Residual strength of blast damaged reinforced concrete columns

Nair, Anand

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**NANYANG
TECHNOLOGICAL
UNIVERSITY**

**RESIDUAL STRENGTH OF BLAST DAMAGED
REINFORCED CONCRETE COLUMNS**

**ANAND NAIR
SCHOOL OF CIVIL & ENVIRONMENTAL
ENGINEERING
2010**

RESIDUAL STRENGTH OF BLAST DAMAGED REINFORCED CONCRETE COLUMNS

ANAND NAIR

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fulfillment of the requirements for the degree of
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TABLE OF CONTENTS

INTRODUCTION	1
1.1 Problem Definition.....	1
1.2 Objectives and Scope	2
1.2.1 Numerical Analysis.....	2
1.2.2 Experimental Study.....	3
1.3 Organization of Report.....	4
 LITERATURE REVIEW	 6
2.1 Introduction	6
2.2 Blast Basics	8
2.2.1 Explosion, Detonation and Deflagration	8
2.2.2 Blast Phenomenon	8
2.2.3 Prediction of Blast Pressure.....	12
2.2.4 Blast Loading.....	14
2.3 Structure Dynamics.....	15
2.4 Structural Response to Blast Loading	16
2.4.1 Elastic SDOF Systems	17
2.4.2 Elasto-Plastic SDOF Systems.....	20
2.5 Material Behaviours at High Strain Rate	21
2.5.1 Dynamic Properties of Concrete under High Strain Rates	22
2.5.2 Dynamic Properties of Reinforcing Steel under High Strain Rates.....	24
2.6 Research Works on Blast Effects on Reinforced Concrete Columns	25
2.6.1 Crawford <i>et al.</i>	25
2.6.2 Krauthammer <i>et al.</i>	34
2.6.3 Hegemier <i>et al.</i>	36
2.6.4 Hayes <i>et al.</i>	38
2.6.5 Residual Axial Capacity of Columns.....	38
2.6.6 Bao and Li.....	39

Table of Contents

NUMERICAL STUDY	40
3.1 Introduction	40
3.2 Blast Simulation Techniques	41
3.2.1 LS-DYNA	41
3.2.2 CONWEP	41
3.3 Finite Element Models	42
3.3.1 Blast Load Calculations	42
3.3.2 Structural Geometry Modeling	45
3.3.3 Concrete Element	53
3.3.4 Reinforcement Element	57
3.3.5 Interaction between Reinforcement and Concrete Element	61
3.3.6 Material Modeling and Parameter Selection	62
3.3.7 Boundary Conditions	78
3.3.8 Analysis Steps and Load Patterns	79
3.3.9 Integration Method Selection	80
3.4 Finite Element Analysis	83
3.4.1 FEM Model of Specimen S1(0.58)	84
3.4.2 FEM Model of Specimen S2(0.19)	90
3.4.3 FEM Model of Specimen S3(0.58)	97
3.4.4 FEM Model of Specimen S4(0.19)	104
3.5 Summary of Numerical Results	111
EXPERIMENTAL STUDY	113
4.1 Introduction	113
4.2 Experiment Specimens	113
4.2.1 Column Specimen Details	114
4.2.2 Column Specimen Materials	117
4.3 Experiment Setup	118
4.3.1 Axial Loading Frame	120
4.3.2 A-Frame	122

Table of Contents

4.3.3	Lateral Load Actuators	125
4.3.4	Axial Load Actuators.....	126
4.4	Test Procedure.....	127
4.4.1	Pre-Axial Loading Stage.....	128
4.4.2	Lateral Loading Stage.....	128
4.4.3	Residual Axial Capacity Stage	129
4.5	Instrumentation	130
4.5.1	Actuators.....	130
4.5.2	Transducers.....	131
4.6	Experimental Results	134
4.6.1	Results of Specimen <i>S1(0.58)</i>	135
4.6.2	Results of Specimen <i>S2(0.19)</i>	139
4.6.3	Results of Specimen <i>S3(0.58)</i>	143
4.6.4	Results of Specimen <i>S4(0.19)</i>	147
4.7	Summary of Experimental Results.....	151
SUMMARY OF NUMERICAL & EXPERIMENTAL RESULTS		152
5.1	Introduction	152
5.2	Residual Axial Capacity Prediction Equation.....	152
5.3	Specimen <i>S1(0.58)</i>	153
5.3.1	Lateral Displacements of <i>S1(0.58)</i>	154
5.3.2	Residual Axial Capacity of <i>S1(0.58)</i>	155
5.3.3	Prediction of Numerical and Experimental Residual Axial Capacity of <i>S1(0.58)</i>	157
5.4	Specimen <i>S2(0.19)</i>	158
5.4.1	Lateral Displacements of <i>S2(0.19)</i>	158
5.4.2	Residual Axial Capacity of <i>S2(0.19)</i>	160
5.4.3	Prediction of Numerical and Experimental Residual Axial Capacity of <i>S2(0.19)</i>	162
5.5	Specimen <i>S3(0.58)</i>	163

Table of Contents

5.5.1	Lateral Displacements of $S3(0.58)$	163
5.5.2	Residual Axial Capacity of $S3(0.58)$	165
5.5.3	Prediction of Numerical and Experimental Residual Axial Capacity of $S3(0.58)$	166
5.6	Specimen $S4(0.19)$	168
5.6.1	Lateral Displacements of $S4(0.19)$	168
5.6.2	Residual Axial Capacity of $S4(0.19)$	170
5.6.3	Prediction of Numerical and Experimental Residual Axial Capacity of $S4(0.19)$	171
5.7	Comparing Results of Various Specimens.....	173
5.7.1	Lateral Displacements.....	173
5.7.2	Residual Axial Capacity	175
5.7.3	Predicting Residual Axial Capacity	176
SUMMARY, CONCLUSIONS & RECOMMENDATIONS.....		178
6.1	Summary	178
6.2	Conclusions	179
6.3	Observations & Recommendations.....	180

ABSTRACT

Columns are key load-bearing elements that hold up framed structures. Exterior columns are probably the most vulnerable structural components to attacks from rebel forces. Their failure could possibly trigger a progressive collapse of an entire structure. Current knowledge of blast-damaged axial load carrying capacity of reinforced concrete columns is rather limited. A better understanding of column behaviour when subjected to blast loadings would be able to provide essential information on its damage assessment enabling the forecasting of a progressive collapse of a structure.

An explosion creates a rapid release of a massive amount of energy. This energy usually takes the form of light, heat, sound and a shock wave. This shock wave is made up of condensed air pressures and travels outwards at supersonic velocities. When this wave encounters a building structure, it could cause it extreme damage if not designed to resist blast effects.

Numerical and experimental studies were carried out to determine the response and behaviour of columns when subjected to blast loadings. The numerical approach utilized computer simulation of blast effects on a specimen modeled to represent actual column specimens. The deflections obtained from the numerical analysis were used to recreate the damaged profile attained by the model on actual column specimens in a laboratory environment. Hydraulically powered actuators were used to push out a column to the blast damaged profile obtained from the computer-generated model. The effects of parameters such as pre-axial loading and transverse reinforcement ratio are investigated in this study.

The performance of the tested column models and specimens subjected blast or lateral loads was found to have enhanced appreciably by providing an increased ratio of transverse reinforcement. Pre-axial loading prescribed on columns were also found to have an effect on the blast-damaged residual deflected profile and residual axial capacity of the column specimens and models tested.

LIST OF FIGURES

Figure 2.1 Blast wave propagation	9
Figure 2.2 Blast wave pressure-time history	10
Figure 2.3 SDOF system [B3]	17
Figure 2.4 Idealized blast loading [B3]	18
Figure 2.5 Simplified resistance function of a elasto-plastic SDOF system	20
Figure 2.6 Maximum response of elasto-plastic SDOF system to triangular load [T6]	21
Figure 2.7 Strain rates associated with different types of loading [N7]	21
Figure 2.8 Stress-strain rates of concrete at different strain rates [N7]	22
Figure 2.9 Dynamic increase factor for peak stress of concrete [C1]	23
Figure 2.10 CTS-1 prior to being tested [C7]	26
Figure 2.11 Two views of column DB6 [C7]	27
Figure 2.12 Influence of CFRP wrap on columns subjected to blast loading	28
Figure 2.13 Field test setup developed by K&C [C7]	29
Figure 2.14 Column response from K&C field tests [C7]	30
Figure 2.15 Schematic of actuator system for laboratory experiments [C7]	31
Figure 2.16 Photograph of 5-actuator system laboratory test [I1]	32
Figure 2.17 Comparison between column DB6 (field test) and the laboratory specimen ...	33
Figure 2.18 Post-test view of field experiment [K8]	34
Figure 2.19 Comparison of experimental and predicted column response [K8]	35
Figure 2.20 Explosive Loading Laboratory Testing Program setup [H4]	36
Figure 2.21 Comparison between field and blast simulator experiments	37
Figure 3.1 Succession of blast pressure on a building [M11]	43
Figure 3.2 Blast loadings on a first storey column [N4]	44
Figure 3.3 Typical reinforcement detail of $S1(0.58)$ & $S3(0.58)$	46
Figure 3.4 Typical model of $S1(0.58)$ & $S3(0.58)$	47
Figure 3.5 Typical reinforcement detail within models of $S1(0.58)$ & $S3(0.58)$	48
Figure 3.6 Typical reinforcement details of Specimens $S2(0.19)$ & $S4(0.19)$	50
Figure 3.7 Typical model of $S2(0.19)$ & $S4(0.19)$	51

List of Figures

Figure 3.8 Typical reinforcement detail within model of $S2(0.19)$ & $S4(0.19)$	52
Figure 3.9 Eight-node solid hexahedron element [L4]	53
Figure 3.10 Hughes-Liu beam element formulation [L4]	58
Figure 3.11 LS-DYNA beam definition [L4]	61
Figure 3.12 Interaction between concrete and reinforcement elements	62
Figure 3.13 Failure surface of concrete in a 3-D stress space [C2]	64
Figure 3.14 Cross-section of the failure surface [M6]	65
Figure 3.15 Pressure versus volumetric strain curve [L4]	66
Figure 3.16 Strength model for concrete material [M6]	67
Figure 3.17 Effects of damage parameters b_1 , b_2 and b_3 [M6]	71
Figure 3.18 DIF for concrete in compression [M3]	72
Figure 3.19 DIF for concrete in tension [M3]	73
Figure 3.20 Rate Enhancement in Tension and Compression [M6]	74
Figure 3.21 Uniaxial behaviour of reinforcing steel bars	76
Figure 3.22 Proposed DIF for reinforcing steel bars [M4]	76
Figure 3.23 Table of curves to represent stress-strain curves at different strain rates [L4] ..	78
Figure 3.24 Boundary conditions of finite element model of column	79
Figure 3.25 Loading steps in simulation	80
Figure 3.26 Numerical residual displacement of $S1(0.58)$	85
Figure 3.27 Numerical x-displacement of $S1(0.58)$	86
Figure 3.28 Numerical residual deflection profile of $S1(0.58)$	87
Figure 3.29 Numerical z-displacement of $S1(0.58)$	88
Figure 3.30 Numerical residual axial capacity of $S1(0.58)$	89
Figure 3.31 Numerical damage-profile of $S1(0.58)$	90
Figure 3.32 Numerical residual displacement of $S2(0.19)$	92
Figure 3.33 Numerical x-displacement of $S2(0.19)$	93
Figure 3.34 Numerical residual deflection profile of $S2(0.19)$	94
Figure 3.35 Numerical z-displacement of $S2(0.19)$	95
Figure 3.36 Numerical residual axial capacity of $S2(0.19)$	96
Figure 3.37 Numerical damage-profile of $S2(0.19)$	97

List of Figures

Figure 3.38 Numerical residual displacement of $S3(0.58)$	99
Figure 3.39 Numerical x-displacement of $S3(0.58)$	100
Figure 3.40 Numerical residual deflection profile of $S3(0.58)$	101
Figure 3.41 Numerical z-displacement of $S3(0.58)$	102
Figure 3.42 Numerical residual axial capacity of $S3(0.58)$	103
Figure 3.43 Numerical damage-profile of $S3(0.58)$	104
Figure 3.44 Numerical residual displacement of $S4(0.19)$	106
Figure 3.45 Numerical x-displacement of $S4(0.19)$	107
Figure 3.46 Residual deflection profile of $S4(0.19)$	108
Figure 3.47 Numerical z-displacement of $S4(0.19)$	109
Figure 3.48 Numerical residual axial capacity of $S4(0.19)$	110
Figure 3.49 Numerical damage-profile of $S4(0.19)$	111
Figure 4.1 Typical geometry and section details of $S1(0.58)$ & $S3(0.58)$	115
Figure 4.2 Typical geometry and section details of $S2(0.19)$ & $S4(0.19)$	117
Figure 4.3 Test setup (elevation view)	119
Figure 4.4 Photograph of test setup	120
Figure 4.5 Front view of test setup	121
Figure 4.6 Rear view of test setup	121
Figure 4.7 Location for commencing experiments	122
Figure 4.8 A-frame and transfer beam in experiment setup	123
Figure 4.9 I-beams and concrete spacer block that support A-frame	124
Figure 4.10 Connection between Column Head, Steel Plates, Concrete Spacer Block and Transfer Beams	125
Figure 4.11 Lateral actuators and strong wall mounting	126
Figure 4.12 Vertically mounted actuator and connection details	127
Figure 4.13 Wire transducers used for the first column specimen	132
Figure 4.14 100 mm LVDT for measuring base slip	133
Figure 4.15 300 mm LVDT for measuring lateral displacement	134
Figure 4.16 Crack development pattern of $S1(0.58)$	136
Figure 4.17 $S1(0.58)$ Cracks from lateral loads	136

List of Figures

Figure 4.18 Experimental Residual Deflection of <i>S1(0.58)</i>	137
Figure 4.19 Experimental residual axial capacity of specimen <i>S1(0.58)</i>	138
Figure 4.20 Crack development pattern of <i>S2(0.19)</i>	140
Figure 4.21 <i>S2(0.19)</i> Cracks from lateral loads	140
Figure 4.22 Experimental Residual Deflection of <i>S2(0.19)</i>	141
Figure 4.23 Experimental residual axial capacity of specimen <i>S2(0.19)</i>	142
Figure 4.24 Crack development pattern of <i>S3(0.58)</i>	144
Figure 4.25 <i>S3(0.58)</i> Cracks from lateral loads	144
Figure 4.26 Experimental Residual Deflection of <i>S3(0.58)</i>	145
Figure 4.27 Experimental residual axial capacity of specimen <i>S3(0.58)</i>	146
Figure 4.28 Crack development pattern of <i>S4(0.19)</i>	148
Figure 4.29 <i>S4(0.19)</i> Cracks from lateral loads	148
Figure 4.30 Experimental Residual Deflection of <i>S4(0.19)</i>	149
Figure 4.31 Experimental residual axial capacity of specimen <i>S4(0.19)</i>	150
Figure 5.1 Numerical and experimental residual deflected profile of <i>S1(0.58)</i>	155
Figure 5.2 Numerical and experimental residual axial capacity of <i>S1(0.58)</i>	156
Figure 5.3 Comparison of residual axial capacities of <i>S1(0.58)</i>	157
Figure 5.4 Numerical and experimental residual deflected profile of <i>S2(0.19)</i>	160
Figure 5.5 Numerical and experimental residual axial capacity of <i>S2(0.19)</i>	161
Figure 5.6 Comparison of residual axial capacities of <i>S2(0.19)</i>	162
Figure 5.7 Numerical and experimental residual deflected profile of <i>S3(0.58)</i>	165
Figure 5.8 Numerical and experimental residual axial capacity of <i>S3(0.58)</i>	166
Figure 5.9 Comparison of residual axial capacities of <i>S3(0.58)</i>	167
Figure 5.10 Numerical and experimental residual deflected profile of <i>S4(0.19)</i>	170
Figure 5.11 Numerical and experimental residual axial capacity of <i>S4(0.19)</i>	171
Figure 5.12 Comparison of residual axial capacities of <i>S4(0.19)</i>	172
Figure 5.13 Summary of deflected profiles	174
Figure 5.14 Bar chart of residual axial capacities.....	176
Figure 6.1 Varied transverse reinforcement detailing	181
Figure 6.2 Non-uniform blast loading	181

LIST OF TABLES

Table 2.1 Peak reflected overpressures (MPa) with corresponding W-R combinations	13
Table 3.1 Summary of $S1(0.58)$ numerical residual x-displacements	86
Table 3.2 Summary of $S2(0.19)$ numerical residual x-displacements	93
Table 3.3 Summary of $S3(0.58)$ numerical residual x-displacements	100
Table 3.4 Summary of $S4(0.19)$ numerical residual x-displacements	107
Table 3.5 Summary of numerical results	112
Table 5.1 Numerical and experimental residual lateral displacements of $S1(0.58)$	154
Table 5.2 Numerical and experimental residual lateral displacements of $S2(0.19)$	159
Table 5.3 Numerical and experimental residual lateral displacements of $S3(0.58)$	164
Table 5.4 Numerical and experimental residual lateral displacements of $S4(0.19)$	169
Table 5.5 Summary of numerical and experimental residual axial capacities.....	175

LIST OF SYMBOLS

A_g	Gross area of the section
f'_c	Characteristic cylinder compressive strengths of the concrete
i_s	Specific impulse
θ	Decay coefficient
P_{s0}	Peak incident pressure
R	Standoff distance
t_a	Arrival time
t_0	Positive phase duration
D_{ijkl}^s	Secant material stiffness
D_{ijkl}^t	Tangent material stiffness
I_1	First invariant of the deviatoric stress tensor
J_2	Second invariant of the deviatoric stress tensor
J_3	Third invariant of the deviatoric stress tensor
$\dot{\epsilon}^p$	Plastic strain rate
E_p	Plastic hardening modulus
σ	Cauchy stress tensor
C	Rank 4 stiffness tensor of the material
ϕ	Shape function
δ	Displacement response
σ_m	Pure hydrostatic stress tensor
p	Pressure
γ	Ratio of specific heats
λ	Modified plastic strain measure
γ_f	Strain rate enhancement factor
f_t	Tensile strength of concrete
b_1, b_2, b_3	Damage Evolution parameters

List of Symbols

$\dot{\epsilon}$	Strain rate
$\dot{\epsilon}_s$	Static strain rate
f_c	Compressive strength of concrete
f_s	Reinforcement static yield strength
M	Mass matrix
K	Stiffness matrix
Q	Vector of the externally applied load
$\ddot{u}$	Acceleration
$\dot{u}$	Velocity
u	Displacement
c_w	Speed at which stress waves travel in the element
ρ_v	Volumetric ratio of transverse reinforcement
ρ_g	Longitudinal reinforcement ratio
y_r	Residual mid-height displacement
L	Clear height of column
v	Ratio of residual axial capacity
P_L	Long-term axial load
P_{max}	Axial capacity of the undamaged column
θ_b	Rotation due to flexure within sections a and b
θ_{fe}	Fixed-end rotation
h_t	Distance between the two transducers
L_s	Original length of the diagonals in the region for column shear
γ_s	Average shear distortion of the region
W	Explosive Charge Weight

1

INTRODUCTION

1.1 Problem Definition

The terrorist attacks in the past decades have riveted our attention to the deficiencies in current structural design practices, regardless of the fact that most of the affected structures and those surrounding them, actually performed well given the extreme loads that were imposed upon them. These attacks served as a call to action to re-evaluate our design practices to accommodate for these severe loads. Incorporating such loadings has not been the standard practice in the design procedures of commercial structures in the past. The complexity of these loads and response mechanisms of individual components has called for the development of a design practice in this field that involves a mix of empirical, analytical and, recently, sophisticated numerical methods.

The bombing of the World Trade Centre in New York City in February 1993, the devastating attack against the Alfred P. Murrah Federal Building in Oklahoma City in April 1995 and the more recent collapse of both World Trade Centre Towers have underscored the attractiveness and vulnerability of civilian buildings as possible terrorist targets. These attacks have also demonstrated that modern terrorism should be considered with the utmost level of importance. Any nation can no longer consider themselves immune to terrorist violence within their own borders. An alarming fact is that a majority of government and civilian buildings remain to be vulnerable targets to terrorist attacks structurally.

Columns are key load-bearing elements within frame structures. Failure of an individual column element could possibly trigger progressive collapse of the entire frame structure. Detonation of explosive devices results in the generation of highly pressurized hot gasses, which expand violently displacing the surrounding air from the volume it previously occupied and can cause the column element to lose its capacity as it accumulates damage.

The post-blast residual axial capacity of a column is therefore narrowed down to be an important aspect of its behaviour to focus upon. It would be extremely important to understand the damage level sustained by a column element and its residual axial capacity available within it, to be able determine if it could possibly trigger a progressive collapse of the entire structure. To date, the research works carried out in this area has been rather limited and no systematic or conclusive findings have been realized from them.

This creates the motivation to understand better the behaviour of columns when subjected to blast loads. This research intends to realize the residual axial capacity of columns subjected to blast loads with two styles of reinforcement detailing while sustaining two levels of pre-axial loads. The author intends to determine how the style of reinforcement detailing within a column matched with two different levels of pre-axial loads affects its residual axial capacity when subjected to such an unforeseen scenario.

1.2 Objectives and Scope

The principle objective of this research is to study structural dynamic responses of reinforced concrete columns when subjected to short standoff blast conditions and subsequently determine their residual axial capacities. The study also extends to determine how increased transverse reinforcements and varied pre-axial loads affect the behaviour, blast resistance, and residual axial capacities of reinforced concrete columns. The study is conducted in two main phases. Namely, they are the numerical analysis and the experimental study phases. The data obtained from these two phases are then utilized to determine the validity of an equation proposed by a previously conducted study to predict blast-damaged residual axial capacity of reinforced concrete columns.

1.2.1 Numerical Analysis

The numerical analysis phase involves creating of finite element column models with varying transverse reinforcement ratios, pre-axially loading them at two levels and then

subjecting them to blast loadings to determine and understand their respective responses with the varying parameters. The numerical simulation is carried out to aid in determining the dynamic response of reinforced concrete columns under blast conditions. The model is the medium used to predict the overall behaviour and failure mechanism of reinforced concrete columns under short standoff blast loadings. The effect of providing increased transverse reinforcement and varying pre-axial loads can also be determined during this phase. The results obtained from this simulation were then used to orchestrate a laboratory-based experiment to confirm the accuracy of residual axial capacity prediction capability of the numerical simulation models and vice versa.

1.2.2 Experimental Study

The experimental study involves subjecting actual life-size column specimens to lateral loads via hydraulically powered actuators in a laboratory. The primary objectives of this phase in the research are to determine the prediction accuracy of both the numerical model and the equation that was proposed from a previous study to predict the post blast-damaged residual axial capacity of columns. The specimens are initially loaded with two levels of pre-axial loads and subsequently loaded with horizontal loads, as it would experience when subjected to a blast wave impact. The blast loadings a column would experience are determined from the numerical analysis phase. Once they have achieved a deflected shape similar to that determined from the numerical study, they would then be subjected to axial loads until failure. The experimental study will help determine the actual residual axial capacity of blast-damaged columns that have been deformed up to a drift level coinciding with the results obtained from the numerical study. Four column specimens with two varying transverse reinforcement ratios were fabricated for this purpose. Each of the two sets of specimens was subjected to two levels of pre-axial loads. The results would provide useful data on the enhancement that the increased transverse reinforcement ratio provides and the effect that pre-axial loads contribute to residual axial capacity of blast-damaged columns. The data obtained would also aid in affirming the findings determined from the numerical model and the equation proposed from studies that were previously conducted.

1.3 Organization of Report

Chapter 1 introduces the background, objectives, and scope of the works of this research. It also lists out the various chapters within this thesis and presents the material covered within its various parts.

Chapter 2 proceeds to describe the literature review carried out by the author prior to carrying out this research study. It includes the basic characteristics of the blast wave phenomenon, the interaction of blast waves with target structures, the effect of air blast loads, and the dynamic response of reinforced concrete columns. There are also brief outlines of some of the various researches carried out within this field. Also highlighted within is the research study carried out previously that proposed an equation capable of predicting post blast-damaged residual axial capacity of reinforced concrete columns.

Chapter 3 deals with the numerical simulations carried out during the course of this research study. The material behaviour of concrete and reinforcing steel and their constitutive models are presented within. The numerical modeling phase includes blast load calculations, structural geometry modeling, material modeling, boundary conditions, analysis steps and integration method selection. Also reported within this chapter are the numerical results obtained from the reinforced concrete column models subjected to short distance blast conditions. The results include their dynamic response, stress and deflection levels encountered by the reinforced concrete column models and the blast-damaged residual axial capacity of each of the column models.

Chapter 4 presents the experimental study that was conducted to validate the numerical results obtained from the previous chapter. It describes individual specimens and their reinforcement details. The entire setup of the experiment including descriptions of the loading frame, actual test set-up procedures, loading sequences and also the various instrumentations and transducers used to record displacements during the laboratory experiment are discussed in detail. The capacities of the hydraulic actuators that were used to simulate all the loadings on the column specimens are also reported within this chapter.

Chapter 5 provides a summary of the results obtained from both the numerical and experimental phases conducted during this study. Charts and figures are compiled within this chapter to describe the behaviour and capacities of each of the column models and specimens. Comparisons are made between the crucial figures that were obtained. Also reported within, is the accuracy of the prediction of an equation proposed from a previously conducted study to determine residual axial capacity of blast-damaged reinforced concrete columns. All these results and comparisons are tabulated and plot into charts or graphs within this chapter, to provide better visualization of the results obtained from each of the phases carried out during this research study.

In Chapter 6, conclusions and recommendations drawn from the entire research study are listed out.

2

LITERATURE REVIEW

2.1 Introduction

Live loads utilized during the design of traditional structural systems have been developed through careful investigation of repetitive events. They involve loads occurring at regular intervals in time, matched with moderate to severe intensity and are predictable using statistical approaches. Blasts or intentional impact events with extreme severity in contrast can be classified as loads that although have a relatively lower frequency of occurrence but have an extremely devastating consequence.

It is only recent years that has brought standard building design practices to form a closer relationship by considering the effects of and for some cases even incorporating for blast loadings. Designers may now be mandatorily required for some structures to consider loads previously thought of as rare and highly improbable. They come mainly in the form of loads caused by intentional attack of buildings by criminals and terrorists with the motive of using the failure of the structure as a means to achieve their ends.

Previous research was conducted primarily for protecting military structures or government buildings. Existing blast design approaches call for modest enhancements in structural design coupled with a buffer zone surrounding the building. This highly effective approach is only feasible where a keep-out zone is available and economically viable. For many urban buildings, this keep-out zone may not be available, and may leave them in a vulnerable position to intended terrorist attacks.

Explosive loads and impact loads are transients, or loads that are applied dynamically as one-half cycle of a high amplitude, short duration air-blast or contact and energy transfer related pulse. This transient load is applied only for a specific and typically short period. In

the case of blast loads, this time is typically less than one-tenth of a second. This means that an additional set of dynamic structural properties such as rate dependent material properties and inertial effects must be incorporated into the design. For most civilian structures, design to resist blast, impact and other extraordinary loads considered for primarily in the context of life safety and not in terms of the serviceability of the structure.

An explosive blast is unlike other types of severe loads caused by extreme events such as earthquakes or high wind. These types of loads generate damage that is limited to very few structural response mechanisms. Earthquake and wind loads are applied globally, not locally, which means that the entire structural system works together to resist these loads. An explosive blast activates many structural response mechanisms within a structure because of its extreme spatial and time variations in magnitude and time of application. This creates the complexity in the design process.

Failure of a single structural column may have a devastating effect on the overall structural integrity of a building. For tall buildings, the structural columns can carry substantial axial load due to gravity, and therefore it is prudent to include the effect of axial load in blast analysis. The axial load in reinforced concrete columns increases the bending capacity of the column, due to the large imbalance in concrete tensile and compressive strengths. Axial load reduces stiffness and strength in tall slender columns due to the buckling phenomenon, which may cause catastrophic failure of the column.

This chapter reviews the significant aspects of previous research studies on the response of reinforced concrete structures when subjected to blast loadings. Firstly, a brief overview of blast loadings is introduced. This is followed by a description of the structural response to blast loadings. Experimental investigations carried out by various researchers on concrete columns subjected to blast loadings are also reviewed.

2.2 Blast Basics

An explosion is defined as a large-scale, rapid and sudden release of energy. It is characterized by an audible blast and a bright flash [B1]. Explosions can be categorized based on their nature as physical, nuclear or chemical events. The energy from a blast is released in two parts. One part of the energy is released as thermal radiation and the remaining part is coupled into the air-blast and to the soil as ground-shocks as radially expanding shock waves. Air-blast is the principle damage mechanism. The Air-blast phenomenon occurs within milliseconds and the local effects of a blast are often over before the structure can globally react to the effects of the blast.

2.2.1 Explosion, Detonation and Deflagration

Blast loads are most often thought of as emanating from exothermic reactions resulting in detonation. However, shock waves in air can result from pressure vessel ruptures and high flame front velocity combustions as well. These are typical of unconfined vapour cloud explosions. When the source material can sustain a supersonic wave or flame front of sufficient velocity to create a local high pressure within the source material, the reaction is considered as a detonation. In this case, all the potential energy is released in the chemical reaction. Flame front velocities in combustion below this velocity, will result in a reaction or explosion short of a detonation and is termed as a deflagration. In this case, only a portion of the stored potential energy is released in the chemical reaction.

Often, the class of explosion, be it deflagration or detonation, is determined by the initial energy available. This initial energy is usually delivered as a strong shock provided by impact, detonation of a primary explosive, or friction.

2.2.2 Blast Phenomenon

The detonation of a condensed high explosive generates hot gasses under pressures of up to 30000 MPa and temperatures of about 3000 – 4000 °C. The hot gas expands forcing out the

volume it occupies. As a consequence, a layer of compressed air (blast wave) forms in front of this gas volume containing most of the energy released by the explosion. The shock front is similar to a moving wall of highly compressed air. The blast wave instantaneously increases to a value of pressure above the ambient atmospheric pressure. This is referred to as the side-on overpressure. The peak blast pressure decays as the shock wave expands outwards from the explosion source. The decay of the pressure over distance is illustrated in **Figure 2.1** [B1].

Figure 2.1 Blast wave propagation

After a short time, the pressure behind the front may drop below ambient pressure. During such a negative phase, a partial vacuum is created and air is sucked in. This is also accompanied by high suction winds that carry the debris for long distances from the explosion source. A typical blast pressure-time profile is shown in **Figure 2.2** [B1].

Figure 2.2 Blast wave pressure-time history

Two main phases can be observed from the pressure-time profile. The portion above ambient is called positive phase of duration and that below ambient is called negative phase of duration. The negative phase is of a longer duration and a low intensity compared with positive phase. Thus, its blast wave parameters are always ignored. The overpressure decay is governed by the decay coefficient, θ , in msec. The exponential form of the overpressure-time history has been suggested in TM5-855-1 [T1] as follows:

$$P_s(t) = P_{s0} \left[1 - \frac{(t - t_a)}{t_d} \right] \exp \left[- \left(\frac{t - t_a}{\theta} \right) \right] \quad (2.1)$$

where $P_s(t)$ = incident pressure at time t , P_{s0} = peak incident pressure, t_0 = positive phase duration, t_d = arrival time, θ = decay coefficient.

The peak overpressures P_{s0} are amplified by a reflection factor as the shock wave encounters an object or structure in its path. Except for specific focusing of high intensity shock waves at near 45° incidence, these reflection factors are typically greatest for normal incidence (a surface adjacent and perpendicular to the source) and diminish with the angle

of obliquity or angular position relative to the source. Reflection factors depend on the intensity of the shock wave, and for large explosives at normal incidence, these reflection factors may enhance the incident pressures by as much as an order of magnitude.

The positive specific impulse per unit area can be obtained by the time integration of the positive phase stated in Equation 2.1:

$$i_s = \int_{t_a}^{t_a+t_0} P_s(t) dt \quad (2.2)$$

Some controlled explosions have been conducted under ideal conditions as reference explosions. To relate other explosions under non-ideal conditions to the reference explosions, blast-scaling laws can be employed. The most widely used form of blast scaling is the cube root scaling law formulated by Hopkinson [H1]. The scaling law states that when two charges of the same explosive and geometry, but of different sizes are detonated in the same medium, the shock waves produced are similar in nature at the same scaled distance. The standoff, Z can be expressed as:

$$Z = \left(\frac{R}{W} \right)^{1/3} \quad (2.3)$$

where, R = standoff distance, W = explosive charge weight

Reference data on blast effects for TNT are available in current manuals [T1, M1]. These data can be extended for the use of other detonation materials, by relating the explosive energy of the equivalent charge weight of those materials to that of an equivalent weight of TNT. The relationship can be given as [T1, B2, M1]:

$$W_{TNT} = \frac{H_{exp}}{H_{TNT}} \times W_{exp} \quad (2.4)$$

where, W_{TNT} is the equivalent TNT charge weight, W_{exp} is the weight of the explosive of interest; H_{TNT} is the heat of detonation of TNT, and H_{exp} is the heat of detonation of the explosive of interest.

2.2.3 Prediction of Blast Pressure

Blast wave parameters for conventional high explosive materials have been the focus of a number of studies. The peak overpressure in KPa is introduced by Mills (1987) [M9], in which, W is expressed as equivalent charge weight in kilograms of TNT, and, Z is the scaled distance:

$$P_{s0} = \frac{1772}{Z^3} - \frac{114}{Z^2} + \frac{108}{Z} \quad (2.5)$$

As the blast wave propagates through the atmosphere, the air behind the shock front is moving outward at a lower velocity. The velocity of the air particles, and hence the wind pressure, depends on the peak overpressure of the blast wave. This later velocity of air is associated with the dynamic pressure, $q(t)$. The maximum value, q_s , say, is given by:

$$q_s = \frac{5P_{s0}^2}{2(P_{s0} + 7P_0)} \quad (2.6)$$

If the blast wave encounters an obstacle perpendicular to the direction of the propagation, reflection increases the overpressure to a maximum reflected pressure, P_r as:

$$P_r = 2P_{s0} \left(\frac{7P_0 + 4P_{s0}}{7P_0 + P_{s0}} \right) \quad (2.7)$$

TM 5-1300 (1990) gives a full discussion and extensive charts for predicting blast pressures and blast durations [T6]. Some numerical values of peak reflected overpressure are given in **Table 2.1**.

Table 2.1 Peak reflected overpressures (MPa) with corresponding W-R combinations

$\begin{matrix} \text{R} \\ \text{W} \end{matrix}$	100 kg TNT	500 kg TNT	1000 kg TNT	2000 kg TNT
1 m	165.80	354.50	464.50	602.90
2.5 m	34.20	89.40	130.80	188.40
5 m	6.65	24.80	39.50	60.19
10 m	0.85	4.25	8.15	14.70
15 m	0.27	1.25	2.53	5.01
20 m	0.14	0.54	1.06	2.13
25 m	0.09	0.29	0.55	1.08
30 m	0.06	0.19	0.33	0.63

For design purposes, reflected overpressure can be idealized by an equivalent triangular pulse of maximum peak pressure, P_r and time duration, t_d , which yields the reflected impulse, i_r :

$$i_r = \frac{1}{2} P_r t_d \quad (2.8)$$

Duration, t_d is related directly to the time taken for the overpressure to be dissipated. Overpressure arising from wave reflection dissipates as the perturbation propagates to the edges of the obstacle at a velocity related to the speed of sound, U_s , in the compressed and heated air behind the wave front. Denoting the maximum distance from an edge as S , the additional pressure due to reflection is considered to reduce from $P_r - P_{s0}$ to zero in the time $3S/U_s$. Conservatively, U_s can be taken as the speed of sound (340 m/s), and the additional impulse to the structure evaluated on the assumption of a linear decay.

2.2.4 Blast Loading

Blast loadings on structures depend on several factors [T1]. They are listed out as follows:

- Type of explosive materials
- Weight of explosive
- Relative location of explosion centre to the structure
- Interaction of the shock front with the structure

Generally, blast conditions include nuclear and conventional detonations [B2, B3]. However, the focus of this study is restricted to conventional explosions. They include bombs placed within vehicles parked outside of buildings. There are two possible scenarios when an explosion occurs outside of a building. One possibility is when the detonation occurs at a large scaled standoff distance and the structure is small as compared with the shock front. In this case, the structure will be loaded as a whole and the entire structure would be required to provide some degree of resistance to the blast loadings. The other possibility is when the detonation occurs at a small-scaled standoff distance. In this case, the blast loadings on the surface of the structure would be uneven and some portions of the structure would be loaded more heavily than others. Some elements would then be subjected to severe initial damage and this may lead to progressive collapse of the entire structure at a later stage.

For any given set of free-field incident pulses, the forces imparted to an aboveground structure can be divided into three general components [T1]. They are as listed below:

- Incident pressure
- Dynamic pressure
- Reflected pressure

Both incident and reflected wave properties can be predicted using graphical tools originally developed by the U.S. Army and as described in the previous section [T1]. Algorithms have been developed to generate the parameters. These polynomial fit-based equations, as well as equations that describe and account for angle of incidence and reflection coefficients, are incorporated into software programs such as ATBlast, BLASTX and CONWEP.

2.3 Structure Dynamics

The ductility and natural period of vibration of a structure governs its response to an explosion. Ductile materials, such as steel and reinforced concrete, can absorb significant amount of strain energy, in contrast to brittle materials, such as timber, masonry and monolithic glass where abrupt failure is inevitable. In the investigation of the dynamic response of structures, there are a few prescribed steps to be followed [E2]. The characteristics of the blast wave and natural period of response of the structure must initially be determined. The positive phase of the blast wave is then compared with the natural period of response of the structure. From this comparison, the response of the structure can be defined as follows:

If the positive phase duration of the blast pressure is,

- shorter than the natural period of vibration of the structure, the response is described as *impulsive*.
- longer than the natural period of vibration of the structure, the response is defined as *quasi-static*.
- close to the natural period of response of the structure, the response of the structure is referred to as *dynamic*.

For an *impulsive* response case, most of the deformation of the structure will occur after the blast loading has diminished. As for the *quasi-static* response case, the blast will cause the structure to deform whilst the loading is still being applied. *Dynamic* response on the other

hand, would result in a deformation of the structure in a function of time. This response is determined by solving the equation of motion of the structural system.

In general, a tall building will have a low frequency and thus a long period of vibration in relation to the duration of the load. Individual elements will have response times that may approach the load duration. Window panels of $1.0 - 1.5 \text{ m}^2$ for instance, have a frequency greater than 10 Hz (or less than 0.1 s response time). Floor slabs have a frequency range of 10 – 30 Hz (or 0.03 – 0.1 s response time). Rigid elements that are unable to respond dynamically within the time period of loading will under some circumstances continue to attract load until they fail in-elastically, and abruptly. Flexible elements, in contrast, will attenuate the load by responding to the blast pressures, enabling the strain energy to be absorbed through deformation. Unlike steelwork, the rebound in concrete is small, as cracking in concrete will result in internal damping. The massive nature of reinforced concrete structures performs well in resisting the impulsive load usually encountered close to the blast point.

2.4 Structural Response to Blast Loading

The complexity in analyzing the dynamic response of blast-loaded structures arises from the effect of high strain rates, non-linear inelastic material behaviours, uncertainties of blast load calculations and time dependent deformations. Therefore, to simplify the analysis, a number of assumptions related to the response of structures and the loads have been proposed and widely accepted. To establish the principles of this analysis, the structure is idealized as a single degree of freedom (SDOF) system and the link between the positive duration of the blast load and the natural period of vibration of the structure is established. This leads to blast load idealization and simplifies the classification of the blast loading regimes.

2.4.1 Elastic SDOF Systems

The simplest discretization of transient problems is by means of the SDOF system approach. The actual structure can be replaced by an equivalent system of one concentrated mass and one weightless spring representing the resistance of the structure against deformation. Such an idealized system is illustrated in **Figure 2.3** [B3].

Figure 2.3 SDOF system [B3]

The structural mass, M , is under the effect of an external force, $F(t)$, and the structural resistance, R , is expressed in terms of the vertical displacement, y , and the spring constant, K . The blast load can also be idealized as a triangular pulse having a peak force, F_m , and a positive phase, t_d . This is illustrated in **Figure 2.4** [B3].

Figure 2.4 Idealized blast loading [B3]

The forcing function is then given as:

$$F(t) = F_m \left(1 - \frac{t}{t_d} \right) \quad (2.9)$$

The blast impulse is approximated as the area under the force-time curve, and is given by:

$$I = \frac{1}{2} F_m t_d \quad (2.10)$$

The equation of motion of the undamped elastic SDOF system for a time ranging from 0 to the positive phase duration, t_d , is given as [B3]:

$$M \ddot{y} + Ky = F_m \left(1 - \frac{t}{t_d} \right) \quad (2.11)$$

The general solution for the displacement can be expressed as:

$$y(t) = \frac{F_m}{K}(1 - \cos \omega t) + \frac{F_m}{K t_d} \left(\frac{\sin \omega t}{\omega} - t \right) \quad (2.12)$$

and the general solution for the velocity is expressed as:

$$\dot{y}(t) = \frac{dy}{dt} = \frac{F_m}{K} \left[\omega \sin \omega t + \frac{1}{t_d} (\cos \omega t - 1) \right] \quad (2.12)$$

in which ω is the natural circular frequency of vibration of the structure and T is the natural period of vibration of the structure which is given by:

$$\omega = \frac{2\pi}{T} = \sqrt{\frac{K}{M}} \quad (2.13)$$

The maximum response is defined by the maximum dynamic deflection y_m that occurs at time t_m . The maximum dynamic deflection y_m can be evaluated by setting dy/dt in **Equation 2.12** equal to zero. This represents when the structural velocity to be zero. The dynamic load factor, DLF , is defined as the ratio of the maximum dynamic deflection y_m to the static deflection y_{st} that would have resulted from the static application of the peak load F_m . This is shown as follows:

$$DLF = \frac{y_{\max}}{y_{st}} = \frac{y_{\max}}{F_m / K} = \phi(\omega t_d) = \psi\left(\frac{t_d}{T}\right) \quad (2.14)$$

ωt_d is used to classify the structural response to blast loading into the three regimes mentioned in **Section 2.3**.

2.4.2 Elasto-Plastic SDOF Systems

Structural elements are expected to undergo large inelastic deformation under blast load or high velocity impact. Analysis of dynamic response is then only possible by systematic numerical solution requiring a nonlinear dynamic finite-element software. However, the degree of uncertainty in both the determination of the loading and interpretation of acceptability of the resulting deformation is such that solution of a postulated equivalent ideal elasto-plastic SDOF system is commonly used [B3]. Interpretation is based on the required ductility factor as defined below and illustrated in **Figure 2.5**.

$$\mu = y_m / y_e \quad (2.15)$$

Figure 2.5 Simplified resistance function of an elasto-plastic SDOF system

The response of the ideal bilinear elasto-plastic system can be evaluated in closed form for the triangular load pulse comprising rapid rise and linear decay, with maximum value, F_m , and duration, t_d . The result for the maximum displacement is generally presented in a chart form as a family of curves for selected values of R_u/F_m showing the required ductility, μ , as a function of, t_d/T , in which, R_u , is the structural resistance of an element and T is its natural period [T6]. The chart is shown in **Figure 2.6**.

Figure 2.6 Maximum response of elasto-plastic SDOF system to triangular load [T6]

2.5 Material Behaviours at High Strain Rate

Blast loads typically produce very high strain rates ($10^2 - 10^4 \text{ s}^{-1}$). This high strain loading rate would alter the dynamic mechanical properties of target structures and, accordingly, the expected damage mechanisms for various structural elements. For reinforced concrete structures subjected to blast effects the strength of concrete and steel reinforcing bars can increase significantly due to strain rate effects. **Figure 2.7** [N7] shows the approximate ranges of the expected strain rates for different loading conditions.

Figure 2.7 Strain rates associated with different types of loading [N7]

2.5.1 Dynamic Properties of Concrete under High Strain Rates

The mechanical properties of concrete under dynamic loading conditions can be quite different from those under static loading. While the dynamic stiffness does not vary a great deal from the static stiffness, the stresses that are sustained for the period of time of dynamic loading may gain values that are remarkably higher than the static compressive strength. This is illustrated in **Figure 2.8** [N7]. Strength magnification factors as high as 4 in compression and up to 6 in tension for strain rates in the range of $10^2 - 10^3 \text{ s}^{-1}$ have been reported by Grote et al. [G6].

Figure 2.8 Stress-strain rates of concrete at different strain rates [N7]

To accommodate the increase in peak compressive stress, f'_c , a dynamic increase factor, *DIF*, is introduced in the CEB-FIP [C9]. This incorporates the strain rate enhancement of concrete. For compressive strength of concrete *DIF* is given as:

$$DIF = \left(\frac{\dot{\varepsilon}}{\dot{\varepsilon}_s} \right)^{1.026\alpha} \quad \text{for } \dot{\varepsilon} \leq 30s^{-1} \quad (2.16)$$

$$DIF = \gamma \left(\frac{\dot{\varepsilon}}{\dot{\varepsilon}_s} \right)^{1/3} \quad \text{for } \dot{\varepsilon} > 30s^{-1} \quad (2.17)$$

where, $\dot{\varepsilon}$, is the dynamic strain rate; the quasi-static strain rate, $\dot{\varepsilon}_s$, is taken as $30 \times 10^{-6} s^{-1}$; $\log \gamma$, is taken as $6.156\alpha - 2$, $\alpha = 1/(5 + 9f_c / f_{co})$ and f_{co} is taken as 10 MPa. The dynamic increase factor for various strain rates of concrete is graphically represented in **Figure 2.9** [C9].

Figure 2.9 Dynamic increase factor for peak stress of concrete [C1]

For concrete in tension, *DIF* is defined as:

$$DIF = \begin{cases} (\dot{\varepsilon}/\dot{\varepsilon}_s)^\delta & \dot{\varepsilon} \leq 1.0s^{-1} \\ \beta(\dot{\varepsilon}/\dot{\varepsilon}_s)^{1/3} & \dot{\varepsilon} > 1.0s^{-1} \end{cases} \quad (2.18)$$

where, ε is the strain rate in the range of 10^{-6} s^{-1} to 160 s^{-1} ; $\dot{\varepsilon}_s = 10^{-6} \text{ s}^{-1}$ (static strain rate); $\log \beta = 6\delta - 2$; $\delta = 1/(1 + 8f_c/f_{co})$; $f_{co} = 10 \text{ MPa}$; f_c is the static compressive strength of concrete.

2.5.2 Dynamic Properties of Reinforcing Steel under High Strain Rates

Due to the isotropic properties of metallic materials, their elastic and inelastic response to dynamic loading can easily be monitored and assessed. Norris et al. [N8] tested steel with two different static yield strengths of 330 and 278 MPa under tension at strain rates from 10^{-5} to 0.1 s^{-1} . Strength increase of 9 – 21% and 10 – 23 % were observed for two types of steel respectively. Dowling and Harding [D3] conducted tensile experiments using the tensile version of Split Hopkinson's Pressure Bar (SHPB) on mild steel using strain rates varying between 10^{-3} s^{-1} and 2000 s^{-1} . It was concluded from this test series that materials of body-centered cubic (BCC) structure (such as mild steel) showed the greatest strain rate sensitivity. It has been found that the lower yield strength of mild steel can almost be doubled; the ultimate tensile strength can be increased by about 50%; and the upper yield strength can be considerably higher. In contrast, the ultimate tensile strain decreases with increasing strain rate.

Malvar [M10] also studied strength enhancement of steel reinforcing bars under the effect of high strain rates. This was described in terms of the dynamic increase factor, DIF , which can be evaluated for different steel grades and yield stresses, f_y , ranging from 290 to 710 MPa as represented by the following equation:

$$DIF = \left(\frac{\dot{\varepsilon}}{10^{-4}} \right)^{\alpha} \quad (2.19)$$

where for calculating yield stress $\alpha = \alpha_{f_y}$ and, α_{f_y} is defined as:

$$\alpha_{fy} = 0.074 - 0.04 \left(\frac{f_y}{414} \right) \quad (2.20)$$

and for ultimate stress calculation $\alpha = \alpha_{fu}$ and is defined by:

$$\alpha_{fu} = 0.019 - 0.009 \left(\frac{f_y}{414} \right) \quad (2.21)$$

2.6 Research Works on Blast Effects on Reinforced Concrete Columns

There is limited literature available documenting the experimental responses of reinforced concrete columns. The following subsections describe some of the efforts made by various research institutions to understand the effects of columns when subjected to blast loading. They include field and laboratory test programs.

2.6.1 Crawford *et al.*

A test series was conducted by Technical Support Working Group (TSWG), USA and the Defense Threat Reduction Agency (DTRA), USA a series of tests were conducted to quantify the effects of terrorist explosions on conventional buildings. Crawford *et al.* [C7] documented the results from these tests in a paper. The full-scale test article, Component Test Structure One (CTS-1), shown in **Figure 2.10** [C7], represents a conventional East Coast reinforced concrete four-story office building located in Seismic Zone 1.

Figure 2.10 CTS-1 prior to being tested [C7]

The column element in the structure is weak in its lateral load capacity and has low ductility. The explosion is set off at a short standoff distance. Two views of the post-blast illustrate the damage of the columns are shown in **Figure 2.11** [C7]. The loosely spaced transverse reinforcement within the column caused the column to fail due to both its shear and flexure capacity being exceeded at both ends.

Figure 2.11Two views of column DB6 [C7]

An additional experiment was conducted by the research team to determine the increase in blast loading capacity the column would attain when reinforced with a layer of CFRP. The comparison between the columns is shown in **Figure 2.12**. From the results, the retrofitted column appears to remain elastic and no permanent deformation was apparent. In contrast, column DB6 had a residual deflection at the mid-height location of about 250 mm. The improvement provided by the retrofit is very evident.

The results indicate that the ductility capacity of a column depends on the amount and distribution of transverse reinforcement available within the plastic hinge region. Hoop reinforcement increases the confinement of the core concrete, thus increasing its ultimate resistance. It also provides lateral restraint against buckling of the longitudinal reinforcement bars. This experiment shows that the transverse reinforcement has a large influence on the response of reinforced concrete columns when subjected to blast loading.

Figure 2.12 Influence of CFRP wrap on columns subjected to blast loading

Karagozian & Case (K&C) developed a column test fixture for conducting field tests [C7]. The setup is illustrated in **Figure 2.13** [C7]. This fixture provides an axial loading mechanism using an axial load beam.

Figure 2.13 Field test setup developed by K&C [C7]

A series of test programs were conducted in this test setup. They involved TNT equivalent weight varying from 500 to 1000 kg and were placed at a standoff distances of 3 to 6 m. The results from these tests are illustrated in **Figure 2.14** [C7]. It can be observed from the results, that the column core concrete was split into small segments by diagonal shear cracks.

Figure 2.14 Column response from K&C field tests [C7]

K&C also proceeded to carry out a test within the laboratory. The aim of this experiment was to obtain detailed response data regarding the behaviour of reinforced concrete columns under carefully controlled conditions. The laboratory setup is illustrated in **Figure 2.15** [C7] and **Figure 2.16** [I1].

Figure 2.15 Schematic of actuator system for laboratory experiments [C7]

Figure 2.16 Photograph of 5-actuator system laboratory test [I1]

The laboratory test setup consists of a reaction frame, five hydraulically powered actuators and a “pantograph” strut system. The lateral load was applied to the column via three horizontally placed on the reaction frame. The horizontal actuators were mounted on the reaction frame. The blast loads were simulated through the horizontally mounted actuators through a displacement-controlled mode. The displacement controlled mode allowed the researchers to track the column resistance up to failure as well as the softening portion. The other two actuators were placed vertically and had the role in applying an axial preload to simulate the service loads an actual column would experience. The “pantograph” strut system had a role to provide the column with rotational restraint and at the same time ensuring that no axial restraint was provided. The design of the footing and top load stub

represented an attempt to provide a fixed-end condition upon application of the initial column axial load.

The test specimens were first loaded axially to represent service loads. Consequently, they were subjected to a quasi-static lateral load from the three horizontally mounted actuators. **Figure 2.17** [C7] illustrates the comparison between the response between the field and laboratory experiment.

Figure 2.17 Comparison between column DB6 (field test) and the laboratory specimen

2.6.2 Krauthammer *et al.*

Collaboration between the protective technology centre of the Pennsylvania state University and the US army engineer research and development centre [D4] conducted a dynamic structure collapse experiment on a 1:4 scale-building model [K8]. **Figure 2.18** shows the post-test view of the experiment.

Figure 2.18 Post-test view of field experiment [K8]

A numerical study was also conducted for a similar specimen by the research team. The finite element analyses using a Lagrangian large deformation code with an explicit-dynamic finite element computer code predicted the response. The numerical results are a good match with the experimental results and are illustrated in **Figure 2.19** [K8].

Figure 2.19 Comparison of experimental and predicted column response [K8]

2.6.3 Hegemier *et al.*

The laboratory experiment setup described in the previous sections is normally used to conduct quasi-static tests. The University of California, San Diego, has developed a Explosive Loading Laboratory Testing Program. This was the first program, which utilized a hydraulic-based blast simulator that had the capability of simulating explosive events without the use of actual explosive material. The test setup is shown in **Figure 2.20** [H4].

Figure 2.20 Explosive Loading Laboratory Testing Program setup [H4]

The main components consists of an array of blast generators together with energy source and control system, an impact mass, a base slab provided with isolators to limit the energy and momentum transmitted to the foundation, a movable reaction wall and a system to provide axial load and end restraint. The blast simulator has performed several controlled blast load simulations on reinforced concrete columns. **Figure 2.21** [H4] shows the comparison made between a field test specimen and an identical column specimen that was subjected to an impulse of 13.8 KPa-sec.

Figure 2.21 Comparison between field and blast simulator experiments

From the comparison, it was observed that the very complex blast simulator is capable of producing effects very similar to those obtained from actual blasts as generated from field tests.

2.6.4 Hayes *et al.*

It is evident from the studies carried out in the previous sections that poorly configured and widely spaced transverse reinforcement, which is inadequate of providing sufficient confinement, would result in severe damage due to shear failure of reinforced concrete columns when subjected to both static and dynamic loads. This led to a suggestion by some engineers that current seismic design provisions could improve the resistance of reinforced concrete columns when subjected to blast loads and prevent progressive collapse of the structure. The U.S. Army Engineering Research and Development Centre (EDRC) to analyze the Alfred P. Murrah Federal Building, which was severely in a 1995 terrorist attack [H5], therefore conducted a study. The building was initially evaluated for seismic vulnerabilities as if it were located in a high-seismicity region. Strengthening schemes in the form of a pier-spandrel system, a special moment concrete frame and a set of internal shear walls were proposed. The strengthened structures were then analyzed for their response to the same explosion that occurred in 1995. The analysis results showed that the pier-spandrel and special moment frame schemes reduced the degree of direct blast induced damage and were capable of subsequently preventing progressive collapse of the building.

2.6.5 Residual Axial Capacity of Columns

Various researchers carried out experimental investigations [T4, T5, L3, and S3] to determine the residual axial capacity of columns when subjected to cyclic loadings. The following observations were noted from these studies.

- Sliding along the diagonal shear cracks was often observed prior to axial failure.
- Axial failure occurred when the shear capacity of the column was reduced close to zero.

- Increased axial loads over prolonged periods reduced the ratio of residual axial strength subsequently available in the columns.
- The drift at axial load failure was directly related to the amount of the transverse reinforcement.
- Drift ratios at axial failure tends to be higher for columns with a larger transverse reinforcement ratio.

These observations revealed that the important parameters that affect the deformation level of reinforced concrete columns at axial load failure include the axial load ratio and the amount of transverse reinforcement.

2.6.6 Bao and Li

In one of the most recent research studies carried out to determine the residual axial capacity of blast-damaged columns, Bao and Li [X1] carried out numerical simulations of the dynamic responses and residual axial strength of reinforced concrete columns subjected to short standoff blast conditions. Their study included an extensive parametric study on a series of twelve column models to investigate the effects of transverse and longitudinal reinforcement ratio, axial load ratio, and column aspect ratio. These parameters were then incorporated into an equation they proposed that was capable of predicting the residual axial capacity of blast damaged columns based on its mid-height damage level.

Some of the findings from that study concluded that the effect of axial load ratio is more critical for columns with lower transverse reinforcement ratio and that the ratio of residual axial capacity increases as longitudinal reinforcement ratio was increased. The comparison made between the results from the proposed equation and the analytical results showed that the equation was capable of predicting residual axial capacity of blast-damaged columns with reasonable accuracy.

3

NUMERICAL STUDY

3.1 Introduction

The study of the dynamic responses of structures under blast loading is commonly carried out utilizing either experimental studies or numerical simulations. Although the former provides real results, it is limited by constraints on cost and time. The research carried out in field experiments are also limited to particular cases under restricted structural dimensions, shapes, loading and boundary conditions.

Numerical methods are based on mathematical equations that describe the basic laws of physics governing a problem. These principles include conservation of mass, momentum, and energy. In addition, the physical behaviour of materials is described by constitutive relationships. These models are commonly termed computational fluid dynamics (CFD) models. The numerical methods used to simulate the blast effects problem typically are based upon a finite volume, finite difference, or finite element method with an explicit time integration scheme.

Computational methods in the area of blast effects mitigation are generally divided into two broad classifications. They are methods used for prediction of blast loads on the structure and calculations to determine structural response to these loads. Computational programs for blast prediction and structural response both use first-principle and semi-empirical methods.

A detailed discussion of finite element models is provided in the following sections. The model created to study the problem on hand and the results obtained from the analysis are presented within.

3.2 Blast Simulation Techniques

High explosive loading and response problems involve a highly nonlinear transient phenomenon. A great range of physical processes must be taken into account to enable accurate characterization of such events. Several hydrocodes using CFD techniques are capable of blast simulation. The nonlinear FEM software LS-DYNA [L4] built-in with CONWEP [H6] was utilized in this study. The specific features of this software are given in the following sub sections.

3.2.1 LS-DYNA

LS-DYNA is a fully integrated engineering analysis code specifically designed for nonlinear dynamic problems. It is particularly suited to the modeling of impact, penetration, blast, and explosion events. LS-DYNA is an explicit numerical analysis code, sometimes referred to as a “hydrocode” where the equations of mass, momentum and energy conservations coupled with material descriptions are solved. Finite difference, finite volume, finite element, and meshless methods can be used depending on the solution technique used.

3.2.2 CONWEP

Kingery and Bulmash [K12] have developed equations to predict air-blast parameters from spherical airbursts. These equations are widely accepted as engineering predictions for determining free-field pressures and loads on structures. The Kingery-Bulmash equations have been automated in the computer program CONWEP.

The Kingery-Bulmash equations are made from a compilation of data observed from explosive tests using charge weights from less than 1 kg to over 400,000 kg. The authors used curve-fitting techniques to represent the data with high-order polynomial equations that were later programmed within the CONWEP software. These equations can also be found in military blast design manuals like TM5-855-1 [T1] in its graphical form.

Unlike TM5-855-1, where an approximate equivalent triangular pulse is proposed to represent the decay of the incident and reflected pressure, CONWEP takes a more realistic approach. It assumes an exponential decay of the pressure with time.

3.3 Finite Element Models

The complete modeling involves blast load calculations, structural geometry modeling, relevant material models and corresponding parameters selection, boundary conditions, analysis steps and integration method. Each of these steps is discussed in detail in the following sub sections.

3.3.1 Blast Load Calculations

Blast loads impose a great intensity of force on a structure within a very short time frame. This creates an extremely dynamic action on the affected structure. As discussed in the previous chapter, the threat for a conventional bomb is defined by two equally important elements, the charge weight, which is measured using the equivalent amount of TNT in this study, and the standoff distance between the blast source and its target.

The weight of the explosive may vary from a relatively small to an extremely large quantity. One may draw the logical conclusion that a larger equivalent weight of TNT would result in higher peak blast pressures over a longer duration; thus causing more severe damage to structural elements. Blast incidents from recent years show that most terrorist attacks on public structures are explosions with short standoff distances of less than 10 m. As such, this study has adopted standoff distances of less than 10 m to understand its effect on a column element. In addition, considering the limitation of the weight of explosive that can be obtained in any particular region, a maximum weight the equivalent of 1 ton of TNT is set for this research.

Exterior reinforced concrete columns are the most important structural elements for holding a building upright and preventing it from collapsing. They are also the most vulnerable to airblast loading. **Figure 3.1** [M11] illustrates the succession of pressures on a building due to an external weapon.

Figure 3.1 Succession of blast pressure on a building [M11]

An explosive load on a column element generates four types of loads on the member. They are impact of primary fragments, impact of secondary fragments, overpressure, and reflected pressure. These blast loads on a first storey column are shown in **Figure 3.2** [N4]. Primary and secondary fragments can cause significant human casualties, but usually neither contributes to severe structural damage. As the overpressure wave is exposed to the front face of a closed target, a reflected pressure is instantly developed. The reflected pressure is normally the cause of destruction to a structure.

Figure 3.2 Blast loadings on a first storey column [N4]

In this study, the explosion centre is assumed to be at the centre of the column and the column surface is taken as the reflected surface. The loading at different points on the front surface of the column for a given charge and standoff distance is computed by LS-DYNA with the built-in CONWEP blast model. The FEM software relates the reflected overpressure to the scaled distance and accounts for the angle of incidence of the blast wave.

3.3.2 Structural Geometry Modeling

A total of four column specimens were fabricated to be tested in the laboratory. The laboratory-testing phase will be described in detail in the next chapter. The columns were primarily different in its reinforcement detail. Two of the column specimens had a transverse reinforcement ratio of 0.19 % and the other two were detailed with 0.58 % of transverse reinforcement. The columns will be labeled as *S1(0.58)*, *S2(0.19)*, *S3(0.58)* and *S4(0.19)* from this point onward. Numerical models of each of these four specimens were created for the analysis. The loading sequence for the two batches of column models will be varied to study its effects on residual axial capacity upon attaining its blast-damaged state. Two pre-axial loads prior to subjecting the model to the blast loads were selected. They are $0.4f'_c A_g$ and $0.2f'_c A_g$. *S1(0.58)* and *S2(0.19)* were pre-axially loaded with $0.4f'_c A_g$ while *S3(0.58)* and *S4(0.19)* were pre-axially loaded with $0.2f'_c A_g$. The geometric model specifications of each of these models are described in the following subsections.

3.3.2.1 Specifications of Specimens *S1(0.58)* and *S3(0.58)*

Specimens *S1(0.58)* and *S3(0.58)* were fabricated with a prescribed concrete grade of 25 N/mm². Both columns have a height of 2.4 m. Their cross-sectional dimension is 260 x 260 mm. Eight T16 bars were arranged as the longitudinal reinforcement and two sets of hoops with R6 bars at 100 mm spacing were used as the transverse reinforcement. Both specimens had a transverse reinforcement ratio of 0.58 %. All the transverse reinforcement was bent at an angle of 135°. **Figure 3.3** shows the typical geometry and reinforcement details of Specimens *S1(0.58)* and *S3(0.58)*.

Figure 3.3 Typical reinforcement detail of S1(0.58) & S3(0.58)

Finite element models for reinforced concrete structures are generally constructed by replacing the composite continuum by an assembly of elements to represent the concrete and steel reinforcement. In this study, a 3-D model was developed to achieve more accurate results from the numerical analysis. The brick element was used to model the concrete

material while both longitudinal and transverse reinforcements were modeled with beam elements. **Figure 3.4** shows a typical schematic of the model created to represent $SI(0.58)$ and $S3(0.58)$.

Figure 3.4 Typical model of $SI(0.58)$ & $S3(0.58)$

Figure 3.5 shows the reinforcement detail within the model of Specimens *S1(0.58)* and *S3(0.58)*.

Figure 3.5 Typical reinforcement detail within models of *S1(0.58)* & *S3(0.58)*

In Finite Element analysis, the general rule of thumb is that the accuracy of the simulation is dependent on the grading of mesh that was used to create the model. Selecting finer mesh to discretize the actual specimen would produce results that are more accurate. However, generating fine mesh is not only a cumbersome process, it also requires a server

with substantial processing capacity capable of carrying out the numerical simulation. In this study, the concrete mesh was established so that the reinforcement nodes coincide with concrete nodes and it was considered able to obtain results with a reasonable level of accuracy. The concrete was discretized using brick elements with an element size of 100 mm. Each element had an aspect ratio of approximately one. Reinforcing bars were discretized using 2-node beam elements with an element size of 100 mm as well. A total of 3626 nodes, 784 beam and 2052 solid elements were used to build each model. Each of these nodes had three degrees of freedom.

3.3.2.2 Specifications of Specimens *S2(0.19)* and *S4(0.19)*

Specimens *S2(0.19)* and *S4(0.19)* were made with a prescribed concrete grade of 25 N/mm². The column height is 2.4 m. Its cross sectional dimension is 260 x 260 mm. Eight T16 bars were arranged as the longitudinal reinforcement and R6 bars at 175 mm spacing were used as the transverse reinforcement. Both the specimens have a transverse reinforcement ratio of 0.19 %. **Figure 3.6** shows the typical geometry and reinforcement details of Specimens *S2(0.19)* and *S4(0.19)*.

Figure 3.6 Typical reinforcement details of Specimens *S2(0.19)* & *S4(0.19)*

3-D finite element models of these two specimens were created to numerically simulate the behaviour of these specimens when subjected to blast loadings while pre-axially loaded to two different levels. *S2(0.19)* was pre-axially loaded to $0.4f'_c A_g$ while *S4(0.19)* was pre-loaded to $0.2f'_c A_g$. The residual axial capacity of these models in its blast-damaged state was of interest to be studied by the author. The brick element was used to represent concrete and the beam element to represent longitudinal and transverse reinforcement. A total of 3626 nodes, 784 beam and 2052 solid elements were used to build each model.

Each of these nodes had three degrees of freedom. **Figure 3.7** shows a typical schematic of the model created to represent $S2(0.19)$ and $S4(0.19)$.

Figure 3.7 Typical model of $S2(0.19)$ & $S4(0.19)$

Figure 3.8 shows the reinforcement detail within the model of Specimens *S2(0.19)* and *S4(0.19)*.

Figure 3.8 Typical reinforcement detail within model of *S2(0.19)* & *S4(0.19)*

3.3.3 Concrete Element

Eight-node solid hexahedron elements with 1-point integration are used to represent the concrete in the models as shown in **Figure 3.9** [L4].

Figure 3.9 Eight-node solid hexahedron element [L4]

If a mesh of 8-node hexahedron solid elements interconnected at nodal points were superimposed on a reference configuration and the particles were tracked through time, i.e.

$$x_i(X_a, t) = x_i(X_a(\xi, \eta, \zeta), t) = \sum_{j=1}^8 \phi_j(\xi, \eta, \zeta) x_i^j(t) \quad (3.1)$$

The shape function ϕ_j is defined for the 8-node hexahedron as

$$\phi_j = \frac{1}{8} (1 + \xi \xi_j) (1 + \eta \eta_j) (1 + \zeta \zeta_j) \quad (3.2)$$

where ξ_j, η_j, ζ_j take on their nodal values of $(\pm 1, \pm 1, \pm 1)$ and x_i is the nodal coordinate of the j^{th} node in the i^{th} direction.

For a solid element, N is the 3x24 rectangular interpolation matrix given by

$$N(\xi, \eta, \zeta) = \begin{bmatrix} \phi_1 & 0 & 0 & \phi_2 & 0 & \dots & 0 & 0 \\ 0 & \phi_1 & 0 & 0 & \phi_2 & \dots & \phi_8 & 0 \\ 0 & 0 & \phi_1 & 0 & 0 & \dots & 0 & \phi_8 \end{bmatrix} \quad (3.3)$$

σ is the stress vector, given by:

$$\sigma^t = (\sigma_{xx}, \sigma_{yy}, \sigma_{zz}, \sigma_{xy}, \sigma_{yz}, \sigma_{zx}) \quad (3.4)$$

B is the 6 x 24 strain-displacement matrix

$$B = \begin{bmatrix} \frac{\partial}{\partial x} & 0 & 0 \\ 0 & \frac{\partial}{\partial y} & 0 \\ 0 & 0 & \frac{\partial}{\partial z} \\ \frac{\partial}{\partial y} & \frac{\partial}{\partial x} & 0 \\ 0 & \frac{\partial}{\partial z} & \frac{\partial}{\partial y} \\ \frac{\partial}{\partial z} & 0 & \frac{\partial}{\partial x} \end{bmatrix} N \quad (3.5)$$

In order to achieve a diagonal mass matrix the rows are summed giving the k^{th} diagonal term as:

$$m_{kk} = \int_V \rho \phi_k \sum_{j=1}^8 \phi_j dV = \int_V \rho \phi_k dV \quad (3.6)$$

Terms in the strain-displacement matrix are readily calculated, which can be written as:

$$\begin{aligned} \frac{\partial \phi_i}{\partial \xi} &= \frac{\partial \phi_i}{\partial x} \frac{\partial x}{\partial \xi} + \frac{\partial \phi_i}{\partial y} \frac{\partial y}{\partial \xi} + \frac{\partial \phi_i}{\partial z} \frac{\partial z}{\partial \xi} \\ \frac{\partial \phi_i}{\partial \eta} &= \frac{\partial \phi_i}{\partial x} \frac{\partial x}{\partial \eta} + \frac{\partial \phi_i}{\partial y} \frac{\partial y}{\partial \eta} + \frac{\partial \phi_i}{\partial z} \frac{\partial z}{\partial \eta} \\ \frac{\partial \phi_i}{\partial \zeta} &= \frac{\partial \phi_i}{\partial x} \frac{\partial x}{\partial \zeta} + \frac{\partial \phi_i}{\partial y} \frac{\partial y}{\partial \zeta} + \frac{\partial \phi_i}{\partial z} \frac{\partial z}{\partial \zeta} \end{aligned} \quad (3.7)$$

which can be rewritten as

$$\begin{bmatrix} \frac{\partial \phi_i}{\partial \xi} \\ \frac{\partial \phi_i}{\partial \eta} \\ \frac{\partial \phi_i}{\partial \zeta} \end{bmatrix} = \begin{bmatrix} \frac{\partial x}{\partial \xi} & \frac{\partial y}{\partial \xi} & \frac{\partial z}{\partial \xi} \\ \frac{\partial x}{\partial \eta} & \frac{\partial y}{\partial \eta} & \frac{\partial z}{\partial \eta} \\ \frac{\partial x}{\partial \zeta} & \frac{\partial y}{\partial \zeta} & \frac{\partial z}{\partial \zeta} \end{bmatrix} \begin{bmatrix} \frac{\partial \phi_i}{\partial x} \\ \frac{\partial \phi_i}{\partial y} \\ \frac{\partial \phi_i}{\partial z} \end{bmatrix} = J \begin{bmatrix} \frac{\partial \phi_i}{\partial x} \\ \frac{\partial \phi_i}{\partial y} \\ \frac{\partial \phi_i}{\partial z} \end{bmatrix} \quad (3.8)$$

Inverting the Jacobian matrix, J, we can solve the desired terms

$$\begin{bmatrix} \frac{\partial \phi_i}{\partial x} \\ \frac{\partial \phi_i}{\partial y} \\ \frac{\partial \phi_i}{\partial z} \end{bmatrix} = J^{-1} \begin{bmatrix} \frac{\partial \phi_i}{\partial \xi} \\ \frac{\partial \phi_i}{\partial \eta} \\ \frac{\partial \phi_i}{\partial \zeta} \end{bmatrix} \quad (3.9)$$

Volume integration is carried out with Gaussian quadrature. Assume g is some function defined over the volume, and n is the number of integration points, then

$$\int_V g dv = \int_{-1}^1 \int_{-1}^1 \int_{-1}^1 g |J| d\xi d\eta d\zeta \quad (3.10)$$

is approximated by

$$\sum_{j=1}^n \sum_{k=1}^n \sum_{l=1}^n g_{jkl} |J_{jkl}| w_j w_k w_l \quad (3.11)$$

where w_j, w_k, w_l are the weighting factors,

$$g_{jkl} = g(\xi_j, \eta_k, \zeta_l) \quad (3.12)$$

and J is the determinant of the Jacobian matrix. For one-point quadrature

$$\begin{aligned} n &= 1 \\ w_i &= w_j = w_k = 2 \\ \xi_1 &= \eta_1 = \zeta_1 = 0 \end{aligned} \quad (3.13)$$

and we can write

$$\int g dv = 8g(0,0,0)|J(0,0,0)| \quad (3.14)$$

where $8|J(0,0,0)|$ approximates the element volume.

The biggest disadvantage of one-point integration is the need to control zero energy modes that arise, referred to as hourglassing modes. To prevent this from occurring, LS-DYNA has several hourglassing control types. In this study, the Flanagan-Belytschko stiffness form was used.

3.3.4 Reinforcement Element

The steel reinforcement is modeled explicitly in the model. The 2-node Hughes-Liu beam element formulation with 2x2 Gauss quadrature integration was used. The formulation has several desirable qualities i.e. it is simple, which usually translates into computational efficiency and robustness; it is compatible with the brick elements because the element is based on a degenerated brick element formulation [L4].

To degenerate the 8-node brick geometry into the 2-node beam geometry, the four nodes at $\xi = -1$ and $\xi = 1$ are combined into a single node with three translational and three rotational degrees of freedom. Orthogonal, inextensible nodal fibers are defined at each

node for treating the rotational degree of freedom. The mapping of the bi-unit cube into the beam elements is separated into three parts, as shown in **Figure 3.10** [L4].

Figure 3.10 Highes-Liu beam element formulation [L4]

$$x(\xi, \eta, \zeta) = \bar{x}(\zeta) + X(\xi, \eta, \zeta) = \bar{x}(\zeta) + X_{\zeta}(\xi, \zeta) + X_{\eta}(\xi, \eta) \quad (3.15)$$

where $\bar{x}$ denotes a position vector to a point on the reference axis of the beam, and X_{ζ} and X_{η} are position vectors at point $\bar{x}$ on the axis that define the fiber directions through that point.

$$\bar{x}(\xi) = N_a(\xi)\bar{x}_a \quad (3.16)$$

$$X_n(\xi, \eta) = N_a(\xi)X_{\eta a}(\eta) \quad (3.17)$$

$$X\zeta(\xi, \zeta) = N_a(\xi)X_{\zeta a}(\zeta) \quad (3.18)$$

Arbitrary points on the reference line $\bar{x}$ are interpolated by the one-dimensional shape function $N(\xi)$. Points off the reference axis are further interpolated by using a one-dimensional shape function along the fiber direction, i.e., $X_{\eta a}(\eta)$ and $X_{\zeta a}(\zeta)$ where

$$X_{\eta a}(\eta) = z_{\eta}(\eta)\hat{X}_{\eta a} \quad (3.19a)$$

$$z_{\eta}(\eta) = N_+(\eta)z_{\eta a}^+ + N_-(\eta)z_{\eta a}^- \quad (3.19b)$$

$$N_+(\eta) = \frac{1+\eta}{2} \quad (3.19c)$$

$$N_-(\eta) = \frac{1-\eta}{2} \quad (3.19d)$$

$$X_{\zeta a}(\zeta) = z_{\zeta}(\zeta)\hat{X}_{\zeta a} \quad (3.20a)$$

$$z_{\zeta}(\zeta) = N_+(\zeta)z_{\zeta a}^+ + N_-(\zeta)z_{\zeta a}^- \quad (3.20b)$$

$$N_+(\zeta) = \frac{1+\zeta}{2} \quad (3.20c)$$

$$N_-(\zeta) = \frac{1-\zeta}{2} \quad (3.20d)$$

where $z_{\zeta}(\zeta)$ and $z_{\eta}(\eta)$ are thickness functions.

To locate the reference axis and define the initial fiber directions, four position vectors are used. $x_{\zeta a}^+$ and $x_{\zeta a}^-$ are two position vectors located on the top and bottom surfaces at node a, respectively.

$$\bar{x}_{\zeta a} = \frac{1}{2}(1 - \bar{\zeta})x_{\zeta a}^- + (1 + \bar{\zeta})x_{\zeta a}^+ \quad (3.21a)$$

$$\hat{X}_{\zeta a} = \frac{(x_{\zeta a}^+ - x_{\zeta a}^-)}{\|x_{\zeta a}^+ - x_{\zeta a}^-\|} \quad (3.21b)$$

$$z_{\zeta a}^+ = \frac{1}{2}(1 - \bar{\zeta})\|x_{\zeta a}^+ - x_{\zeta a}^-\| \quad (3.21c)$$

$$z_{\zeta a}^- = -\frac{1}{2}(1 + \bar{\zeta})\|x_{\zeta a}^+ - x_{\zeta a}^-\| \quad (3.21d)$$

$$\bar{x}_{\eta a} = \frac{1}{2}(1 - \bar{\eta})x_{\eta a}^- + (1 + \bar{\eta})x_{\eta a}^+ \quad (3.22a)$$

$$\hat{X}_{\eta a} = \frac{(x_{\eta a}^+ - x_{\eta a}^-)}{\|x_{\eta a}^+ - x_{\eta a}^-\|} \quad (3.22b)$$

$$z_{\eta a}^+ = \frac{1}{2}(1 - \bar{\eta})\|x_{\eta a}^+ - x_{\eta a}^-\| \quad (3.22d)$$

$$z_{\eta a}^- = -\frac{1}{2}(1 + \bar{\eta})\|x_{\eta a}^+ - x_{\eta a}^-\| \quad (3.22d)$$

where $\| \cdot \|$ is the Euclidean norm.

The same parametric representation used to describe the geometry of the beam elements is used to interpolate the beam element displacements. The displacements are separated into the reference axis displacements and rotations associated with the fiber directions:

$$u(\xi, \eta, \zeta) = \bar{u}(\xi) + U(\xi, \eta, \zeta) = \bar{u}(\xi) + U_{\zeta}(\xi, \zeta) + U_{\eta}(\xi, \eta) \quad (3.23a)$$

$$\bar{u}(\xi) = N_a(\xi)\bar{u}_a \quad (3.23b)$$

$$U_{\eta}(\xi, \eta) = N_a(\xi)U_{\eta a}(\eta) \quad (3.23c)$$

$$U_{\zeta}(\xi, \zeta) = N_a(\xi)U_{\zeta a}(\zeta) \quad (3.23d)$$

$$U_{\eta a}(\eta) = z_{\eta a}(\eta)\hat{U}_{\eta a} \quad (3.23e)$$

$$U_{\zeta a}(\zeta) = z_{\zeta a}(\zeta)\hat{U}_{\zeta a} \quad (3.23f)$$

where u is the displacement of a generic point, u is the displacement of a point on the reference surface, and U is the fiber displacement rotations.

This type of beam contains six translational and six rotational degrees of freedom. A reference node is needed for each beam to determine the initial orientation of the cross-sections, as shown in **Figure 3.11** [L4].

Figure 3.11 LS-DYNA beam definition [L4]

3.3.5 Interaction between Reinforcement and Concrete Element

Bond is the interaction between reinforcing steel and the surrounding area. The force transfer from steel to concrete are attributed to three different phenomena: (1) chemical adhesion between mortar paste and bar surface; (2) friction and wedging action of small dislodged sand particles and the surrounding concrete; and (3) mechanical interaction between concrete and steel. Deformed bars have better bond than plain bars, because most of the force is transferred through the lugs. Friction and chemical adhesion forces are secondary and are quickly lost as the reinforcing bars start to slip.

In this analysis, complete compatibility of strains between concrete and steel is assumed, which implies perfect bond. Although it does impose some errors, it is the practical solution considering the unmanageable size of the computational works and the lack of reliable data on this topic in blast environments. In order to obtain the residual axial capacity of the columns, the reinforcing bars are modeled explicitly using beam element instead of being uniformly distributed over the concrete elements as in smeared model. As a result, there are “shared nodes” between the concrete mesh and reinforcement mesh and no slip at these shared nodes, as shown in **Figure 3.12**.

Figure 3.12 Interaction between concrete and reinforcement elements

3.3.6 Material Modeling and Parameter Selection

Success in analyzing reinforced concrete element problems depends significantly on making sensible choices regarding the material parameters of concrete and reinforcing bars. Given the complex behaviour of concrete, it is not surprising that a large number of distinctively different constitutive models for concrete have been developed and incorporated into various FEM computer codes. The material model used in this present study will be discussed in depth in the following sub sections.

3.3.6.1 Concrete Material Model

Finite element code LS-DYNA, which is used in this research, contains several material models that can be used to represent concrete. Namely, they are material type 5 (Soil and Crushable Foam), material type 14 (Soil and Crushable Foam Failure), material type 16 (pseudo tensor), material type 25 (geological cap model), material type 72RW3 (concrete damage), material type 84 (Winfrith concrete), and material type 96 (brittle damage). Material type 72 RW3 (MAT_CONCRETE_DAMAGE), was the third release of Karagozian and Case (K&C) concrete model. It is a plasticity-based model, using three shear failure surfaces that include damage and strain-rate effects. The most significant user improvement provided by Release III is the default parameter generation function based solely on the unconfined compressive strength. It provides a robust representation of complex concrete laboratory responses [S4]. It has been used successfully to model the behaviour of standard reinforced concrete dividing walls subjected to blast loads [M6]. It was chosen to model concrete elements in this study.

3.3.6.1.1 Failure Surface

This model assumes concrete behaves in a phenomenological manner as an isotropic material so that a failure surface can be represented as an isotropic function of its stress invariants. Failure limits can then be represented as surfaces in a three-dimensional principal-stress space as shown in **Figure 3.13** [C2]. The failure surface is a convex cone with curved meridians and noncircular deviatoric sections defined by the maximum allowable strength values.

Figure 3.13 Failure surface of concrete in a 3-D stress space [C2]

The stress-tensor σ_{ij} is expressed as the sum of a purely hydrostatic stress σ_m and a deviation S_{ij} from the hydrostatic state:

$$\sigma_{ij} = S_{ij} + \sigma_m \delta_{ij} \quad (3.24)$$

$$\sigma_m = \frac{1}{3}(\sigma_x + \sigma_y + \sigma_z) \quad (3.25)$$

is termed as the pure hydrostatic stress and S_{ij} as the deviatoric stress or the deviatoric stress tensor, which represents a state of pure shear.

As discussed earlier, the geometric construction of such a surface is usually represented in terms of hydrostatic and deviatoric sections. The hydrostatic section defines the meridians of the failure surface that are the intersections of curves between the failure surface and a plane containing the hydrostatic axis. The deviatoric section is the intersection between the failure surface and a deviatoric plane that is perpendicular to the hydrostatic axis. **Figure 3.14** [M6] shows an example of deviatoric section. The two extreme meridian planes (farthest and closest intersections from the hydrostatic axis) are called the compressive and tensile meridian, respectively. The path between the compressive meridian and the tensile

meridian (distance r as a function of θ) is defined by an elliptical curve developed by William and Warnke [W1].

Figure 3.14 Cross-section of the failure surface [M6]

The hydrostatic stress tensor changes the concrete volume and the deviatoric stress tensor conjugates its shape deformation. Hydrostatic pressure has a significant influence on the strain hardening and failure of concrete. Under hydrostatic pressure, concrete can be consolidated beyond the limit of elasticity but cannot be crushed to failure [L4]. Therefore, concrete compression failure is governed by the deviatoric components (or shear components) of the stress state.

For the hydrostatic stress tensor, the compaction model is a multi-linear approximation of internal energy. Pressure is defined by

$$p = C(\varepsilon_v) + \gamma T(\varepsilon_v)E \quad (3.26)$$

where, E is the internal energy per initial volume and γ is the ratio of specific heats. The volumetric strain, (ε_v) , is given by the natural logarithm of the relative volume.

As shown in **Figure 3.15** [L4], the model contains an elastic path from the hydrostatic tension cutoff to the point T of elastic limit.

Figure 3.15 Pressure versus volumetric strain curve [L4]

When tension stress exceeds the hydrostatic tension cutoff, tensile failure occurs, which corresponds to the bulk failure region. When the volumetric strain is greater than point T, compaction occurs and gradually the concrete converts into a granular kind of material. Then the volumetric strain almost does not increase any further. Unloading occurs along the unloading bulk modulus to the pressure cutoff. Reloading always follows the unloading path to the point where unloading began, and continues along the loading path.

During initial loading or reloading, the deviatoric stresses remain elastic until the stress point reaches the initial yield surface. The deviatoric stresses then can be further increased until the maximum yield surface is reached. Beyond this stage, the response can be perfectly plastic or softened to a residual surface. **Figure 3.16** [M6] shows the strength

model used to analyze the deviatoric strength tensor, which changes its shape depending on the pressure.

Figure 3.16 Strength model for concrete material [M6]

The pressure is defined as follows:

$$p = \frac{1}{3}(\sigma_x + \sigma_y + \sigma_z) \quad (3.27)$$

The curves above and below the p -axis, correspond to the compressive and tensile meridians, respectively. The failure surfaces are defined as follows [M6]:

$$\Delta\sigma_m = a_0 + \frac{p}{a_1 + a_2 p} \quad (\text{maximum failure surface}) \quad (3.28)$$

$$\Delta\sigma_r = \frac{p}{a_{1f} + a_{2f} p} \quad (\text{residual failure surface}) \quad (3.29)$$

$$\Delta\sigma_y = a_{0y} + \frac{p}{a_{1y} + a_{2y} p} \quad (\text{yield failure surface}) \quad (3.30)$$

where $\Delta\sigma$ is the failure surface for the deviatoric stresses, defined as:

$$\Delta\sigma = \sqrt{3J_2} \quad (3.31)$$

and J_2 is the second invariant of the deviatoric stress tensor as previously defined.

The parameters (a_0, a_1, a_3) can be determined from available laboratory data in unconfined compression tests and conventional triaxial compression tests at a range of confining pressures.

3.3.6.1.2 Damage Accumulation

- Shear damage accumulation

After reaching the initial surface but prior to the maximum failure surface, the current surface is obtained as a linear interpolation between the two:

$$\Delta\sigma = \eta(\Delta\sigma_m - \Delta\sigma_y) + \Delta\sigma_y \quad (3.32)$$

The parameter η is a user-defined function of a modified plastic strain measure, λ . The function $\eta(\lambda)$ is intended to first increase from some initial value up to unity, then decrease to zero to represent softening.

After reaching the maximum surface, the current failure surface is similarly interpolated between the maximum and the residual points:

$$\Delta\sigma = \eta(\Delta\sigma_m - \Delta\sigma_r) + \Delta\sigma_r \quad (3.33)$$

To obtain current failure surface, whether to use **Equation 3.32** or **Equation 3.33** depends on whether $\lambda \leq \lambda_m$ or $\lambda \geq \lambda_m$. The modified effective plastic strain, λ is then defined as:

$$\lambda = \begin{cases} \int_0^{\varepsilon^p} \frac{d\varepsilon^p}{r_f (1 + p/r_f f_t)^{b_1}} & \text{for } p \geq 0 \\ \int_0^{\varepsilon^p} \frac{d\varepsilon^p}{r_f (1 + p/r_f f_t)^{b_2}} & \text{for } p < 0 \end{cases} \quad (3.34)$$

where r_f is the strain rate enhancement factor, f_t is the tensile strength of concrete, p is the pressure as previously defined, and b_1, b_2 are damage evolution parameters.

- Volumetric damage accumulation

With damage accumulation described in Equation (3.34), suppose a triaxial tensile test is modeled, if the pressure decreases from 0 to $-f_t$ with no deviators, then no damage accumulation occurs. The equation of state decreases the pressure to $-f_t$ and remains at that level thereafter. When the stress path is close to the triaxial tensile test path, a volumetric damage increment can be added to the deviatoric damage to implement pressure decay after tensile failure.

The closeness to this path is measured by the ratio $|\sqrt{3J_2}/p|$, which, for example, is 1.5 for the biaxial tensile test. To limit the effects of this change to the paths close to the triaxial tensile path, the incremental damage is multiplied by a factor f_d given by:

$$f_d = \begin{cases} 1 - \frac{|\sqrt{3J_2}/p|}{0.1}, & 0 \leq |\sqrt{3J_2}/p| < 0.1 \\ 0, & |\sqrt{3J_2}/p| \geq 0.1 \end{cases} \quad (3.35)$$

The modified effective plastic strain is incremented by:

$$\Delta\lambda = b_3 f_d k_d (\varepsilon_v - \varepsilon_{v,yield}) \quad (3.36)$$

where, b_3 is the input scalar multiplier, k_d is the internal scalar multiplier, ε_v is the volumetric strain, and $\varepsilon_{v,yield}$ is the volumetric strain at yield.

The values of b_2 and b_3 govern the softening part of the unconfined uniaxial tension stress-strain curve. These parameters are used to compute the modified effective plastic strain λ and the value of λ will determine η , which is then used to compute the current surface, as indicated in **Equations (3.32) and (3.33)**. **Figure 3.17 (a) and (b)** [M6] shows an example of the effects of b_2 and b_3 on the stress-strain curve for a uniaxial unconfined tensile test. The parameter b_1 governs the softening part of the compressive strength, and it is used to match observed compression behaviour at various levels of transverse confinement, as shown in **Figure 3.17 (c)** [M6].

(a) Parameter b_2 : effect on uniaxial tension strain softening

(b) Parameter b_3 : effect on triaxial tension strain softening

(c) Model representation of triaxial compression tests

Figure 3.17 Effects of damage parameters b_1 , b_2 and b_3 [M6]

3.3.6.1.3 Rate Effects

Experimental studies show that concrete is a highly rate-dependent material. This strain rate sensitivity is traditionally expressed in terms of the Dynamic Increase Factor (*DIF*),

the ratio of the dynamic to static strength. It is a convenient format for implementation into numerical models. The expressions proposed by Malvar and Crawford [M3] are utilized in this project.

The *DIF* for compressive strength of concrete is given in Section 2.5.1 of this report in **Equations 2.16 and 2.17**. **Figure 3.18** [M3] shows the plot of *DIF* for compressive strength of concrete based on the expressions proposed by Malvar and Crawford [M3].

Figure 3.18 *DIF* for concrete in compression [M3]

The *DIF* for the tensile strength of concrete is also given in Section 2.5.1 of this thesis and expressed in **Equation 2.18**. **Figure 3.19** [M3] shows the plot of *DIF* for tensile strength of concrete based on the expressions proposed by Malvar and Crawford [M3].

Figure 3.19 *DIF* for concrete in tension [M3]

It can be seen that the tensile response is more sensitive to strain rate than compressive response. Therefore, different rate enhancements are included in “tension and compression” in the concrete material model employed in this study.

In the numerical model, the strain rate effect is incorporated as follows. At any given pressure, the failure surfaces are expanded by a rate enhancement factor that depends on the effective deviatoric strain rate, as shown in **Figure 3.20** [M6].

Figure 3.20 rate enhancement in tension and compression [M6]

Let r_f be the strain rate enhancement factor and p the pressure, an “unenhanced” pressure p/r_f is first obtained, then the unenhanced strength $\Delta\sigma(p/r_f)$ is calculated for the specified failure surface. Finally, the enhanced strength is given by:

$$\Delta\sigma_e = r_f \Delta\sigma(p/r_f) \quad (3.30)$$

Strength is enhanced equally along any radial stress path, including uniaxial, biaxial and triaxial tension, and uniaxial and biaxial compression. The effective strain rate versus deviatoric strength enhancement is given by the LS-DYNA Define Curve keyword.

3.3.6.1.4 Material Type 72 RW3

Material type 72 RW3 (MAT_CONCRETE_DAMAGE) is selected to be utilized for this research. Listed below are the characteristics of this material type that led to the selection of the material type:

- Concrete material behaviour along multiple radial paths in the $\Delta\sigma$ versus p space, including stress strain response, tension cracking, biaxial stiffening, and strain softening phenomena allows for proper modeling of concrete.
- Damage accumulations are included with damage evolution parameters.
- Strain rate effects are properly captured in both tension and compression states.

Along with these features, this material model has seven material card images that are required to be defined to attain a complete set of model parameters. An Equation-of-State (EOS) is also included to define the pressure-volume strain response.

3.3.6.2 Reinforcing Steel Material Model

The reinforcement steel within the column is modeled as a strain sensitive uniaxial elasto-plastic material to account for its strain rate sensitivity as well as stress-strain history dependence. For simplicity, bilinear idealization has been adopted. This bilinear idealization is capable of representing both the linear isotropic hardening and elasto-plastic behaviour of steel. This is illustrated in **Figure 3.21**. The curve is assumed identical in tension and compression. In addition, unloading is assumed to occur elastically.

Figure 3.21 Uniaxial behaviour of reinforcing steel bars

The expressions proposed by Malvar and Crawford [M4] on strain rate sensitivity are utilized in this study. **Figure 3.22** [M4] shows this proposed *DIF* for both yield and ultimate stress.

Figure 3.22 Proposed *DIF* for reinforcing steel bars [M4]

For the yield stress of reinforcing bars:

$$DIF = (\dot{\epsilon} / 10^{-4})^{\alpha} \quad (3.31)$$

where $\alpha = \alpha_{fy}$; $\alpha_{fy} = 0.074 - 0.04 f_y / 414$; f_y is the reinforcement static yield strength in MPa. This formula is valid for reinforcement with yield stress between 290 and 710 MPa and for strain rates between $10^{-4} s^{-1}$ and $225 s^{-1}$.

For the ultimate stress of reinforcing bars:

$$DIF = (\dot{\epsilon} / 10^{-4})^{\alpha} \quad (3.32)$$

where $\alpha = \alpha_{fu}$; $\alpha_{fu} = 0.019 - 0.009 f_u / 414$; f_u is the reinforcement static ultimate strength in MPa. This formula is valid for reinforcement with yield stress between 290 and 710 MPa and for strain rates between $10^{-4} s^{-1}$ and $225 s^{-1}$.

Material type 24 (MAT_PIECEWISE_LINEAR_PLASTICITY) from LS-DYNA is employed for this study to incorporate for the strain rate sensitivity in the reinforcing bars. In this material model, rate effects are accounted for by defining a table of curves, where effective plastic strain versus yield stress is defined for each strain rate as shown in **Figure 3.23** [L4].

Figure 3.23 Table of curves to represent stress-strain curves at different strain rates [L4]

3.3.7 Boundary Conditions

A typical column element within a structure would be connected to a footing either at the bottom end or secondary floor beams or slabs at both ends. The sample specimen is intended to be tested in the laboratory to simulate this actual-life representation as close as possible. To best simulate these boundary conditions in the model, two very stiff, linear elastic steel blocks were utilized as the boundary conditions for the finite element model of the column. In addition, a rigid plate that was only allowed to move vertically was attached to top stiff block of the column. This rigid plate was modeled to allow axial loads to be applied throughout the entire cross-section area of the column model. **Figure 3.24** illustrates the location of these stiff blocks and rigid plate in the model.

Rigid plate allowing for vertical
movement with end rotation fixity

Figure 3.24 Boundary conditions of finite element model of column

3.3.8 Analysis Steps and Load Patterns

The purpose of the finite element simulation was to obtain the deflection of the column specimen at three specific locations where the actuators are positioned in the laboratory. This deflection would then be programmed into the displacement-controlled actuator

settings during the experimental study phase to achieve a column deflection profile that simulates effects from a blast load.

- Apply gravity loading via a slow ramp ($0.4\text{f}^{\circ}\text{cAg}$ or $0.2\text{f}^{\circ}\text{cAg}$)
- Perform static finite element analysis to determine shortening of the column model if any
- Apply blast loading from the equivalent of 1 ton of TNT placed at a standoff distance of 7.5 m
- Obtain the lateral displacement profile of the column model
- Increase axial loading up to a point where the axial capacity of the column mode begins to take a dip

These loading steps are illustrated in **Figure 3.25**.

Figure 3.25 Loading steps in simulation

3.3.9 Integration Method Selection

The general form of the equation of motion that governs structural dynamic responses is expressed as:

$$M\ddot{u} + C\dot{u} + Ku = Q \quad (3.33)$$

where, M , C and K are mass, damping and stiffness matrices; Q is the vector of the externally applied load; and, $\ddot{u}$, $\dot{u}$ and u are the acceleration, velocity and displacement responses respectively.

The explicit method using an explicit solver and the implicit method, which utilizes the implicit solver, are the two numerical direct integration methods that can be employed to solve the differential equation of motion. The terms explicit and implicit refer to time integration algorithms.

The explicit solution method is a true dynamic procedure originally developed to model high-speed impact events in which, inertia plays a dominant role in its solution. In the explicit approach, internal and external forces are summed at each node point, and a nodal acceleration is computed by dividing by the nodal mass. Out-of-balance forces are propagated as stress waves between neighboring elements while solving for a state of dynamic equilibrium. The explicit integration method uses the central difference operator for integration of the equation of motion. Suppose the semi-discrete equations of motion at time n are:

$$Ma^n = P^n - F^n + H^n \quad (3.34)$$

where, M is the mass matrix, P^n represents external and body force loads, F^n is the stress divergence vector, and H^n is the hourglass resistance. To move to time t^{n+1} , central difference time integration is used:

$$a^n = M^{-1}(P^n - F^n + H^n) \quad (3.35)$$

$$\dot{u}^{n+1/2} = \dot{u}^{n-1/2} + a^n \Delta t^n \quad (3.36)$$

$$u^{n+1} = u^n + \dot{u}^{n+1/2} \Delta t^{n+1/2} \quad (3.37)$$

where

$$\Delta t^{n+1/2} = \frac{(\Delta t^n + \Delta t^{n+1})}{2} \quad (3.38)$$

and, $\dot{u}$ and u are the velocity and displacement vectors respectively. The geometry is updated by adding the displacement increments to the initial geometry:

$$x^{n+1} = x^0 + u^{n+1} \quad (3.39)$$

Once the displacements are updated, strains can be obtained, which are then used to determine the stresses and eventually nodal forces. Stable integration using the central difference method for undamped problems requires the following time step limit:

$$\Delta t \leq \frac{L}{c_w} \quad (3.40)$$

where, L is related to the element size and c_w is the speed at which stress waves travel in the element. The physical interpolation of this condition for linear displacement elements is that Δt must be small enough so that information does not propagate across more than one element in a time step [C8]. The explicit integration method tries to obtain values for dynamic quantities at time $n + 1$ based entirely on available values at time, n , thus, it is simple and computationally efficient since no iterative calculation is involved.

The implicit integration method uses Hilber-Hughes-Taylor operator, which is an extension of the trapezoidal rule. It tries to solve for dynamic quantities at time $n + 1$ based not only on values at time n , but also on these quantities at time $n + 1$. The Hilber-Hughes-Taylor operator is implicit. The integration operation matrix must be inverted, and a set of simultaneous nonlinear dynamic equilibrium equations must be solved at each step of time increment. This solution is done iteratively using Newton's method. This nonlinear equation solving process is cumbersome, and if the equations are extremely nonlinear, it may lead to a very complex route to obtain its solution. For example, material degradation

and failure often lead to severe convergence difficulties in implicit analysis programs, i.e. concrete cracking model, in which tensile cracking causes the material stiffness to become negative, but the explicit method deals these kind of problem quite well due to its simplicity.

While explicit analysis is well suited to dynamic simulations such as impact and crash, it can be prohibitively expensive to conduct long duration or static analyses. Therefore, to adopt explicit integration method, a suitable simulation time need to be chosen so that inertial effects would be negligible in the gravity load application stage and meantime can save runtime since the analysis is performed in a 3.2 GHz personal computer.

3.4 Finite Element Analysis

The finite element analysis was carried out prior to the laboratory experiments. The outcomes to be obtained from the analysis are the displacement of the column on specific locations when subjected to a blast load, its complete blast-damaged displacement profile and the residual axial capacity of the column upon being damaged by the blast loads. The locations at which the displacements are monitored at would coincide with the positions of the actuators in the laboratory setup. The laboratory setup will be discussed in detail in the following chapter of this report.

Two batches of specimens were modeled and initially pre-axially loaded to either $0.4f'_cA_g$ or $0.2f'_cA_g$. Each batch had two models with varying transverse reinforcement ratio within them. The TNT equivalent weight was set at 1 ton. This was selected considering that it would be difficult for any extremist groups to obtain or transport any explosive of charge weight higher than this quantity. The standoff distance from the blast was set at 7.5 m. The column displacement profile and residual axial capacity of the blast-damaged column models were two important parameters, which the author intended to obtain from this analysis before moving on to the laboratory experiment stage. The following sub-sections

describe the process through which the parameters were obtained from the numerical modeling for each of the models that were created.

3.4.1 FEM Model of Specimen *SI(0.58)*

The model of specimen *SI(0.58)* stands 2.4 m tall and has a cross sectional dimension of 260 x 260 mm. It has a longitudinal reinforcement ratio of 2.3 % and a transverse reinforcement ratio of 0.58 %. This model was initially subjected to a pre-axial load of $0.4f_c A_g$. This pre-axial load was to represent the service loads a column would be subjected to prior to blast wave loads striking its surface. Although the actual specimen was designed with a prescribed concrete strength of 25 MPa, the cylinder test carried out in the laboratory upon casting of the specimen showed that the concrete had a compressive strength of approximately 30 MPa. The detailed cylinder test results are presented in the experimental study phase of this report. As such, 40 % of its compressive strength works out to 811 kN for a column with a 260 x 260 mm cross section. The blast pressures are calculated with the CONWEP software that is in-built within LS-DYNA. The various data obtained from the numerical blast simulation are contained in the following sub-sections.

3.4.1.1 Numerical x-displacement of Specimen *SI(0.58)*

There are in total three loading stages involved in the numerical simulation. The first stage involves applying the pre-axial load to the model. This specimen had a pre-axial load of 811 kN to replicate the service loadings a column would be subjected to as in a real life structure. The blast loading is set to go off at 0.3 sec. **Figure 3.26** shows the model after being subjected to the blast loadings. The residual deflection is defined as the final shape the specimen takes up after it vibrates due to the blast loading. The residual deflected shape of model *SI(0.58)* is attained at a time step of 0.349 sec. LS-DYNA provides erosion criteria for elements that includes principle tensile strain and shear strain. This study only considered the principle tensile strain criterion to determine erosion of elements.

Figure 3.26 Numerical residual displacement of *SI(0.58)*

The x-direction is the horizontal direction that is perpendicular to the height of the column model. A plot of the x-displacement of the model of specimen *SI(0.58)* is provided in **Figure 3.27**. The nodes on the model numbered 2828, 2864 and 2900 coincide with the locations of the top middle and bottom actuator respectively, in the laboratory setup. The residual mid-span deflection of *SI(0.58)* after subjected to the blast load is 28.3 mm. This works out to a drift of 1.18 %. **Table 3.1** tabulates the deflections at the three locations at the time-step when the residual displacement profile is attained by the model.

Figure 3.27 Numerical x-displacement of $S1(0.58)$ Table 3.1 Summary of $S1(0.58)$ numerical residual x-displacements

Node Number	2900	2864	2828
Residual Displacement (mm)	18.54	28.31	22.61

Figure 3.28 plots these points to be able to visualize the deflected shape of the model. This deflected shape would be the target to be achieved by the hydraulic actuators during the laboratory experiment phase of this study.

Figure 3.28 Numerical residual deflection profile of *S1(0.58)*

3.4.1.2 Numerical z-displacement of Specimen *S1(0.58)*

The z-displacement is measured in a direction that is parallel to the height of the column. This vertical movement is monitored to determine the z-displacement of the model when the vibration due to the blast load has ceased. This marks the point or the displacement at which the model is subjected to the axial crushing phase of the loading sequence. This blast residual vertical displacement will be used to determine the residual axial capacity of the model in the next sub-section. A plot of the z-displacement of the model when subjected to the blast load is presented in **Figure 3.29**.

Figure 3.29 Numerical z-displacement of *S1(0.58)*

It can be determined from the plot in **Figure 3.29** that when the vibration due to the blast load ceased in the simulation at a time-step of 0.35 sec through observation, that the z-displacement of the column measured to be 8.48 mm. After this vertical displacement, the column model was subjected to axial loads up until it fails to determine its residual axial capacity.

3.4.1.3 Numerical Residual Axial Capacity of Specimen *S1(0.58)*

The final phase in the loading regime involves increasing the displacement of the model until it is unable to sustain any more loading. The failure load is recorded by measuring the reaction force at the bottom of the model. The model is deemed to have failed once its z-displacement increases while the reaction force at the bottom of the specimen drops. **Figure 3.30** shows a plot of the axial force against the z-displacement the model was subjected to from the pre-axial, blast and residual axial capacity loading stages.

Figure 3.30 Numerical residual axial capacity of S1(0.58)

It can be observed from **Figure 3.30** that upon completion of the vibration from the blast load that S1(0.58) was subjected to, there was an initial increase in axial load. This point can also be traced from the previous figure, to correspond to a vertical z-displacement of 8.48 mm. However, the axial force peaked at 1076 kN at a z-displacement of 13.3 mm and failed to increase any further. This is deemed to be the post-blast residual axial capacity of S1(0.58). This analysis shows that the model would have sufficient post-blast residual axial capacity to sustain its service loadings and therefore can be deemed to have survived the blast loadings.

Figure 3.31 shows the damage-profile of $S1(0.58)$ from the numerical analysis.

Figure 3.31 Numerical damage-profile of $S1(0.58)$

3.4.2 FEM Model of Specimen $S2(0.19)$

The model of specimen $S2(0.19)$ stands 2.4 m tall and has a cross sectional dimension of 260 x 260 mm. It has a longitudinal reinforcement ratio of 2.3 % and a transverse reinforcement ratio of 0.19 %. This model was initially subjected to a pre-axial load of $0.4f_c A_g$. This pre-axial load was to represent the service loads a column would be subjected to prior to blast wave loads striking its surface. Although the actual specimen was designed

with a prescribed concrete strength of 25 MPa, the cylinder test carried out in the laboratory upon casting of the specimen showed that the concrete had a compressive strength of approximately 30 MPa. The detailed cylinder test results are presented in the experimental study phase of this report. As such, 40 % of its compressive strength works out to 811 kN for a column with a 260 x 260 mm cross section. The blast pressures are calculated with the CONWEP software that is in-built within LS-DYNA. The various data obtained from the numerical blast simulation are contained in the following sub-sections.

3.4.2.1 Numerical x-displacement of Specimen S2(0.19)

There are in total three loading stages involved in the numerical simulation. The first stage involves applying the pre-axial load to the model. This specimen had a pre-axial load of 811 kN to replicate the service loadings a column would be subjected to as in a real life structure. The blast loading is set to go off at 0.3 sec. **Figure 3.32** shows the model after being subjected to the blast loadings. The residual deflection is defined as the final shape the specimen takes up after it vibrates due to the blast loading. The residual deflected shape of model S2(0.19) is attained at a time step of 0.35 sec. LS-DYNA provides erosion criteria for elements that includes principle tensile strain and shear strain. This study only considered the principle tensile strain criterion to determine erosion of elements.

Figure 3.32 Numerical residual displacement of *S2(0.19)*

The x-direction is the horizontal direction that is perpendicular to the height of the column model. A plot of the x-displacement of the model of specimen *S2(0.19)* is provided in **Figure 3.33**. The nodes on the model numbered 2828, 2864 and 2900 coincide with the locations of the top middle and bottom actuator respectively, in the laboratory setup. The residual mid-span deflection of *S2(0.19)* after subjected to the blast load is 46.5 mm. This works out to a drift of 1.94 %. **Table 3.2** tabulates the deflections at the three locations at the time-step when the residual displacement profile is attained by the model.

Figure 3.33 Numerical x-displacement of *S2(0.19)*Table 3.2 Summary of *S2(0.19)* numerical residual x-displacements

Node Number	2900	2864	2828
Residual Displacement (mm)	36.56	46.53	36.53

Figure 3.29 plots these points to be able to visualize the deflected shape of the model. This deflected shape would be the target to be achieved by the hydraulic actuators during the laboratory experiment phase of this study.

Figure 3.34 Numerical residual deflection profile of S2(0.19)

3.4.2.2 Numerical z-displacement of Specimen S2(0.19)

The z-displacement is measured in a direction that is parallel to the height of the column. This vertical movement is monitored to determine the z-displacement of the model when the vibration due to the blast load has ceased. This marks the point or the displacement at which the model is subjected to the axial crushing phase of the loading sequence. This blast residual vertical displacement will be used to determine the residual axial capacity of the model in the next sub-section. A plot of the z-displacement of the model when subjected to the blast load is presented in **Figure 3.35**.

Figure 3.35 Numerical z-displacement of S2(0.19)

It can be determined from the plot in **Figure 3.35** that when the vibration due to the blast load ceased in the simulation at a time-step of 0.35 sec through observation, that the z-displacement of the column measured to be 11.73 mm. After this vertical displacement, the column model was subjected to axial loads up until it fails to determine its residual axial capacity.

3.4.2.3 Numerical Residual Axial Capacity of Specimen S2(0.19)

The final phase in the loading regime involves increasing the displacement of the model until it is unable to sustain any more loading. The failure load is recorded by measuring the reaction force at the bottom of the model. The model is deemed to have failed once its z-displacement increases while the reaction force at the bottom of the specimen drops. **Figure 3.36** shows a plot of the axial force against the z-displacement the model was subjected to from the pre-axial, blast and residual axial capacity loading stages.

Figure 3.36 Numerical residual axial capacity of S2(0.19)

It can be observed from **Figure 3.36** that during the blast-loading phase, the model had already lost the capacity to hold its pre-axial load. As such, this model can be deemed to have failed due the blast loadings. It only had a peak axial capacity of 715 kN while shortened by 11.73 mm after being subjected to the blast loads. The z-displacement of S2(0.19) kept increasing following the blast loads. This analysis therefore signifies that the model has failed catastrophically during the blast.

Figure 3.37 shows the damage-profile of $S2(0.19)$ from the numerical analysis.

Figure 3.37 Numerical damage-profile of $S2(0.19)$

3.4.3 FEM Model of Specimen $S3(0.58)$

The model of specimen $S3(0.58)$ stands 2.4 m tall and has a cross sectional dimension of 260 x 260 mm. It has a longitudinal reinforcement ratio of 2.3 % and a transverse reinforcement ratio of 0.58 %. This model was initially subjected to a pre-axial load of $0.2f'_cA_g$. This pre-axial load was to represent the service loads a column would be subjected to prior to blast wave loads striking its surface. Although the actual specimen was designed with a prescribed concrete strength of 25 MPa, the cylinder test carried out in the

laboratory upon casting of the specimen showed that the concrete had a compressive strength of approximately 30 MPa. The detailed cylinder test results are presented in the experimental study phase of this report. As such, 40 % of its compressive strength works out to 406 kN for a column with a 260 x 260 mm cross section. The blast pressures are calculated with the CONWEP software that is in-built within LS-DYNA. The various data obtained from the numerical blast simulation are contained in the following sub-sections.

3.4.3.1 Numerical x-displacement of Specimen S3(0.58)

There are in total three loading stages involved in the numerical simulation. The first stage involves applying the pre-axial load to the model. This specimen had a pre-axial load of 406 kN to replicate the service loadings a column would be subjected to as in a real life structure. The blast loading is set to go off at 0.3 sec. **Figure 3.38** shows the model after being subjected to the blast loadings. The residual deflection is defined as the final shape the specimen takes up after it vibrates due to the blast loading. The residual deflected shape of model S3(0.58) is attained at a time step of 0.38 sec. LS-DYNA provides erosion criteria for elements that includes principle tensile strain and shear strain. This study only considered the principle tensile strain criterion to determine erosion of elements.

Figure 3.38 Numerical residual displacement of *S3(0.58)*

The x-direction is the horizontal direction that is perpendicular to the height of the column model. A plot of the x-displacement of the model of specimen *S3(0.58)* is provided in **Figure 3.29**. The nodes on the model numbered 2828, 2864 and 2900 coincide with the locations of the top middle and bottom actuator respectively, in the laboratory setup. The residual mid-span deflection of *S3(0.58)* after subjected to the blast load is 52.8 mm. This works out to a drift of 2.2 %. **Table 3.3** tabulates the deflections at the three locations at the time-step when the residual displacement profile is attained by the model.

Figure 3.39 Numerical x-displacement of $S3(0.58)$ Table 3.3 Summary of $S3(0.58)$ numerical residual x-displacements

Node Number	2900	2864	2828
Residual Displacement (mm)	32.53	52.81	36.94

Figure 3.40 plots these points to be able to visualize the deflected shape of the model. This deflected shape would be the target to be achieved by the hydraulic actuators during the laboratory experiment phase of this study.

Figure 3.40 Numerical residual deflection profile of S3(0.58)

3.4.3.2 Numerical z-displacement of Specimen S3(0.58)

The z-displacement is measured in a direction that is parallel to the height of the column. This vertical movement is monitored to determine the z-displacement of the model when the vibration due to the blast load has ceased. This marks the point or the displacement at which the model is subjected to the axial crushing phase of the loading sequence. This blast residual vertical displacement will be used to determine the residual axial capacity of the model in the next sub-section. A plot of the z-displacement of the model when subjected to the blast load is presented in **Figure 3.41**.

Figure 3.41 Numerical z-displacement of S3(0.58)

It can be determined from the plot in **Figure 3.41** that when the vibration due to the blast load ceased in the simulation at a time-step of 0.38 sec through observation, that the z-displacement of the column measured to be 3.47 mm. After this vertical displacement, the column model was subjected to axial loads up until it fails to determine its residual axial capacity.

3.4.3.3 Numerical Residual Axial Capacity of Specimen S3(0.58)

The final phase in the loading regime involves increasing the displacement of the model until it is unable to sustain any more loading. The failure load is recorded by measuring the reaction force at the bottom of the model. The model is deemed to have failed once its z-displacement increases while the reaction force at the bottom of the specimen drops. **Figure 3.42** shows a plot of the axial force against the z-displacement the model was subjected to from the pre-axial, blast and residual axial capacity loading stages.

Figure 3.42 Numerical residual axial capacity of S3(0.58)

It can be observed from **Figure 3.42** that upon completion of the vibration from the blast load that S3(0.58) was subjected to, there was an initial increase in axial load. This point can also be traced from the previous figure, to correspond to a vertical z-displacement of 3.47 mm. However, the axial force peaked at 621 kN at a z-displacement of 7.15 mm and failed to increase any further. This is deemed to be the post-blast residual axial capacity of S3(0.58). The analysis shows that as this model that was only subjected to a pre-axial load of $0.2f_c A_g$, has a post-blast residual axial capacity that is sufficient to sustain its service loadings and is deemed to have survived the blast.

Figure 3.43 shows the damage-profile of *S3(0.58)* from the numerical analysis.

Figure 3.43 Numerical damage-profile of *S3(0.58)*

3.4.4 FEM Model of Specimen *S4(0.19)*

The model of specimen *S4(0.19)* stands 2.4 m tall and has a cross sectional dimension of 260 x 260 mm. It has a longitudinal reinforcement ratio of 2.3 % and a transverse reinforcement ratio of 0.19 %. This model was initially subjected to a pre-axial load of $0.2f'_cA_g$. This pre-axial load was to represent the service loads a column would be subjected to prior to blast wave loads striking its surface. Although the actual specimen was designed

with a prescribed concrete strength of 25 MPa, the cylinder test carried out in the laboratory upon casting of the specimen showed that the concrete had a compressive strength of approximately 30 MPa. The detailed cylinder test results are presented in the experimental study phase of this report. As such, 40 % of its compressive strength works out to 406 kN for a column with a 260 x 260 mm cross section. The blast pressures are calculated with the CONWEP software that is in-built within LS-DYNA. The various data obtained from the numerical blast simulation are contained in the following sub-sections.

3.4.4.1 Numerical x-displacement of Specimen *S4(0.19)*

There are in total three loading stages involved in the numerical simulation. The first stage involves applying the pre-axial load to the model. This specimen had a pre-axial load of 406 kN to replicate the service loadings a column would be subjected to as in a real life structure. The blast loading is set to go of at 0.3 sec. **Figure 3.44** shows the model upon being subjected to the blast loadings. The residual deflection is defined as the final shape the specimen takes up after it vibrates due to the blast loading. The residual deflected shape of model *S4(0.19)* is attained at a time step of 0.35 sec. LS-DYNA provides erosion criteria for elements that includes principle tensile strain and shear strain. This study only considered the principle tensile strain criterion to determine erosion of elements.

Figure 3.44 Numerical residual displacement of *S4(0.19)*

The x-direction is the horizontal direction that is perpendicular to the height of the column model. A plot of the x-displacement of the model of specimen *S4(0.19)* is provided in **Figure 3.45**. The nodes on the model numbered 2828, 2864 and 2900 coincide with the locations of the top middle and bottom actuator respectively, in the laboratory setup. The residual mid-span deflection of *S4(0.19)* after subjected to the blast load is 64.75 mm. This works out to a drift of 2.69 %. **Table 3.4** tabulates the deflections at the three locations at the time-step when the residual displacement profile is attained by the model.

Figure 3.45 Numerical x-displacement of *S4(0.19)*Table 3.4 Summary of *S4(0.19)* numerical residual x-displacements

Node Number	2900	2864	2828
Residual Displacement (mm)	41.65	64.75	49.52

Figure 3.46 plots these points to be able to visualize the deflected shape of the model. This deflected shape would be the target to be achieved by the hydraulic actuators during the laboratory experiment phase of this study.

Figure 3.46 Residual deflection profile of *S4(0.19)*

3.4.4.2 Numerical z-displacement of Specimen *S4(0.19)*

The z-displacement is measured in a direction that is parallel to the height of the column. This vertical movement is monitored to determine the z-displacement of the model when the vibration due to the blast load has ceased. This marks the point or the displacement at which the model is subjected to the axial crushing phase of the loading sequence. This blast residual vertical displacement will be used to determine the residual axial capacity of the model in the next sub-section. A plot of the z-displacement of the model when subjected to the blast load is presented in **Figure 3.47**.

Figure 3.47 Numerical z-displacement of S4(0.19)

It can be determined from the plot in **Figure 3.47** that when the vibration due to the blast load ceased in the simulation at a time-step of 0.35 sec through observation, that the z-displacement of the column measured to be 8.64 mm. After this vertical displacement, the column model was subjected to axial loads up until it fails to determine its residual axial capacity.

3.4.4.3 Residual Axial Capacity of Specimen S4(0.19)

The final phase in the loading regime involves increasing the displacement of the model until it is unable to sustain any more loading. The failure load is recorded by measuring the reaction force at the bottom of the model. The model is deemed to have failed once its z-displacement increases while the reaction force at the bottom of the specimen drops. **Figure 3.48** shows a plot of the axial force against the z-displacement the model was subjected to from the pre-axial, blast and residual axial capacity loading stages.

Figure 3.48 Numerical residual axial capacity of *S4(0.19)*

It can be observed from **Figure 3.48** that upon completion of the vibration from the blast load that *S4(0.19)* was subjected to, there was a slight increase in axial load. This point can also be traced from the previous figure, to correspond to a vertical z-displacement of 8.64 mm. However, the axial force peaked at 491 kN at a z-displacement of 12.29 mm and failed to increase any further. This is deemed to be the post-blast residual axial capacity of *S4(0.19)*. The analysis shows that as this model that was only subjected to a pre-axial load of $0.2f'_cA_g$, has a post-blast residual axial capacity that is sufficient to sustain its service loadings and is deemed to have survived the blast.

Figure 3.49 shows the damage-profile of $S4(0.19)$ from the numerical analysis.

Figure 3.49 Numerical damage-profile of $S4(0.19)$

3.5 Summary of Numerical Results

Table 3.5 provides a summary of the model specifications, pre-axial loadings, and the numerical results obtained from the analysis that was conducted. It serves to provide a convenient source of comparing between the results that were obtained from this stage in the study. The validity of these results and the entire analysis that was carried out will be determined by comparing the residual axial capacities that are obtained from an

Chapter 3: Numerical Study

experimental study conducted in a laboratory as well as with the residual axial capacity prediction equation as proposed by Bao and Li [X1]. These results and comparisons will be presented in the following two chapters of this thesis.

Table 3.5 Summary of numerical results

Model	Height	Cross-Sectional Dimension	Longitudinal Reinforcement Ratio	Transverse Reinforcement Ratio	Pre-Axial Load	Blast Residual Mid-Span X-Deflection	Drift Ratio	Blast Residual Z-Displacement	Blast-Damaged Residual Axial Capacity
S1(0.58)	2400 mm	260 x 260 mm	2.30%	0.58%	811 kN	28.31 mm	1.18%	8.48 mm	1076 kN
S2(0.19)	2400 mm	260 x 260 mm	2.30%	0.19%	811 kN	46.53 mm	1.94%	11.73 mm	715 kN
S3(0.58)	2400 mm	260 x 260 mm	2.30%	0.58%	406 kN	52.81 mm	2.20%	3.47 mm	621 kN
S4(0.19)	2400 mm	260 x 260 mm	2.30%	0.19%	406 kN	64.75 mm	2.69%	8.64 mm	491 kN

4

EXPERIMENTAL STUDY

4.1 Introduction

The experimental study phase was carried out in the Protective Engineering Laboratory at NTU to simulate blast effects on four column specimens. The lateral and axial loadings on the specimens were applied through hydraulically controlled actuators. The focus of this experimental study was to initially simulate the blast loads that are applied on a column by achieving its displacement profile obtained from the numerical study and subsequently determining its blast-damaged residual axial load carrying capacity. The laboratory experiments commenced only after the finite element analysis on all of the four column specimen models were completed. A testing frame was designed, developed, and set up in the laboratory with the aim of understanding the residual axial capacity of blast-damaged columns.

This chapter describes the specimen and loading frame details, the test setup, the loading sequence, and instrumentations used to record important parameters during the experimental study phase. A summary of the test results obtained from the experiments conducted are also reported herein.

4.2 Experiment Specimens

A total of four reinforced concrete column specimens were designed and fabricated for the purpose of this experimental study program. The four specimens are made up of two batches. Each batch has two similarly detailed specimens that were pre-loaded with two levels of axial loads. The batches differ from each other by the ratio of the transverse reinforcement detailing within each of their two specimens. The first batch has a higher transverse reinforcement ratio of 0.58 % and the second batch has a lower transverse

reinforcement ratio of 0.19 %. They are labeled *S1(0.58)*, *S2(0.19)*, *S3(0.58)* and *S4(0.19)* to distinguish the amount of transverse reinforcement each specimen is detailed with from this point on. The specimen labels are also synchronized with the labels of the numerical models that were created and discussed in the previous chapter. One of the objectives of this study is to determine the additional resistance that the increased transverse reinforcements within the column specimens are able to provide to reduce the effects of the blast loadings and thus increasing its blast-damaged residual axial capacity. Detailed specifications of the two specimens tested are provided in the following sub-sections.

4.2.1 Column Specimen Details

The column specimens are mounted between two subassemblies. They are the top head and a foundation that hold the column upright at its two ends. All the column specimens have cross-sectional dimensions of 260 x 260 mm and a vertical height of 2400 mm.

4.2.1.1 Specimen *S1(0.58)* & *S3(0.58)* Reinforcement Detail

Specimens *S1(0.58)* and *S3(0.58)* were designed based on the specifications provided by standard design codes. The column specimens are 2400 mm tall and are held upright by two subassemblies. The base subassembly stands 500 mm high while the head subassembly rises 750 mm above the column specimens. The column specimens have longitudinal reinforcements provided for by eight T16 bars and transverse reinforcement provided for by shear links formed with double hoops of R6 bars placed 100 mm apart along its height. The longitudinal reinforcement ratio provided is 2.3 % and the volumetric ratio of transverse reinforcement works out to 0.58 %. The closely spaced transverse reinforcement in Specimen is to increase the confinement of the concrete within its core and to provide additional seismic resistance as compared to the next batch of specimens. The aspect ratio of the column specimen is approximately 9. Typical geometry and section details of *S1(0.58)* and *S3(0.58)* are provided in **Figure 4.1**. Detailed drawings used during the

procurement of the contract for the casting of the Specimen are included in the Appendix of this report.

Figure 4.1 Typical geometry and section details of *S1(0.58)* & *S3(0.58)*

4.2.1.2 Specimen *S2(0.19)* & *S4(0.19)* Reinforcement Detail

Specimens *S2(0.19)* and *S4(0.19)* were designed based on the specifications provided by standard design codes. The column specimens are 2400 mm tall and are held upright by two subassemblies. The base subassembly stands 500 mm high while the head subassembly rises 750 mm above the column specimens. The column specimens have longitudinal reinforcements provided for by eight T16 bars and transverse reinforcement provided for

by shear links formed by R6 bars placed at 175 mm apart. The longitudinal reinforcement ratio provided is 2.38 % and the volumetric ratio of transverse reinforcement works out to 0.19 %. This batch of specimens have more widely spaced transverse reinforcements to serve as a comparison as to how much better the first batch of specimens with closely spaced transverse reinforcements would perform when subjected to lateral loads. It is believed that this poorer shear capacity of the specimens would eventually lead to lower residual axial capacity of the specimens once they have been deformed to the deflected shapes as prescribed from the numerical study. The aspect ratio of the column is approximately 9. The geometry and section details of *S2(0.19)* and *S4(0.19)* are provided in **Figure 4.2**. Detailed drawings used during the procurement of the contract for the casting of the Specimen are included in the Appendix of this report.

Figure 4.2 Typical geometry and section details of S2(0.19) & S4(0.19)

4.2.2 Column Specimen Materials

The specifications of the concrete and reinforcement bars used during to fabricate the specimens are provided in the following sub-sections.

4.2.2.1 Concrete Specifications

The four specimens were cast with ready-mix concrete that was specified to achieve a characteristic strength of not lower than 25 MPa within 28 days. The aggregates were specified to have a size of 13 mm and the concrete pour was required to produce a slump of

125 mm to ensure its workability. Twelve 150 x 300 mm test cylinders (three per individual specimen) were also cast and cured under the same conditions to make them representative of their respective column specimens. The cylinders were individually tested for their compressive strength, f'_c to determine the compressive strengths of the concrete used to cast the specimens. The results obtained from the cylinder compressive tests are tabulated in **Table 4.2**. These compressive strengths of the specimens were also utilized to carry out the numerical simulations reported in the previous chapter.

Table 4.1 Cylinder test compressive strength results

Batch	Specimen Tag	Cylinder 1 f'_c (MPa)	Cylinder 2 f'_c (MPa)	Cylinder 3 f'_c (MPa)	Average Compressive Strength f'_c (MPa)
1	S1(0.58)	27.83	34.75	32.71	31.76
2	S2(0.19)	35.27	30.85	31.76	32.63
1	S3(0.58)	32.61	32.73	34.68	33.34
2	S4(0.19)	30.34	34.80	32.53	32.56

4.2.2.2 Reinforcement Bar Specifications

There are in total two variations of steel bars that were used as reinforcement within all of the four specimens. High tensile strength steel bars of 16 mm diameter with nominal yield strength of 460 MPa made up the longitudinal reinforcement and mild steel bars of 6 mm diameter with nominal yield strength of 250 MPa provided transverse reinforcement the column specimens. The two batches of specimens had variations in terms of the amount of transverse reinforcement provided within them. The first batch had had double hoops formed from of R6 bars placed at 100 mm spacing. The second batch had single hoops formed from R6 bars placed 175 mm spacing.

4.3 Experiment Setup

The experiment setup required a loading frame that was capable of maintaining axial loads on these column specimens initially, which was followed by the application of lateral loads to enable the specimen to take up the deflected shape as were achieved from the numerical

study conducted on models of these specimens. Once the specimens have achieved the deflected shape as they would be damaged from a blast load, their axial loads would be gradually increased to determine the additional amount of residual axial capacity within them upon attaining this deflected form.

The test setup was designed to achieve the initially set research objectives. In the planning stage, considerations were given to various factors such as limitations of equipments, laboratory space constraint, assembly procedures, and safety issues. The setup comprises of a steel A-frame, plates, bolts and transfer beams, concrete spacer blocks and five hydraulically powered MTS actuators. A sketch and photograph of the test setup is illustrated in **Figure 4.3** and **Figure 4.4** respectively.

Figure 4.3 Test setup (elevation view)

Figure 4.4 Photograph of test setup

4.3.1 Axial Loading Frame

The pre-axial load was applied to the column via a loading frame. The loading frame was made up of a transfer beam that was attached to vertically mounted actuators. The actuators were mounted on reinforced concrete spacer blocks to provide it with sufficient stroke distance. The transfer beam allowed an even distribution of the axial load on to the column specimen. The transfer beam also simulates the stiffness of upper floors that provide end rotation fixity. **Figure 4.5** and **Figure 4.6** shows the front and rear views of the test setup, A-frame and the two vertically mounted actuators that apply the axial load on to the column specimen respectively.

Figure 4.5 Front view of test setup

Figure 4.6 Rear view of test setup

The foundation of the column was attached to the strong floor by high tensile strength steel rods. All these steel rods were pre-tensioned prior to commencing each of the experiments to ensure that neither the specimen nor the setup should rotate or slide while conducting the experiments. The initial pre-axial loads were applied to the column specimen through the two vertically mounted hydraulically powered actuators. **Figure 4.7** shows the location at the Protective Engineering Laboratory at which the setup was to be placed.

Figure 4.7 Location for commencing experiments

4.3.2 A-Frame

The top head of the column was attached to an A-frame. The purpose of the A-frame is to prevent the specimen from slipping. It also has three pin-connected transfer beams to prevent rotation of the column head. The A-frame was connected to eight I-beams that

were in turn mounted above two concrete spacer blocks, which were bolted down on to the strong floor. All the bolt connections to the strong floor had a pre-tension force applied onto them before the nuts were tightened to ensure no slippage of the specimen or the frame would occur when the experiment was being carried out. **Figure 4.8** shows a photograph of the A-frame and three pinned transfer beams while **Figure 4.9** shows the connection between the I-beams concrete spacer block and the strong floor.

Figure 4.8 A-frame and transfer beam in experiment setup

Figure 4.9 I-beams and concrete spacer block that support A-frame

The frame was attached to the top head of the column through a pair of steel plates at each end and three small transfer beams, which are pin-connected at both ends. An additional concrete spacer block placed above the specimen to provide it the additional height it required to reach the axial load transfer beam and the A-frame. **Figure 4.10** shows the connection between the column head, steel plates, concrete spacer block, and the three pin-jointed transfer beams.

Figure 4.10 Connection between column head, steel plates, concrete spacer block and transfer beams

4.3.3 Lateral Load Actuators

The hydraulically powered lateral load actuators were mounted on to steel plates that were bolted on the strong wall of the laboratory. These steel plates were fabricated especially so that the positions of the lateral load actuators would coincide as closely as possible with the locations that were measured from the numerical results. The actuators are powered by hydraulic fluids and would be controlled through a computer software in a displacement mode to ensure the deflected shape of the column specimens are similar to the blast-damaged deflected shape obtained from the numerical analysis. The top and bottom actuators had a 100-ton capacity while the middle actuator had a 50-ton capacity. **Figure 4.11** shows the steel plates that were mounted on the strong wall and the lateral load actuators that were bolted on to these steel plates. The alignment of each actuator was

checked thoroughly prior to commencement of the experiments to ensure the lateral loads were applied on to the column specimen in the correct angle.

Figure 4.11 Lateral actuators and strong wall mounting

4.3.4 Axial Load Actuators

The axial loading was applied on to the column specimens via two vertically mounted hydraulically powered actuators that were connected to the specimen by an axial load transfer beam. These vertically mounted actuators have each a 100-ton capacity. The actuators were mounted on concrete spacer blocks to enable them to be able to reach the axial load transfer beam with their available stroke distance. The connection between the concrete block and the vertically placed actuators was made possible through a steel plate that was bolted down to the strong floor along with the concrete block. All the holding down bolts used in the setup initially was stressed with a pre-tension force prior to tightening of its nuts. The top head of the vertically mounted actuator was connected to the

axial load transfer beam. The connection was made through four bolts. **Figure 4.12** shows all these connection details.

Figure 4.12 Vertically mounted actuator and connection details

4.4 Test Procedure

The complete experimental study phase of this research was conducted in the Protective Engineering Lab at NTU. The first stage of this experimental program involved assembling the entire loading frame, actuators and specimen as described in the previous sections. Upon setting up the entire test rig, the verticality of each element of the test rig was ensured

with spirit levels to uphold the level of accuracy of the experimental study. All the actuators were also checked for positioning and their alignment. The column specimen was white washed to allow better visualization of crack development patterns. Gridlines at 100 x 100 mm were drawn on the column specimen for crack size monitoring. Safety checks were also conducted on all the hydraulic hoses for leaks or any other defects. The cables were ensured to have been connected properly to prevent any disruptions during the actual testing. All the transducers and actuators were calibrated and zeroed into position with the specimens prior to commencement of the tests. The complete testing procedure comprised of three stages: Pre-Axial Load Application, Lateral Load Test and Axial Failure Test. The following sub-sections describe each of the stages.

4.4.1 Pre-Axial Loading Stage

The first stage involves applying the pre-axial loads on each of the column specimens. This pre-axial load was to simulate on to the specimen the service loadings an actual column would be subjected to in a frame structure. The two vertically mounted actuators would apply this pre-axial loading on to the column specimen via the axial load transfer beam. The loading was applied in a quasi-static manner with small increments at each time step. The actuators are triggered through a displacement-controlled mode. The loading that is applied on the specimen is monitored through a load cell that is in-built within the hydraulic actuators. Once the desired level of loading is achieved, the operator can stop the actuator from displacing any further. Two target level pre-axial loads are to be varied between each batch of specimen. They are $0.4f'_c A_g$ and $0.2f'_c A_g$. Once the required pre-axial load level is achieved for each batch of specimens, it was to be maintained at that level throughout the lateral loading process.

4.4.2 Lateral Loading Stage

Once the respective pre-axial loads to be applied on the column specimen from each batch have stabilized, the lateral loads were applied via the three horizontally mounted actuators

in order to achieve their respective blast-damaged displacement profile. The blast-damaged displacement profile is obtained from the numerical study conducted on models of each of the four column specimens and is presented in the previous chapter of this report. It is close to impossible to recreate the exact stresses and strains within the concrete and reinforcement of a column, as it would experience from actual explosive loads through a non-dynamic laboratory-testing scheme. Therefore, the main aim of this experimental phase is to obtain a laterally deflected shape that is as close as possible to the blast-damaged lateral displacement profile obtained from the numerical study. The laboratory setup adopted was similar to the tests carried out by Crawford *et al.* [C7]. The lateral load application actuators were set to a displacement control mode in order to achieve this blast-damaged deflection profile on the column specimens. The displacement profile of the column specimens were monitored through three LVDTs that were mounted at similar heights at which the lateral actuators were placed. The actuators in the laboratory applied the loading at a quasi-static loading rate. As it is the focus of this study to determine the blast-damaged residual axial capacity of columns, an important aim was to achieve a displacement profile as close as possible to the numerically determined blast-damaged column model profile.

4.4.3 Residual Axial Capacity Stage

Upon attaining the blast-damaged deflected profile on each of the column specimens, the lateral load actuators were individually retracted. The column is assumed to be in a similar damage level as when an actual blast loading would be applied on it. All the LVDTs are also removed for preserving the quality and accuracy of these instruments for future tests. The next phase in the experiment would be to determine the amount of residual axial capacity the column specimen possesses after this inflicted lateral damage. This would determine if an actual column in this damage profile state would be able carry the loadings of the upper floors in a real frame structure. The vertically mounted actuators are then activated via a displacement-controlled mode to determine the residual axial capacity of each of the column specimens. While the displacement levels of both these actuators are

controlled in-synch, the load cell within them would record the amount of resistance provided to them while doing so. This resistance is taken to be the axial capacity of the damaged column specimens. The experiment is halted either when this resistance force starts to drop drastically or when the force remains constant while the displacement increases rapidly. The column specimen is deemed to have reached its failure state when either one of these scenario occurs. The in-built data logger within the actuator operation software records all of the data during this entire process.

4.5 Instrumentation

A few devices were used to measure and record key data throughout the experimental program. These devices and instruments will provide the information that would quantify the experimental program. The data collected will form as a means of comparison of results between the experimental and numerical studies as well as the residual axial capacity prediction equation proposed by previously conducted researches. These comparisons are compiled within the following chapter of this report. The parameters that were required to be recorded from the experiments were narrowed down after the numerical study. The parameters obtained from this phase must be compatible with the data obtained from the numerical study to form a meaningful comparison between the two phases. The following sub-sections describe the instruments that were used during this study to record the required data.

4.5.1 Actuators

A five-actuator system was utilized for the experimental study. They consist of two 100-ton capacity vertical actuators, two 100-ton capacity horizontal actuators and along with another 50-ton capacity horizontal actuator. The 50-ton capacity actuator was placed in-between of the two 100-ton capacity horizontally mounted actuators. The vertically placed actuators would apply the axial loads while the horizontally placed actuators would simulate the lateral blast loads on to the column specimen. All of these actuators were

connected into a computer server and would advance or retract completely under its operator's command. The actuators have an in-built force and displacement sensor to detect the force and displacement levels at various stages during the experiment. The data-logging feature within the software that controls the actuators allows the user to specify the type and frequency of data to be recorded while the experiment is underway. The data that would be required from the actuators load and displacement sensors would be the loads applied by the vertically placed actuators and their corresponding displacements to signify the axial loads applied on to and the shortening of the column specimen.

4.5.2 Transducers

There were in total three types of transducers used for the experimental study program. They include wire transducers, 300 mm travel distance Linear Variable Differential Transducers (LVDT) and 100 mm travel distance LVDTs. The LVDTs were mounted on magnetic holders and positioned into the required height placements via a transducer stand. The following sub-sections give a brief description of each of these transducers and illustrate where they were placed to obtain data from the specimens.

4.5.2.1 Wire Transducers

The initial setup of the first specimen utilized three wire transducers. They were placed at heights corresponding to the positions of the lateral actuators. Their wires were extended out and connected to a steel rod that was embedded into the middle of the cross-section of each column specimen. The transducers measure displacement by monitoring the amount of wire that is displaced or retracted from its housing. This would provide the closest representation of the actual lateral displacement of the column specimens. This device was however only used to measure the lateral displacement from the first column specimen. Subsequently, it was found that the sensitivity of the wire that was used to measure the displacement was of hindrance to obtaining accurate data due to its relentless fluctuations. It was thus replaced with the 300 mm travel distance LVDT for the subsequent three

column specimens. Brief descriptions of these 300 mm travel distance LVDT is provided in the following sub-section. Pictures of these wire transducers as set up for the first column specimen are shown in **Figure 4.13**.

Figure 4.13 Wire transducers used for the first column specimen

4.5.2.2 100 mm LVDT

The 100 mm travel distance LVDT was used to determine the fixity of the column base. It was pertinent that the column specimen was firmly fixed to the strong floor. This was important as the lateral displacements measured from the column itself needs to be as accurate as possible to represent the deflected profile of the specimen. The 100 mm travel distance LVDT was attached to a magnetic stand and placed on a steel plate behind the column specimen in a direction parallel to the lateral actuators. The values read off this LVDT were constantly monitored during the commencement of every experiment. If any base slip was observed from the data, the experiment was temporarily halted and the steel bolts that attached the column specimens' base were tightened further to prevent any more slippage. However, there was no base slippage noticed from any of the four column specimens that were tested during this experimental phase. **Figure 4.14** shows pictures of the location of this transducer.

Figure 4.14 100 mm LVDT for measuring base slip

4.5.2.3 300 mm LVDT

After flaws were noticed from wire transducers that were used to monitor the lateral deflection of the column specimen from the first test, three 300 mm travel distance LVDTs were used in place of them. The problem caused by the wire transducers were due to the highly sensitive wires that were causing many fluctuations in the results that it recorded. This problem was rectified by replacing the wire transducers with 300 mm travel distance LVDTs. These LVDTs were attached on to a magnetic stand and mounted on a transducer stand to coincide with the vertical heights of the lateral load actuators. There were three steel rods embedded into the middle of the cross section of all the column specimens. The heights of these steel rods were at the locations of the lateral load actuators. A steel plate was bolted on to each of these steel rods. The displacement of this steel plate was taken to be representative of the column specimens' lateral displacement. The sturdy built and proper mounting of this instrument provided this study with more accurate data enabling the numerically obtained blast-damaged displacement profile to be achieved with greater

ease. **Figure 4.15** shows these 300 mm travel distance LVDTs and the steel plates that were bolted on to the column specimen.

Figure 4.15 300 mm LVDT for measuring lateral displacement

4.6 Experimental Results

A total of four column specimens were cast for this experimental study. They are labeled *S1(0.58)*, *S2(0.19)*, *S3(0.58)* and *S4(0.19)* to reflect the amount of transverse reinforcement that was embedded within the column specimens. *S1(0.58)* and *S2(0.19)* were subjected to a pre-axial load of $0.4f'_cA_g$ while *S3(0.58)* and *S4(0.19)* were subjected to $0.2f'_cA_g$. The experimental results reported include the crack development pattern, final displacement profile and residual axial capacity for each of the four specimens. Details of the abovementioned are provided and organized for each specimen in the following subsections.

4.6.1 Results of Specimen *SI(0.58)*

Specimen *SI(0.58)* has a longitudinal and transverse reinforcement ratio of 2.38 % and 0.58 % respectively. This specimen was initially subjected to a pre-axial loading of $0.4f_cA_g$ prior to the lateral loads being applied to attain its blast deflected profile. This works out to 811 kN. The target mid-span deflection to be achieved on this specimen was 28.3 mm or a drift ratio of 1.18 %. The mid-span deflections are based on the numerical simulations that were previously conducted and reported in the preceding chapter. The crack development pattern, final displacement profile and residual axial capacity of *SI(0.58)* are reported within the following sub-sections.

4.6.1.1 Crack Development Pattern of *SI(0.58)*

The pre-axial load of 811 kN was applied through a displacement-controlled mode via the vertically placed actuators. The displacements on both these actuators were adjusted harmoniously until a resistance force of close to the desired pre-axial load was recorded by the load cells within these actuators. Once the pre-axial load was maintained, the lateral actuators were controlled through a displacement-controlled mode as well to achieve the targeted displacement profile on this specimen. The crack development pattern that was observed during the experiment is sketched in **Figure 4.16**. Photographs of *SI(0.58)* upon being subjected to the lateral loads is presented in **Figure 4.17**.

Figure 4.16 Crack development pattern of $S1(0.58)$

4.6.1.2 Final Displacement Profile of $SI(0.58)$

Column specimen $SI(0.58)$ achieved a mid-span residual deflection of 30.14 mm. A plot of the experimental deflected profile of $SI(0.58)$ is provided in **Figure 4.18**. The final residual deflections observed at the locations of the three lateral actuators are also tabulated in **Table 4.3**. The pre-axial load of 811 kN was maintained by the vertical actuators while the column attained this deflected profile. The axial load history imposed on to this column specimen is presented in the next sub-section.

Figure 4.18 Experimental Residual Deflection of $SI(0.58)$

Table 4.2 Summary of experimental lateral displacements of $SI(0.58)$

Column Height (mm)	530	1200	1870
Residual Displacement (mm)	28.92	30.14	24.96

4.6.1.3 Residual Axial Capacity of *SI(0.58)*

The laterally mounted actuators were completely retracted once the desired deflected profile was achieved on column specimen *SI(0.58)*. The next stage in the loading process involved increasing the axial loads applied on the column specimen through the vertically mounted actuators to determine the amount of residual axial capacity that was available within this laterally displaced column specimen. All the instruments that were recording the data were removed to ensure that they would not be damaged in a case where the specimen completely fractured due to its failure. The load cells in-built within the vertically mounted actuators were still active in logging the applied axial loads and displacements that the specimen encountered due to these loads during this final loading phase. A plot of the applied axial loads against their respective vertical deformations of column specimen *SI(0.58)* is provided in **Figure 4.19**.

Figure 4.19 Experimental residual axial capacity of specimen *SI(0.58)*

It can be observed from **Figure 4.19** that the initial pre-axial load of 811 kN caused a vertical shortening of column specimen *SI(0.58)* of approximately 4.5 mm. The lateral loads were then applied on to the column specimen. This increased the vertical shortening to approximately 6.7 mm. In the final axial loading phase, the laterally damaged column

specimen was able to sustain a further axial loading of approximately 982 kN before it started to take a dip. The test was brought to a halt when the axial load that the column specimen could carry dropped to 424 kN. The column specimen had shortened by approximately 16 mm at that axial load.

4.6.2 Results of Specimen S2(0.19)

Specimen S2(0.19) has a longitudinal and transverse reinforcement ratio of 2.38 % and 0.19 % respectively. This specimen was initially subjected to a pre-axial loading of $0.4f'_cA_g$ prior to the lateral loads being applied to attain its blast deflected profile. This works out to 811 kN. The target mid-span deflection to be achieved on this specimen was 46.53 mm or a drift ratio of 1.94 %. The mid-span deflections are based on the numerical simulations that were previously conducted and reported in the preceding chapter. The crack development pattern, final displacement profile and residual axial capacity of S2(0.19) are reported within the following sub-sections.

4.6.2.1 Crack Development Pattern of S2(0.19)

The pre-axial load of 811 kN was applied through a displacement-controlled mode via the vertically placed actuators. The displacements on both these actuators were adjusted harmoniously until a resistance force of close to the desired pre-axial load was recorded by the load cells within these actuators. Once the pre-axial load was maintained, the lateral actuators were controlled through a displacement-controlled mode as well to achieve the targeted displacement profile on this specimen. The crack development pattern that was observed during the experiment is sketched in **Figure 4.20**. Photographs of S2(0.19) upon being subjected to the lateral loads is presented in **Figure 4.21**.

Figure 4.20 Crack development pattern of S2(0.19)

Figure 4.21 S2(0.19) Cracks from lateral loads

4.6.2.2 Final Displacement Profile of S2(0.19)

Column specimen S2(0.19) achieved a mid-span residual deflection of 41.23 mm. A plot of the experimental deflected profile of S2(0.19) is provided in **Figure 4.22**. The final residual deflections observed at the locations of the three lateral actuators are also tabulated in **Table 4.4**. This column specimen was unable to sustain the initially applied pre-axial load while the lateral actuators were trying to achieve this deflected profile. The axial load history imposed on to this column specimen is presented in the next sub-section.

Figure 4.22 Experimental Residual Deflection of S2(0.19)

Table 4.3 Summary of experimental lateral displacements of S2(0.19)

Column Height (mm)	530	1200	1870
Residual Displacement (mm)	44.68	41.23	34.96

4.6.2.3 Residual Axial Capacity of $S2(0.19)$

The laterally mounted actuators were completely retracted once the desired deflected profile was achieved on column specimen $S2(0.19)$. This column specimen was unable to sustain the initially applied pre-axial load while the lateral actuators were attempting to obtain its desired deflected shape. However, once the deflected shape was achieved, the axial loads were increased to determine whatever amount of residual capacity the specimen possessed. All the instruments that were recording the data were removed to ensure that they would not be damaged in a case where the specimen completely fractured due to its failure. The load cells in-built within the vertically mounted actuators were still active in logging the applied axial loads and displacements that the specimen encountered due to these loads during this final loading phase. A plot of the applied axial loads against their respective vertical deformations of column specimen $S2(0.19)$ is provided in **Figure 4.23**.

Figure 4.23 Experimental residual axial capacity of specimen $S2(0.19)$

It can be observed from **Figure 4.23** that the initial pre-axial load of 811 kN caused a vertical shortening of column specimen $S2(0.19)$ of approximately 5.9 mm. The lateral loads were then applied on to the column specimen. When the lateral actuators was almost about to achieve the desired deflection profile on this specimen there was a sudden drop of

the axial load the column was sustaining. This plummet in the axial load occurred at a vertical displacement of approximately 8.97 mm. The axial load at this point dropped to approximately 680 kN. Subsequently the lateral loading was stopped and the specimen was put through further axial load increments to determine if it possessed any residual axial capacity. The axial loading at this point increased to a peak value of approximately 771 kN and then failed to raise any further from that point. The experiment was brought to a halt when the axial load dropped further to 380 kN while the column specimen had shortened by almost 11 mm.

4.6.3 Results of Specimen S3(0.58)

Specimen S3(0.58) has a longitudinal and transverse reinforcement ratio of 2.38 % and 0.58 % respectively. This specimen was initially subjected to a pre-axial loading of $0.2f'_cA_g$ prior to the lateral loads being applied to attain its blast deflected profile. This works out to 406 kN. The target mid-span deflection to be achieved on this specimen was 52.81 mm or a drift ratio of 2.20 %. The mid-span deflections are based on the numerical simulations that were previously conducted and reported in the preceding chapter. The crack development pattern, final displacement profile and residual axial capacity of S3(0.58) are reported within the following sub-sections.

4.6.3.1 Crack Development Pattern of S3(0.58)

The pre-axial load of 406 kN was applied through a displacement-controlled mode via the vertically placed actuators. The displacements on both these actuators were adjusted harmoniously until a resistance force of close to the desired pre-axial load was recorded by the load cells within these actuators. Once the pre-axial load was maintained, the lateral actuators were controlled through a displacement-controlled mode as well to achieve the targeted displacement profile on this specimen. The crack development pattern that was observed during the experiment is sketched in **Figure 4.24**. Photographs of S3(0.58) upon being subjected to the lateral loads is presented in **Figure 4.25**.

Figure 4.24 Crack development pattern of S3(0.58)

Figure 4.25 S3(0.58) Cracks from lateral loads

4.6.3.2 Final Displacement Profile of S3(0.58)

Column specimen *S3(0.58)* achieved a mid-span residual deflection of 55.03 mm. A plot of the experimental deflected profile of *S3(0.58)* is provided in **Figure 4.26**. The final residual deflections observed at the locations of the three lateral actuators are also tabulated in **Table 4.5**. The pre-axial load of 406 kN was maintained by the vertical actuators while the column attained this deflected profile.

Figure 4.26 Experimental Residual Deflection of *S3(0.58)*

Table 4.4 Summary of experimental lateral displacements of *S3(0.58)*

Column Height (mm)	530	1200	1870
Residual Displacement (mm)	38.50	55.03	51.91

4.6.3.3 Residual Axial Capacity of S3(0.58)

The laterally mounted actuators were completely retracted once the desired deflected profile was achieved on column specimen S3(0.58). The next stage in the loading process involved increasing the axial loads applied on the column specimen through the vertically mounted actuators to determine the amount of residual axial capacity that was available within this laterally displaced column specimen. All the instruments that were recording the data were removed to ensure that they would not be damaged in a case where the specimen completely fractured due to its failure. The load cells in-built within the vertically mounted actuators were still active in logging the applied axial loads and displacements that the specimen encountered due to these loads during this final loading phase. A plot of the applied axial loads against their respective vertical deformations of column specimen S3(0.58) is provided in **Figure 4.27**.

Figure 4.27 Experimental residual axial capacity of specimen S3(0.58)

It can be observed from **Figure 4.27** that the initial pre-axial load of 406 kN caused a vertical shortening of column specimen S3(0.58) of approximately 3 mm. The lateral loads

were then applied on to the column specimen. This increased the vertical shortening to approximately 8.96 mm. In the final axial loading phase, the laterally damaged column specimen was able to sustain a further axial loading of approximately 535 kN before it started to take a dip. The test was brought to a halt when the axial load that the column specimen could carry dropped to 190 kN. The column specimen had shortened by approximately 14 mm at that axial load.

4.6.4 Results of Specimen *S4(0.19)*

Specimen *S4(0.19)* has a longitudinal and transverse reinforcement ratio of 2.38 % and 0.19 % respectively. This specimen was initially subjected to a pre-axial loading of $0.2f'_cA_g$ prior to the lateral loads being applied to attain its blast deflected profile. This works out to 406 kN. The target mid-span deflection to be achieved on this specimen was 64.75 mm or a drift ratio of 2.69 %. The mid-span deflections are based on the numerical simulations that were previously conducted and reported in the preceding chapter. The crack development pattern, final displacement profile and residual axial capacity of *S4(0.19)* are reported within the following sub-sections.

4.6.4.1 Crack Development Pattern of *S4(0.19)*

The pre-axial load of 406 kN was applied through a displacement-controlled mode via the vertically placed actuators. The displacements on both these actuators were adjusted harmoniously until a resistance force of close to the desired pre-axial load was recorded by the load cells within these actuators. Once the pre-axial load was maintained, the lateral actuators were controlled through a displacement-controlled mode as well to achieve the targeted displacement profile on this specimen. The crack development pattern that was observed during the experiment is sketched in **Figure 4.28**. Photographs of *S4(0.19)* upon being subjected to the lateral loads is presented in **Figure 4.29**.

Figure 4.28 Crack development pattern of S4(0.19)

4.6.4.2 Final Displacement Profile of $S4(0.19)$

Column specimen $S4(0.19)$ achieved a mid-span residual deflection of 61.17 mm. A plot of the experimental deflected profile of $S4(0.19)$ is provided in **Figure 4.30**. The final residual deflections observed at the locations of the three lateral actuators are also tabulated in **Table 4.6**. The pre-axial load of 406 kN was maintained by the vertical actuators while the column attained this deflected profile.

Figure 4.30 Experimental Residual Deflection of $S4(0.19)$

Table 4.5 Summary of experimental lateral displacements of $S4(0.19)$

Column Height (mm)	530	1200	1870
Residual Displacement (mm)	61.70	61.17	49.05

4.6.4.3 Residual Axial Capacity of *S4(0.19)*

The laterally mounted actuators were completely retracted once the desired deflected profile was achieved on column specimen *S4(0.19)*. The next stage in the loading process involved increasing the axial loads applied on the column specimen through the vertically mounted actuators to determine the amount of residual axial capacity that was available within this laterally displaced column specimen. All the instruments that were recording the data were removed to ensure that they would not be damaged in a case where the specimen completely fractured due to its failure. The load cells in-built within the vertically mounted actuators were still active in logging the applied axial loads and displacements that the specimen encountered due to these loads during this final loading phase. A plot of the applied axial loads against their respective vertical deformations of column specimen *S4(0.19)* is provided in **Figure 4.31**.

Figure 4.31 Experimental residual axial capacity of specimen *S4(0.19)*

It can be observed from **Figure 4.31** that the initial pre-axial load of 406 kN caused a vertical shortening of column specimen *S4(0.19)* of approximately 3.6 mm. The lateral loads were then applied on to the column specimen. This increased the vertical shortening to approximately 10.5 mm. In the final axial loading phase, the laterally damaged column

specimen was able to sustain a further axial loading of approximately 526 kN before it started to take a dip. The test was brought to a halt when the axial load that the column specimen could carry dropped to 280 kN. The column specimen had shortened by approximately 13.5 mm at that axial load.

4.7 Summary of Experimental Results

Table 4.7 provides a summary of the model specifications, pre-axial loadings, and the experimental results obtained from the study that has been conducted. It serves to provide a convenient source of comparing between the results that were obtained from this stage in the study. The validity of these experimental results will be determined by comparing the residual axial capacities that were obtained from the numerical study previously conducted and reported in the following chapter, as well as with the residual axial capacity prediction equation as proposed by Bao and Li [X1]. These results and comparisons will be presented in the following chapter of this report.

Table 4.6 Summary of experimental results

Batch	Model	Height	Cross-Sectional Dimension	Longitudinal Reinforcement Ratio	Transverse Reinforcement Ratio	Pre-Axial Load	Blast Residual Mid-Span X-Deflection	Drift Ratio	Blast-Damaged Residual Axial Capacity
1	S1(0.58)	2400 mm	260 x 260 mm	2.30%	0.58%	811 kN	30.14 mm	1.25%	982 kN
2	S2(0.19)	2400 mm	260 x 260 mm	2.30%	0.19%	811 kN	41.23 mm	1.72%	771 kN
1	S3(0.58)	2400 mm	260 x 260 mm	2.30%	0.58%	406 kN	55.03 mm	2.29%	535 kN
2	S4(0.19)	2400 mm	260 x 260 mm	2.30%	0.19%	406 kN	61.17 mm	2.55%	526 kN

5

SUMMARY OF NUMERICAL & EXPERIMENTAL RESULTS

5.1 Introduction

Numerical analysis through finite element simulations and laboratory-based experiments have been carried out on four column specimens and models to determine their residual axial capacity upon each achieving a lateral displacement profile to represent a blast-damaged state as was determined from the former analysis. The aim of this study was to determine the residual axial capacity of blast-damaged reinforced concrete columns while varying parameters such as pre-axial loading and transverse reinforcement ratio. This chapter compiles the results obtained from each of these study methods that were adopted and serves as a comparison between the results attained from both approaches. There is also a comparison made between the results obtained from the two approaches of this study with those of an equation capable of predicting the residual axial load capacity of blast-damaged reinforced concrete column. This prediction equation has been proposed through a previously conducted study. A brief description of this equation and the definitions of its various terms are given in the following sub-section. This is followed by a detailed examination of the results obtained from each phase of this study placed in contrast to the predictions from this equation.

5.2 Residual Axial Capacity Prediction Equation

In a parametric study that was previously conducted [X1], the significance of selected parameters that affected the residual axial strength of a blast-damaged reinforced concrete column was revealed. This formula was capable of providing a term, v , described as the ratio of the residual axial strength of a blast-damaged column. This ratio is defined by the following equation:

Chapter 5: Summary of Numerical & Experimental Results

$$v = \frac{(P_r - P_L)}{(P_{max} - P_L)} \quad (5.1)$$

where, P_r is the residual axial capacity of a blast-damaged column, P_L is the pre-axial load that the column is designed to sustain and P_{max} is the axial capacity of an undamaged column.

As such, when a column is undamaged, $P_r = P_{max}$ and the value of $v = 1$. Likewise, when the column has totally lost the ability to sustain its pre-axial load, $P_r = P_L$, the value of $v = 0$. This also refers to the ultimate state of the column.

The various parameters utilized in this study provided a means of determining this residual axial strength ratio through a multi-variable regression analysis. This ratio was proposed to be determined by its authors as:

$$v = [73.65\rho_v + 8.465\rho_g - 0.020879(L/b) + 0.104]e^{[89284.22\rho_v - 1308.64221\rho_g - 9.684203(L/b) - 382.12](y_r/L)(P_L/f'_c/A_g)} \quad (5.2)$$

where, ρ_v was defined as the transverse reinforcement ratio, ρ_g the longitudinal reinforcement ratio, L the length of the column, b its width, y_r its residual mid-span deflection due to the blast, P_L the pre-axial load, f'_c the compressive strength of the concrete utilized, and A_g the gross column cross-sectional area.

5.3 Specimen *SI(0.58)*

Specimen *SI(0.58)* has a vertical height of 2.4 m and a cross-sectional dimension of 260 x 260 mm. It is reinforced with a longitudinal and transverse reinforcement with a ratio of 2.38 % and 0.58 % respectively. This specimen was subjected to a pre-axial loading of $0.4f'_cA_g$ prior to imposing the model to the blast loadings as defined in the numerical analysis and the lateral loadings applied through the actuators on to the specimen during the laboratory experiment. This pre-axial loading works out to 811 kN based on the

Chapter 5: Summary of Numerical & Experimental Results

representative cylinder compressive tests carried out in the laboratory. The following subsections describe the various data obtained from the numerical and experimental phases carried out during this study and make a comparison of those with the predicted residual axial capacity of a reinforced concrete column of similar specifications that has been damaged up to this level based on the prediction equation that is presented in the previous section.

5.3.1 Lateral Displacements of *S1(0.58)*

The first phase in the study involved creating a finite element model of the specimen. This model was then subjected to a pre-axial loading to simulate the service loadings an actual column would be subjected to in a frame structure. A blast loading as calculated through the in-built CONWEP within LS-DYNA is then applied on to the column model. The lateral displacements achieved from this simulations is assumed to be the effects an actual column would be subjected to when exposed to a blast loading of the defined equivalent TNT charge weight placed at the specified standoff distance. These lateral displacements are then taken as the target displacement profile to be achieved on a column specimen in a controlled laboratory experiment. **Table 5.1** lists out the residual lateral displacements of specimen *S1(0.58)* obtained from the numerical and experimental stages of this study.

Table 5.1 Numerical and experimental residual lateral displacements of *S1(0.58)*

FEM Node Height (mm)	S1(0.58) Numerical Lateral Displacement (mm)	Lateral Actuator Height (mm)	S1(0.58) Experimental Lateral Displacement (mm)
600	18.54	530	28.92
1200	28.31	1200	30.14
1800	22.61	1870	24.96

These points are plot on to a graph to provide better visualization of the deflected profiles obtained from each stage of this study. This plot is provided in **Figure 5.1**. It can be observed from the plot that the experimental residual deflected profile had larger displacements at all the three regions monitored as compared to the numerical deflected

Chapter 5: Summary of Numerical & Experimental Results

profile. As the lateral actuators in the laboratory testing were controlled individually, it was extremely difficult to obtain a laterally displaced profile that was the same as obtained from the numerical simulations. However, the aim to achieve a mid-span displacement within 10% of that obtained by the model in the numerical simulation was achieved on this and all the models.

Figure 5.1 Numerical and experimental residual deflected profile of $S1(0.58)$

5.3.2 Residual Axial Capacity of $S1(0.58)$

The next stage in the loading involved applying an axial load to the laterally displaced column to determine the amount of residual axial capacity it still possesses. As it can be seen from the deflected profile that was obtained from the two phases of the study, the experimental column specimen had a larger deflection at all the three regions that were monitored as compared to the numerical model of $S1(0.58)$. This would logically cause the experimental column specimen to have a lower residual axial capacity than its

Chapter 5: Summary of Numerical & Experimental Results

numerical model counterpart due to the larger amount of damage inflicted upon it. The numerical and experimental axial load plot against the vertical displacement of the column is provided in **Figure 5.2**.

Figure 5.2 Numerical and experimental residual axial capacity of $S1(0.58)$

It can be seen from **Figure 5.2** that both the numerical and experimental studies resulted in almost similar residual axial capacities. The numerical residual axial capacity of model $S1(0.58)$ is approximately 1076 kN. In contrast, the experimental column specimen $S1(0.58)$ had a residual axial capacity of approximately 982 kN. However, the initial stiffness of the specimen while the pre-axial load was applied shows the numerical model to have a much greater stiffness. This could be due to the gap that was present in the experimental setup between the axial load transfer beam, concrete spacer block and the column specimen itself. In addition, controlling any one of the actuators caused a change in displacement on other actuators. This posed as a severe problem in attaining the exact displacements as obtained from the numerical simulation. The approximately 10 % reduction in the residual axial capacity of the experimental column specimen could be caused by the larger lateral displacement profile it adopted as compared to the numerical model.

5.3.3 Prediction of Numerical and Experimental Residual Axial Capacity of $S1(0.58)$

The equation proposed to predict the blast damaged residual axial capacity of columns is based on parameters such as longitudinal and transverse reinforcement ratios, column aspect ratio, residual mid-span deflection and the applied pre-axial loading. The ratio of residual axial strength is determined based on these parameters. This ratio can subsequently be used to determine the residual axial capacity of a blast-damaged column. The numerical model and experimental specimen of $S1(0.58)$ have a slightly different blast-damaged residual mid-span deflection. This difference in the residual mid-span deflection would produce two different predictions of residual axial capacities by utilizing the proposed equation. The bar chart in **Figure 5.3** shows the residual axial capacity obtained from the numerical model, experimental specimen and each of their predicted residual axial capacities based on their residual mid-span deflection from this equation.

Figure 5.3 Comparison of residual axial capacities of $S1(0.58)$

Chapter 5: Summary of Numerical & Experimental Results

It can be noted from the bar chart in **Figure 5.3** that the prediction equation has provided an over-estimate of the residual axial capacity of blast damaged reinforced concrete columns. The prediction for the numerically simulated model with a residual mid-span deflection of 28.31 mm was 1511 kN while the prediction for the specimen from the experimental study with a lateral residual mid-span deflection of 30.14 mm was 1515 kN. The predictions were almost 50 % more than the residual axial capacities that were recorded from the study in each phase. It is also to be noted that while all the other parameters were held constant, utilizing the equation for the experimental column specimen with the larger residual mid-span deflection resulted in a higher residual axial capacity as compared to the numerical column model with the lower residual mid-span deflection.

5.4 Specimen S2(0.19)

Specimen S2(0.19) has a vertical height of 2.4 m and a cross-sectional dimension of 260 x 260 mm. It is reinforced with a longitudinal and transverse reinforcement with a ratio of 2.38 % and 0.19 % respectively. This specimen was subjected to a pre-axial loading of $0.4f'_cA_g$ prior to imposing the model to the blast loadings as defined in the numerical analysis and the lateral loadings applied through the actuators on to the specimen during the laboratory experiment. This pre-axial loading works out to 811 kN based on the representative cylinder compressive tests carried out in the laboratory. The following sub-sections describe the various data obtained from the numerical and experimental phases carried out during this study and make a comparison of those with the predicted residual axial capacity of a reinforced concrete column of similar specifications that has been damaged up to this level based on the prediction equation that is presented in the previous section.

5.4.1 Lateral Displacements of S2(0.19)

The first phase in the study involved creating a finite element model of the specimen. This model was then subjected to a pre-axial loading to simulate the service loadings an

Chapter 5: Summary of Numerical & Experimental Results

actual column would be subjected to in a frame structure. A blast loading as calculated through the in-built CONWEP within LS-DYNA is then applied on to the column model. The lateral displacements achieved from this simulations is assumed to be the effects an actual column would be subjected to when exposed to a blast loading of the defined equivalent TNT charge weight placed at the specified standoff distance. These lateral displacements are then taken as the target displacement profile to be achieved on a column specimen in a controlled laboratory experiment. **Table 5.2** lists out the residual lateral displacements of specimen *S2(0.19)* obtained from the numerical and experimental stages of this study.

Table 5.2 Numerical and experimental residual lateral displacements of *S2(0.19)*

FEM Node Height (mm)	S2(0.19) Numerical Lateral Displacement (mm)	Lateral Actuator Height (mm)	S2(0.19) Experimental Lateral Displacement (mm)
600	36.55	530	44.68
1200	46.53	1200	41.23
1800	36.53	1870	34.96

These points are plot on to a graph to provide better visualization of the deflected profiles obtained from each stage of this study. This plot is provided in **Figure 5.4**. It can be observed from the plot that the experimental residual deflected profile had larger displacements at both the top and bottom regions but a lower mid-span residual deflection when compared to the numerical deflected profile. As the lateral actuators in the laboratory testing were controlled individually, it was extremely difficult to obtain a laterally displaced profile that was the same as obtained from the numerical simulations. However, the aim to achieve a mid-span displacement within 10% of that obtained by the model in the numerical simulation was achieved on this and all the models.

Figure 5.4 Numerical and experimental residual deflected profile of $S2(0.19)$

5.4.2 Residual Axial Capacity of $S2(0.19)$

The next stage in the loading involved applying an axial load to the laterally displaced column to determine the amount of residual axial capacity it still possesses. As it can be seen from the deflected profile that was obtained from the two phases of the study, the experimental column specimen had a smaller residual deflection only at its mid-span as compared to the numerical model of $S2(0.19)$. The numerical and experimental axial load plot against the vertical displacement of the column is provided in **Figure 5.5**.

Chapter 5: Summary of Numerical & Experimental Results

Figure 5.5 Numerical and experimental residual axial capacity of *S2(0.19)*

It can be seen from **Figure 5.5** that both the numerical model and the experimental column of specimen *S2(0.19)* failed while they were laterally loaded. This failure was probably caused by the lower transverse reinforcement within the specimen causing it to have a lesser shear resistance as compared to *S1(0.58)*. The numerical residual axial capacity of model *S2(0.19)* at its damaged state is approximately 715 kN. In contrast, the experimental residual axial capacity is approximately 771 kN. In addition, the initial stiffness of the specimen while the pre-axial load was applied shows the numerical model to have a much greater stiffness. This could be due to the gap that was present in the experimental setup between the axial load transfer beam, concrete spacer block and the column specimen itself. In addition, controlling any one of the actuators caused a change in displacement on other actuators. This posed as a severe problem in attaining the exact displacements as obtained from the numerical simulation. It is also evident from the analysis that both the numerical model and the experimental column specimen of *S2(0.19)* did not have sufficient residual axial capacity to sustain the pre-axial loading. As such, this column is deemed have failed when it attains this deflected profile while subjected to a pre-axial load of 40 % of its capacity.

5.4.3 Prediction of Numerical and Experimental Residual Axial Capacity of S2(0.19)

The equation proposed to predict the blast damaged residual axial capacity of columns is based on parameters such as longitudinal and transverse reinforcement ratios, column aspect ratio, residual mid-span deflection and the applied pre-axial loading. The ratio of residual axial strength is determined based on these parameters. This ratio can subsequently be used to determine the residual axial capacity of a blast-damaged column. The numerical model and experimental specimen of S2(0.19) have a slightly different blast-damaged residual mid-span deflection. This difference in the residual mid-span deflection would produce two different predictions of residual axial capacities by utilizing the proposed equation. The bar chart in **Figure 5.6** shows the residual axial capacity obtained from the numerical model, experimental specimen and each of their predicted residual axial capacities based on their residual mid-span deflection from this equation.

Figure 5.6 Comparison of residual axial capacities of S2(0.19)

Chapter 5: Summary of Numerical & Experimental Results

It can be noted from the bar chart in **Figure 5.6** that the prediction equation has provided a slight over-estimate of the residual axial capacity of blast damaged reinforced concrete columns. The prediction for the numerically simulated model with a residual mid-span deflection of 46.53 mm was 833 kN while the prediction for the specimen from the experimental study with a lateral residual mid-span deflection of 41.23 mm was 841 kN. The predictions were approximately 20 % more than the residual axial capacities that were recorded from the study in each phase. It is also to be noted that while all the other parameters were held constant, utilizing the equation for the experimental column specimen with the lower residual mid-span deflection resulted in a larger residual axial capacity as compared to the numerical column model with the larger residual mid-span deflection.

5.5 Specimen S3(0.58)

Specimen S3(0.58) has a vertical height of 2.4 m and a cross-sectional dimension of 260 x 260 mm. It is reinforced with a longitudinal and transverse reinforcement with a ratio of 2.38 % and 0.58 % respectively. This specimen was subjected to a pre-axial loading of $0.2f'_cA_g$ prior to imposing the model to the blast loadings as defined in the numerical analysis and the lateral loadings applied through the actuators on to the specimen during the laboratory experiment. This pre-axial loading works out to 406 kN based on the representative cylinder compressive tests carried out in the laboratory. The following sub-sections describe the various data obtained from the numerical and experimental phases carried out during this study and make a comparison of those with the predicted residual axial capacity of a reinforced concrete column of similar specifications that has been damaged up to this level based on the prediction equation that is presented in the previous section.

5.5.1 Lateral Displacements of S3(0.58)

The first phase in the study involved creating a finite element model of the specimen. This model was then subjected to a pre-axial loading to simulate the service loadings an

Chapter 5: Summary of Numerical & Experimental Results

actual column would be subjected to in a frame structure. A blast loading as calculated through the in-built CONWEP within LS-DYNA is then applied on to the column model. The lateral displacements achieved from this simulations is assumed to be the effects an actual column would be subjected to when exposed to a blast loading of the defined equivalent TNT charge weight placed at the specified standoff distance. These lateral displacements are then taken as the target displacement profile to be achieved on a column specimen in a controlled laboratory experiment. **Table 5.3** lists out the residual lateral displacements of specimen *S3(0.58)* obtained from the numerical and experimental stages of this study.

Table 5.3 Numerical and experimental residual lateral displacements of *S3(0.58)*

FEM Node Height (mm)	S3(0.58) Numerical Lateral Displacement (mm)	Lateral Actuator Height (mm)	S3(0.58) Experimental Lateral Displacement (mm)
600	32.53	530	38.50
1200	52.81	1200	55.03
1800	36.94	1870	51.91

These points are plot on to a graph to provide better visualization of the deflected profiles obtained from each stage of this study. This plot is provided in **Figure 5.7**. It can be observed from the plot that the experimental residual deflected profile had larger displacements at all the three regions monitored as compared to the numerical deflected profile. As the lateral actuators in the laboratory testing were controlled individually, it was extremely difficult to obtain a laterally displaced profile that was the same as obtained from the numerical simulations. However, the aim to achieve a mid-span displacement within 10% of that obtained by the model in the numerical simulation was achieved on this and all the models.

Figure 5.7 Numerical and experimental residual deflected profile of S3(0.58)

5.5.2 Residual Axial Capacity of S3(0.58)

The next stage in the loading involved applying an axial load to the laterally displaced column to determine the amount of residual axial capacity it still possesses. As it can be seen from the deflected profile that was obtained from the two phases of the study, the experimental column specimen had a larger deflection at all the three regions that were monitored as compared to the numerical model of S3(0.58). This would logically cause the experimental column specimen to have a lower residual axial capacity than its numerical model counterpart due to the comparatively larger amount of damage inflicted upon it. The numerical and experimental axial load plot against the vertical displacement of the column is provided in **Figure 5.8**.

Figure 5.8 Numerical and experimental residual axial capacity of S3(0.58)

It can be seen from **Figure 5.8** that both the numerical and experimental studies resulted in almost similar residual axial capacities. The numerical residual axial capacity of model S3(0.58) is approximately 621 kN. In contrast, the experimental column specimen S3(0.58) had a residual axial capacity of approximately 535 kN. The approximately 15 % reduction in the residual axial capacity of the experimental column specimen could be caused by the larger lateral displacement profile it adopted as compared to the numerical model.

5.5.3 Prediction of Numerical and Experimental Residual Axial Capacity of S3(0.58)

The equation proposed to predict the blast damaged residual axial capacity of columns is based on parameters such as longitudinal and transverse reinforcement ratios, column aspect ratio, residual mid-span deflection and the applied pre-axial loading. The ratio of residual axial strength is determined based on these parameters. This ratio can subsequently be used to determine the residual axial capacity of a blast-damaged column. The numerical model and experimental specimen of S3(0.58) have a slightly different blast-damaged residual mid-span deflection. This difference in the residual mid-span deflection would produce two different predictions of residual axial capacities by

Chapter 5: Summary of Numerical & Experimental Results

utilizing the proposed equation. The bar chart in **Figure 5.9** shows the residual axial capacity obtained from the numerical model, experimental specimen and each of their predicted residual axial capacities based on their residual mid-span deflection from this equation.

Figure 5.9 Comparison of residual axial capacities of S3(0.58)

It can be noted from the bar chart in **Figure 5.9** that the prediction equation has provided an over-estimate of the residual axial capacity of blast-damaged reinforced concrete columns. The prediction for the numerically simulated model with a residual mid-span deflection of 52.81 mm was 1334 kN while the prediction for the specimen from the experimental study with a lateral residual mid-span deflection of 55.03 mm was 1337 kN. The predictions were almost more than double the residual axial capacities that were recorded from the study in each phase. It is also to be noted that while all the other parameters were held constant, utilizing the equation for the experimental column specimen with the larger residual mid-span deflection resulted in a higher residual axial capacity as compared to the numerical column model with the lower residual mid-span deflection.

5.6 Specimen *S4(0.19)*

Specimen *S4(0.19)* has a vertical height of 2.4 m and a cross-sectional dimension of 260 x 260 mm. It is reinforced with a longitudinal and transverse reinforcement with a ratio of 2.38 % and 0.19 % respectively. This specimen was subjected to a pre-axial loading of $0.2f'_cA_g$ prior to imposing the model to the blast loadings as defined in the numerical analysis and the lateral loadings applied through the actuators on to the specimen during the laboratory experiment. This pre-axial loading works out to 406 kN based on the representative cylinder compressive tests carried out in the laboratory. The following sub-sections describe the various data obtained from the numerical and experimental phases carried out during this study and make a comparison of those with the predicted residual axial capacity of a reinforced concrete column of similar specifications that has been damaged up to this level based on the prediction equation that is presented in the previous section.

5.6.1 Lateral Displacements of *S4(0.19)*

The first phase in the study involved creating a finite element model of the specimen. This model was then subjected to a pre-axial loading to simulate the service loadings an actual column would be subjected to in a frame structure. A blast loading as calculated through the in-built CONWEP within LS-DYNA is then applied on to the column model. The lateral displacements achieved from this simulations is assumed to be the effects an actual column would be subjected to when exposed to a blast loading of the defined equivalent TNT charge weight placed at the specified standoff distance. These lateral displacements are then taken as the target displacement profile to be achieved on a column specimen in a controlled laboratory experiment. **Table 5.4** lists out the residual lateral displacements of specimen *S4(0.19)* obtained from the numerical and experimental stages of this study.

Chapter 5: Summary of Numerical & Experimental Results

Table 5.4 Numerical and experimental residual lateral displacements of S4(0.19)

FEM Node Height (mm)	S4(0.19) Numerical Lateral Displacement (mm)	Lateral Actuator Height (mm)	S4(0.19) Experimental Lateral Displacement (mm)
600	41.69	530	61.70
1200	64.75	1200	61.17
1800	49.52	1870	49.05

These points are plot on to a graph to provide better visualization of the deflected profiles obtained from each stage of this study. This plot is provided in **Figure 5.10**. It can be observed from the plot that the experimental residual deflected profile had larger displacements at the top and bottom regions monitored but a smaller deflection at the mid-span region as compared to the numerical deflected profile. As the lateral actuators in the laboratory testing were controlled individually, it was extremely difficult to obtain a laterally displaced profile that was the same as obtained from the numerical simulations. However, the aim to achieve a mid-span displacement within 10% of that obtained by the model in the numerical simulation was achieved on this and all the models.

Figure 5.10 Numerical and experimental residual deflected profile of *S4(0.19)*

5.6.2 Residual Axial Capacity of *S4(0.19)*

The next stage in the loading involved applying an axial load to the laterally displaced column to determine the amount of residual axial capacity it still possesses. As it can be seen from the deflected profile that was obtained from the two phases of the study, the experimental column specimen had a larger deflection at the top and bottom regions that were monitored but a slightly smaller displacement in the mid-span as compared to the numerical model of *S4(0.19)*. The numerical and experimental axial load plot against the vertical displacement of the column is provided in **Figure 5.11**.

Figure 5.11 Numerical and experimental residual axial capacity of *S4(0.19)*

It can be seen from **Figure 5.11** that both the numerical and experimental studies resulted in almost similar residual axial capacities. The numerical residual axial capacity of model *S4(0.19)* is approximately 491 kN. In contrast, the experimental column specimen *S4(0.19)* had a residual axial capacity of approximately 526 kN. However, the initial stiffness of the specimen while the pre-axial load was applied shows the numerical model to have a much greater stiffness. This could be due to the gap that was present in the experimental setup between the axial load transfer beam, concrete spacer block and the column specimen itself. In addition, controlling any one of the actuators caused a change in displacement on other actuators. This posed as a severe problem in attaining the exact displacements as obtained from the numerical simulation. The approximately 10 % increase in the residual axial capacity of the experimental column specimen could be caused by the different deflected profile it adopted as compared to the numerical model.

5.6.3 Prediction of Numerical and Experimental Residual Axial Capacity of *S4(0.19)*

The equation proposed to predict the blast damaged residual axial capacity of columns is based on parameters such as longitudinal and transverse reinforcement ratios, column aspect ratio, residual mid-span deflection and the applied pre-axial loading. The ratio of residual axial strength is determined based on these parameters. This ratio can

Chapter 5: Summary of Numerical & Experimental Results

subsequently be used to determine the residual axial capacity of a blast-damaged column. The numerical model and experimental specimen of *S4(0.19)* have a slightly different blast-damaged residual deflection profile. This difference in the residual mid-span deflection would produce two different predictions of residual axial capacities by utilizing the proposed equation. The bar chart in **Figure 5.12** shows the residual axial capacity obtained from the numerical model, experimental specimen and each of their predicted residual axial capacities based on their residual mid-span deflection from this equation.

Figure 5.12 Comparison of residual axial capacities of *S4(0.19)*

It can be noted from the bar chart in **Figure 5.12** that the prediction equation has provided a slight over-estimate of the residual axial capacity of blast-damaged reinforced concrete columns. The prediction for the numerically simulated model with a residual mid-span deflection of 64.75 mm was 472 kN while the prediction for the specimen from the experimental study with a lateral residual mid-span deflection of 61.17 mm was 479 kN. The predictions were approximately 10 % more than the residual axial capacities that were recorded from the study in each phase. It is also to be noted that while all the other

Chapter 5: Summary of Numerical & Experimental Results

parameters were held constant, utilizing the equation for the experimental column specimen with the lower residual mid-span deflection resulted in a higher residual axial capacity as compared to the numerical column model with the higher residual mid-span deflection.

5.7 Comparing Results of Various Specimens

The specimens were made up of two batches in general. Each batch had two similar specimens that were studied in two different phases. Namely, they are the numerical and experimental study phase. The results obtained from the two phases were eventually checked with each other and with an equation capable of predicting the residual axial capacity of blast-damaged reinforced concrete columns that was proposed from a previously conducted study. The following sub-sections compare the results obtained from each of the two batches of specimens that were created for this study to understand the effects of transverse reinforcement ratio and pre-axial loading on the residual axial capacity of blast-damaged reinforced concrete columns. It also contains an analysis carried out that is based on the current study conducted, to determine the accuracy of the equation that was proposed to determine the residual axial capacity of blast-damaged columns.

5.7.1 Lateral Displacements

The lateral displacements that damage the column model were caused by the simulated blast loading as was administered through the CONWEP software that is in-built within LS-DYNA. **Figure 5.13** show the deflected profile of all four column models and four column specimens.

Chapter 5: Summary of Numerical & Experimental Results

Figure 5.13 Summary of deflected profiles

It is evident from the numerical deflected profiles that the additional transverse reinforcements of $S1(0.58)$ and $S3(0.58)$ resulted in those specimens having a less extreme lateral deflection profile. Also, the reduced pre-axial loading in specimens $S3(0.58)$ and $S4(0.19)$ resulted in the column being less stiff globally. This reduced global stiffness of the column and produced a comparatively more extreme deflected profile in those specimens.

5.7.2 Residual Axial Capacity

The residual axial capacity of the column models and specimens were determined after they attained their respective lateral displacement profiles. The axial load was increased upon which, to determine the available axial load carrying capacity there was within each of the models and specimens. **Table 5.5** tabulates the amount of residual axial capacity that was within each of these numerical models and specimens.

Table 5.5 Summary of numerical and experimental residual axial capacities

Numerical Residual Axial Capacity				
Pre-Axial Load	$0.4f'_cA_g$		$0.2f'_cA_g$	
Model ID	<i>S1(0.58)</i>	<i>S2(0.19)</i>	<i>S3(0.58)</i>	<i>S4(0.19)</i>
Longitudinal Reinforcement (%)	2.38	2.38	2.38	2.38
Transverse Reinforcement (%)	0.58	0.19	0.58	0.19
Residual Axial Capacity (kN)	1076	715	621	491
Experimental Residual Axial Capacity				
Pre-Axial Load	$0.4f'_cA_g$		$0.2f'_cA_g$	
Model ID	<i>S1(0.58)</i>	<i>S2(0.19)</i>	<i>S3(0.58)</i>	<i>S4(0.19)</i>
Longitudinal Reinforcement (%)	2.38	2.38	2.38	2.38
Transverse Reinforcement (%)	0.58	0.19	0.58	0.19
Residual Axial Capacity (kN)	982	771	535	526

It can be understood from **Table 5.5** that all of the specimens except for *S2(0.19)* had sufficient residual axial capacity to sustain its pre-axial loading despite attaining its respective laterally deflected form. The lower transverse reinforcement ratio within this specimen caused it to have a lower shear resistance as compared to specimen *S1(0.58)*, which was also loaded with the same pre-axial load. The poor shear resistance in turn caused the column to have a severe deflected profile when damaged by the same blast wave that was imposed upon all specimens. This large deflected profile only allowed the specimen to sustain 715 kN from the numerical analysis and 771 kN from the experimental study. In contrast, specimen *S4(0.19)* with the same amount of transverse reinforcement as *S2(0.19)* was able to sustain its pre-axial load despite being displaced to a drift of almost 2.7 %. Thus, the 50 % reduction in pre-axial load manages to keep the column from failing and in a real-life scenario might prevent progressive collapse of the entire structure.

5.7.3 Predicting Residual Axial Capacity

A previously conducted study [X1] proposed an equation that was capable of predicting the blast-damaged residual axial capacity of reinforced concrete columns. The prediction was based on parameters such as longitudinal and transverse reinforcement ratios, column aspect ratio, residual mid-span deflection and the applied pre-axial loading. Detailed equations that were concluded from that study are presented in **Section 5.2** of this Chapter in the thesis. A bar chart showing the respective residual axial capacities of the four numerical models and experimental specimens, and their respective predicted residual axial capacities are presented in **Figure 5.14**.

Figure 5.14 Bar chart of residual axial capacities

It is evident from the bar chart in **Figure 5.14** that the prediction equation when utilized to predict the residual axial capacities of *S1(0.58)* and *S3(0.58)* produced a large over-estimate. This is due to the form of the equation not being able to analyze column specimens with higher transverse reinforcement ratios. It was observed from the study

Chapter 5: Summary of Numerical & Experimental Results

that the residual axial capacity produced through the prediction of the equation for columns with a transverse reinforcement ratio of 0.58 %, resulted in a residual axial capacity that increased as the mid-span residual deflection increased. This would not make very much logical sense because a column with very high mid-span damage from blast impact should have a lower residual axial capacity after being subjected to these lateral loads with all other parameters held constant. However, the equation was able to give logical results for specimens with a transverse reinforcement ratio of 0.19 %. It produced predictions of residual axial capacities for these columns that reduced as their respective mid-span lateral damage due to blast loads increased with all other parameters held constant. This is evident as the equation was able to predict the residual axial capacity of specimens and models of *S2(0.19)* and *S4(0.19)* with an accuracy of approximately 20 % as illustrated in the bar chart.

6

SUMMARY, CONCLUSIONS & RECOMMENDATIONS

6.1 Summary

Exterior columns are primary load-bearing elements in framed structures and are often susceptible to close-in explosions as intended by extreme terrorist organizations. Their response to direct blast loads can be deemed crucial to the overall performance of the structure they support. However, current knowledge on the evaluation of the residual axial capacity of blast-damaged reinforced concrete columns is rather limited.

This research study adopted two phases to determine the blast-damaged residual axial capacity of reinforced concrete columns. The accuracy of the results obtained from two phases, made up of the numerical modeling and simulation phase and the experimental study phase were checked with each other as well as with a blast-damaged residual axial capacity prediction equation that was proposed through a previously conducted study.

Blast loadings were initially simulated through CONWEP that is in-built within LS-DYNA and imposed upon finite element models of the four specimens that were fabricated for the experimental study phase. The column specimens comprised of two batches that were detailed with two variable transverse reinforcement ratios. Each batch was made up of two specimens and was subjected to one of two variable pre-axial loadings prior to imposing the lateral loadings upon them. The lateral displacements due to the simulated blast loadings were recorded at three locations along the face of the models. The laterally deflected shape of the model was then targeted to be achieved upon on the column specimens within a laboratory setting through hydraulically powered actuators. Once the respective laterally deflected profile was achieved on the specimens, they were axially loaded to determine the amount of residual axial capacity they possess despite the lateral damage inflicted upon them. The primary intention of the study was to

determine the effects of transverse reinforcement ratio and pre-axial loading on the residual axial capacity of these columns.

6.2 Conclusions

The following conclusions can be drawn based on the numerical and experimental study conducted:

- The effect of blast loadings on columns determined through the numerical analysis aids in providing a better understanding to the behaviour of the column element when subjected to these explosive loads.
- The results obtained indicate that a pre-axial loading of $0.2f'_cA_g$ that was applied upon models and specimens *S3(0.58)* and *S4(0.19)* prior to the lateral loadings resulted in those specimens having a less extreme lateral deflection profile as compared to models and specimens *S1(0.58)* and *S2(0.19)*. It can be concluded that the 50 % increase in pre-axial loading that was applied on to models and specimens *S1(0.58)* and *S2(0.19)* caused them to have an increase in stiffness that resulted in them attaining a smaller deflected profile. This is because increasing axial load increases the moment capacity and nominal shear strength of reinforced concrete columns. However, if the impulse and the corresponding displacement exceed certain critical values, the lateral displacement would increase tremendously with increasing axial load due to the P-Δ effect. It is also noted from this study that this effect is more prominent in columns with lower transverse reinforcement ratios.
- The additional transverse reinforcement within models and specimens *S1(0.58)* and *S3(0.58)* provided them with additional shear strength. This additional shear strength resulted in these models and specimens attaining a smaller lateral residual deflection profile as attained by models and specimens of *S2(0.19)* and *S4(0.19)*. Low transverse reinforcement, usually utilized in low seismicity regions is primarily designed with no special consideration for ductility demand. This usually causes the column to fail through a diagonal shear mode. Higher transverse reinforcement ratio provides additional restraint for the longitudinal bars and increased confinement for the core

Chapter 6: Summary, Conclusions & Recommendations

concrete. This improves the shear capacity of the column. The numerical results from all the models show that the increase in transverse reinforcement ratio significantly reduced the degree of direct blast-induced damage on columns.

- Comparing between the results obtained from models and specimens $S1(0.58)$ and $S3(0.58)$ shows that these columns had sufficient residual axial capacity to sustain their prescribed pre-axial loads. The increased transverse reinforcement within these models and specimens reduced the severity of the lateral displacement profile they attained due to the lateral loads. This less severe damaged state in turn provided them with a higher residual axial capacity.
- The lower transverse reinforcement within $S2(0.19)$ resulted in that model and specimen attaining a more severe lateral deflection profile. This damage profile resulted in those models and specimens being unable to sustain its prescribed pre-axial load. In contrast, similarly detailed $S4(0.19)$ had a lateral deflected profile even more severe than that attained by $S2(0.19)$. This was due to the lower pre-axial load that was prescribed on $S4(0.19)$. However, despite the more severe lateral deflected profile attained by $S4(0.19)$, this column had sufficient capacity to sustain its pre-axial loading.

6.3 Observations & Recommendations

The following are the observations and recommendations made by the author from the study conducted:

- It was observed that columns designed to sustain a pre-axial loading of $0.4f'_cA_g$ would require them to have increased transverse reinforcement to enable them to sustain the prescribed pre-axial loading when subjected to impulsive loadings. The increase in transverse reinforcement ratio would provide the column with additional shear strength and control the amount of lateral deflection it would attain when subjected to impulsive loadings.
- In contrast, it was noted from the results obtained that prescribing a column to a reduced pre-axial loading of $0.2f'_cA_g$ would leave it with sufficient residual axial capacity to sustain this loading despite of the more severe lateral deflection profile it would adopt when subjected to the blast loads.

Chapter 6: Summary, Conclusions & Recommendations

- Checking the blast-damaged residual axial capacity of reinforced concrete column equation that was proposed by a previously conducted study, has surfaced some flaws embedded within it. The equation is able to predict with decent accuracy for columns with low transverse reinforcement. However, it provides a rather large over-estimate of the residual axial capacity of columns with higher transverse reinforcement ratio. The applicability bandwidth of this equation has to be refined further through future studies.
- More numerical and experimental works can be carried out on this topic while varying individual parameters to gain a deeper understanding on their individual effects. Some of these parameters include varying transverse reinforcement spacing as illustrated in **Figure 6.1** and varying the explosion location, which could result in plastic hinge formation below the mid-height of the column as illustrated in **Figure 6.2**.

Figure 6.1 Varied transverse reinforcement detailing

Figure 6.2 Non-uniform blast loading

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APPENDIX A – SPECIMEN DRAWINGS

Appendix A

Appendix A

APPENDIX B – SETUP DRAWINGS

Appendix B

Appendix B

Appendix C