirals made of Grade 40 plain wire. Column longitudinal bars were fully embedded into the footings with 24 in. (610 mm) 90-degree hooks.	MPa), beams/columns 4700 psi (32 MPa), split-cylinder tensile strength 450 psi (3.1 MPa). Representative reinforcement values: #8 bar $f_y = 69$ ksi (476 MPa), $f_u = 94$ ksi (648 MPa), rupture strain 21%; #10 bar $f_y = 73$ ksi (503 MPa), $f_u = 106$ ksi (731 MPa), rupture strain 16%.	1/4 in. (32 mm) post-tensioning bars, each tensioned to 150 kip (667 kN), giving a total clamping force of 1500 kip (6670 kN) per footing. Instrumentation included LVDTs, load cells, position transducers, inclinometers, joint measurement targets, and reinforcement strain gauges.	mm) displacement. Ultimate-to-initial peak load ratio: about 1.85. Rotation at failure: about 0.185 rad, much larger than the seismic-test-based deformation capacity of about 0.026 rad.	fracture of one or more bottom longitudinal beam bars at a major crack near the center column. The specimen had lower absolute capacity than SMF but showed a larger reserve ratio from first peak to ultimate load.
SMF	NIST Technical Note 1720 / Lew et al. (2011). Full-scale Special Moment Frame assembly, extracted from a ten-story RC moment-frame building designed for Seismic Design Category D.	Full-scale RC beam-column assembly with the same general layout as IMF: three columns and two beams, with 240 in. (6096 mm) beam span on each side. The assembly represents a special moment-frame configuration with	Same general anchorage philosophy as IMF: threaded couplers at beam mid-span, mechanical anchorage devices and 3/4 in. steel plates at exterior joints, spiral confinement around anchorage zones, and fully embedded	Nominal design materials: 4000 psi (27.6 MPa) concrete and ASTM A706 Grade 60 reinforcement. Measured concrete strength: footings 5300 psi (37 MPa), beams/columns 5200 psi (36 MPa), split-cylinder tensile strength 420	Monotonically increasing vertical displacement imposed at the center column, with footing clamping and instrumentation strategy equivalent to the IMF test. The test measured global load-displacement response, beam/column movement, joint deformation,	Initial peak load: 203 kip (903 kN) at about 4.4 in. (112 mm) center-column displacement. Ultimate load: 277 kip (1232 kN) at 48.0 in. (1219 mm). Ultimate-to-initial peak load ratio: about 1.36. Rotation at	The SMF specimen developed substantially larger absolute load capacity than IMF because of stronger seismic design and detailing. The final response was again governed by large-deformation mechanisms, with concrete damage near the center column and fracture of bottom longitudinal beam reinforcement. The lower ultimate-to-first-

		stronger seismic detailing and larger resistance demand than the IMF specimen.	column longitudinal reinforcement with 24 in. (610 mm) 90-degree hooks. Compared with IMF, the SMF specimen corresponds to a stronger seismic detailing level and substantially larger force resistance.	psi (2.9 MPa). Reinforcement properties varied by bar size and heat; representative values include #8 and #10 bars with yield strengths around 69-73 ksi and ultimate strengths around 94-106 ksi.	rotations, and reinforcement strains to track transition from arching to catenary behavior.	failure: about 0.200 rad, much larger than the seismic-test-based deformation capacity of about 0.025 rad.	peak ratio compared with IMF shows that stronger initial flexural/arching capacity does not necessarily imply proportionally larger catenary reserve.
--	--	--	--	--	---	--	---

Table 29: Overall summary of the experimental database

CHAPTER 3: Non-Linear Finite Element Analysis

3.1 Introduction

The assessment of reinforced concrete structures subjected to progressive collapse scenarios requires advanced numerical procedures capable of reproducing highly nonlinear structural behavior. Experimental investigations provide valuable information regarding the response mechanisms of reinforced concrete members; however, they are generally limited in terms of the number of specimens, material variability, and possible reinforcement configurations that can be investigated. For this reason, nonlinear finite element analysis (NLFEA) was adopted in the present thesis to reproduce the structural response observed experimentally and extend the investigation toward a probabilistic assessment framework. [7], [13]

The numerical methodology developed in this thesis was directly connected to the experimental campaign presented in Chapter 2. Each tested specimen was initially reproduced numerically using ATENA 2D in order to simulate crack development, stiffness degradation, compressive arch action, and catenary action evolution under increasing displacement demand. Once the numerical models achieved acceptable agreement with the experimental response, the calibrated models were used as the basis for the probabilistic analyses discussed in the following chapter. [8], [13]

Unlike simplified deterministic approaches focused exclusively on ultimate load capacity, the adopted methodology aimed to reproduce the entire structural response, including the transition from flexural behavior to compressive arch action (CAA) and finally to catenary action (CA). This aspect was particularly important because the stability of catenary activation strongly depends on reinforcement continuity, anchorage quality, material variability, and large displacement behavior. Consequently, the nonlinear analysis procedure focused not only on strength prediction but also on mechanism evolution throughout the loading history. [5], [9]

3.2 Numerical Modeling Strategy

ATENA 2D was selected as the numerical solver because it is specifically developed for nonlinear analysis of reinforced concrete structures. The software includes constitutive material models capable of reproducing concrete cracking, crushing, reinforcement yielding, bond-related effects, and post-peak instability under monotonic loading conditions. In addition, ATENA provides displacement-controlled analysis procedures and arc-length stabilization methods that allow stable tracing of highly nonlinear post-peak structural behavior. These features were essential for reproducing the response associated with catenary action development after significant stiffness degradation. [13]

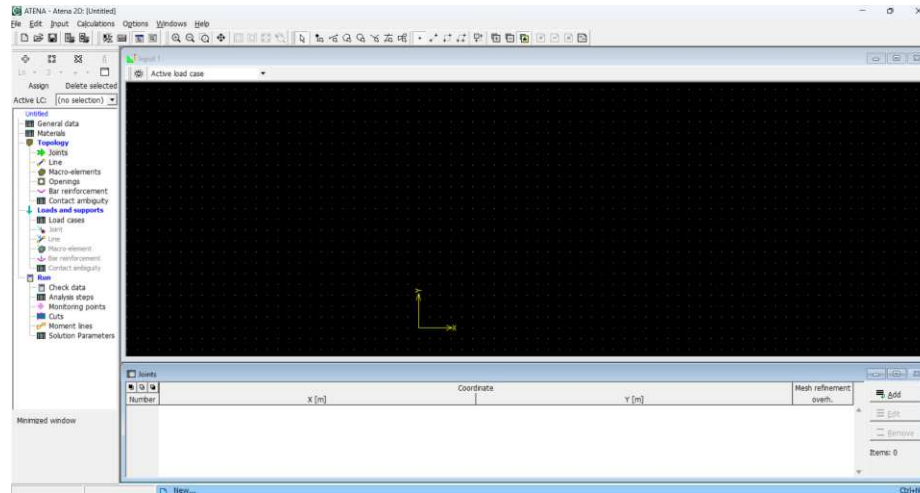


Figure 37: Atena 2D Working Space

The numerical models were developed as two-dimensional beam-column sub-assemblages consistent with the experimental configurations investigated in this thesis. The selected idealization was considered appropriate because the dominant failure mechanisms and load transfer paths developed primarily within the loading plane. The simplified two-dimensional representation significantly reduced computational effort while preserving the main structural characteristics governing progressive collapse resistance. The models included beams, columns, reinforcement layers, support conditions, loading points, and monitoring locations required for extraction of the structural response. [10]

Special attention was given to the representation of boundary conditions because the restraint level strongly influences the development of compressive arch action and the subsequent transition toward catenary behavior. The support conditions were therefore defined in order to reproduce the restraint configuration adopted during the experimental tests. Axial restraint was considered particularly important because it directly affects the compressive membrane forces developed during the intermediate loading stages prior to catenary activation. [12]

Displacement-controlled loading was adopted throughout the nonlinear analyses because it provides improved numerical stability during highly nonlinear post-peak response. Unlike force-controlled procedures, displacement-controlled analysis allows continuous tracing of the structural response even after local stiffness degradation and crack propagation occur. This aspect was essential for capturing the complete evolution of catenary action and the redistribution of internal forces after local damage. [13]

3.3 Material Modeling and Calibration Procedure

Concrete behavior was represented using nonlinear fracture-plastic constitutive models capable of reproducing tensile cracking, crack localization, compressive crushing, and softening behavior. The adopted material model included parameters associated with compressive strength, tensile strength, fracture energy, elastic modulus, and crack propagation characteristics. These parameters directly controlled the evolution of damage and stiffness degradation within the structural members. Since the transition from flexural response toward catenary action is governed by extensive cracking and redistribution of internal forces, accurate representation of concrete nonlinear behavior was considered essential. [13], [14]

Calibration target	Reason for calibration	Typical adjusted input
Initial stiffness	Controls early flexural response and crack initiation	Concrete elastic modulus, tensile parameters, mesh/support representation
First peak / CAA peak	Represents compression-dominated arching stage	Concrete compressive strength, compression softening, support stiffness
Post-peak degradation	Represents cracking, crushing and transition between mechanisms	Tensile local strain, crack reduction, compression softening
Catenary-stage capacity	Represents tensile resistance of reinforcement at large displacement	Steel yield/ultimate strength, ultimate strain, bar fracture assumptions
Boundary effect	Controls axial force development and restraint stiffness	Spring stiffness or equivalent restraint parameters

Table 30: Calibration process summary

The reinforcement behavior was modeled using elastic–plastic constitutive laws including yielding and strain-hardening capabilities. The representation of reinforcement response was particularly important because catenary action develops primarily through tensile membrane forces generated in the reinforcement after large structural deformations. Consequently, the reinforcement properties directly influenced the stability of the post-peak response and the ability of the structure to maintain load-carrying capacity after flexural failure mechanisms had fully developed. [7]

The numerical calibration process constituted the first stage of the nonlinear finite element methodology. Each experimental specimen was calibrated independently by comparing the numerical and experimental load–displacement curves and progressively adjusting the material parameters until satisfactory agreement was achieved. The calibration process focused not only on matching the ultimate capacity but also on reproducing the entire response history, including stiffness degradation, crack evolution, mechanism transition, and post-peak behavior. This approach ensured that the calibrated numerical models reproduced the governing physical mechanisms observed experimentally. [6], [8]

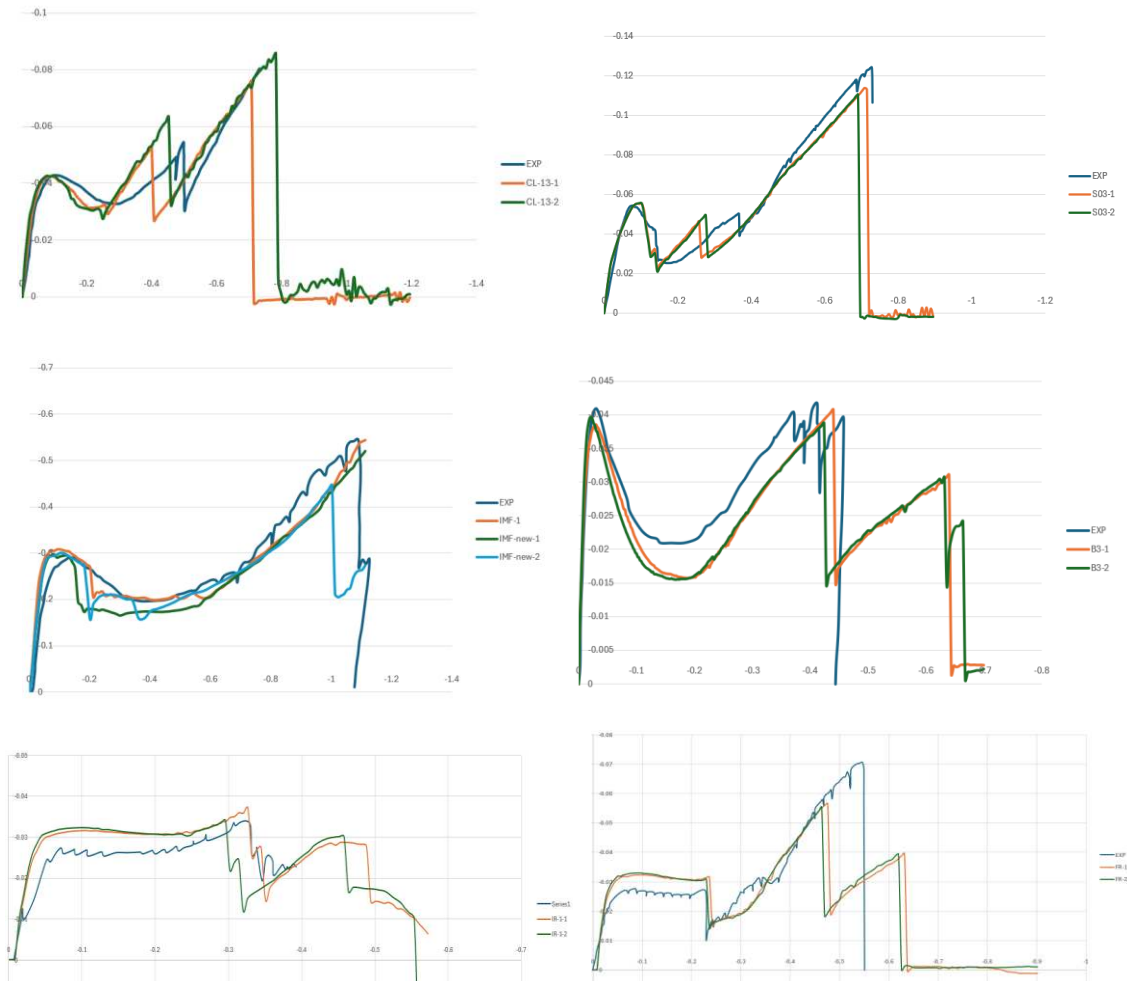


Figure 38: Some graphs (Capacity Curves) related to calibration of the model with respect to its experimental data

Particular attention was devoted to reproducing the transition between flexural action, compressive arch action, and catenary action observed experimentally. The calibrated models therefore represented not only the load-carrying capacity of the investigated sub-assemblages but also the progressive redistribution of internal forces occurring after significant cracking and stiffness degradation. The ability to reproduce the post-peak

response was considered a critical requirement because the probabilistic framework developed later in the thesis relied directly on the stability of the simulated catenary response. [15]

3.4 MATLAB–ATENA Integrated Workflow

The calibrated deterministic models subsequently served as the basis for the probabilistic framework developed in this thesis. Following calibration, the material parameters associated with each specimen were exported into Excel-based templates containing the constitutive properties used in the ATENA models.

[illegible]

Figure 39: Examples of Excel-based templates for calibration of the samples (gray rows where the parameters that would be calibrated and be probabilistically changed after running the MATLAB Code (LHS))

These templates constituted an intermediate data management layer connecting the deterministic finite element models with the MATLAB-based probabilistic workflow. The use of Excel significantly improved consistency between simulations and reduced the possibility of manual input errors during the generation of large numerical datasets. [16]

```

1 |
2 %INPUT
3 %Number of samples for LHS
4 N=30;
5 %Data
6
7 fileID = fopen('filename.txt','r');
8 filename = fscanf(fileID,'%s\n');
9 fclose(fileID);
10
11 A = readmatrix(filename,'Sheet',1,'Range','AH6:AH27')
12 % A = xlsread('WT2.xlsx' filename,'Materials_Geometry','AH6:AH14');
13
14 %Correlation matrix Ro in input
15 %   fc  fy  fu  Es  epsu
16 %fc  1  0  0  0  0
17
18 %fy  0  1  ..  0  ..
19
20 %fu  0  ..  1  0  ..
21
22 %Es  0  0  0  1  0
23
24

```

Figure 40: A view of the LHS MATLAB code

A dedicated MATLAB framework was developed in order to automate the generation of probabilistic realizations and integrate them with the nonlinear finite element models. The developed scripts operated directly on the Excel templates associated with each calibrated specimen and automatically generated probabilistic material realizations through Latin Hypercube Sampling (LHS). The generated parameters were then transferred into ATENA-compatible CCT files used for nonlinear analysis. This automated integration enabled efficient management of a large number of nonlinear simulations while preserving consistency between the experimental database and the probabilistic framework. [17]

Script	Input	Core operation	Output
LHS.m	filename.txt and calibrated Excel file	Generates 30 correlated lognormal material samples using Latin Hypercube Sampling	Writes 22 x 30 material matrix
CCT.m	filename.txt and calibrated Excel file	Exports each selected simulation column of the template as one ATENA CCT text file	Sequential CCT files named 1.cct, 2.cct, etc.

Table 31: MATLAB scripts division

[illegible]

[illegible]

Figure 41: Excel-based templates before and after running the LHS MATLAB code

```

FILETYPE ATENA_TEXT
VERSION <version, currently 0>

BEG_HEADER
  DESCRIPTION "<task name>"
  COMMENT "<comment to task>"
  NUMPLANES <no. of layers of smeared material>
  STRUCTTYPE "<analysis type, 2D or 3D>"
END_HEADER

BEG_MATERIALS
COUNT <no. of materials>

BEG_MAT <material number id>
  ver_mat <version zápisu materiálu>
  <CCM file>
END_MAT <material number id>

END_MATERIALS

BEG_TOPOLOGY

  BEG_JOINTS
  COUNT <number of joints>
  <joint no.> <X> <Y> for each joint
  END_JOINTS

```

Materials

```

Materials
Material n. 1
Name : SBeta Material
Type: COSEBETAMaterial
Elastic modulus E = 3.1702E+04 [MPa]
Poisson's ratio am = 0.200 [-]
Tensile strength Ft = 1.640E+08 [MPa]
Compressive strength Fc = -2.848E+01 [MPa]
Type of tension softening : Exponential
Specific fracture energy Gf = 1.000E-04 [MJ/m]
Crack model: Flied
Compressive strain at comp. strength in the uniax. test EpsC = -1.795E-03 [-]
Reduction of compressive strength due to cracks CompRed = 0.800 [-]
Type of compression softening : Crush Band
Critical compressive displacement Wd = -5.0000E-04 [m]
Shear Retention Factor Varshe = 0.000 [-]
Tension-compression interaction : Linear
Specific material weight Rho = 2.300E-03 [MN/m3]
Coefficient of thermal expansion Alpha = 1.200E-05 [1/K]

Material n. 2
Name : Plane Stress Elastic Isotropic
Type: OCPlaneStressElastIsotropic
Elastic modulus E = 2.000E+05 [MPa]
Poisson's ratio am = 0.300 [-]
Specific material weight Rho = 7.850E-03 [MN/m3]
Coefficient of thermal expansion Alpha = 1.200E-05 [1/K]

Material n. 3
Name : Reinforcement
Type: OCReinforcement
Typ: BiLinear
Elastic modulus E = 2.080E+05 [MPa]
Sigmak = 560.000 [MPa]
Specific material weight Rho = 7.850E-03 [MN/m3]
Coefficient of thermal expansion alEPA = 1.200E-05 [1/K]

```

Figure 42: Part of Atena CCT file Configuration

CCT block / input item	Technical content	Use in this thesis
BEG_HEADER	Task name, comment, number of smeared reinforcement layers, structure type	Identifies each generated sample and preserves the 2D model definition
BEG_MATERIALS	Concrete, reinforcement, interface or spring material definitions	Receives calibrated and LHS-generated material parameters
BEG_TOPOLOGY	Joints, lines, macro-elements, openings and reinforcement geometry	Preserves specimen geometry and reinforcement layout

CCT block / input item	Technical content	Use in this thesis
BEG_LOADING	Supports, forces, prescribed deformations and load cases	Represents column-loss pushdown/loading scenario
Analysis steps	Load cases, solution methods and coefficients	Controls nonlinear incremental procedure
Monitoring points	Nodal displacements and reactions	Provides data for capacity curves, P_u and P_o extraction

Table 32: Atena CCT file contents

The probabilistic framework considered uncertainty in the main material properties governing structural response. The generated random variables included concrete compressive strength (F_c), tensile strength (F_t), elastic modulus (E), and reinforcement properties. The statistical distributions were centered around the calibrated mean values derived from the deterministic numerical models. The LHS technique was adopted because it provides efficient statistical coverage of the parameter space while requiring fewer simulations compared to conventional Monte Carlo methods. [18], [19]

Stage	Operation	Main output	Role in thesis
1	Original ATENA model preparation	Reference model/CCT file for each experimental sample	Preserves geometry, reinforcement layout, boundary conditions and loading scheme
2	Calibration of Excel/CCT input	Calibrated baseline Excel file	Ensures agreement between numerical and experimental capacity curves
3	LHS material sampling in MATLAB	Thirty material-variable sub-samples per sample	Introduces material uncertainty while keeping geometry and reinforcement unchanged
4	CCT regeneration from Excel	Importable CCT files	Creates the model input files for ATENA analyses
5	ATENA 2D analysis	Load-displacement/capacity curves for sub-samples	Provides numerical output for Chapter 4 probabilistic analysis

Table 33: Summary of the numerical workflow adopted in the thesis, from the preparation and calibration of the original ATENA model to material sampling, CCT regeneration, and ATENA 2D analyses for obtaining probabilistic capacity curves.

Modeling component	ATENA concept	Influence on capacity curve	Relevance to thesis
Concrete tension	Smearred cracking, tensile strength, tension softening	Controls first cracking and stiffness degradation	Flexural stage and crack localization
Concrete compression	Nonlinear compression, crushing and post-peak softening	Controls CAA peak and compression failure	Arching action and concrete crushing
Cracked concrete shear	Shear retention factor	Controls shear transfer across cracked regions	Damage propagation and post-peak branch
Reinforcement	Bilinear or multi-linear steel law	Controls yielding, tensile reserve and bar failure	Flexural capacity and catenary action
Bond/anchorage	Bond-slip laws or effective interaction assumptions	Controls force transfer between steel and concrete	Joint integrity and catenary activation
Supports/springs	Boundary conditions and spring stiffness	Controls axial restraint and horizontal displacement	CAA and CA development

Table 34: Main ATENA modelling components used in the nonlinear finite element analysis and their influence on the capacity-curve response, highlighting their relevance to flexural behaviour, concrete crushing, shear transfer, reinforcement yielding, ...

For each calibrated specimen, thirty probabilistic sub-samples were generated and analyzed. Considering the twenty-six baseline experimental specimens investigated in the thesis, the developed workflow resulted in a total of 780 nonlinear finite element analyses for the original reinforcement configuration alone. Additional analyses were also performed for the modified reinforcement arrangements investigated in the probabilistic assessment phase. The automated workflow significantly reduced manual modeling effort and enabled systematic investigation of the influence of material uncertainty on catenary activation and progressive collapse resistance. [20]

3.5 Nonlinear Analysis Procedure and Structural Response

The generated material realizations were automatically written into the Excel templates and subsequently converted into ATENA-compatible CCT files through dedicated MATLAB routines. Each generated CCT file represented a unique nonlinear finite element model with distinct probabilistic material properties while preserving the same geometry,

loading configuration, support conditions, and reinforcement arrangement associated with the baseline specimen. This procedure ensured that the observed variability in structural response originated directly from material uncertainty rather than inconsistencies in the structural model itself. [16]

The CCT files generated through MATLAB were imported directly into ATENA for nonlinear finite element analysis. Incremental–iterative solution procedures combined with arc-length stabilization techniques were employed in order to ensure convergence during highly nonlinear post-peak stages characterized by severe stiffness degradation and instability. Displacement-controlled loading was adopted because it allows continuous tracing of the structural response even after the occurrence of local cracking, yielding, and softening mechanisms. This aspect was essential for capturing the complete evolution of catenary action. [16]

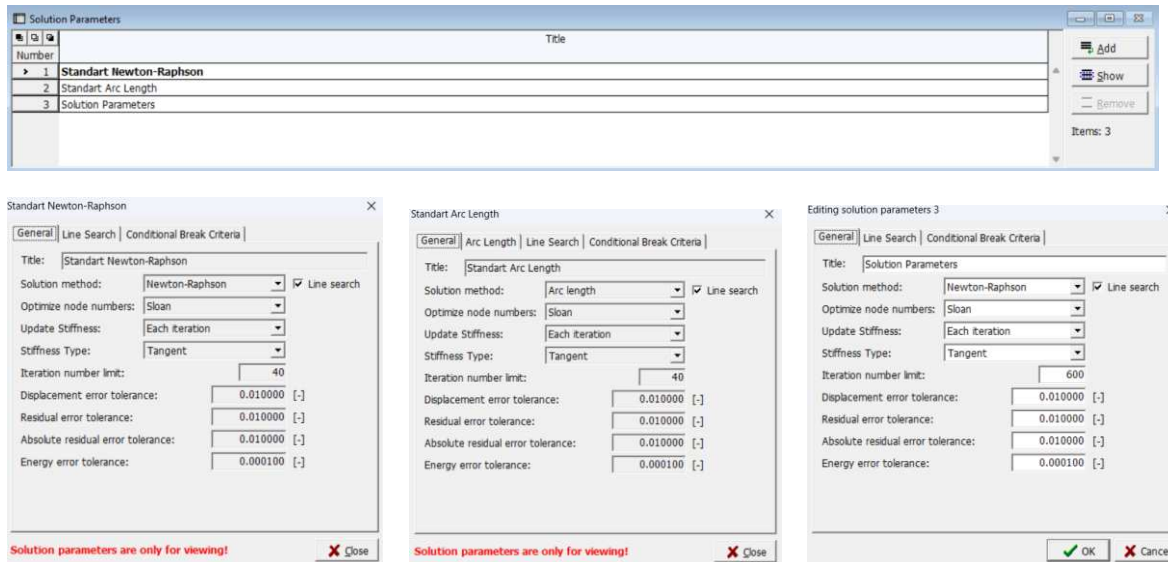


Figure 43: Indication of solution parameters for the sample B3

The structural response associated with each nonlinear analysis was extracted through predefined monitoring points recording displacement and reaction forces throughout the loading history. The resulting load–displacement curves constituted the primary output used for probabilistic interpretation of the structural behavior. Particular attention was given to the identification of characteristic response points associated with mechanism transition and catenary activation. These parameters were later used for probabilistic assessment of structural robustness. [16]



Figure 44: Pre-defined monitoring points for the sample B3

The parameter P_o was associated with the first significant resistance peak observed in the structural response, generally corresponding to the transition from flexural behavior toward compressive arch action. The parameter P_u represented the ultimate resistance associated with fully developed catenary behavior during the final response stage. The comparison between P_o and P_u provided important information regarding the stability of catenary activation and the residual load-carrying capacity of the investigated sub-assemblages after significant damage development. [6], [9]

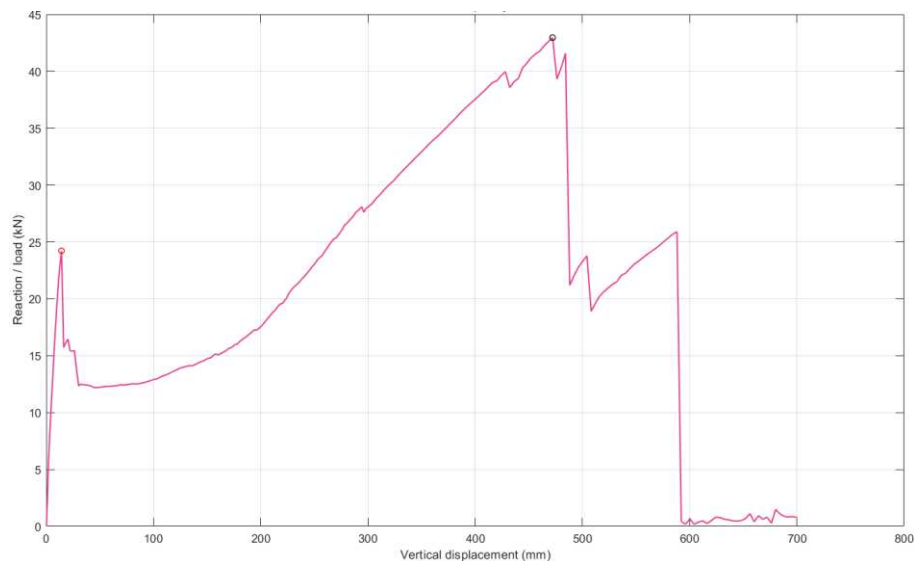


Figure 45: Indication of P_o and P_u

The generated numerical database demonstrated that the development of catenary action is highly sensitive to reinforcement detailing, anchorage quality, and material variability. In several probabilistic realizations, the post-peak response associated with catenary action exhibited significant variability compared to the initial flexural stage. This observation

confirmed the importance of adopting a probabilistic interpretation framework rather than relying exclusively on deterministic peak-load evaluations. [2], [15]

3.6 Concluding Remarks

The nonlinear finite element framework developed in this thesis established a direct connection between experimentally calibrated deterministic models and probabilistic structural assessment. The proposed methodology integrated experimental validation, nonlinear constitutive modeling, automated parameter generation, MATLAB-based processing, and ATENA nonlinear analysis within a unified computational workflow. The resulting numerical database provided the basis for the statistical analyses and probabilistic investigations presented in Chapter 4. [13], [18]

The adopted methodology demonstrated that nonlinear finite element analysis can provide reliable interpretation of progressive collapse mechanisms when combined with appropriate calibration procedures and probabilistic assessment techniques. In particular, the integration between ATENA and MATLAB enabled efficient generation and management of a large nonlinear numerical database while maintaining consistency with the experimental observations presented in the thesis. [17], [20]

CHAPTER 4: Probabilistic Analysis Framework

4.1 Introduction

The deterministic interpretation of progressive collapse resistance is generally insufficient for describing the actual structural variability associated with reinforced concrete sub-assemblages subjected to column-loss scenarios. Experimental investigations have demonstrated that specimens with similar geometry may exhibit significantly different post-peak responses depending on reinforcement detailing, anchorage conditions, material properties, and nonlinear mechanism evolution. For this reason, a probabilistic framework was developed in the present thesis in order to quantify the variability of the structural response and evaluate the probability of catenary action activation under uncertain material conditions. [15], [20]

The probabilistic methodology adopted in this work was directly connected to the nonlinear finite element framework described in Chapter 3. After calibration of the deterministic ATENA models against the experimental results, probabilistic material distributions were introduced through MATLAB-based Latin Hypercube Sampling (LHS). The generated material realizations were subsequently converted into ATENA-compatible CCT files and analyzed numerically in order to produce a large probabilistic database of nonlinear structural responses. [13], [17]

The primary objective of the probabilistic framework was not limited to the evaluation of ultimate resistance. Instead, the developed methodology aimed to investigate the variability associated with the transition from flexural behavior toward compressive arch action and finally to catenary action. Particular attention was therefore devoted to the probabilistic interpretation of the post-peak response and the stability of catenary activation under uncertain material conditions. [7], [9]

4.2 Definition of the Probabilistic Framework

The probabilistic framework was established on the basis of the experimentally calibrated nonlinear finite element models presented in the previous chapter. Each calibrated specimen represented a deterministic baseline configuration from which multiple probabilistic realizations were generated. The uncertainty was introduced primarily through the variability of the material parameters governing the nonlinear constitutive behavior of concrete and reinforcement. [18]

The selected random variables included concrete compressive strength, tensile strength, elastic modulus, and reinforcement properties associated with yielding and strain-hardening behavior. These variables were selected because they directly influence crack development, stiffness degradation, post-peak resistance, and catenary activation stability.

The probabilistic distributions were centered around the calibrated mean values obtained during the deterministic calibration process. [19], [21]

For each of the twenty-six calibrated experimental specimens, thirty probabilistic sub-samples were generated through MATLAB-based Latin Hypercube Sampling. Consequently, the original reinforcement configuration alone resulted in a total of 780 nonlinear finite element simulations. Additional analyses were later performed for the modified reinforcement arrangements investigated in the thesis. [17], [20]

4.3 Latin Hypercube Sampling and Automated Model Generation

Latin Hypercube Sampling (LHS) was adopted because it provides efficient statistical coverage of the parameter space while requiring a reduced number of simulations compared to conventional Monte Carlo procedures. The method divides the probability space of each variable into intervals with equal probability and generates representative combinations of the input parameters. This approach significantly improves sampling efficiency and reduces clustering effects within the generated probabilistic realizations. [17], [22]

The generated material realizations were automatically written into Excel templates associated with each calibrated specimen. These templates contained the constitutive parameters used by the ATENA numerical models and therefore acted as an intermediate interface between MATLAB and the nonlinear finite element environment. The use of Excel-based parameter management ensured consistency among the generated realizations while preserving the geometry, reinforcement arrangement, loading configuration, and support conditions associated with the baseline specimens[16]

Dedicated MATLAB routines were subsequently used to convert the generated parameter sets into ATENA-compatible CCT files. Each generated CCT file represented a unique nonlinear finite element model characterized by distinct probabilistic material properties. This automated workflow significantly reduced manual modeling effort and enabled systematic generation of a large nonlinear numerical database. [23]

4.4 Extraction of Capacity Curves and Structural Parameters

Following completion of the nonlinear finite element analyses in ATENA, the structural response associated with each probabilistic realization was extracted through predefined monitoring points recording displacement and reaction forces throughout the loading history. The resulting load–displacement curves constituted the primary output used for probabilistic interpretation of the structural behavior. [16]

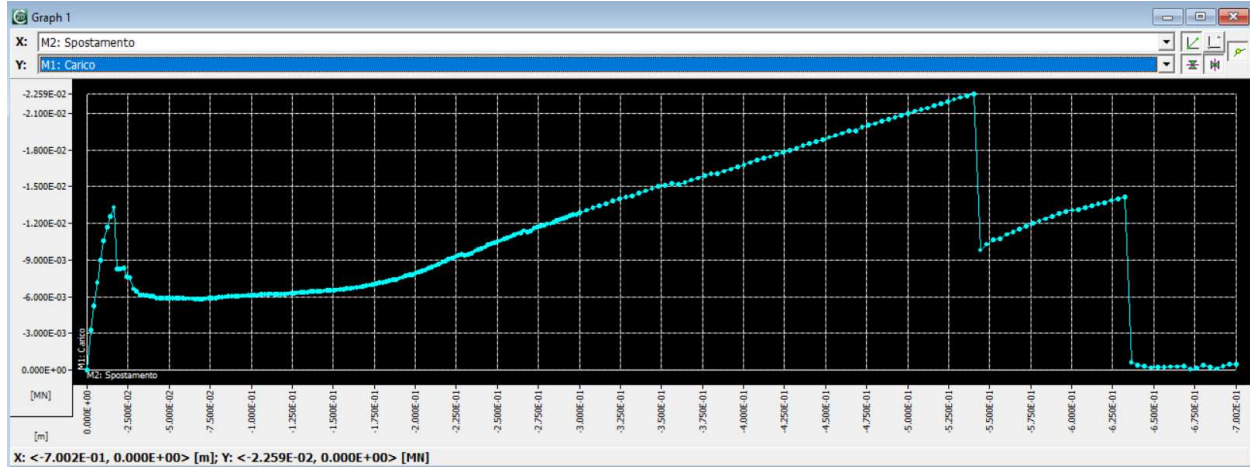


Figure 46: Capacity curve generated after the analysis inside Atena 2D

Particular attention was devoted to the identification of characteristic response points associated with mechanism transition and catenary activation. The parameter P_o represented the first significant resistance peak generally associated with the transition from flexural behavior toward compressive arch action. The parameter P_u represented the ultimate resistance associated with fully developed catenary behavior during the final stage of the response. The comparison between P_o and P_u therefore provided direct information regarding the stability of catenary activation and the residual structural capacity after severe damage development. [6], [9]

The extraction of P_o and P_u values constituted one of the most critical stages of the probabilistic framework because the subsequent statistical interpretation depended directly on the reliability of the detected characteristic points. During the initial stages of the research, a fully automatic MATLAB procedure was developed in order to identify both P_o and P_u directly from the nonlinear capacity curves. The code automatically detected local peaks and attempted to classify the characteristic response points based on smoothing and drop-detection procedures.

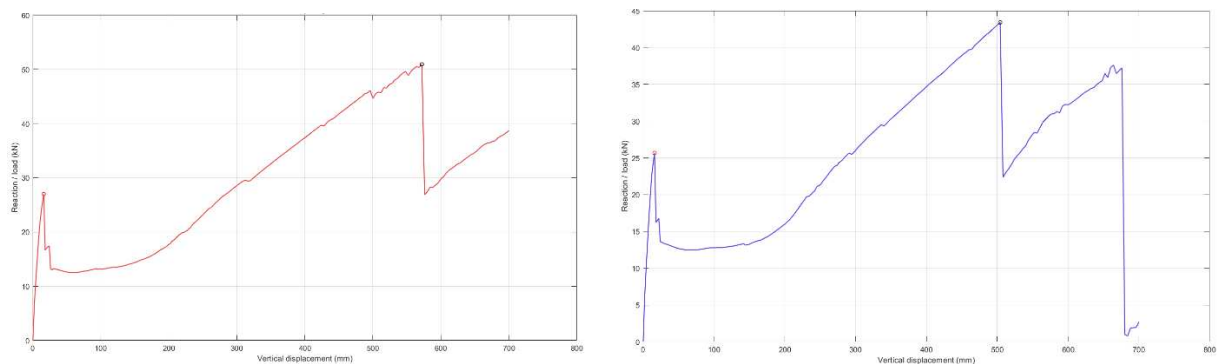


Figure 47: Indication of selected P_o (Red circle) and P_u (Black circle) with automatic MATLAB code

4.5 Statistical Interpretation and Lognormal Fitting

After extraction of the P_o and P_u values for each probabilistic realization, the generated numerical database was statistically processed using dedicated MATLAB routines. Mean values, standard deviations, and variability indicators were calculated for both P_o and P_u in order to quantify the dispersion associated with the nonlinear structural response. The statistical interpretation demonstrated that the catenary-related P_u response generally exhibited larger variability compared to the initial flexural stage represented by P_o . [20]

The probabilistic distributions of P_o and P_u were subsequently approximated through lognormal fitting procedures implemented in MATLAB. Lognormal distributions were selected because structural resistance parameters are positive-definite quantities and typically exhibit multiplicative variability associated with material uncertainty and nonlinear mechanism interaction. Probability density functions (PDFs) and cumulative distribution functions (CDFs) were generated for each reinforcement arrangement and specimen configuration. [18]

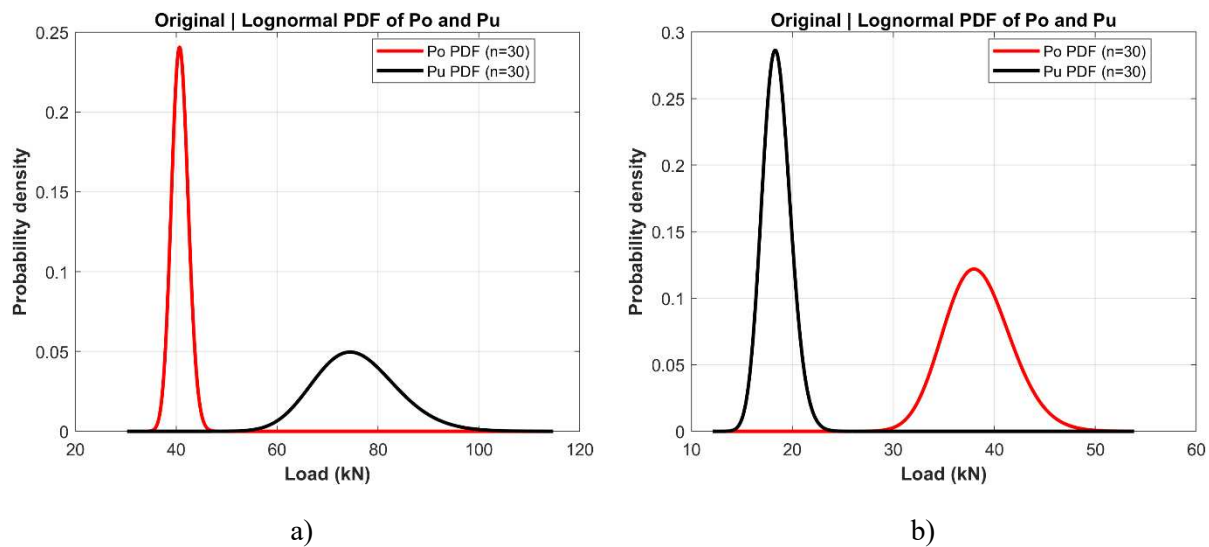


Figure 48: PDF graphs generated based on lognormal for samples a) Specimen-02, b) S2

The MATLAB procedure developed for lognormal fitting automatically imported the P_o and P_u values from the Excel result tables generated during the nonlinear analyses. The script identified the reinforcement arrangements associated with each specimen, filtered the accepted realizations, and generated probabilistic curves describing the statistical distribution of the structural capacities. The resulting plots enabled direct comparison between the flexural stage and the catenary stage of the response.

The probabilistic interpretation additionally enabled evaluation of the probability of catenary activation. The activation probability was defined through comparison between

the probabilistic distributions associated with Po and Pu. Stable catenary activation was associated with realizations characterized by a physically meaningful secondary resistance increase after the initial degradation stage. The resulting probabilistic curves therefore provided direct information regarding the robustness sensitivity of the investigated specimens under uncertain material conditions. [7], [15]

4.6 Merging of Probabilistic Results and Global Comparison

Following statistical processing of the individual specimens, an additional MATLAB procedure was developed in order to merge and compare the probabilistic activation curves associated with all investigated samples. The implemented code automatically entered each sample folder, detected the Po/Pu Excel result files, imported the corresponding statistical parameters, and generated unified probabilistic plots combining all specimens within the same graphical representation.

The merged probabilistic plots constituted one of the most important outputs of the research because they enabled direct comparison between the robustness characteristics of the different experimental groups. The generated figures used unique colors, markers, and line styles for each specimen in order to improve visual interpretation of the probabilistic trends. The resulting plots additionally allowed direct comparison between the original reinforcement configuration and the modified reinforcement arrangements investigated in the thesis.

The merged probabilistic interpretation demonstrated significant differences between the investigated specimens in terms of catenary activation stability and post-peak robustness. Specimens characterized by improved reinforcement continuity and stronger anchorage generally exhibited higher activation probabilities and more stable post-peak behavior. Conversely, specimens characterized by weaker detailing frequently exhibited unstable probabilistic trends and lower catenary activation reliability. [2], [5]

4.7 Concluding Remarks

The probabilistic framework developed in this thesis combined experimentally calibrated nonlinear finite element models, automated MATLAB-based parameter generation, statistical post-processing, and engineering interpretation within a unified computational methodology. The resulting framework enabled systematic evaluation of the variability associated with progressive collapse resistance and catenary action development under uncertain material conditions. [20], [23]

The integration between ATENA nonlinear analysis and MATLAB-based probabilistic post-processing constituted a major contribution of the research because it enabled efficient management of a large nonlinear numerical database while preserving engineering

reliability during interpretation of the structural response. In particular, the transition from fully automatic P_u detection toward a semi-automatic engineering-based procedure significantly improved the accuracy and physical consistency of the probabilistic analysis.

The developed probabilistic framework provided the basis for the comparative investigations and reinforcement optimization studies presented in the following chapters. The generated probabilistic curves and activation probabilities enabled direct interpretation of the influence of reinforcement arrangements on structural robustness and catenary action stability. [7], [15]

CHAPTER 5: Improved Numerical Models and Reinforcement Enhancement Strategy

5.1 Introduction

The probabilistic analyses presented in the previous chapter demonstrated that the activation and stability of catenary action are highly sensitive to reinforcement continuity, reinforcement ratio, anchorage conditions, and post-peak tensile resistance. Although several of the original numerical models exhibited stable catenary behavior, other specimens demonstrated unstable post-peak response characterized by limited secondary resistance development and premature degradation. For this reason, an additional investigation was performed in order to evaluate the influence of reinforcement enhancement on the probabilistic robustness of the investigated reinforced concrete sub-assemblages. [7], [15]

The objective of the present chapter is to describe the development of the improved numerical models and explain the reinforcement enhancement methodology adopted in the thesis. The improvement strategy consisted of introducing additional longitudinal reinforcement layers into the original calibrated numerical models and subsequently repeating the entire probabilistic workflow developed in Chapters 3 and 4. The modified models therefore constituted enhanced structural configurations designed to improve catenary activation stability and increase the residual load-carrying capacity after severe damage development. [2], [9]

The reinforcement enhancement methodology was developed systematically using the updated Excel database containing the information associated with all twenty-six investigated specimens. The same probabilistic workflow previously adopted for the original models was repeated for the enhanced reinforcement configurations in order to preserve consistency between the numerical investigations. [16]

5.2 Reinforcement Enhancement Methodology

The enhancement strategy started from evaluation of the existing reinforcement ratio associated with each specimen. The original reinforcement information available in the Excel database was used to calculate the corresponding additional reinforcement area required for the 25% and 30% enhancement configurations. The selected enhancement ratios were considered sufficient to produce measurable changes in the post-peak response while remaining compatible with realistic strengthening configurations applicable to reinforced concrete sub-assemblages. [21]

Sample name	Initial reinforcement ratio	Reinforcement ratio for 25%	Reinforcement ratio for 30%
IMF	0.009187	0.011483	0.011942
SMF	0.011544	0.014430	0.015007
B2	0.015863	0.019829	0.020622
B3	0.013484	0.016854	0.017529
S1	0.011911	0.014889	0.015484
S2	0.010467	0.013083	0.013607
S3	0.014800	0.018500	0.019240
S4	0.017689	0.022111	0.022995
S5	0.017689	0.022111	0.022995
S6	0.028511	0.035639	0.037065
S7	0.017689	0.022111	0.022995
MEHRDAD	0.007850	0.009813	0.010205
CL-13	0.025120	0.031400	0.032656
CL-11	0.025120	0.031400	0.032656
SPECIMEN-01	0.009691	0.012114	0.012599
SPECIMEN-02	0.009691	0.012114	0.012599
SPECIMEN-03	0.009691	0.012114	0.012599
SPECIMEN-04	0.012114	0.015143	0.015748
SPECIMEN-05	0.012114	0.015143	0.015748
SPECIMEN-06	0.012114	0.015143	0.015748
FR	0.021806	0.027257	0.028347
FR-S	0.021806	0.027257	0.028347
FR-R	0.028958	0.036197	0.037645
IR-1	0.021806	0.027257	0.028347

Table 35: Comparison of initial and enhanced reinforcement ratios¹ for the investigated samples, showing the baseline reinforcement ratio and the modified values obtained by applying 25% and 30% reinforcement increases.

¹ Reinforcement ratio: The reinforcement ratio represents the proportion of steel reinforcement area (A_s) to the effective concrete section (A_c). In this study, reinforcement enhancement was applied by increasing the reinforcement area, which consequently increased the reinforcement ratio of the modified specimens.

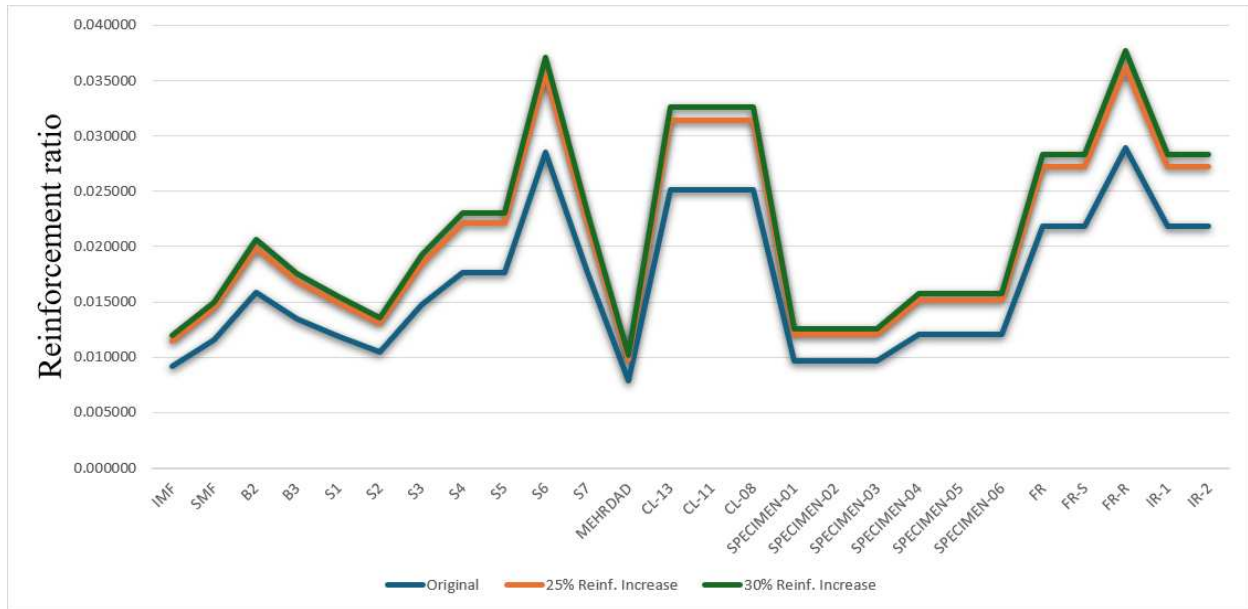
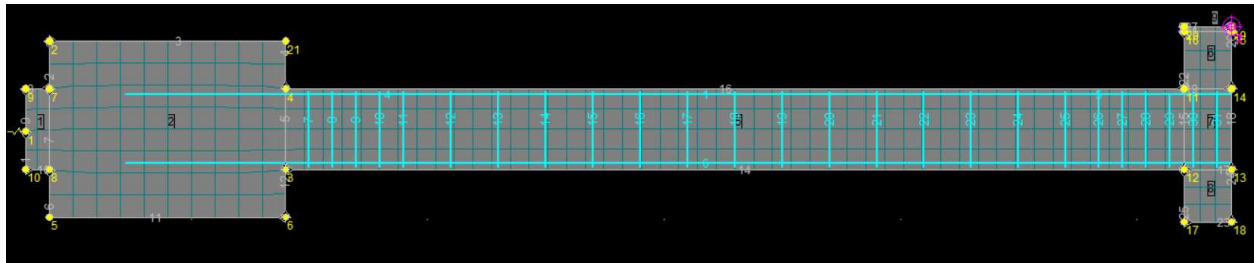
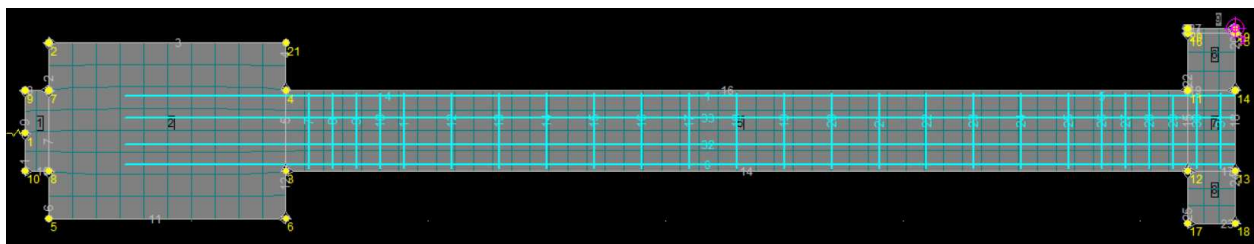


Figure 49: Variation of reinforcement ratio across the analyzed specimens for the baseline configuration and the two strengthened cases, corresponding to 25% and 30% increases in reinforcement.

The reinforcement enhancement procedure specifically targeted the post-peak catenary stage of the structural response rather than only the flexural resistance of the specimens. Consequently, the additional reinforcement was positioned within intermediate regions between the existing top and bottom reinforcement layers in order to improve tensile force redistribution and stabilize membrane action during large deformations. [7]



a)



b)

Figure 50: Reinforcement arrangements a) Before adding additional reinforcements, b) After adding additional reinforcements

The additional reinforcement layers were distributed symmetrically within the beam section. For the 25% and 30% enhancement configurations, the total additional reinforcement area was divided into two reinforcement layers located approximately at one-third and two-thirds of the beam depth. This arrangement preserved the structural symmetry of the investigated sub-assemblages and improved the efficiency of tensile force redistribution during catenary activation. [2], [9]

5.3 Geometric Definition of the Additional Reinforcement

The geometric definition of the additional reinforcement layers was performed using the coordinates associated with the original longitudinal reinforcement already defined within the ATENA numerical models. The x-coordinates of the newly introduced reinforcement generally remained identical to the existing reinforcement layers, while the y-coordinates were modified in order to position the new reinforcement at intermediate vertical locations within the beam depth. [16]

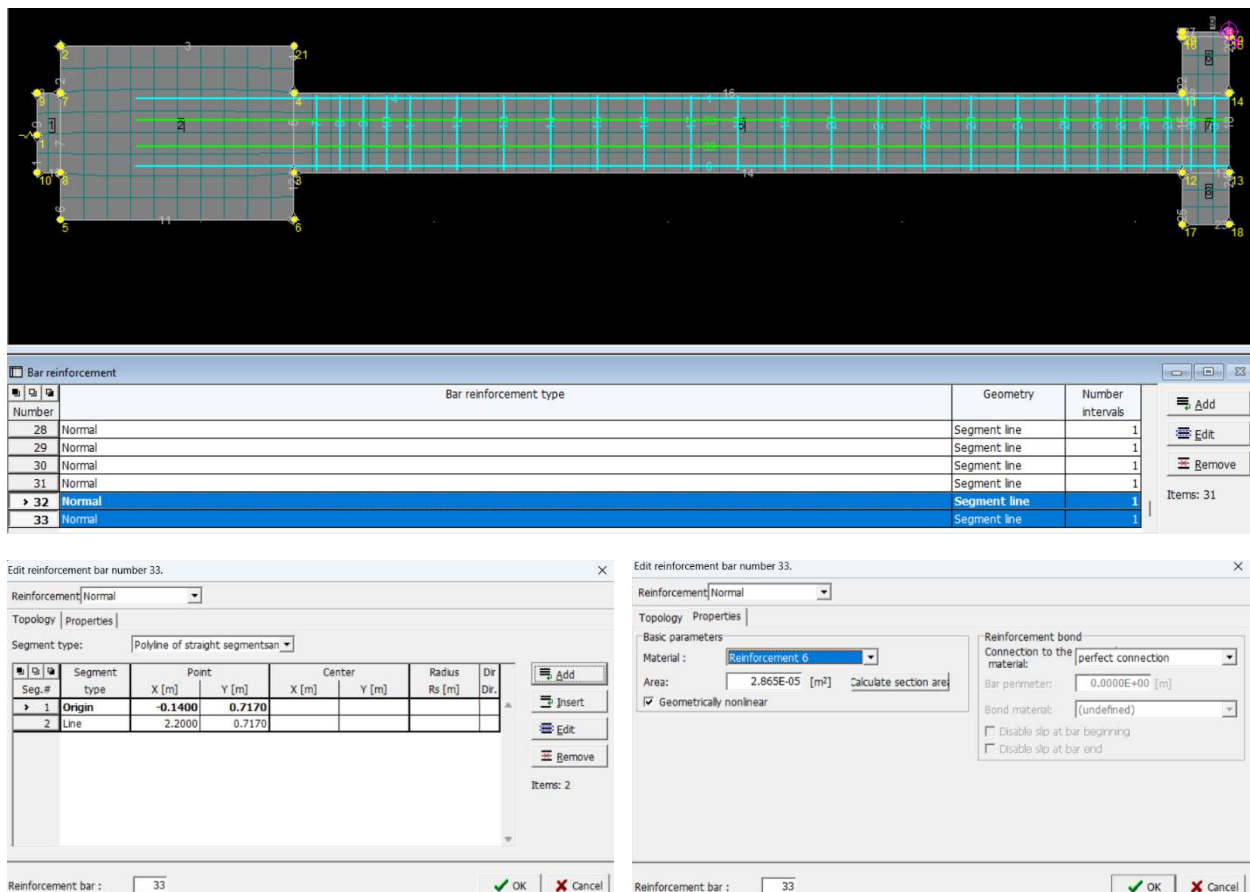


Figure 51: ATENA 2D reinforcement modelling interface for the investigated specimen, illustrating the discrete bar reinforcement arrangement and the assigned reinforcement parameters, including segment geometry, steel material model, cross-sectional area.

The reinforcement enhancement methodology preserved compatibility between the original and modified numerical models because the additional reinforcement layers were generated directly from the original reinforcement geometry. Consequently, the enhancement process modified only the reinforcement quantity and vertical reinforcement distribution while preserving the original geometry, support conditions, and loading configuration associated with the calibrated baseline models. [13]

5.4 Modification of the Numerical Models

After calculation of the reinforcement enhancement ratios and reinforcement coordinates, the additional reinforcement layers were manually inserted into the Excel templates associated with the calibrated numerical models. These templates already contained the constitutive material properties, reinforcement information, geometric parameters, and analysis settings required by the ATENA nonlinear finite element environment. [16]

Sample Name	Add. Reinforcement	Reinforcement ID	Starting X	Starting Y	Ending X	Ending Y
IMF	25-30%	109 - 110	0.05	1.7438	6.4516	1.7438
	25-30%	109 - 110	0.05	1.9128	6.4516	1.9128
SMF	25-30%	109 - 110	0.05	1.7182	6.5278	1.7182
	25-30%	109 - 110	0.05	1.9382	6.5278	1.9382
B2	25-30%	32 - 33	-0.14	0.661	2.2	0.661
	25-30%	32 - 33	-0.14	0.717	2.2	0.717
B3	25-30%	32 - 33	-0.14	0.661	2.2	0.661
	25-30%	32 - 33	-0.14	0.719	2.2	0.719
S1	25-30%	69 - 70	-0.38	0.09	2.875	0.09
	25-30%	69 - 70	-0.38	0.16	2.875	0.16
S2	25-30%	55 - 56	-0.38	0.093	2.875	0.093
	25-30%	55 - 56	-0.38	0.156	2.875	0.156
S3	25-30%	55 - 56	-0.38	0.09	2.875	0.09
	25-30%	55 - 56	-0.38	0.16	2.875	0.16
S4-S5-S6	25-30%	55 - 56	-0.38	0.09	2.875	0.09
	25-30%	55 - 56	-0.38	0.16	2.875	0.16
S7	25-30%	48 - 49	-0.38	0.09	2.2750	0.09
	25-30%	48 - 49	-0.38	0.16	2.2750	0.16
Mehrdad 02	25-30%	48 - 49	0.0	0.145	2.37	0.145

	25-30%	48 - 49	0.0	0.195	2.37	0.195
CL-13	25-30%	58 - 59	0.0	0.441	2.37	0.441
	25-30%	58 - 59	0.0	0.507	2.37	0.507

Table 36: Additional reinforcements' coordinates for each sample

The newly introduced reinforcement layers were assigned new reinforcement identifiers within the ATENA input structure while maintaining the same constitutive material properties adopted for the original reinforcement. This approach ensured that the observed differences between the original and enhanced numerical models were associated primarily with the modified reinforcement arrangement rather than changes in material behavior. [13]

The reinforcement insertion procedure additionally required verification of reinforcement continuity and compatibility with the finite element mesh. Since the new reinforcement layers were introduced within intermediate regions of the beam depth, particular attention was devoted to avoiding numerical overlap between reinforcement elements and preserving proper interaction between reinforcement and surrounding concrete material. [16]

The before-and-after reinforcement configurations clearly illustrated the conceptual objective of the enhancement strategy. In the original models, the reinforcement arrangement consisted mainly of top and bottom reinforcement layers. In the enhanced configurations, additional intermediate reinforcement layers were inserted in order to improve tensile force redistribution and stabilize catenary action during the post-peak response.

5.5 Repetition of the Probabilistic Workflow

Following modification of the reinforcement configurations, the complete probabilistic workflow previously developed for the original models was repeated for the enhanced numerical database. The same MATLAB-based Latin Hypercube Sampling (LHS) procedure was adopted in order to generate probabilistic material realizations for the 25% and 30% reinforcement enhancement configurations.

For each of the twenty-six baseline specimens, thirty probabilistic sub-samples were generated for the 25% enhancement configuration and an additional thirty probabilistic sub-samples were generated for the 30% enhancement configuration. Consequently, the enhancement stage produced an additional $26 \times 2 \times 30$ nonlinear finite element simulations beyond the original probabilistic database developed in the previous chapters.

The generated probabilistic material realizations were automatically written into the updated Excel templates containing the modified reinforcement configurations. Dedicated

MATLAB procedures subsequently converted the updated parameter sets into ATENA-compatible CCT files exactly as performed for the original reinforcement models. This approach preserved methodological consistency throughout the entire numerical investigation.

Each generated CCT model was imported into ATENA 2D and analyzed using the same nonlinear finite element procedures previously adopted for the original models. Incremental–iterative solution procedures, displacement-controlled loading, and arc-length stabilization methods were maintained throughout the enhanced analyses in order to ensure direct comparability between the original and improved structural responses. [16]

5.6 Structural Behavior of the Improved Models

The enhanced numerical models demonstrated substantial differences compared to the original reinforcement configurations, particularly during the post-peak response stages associated with catenary action development. In many specimens, the additional reinforcement layers improved the stability of the secondary resistance increase and reduced the occurrence of unstable degradation branches within the load–displacement curves. [7], [9]

The probabilistic analyses additionally indicated that reinforcement enhancement influenced not only the magnitude of the ultimate catenary resistance but also the variability associated with the nonlinear structural response. In several cases, the improved configurations exhibited narrower probabilistic distributions compared to the original models, indicating improved robustness consistency under uncertain material conditions. [15], [20]

The introduction of intermediate reinforcement layers proved particularly effective for specimens characterized by limited reinforcement continuity or unstable post-peak behavior in the original configuration. The enhanced reinforcement arrangement improved tensile force redistribution within the beam section and contributed to more stable membrane action during large deformation stages. [2], [5]

Despite the general improvement observed in the enhanced models, the effectiveness of the reinforcement enhancement varied among the investigated specimens. This observation demonstrated that the structural response depends not only on reinforcement quantity but also on the original detailing configuration, geometric characteristics, restraint conditions, and failure mechanisms governing each specimen. [8]

5.7 Concluding Remarks

The improved numerical models developed in this chapter constituted a direct extension of the probabilistic framework established in the previous chapters. The adopted methodology integrated reinforcement enhancement, probabilistic material generation, MATLAB-based automation, and nonlinear finite element analysis within a unified computational workflow. [13], [17]

The reinforcement enhancement strategy specifically targeted the improvement of the post-peak catenary response rather than simply increasing the flexural resistance of the investigated sub-assemblages. The introduction of additional intermediate reinforcement layers significantly improved tensile force redistribution and enhanced the stability of catenary activation in many of the investigated specimens. [7], [9]

The expanded numerical database generated from the enhanced models provided the basis for the comparative investigations presented in the following chapter. The probabilistic comparison between the original and improved configurations enabled direct evaluation of the influence of reinforcement enhancement on structural robustness, catenary activation probability, and residual resistance under progressive collapse scenarios. [15], [20]

CHAPTER 6: Comparative Analysis between Original and Improved Models

6.1 Introduction and purpose of the comparative analysis

The previous chapters established the complete numerical framework of the thesis: Chapter 3 presented the calibrated nonlinear finite element models, Chapter 4 described the probabilistic post-processing of the original configurations, and Chapter 5 introduced the improved reinforcement arrangements obtained by adding 25% and 30% longitudinal reinforcement. The purpose of the present chapter is therefore not to repeat the modeling procedure, but to compare the structural consequences of the reinforcement enhancement in a results-oriented manner. [13], [17]

The comparison is performed between three reinforcement arrangements for each of the twenty-six investigated specimens: the Original configuration, the +25% improved configuration, and the +30% improved configuration. The complete probabilistic dataset contains the mean P_o and P_u values, their lognormal parameters, and the probability that the catenary-stage resistance exceeds the first resistance peak. These indicators allow the improved models to be judged not only by higher load capacity, but also by the stability and reliability of catenary action. [18], [20]

The central question of this chapter is whether the additional reinforcement layers introduced in Chapter 5 actually improve robustness in a meaningful way. In this thesis, an improvement is considered meaningful when it increases the ultimate/catenary resistance, improves or preserves the probability of catenary activation, and does not merely increase the early flexural or arching peak without enhancing the large-deformation response. This distinction is essential because progressive collapse resistance is governed by mechanism evolution rather than by a single peak load. [7], [15]

6.2 Comparison indicators and evaluation logic

Three principal indicators were used for the comparative analysis. The first indicator is P_o , defined as the first significant resistance peak in the nonlinear load–displacement curve. This value is generally associated with the flexural-to-arching stage and therefore reflects the initial resistance of the specimen before full catenary action is mobilized. Although P_o is important, it does not by itself demonstrate robustness at large deformation. [6], [9]

The second indicator is P_u , defined as the ultimate resistance associated with the catenary-dominated stage selected during the semi-automatic MATLAB post-processing procedure. P_u is the most relevant load indicator for the improved models because the added intermediate reinforcement layers were intended to strengthen the tensile membrane mechanism that develops after the first peak and after substantial damage accumulation. [2], [5]

The third indicator is the probability of catenary activation, calculated as the probability that P_u exceeds P_o under the fitted lognormal distributions. This metric is particularly important because it summarizes the probabilistic stability of the mechanism transition. A model may have a higher mean P_u but still show limited activation reliability if its response is highly scattered or if the catenary branch remains unstable. [15], [20]

Arrangement	Average P_o (kN)	Average P_u (kN)	Average Pactivation	Median P_u (kN)	Samples with Pactivation $\geq$ 0.95
Original	94.88	132.40	0.681	73.22	15
+25%	104.33	146.77	0.707	72.85	14
+30%	106.36	150.84	0.706	73.34	14

Table 37: Global comparison indicators across the 26 specimens.

Table 40 shows that the average P_o increased from 94.88 kN in the Original models to 104.33 kN in the +25% models and 106.36 kN in the +30% models. The corresponding average P_u increased from 132.40 kN to 146.77 kN and 150.84 kN. These values indicate that the reinforcement enhancement produced a stronger global effect on the ultimate/catenary stage than on the initial peak response.

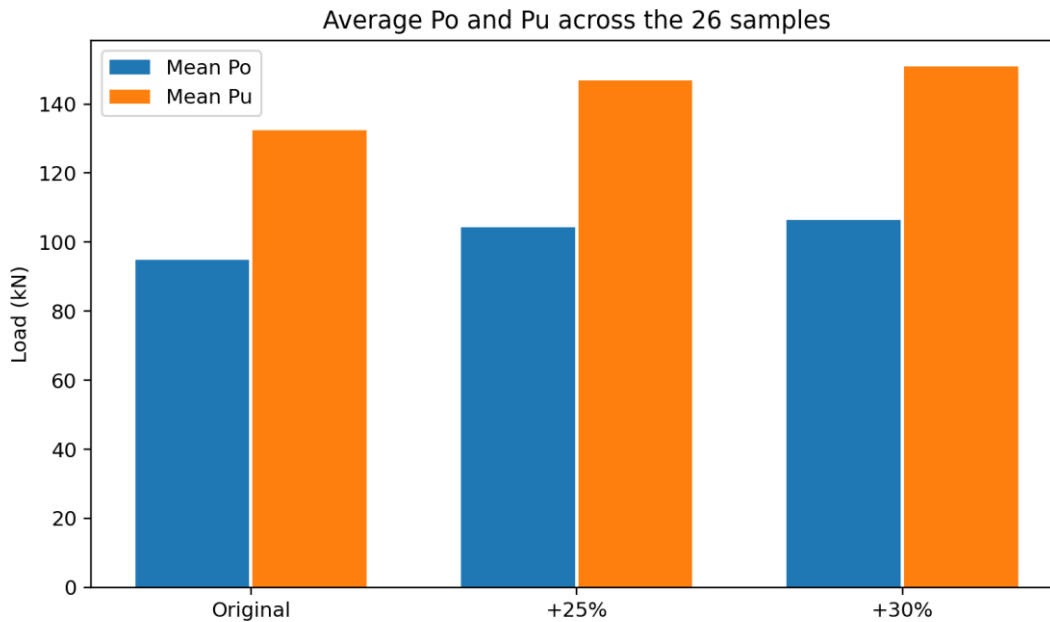


Figure 52: Average P_o and P_u for Original, +25%, and +30% configurations.

6.3 Comparison of P_o and P_u values

The comparison of P_o confirms that the added reinforcement affected the early response in most specimens, but the magnitude of this increase was generally moderate compared with the changes observed in P_u . This is mechanically reasonable because P_o is influenced by

flexural behavior, concrete cracking, axial restraint, and compressive arch action, whereas the added intermediate reinforcement was mainly intended to improve tensile force redistribution at large displacement. [7], [12]

The comparison of P_u provides a clearer measure of the effectiveness of the improved reinforcement arrangements. Fifteen of the twenty-six specimens exhibited a positive increase in mean P_u for both the +25% and +30% configurations, while several specimens showed a reduction in P_u despite the added reinforcement. This result demonstrates that reinforcement enhancement is not universally beneficial; its effectiveness depends on whether the original failure mechanism allows the added steel to participate in the catenary stage. [8], [9]

Sample	P_u Original	P_u +25%	P_u +30%	ΔP_u +25%	ΔP_u +30%
B2	44.86	63.24	66.33	41.0%	47.9%
IMF	458.40	645.50	661.02	40.8%	44.2%
IR-1	69.47	89.83	95.30	29.3%	37.2%
B3	34.08	44.18	46.11	29.6%	35.3%
Specimen-03	19.24	24.09	25.41	25.2%	32.1%
S6	151.21	122.40	123.32	-19.1%	-18.4%
S4	111.48	94.90	93.71	-14.9%	-15.9%
S3	109.66	96.43	94.98	-12.1%	-13.4%
CL-11	87.21	75.91	77.61	-13.0%	-11.0%

Table 38: Representative P_u changes: strongest increases and reductions.

The strongest positive P_u changes were observed for B2, IMF, IR-1, B3, and several FarhangVesali specimens. These improvements are mainly associated with cases where the added reinforcement increased the available tensile resistance or improved the stability of the large-deformation branch. In contrast, reductions in P_u occurred in specimens such as S6, S4, S3, and CL-11, indicating that the response of these models was controlled by mechanisms other than the simple amount of longitudinal reinforcement.

For completeness, Tables 42 and 43 report the full P_o and P_u comparisons for all specimens. These two tables are intentionally separated because the physical meaning of the two quantities is different: P_o is mainly related to early-stage flexural or arching resistance, while P_u represents the late-stage catenary resistance selected after inspection of the capacity curves. This separation prevents the interpretation from confusing early resistance increase with genuine robustness improvement. [7]

Sample	Po Original	Po +25%	Po +30%	ΔP_o +25%	ΔP_o +30%
B2	27.10	28.51	28.88	5.2%	6.6%
B3	39.68	41.41	41.72	4.4%	5.1%
CL-11	45.91	53.84	55.28	17.3%	20.4%
CL-13	41.86	47.25	48.33	12.9%	15.5%
FR	32.87	38.52	39.60	17.2%	20.5%
FR-R	32.08	37.62	38.48	17.3%	19.9%
FR-S	32.03	38.09	39.27	18.9%	22.6%
IMF	301.30	345.16	353.93	14.6%	17.5%
IR-1	63.36	74.81	76.94	18.1%	21.4%
Mehrdad	66.57	70.03	70.84	5.2%	6.4%
S1	45.50	50.68	51.33	11.4%	12.8%
S2	40.77	46.82	48.04	14.8%	17.8%
S3	55.47	63.39	64.77	14.3%	16.8%
S4	64.21	73.92	75.31	15.1%	17.3%
S5	70.75	75.70	76.60	7.0%	8.3%
S6	73.92	81.14	84.45	9.8%	14.2%
S7	93.69	104.58	106.84	11.6%	14.0%
SMF	976.19	1050.44	1068.19	7.6%	9.4%
Specimen-01	42.93	45.00	45.50	4.8%	6.0%
Specimen-02	38.40	40.38	40.91	5.1%	6.5%
Specimen-03	42.61	44.72	45.04	5.0%	5.7%
Specimen-04	44.96	47.28	47.86	5.2%	6.4%
Specimen-05	47.86	48.32	48.85	1.0%	2.1%
Specimen-06	45.52	47.89	48.40	5.2%	6.3%

Table 39: Full comparison of mean P_o values for all specimens.

The P_o table shows that most specimens experienced an increase in the first resistance peak after reinforcement enhancement. However, this increase should be interpreted carefully because a higher first peak may also raise the threshold that the catenary stage must exceed to be counted as activated. Therefore, an increase in P_o without a sufficient increase in P_u can actually reduce the calculated probability of catenary activation. [20]

Sample	Pu Original	Pu +25%	Pu +30%	ΔP_u +25%	ΔP_u +30%
B2	44.86	63.24	66.33	41.0%	47.9%
B3	34.08	44.18	46.11	29.6%	35.3%
CL-11	87.21	75.91	77.61	-13.0%	-11.0%
CL-13	78.31	70.20	75.50	-10.4%	-3.6%
FR	52.15	60.22	62.45	15.5%	19.8%
FR-R	76.50	75.50	73.30	-1.3%	-4.2%
FR-S	53.65	57.90	58.26	7.9%	8.6%
IMF	458.40	645.50	661.02	40.8%	44.2%
IR-1	69.47	89.83	95.30	29.3%	37.2%

Mehrdad	70.71	87.68	85.83	24.0%	21.4%
S1	81.23	75.92	73.38	-6.5%	-9.7%
S2	75.73	67.92	67.53	-10.3%	-10.8%
S3	109.66	96.43	94.98	-12.1%	-13.4%
S4	111.48	94.90	93.71	-14.9%	-15.9%
S5	112.20	108.45	109.61	-3.3%	-2.3%
S6	151.21	122.40	123.32	-19.1%	-18.4%
S7	128.89	124.50	125.72	-3.4%	-2.5%
SMF	1370.17	1568.50	1639.28	14.5%	19.6%
Specimen-01	19.82	20.83	21.18	5.1%	6.9%
Specimen-02	18.46	20.35	22.00	10.3%	19.2%
Specimen-03	19.24	24.09	25.41	25.2%	32.1%
Specimen-04	24.77	31.54	32.42	27.3%	30.9%
Specimen-05	32.42	32.61	32.88	0.6%	1.4%
Specimen-06	25.71	31.55	33.19	22.7%	29.1%

Table 40: Full comparison of mean P_u values for all specimens.

The P_u table confirms that the reinforcement enhancement did not lead to a uniform improvement across all specimens. Some models gained substantial catenary resistance, while others showed modest improvement or reduction. This result is important because it proves that the effectiveness of added reinforcement depends on the governing failure mechanism, not only on the amount of steel added to the section. [8], [15]

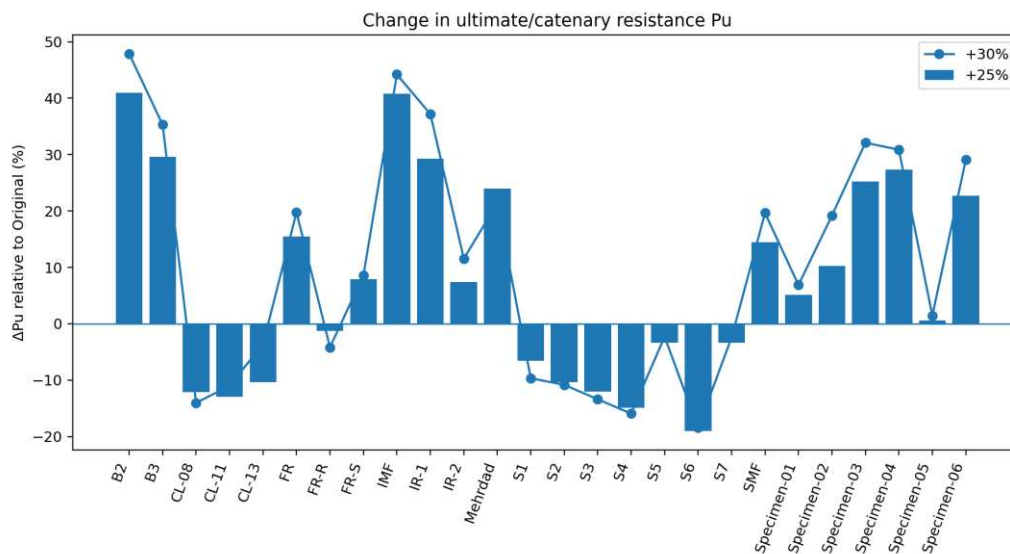


Figure 53: Percentage change in mean P_u relative to the original configuration.

6.4 Probability of catenary activation

The probability of catenary activation provides a more direct robustness interpretation than mean load values alone. In this thesis, activation probability was interpreted as the

likelihood that the catenary-stage resistance P_u exceeds the first significant resistance peak P_o . A high activation probability indicates that the specimen is likely to develop a meaningful secondary resistance mechanism after initial degradation. [15], [20]

The global average activation probability changed only slightly, increasing from approximately 0.681 in the Original models to approximately 0.707 for +25% and 0.706 for +30%. This modest global increase does not mean that the enhancement was ineffective. Instead, it reflects the heterogeneous nature of the dataset: several specimens already had activation probabilities close to one in the Original configuration, whereas other specimens remained unable to activate catenary action even after reinforcement enhancement because their response was governed by arching, local degradation, or unstable failure modes.

Sample	Pactivation n Original	Pactivation n +25%	Pactivation n +30%	Δ Pactivation n +25%	Δ Pactivation n +30%	Improvement trend
B2	0.978	1.000	1.000	0.022	0.022	small but visible activation gain
B3	0.192	0.615	0.660	0.423	0.468	clear activation gain
CL-11	1.000	0.997	0.986	-0.003	-0.014	capacity and/or activation decreased
CL-13	1.000	0.995	0.992	-0.005	-0.008	capacity and/or activation decreased
FR	0.963	0.967	0.985	0.004	0.022	small but visible activation gain
FR-R	1.000	1.000	1.000	-0.000	0.000	capacity and/or activation decreased
FR-S	0.994	0.993	0.990	-0.001	-0.005	limited change in activation
IMF	0.994	1.000	1.000	0.006	0.006	activation already high; P_u improved
IR-1	0.782	0.945	0.966	0.162	0.184	clear activation gain
Mehrdad	0.544	0.841	0.854	0.297	0.310	clear activation gain

S1	1.000	1.000	0.998	0.000	-0.002	capacity and/or activation decreased
S2	1.000	0.988	0.994	-0.012	-0.006	capacity and/or activation decreased
S3	1.000	1.000	0.998	-0.000	-0.002	capacity and/or activation decreased
S4	1.000	0.989	0.968	-0.011	-0.032	capacity and/or activation decreased
S5	1.000	0.996	0.998	-0.004	-0.001	capacity and/or activation decreased
S6	0.989	0.842	0.852	-0.147	-0.137	capacity and/or activation decreased
S7	0.765	0.967	0.949	0.202	0.183	clear activation gain
SMF	1.000	0.999	0.997	-0.000	-0.003	activation already high; Pu improved
Specimen -01	0.000	0.000	0.000	0.000	0.000	limited change in activation
Specimen -02	0.000	0.000	0.000	0.000	0.000	Pu improved, activation still limited/saturate d
Specimen -03	0.000	0.000	0.000	0.000	0.000	Pu improved, activation still limited/saturate d
Specimen -04	0.000	0.000	0.011	0.000	0.011	Pu improved, activation still limited/saturate d
Specimen -05	0.011	0.001	0.000	-0.010	-0.011	limited change in activation
Specimen -06	0.000	0.000	0.000	0.000	0.000	Pu improved, activation still limited/saturate d

Table 41: Catenary activation probability for all specimens.

Table 44 shows that the most important activation gains occurred in B3, IR-1, Mehrdad, and S7. These specimens moved from uncertain or moderate catenary activation toward much more reliable activation after the reinforcement enhancement. By contrast, specimens such as Specimen-01 to Specimen-06 remained close to zero activation probability, confirming that their behavior was not transformed into a true catenary-dominated response by the added reinforcement alone.

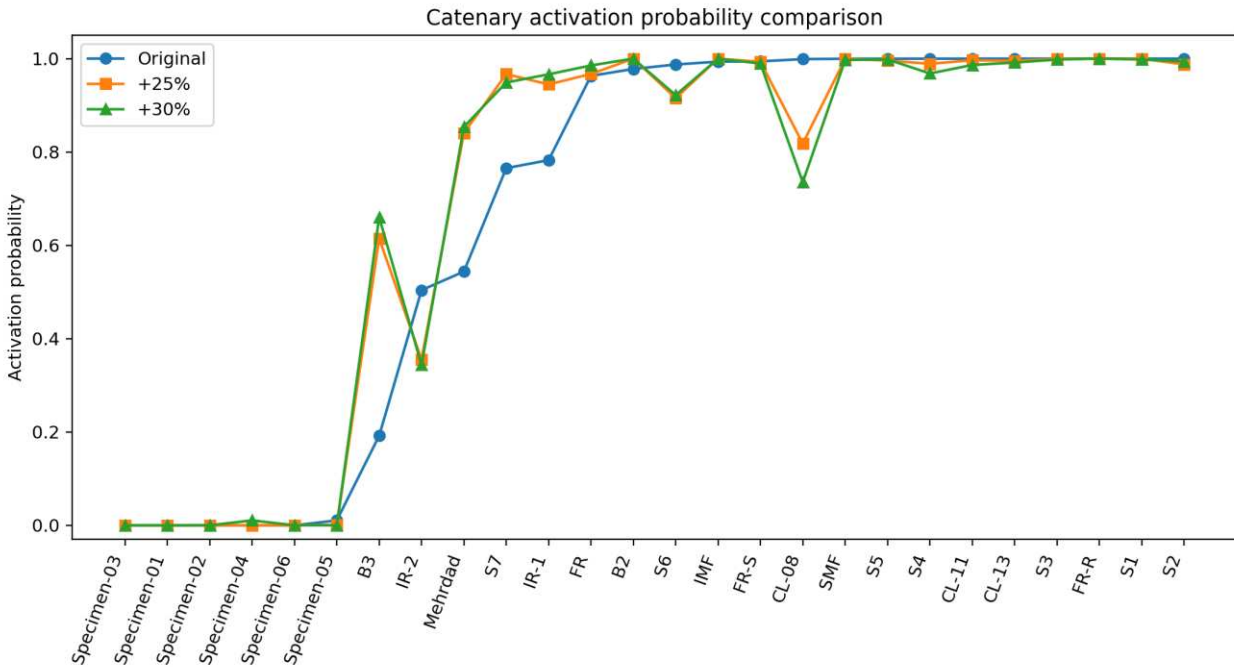


Figure 54: Catenary activation probability for the 26 specimens.

6.5 Classification of improvement trends

The specimens were classified into improvement groups using the activation probability and the change in P_u . Group A includes specimens where the +25% and/or +30% models produced a clear increase in activation probability. Group B includes specimens where activation was either moderately improved or already close to saturation but the ultimate/catenary resistance increased meaningfully. Group C includes specimens where the added reinforcement produced limited, negative, or non-transformative activation improvement. [20]

Group A contains B3, IR-1, Mehrdad, and S7. These are the cases where the improved reinforcement arrangements most clearly increased the likelihood of catenary activation. The behavior of this group suggests that the original models had enough deformation capacity to benefit from additional tensile reinforcement, but their baseline catenary branch was not fully stable.

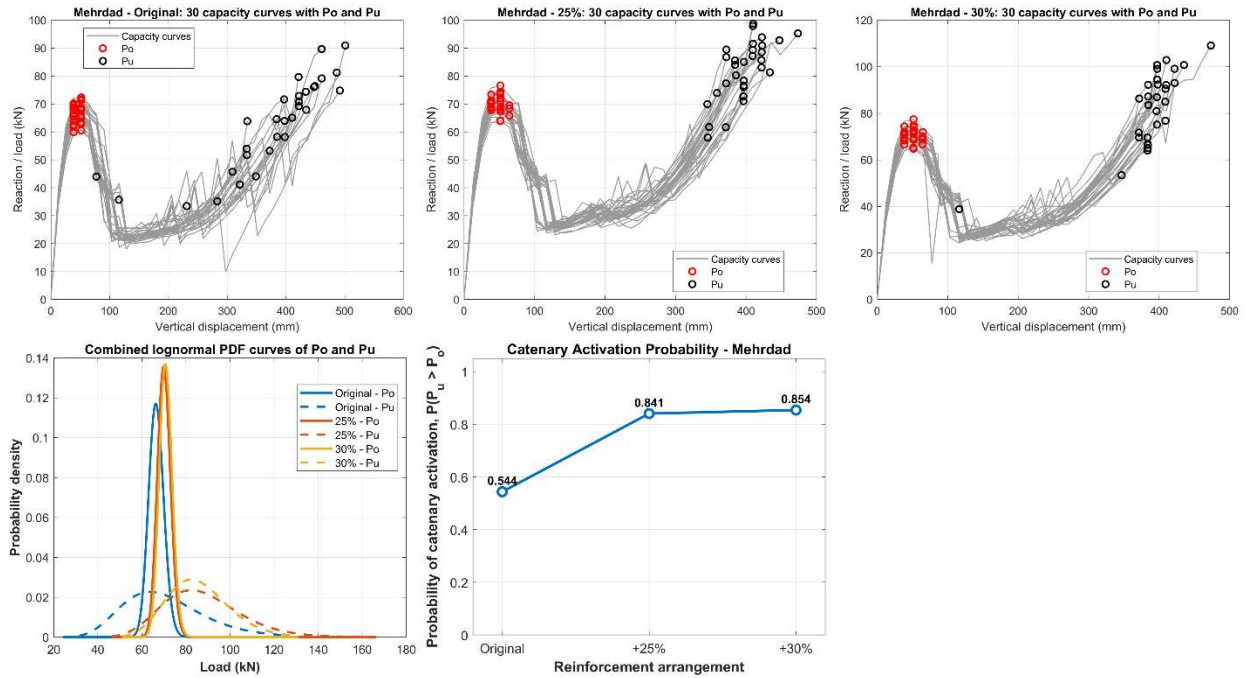


Figure 55: Probabilistic evaluation of the Mehrdad specimen for different reinforcement arrangements, the fitted lognormal probability density functions, and the probability of catenary activation

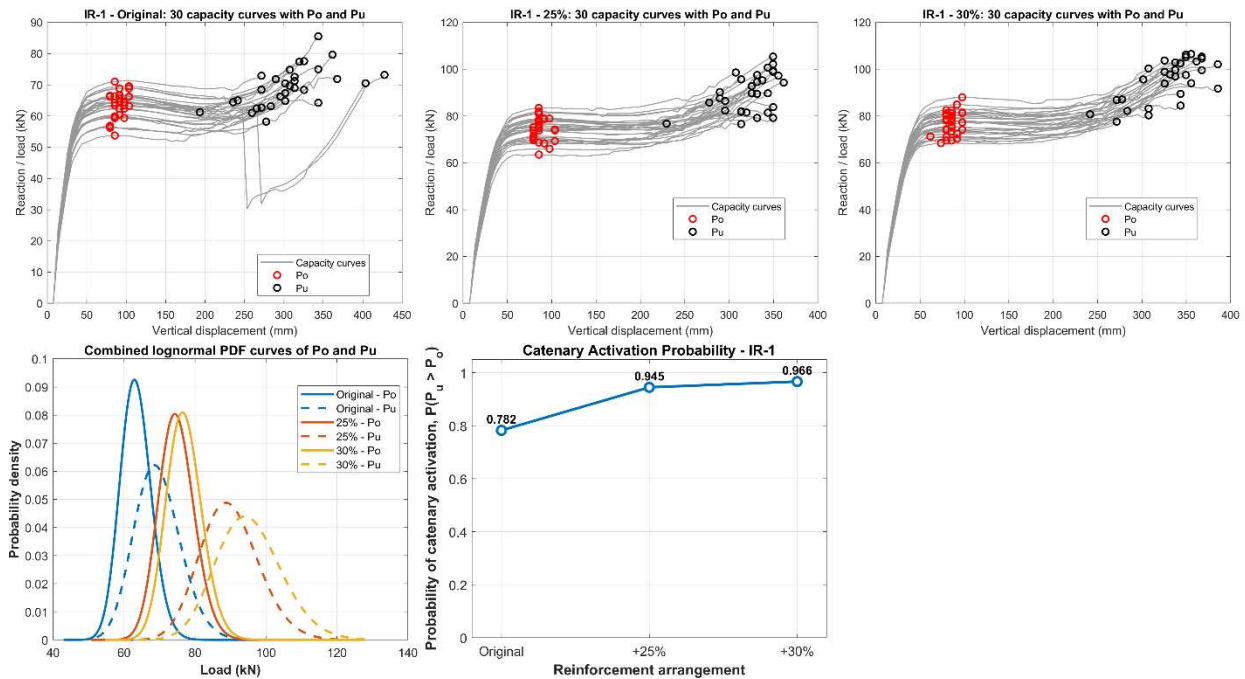


Figure 56: Probabilistic evaluation of the IR-1 specimen for different reinforcement arrangements, the fitted lognormal probability density functions, and the probability of catenary activation

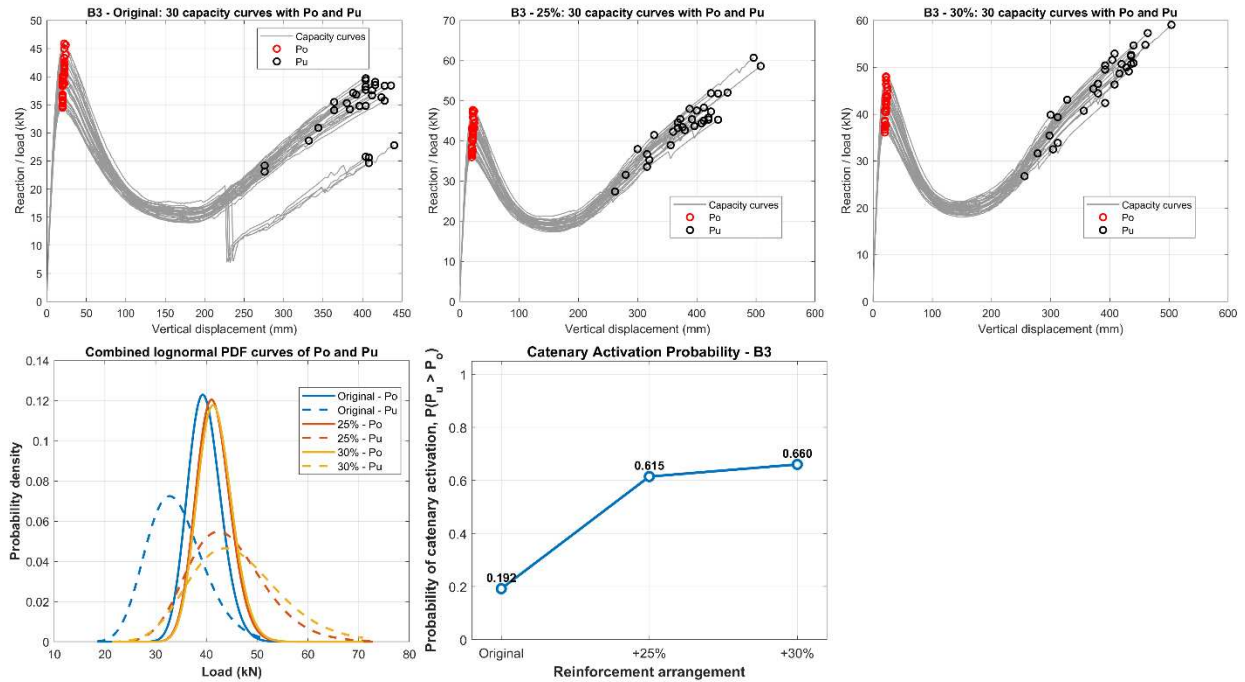


Figure 57: Probabilistic evaluation of the B3 specimen for different reinforcement arrangements, the fitted lognormal probability density functions, and the probability of catenary activation

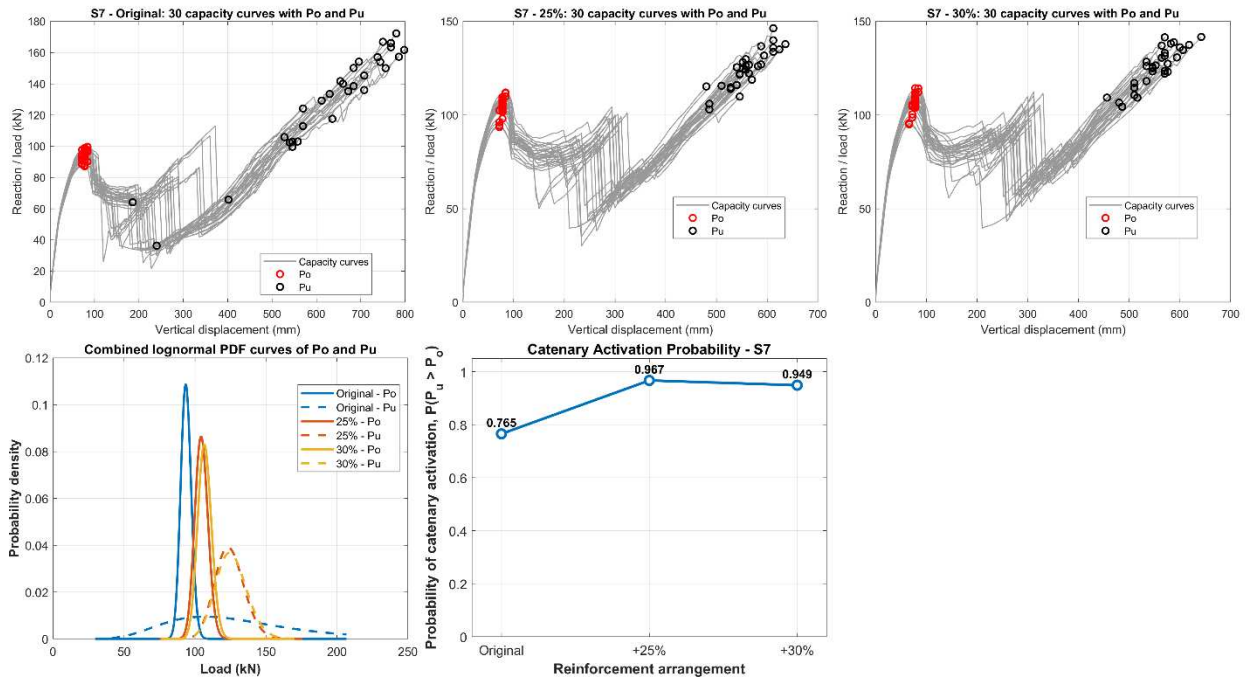


Figure 58: Probabilistic evaluation of the S7 specimen for different reinforcement arrangements, the fitted lognormal probability density functions, and the probability of catenary activation

Group B includes specimens whose activation probability was either already high in the Original configuration or improved only moderately, while the ultimate resistance still

increased in a meaningful way. Examples include B2, FR, and IMF, where the activation probability was close to one or increased only slightly, but the improved models still produced clear gains in P_u . These specimens show that activation probability and capacity improvement must be interpreted together. [2]

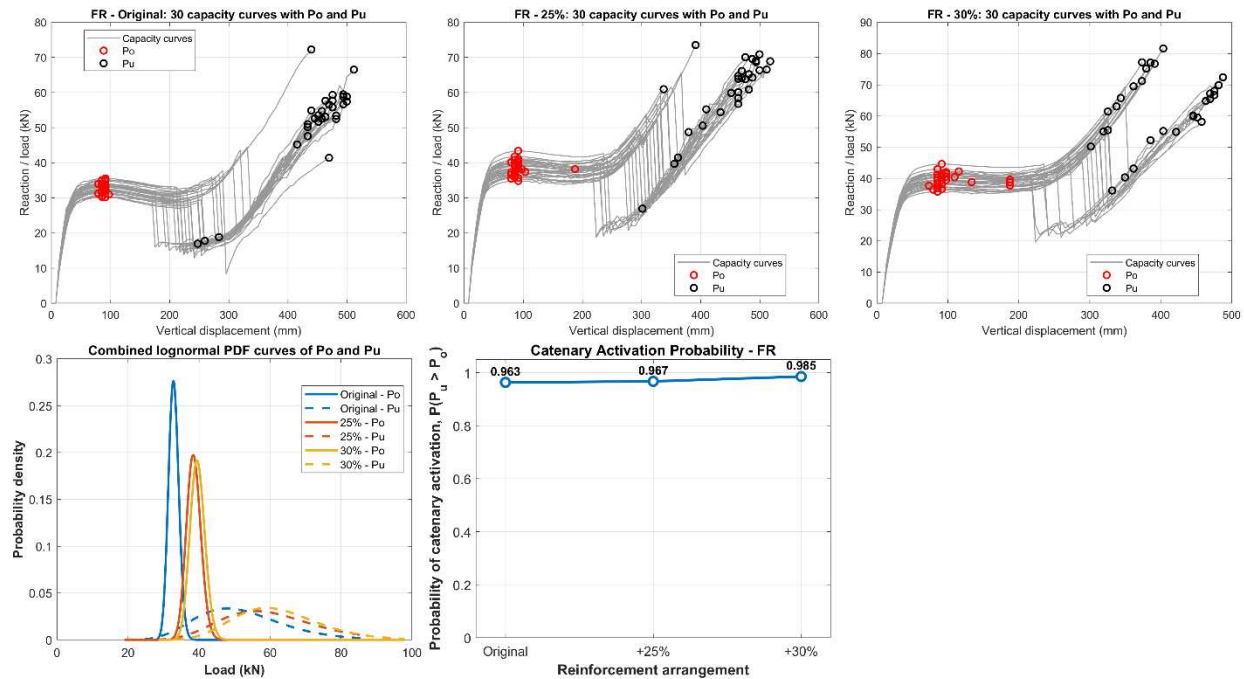


Figure 59: Probabilistic evaluation of the FR specimen for different reinforcement arrangements, the fitted lognormal probability density functions, and the probability of catenary activation

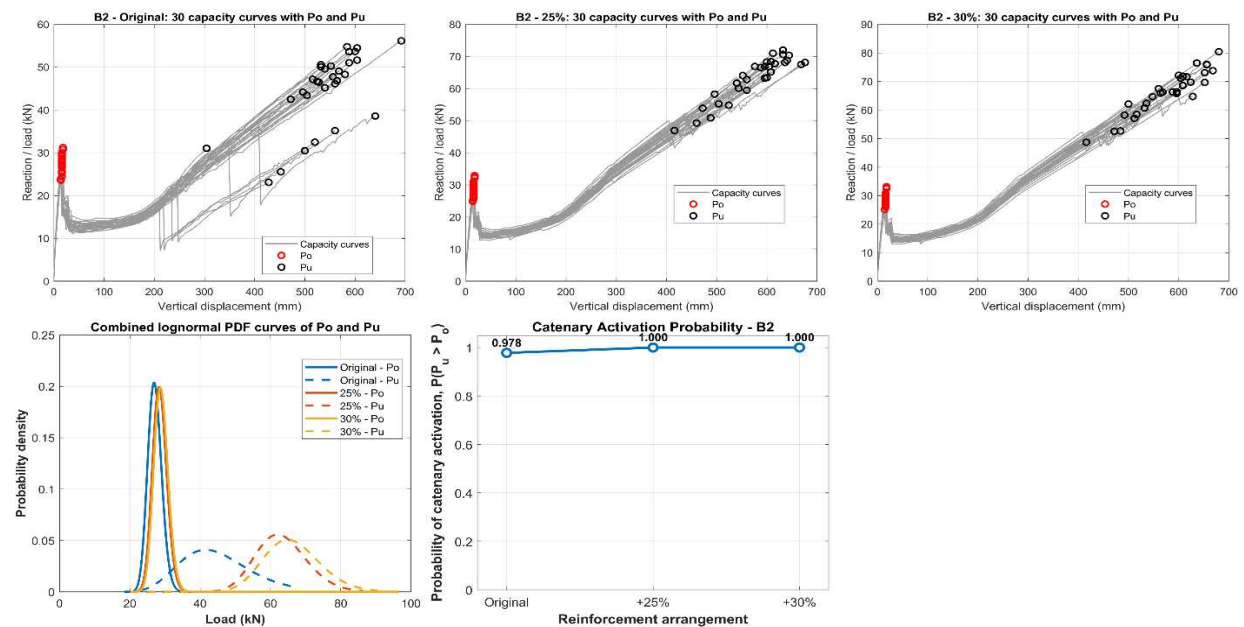


Figure 60: Probabilistic evaluation of the B2 specimen for different reinforcement arrangements, the fitted lognormal probability density functions, and the probability of catenary activation

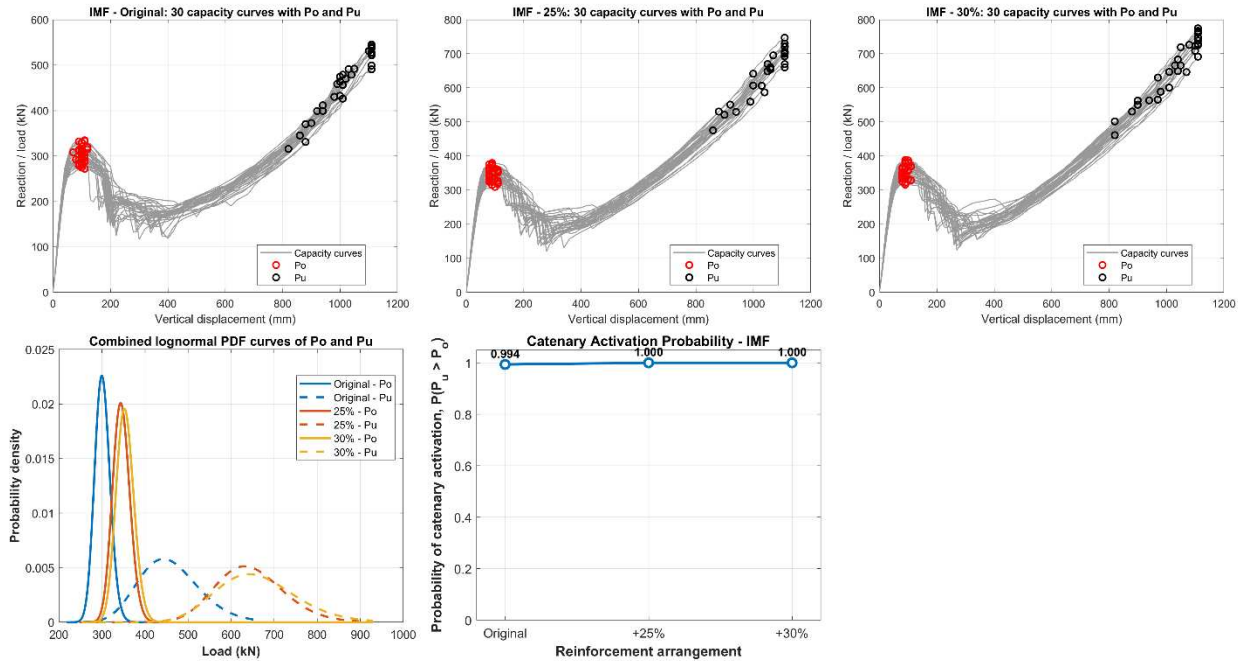


Figure 61: Probabilistic evaluation of the IMF specimen for different reinforcement arrangements, the fitted lognormal probability density functions, and the probability of catenary activation

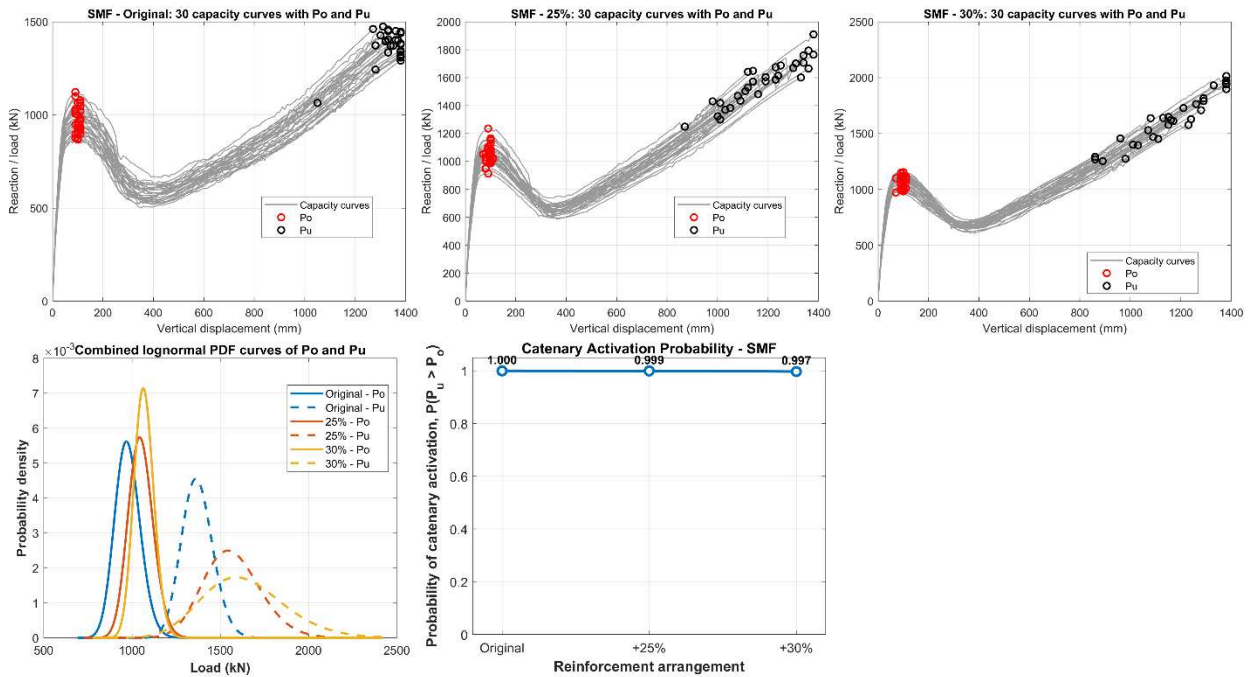


Figure 62: Probabilistic evaluation of the SMF specimen for different reinforcement arrangements, the fitted lognormal probability density functions, and the probability of catenary activation

Group C includes specimens where the additional reinforcement did not produce a significant activation benefit, and in some cases reduced the probabilistic activation indicator. This group includes several specimens that were already saturated in activation

probability, specimens governed by mechanisms other than tensile reinforcement capacity, and FarhangVesali specimens where the response was fundamentally arching-controlled rather than catenary-controlled. [12]

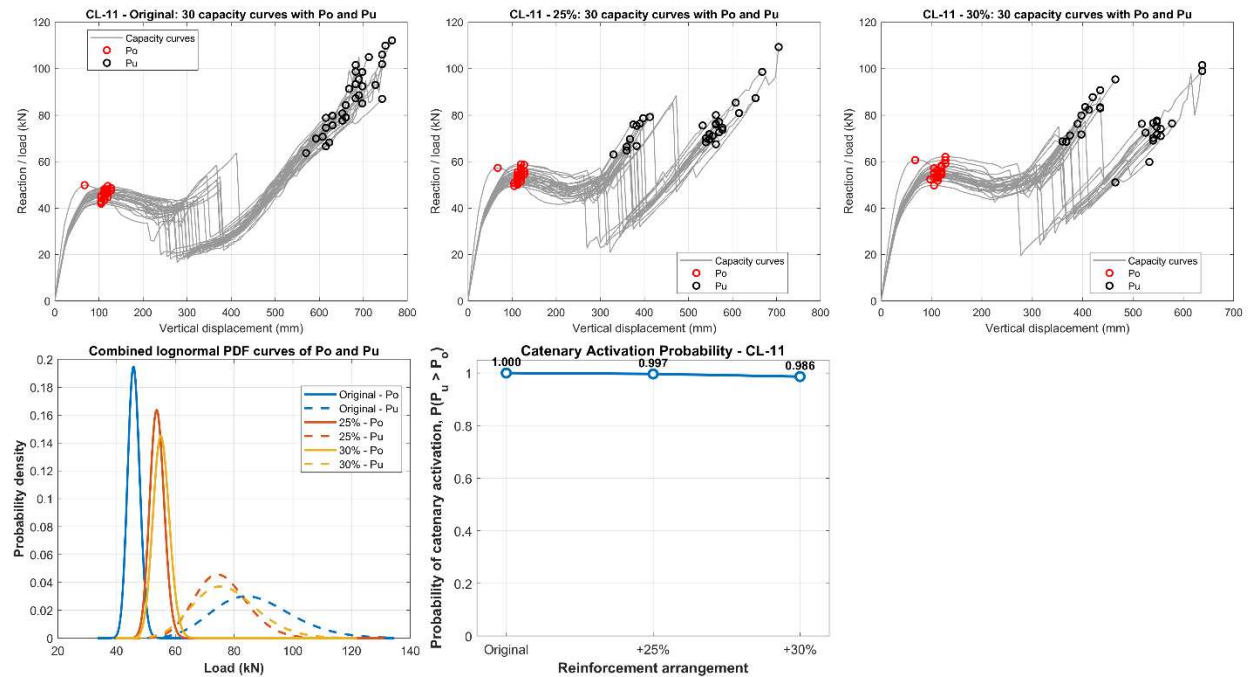


Figure 63: Probabilistic evaluation of the CL-11 specimen for different reinforcement arrangements, the fitted lognormal probability density functions, and the probability of catenary activation

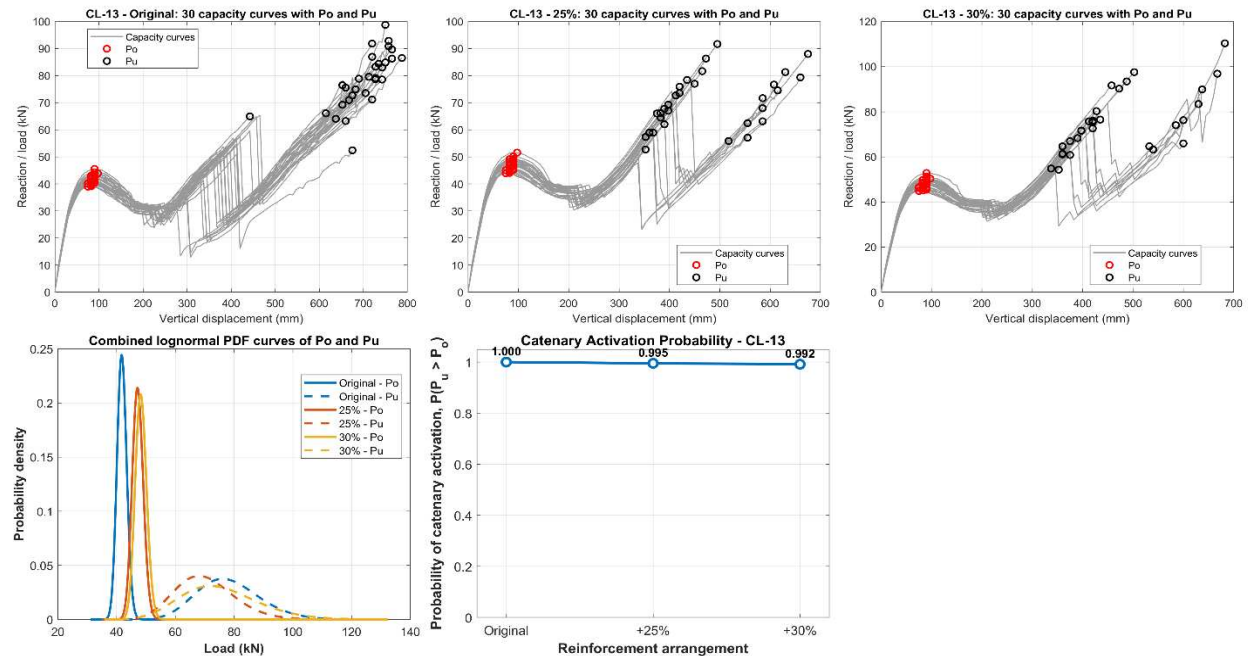


Figure 64: Probabilistic evaluation of the CL-13 specimen for different reinforcement arrangements, the fitted lognormal probability density functions, and the probability of catenary activation

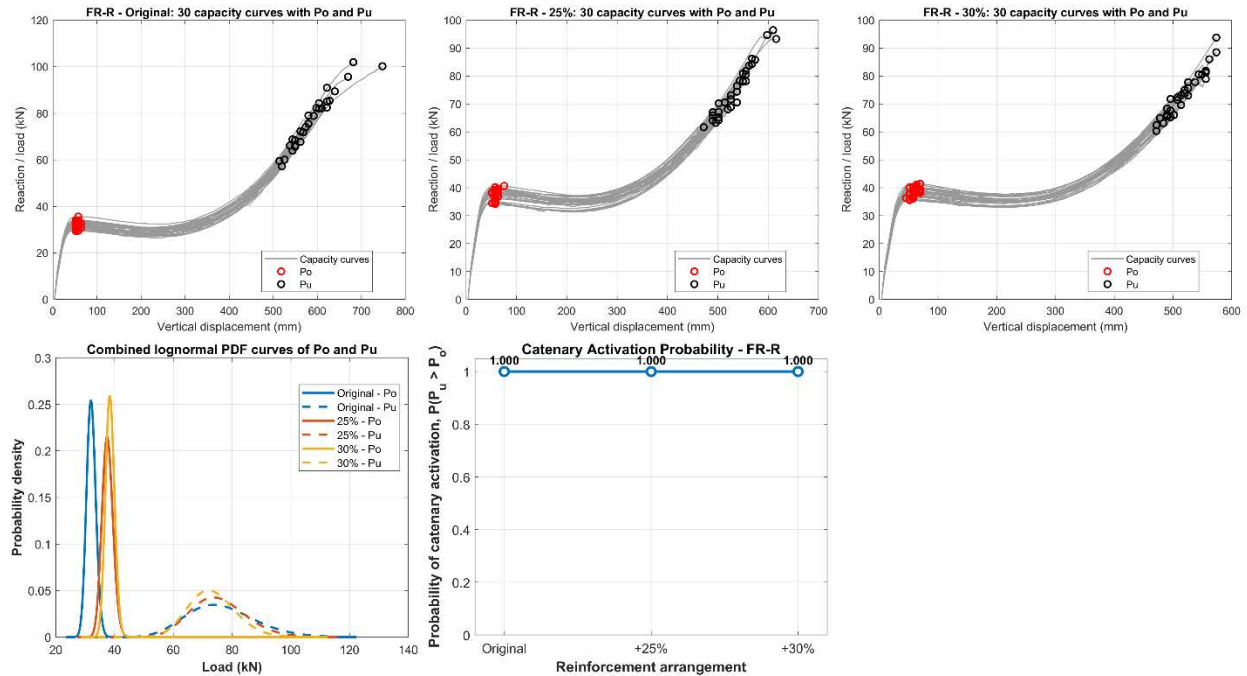


Figure 65: Probabilistic evaluation of the FR-R specimen for different reinforcement arrangements, the fitted lognormal probability density functions, and the probability of catenary activation

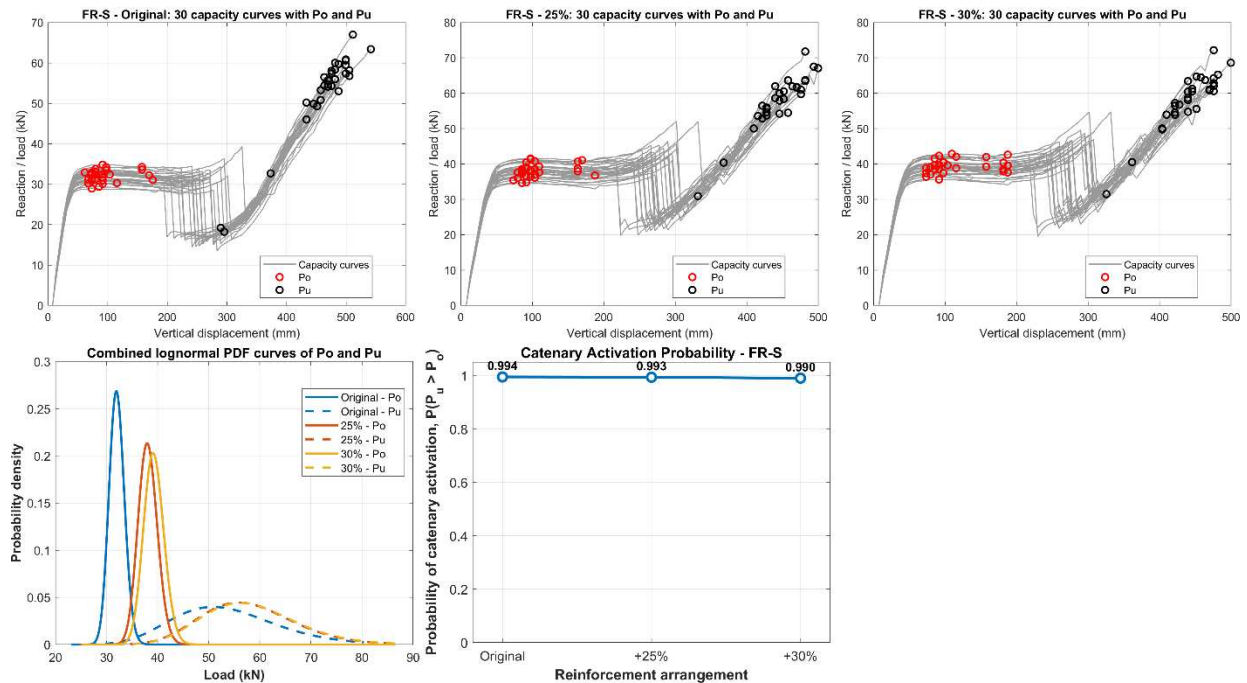


Figure 66: Probabilistic evaluation of the FR-S specimen for different reinforcement arrangements, the fitted lognormal probability density functions, and the probability of catenary activation

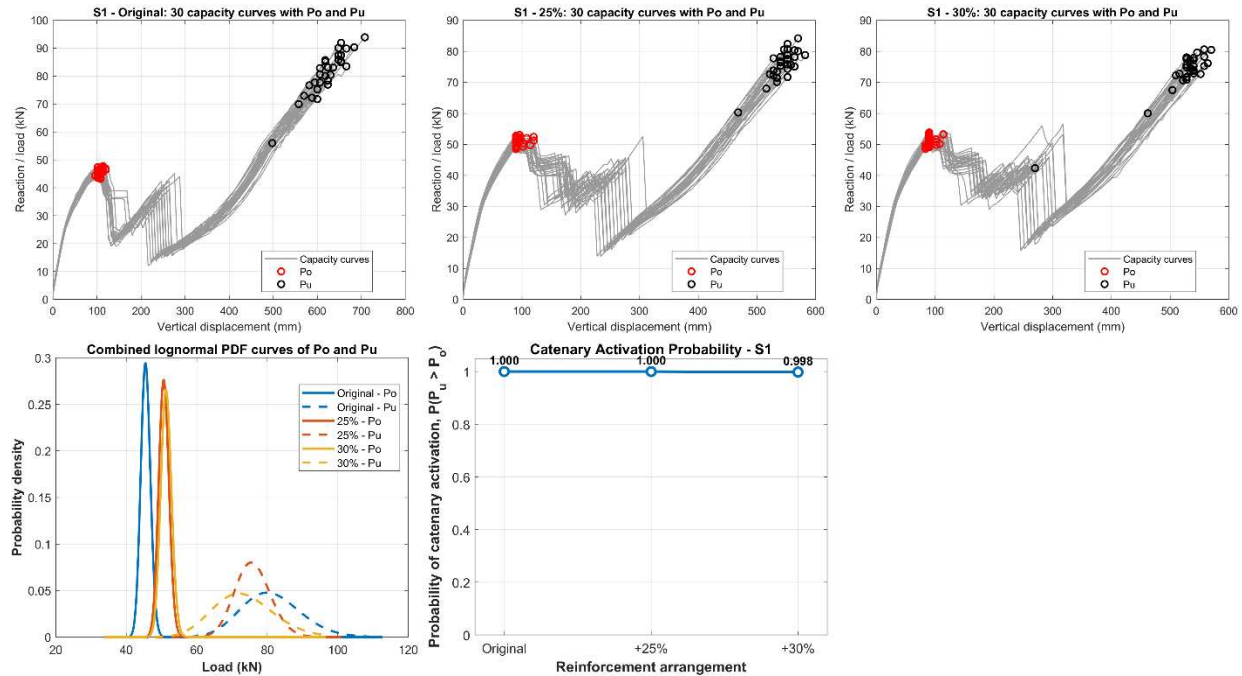


Figure 67: Probabilistic evaluation of the S1 specimen for different reinforcement arrangements, the fitted lognormal probability density functions, and the probability of catenary activation

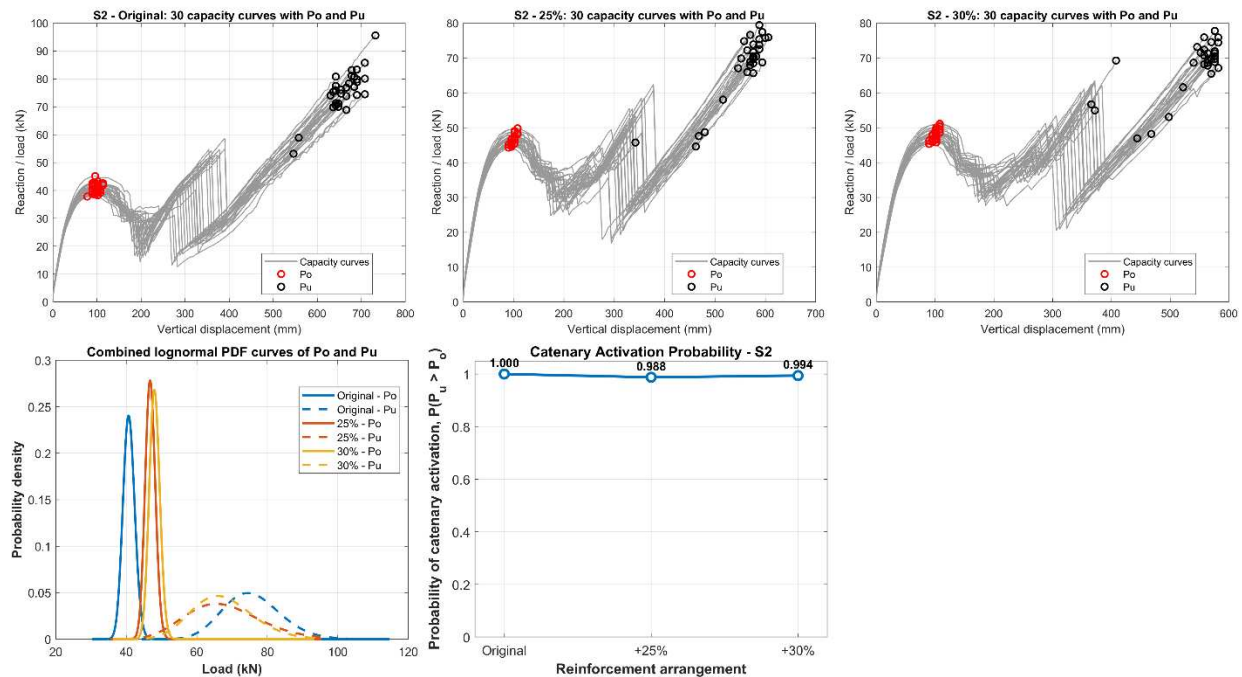


Figure 68: Probabilistic evaluation of the S2 specimen for different reinforcement arrangements, the fitted lognormal probability density functions, and the probability of catenary activation

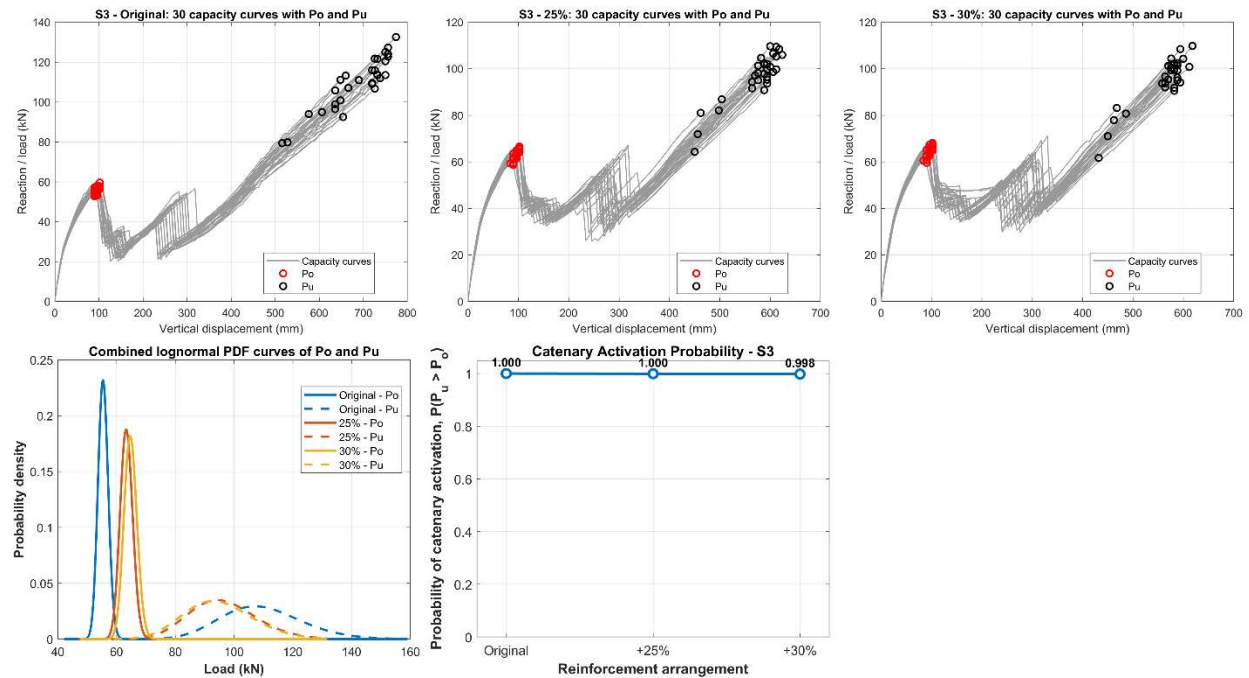


Figure 69: Probabilistic evaluation of the S3 specimen for different reinforcement arrangements, the fitted lognormal probability density functions, and the probability of catenary activation

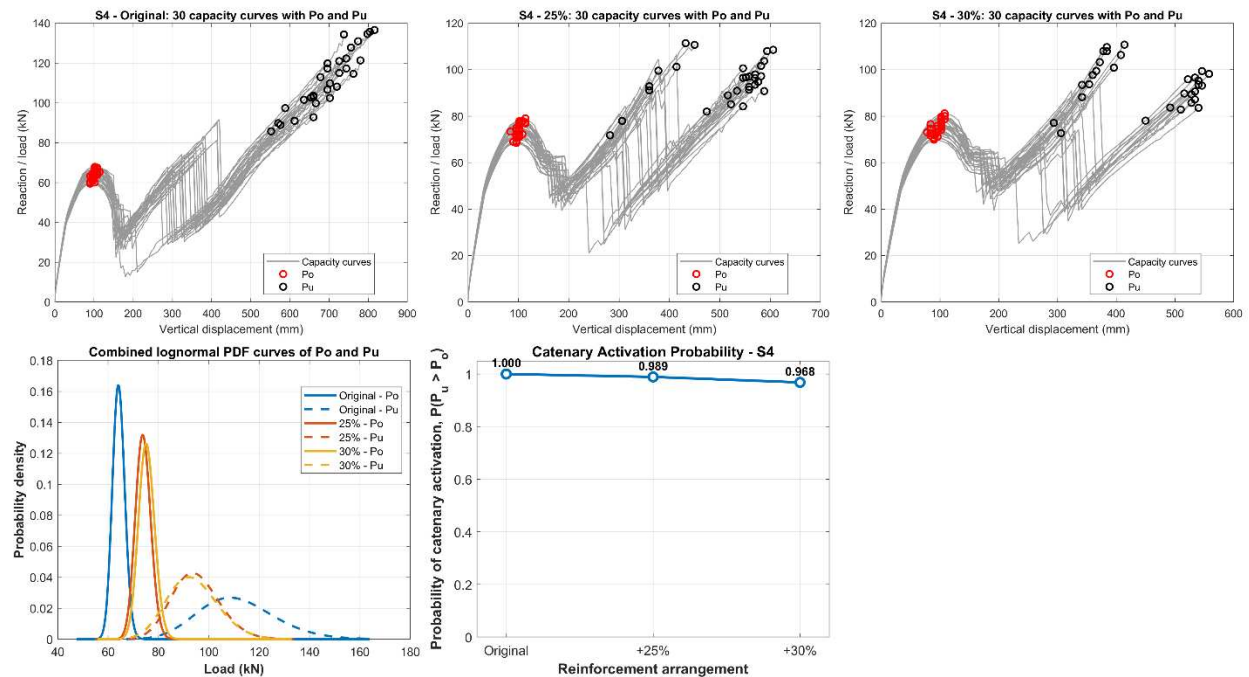


Figure 70: Probabilistic evaluation of the S4 specimen for different reinforcement arrangements, the fitted lognormal probability density functions, and the probability of catenary activation

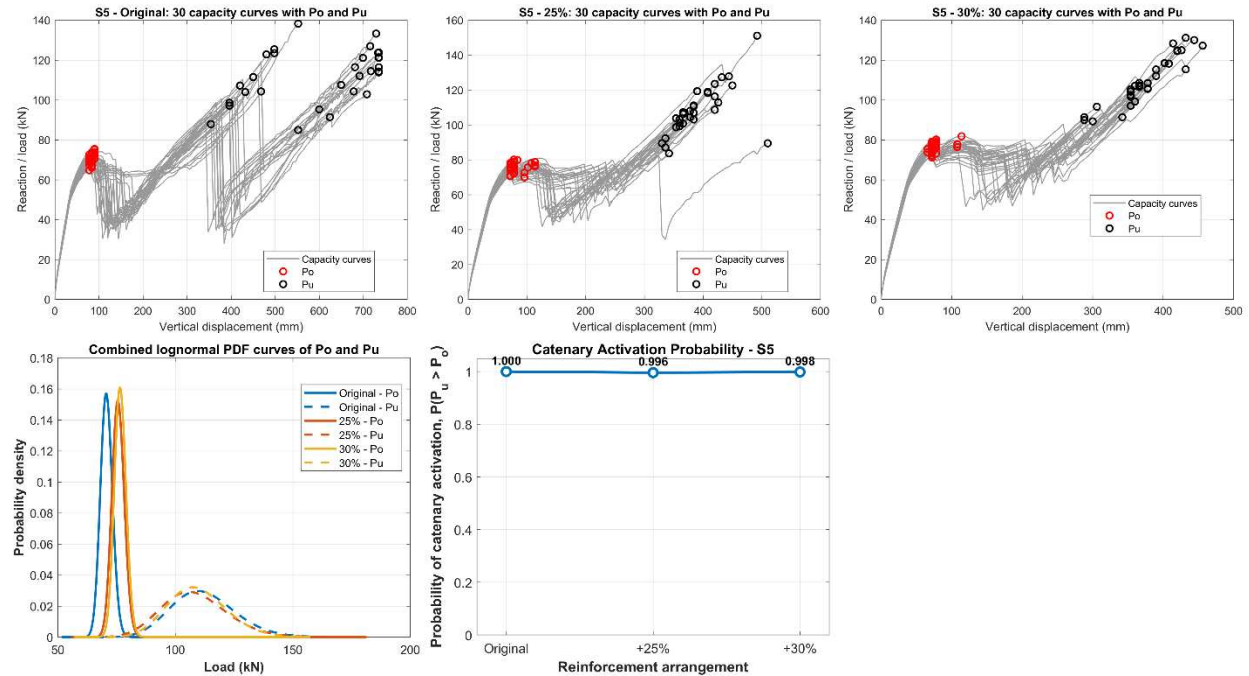


Figure 71: Probabilistic evaluation of the S5 specimen for different reinforcement arrangements, the fitted lognormal probability density functions, and the probability of catenary activation

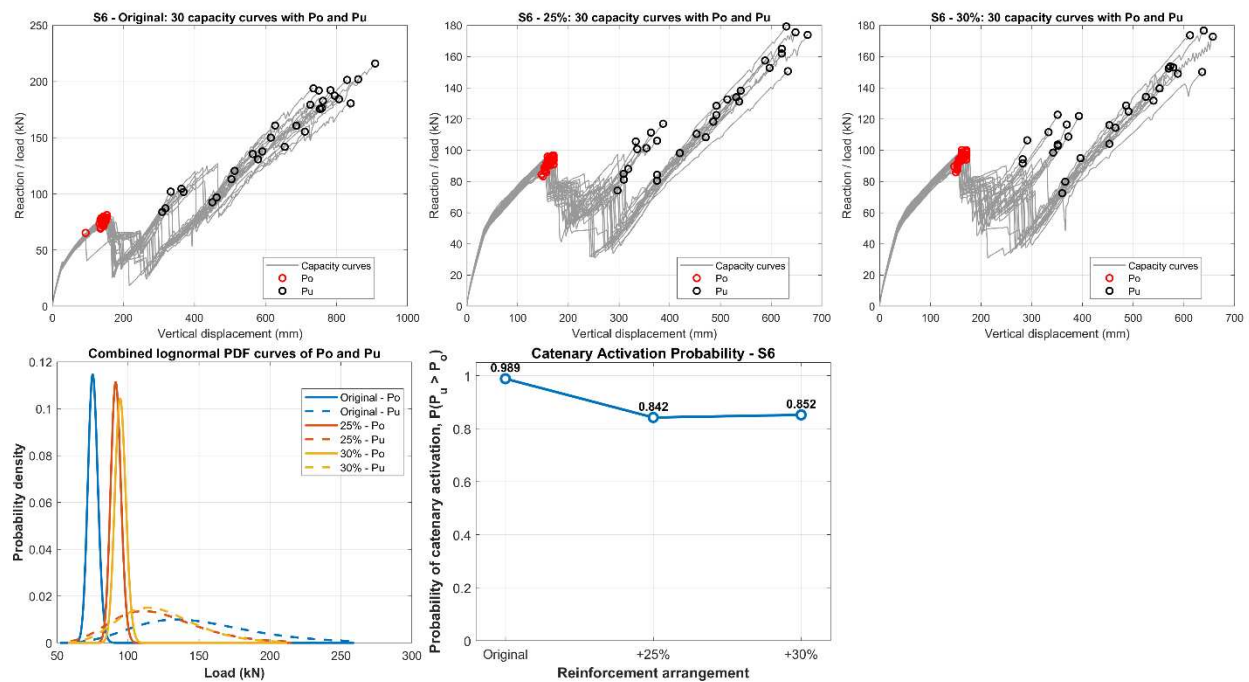


Figure 72: Probabilistic evaluation of the S6 specimen for different reinforcement arrangements, the fitted lognormal probability density functions, and the probability of catenary activation

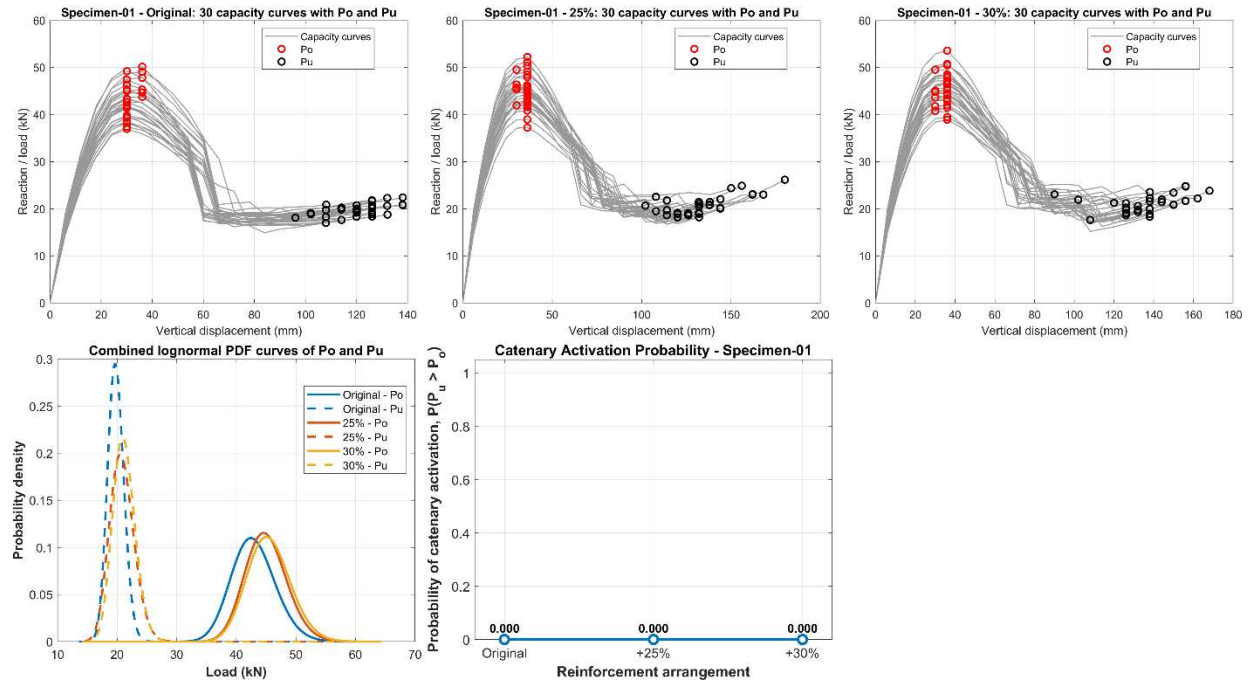


Figure 73: Probabilistic evaluation of the Specimen-01 specimen for different reinforcement arrangements, the fitted lognormal probability density functions, and the probability of catenary activation

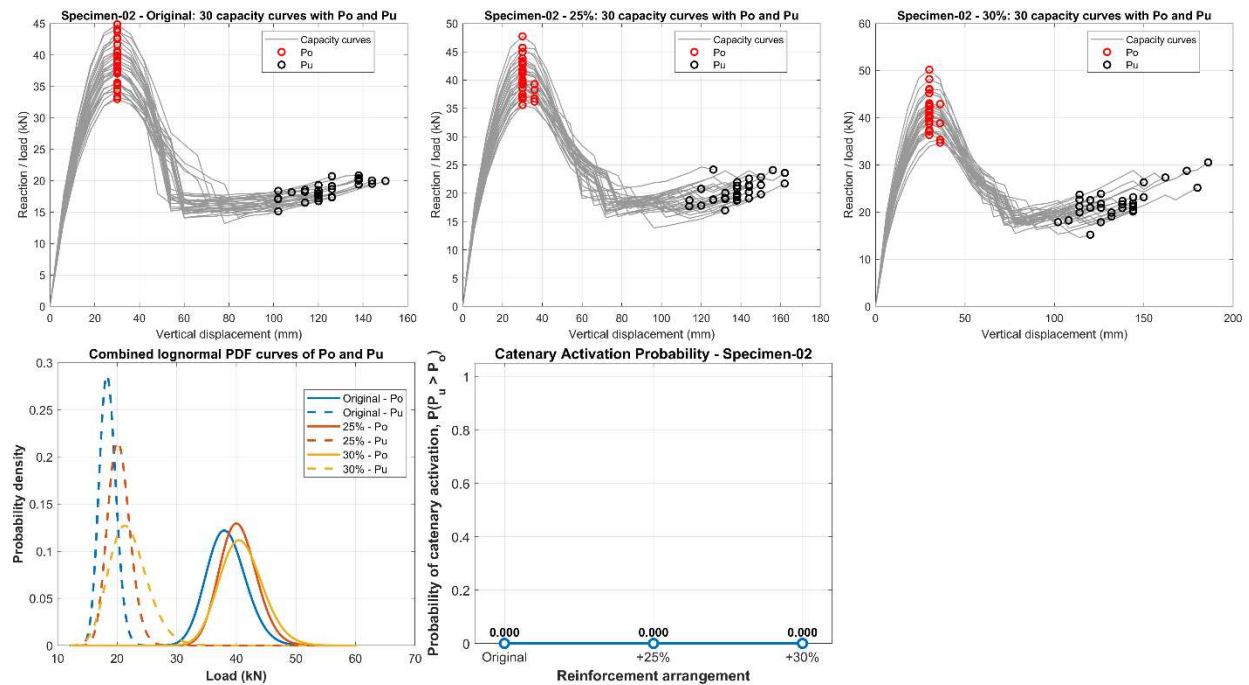


Figure 74: Probabilistic evaluation of the Specimen-02 specimen for different reinforcement arrangements, the fitted lognormal probability density functions, and the probability of catenary activation

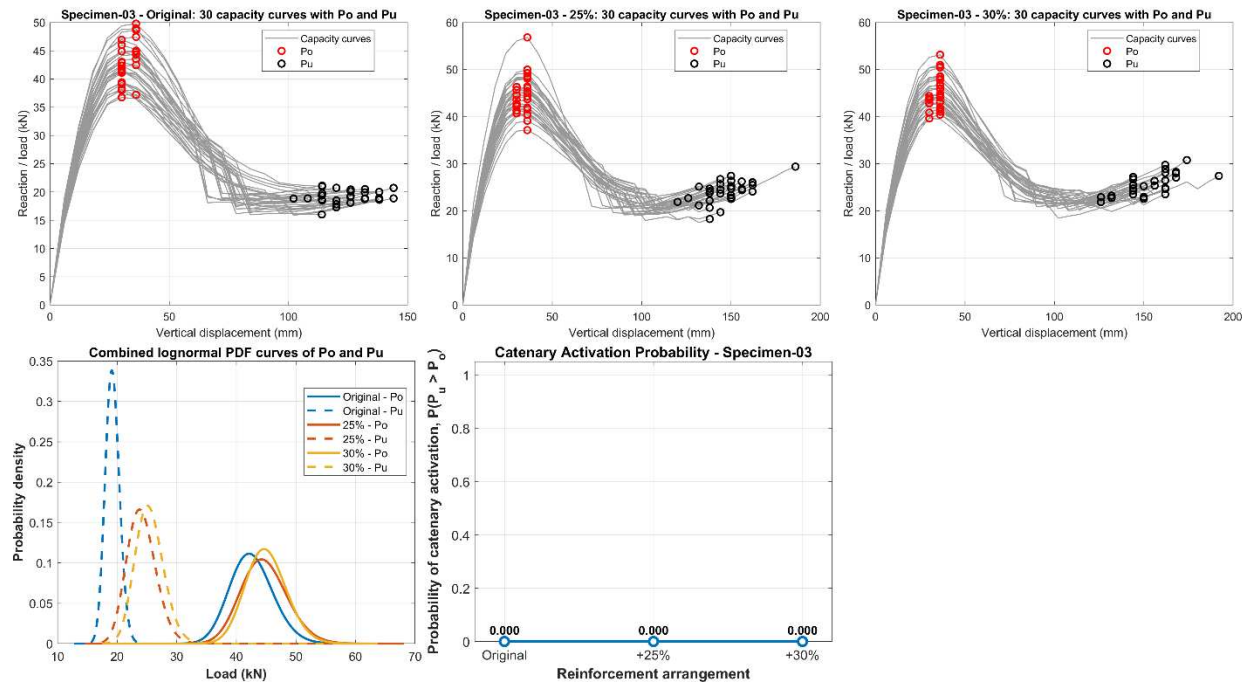


Figure 75: Probabilistic evaluation of the Specimen-03 specimen for different reinforcement arrangements, the fitted lognormal probability density functions, and the probability of catenary activation

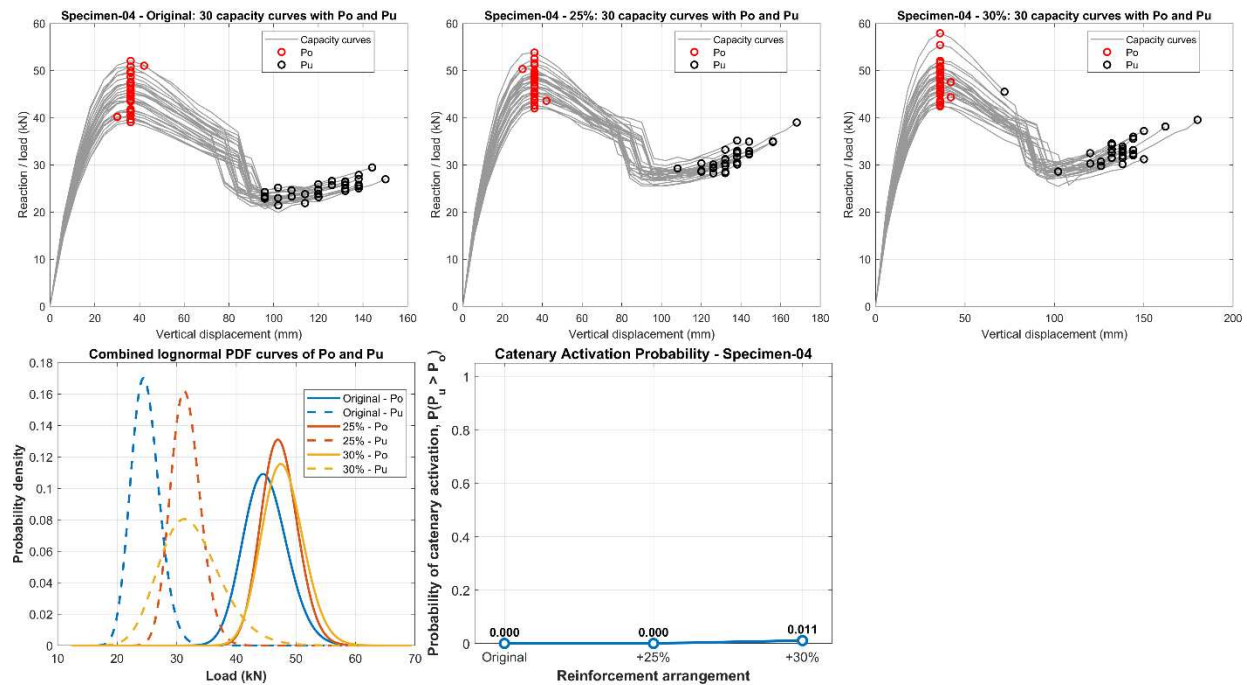


Figure 76: Probabilistic evaluation of the Specimen-04 specimen for different reinforcement arrangements, the fitted lognormal probability density functions, and the probability of catenary activation

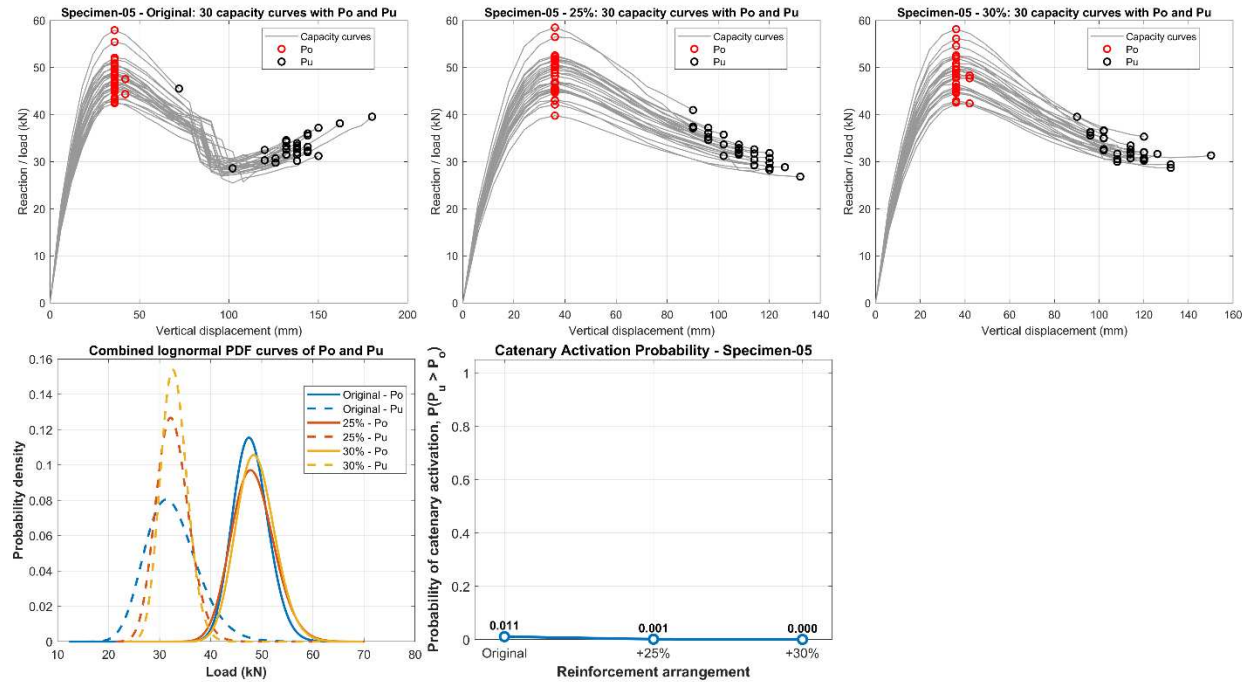


Figure 77: Probabilistic evaluation of the Specimen-05 specimen for different reinforcement arrangements, the fitted lognormal probability density functions, and the probability of catenary activation

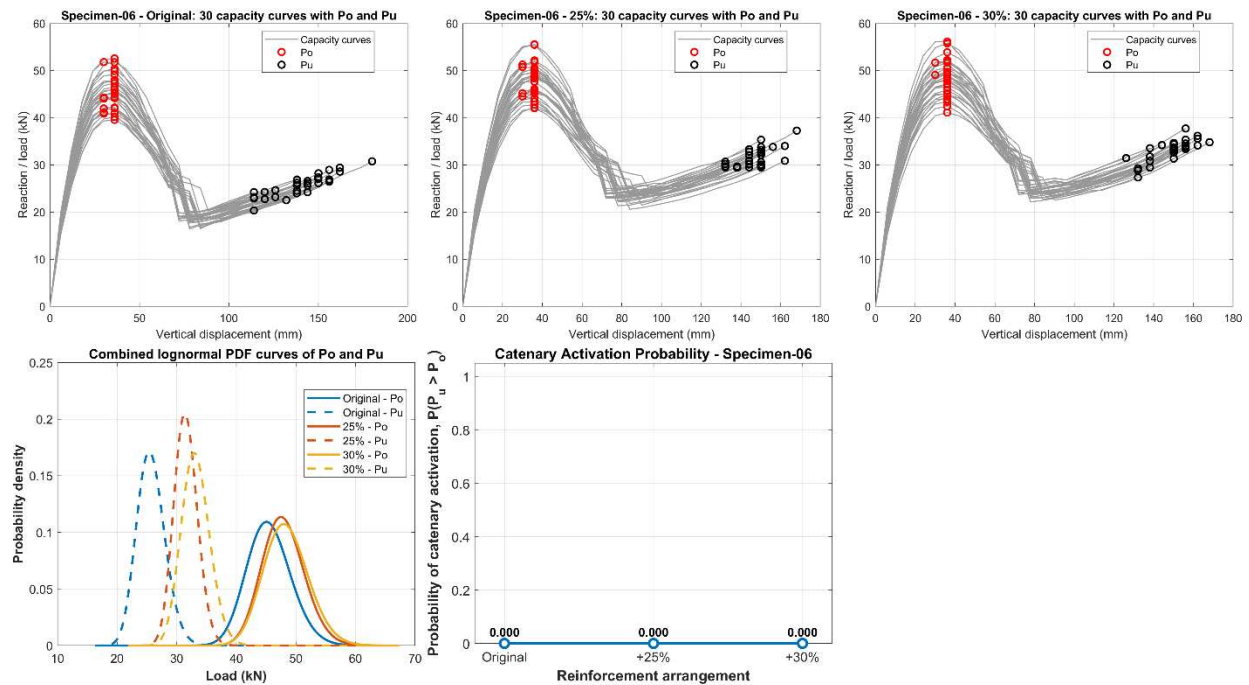


Figure 78: Probabilistic evaluation of the Specimen-06 specimen for different reinforcement arrangements, the fitted lognormal probability density functions, and the probability of catenary activation

6.6 Detailed interpretation of the improvement groups

The behavior of Group A is particularly important because it identifies the configurations where reinforcement enhancement changed the probability of mechanism activation, not only the magnitude of resistance. B3, IR-1, Mehrdad, and S7 had original activation probabilities below the nearly saturated range and therefore provided enough margin for the additional reinforcement to modify the probabilistic response. These specimens are the clearest evidence that the added intermediate bars can improve catenary reliability when the baseline system is deformable enough to mobilize tensile membrane action. [7]

B3 is a representative example of strong improvement because its original activation probability was low, while the +25% and +30% models increased both the activation probability and the mean P_u . This behavior is consistent with the experimental interpretation of Ren et al., where B3 had a comparatively high beam mechanism but weaker catenary performance than B2. The improved numerical models suggest that additional reinforcement can partially compensate for this limitation by supporting the large-deformation tensile stage. [9]

IR-1 also belongs to Group A, but its interpretation is different from B3. The original IR-1 model already had moderate activation probability, and the improvement increased both activation reliability and ultimate resistance. This response suggests that the system was not completely restrained like FR, but still retained enough deformation capacity and load-path continuity for the added reinforcement to become effective. [10]

Mehrdad shows a strong activation improvement combined with a meaningful increase in P_u . This result is important because the corresponding experimental background emphasizes bar fracture and post-fracture redistribution. The improved configuration appears to provide a more stable residual load path after the initial degradation stage, which is consistent with the idea that catenary action can continue after local fracture if sufficient reinforcement continuity remains available. [5]

S7 presents a special case within Group A. The activation probability increased strongly, but the mean P_u did not increase compared with the Original model. This means that the improvement made the catenary branch more statistically reliable relative to P_o , but did not increase the absolute ultimate resistance. This distinction is useful because it shows that activation probability and load capacity do not always move in the same direction. [7]

Group B contains cases in which the improvement was meaningful but not necessarily visible as a large activation-probability increase. B2, FR, and IMF are representative examples because their original activation probabilities were already high. In these cases, activation probability was close to saturation, so the main benefit of reinforcement

enhancement appeared through increases in P_u rather than through large changes in activation probability. [2], [20]

B2 is especially important because its activation probability was already close to one, but the +25% and +30% models produced a large increase in P_u . This means that B2 did not need reinforcement enhancement to activate catenary action, but the added reinforcement increased the strength level at which the mechanism operated. Such behavior should be interpreted as capacity improvement under saturated activation rather than as a new activation mechanism. [9]

IMF and SMF demonstrate the importance of distinguishing between absolute capacity and relative reserve. The IMF model showed a strong P_u increase while activation probability reached saturation after improvement, whereas the SMF model already had very high activation probability and very high absolute resistance. This result is consistent with the NIST interpretation that stronger detailing improves large-deformation performance but may also reduce the relative margin available for additional activation-probability improvement. [2]

Group C contains the largest number of specimens, but this does not mean that the reinforcement enhancement failed in a simple sense. The group includes multiple behavioral categories: specimens with already saturated activation probability, specimens where P_u decreased after adding reinforcement, and specimens whose original mechanism was not catenary-dominated. Therefore, Group C must be interpreted through the controlling mechanism of each specimen rather than through a single generalized explanation. [15]

The FarhangVesali specimens remained close to zero activation probability because their response was primarily associated with compressive arch action in restrained beams rather than a fully developed catenary mechanism. The increase in P_u observed in some of these models therefore does not necessarily indicate true catenary activation. This confirms the experimental lesson that arching-controlled specimens should not be judged using the same criteria as beam-column sub-assemblages designed to reach large tensile membrane action. [12]

Several Yu and Tan and Qian specimens also fall into Group C because their original activation probabilities were already high, while the added reinforcement increased P_o more than P_u or reduced the relative advantage of the catenary branch. This behavior explains why a reinforcement increase may reduce the calculated activation probability even when the model remains structurally strong. In probabilistic terms, the threshold represented by P_o moved upward, while P_u did not increase proportionally. [7], [8]

The group interpretation therefore leads to a central conclusion of the comparative analysis: reinforcement enhancement is beneficial only when it is compatible with the controlling mechanism of the specimen. If the specimen is tensile-mechanism-sensitive, added reinforcement improves catenary action. If the specimen is arching-controlled, shear-sensitive, restraint-limited, or already saturated in activation, the benefit may be limited, indirect, or visible only in capacity rather than probability. [15], [20]

6.7 Mechanism-based interpretation of the comparative results

The comparison confirms that the additional reinforcement did not act uniformly across all stages of response. The P_o values increased in most specimens because adding steel modifies the sectional stiffness and the early resistance, but the principal objective of the enhancement was the large-displacement response. Therefore, the most meaningful indicator of the improved models is not the absolute increase in P_o , but the behavior of P_u and the probability of catenary activation. [7], [9]

The cases where P_u increased significantly can be interpreted as reinforcement-sensitive catenary responses. In these models, the additional intermediate reinforcement layers were able to participate in tensile force transfer after major cracking and stiffness degradation. This behavior is consistent with experimental evidence showing that catenary action depends strongly on reinforcement continuity, anchorage, and the ability of the reinforcement to sustain large tensile strains. [2], [5]

The cases where P_u decreased or activation probability reduced should not be interpreted as computational errors automatically. In nonlinear reinforced concrete systems, increasing reinforcement quantity may change the balance between flexural, arching, and tensile mechanisms. If the response is controlled by concrete crushing, shear localization, joint degradation, or boundary restraint, additional longitudinal reinforcement may increase early stiffness or P_o without improving the final catenary resistance. [8], [15]

This mechanism-based interpretation is particularly important for the Yu and Tan and Qian specimens, where some models already had high original activation probabilities and strong catenary branches. In these cases, the additional reinforcement sometimes shifted the response toward a higher P_o without producing a proportional increase in P_u . The result is a smaller or even negative activation-probability change despite the presence of added reinforcement. [7], [8]

6.8 Comparison between +25% and +30% reinforcement

The comparison between the +25% and +30% configurations shows that the additional 5% reinforcement increase did not always produce a proportional improvement. Although the +30% configuration produced a higher global average P_u than the +25% configuration, the

activation probability was nearly the same. This indicates that the reinforcement enhancement reached a saturation point in several specimens, beyond which other mechanisms controlled the response.

In some specimens, +30% clearly improved the catenary resistance relative to +25%, as observed in B2, IMF, SMF, IR-1, and several Farhang Vesali specimens. These cases suggest that the additional reinforcement area was effectively mobilized in the large-deformation stage or contributed to increased residual resistance.

In other specimens, the +30% configuration produced little additional benefit or even a slight reduction compared with +25%. This behavior is structurally plausible because the governing failure mode may move away from reinforcement tensile capacity and toward local crushing, shear damage, support restraint, or unstable deformation geometry. Therefore, the comparison suggests that +30% reinforcement should not be assumed to be automatically superior for every specimen. [9], [15]

6.9 Chapter conclusions

This chapter compared the Original, +25%, and +30% reinforcement configurations using the probabilistic outputs generated in Chapters 4 and 5. The comparison showed that the improved models generally increased the average P_o and P_u values, with the larger improvement occurring in P_u . This confirms that the reinforcement enhancement strategy mainly affected the catenary-stage response rather than only the initial flexural/arching resistance.

The activation-probability comparison revealed a more nuanced result. Some specimens exhibited strong activation improvement, some were already saturated in the Original configuration, and others remained limited despite the added reinforcement. This confirms that catenary action is a system mechanism governed not only by the amount of reinforcement but also by anchorage, restraint, deformation capacity, and local failure mode. [2], [15]

The main conclusion is that reinforcement enhancement is effective when the structural system is mechanically capable of mobilizing catenary action. When premature local failure, arching-dominated response, or inadequate deformation capacity controls the behavior, increasing longitudinal reinforcement alone may not be sufficient. This finding provides the basis for the final conclusions and design-oriented recommendations of the thesis. [7], [20]

CHAPTER 7: Conclusions and Future Developments

7.1 General conclusions

This thesis investigated the robustness of reinforced concrete structures subjected to column-loss scenarios through a combined experimental, numerical, probabilistic, and comparative framework. The work started from the interpretation of structural robustness as the ability of a system to tolerate local damage without disproportionate collapse, and then progressively moved from literature-based evidence to nonlinear finite element modelling and probabilistic evaluation. The study confirmed that the response of reinforced concrete sub-assemblages after column removal cannot be described by a single resistance value, because the load-carrying mechanism changes with deformation. Flexural action governs the early stage, compressive arch action may increase the resistance when axial restraint is available, and catenary action can provide residual capacity at large displacement if reinforcement continuity, anchorage, and deformation capacity are sufficient. [4], [7], [15]

The experimental database developed in Chapter 2 provided the conceptual foundation for the whole thesis. The selected studies showed that progressive collapse resistance is controlled by the interaction between global load redistribution and local damage mechanisms such as reinforcement fracture, bond-slip, concrete crushing, joint degradation, and support instability. This interpretation was essential because it prevented the later numerical work from being treated as a purely computational exercise. Instead, the ATENA models and the probabilistic analyses were interpreted against the physical mechanisms observed in experimental campaigns, especially the transition between flexural behavior, compressive arch action, and catenary action. [2], [5], [9]

7.2 Main methodological outcomes

The nonlinear finite element phase established a consistent numerical platform for reproducing the behavior of the selected reinforced concrete specimens. ATENA 2D was used to represent concrete cracking, stiffness degradation, reinforcement yielding, large-displacement effects, and the post-peak response of the sub-assemblages. The calibrated models were not judged only by their maximum load, but by their ability to reproduce the sequence of mechanisms and the general shape of the capacity curves. This mechanism-based calibration was important because the later probabilistic analysis depended directly on the reliability of the numerical response, particularly in the large-deformation range where catenary action is activated. [8], [13], [16]

A major methodological contribution of the thesis was the integration between ATENA, Excel-based model templates, and MATLAB routines. After calibration, the material

parameters of each specimen were organized in structured Excel files, probabilistic realizations were generated by Latin Hypercube Sampling, and the resulting data were converted into ATENA-compatible CCT files. This workflow made it possible to generate thirty probabilistic sub-samples for each baseline specimen and to repeat the same process for the improved reinforcement configurations. As a result, the thesis moved from individual deterministic models toward a structured numerical database suitable for statistical interpretation. [17], [18], [20]

7.3 Conclusions from the improved-model comparison

The improved models were developed by adding 25% and 30% longitudinal reinforcement relative to the original reinforcement amount. The additional reinforcement was positioned as side face rebars, approximately at one-third and two-thirds of the beam depth, with the intention of enhancing tensile force redistribution during large deformation. The same probabilistic workflow used for the original models was then repeated for the improved configurations. This allowed the original, +25%, and +30% models to be compared using common indicators: P_o , P_u , and the probability of catenary activation. [2], [7], [21]

The comparative analysis in Chapter 6 showed that the reinforcement enhancement generally increased the average P_o and P_u values, but the improvement was more meaningful in the catenary-stage resistance than in the first peak. This result is consistent with the purpose of the strengthening strategy: the added intermediate reinforcement was not primarily intended to increase the initial flexural or arching response, but to improve the tensile membrane mechanism at large displacement. Specimens such as B3, IR-1, Mehrdad, and S7 showed clear activation improvement, while B2, IMF, and SMF demonstrated that high activation probability and capacity increase must be interpreted together because some specimens were already close to activation saturation in the original configuration. [2], [9]

The results also demonstrated that additional reinforcement does not automatically guarantee a proportional improvement in robustness. In some specimens, the added reinforcement increased P_o more than P_u or did not overcome the governing failure mechanism. This means that the effectiveness of strengthening depends on whether the specimen is capable of mobilizing catenary action in the first place. When the response is controlled by tensile reinforcement continuity and deformation capacity, additional reinforcement can be beneficial. When the response is governed by concrete crushing, shear localization, restraint limitations, or arching-dominated behavior, increasing longitudinal reinforcement alone may provide only limited benefit. [8], [12], [15]

The comparison between the +25% and +30% configurations indicated that the additional 5% reinforcement increase did not always produce a proportional gain. Although the +30%

models generally produced slightly higher average P_u values, the activation probability did not increase substantially compared with the +25% models. This suggests that several specimens reached a saturation point where other mechanisms, rather than reinforcement quantity, controlled the response. Therefore, the +30% configuration should not be interpreted as universally superior; instead, the optimal reinforcement enhancement depends on the governing mechanism and on the balance between added tensile capacity, deformation demand, and local failure resistance. [7], [20]

7.4 Future developments

Future work should extend the present framework in three main directions. First, selected specimens could be reanalyzed using refined modelling strategies that explicitly represent bond-slip, reinforcement fracture, and local joint degradation. Second, the improved reinforcement concept could be investigated through additional configurations, including different reinforcement positions, anchorage strategies, and strengthening ratios beyond the two values considered here. Third, the probabilistic framework could be expanded to include uncertainty in boundary stiffness, loading protocol, and geometric parameters, not only material properties. These developments would provide a more comprehensive reliability-based assessment of robustness under realistic column-loss conditions. [8], [15], [18]

Overall, the thesis confirms that robustness in reinforced concrete structures is a mechanism-dependent and uncertainty-sensitive property. The most reliable improvement is achieved when additional reinforcement is compatible with the structural system's ability to develop catenary action. The work therefore supports a design philosophy in which robustness is not evaluated only by initial strength, but by the capacity of the structure to redistribute forces, sustain large deformation, and preserve alternative load paths after severe local damage. [4], [7]

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