

VULNERABILITY OF REINFORCED CONCRETE COLUMNS TO EXTERNAL BLAST LOADING

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Abstract

Reinforced concrete columns are essential elements that are responsible for overall strength and stability of structures. Loss of a column within a frame can cause progressive collapse. While some research has been conducted on blast performance of reinforced columns, primarily under far-field explosions, very limited work exists on the effects of close-in explosions. Dynamic response of concrete columns, in multi storey building, was investigated under close-in blast loads numerically, using FEM software LS-DYNA. A six-storey reinforced concrete building was selected for this purpose. Different standoff distance/charge mass combinations were used to investigate the failure modes of external building columns. Three different charge masses were used; i) backpack bomb having 22.67 kg (50 lbs) of TNT, ii) compact sedan car bomb with 227 kg (500 lbs) of TNT and iii) sedan car bomb with 454 kg (1000 lbs) of TNT. The explosives were placed at different distances relatively close to the structure, triggering different failure modes. Effects of transverse reinforcement and column location (edge versus corner column) were studied under different combinations of charge weight and standoff distance.

Column response under dynamic blast load was identified as either local or global. The results show that the failure mode with backpack bombs located at small standoff distance is either local breaching or concrete scabbing. Direct shear failure occurred at column supports when higher charge masses were detonated at close distances. As the standoff distance increased the response changed from breaching or direct shear to diagonal tension and flexure. The column transverse reinforcement played a major role in controlling diagonal shear cracks and promoting flexural response. Hence, the amount and spacing of transverse reinforcement were observed to be important design parameters.

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List of Abbreviations

<u>Symbols</u>	<u>Definition</u>
$A_{e_{max}}$	Area of the largest side of the solid element
a	Stability factor
b	Waveform parameter
c	Wave speed
$C \ \& \ P$	Cowper-Symonds constants
$C_{r\alpha}$	Reflected Pressure Coefficient
E	Young's modulus
F_c	Hardening cap
f_c	Dynamic concrete compressive strength
f_{cs}	Static concrete compressive strength
f_{dy}	Dynamic steel yield strength
f_t	Dynamic concrete tensile strength
f_{ts}	Static concrete tensile strength
f_y	Static steel yield strength
H_{EXP}^d	Heat of detonation of explosive
H_{TNT}^d	Heat of detonation of TNT
I^+	Positive specific impulse
I^-	Negative specific impulse
J_1	First invariants of the deviatoric stress tensors
J_2'	Second invariants of the deviatoric stress tensors
J_3'	Third invariants of the deviatoric stress tensors
K_L	Load factor
K_{LM}	Ratio of mass factor and load factor
K_M	Mass factor
L	Length
m	Mass
N	Number of elements

P	Pressure
P_o	Ambient atmospheric pressure
P_r	Reflected pressure
P_{so}	Incident pressure
R	Standoff distance
S_{ij}	Deviatoric stress tensors
t_a	Arrival time
t_d^+	Positive time duration
t_d^-	Negative time duration
u_i	Displacement, velocity and acceleration
$\dot{u}_i$	Velocity
$\ddot{u}_i$	Acceleration
u_{i+1}	Displacement at next time step
u_{i-1}	Displacement at previous time step
V_e	Volume of the solid element
W	Explosive equivalent weight of TNT
W_E	Effective explosive weight (TNT)
W_{EXP}	Explosive weight
Z	Scaled standoff distance
$\alpha_2, \beta_2, \lambda_2 \text{ \& } \theta_2$	computed by conducting triaxial tension test
θ	angle of incidence
$\emptyset(x)$	Shape function
$\dot{\epsilon}$	Strain rate
κ	Hardening cap parameter
R	Rubin three invariant reduction factor
Δt	Time increment
Δt_e	Critical time step
ρ	Specific density
ν	Poisson's ratio

<u>Acronym</u>	<u>Definition</u>
ALE	Arbitrary Lagrange Euler
ANFO	Ammonium Nitrite Fuel Oil
C4	Composition 4
CEB	Comité Européen du Béton
CFD	Computational Fluid Dynamic
CONWEP	Conventional Weapons Effects Program
CSA	Cement Association of Canada
CSCM	Continuous Surface Cap Model
DIF	Dynamic Increase Factor
FEM	Finite Element Method
LOP	Level of Performance
LSTC	Livermore Software Technology Corporation
MDOF	Multiple Degree of Freedom
NBCC	National Building Code of Canada
SDOF	Single Degree of Freedom

1 Chapter 1

Introduction

1.1 General

Blast proofing is becoming more essential in high security buildings due to safety measures against maliciously intended terrorist attacks. Designing embassies and some important government buildings may be classified as post-disaster and/or high security buildings and may have to be designed against blast loads. Hazards caused by blast loads that may lead to loss of lives and injuries include the effects of high air pressure associated with blast shock waves, fragmentation and debris impact and fire due to the fireball effect of explosives. Structural and non-structural damage may also interfere with relief and response efforts, potentially blocking safety routes and preventing evacuation.

The main objective of designing protective structures is to provide safe buildings against blast loads, to protect building occupants, as well as valuable assets in and around the buildings. The analysis and design of such buildings require a thorough understanding of the effects and behavior of blast loads, blast phenomena, and dynamic response of structures. Reinforced concrete columns, located at the exterior of building, forming part of the façade are especially vulnerable to external attacks and are essential gravity load carrying elements, failure of which may lead to progressive collapse. Figure 1-1 illustrates the Murrah Federal Building in Oklahoma City that suffered partial collapse under blast loads in 1995. This nine-storey reinforced concrete frame building experienced damage to its front façade and experienced progressive collapse because of the loss of four of the first storey columns. Column failure was caused by extremely high blast pressure generated by a truck bomb, carrying 1814 kg of ammonium nitrite fuel oil (ANFO) detonated at a very small distance from the building exterior. The results from the explosion were devastating as 168 people were killed.

Different size bombs placed at different distances relative to the target structure result in different structural response and failure modes. Limited research has been conducted on reinforced concrete

column response under blast loads where the explosives were placed at close distance to the structure. More research is needed to establish blast risk for reinforced concrete columns in building façade, subjected to different bomb sizes located at different locations.

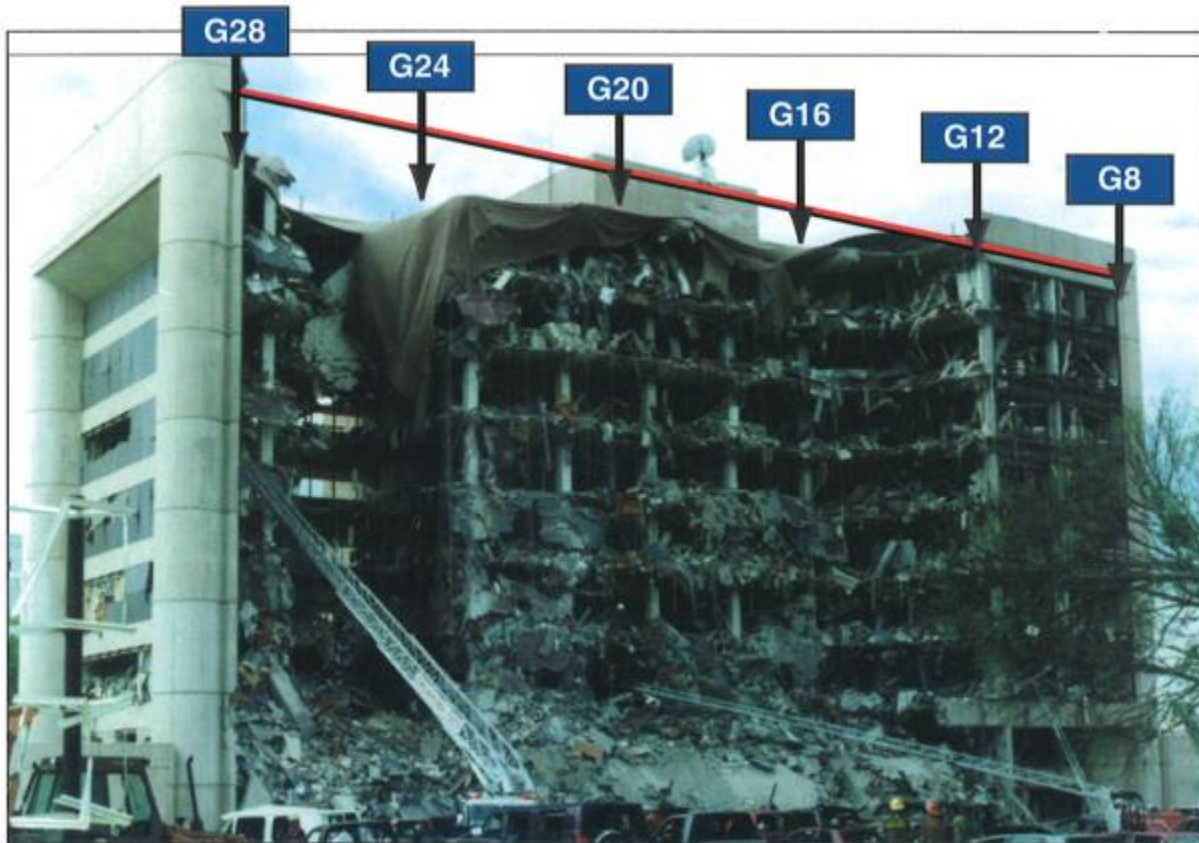


Figure 1-1: Oklahoma City Bombing 1995 (FEMA 227 1996)

1.2 Objectives and Scope

The objective of this research project is to assess blast risk and behaviour of reinforced concrete columns subjected to different bomb threat scenarios. The main focus is placed on identifying failure modes, damage levels and the effects of design parameters due to loads generated by car or backpack bombs that are relatively close to the building.

The following forms the scope of the project:

- Review the literature on blast loads and response of reinforced concrete columns to blast loading.
- Select computer software for dynamic nonlinear analysis.
- Develop an analytical model and verify the model against published test results.
- Select a building and assess its behavior under different charge sizes and standoff distances.
- Investigate the effects of concrete confinement on column response to blast loading.
- Summarize results and identify failure modes of reinforced concrete columns under different scenario cases with emphasis on the effects of close-in explosives.

1.3 Thesis Structure

This thesis consists of six chapters:

- Chapter 1: Presents Introduction, objective and scope of the research project.
- Chapter 2: Reviews the literature on explosives, blast load parameters, methods for predicting blast loading and structural response. The literature review includes those on previous experimental and analytical research conducted on reinforced concrete columns subjected to blast loads.
- Chapter 3: Provides background on LS-DYNA software elected for use in the current research project by presenting material models and the methods used to simulate blast loading. Moreover, the validation of model performance is compared with experimental data in the literature.
- Chapter 4: Presents the details of the selected building and the threats used to identify column response under blast loading.
- Chapter 5: Presents the results obtained using the numerical model. The effects of transverse reinforcement spacing and column location (edge versus corner column) on structural response, as well as different blast scenarios consisting of different combinations of charge weight and standoff distance are presented.
- Chapter 6: Provides conclusions and recommendations for future research.

2 Chapter 2

Literature Review

2.1 Introduction

This chapter provides a discussion on previous literature, covering the estimation of blast loads, effects of blast loads on structures, structural and material behaviour under blast loads, and possible failure modes. The emphasis is placed on vulnerability of reinforced concrete columns as part of the building façade under different threat scenarios. The literature review also presents previous experimental and numerical research on performance of reinforced concrete columns under blast loads.

2.2 Explosives

2.2.1 Explosion

An explosion is a large and sudden release of energy. Explosives come in forms of solids, liquids, or gases in normal state. It is also categorized as being physical, chemical or nuclear. Depending on how explosives are sensitive to ignition, two main types of explosives are found: primary and secondary (Ngo et al. 2007).

Primary explosives are directly ignited by a spark, flame, or impact. Secondary explosives, when detonated, create what is known as a blast (shock wave). The term “detonated” describes a very quick and stable chemical reaction. The chemical reaction converts the liquid, solid or gas explosives into a very hot, dense, and high-pressure gas. The volume containing this reaction holds massive energy that forces the volume to expand, hence, forcing out the energy at a very high velocity and creating blast waves. The results from secondary explosives are much more overwhelming. Examples of secondary explosives are trinitrotoluene (TNT) and ammonium nitrite fuel oil (ANFO). Depending on the weight of the explosive, the location of the explosive source relative to the structure and the interaction of the blast wave with the structure, blast waves can carry an enormous pressure ranging 2,700 to 4,900 ksi (UFC 03-0340-02 2008).

2.2.2 Explosive Sources

In terrorist attacks, bombing is considered to be the most desired activity of an attack. Explosive devices are associated with bombings and comes in the form of suicide bombs, package bombs and vehicular bombs. The quantity of explosive being used in a bombing attack is difficult to establish. The capacity of an explosive device is usually used to determine the mass of the explosive. According to the National Counterterrorism Center (NCTC), suicide bomber, package and vehicular devices are divided and categorized as shown in Figure 2-1. The explosive mass capacities presented in the figure are TNT equivalent.

Threat Description	Explosives Capacity ¹ (TNT Equivalent)	Building Evacuation Distance ²	Outdoor Evacuation Distance ³
 Pipe Bomb	5 LBS/ 2.3 KG	70 FT/ 21 M	850 FT/ 259 M
 Briefcase/ Suitcase Bomb	50 LBS/ 23 KG	150 FT/ 46 M	1,850 FT/ 564 M
 Compact Sedan	500 LBS/ 227 KG	320 FT/ 98 M	1,500 FT/ 457 M
 Sedan	1,000 LBS/ 454 KG	400 FT/ 122 M	1,750 FT/ 533 M
 Passenger/ Cargo Van	4,000 LBS/ 1,814 KG	600 FT/ 183 M	2,750 FT/ 838 M
 Small Moving Van/ Delivery Truck	10,000 LBS/ 4,536 KG	860 FT/ 262 M	3,750 FT/ 1,143 M
 Moving Van/ Water Truck	30,000 LBS/ 13,608 KG	1,240 FT/ 378 M	6,500 FT/ 1,981 M
 Semi-Trailer	60,000 LBS/ 27,216 KG	1,500 FT/ 457 M	7,000 FT/ 2,134 M

Figure 2-1: Bomb Threats According the National Counterterrorism Center (NCTC 2006)

2.3 Blast Loads

2.3.1 Blast Loads-Structure Interaction

Explosions and blast loads acting upon structures can be divided into two types: confined explosions and unconfined explosions. Confined explosion occurs within a building or a vessel, while, unconfined explosion occurs out in the open space (UFC 03-0340-02 2008).

Unconfined explosions can be subdivided into free air burst explosion, air burst explosion and surface burst explosion. Free air burst explosion takes place in the free air where the blast waves, propagating from the center of detonation with a spherical shape as shown in Figure 2-2 (a), move towards the structure without any interference that would amplify the blast loads.

Air burst explosions take place in free air at a distance close to the ground surface. This means that the blast loads generated from blast shock waves acting on the structure would get amplified after hitting the ground surface as shown in Figure 2-2 (b).

Finally, surface burst explosion takes place when the detonation is close to or at the ground surface level. This indicates that blast loads would get amplified after hitting the ground, and the blast waves move towards the structure with a hemispherical shape as shown in Figure 2-2 (c).

2.3.2 Blast Wave Phenomena

As described earlier, blast waves are the result of an enormous and instantaneous energy due to an explosion. Those waves expand outward from the centre of explosion and travel into the atmosphere in radial form. There are two major types of pressures that form due to an explosion. These pressures have a direct effect on structures: incident pressure (P_{so}) and reflected pressure (P_r), as shown in Figure 2-3.

Blast waves are also described as shock waves because of sudden extreme change in pressure and velocity. This change in pressure generates a diminishing shock velocity extending beyond the sonic velocity in the medium. Moreover, an enormous pressure reaching levels higher than the ambient atmospheric pressure (P_o) is formed, also known as incident pressure.

The reflected pressure is generated when shock waves impact a structure and reflect from the surrounding structures in a complex pattern. The time at which a shock wave reaches a structure is known as arrival time (t_a). The magnitude of the reflected pressure depends on the magnitude of the shock wave, the reflected pressure coefficient ($C_{r\alpha}$) and the angle of incidence on a structure. When the incident wave hits a structure on a normal line, the incident pressure gets amplified by a considerable amount and the reflected pressure becomes maximum. This maximum

reflected pressure can be used in design for each specific element of the structure (UFC 03-0340-02 2008).

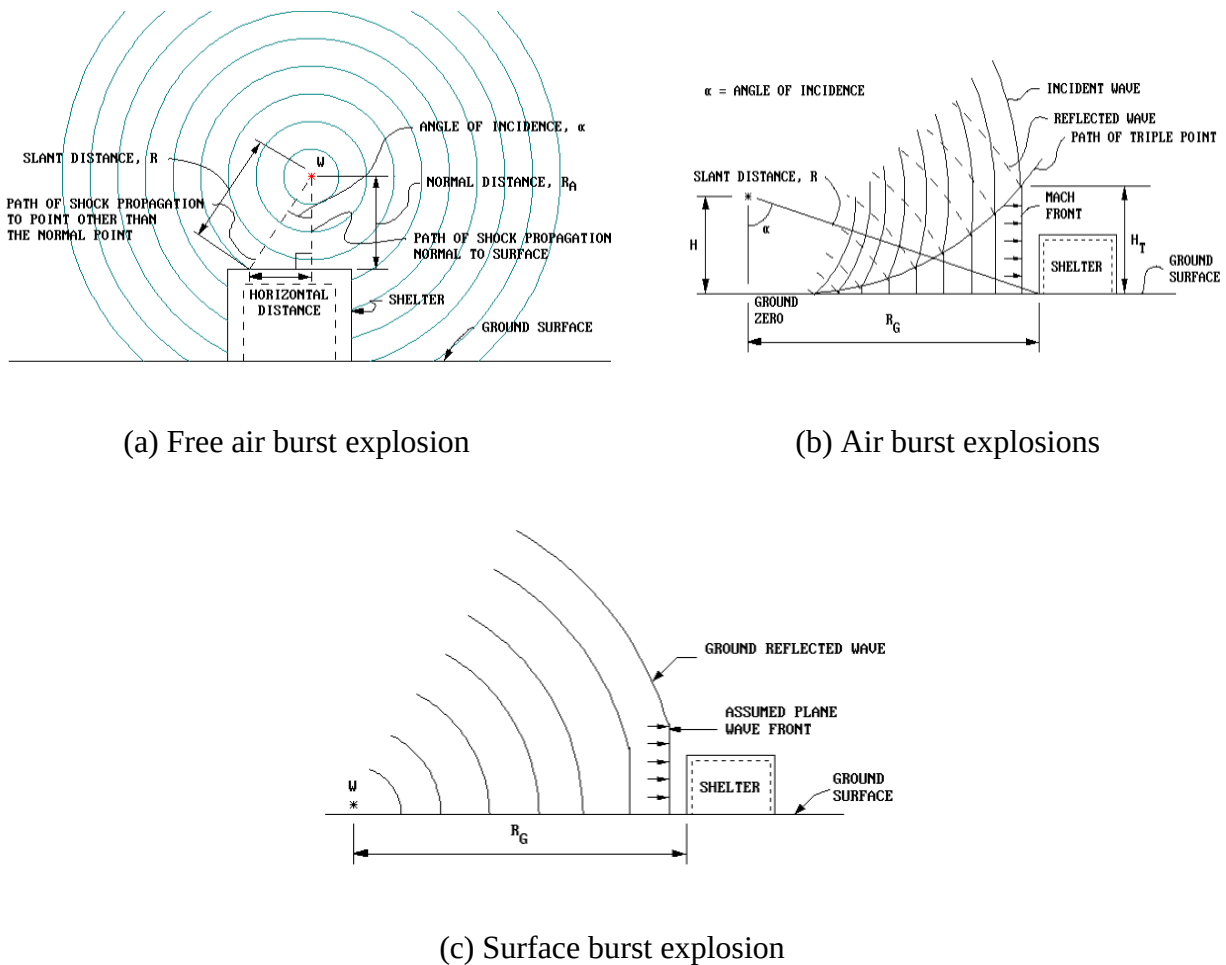


Figure 2-2: External Explosions (UFC 03-0340-02 2008)

The effect of a blast can be categorized over two durations: positive and negative. The positive pressure occurs when the incident pressure acting upon the structure is higher than the ambient atmospheric pressure. The period of time in which the incident pressure is above the ambient pressure is referred to as the positive time duration (t_d^+). The area under the positive duration of blast loading is identified as positive specific impulse (I^+). During that period, the blast waves are very strong and destructive for they move in small volume and carry very high pressures. Pressure in positive duration is of significant importance and it is used in load calculations for structural design against blast.

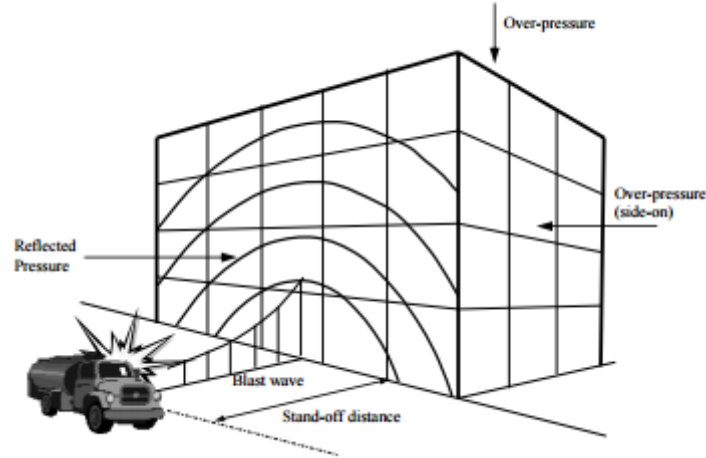


Figure 2-3: Pressures Generated from an Explosion (Ngo et al. 2007)

The negative pressure occurs when the incident pressure acting upon the structure is less than the ambient atmospheric pressure. The negative duration pressure is far less severe than the positive duration pressure. However, the damaged structure may be further damaged due to the debris carried during the negative time duration (t_d^-) (Ngo et al. 2007). The area under the negative duration of blast loading is identified as negative specific impulse (I^-).

Figure 2-4 illustrates a typical pressure time history for blast pressure. The pressure time history can be described using Friedlander equation as shown in Equation 2.1 (Smith and Hetherington 1994):

$$P(t) = P_{so} \left(1 - \frac{t}{t_d^+} \right) e^{\frac{-bt}{t_d^+}} \quad (2.1)$$

Where b is a non dimensionless number known as the waveform parameter and can be obtained from a table provided by Kinney and Graham (1985). Another way to calculate the waveform parameter is by equating the area under the positive duration of the loading (which is the integral of equation 2.1) to the positive specific impulse and solving for b .

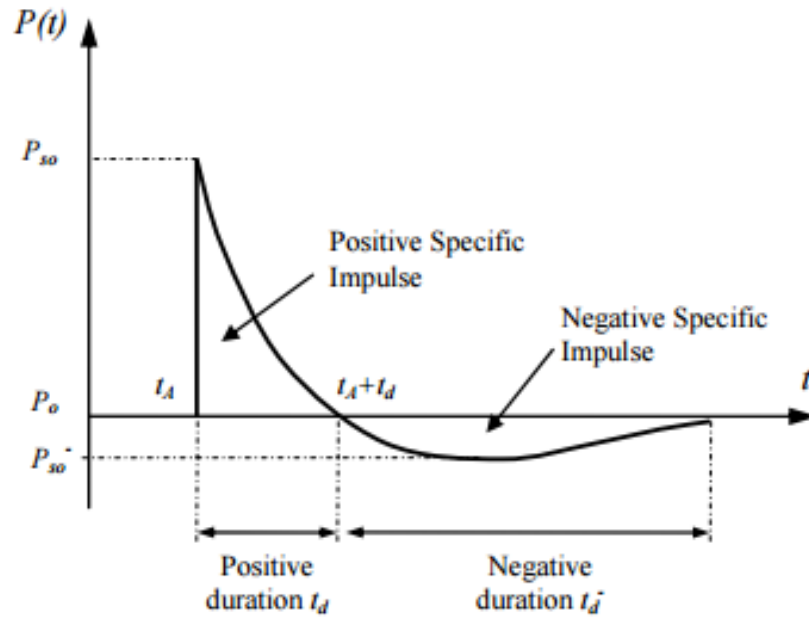


Figure 2-4: Typical Pressure Time History for Blast Loads (Ngo et al. 2007)

2.3.3 Factors Affecting Blast Pressure

Once an explosion takes place, the blast waves start travelling in every direction and have direct effect on every object on its path. Several factors contribute to the magnitude and distribution of blast loads. These include:

- Shape and type of explosive, detonation speed, and travel distance between the explosive and structure under blast loading.
- The clarity of the path that the shock waves travel through (obstacles change shockwave behaviour).
- The geometrical shape and elevation of the structure.
- Open versus closed structures

The above factors can significantly affect the intensity of blast. For instance, buildings with openings allow for incident pressure to propagate inside of the structure and that will create high reflected pressure between the interior walls. Also, the geometrical shape will change the reflection of shock wave line, due to the change in reflection angle (UFC 03-0340-02 2008). Finally, the distance between the point of blast and the structure will control the intensity of shock wave.

Distant explosions also known as far-field explosions produce uniform pressure on the target structure. The shock wave developing from explosives becomes more planer and uniform as distance increases. Explosion near a structure, which is known as close-in explosion, produces high non uniform pressures on the target structure. The shock wave is non planer and very complex to predict. Limited research was conducted on close-in effects of explosions on structures even though it causes more damage than far-field explosions. In cases where the standoff distance around a building is not restricted, terrorists place the explosives as close as possible to the building. The impact of shock waves on a structure generates extremely high reflected pressures, causing localized failures. This may affect the global behaviour of the structure.

2.3.4 Scaling Blast Wave Parameters

In order to compare different explosive types with different masses detonated at different locations, blast wave parameters such as pressure, impulse and duration are typically presented in a scaled form. The most common method used to scale explosions is the Hopkinson-Cranz scaling law or the cube root scaling method. This method is presented in UFC 03-0340-02. Fundamentally, the law implies that; when two different weights of the same explosive type are detonated in the same atmospheric conditions, same peak reflected pressures are produced by both explosives at the same scaled distance (Baker et al. 1991).

The dimensional scaled distance, Z is computed using Equation 2.2:

$$Z = \frac{R}{W^{1/3}} \quad (2.2)$$

Where, R is the standoff distance measured from the centre of explosion to the target structure, and W is the explosive weight expressed in terms of equivalent weight of Trinitrotoluene (TNT). Similarly, the positive time duration and impulse can be presented in a scaled form.

The energy released by an explosion that is used to calculate blast loads, is typically measured relative to TNT explosion. A majority of the data presented in the UFC 03-0340-02 (2008) are based on the spherical TNT explosion. Other mass-detonating materials can be taken into consideration; however, they have to be converted to an equivalent mass of TNT explosive. This can be done by relating explosion energy of the “effective charge weight” to the explosion energy

of an equivalent TNT explosive. Equation 2.3 is presented in UFC 03-0340-02 (2008) to calculate the blast pressure generated by an explosive expressed in terms of TNT charge-weight for unconfined detonations. This expression can be used to express the effect of energy output as a function of the heat of detonation:

$$W_E = \frac{H_{EXP}^d}{H_{TNT}^d} W_{EXP} \quad (2.3)$$

Where;

- W_E is effective charge weight (TNT equivalent)
- H_{EXP}^d is heat of detonation of explosive
- H_{TNT}^d is heat of detonation of explosive of TNT
- W_{EXP} is weight of explosive

The ratio H_{EXP}^d/H_{TNT}^d is known as the TNT equivalent factor. Some common TNT equivalent factors for various types of explosives for both peak pressure and impulse are present in Table 2-1 (Hyde 1992).

2.3.5 Determination of Blast Pressures

An empirical method is available for determining the blast load parameters generated for an explosion. The empirical method uses blast curves which relates blast wave parameters and the scaled standoff distance between the explosive source and the structure (Dusenberry 2009). The blast curves were constructed using Kingery-Bulmash (1984) polynomials, which were obtained from actual large-scale blast explosions conducted using 5-500 tons of TNT explosives in Canada between 1959 and 1964 (Swisdak 1994). Kingery-Bulmash (1984) polynomials are the basis for Conventional Weapons (CONWEP) software that is used for blast load parameter generation (Hyde 1988). The software generates blast load parameters for both spherical and hemispherical explosions.

Table 2-1: TNT Equivalent Masses for Various Explosives (Hyde 1992)

Explosive Type	Equivalent TNT Mass Factor	
	Pressure	Impulse
ANFO	0.82	0.98
Composition A-3	1.09	1.07
Composition B	1.11	0.98
Composition C-4	1.37	1.19
H-6	1.38	1.15
HBX-1	1.17	1.16
Octal	1.06	1.06
Pentolite	1.42	1.00
RDX	1.14	1.09
TNT	1.00	1.00
Tritonal	1.07	0.96

The empirical procedure to estimate blast load parameters presented in UFC 03-0340-02 (2008) for a hemispherical surface burst is presented below:

- The first step is to convert the explosive weight charge to an equivalent weight in TNT.
- Then, the scaled distance can be computed based on the distance from the explosion center to the point of interest.
- Incident blast wave properties such as incident pressure, positive time duration and impulse can be computed by using the appropriate figure given in UFC 03-0340-02. Figure 2-5 illustrates curves that can be used to determine the positive parameters of blast loading for a hemispherical surface burst explosion.
- Figure 2-6 illustrates a chart that can be used to determine the parameters for negative blast loading for a hemispherical surface burst explosion.
- Then, the angle of incidence (θ), which represents the angle at which the shock waves interact with the surface of the element relative to the normal line to the surface can be established.
- From the angle of incidence, one can compute the reflected pressure coefficient ($C_{r\alpha}$) from Figure 2-7. The magnification will be used to determine the reflected pressure (P_r) acting on the element by using the appropriate blast curve.

- The reflected pressure can be computed by multiplying the incident pressure (P_{so}) by the reflected pressure coefficient (C_{ra}).
- Finally, Friedlander exponential decay equation can be used to establish pressure time history for blast loading.

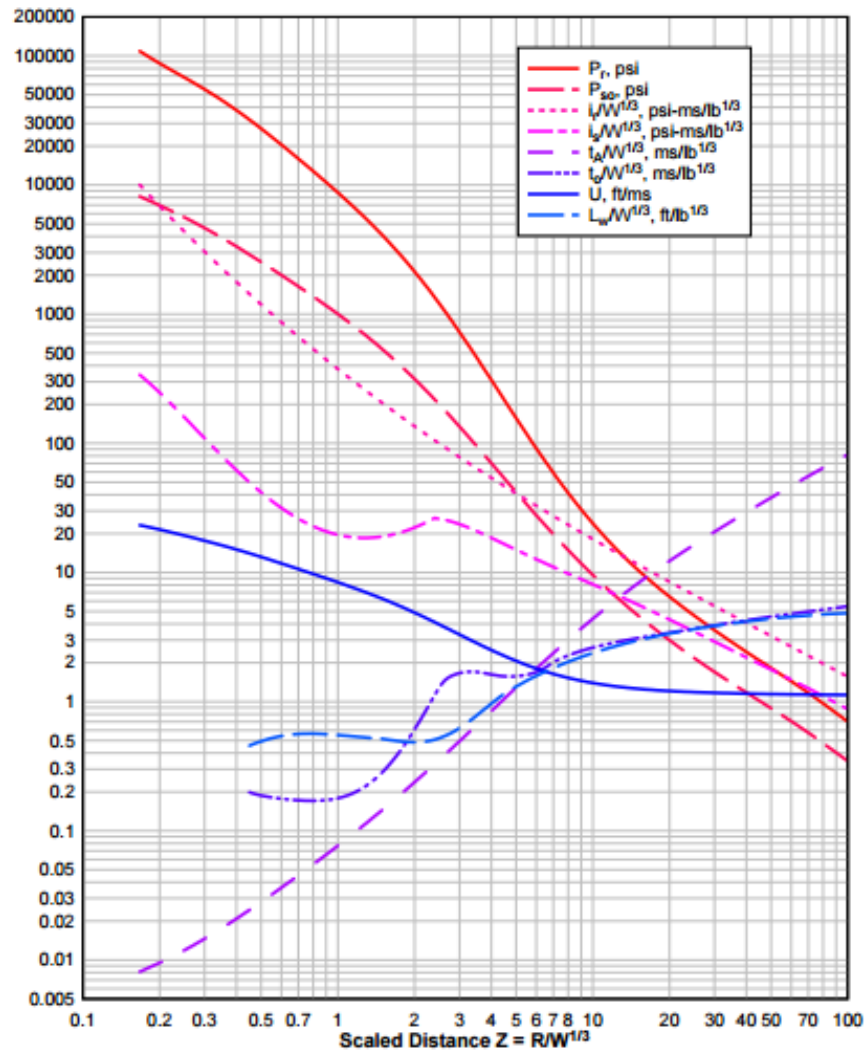


Figure 2-5: Positive Blast Parameters for a Hemispherical Air Burst (UFC 03-0340-02 2008)

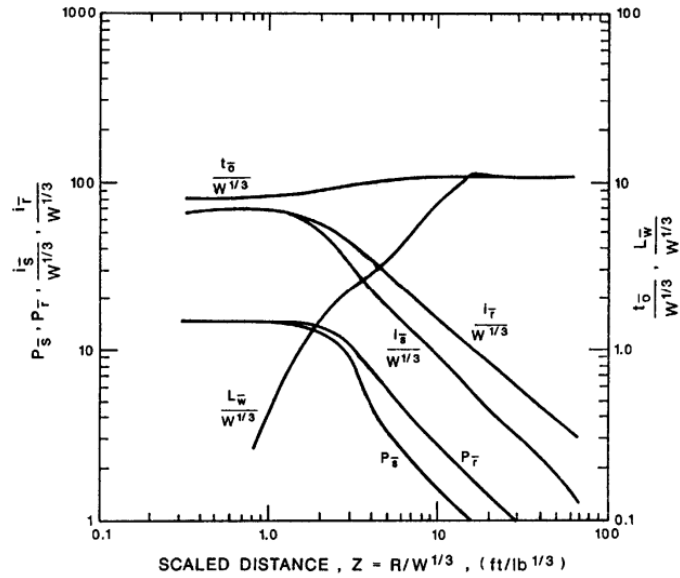


Figure 2-6: Positive Blast Parameters for a Hemispherical Air Burst (UFC 03-0340-02 2008)

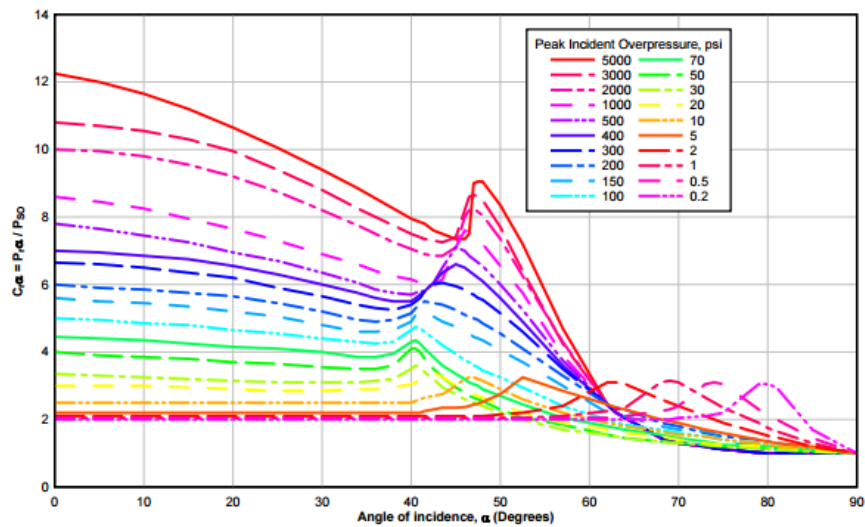


Figure 2-7: Magnification Factor Coefficient (UFC 03-0340-02 2008)

2.4 Structural Response to Blast Loading

2.4.1 Analysis Approach

A structure subjected to an explosion experiences time varying loads over a short period of time. The duration of blast loading usually varies between 1.0 and 50 milliseconds, and the duration can be longer as the explosion moves away from the building. Because of the time varying nature of

the load, dynamic analysis must be conducted to establish the behaviour of the structure. In practice, three methods that are widely used for analysis: i) single degree of freedom (SDOF) analysis, ii) multiple degree of freedom (MDOF) analysis and iii) finite element method (FEM) of analysis.

The number of independent coordinates required to define the deflected shape of a structure is known as the number of degrees of freedom. All structures produce multiple degrees of freedom. However, in blast analysis, a single mode can be assumed to govern the structural response during the short duration of the blast load (UFC 03-0340-02 2008). Therefore, the mode of vibration is dependent on a single variable, which is the displacement of the structure in one direction. This system is called single degree of freedom system, which is an idealization of the whole system. Components of this idealized dynamic system consist of a mass subjected to a concentrated load, a weightless spring attached to the mass, and a damper, as is illustrated in Figure 2-8.

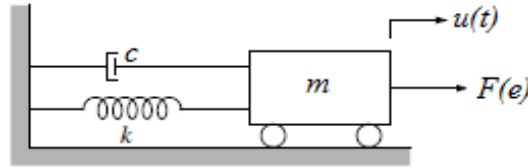


Figure 2-8: Idealized SDOF System (Modified from Clough and Penzien 2003)

The equation of motion for the dynamic system can be written as follows:

$$F_i + F_d + F_s = F_e \quad (2.4)$$

Where F_i is the inertia force due to the acceleration of the mass, F_d is the damping force due to the restraint provided by the surrounding viscous fluids, F_s is the resistance forcing function due to the stiffness of the spring, and F_e is the external blast force acting on the mass. Since blast occurs over a short period of time, damping can be neglected. Hence, the above equation reduces to Equations 2.5 and 2.6:

$$F_i + F_s = F_e \quad (2.5)$$

$$m\ddot{u}(t) + ku(t) = F_e(t) \quad (2.6)$$

The above equation is valid for an idealized system with a concentrated mass and spring. An actual structural element with uniformly distributed mass, as often is the case for a single structural element such as a column, needs to be transformed into an equivalent concentrated mass for the application of Equation 2.6. Similarly, the stiffness of the structural element and the uniformly distributed blast load need to be transformed into equivalent spring stiffness and concentrated force, respectively. These transformation factors can be established by assuming a single shape function $\phi(x)$ giving the deflected shape of the element. Then the principles of energy may be employed to derive the transformation factors. Since the deflected shape of an element is different in the elastic range when compared with the elasto-plastic range and the plastic range, the transformation factors are different in each region.

The first factor is called the mass factor (K_M) which represents ratio of the equivalent mass of the structure and the actual mass of the structure. It can be calculated from Equation 2.7:

$$K_M = \frac{\int_0^L m(x)\phi^2(x)dx}{mL} \quad \langle 2.7 \rangle$$

Where, m is the uniform mass of the structural element per unit length, and L is the total member length. The second factor is called the load factor (K_L) which represents the ratio of the equivalent load acting on the structural element and the actual load. It can be calculated as follow:

$$K_L = \frac{\int_0^L F_e(x,t)\phi(x)dx}{F_e(t)L} \quad \langle 2.8 \rangle$$

The same transformation factor can be used as the transformation factor for stiffness to find the equivalent stiffness of the SDOF model. This is because stiffness is load per unit deflection and the relationship between the uniformly distributed load on structural element producing maximum deflection and the equivalent concentrated force producing the same deflection is the same as the relationship between the stiffness of a real element and the spring stiffness of a concentrated mass. Substituting the transformation factors into Equation 2.6, one can obtain the following expression.

$$K_M m \ddot{u}(t) + K_L k u(t) = K_L F_e(t) \quad \langle 2.9 \rangle$$

Dividing all the terms in the above expression by K_L results in the definition of the load-mass factor (K_{LM}) as shown below, which is a ratio between the mass factor to the load factor.

$$K_{LM} = \frac{K_M}{K_L} \quad (2.10)$$

The equation of motion under blast loading for the equivalent system can then be rewritten as follow:

$$K_{LM}m\ddot{u}(t) + ku(t) = F_e(t) \quad (2.11)$$

Transformation factors for both one-way and two-way elements were derived by Biggs (1964). They also appear in UFC 03-0340-02 (2008). The transformation factors for one-way elements under uniformly distributed load, which is applicable to column analysis, with different boundary conditions are presented in Table 2-2 (Extracted from UFC 03-0340-02 2008).

**Table 2-2: Transformation Factors for One-Way Elements for Uniform Loading
(Extracted from UFC 03-0340-02 2008)**

Edge Conditions	Behavior	Load Factor, K_L	Mass Factor, K_M	Load-Mass Factor, K_{LM}
Pin-Pin	Elastic	0.64	0.50	0.78
	Plastic	0.50	0.33	0.66
Fix-Free	Elastic	0.58	0.45	0.78
	Elasto-Plastic	0.64	0.50	0.78
	Plastic	0.50	0.33	0.66
Fix-Fix	Elastic	0.53	0.41	0.77
	Elasto-Plastic	0.64	0.50	0.78
	Plastic	0.50	0.33	0.66

Equation 2.11 can be solved numerically under elastic or inelastic response assumption to determine the response history of the structure. The structural response using SDOF cannot capture possible local structural responses due to an extremely high blast loading. Hence, for such cases, multiple-degree-of-freedom (MDOF) method of analysis can be used to accurately predict structural response to blast loading. MDOF can be very costly to perform as it depends on the number of coordinates chosen to define the deflected shape of the structure. An alternative to

conducting more accurate analysis is to use Finite Element Method (FEM) of analysis. FEM analysis software is widely used to predict structural response under blast loading. However, the FEM analysis can be costly. The cost of using FEM software to perform nonlinear dynamic analysis stems from the computational time required, which may be excessive at times. Proper choices can be made to reduce the computational time for the analysis.

2.4.2 Modes of Failure

Columns are essential components within a building, responsible for overall strength and stiffness of the structure; as they carry axial loads comprised of dead and live loads within the building frame. Loss of a column can lead to the failure of the whole building if the building frame cannot redistribute the loads to other components of the frame. This phenomenon is known as progressive collapse, and it becomes a concern for buildings subjected to extreme blast loads. Possible modes of failure of reinforced concrete columns needs to be looked at carefully in order to assess its vulnerability in a building subjected to blast loads. Reinforced concrete columns under blast loads can either fail globally or locally as illustrated in Figure 2-9. Depending on the boundary conditions of a column, and the rate of blast loading, different modes of failure governs the overall dynamic response of columns under blast loads (Ngo et al. 2007).

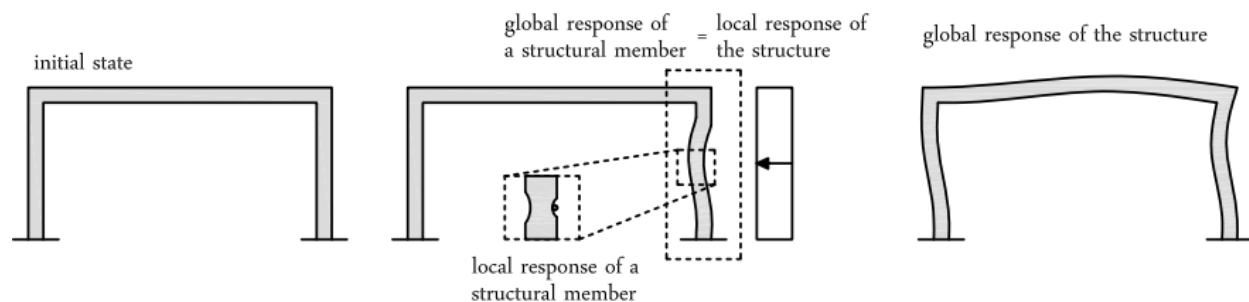


Figure 2-9: Local and Global Responses of Reinforced Concrete Column (Ekström 2015)

2.4.2.1 Global Failure Modes

Generally, global failure modes of reinforced concrete columns under blast loads can be divided into flexure and shear failures. When a column is subjected to blast loading, it displaces laterally. The kinetic energy is absorbed by the column, turning into elastic and inelastic deformations (Rodriquez-Nikl 2006). If a column has adequate shear capacity, it can exhibit large inelastic

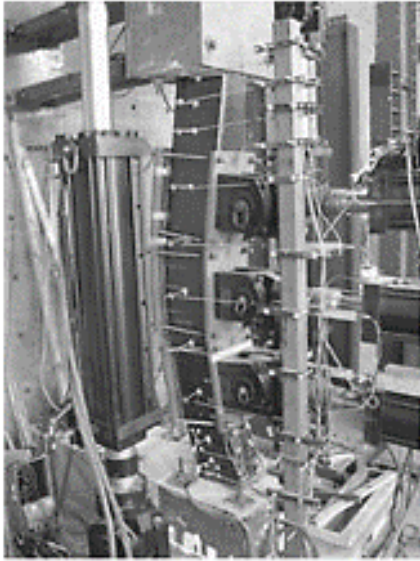
deformations prior to flexural failure. Flexural mode of failure is associated with the formation of plastic hinges in critical regions (regions of maximum moment). Flexural failure in reinforced concrete columns is illustrated in Figure 2-10 (a).

Shear failure mode in reinforced concrete columns can be subdivided into direct shear failure and diagonal shear failure. Diagonal shear failure can be further subdivided into diagonal tension and diagonal compression failures.

Direct Shear failure occurs within a very short period of time after the arrival of shock wave due to the concentration of shear stresses at column supports. Direct shear failure mode occurs prior to the development of flexural failure mode as the column doesn't have the time to develop flexural inelastic deformations (Ngo et al. 2007). The direct shear failure mode in reinforced concrete columns is illustrated in Figure 2-10 (b).

Diagonal tension failure occurs due to lack of shear capacity by not providing enough transverse reinforcements. Failure due to diagonal tension is associated with brittle non ductile behaviour. Diagonal compression failure occurs due to the over-reinforcement of member against diagonal tension. Because diagonal tension and diagonal compression are components of applied shear forces, increasing diagonal tension capacity places the burden on concrete compressive struts, which may develop compression crushing of members. Diagonal shear failure mode in reinforced concrete columns is illustrated in Figure 2-10 (c).

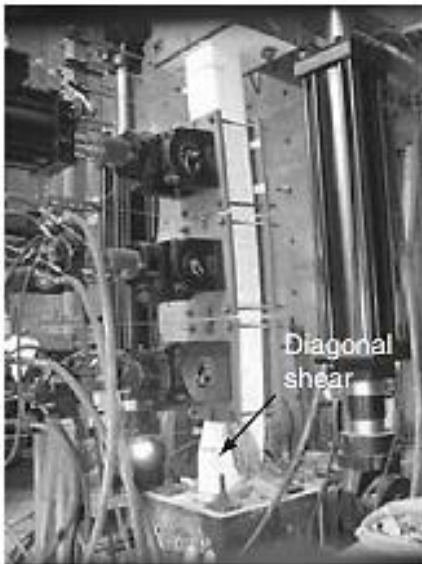
Sometimes, depending on relative strengths in flexure and shear, a combination of flexural and shear modes of failure may occur in columns under blast loads. This is illustrated in Figure 2-10 (d).



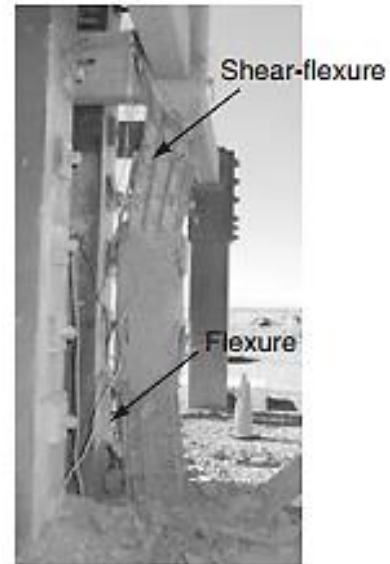
(a) Flexural Failure



(b) Direct Shear Failure



(c) Diagonal Shear Failure



(d) Combination of Shear and Flexural Failures

Figure 2-10 : Global Failure Modes (Crawford et al. 2013)

2.4.2.2 Local Failure Modes

Local failure mode in a column depends on the standoff distance between the centre of explosion and the column, and it occurs under close-in explosions (Ngo et al. 2007). Basically, as the standoff distance becomes small, enormous pressure generated by a close-in explosion over a very short period of time results in local failures. The shock waves generated from the explosion propagates

and reflects within the column few times before the column has a chance to attain its global shear or flexural capacities. The compressive shock wave, when interacts with the column, spalls off concrete soon after the arrival time. Spalling occurs when concrete compressive capacity is exceeded on the face of the column under rapidly applied compressive shock wave as illustrated in Figure 2-11 (a).

The compressive shock wave on the face of the column is reflected at the back face, forming a tensile shock wave at the back face. This puts concrete at the back face in tension. Scabbing at the back face occurs because tensile stresses generated from the shockwave exceed the tensile capacity of concrete, which is very low relative to its compressive capacity (Conrath et al. 1999). The tensile capacity of concrete under slow rate of loading is approximately equal to 10% of its capacity in tension. Scabbing failure mode is presented in Figure 2-11 (b).

As the shock wave travels through the column, it could cause a partial or complete breaching of the column due to the formation of spalling and scabbing (Ngo et al. 2007). Figure 2-11 (c) illustrates a fully breached column. This phenomenon is common in close-in explosions, hence forms one of the failure modes that is under investigation in the current research project.

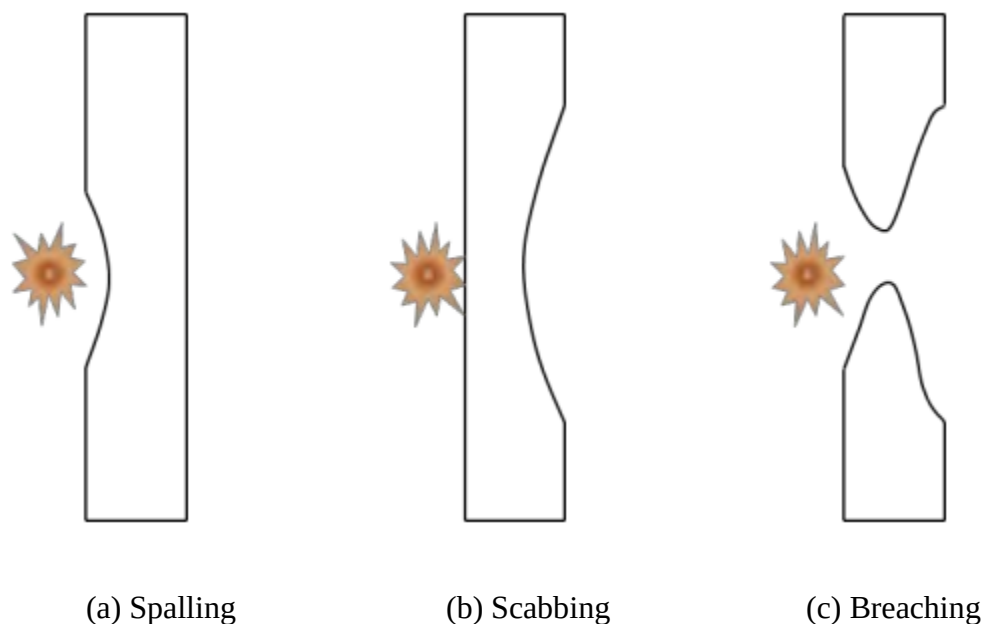


Figure 2-11: Local Failure Modes (Nyström 2008)

2.5 Material Response to Blast Loading

Material response under blast loading shows increased strength and stiffness when compared with behaviour under slowly applied static loading. This increase in material strength is associated with high strain rates produced by blast loading. Researchers reported that blast loads produce strain rates that range between 10^2 - 10^4 sec^{-1} (Ngo et al 2007). These rates under various dynamic loads are shown in Figure 2-12. Static loading produces strain rates of 10^{-6} - 10^{-5} sec^{-1} . The strength enhancement in materials is quantified by dynamic increase factor (DIF), which is the ratio of strength of the material under blast loading to that under static loading. The strength enhancement depends on the material type and the type of stress (whether the material is under tensile or compressive loading). The higher the strain rate produced by blast loading, the higher the strength enhancement would be. Reinforced concrete is comprised of two materials: concrete and steel. The following section explains the different models developed to find strength enhancements in these two materials.

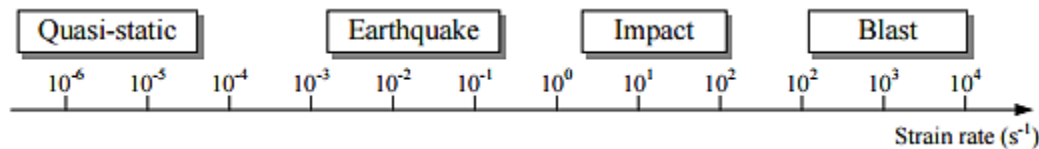


Figure 2-12: Strain Rate for Different Types of Loadings (Ngo et al. 2007)

2.5.1 Concrete

Compressive and tensile strengths of concrete under blast loads show increases due to the rapid rate of blast loads. Figure 2-13 illustrates the change in stress-strain relationship of concrete due to the strain rate effect, with clear increase in compressive strength under blast loading. According to Johansson (2000), the strength enhancement in concrete is due to viscous and inertial effects. Under lower strain rates, the viscous effect dominates the behaviour due to the fact that the pore water pressure in concrete increases with the increasing rate of loading. For higher strain rates, inertial and confinement effects become more dominant. As shown in Figures 2-14 and 2-15, the dynamic increase factor for concrete in compression and tension increases linearly in the lower strain rate range, but the rate of increase changes (increases) at 30^{-1} sec^{-1} due to the inertia effects (Bischoff and Perry 1991).

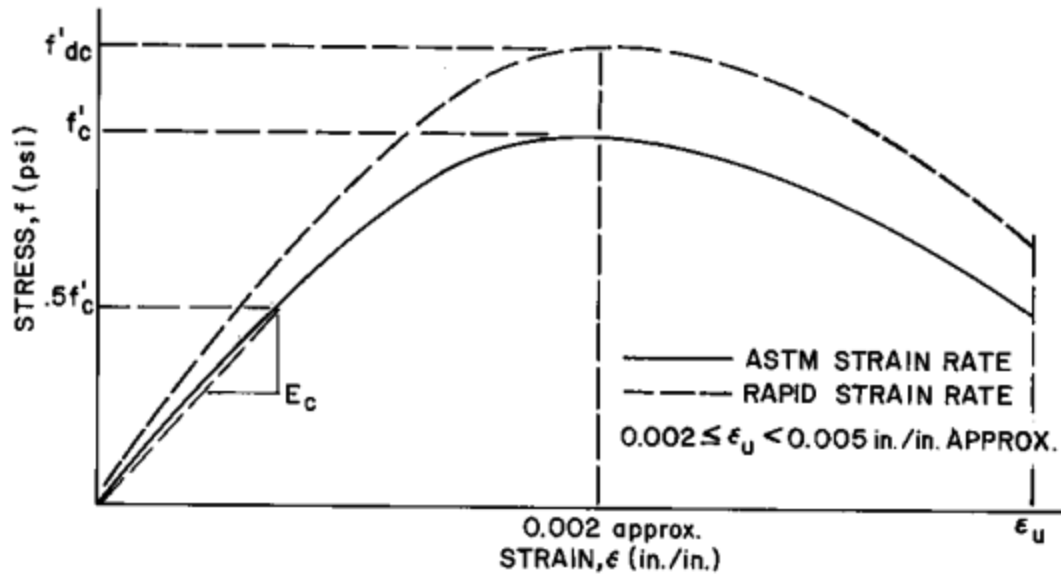


Figure 2-13: Stress-Strain Relationship for Concrete under Static and Rapid Loading
(UFC 03-0340-02 2008)

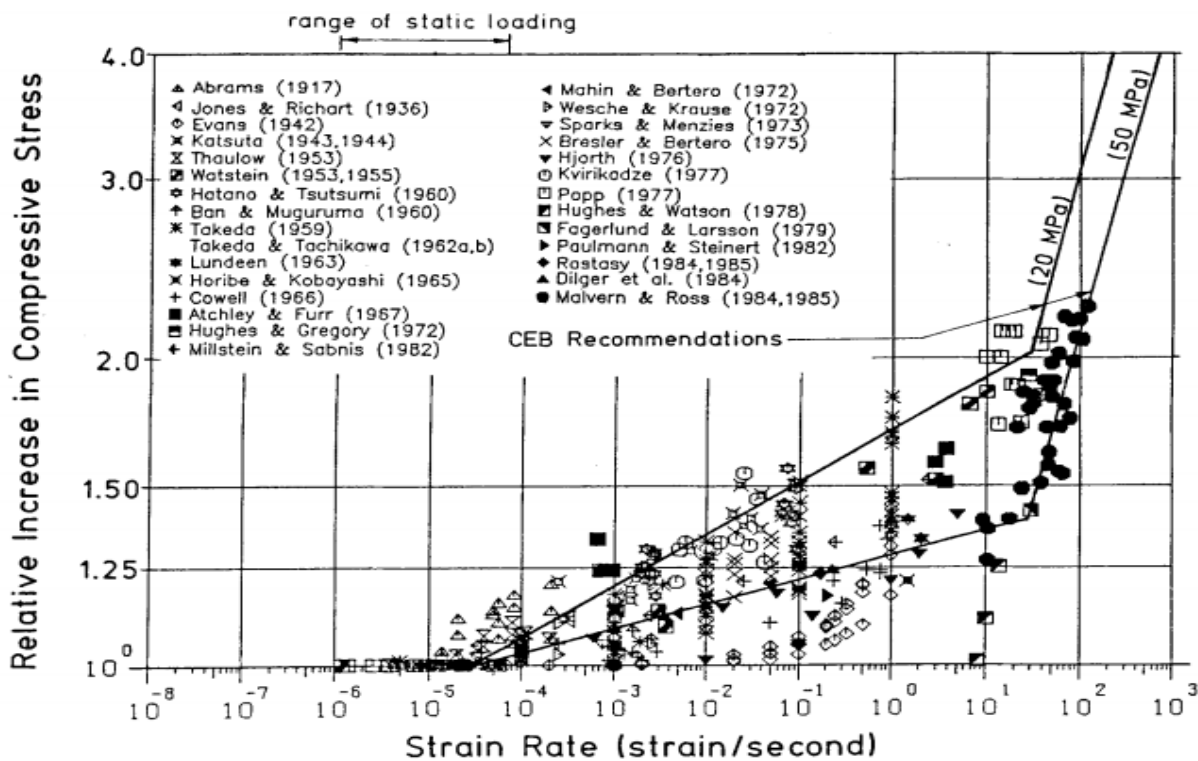


Figure 2-14: Strength Enhancement Factor for Concrete in Compression
(Bischoff and Perry 1991)

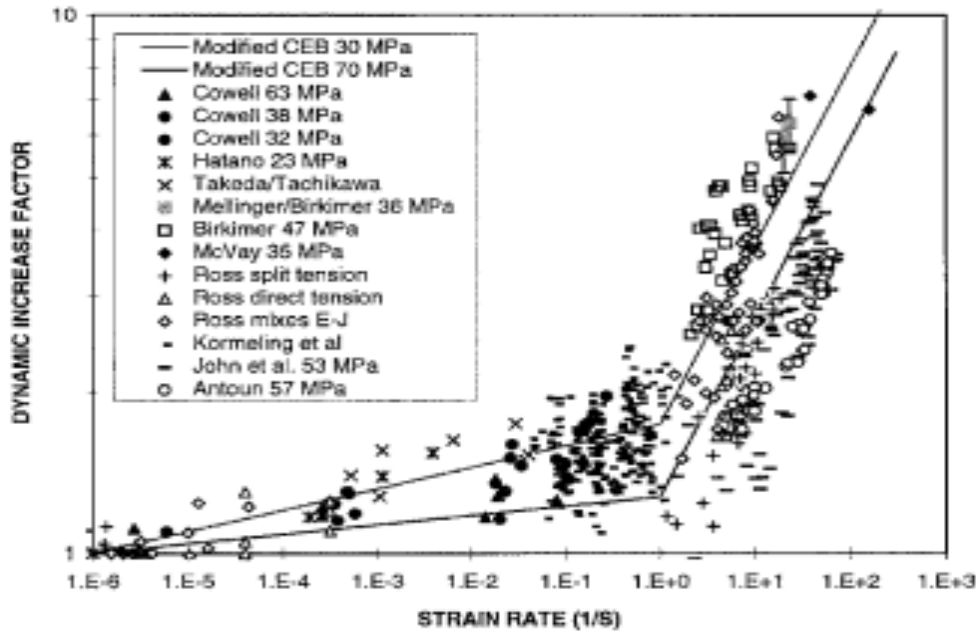


Figure 2-15 Strength Enhancement Factor for Concrete in Tension (Bischoff and Perry 1991)

The strength enhancement for concrete can be calculated using the Comité Euro-International du Béton (CEB) model (CEB 1993), which is widely used among researchers. As shown in Figure 2-14, the CEB recommendation for concrete in compression compares well with tests conducted by other researchers. Accordingly, the strength enhancement can be calculated using Equations 2.12 and 2.13:

$$\frac{f_c}{f_{cs}} = \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s} \right)^{1.026\alpha_s} \quad \text{for } \dot{\epsilon} \leq 30s^{-1} \quad (2.12)$$

$$\frac{f_c}{f_{cs}} = \gamma_s \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s} \right)^{1/3} \quad \text{for } \dot{\epsilon} > 30s^{-1} \quad (2.13)$$

Where;

- f_c is dynamic compressive strength at $\dot{\epsilon}$
- f_{cs} is static compressive strength at $\dot{\epsilon}_s$
- f_c/f_{cs} is compressive strength dynamic increase factor (DIF)

- $\dot{\epsilon}$ is strain rate in the range of 30×10^{-6} to 300 s^{-1}
- $\dot{\epsilon}_s = 30 \times 10^{-6} \text{ s}^{-1}$ static strain rate
- $\log \gamma_s = 5.056\alpha - 2$
- $\alpha_s = 1/(5 + 9 f_{cs}/f_{co})$
- $f_{co} = 10 \text{ MPa} = 1450 \text{ psi}$

Figure 2-15 compares the CEB recommendation for concrete strength enhancement in tension with tests conducted by other researchers. The CEB recommendation for strength enhancement is given in Equations 2.14 and 2.15:

$$f_t/f_{ts} = \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s}\right)^{1.016\alpha_s} \quad \text{for } \dot{\epsilon} \leq 30 \text{ s}^{-1} \quad \langle 2.14 \rangle$$

$$f_c/f_{cs} = \beta \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s}\right)^{1/3} \quad \text{for } \dot{\epsilon} > 30 \text{ s}^{-1} \quad \langle 2.15 \rangle$$

Where;

- f_t is dynamic tensile strength at $\dot{\epsilon}$
- f_{ts} is static tensile strength at $\dot{\epsilon}_s$
- f_t/f_{ts} is tensile strength dynamic increase factor (DIF)
- $\dot{\epsilon}$ is strain rate in the range of 3×10^{-6} to 300 s^{-1}
- $\dot{\epsilon}_s = 3 \times 10^{-6} \text{ s}^{-1}$ static strain rate
- $\log \beta = 7.11\delta - 2.33$
- $\delta = 1/(10 + 6 f_{cs}/f_{co})$
- $f_{co} = 10 \text{ MPa} = 1450 \text{ psi}$

2.5.2 Steel

Steel under high strain rates show increases in both the yield strength and the ultimate strength. Figure 2-16 illustrates strength enhancement in steel due to blast loading. As shown in the figure, the enhancement increase for the yield stress is far more than the increase on the ultimate stress. The strength enhancement of steel reinforcement can be calculated based on Cowper-Symonds model (LSTC 2015). This model scales yield stress by a strain rate dependant factor using Equation 2.16.

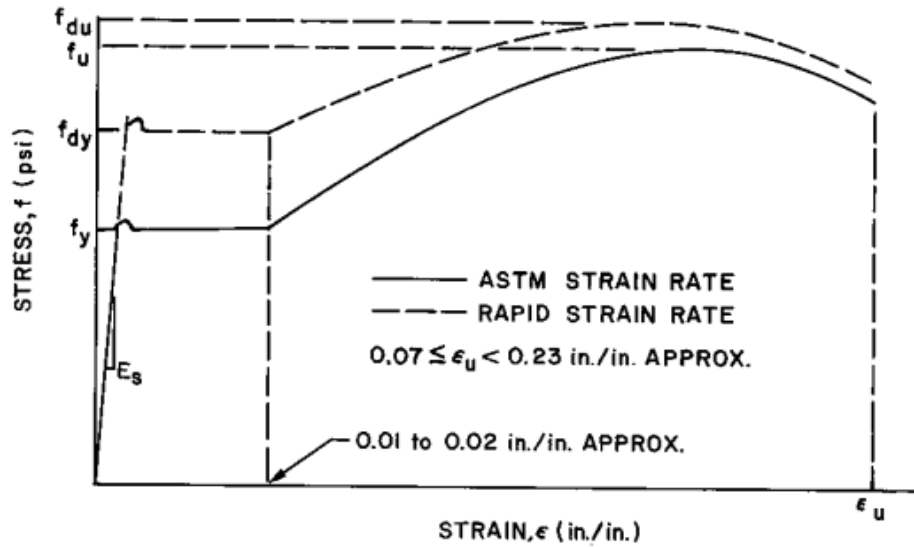


Figure 2-16: Stress-Strain Relationship for Steel under Static and Rapid Loading (UFC 03-0340-02 2008)

$$f_{dy} = f_y \left[1 + \left(\frac{\dot{\epsilon}}{C} \right)^{\frac{1}{P}} \right] \quad (2.16)$$

where;

- f_{dy} is the dynamic yield strength
- f_y is the static yield strength
- $\dot{\epsilon}$ is the strain rate
- C and P are user defined Cowper-Symonds constants

2.6 Previous Research

2.6.1 Experimental Research

This section presents an overview of experimental research conducted on reinforced concrete columns subjected to blast loads.

NCHRP Report 645 Project 12-72 (2010)

The main objective of the research project reported by NCHRP (2010) was to come up with design guidelines for reinforced concrete bridge columns subjected to blast loading. During Phase II of the project, ten half-scale reinforced concrete columns were tested with small standoff distances to investigate possible modes of failures experienced by bridge columns. Five different parameters were used in the test program, which include scaled standoff distance, column geometry, type and percentage of transverse reinforcements, and longitudinal reinforcement lap splice location. Charge-mass and standoff-distance were not disclosed for security reasons.

The columns were either circular or square in shape with longitudinal reinforcement ratio varying between 1.04% and 1.18%. The transverse reinforcement consisted of either ties or spiral for the circular columns, and ties for the square columns. Some of the columns were designed as typical columns, some as seismic-resistant columns, and the rest as blast-resistant columns. The boundary condition for the columns in a reaction structure simulates fixed free condition. The columns were loaded so as to generate flexural or shear responses. Then, the same columns were tested (ensuring the first test didn't cause heavy damage to the columns) with load combinations involving small standoff distances to identify the localized failure modes. Some of the observed failure modes during testing are illustrated in Figure 2-17.

The following was concluded from this project:

1. Performance of columns improves significantly with the increase of standoff distance.
2. Larger cross sections in columns helped prevent breaching failure mode.
3. Using spirals as transverse reinforcements increased column confinement, improving behaviour under blast loads.

4. Columns with higher transverse reinforcement ratio helped prevent failures of some columns.



Figure 2-17: Observed Modes of Failure during NCHRP Project (NCHRP Report 645 2010)

A. Lloyd (2010)

The author's primary objective was to study the effect and response of blast loading on reinforced concrete columns. Fourteen half-scale columns were tested at the University of Ottawa using a Shock Tube. Those columns were designed according to the Canadian Standard Association CSA23.3 (2006) to represent first storey exterior columns located in seismic and non-seismic regions.

Twelve columns were rectangular with cross sectional dimensions of 100 mm x 150 mm and the other two columns were square with dimensions of 150 mm x 150 mm. All columns were 2438 mm tall. The columns were reinforced with 4–10M longitudinal bars. Different transverse reinforcement spacing was used; 25 mm, 50 mm and 75 mm. The compressive strength of concrete for the columns was either 46 MPa or 58 MPa. The columns were designed to be fixed-fixed columns but full fixity was not achieved. An axial load of either 0 kN, 350 kN or 400 kN was applied on the columns using a hydraulic jack. Various blast load parameters were generated using the shock tube, and applied at the front face of the columns. Figure 2-18 illustrates the crack pattern observed on the tension side of one of the columns that formed a plastic hinge at mid span region.

The author concluded that reducing the spacing of transverse reinforcements had no significant effect on column response under large pressures, especially under low levels of axial load, as the neutral axis would approach the cover concrete, reducing the effectiveness of confinement reinforcement. Also, the drop in axial load due to lateral displacement was significant on column response.

S.C. Woodson, J.T. Baylot (1999)

Authors carried out an experimental investigation on response of reinforced concrete columns under blast loads. Four two-storey quarter-scale models were tested with blasts at different standoff distances. The second parameter that was considered in this study was the configuration of the cladding system. Masonry infill walls were used in quarter scale models to assess exterior column behaviour in their presence. Figure 2-19 illustrates the reaction structure model. The explosive type that was used was C4 with a mass of 7.1 kg at a height of 0.229 m from the ground surface in front of the middle column.



Figure 2-18: Flexural Cracks at the Mid-Span of the Column (Lloyd 2010)

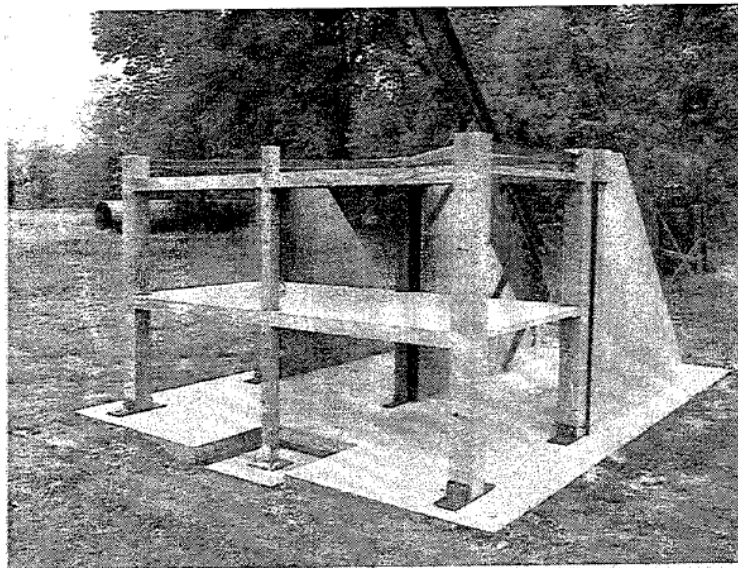


Figure 2-19: Two Storey Quarter Scale Model (Woodson and Baylot 1999)

The first model was tested with a standoff distance of 1.524 m and represented an open frame structure. There was no damage observed in this experiment and the same quarter scale model was

used for the second experiment. The standoff distance was reduced to 1.067 m for the second experiment which resulted in a permanent displacement of 6.35 in the lower part of the column, but the column was able to remain in-place after the explosion. Also, some cracks were observed in the test model.

The third experiment was conducted with a standoff distance of 1.067 m on a quarter scale model with concrete masonry unit walls. This experiment resulted in a peak displacement of 114.3 mm in the lower section and 25.4 mm in the upper section of the column. The column couldn't carry any more axial load but remained in place as the edge beams were able to carry the lost load from the column.

The fourth experiment was conducted with a standoff distance of 1.067 m on a quarter scale model with concrete masonry unit walls with window openings. This experiment resulted in a peak displacement of 76.2 mm in the lower section and 14.3 mm in the upper section of the column. The presence of window opening led to severe deformation of the slab in between the two floors.

Lastly, the fifth experiment was conducted with a standoff distance of 1.067 m on a quarter scale model with concrete masonry walls in the second storey only. This experiment resulted in a permanent displacement of 76.2 mm in the upper section of the column and extensive damage to the slab in between the two floors.

The authors concluded that cladding systems were exposed to a larger impulse generated from the blast loading and had a significant effect on the overall response of reinforced concrete columns in the building frames.

2.6.2 Numerical Research

This section presents an overview of numerical research conducted on reinforced concrete columns subjected to blast loads.

J. Cui, Y. Shi, Z. Li, and L. Chen (2015)

The authors of this paper analyzed and assessed damage to reinforced concrete columns under close-in explosions using LS-DYNA. In this study, the blast shock wave was modelled explicitly

using the arbitrary Lagrange Euler method (ALE). Two-dimensional air mesh domain was remapped to a three-dimensional domain in which the TNT explosive shock waves propagated through. The mesh size used in the 2-dimensional analysis was 1 mm. The mesh size that was used in the direction of the shock wave propagation was 5 mm, and 10 mm in the two orthogonal directions for the 3-dimensional analysis.

Reinforced concrete column was modelled within the air mesh domain using MAT072-CONCRETE_RELATIVE_DAMAGE for concrete and MAT024-PIECEWISE_LINEAR_PLASTICITY for steel reinforcements. Concrete mesh was the same as the air mesh 5 mm along the wave propagation and 10 mm in the other two directions, while the reinforcements mesh was 10 mm. Maximum principal strain erosion criteria was used to delete elements that lost their strength. A parametric study was done on the erosion criteria for the following values: 0.1, 0.12, 0.15, 0.18 and 2.0 which yielded 0.15 as the governing value.

Three different charge masses were used 3 kg, 4 kg and 6 kg placed at a standoff distance of 0.2 m at a height of 1 m from the ground surface to simulate a close-in blast load. Three different column configurations were used, consisting of 400 mm x 400 mm x 3600 mm, 300 mm x 300 mm x 2700 mm and 200 mm x 200 mm x 1800 mm. The columns had three different longitudinal reinforcement ratios, consisting of 0.68%, 1.13% and 1.69%. Three different transverse reinforcement ratios were used, as 0.13%, 0.22% and 0.50%. Two different concrete covers were used, either 20 mm or 40 mm. The columns were subjected to axial load equivalent to 20% of their capacity. As a result of the analysis, the authors concluded the following:

1. Column failures were local, either spalling of concrete on the tension side or concrete crushing on the compression side, as illustrated in Figure 2-20.
2. Column blast resistance increased when the column cross sectional dimensions increased, stirrup ratio increased, stirrup spacing decreased and concrete cover was reduced.
3. A column damage assessment criterion was proposed based on column residual displacement and column depth.

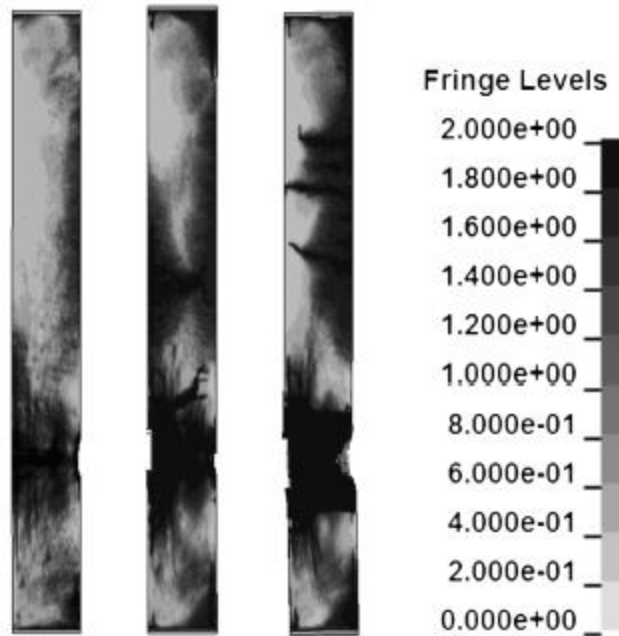


Figure 2-20: Local Failure Modes for Columns with Different Standoff Distance/Mass Combinations (Cui et al. 2015)

J. Li, H. Hao (2014)

The researchers carried out a numerical study on concrete spall damage in reinforced concrete columns under blast loads, using LS-DYNA. They first validated their 3-dimensional model with existing data for a reinforced concrete slab subjected to close-in blast loading. The validation was done for a cylindrical explosive of 0.64 kg mass at a distance of 0.5 m away from the slab. Then, different column configurations and standoff distance/mass combinations were used to evaluate the spall damage of reinforced concrete columns.

Blast load was simulated using the LOAD-BLAST_ENHANCED key card in LS-DYNA. Concrete was modelled using MAT072-CONCRETE_RELATIVE_DAMAGE, while, steel reinforcement was modelled using MAT024-PIECEWISE_LINEAR_PLASTICITY. Strain rate effect was included for both concrete and steel reinforcement. The mesh size that was used for both concrete and steel elements was 5 mm. The erosion algorithm was based on the maximum principle strain of failure with a value of 0.01.

Two different mass/standoff combinations were used: i) 5 kg of TNT mass at 0.5 m and ii) 1 kg of TNT mass at 0.2 m. Three column configurations were used with a constant width of 300 mm but different depths of 200 mm, 300 mm and 400 mm. The column heights were 2400 mm, 3000 mm, and 4500 mm, respectively. The longitudinal and transverse reinforcement ratios were 1.5% and 0.2%, respectively for all the simulations. Longitudinal reinforcements were spaced at 210 mm, 100 mm, 65 mm. Transverse reinforcements were: 10 mm in diameter, spaced at 0.3 m; 8 mm in diameter spaced at 0.2 m; or 7 mm in diameter spaced at 0.15 m.

The following were the conclusions attained from the research project:

1. Column depth had a huge impact on spall damage. An increase in column depth reduced the local damage caused by close-in explosions.
2. Column height, which affected flexural stiffness, had minimal effect on the spall damage. This is shown in Figure 2-21.
3. Closely spaced longitudinal and transverse reinforcement helped reduce spall damage. It was postulated that increasing reinforcements helped disperse the stress wave generated in column elements.

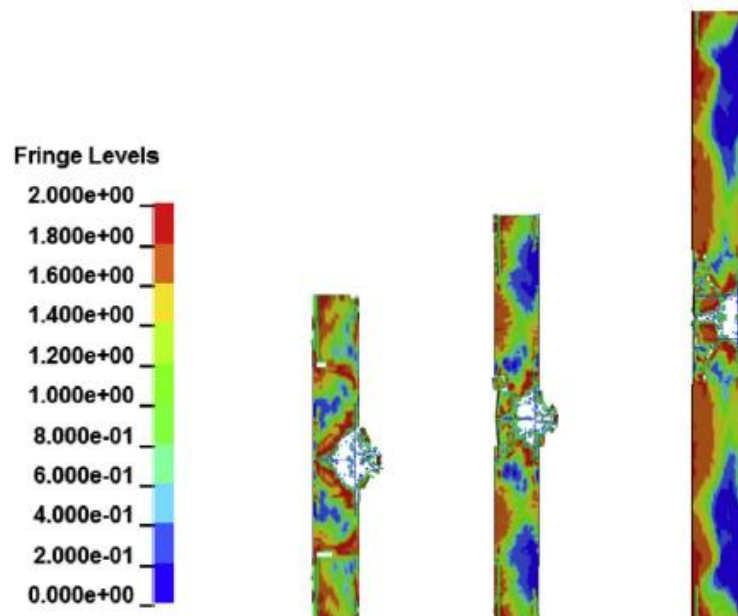


Figure 2-21: Spall Damage for Columns with Different Heights (Li et al. 2014)

K. Wu, B. Li, K. Tsai (2011)

The objective of the research project that was reported by the authors was to establish the residual axial compression capacity of damaged reinforced concrete columns under close-in explosions. Numerical and experimental studies were conducted using explosive charges placed at a close distance from the column. Parametric study was carried out to find a relationship between the column residual capacity and a series of parameters consisting of material strength, column detailing and blast variables.

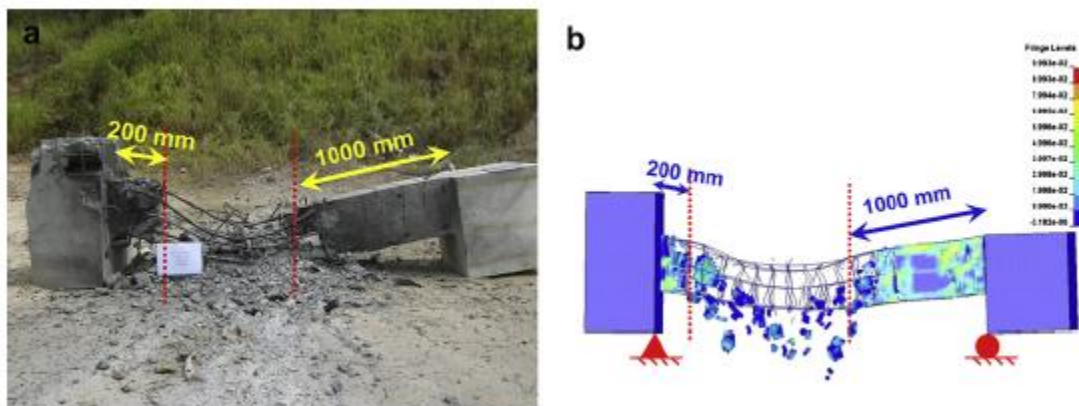
MAT072-CONCRETE_RELATIVE_DAMAGE was used to model the concrete, and MAT024-PIECEWISE_LINEAR_PLASTICITY was used to model the steel reinforcement. Strain rate effect was taken into consideration. The mesh size for all types of elements was 50 mm. The column footing and the head were used to construct the column model to obtain the best representative boundary conditions.

The axial load was applied and increased linearly (i.e. initially the load was zero, and then increased to maximum load at time 50 ms). The axial load ratio varied between 0.2 and 0.4. Then, the blast load was applied on the front face of the column until the velocities of the nodes were very small. The blast load was simulated using the arbitrary Lagrange Euler method (ALE). Air was assumed to be ideal gas. The explosive geometry was the same as what had been used in the experiment. The axial load was applied at the column top nodes to establish the column residual capacity. Finally, the residual capacity index was obtained by dividing the residual capacity by the nominal capacity of the damaged column.

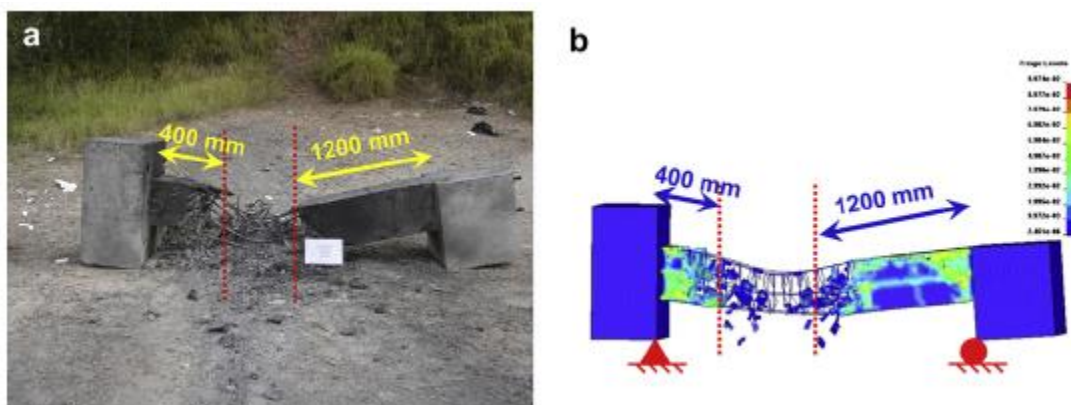
The numerical study was validated with experimental data. Experimentally, both the footing and the head were built for two columns, for a better representation of the boundary conditions. As charge, 25 kg of TNT was placed at 200 mm and 500 mm from the front face of the column. The columns had 400 mm square cross-sections with a 2400 mm column height. The longitudinal reinforcement ratio was 1.57% and the volumetric ratio of transverse reinforcement was 0.21%. Both columns suffered extensive local damage and deemed to have no residual capacity. The local damage zone length was the same experimentally and numerically. Figure 2-22 illustrates the

experiment and numerical findings. From the numerical parametric study, authors concluded the following:

1. Increasing column depth reduced local damage caused by near field explosions.
2. Residual axial capacity increased when the axial load ratio increased.
3. Increasing transverse reinforcement ratio enhanced the residual axial capacity.
4. Increasing longitudinal reinforcement ratio did not increase the residual axial capacity.
5. Residual capacity increased with standoff distance from the ground.



(a) 25 kg of TNT at 200



(b) 25 kg of TNT at 500

Figure 2-22: Experimental Versus Numerical Findings (Wu et al. 2011)

X. Bao, B. Li (2010)

The objective of the research projected reported by the authors was to determine the residual strength of reinforced concrete columns subjected to close-in blast explosions numerically. Numerical simulations were conducted on 12 columns using the LS-DYNA software. Columns were fixed at the bottom, and free to move in the vertical direction. The axial load ratio applied to simulate gravity loads varied between 0.1-0.4 of column nominal capacity. The columns were tested under different parameters, which included height, axial load ratio, longitudinal reinforcement ratio, transverse reinforcement ratio and column aspect ratio. Concrete was modelled using MAT072-CONCRETE_RELATIVE_DAMAGE. Blast was simulated using *LOAD-BLAST_ENHANCED key card. The explosive was triggered at a distance of 5 m from columns and had a charge mass weight value varying between 0 ton and 1 ton. The following was concluded from the paper:

1. Detailing columns seismically enhanced blast resistance.
2. Axial load effects on columns became more dominant when the transverse reinforcement ratio was small.
3. Reducing column aspect ratio and increasing transverse reinforcement ratio helped increase column axial capacity.

R. Jayasooriya, D. P. Thambiratnam, N. J. Perera, V. Kosse (2011)

Numerical work was conducted by the researcher to determine blast and residual capacities of typical reinforced concrete frames in a building. The frame analysis included column response due to near field explosions. Ten storey building was analyzed using SAP2000 and LS-DYNA. Global first order linear elastic uncoupled time history analysis was conducted using SAP2000. The purpose of the global analysis was to establish parameters that affected the interaction of the elements of the lower part of the building with the entire building. Then, uncoupled second order elasto-plastic analysis was conducted using LS-DYNA. The purpose of the analysis was to determine local effects of near field explosions on the lower part of the building. From the second analysis, blast and residual capacities of columns and beams were determined. Blast load was simulated using *LOAD-BLAST_ENHANCED key card for an explosive charge of 500 kg of

TNT at a standoff distance of 5 m from the building, placed at a height of 1.0 m from the ground surface.

The following were drawn from the research project as conclusions:

1. Global and local analyses were necessary to determine blast load effects on the entire building.
2. Global analysis led to the determination of frames capacity within the 10-storey building.
3. Local analysis resulted in damage levels in critical components of the building.
4. Damage levels were determined from effective plastic strains as shown in Figure 2-23.

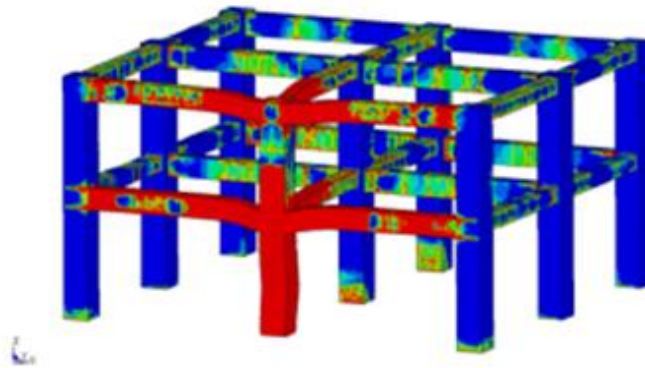


Figure 2-23: Damage Levels in the Frame (Jayasooriya et al. 2011)

2.7 Research Needs

The review of previous literature revealed that some research has been conducted on reinforced concrete column response under high loading rates. The results indicate that, explosives placed at a distance very close to the target structure produce extremely high blast loading rates that change the failure modes in columns. Research on the effects of close-in explosions is rare in the literature. Structural response/failure modes under close in explosions need to be examined further under different blast loading rates. The current research project is intended to fill gaps that exist related to the most common cases of terrorist attacks on infrastructure i.e., vehicular bombs and suicide bombs that happen to be the most common types of malicious attacks. Therefore, the effects of small and large masses of explosives need to be examined when detonated at close distances to buildings. Failure modes associated with loading rates need to be examined. Existing concrete

columns lack shear capacity. Hence, the effects of transverse reinforcement spacing used in old construction, current conventional construction, and seismically detailed construction need to be assessed.

3 Chapter 3

Computer Software LS-DYNA and Numerical Validation

3.1 Introduction

Finite element method (FEM) of analysis provides a powerful tool to study the effects of blast loads on structures. Using equilibrium equations implemented in FEM software can estimate structural response under blast loads accurately. The main concept in such programming tools is to discretize the structure into smaller finite elements. These elements are connected at common nodes and along common edges. Continuity is assured at the common nodes and edges by enforcing compatibility (Logan 2007).

The first step to analyze structures under blast loads is to divide the structural members into finite elements and choose the most suitable element that can represent the physical characteristics of structural members. Choosing an element depends on the physical behaviour of the structure, geometry of the structure and degree of accuracy needed to numerically solve the problem. There are different types of elements that can represent a structure, such as 1-D elements, 2-D elements and 3-D elements.

Displacement functions are assumed within each element. The function must be complete and it must allow rigid body motion. Compatibility is employed by relating strains to nodal displacements. Stresses are related to strains through appropriate constitutive models that represent material behaviour. The FEM computer software that was selected to conduct blast analysis of reinforced concrete columns in the current investigation is LS-DYNA.

3.2 LS-DYNA

LS-DYNA software package is a powerful programming tool that was developed by Livermore Software Technology Corporation (LSTC). LS-DYNA is used to solve nonlinear transient dynamic problems using explicit time integration. Nonlinearity is associated with material

behaviour as well as excessive deformation problems. Transient dynamics is associated with short term loading cases and high speed problems. LS-DYNA has a large library for material models and different element types that can be used to accurately represent real life problems. The following sections provide the details of the software package.

3.3 LS-DYNA Solver

The equation of motion (Equation 2.4) discussed in section 2.4 can be solved using LS-DYNA by performing explicit or implicit analysis. Explicit analysis is the default option in LS-DYNA and widely used as the implicit analysis approach is computationally expensive for problems that needs to be discretized into small finite elements.

3.3.1 Explicit Analysis

The explicit analysis in LS-DYNA is based on the central difference method that is used to solve the equation of motion given in Equation 3.1:

$$m\ddot{u}_i + c\dot{u}_i + k(t)u_i = F_i \quad \langle 3.1 \rangle$$

where, $u_i(t = 0)$, $\dot{u}_i(t = 0)$ and $\ddot{u}_i(t = 0)$ are the initial conditions of the problem and represent displacement, velocity and acceleration, respectively. The procedure to solve the equation of motion is presented below as explained by Kwon and Bang (2000).

The central difference method employs Taylor's series expansion to find the previous and next displacements, u_{i-1} and u_{i+1} , respectively as follow:

$$u_{i+1} = u_i + h\dot{u}_i + \frac{h^2}{2}\ddot{u}_i + \dots \quad \langle 3.2 \rangle$$

$$u_{i-1} = u_i - h\dot{u}_i + \frac{h^2}{2}\ddot{u}_i + \dots \quad \langle 3.3 \rangle$$

where;

- u_i is the initial displacement at time, $t_i(t = 0)$.
- $h = t_{i+1} - t_i = \Delta t$, which is the time increment.

By taking the first two terms from Equations 3.2 and 3.3 and subtracting the second term from the first, the initial velocity, $\dot{u}_i(t = 0)$ is found as indicated below:

$$\dot{u}_i = \frac{1}{2h} (u_{i+1} - u_{i-1}) \quad (3.4)$$

Adding Equation 3.2 to 3.3 leads to the calculation of the initial acceleration, $\ddot{u}_i(t = 0)$:

$$\ddot{u}_i = \frac{1}{h^2} (u_{i-1} - 2u_i + u_{i+1}) \quad (3.5)$$

Substituting Equations 3.4 and 3.5 in Equation 3.1:

$$m \left[\frac{1}{\Delta t^2} (u_{i-1} - 2u_i + u_{i+1}) \right] + c \left[\frac{1}{2\Delta t} (u_{i+1} - u_{i-1}) \right] + k(t)u_i = F_i \quad (3.6)$$

Solving for u_{i+1} :

$$u_{i+1} = \left\{ \frac{1}{\frac{m}{\Delta t^2} + \frac{c}{2\Delta t}} \right\} \left[\left\{ \frac{m}{\Delta t^2} - k \right\} \{u_i\} + \left\{ \frac{c}{2\Delta t} - \frac{m}{\Delta t^2} \right\} u_{i-1} + F_i \right] \quad (3.7)$$

The initial conditions $u_i(t = 0)$ and $\dot{u}_i(t = 0)$ are known prior to solving the problem. To find, u_{i-1} , the acceleration, $\ddot{u}_i(t = 0)$, needs to be determined first from Equation 3.1:

$$\ddot{u}_i = \frac{1}{m} [F(t = 0) - c\dot{u}_i - ku_i] \quad (3.8)$$

From Equation 3.3, u_{i-1} :

$$u_{i-1} = u_i - \Delta t \dot{u}_i + \frac{\Delta t^2}{2} \ddot{u}_i \quad (3.9)$$

The accuracy of this method depends on the selection of the time increment, Δt . The smaller the time increment is, the more accurate the method becomes. There are some factors that need to be considered when selecting the time increment. These include the rate of change of the loading function, the complexity of the nonlinear behaviour of structure and the period of vibration of the structure in the free vibration mode (Clough and Penzien 2003).

3.3.2 Time Step

The calculation of time step in LS-DYNA is dependent on the element dimension and the material sound speed (LSTC 2006). The critical time step, Δt_e , for Hughes-Liu beam elements, which is used in the current project, is calculated as follow:

$$\Delta t_e = \frac{L}{c} \quad \langle 3.10 \rangle$$

where L is the length of the beam element and c is the speed wave, computed from Equation 3.11:

$$c = \sqrt{\frac{E}{\rho}} \quad \langle 3.11 \rangle$$

where E is young's modulus and ρ is the specific density of the beam element.

For 8-node solid element:

$$L = \frac{V_e}{A_{e_{max}}} \quad \langle 3.12 \rangle$$

where V_e is the volume of the solid element and $A_{e_{max}}$ is the area of the largest side of the solid element. The speed wave for elastic materials is given by:

$$c = \sqrt{\frac{E(1-\nu)}{(1+\nu)(1-2\nu)\rho}} \quad \langle 3.13 \rangle$$

where ν is Poisson's ratio.

For a stable numerical solution, LS-DYNA calculates the critical time step for every single element at each time step during the stress and force update process in solving the equation of motion. It then selects the minimum critical time step value which is used at every step in the simulation, as shown in Equation 3.14.

$$\Delta t^{n+1} = a. \min[\Delta t_1, \Delta t_2, \Delta t_3, \dots, \Delta t_N] \quad \langle 3.14 \rangle$$

where, N is the number of elements and a is a stability factor, which is set equal to 0.9 by default.

The user has an option of inputting the time step value or using the critical time step. If the inputted value is larger than the critical time step calculated by LS-DYNA, critical time step is used (LSTC 2006).

3.4 Reinforced Concrete Modelling

3.4.1 Concrete

3.4.1.1 Concrete Element

Three dimensional solid mesh elements (***SECTION_SOLID**) are widely used to model concrete for analysis of reinforced concrete structures with LS-DYNA. The 3-D mesh elements can have four or more nodes. The eight-node hexahedron solid mesh element used to model concrete in the current investigation can be seen in Figure 3-1.

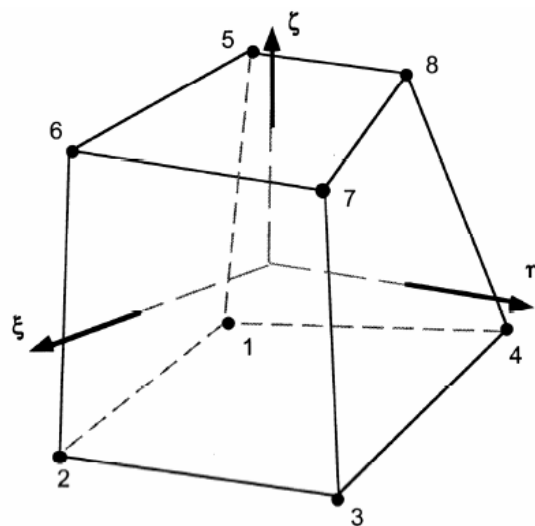


Figure 3-1: Eight-Node Solid Element (LSTC 2006)

There is a wide range of solid element formulations available in LS-DYNA for modelling concrete. These elements can have one or more volume integration points. It is generally recommended to use fully integrated elements to control zero energy modes (LSTC 2006). The cost of using fully integrated elements can be high as the computational time increases substantially. Also, zero energy modes can be controlled by performing a parametric study on the hourglass coefficient,

which is explained in the subsequent section. These solid elements work well for problems involving excessive deformations as in the case of blast events.

3.4.1.2 Concrete Material

Concrete is known to be a nonlinear material that exhibits inelastic behaviour, especially under extreme loading. Numerous constitutive models were developed in the past to predict the behaviour of concrete under axial, lateral and cyclic loading. Concrete constitutive material models can be categorized into elastic models, plastic models, elasto-plastic models, continuum models, endochronic models, and fracture models. LS-DYNA offers a number of constitutive material models for concrete. The selection of a material model depends on the fundamental behaviour of constitutive models and their effects on the problem at hand. The primary factors for blast analysis are post peak softening, stiffness degradation, shear dilation, confinement, and strain rate effects (Wu et al. 2012).

There are three material models in LS-DYNA that are commonly used by researchers for blast load analysis. These are listed below:

- *MAT072R-CONCRETE_DAMAGE_REL3
- *MAT084/85-WINFRITH_CONCRETE
- *MAT159-CSCM_CONCRETE

***MAT159-CSCM_CONCRETE** has been selected for all simulations in the current research project. The CSCM (continuous surface cap model) was first derived for assessing safety of roadside concrete structures (Murray 2007). Subsequently, it was implemented in LS-DYNA to evaluate response of reinforced concrete buildings to dynamic loads. It is also used to assess concrete damage under such loads. The required input for this model consists of the elastic modulus, tensile and compressive strengths, as well as softening and hardening parameters. The user has the option of inputting these properties from available test data. Alternatively, the automatic data generation option can be used. Laboratory test data used to fit the material model for the latter option, which has been implemented in LS-DYNA to generate the required parameters. These default values are intended for concretes with unconfined concrete strengths in the range of 20 MPa to 58 MPa, and aggregate sizes in the range of 8 mm to 32 mm (Murray 2007).

The model requires only the input of concrete density, unconfined compressive strength and aggregate size. It has three invariant yield surfaces as shown in Figure 3-2 (a). They define yield surfaces at the onset of plastic stage. The yield surface is a combination of shear failure surface and hardening cap, connected continuously and smoothly as shown in Figure 3-2 (b).

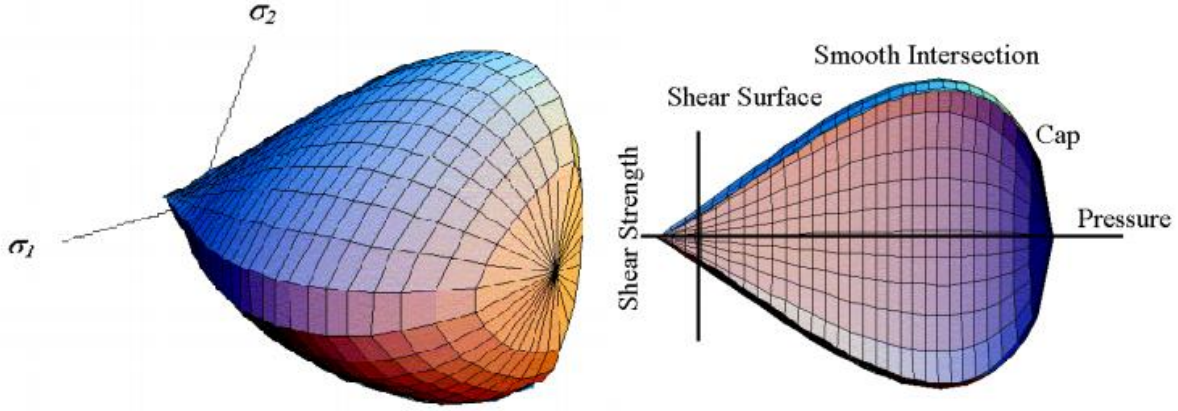


Figure 3-2: a) 3-Dimensional view of the model yield surface b) 2-Dimensional view of the model yield surface (Murray 2007)

The yield surface is derived from three invariants of deviatoric stress tensors; J_1, J'_2 and J'_3 , and the hardening cap parameter, κ (Murray 2007). The three invariants are functions of the deviatoric stress tensors, S_{ij} and pressure, P . The yield surfaces can be calculated as illustrated in Equation 3.15:

$$f(J_1, J'_2, J'_3, \kappa) = J'_2 - R^2 F_f^2 F_c \quad (3.15)$$

Where; F_f is the shear failure surface, F_c is the hardening cap and R is the Rubin three invariant reduction factor.

Concrete strength in tensile and low confinement regions is determined from the shear failure surface, F_f , which is a function of the first invariant of the deviatoric stress tensor, J_1 . It is computed as follows:

$$F_f(J_1) = \alpha - \lambda e^{-\beta J_1} + \theta J_1 \quad (3.16)$$

Where α, β, λ and θ are established by conducting triaxial compression tests and fitting the data to the model surface.

Concrete strength in tension: Concrete strength in tension is calculated by multiplying Rubin scaling function, R and the shear failure surface, F_f , which is named as $Q_2 F_f$. Q_2 is calculated as follow:

$$Q_2 = \alpha_2 - \lambda_2 e^{-\beta_2 J_1} + \theta_2 J_1 \quad (3.17)$$

Where $\alpha_2, \beta_2, \lambda_2$ and θ_2 are computed by conducting triaxial extension tests.

Concrete strength in low to high confinement regions: Concrete strength in low to high confinement regions is reflected by the cap hardening surface, F_c . This hardening surface is a function of the first invariant of the deviatoric stress tensors J_1 and the hardening cap parameter, κ . It is calculated as follow:

$$F_c(J_1, \kappa) = 1 - \frac{[J_1 - L(\kappa)][|J_1 - L(\kappa)| + J_1 - L(\kappa)]}{[X(\kappa) - L(\kappa)]^2} \quad (3.18)$$

Where $L(\kappa)$ is κ when $\kappa > \kappa_0$. Otherwise, κ_0 and $X(\kappa)$ can be established by fitting the unknown variables to pressure versus volumetric strain curve. The cap hardening surface models the effect of pore collapse on concrete strength.

Post Peak Strength: The post-peak strength is determined from damage accumulation. This region of the relationship incorporates i) strain softening due to strength degradation, and ii) modulus reduction due to loading and unloading.

The model uses a scalar damage parameter, d , which transforms undamaged stress to damaged stress. The scalar damage value d used in the model ranges between 0 and 1 ($d = 0$ when no damage and $d = 1.0$ when full damage). Damage is characterized in two regions:

- **Brittle Damage:** This damage occurs when pressure is in tension. It accumulates based on the maximum principle tensile strain.

- **Ductile Damage:** This damage occurs when the pressure is compression. It accumulates based on the total strain.

Damage in the model is initiated as soon as the plastic state is reached. The initial damage surface coincides with the plastic shear surface at peak strengths. The damage starts to accumulate at the peak strength, and when the plastic strain increases as shown in Figure 3-3. The scalar damage is calculated as outlined in Murray (2007).

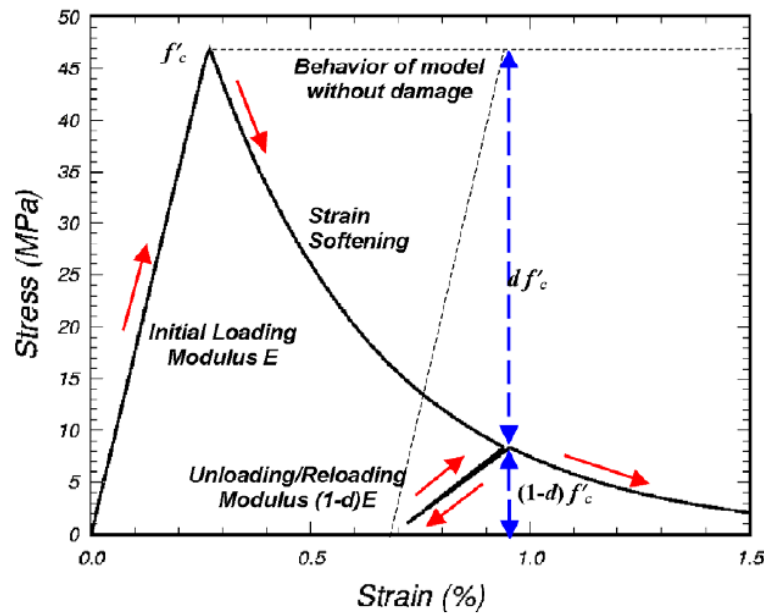


Figure 3-3: Stain Softening and Modulus Reduction in Concrete (Murray 2007)

Figure 3-4 illustrates a typical scalar damage output by the model during simulations. Blue represents the state of undamaged element ($d = 0$), indicating that the strain levels are less than those at peak tensile or compressive strengths. Red represents the state of fully damaged element ($d = 1$) with 0 residual strength. Values in between represent residual strength.

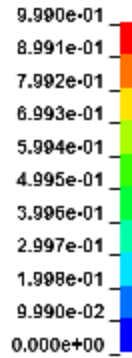


Figure 3-4: Typical Scaler Damage Output for *159-CSCM_CONCRETE

Figure 3-5 illustrates a typical stress-strain relationship generated by the material model in LS-DYNA for 30 MPa concrete under static loading.

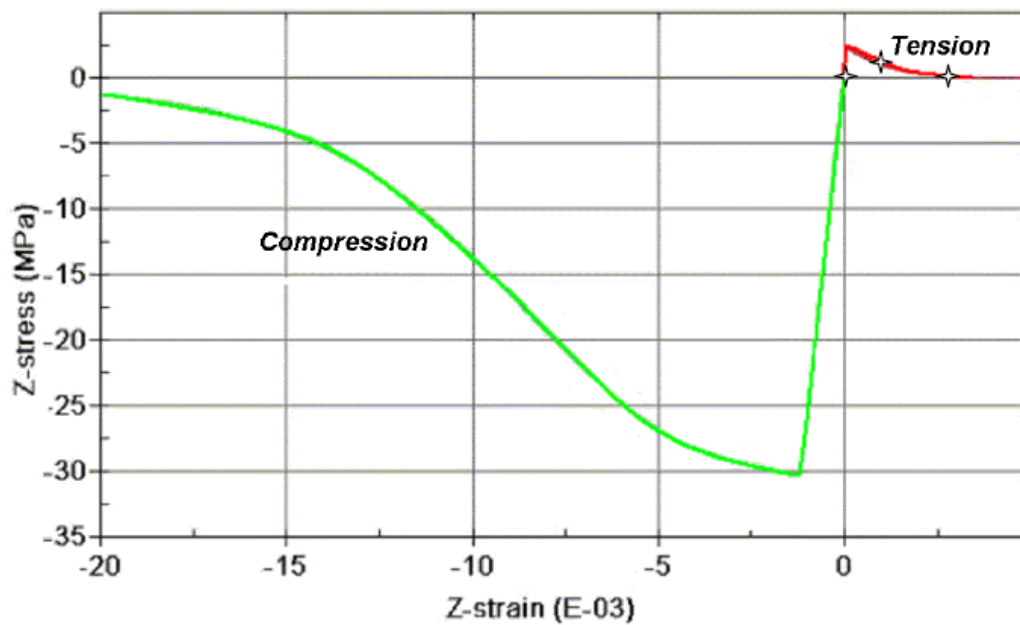


Figure 3-5: Typical Stress-Strain Relationship for 30 MPa Concrete (Murray 2007)

Figure 3-6 illustrates the effect of concrete confinement on 28 MPa concrete for a single concrete element under triaxial compression test (Murray et al. 2007) using LS-DYNA. The figure illustrates the effects of three levels of lateral confining pressures on concrete.

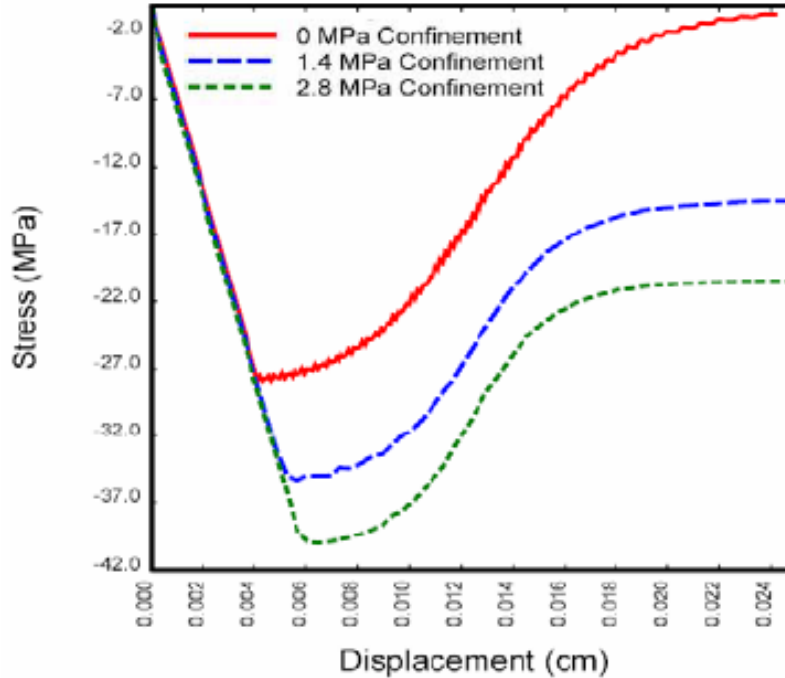


Figure 3-6: Confining Pressure Effect (Murray et al. 2007)

This material model has the option of turning strain rate effects on or off. The strain rate effects are turned on for the analyses conducted in the current study. The damage levels shift up with increase in strength due to the strain rate effects.

The element is fully damaged when the scalar damage is equal to a value of 1.0. The user has the option to delete only fully damaged elements through the maximum principal strain criteria. The elements are deleted from the simulation when **ERODE=1.0** is reached. To delete elements at a maximum principal strain value of 0.05, the user inputs **ERODE = 1.05**. Deleting elements when they reach the principal tensile strain is desirable as elements with low stiffness can cause computational problems (Murray 2007). ERODE value between 1.05 - 1.1 is recommended (Murray et al. 2007). The value of 1.05 was used in the current research project.

3.4.2 Steel Reinforcement

3.4.2.1 Steel Element

Two-dimensional beam mesh elements are widely used to model longitudinal and transverse reinforcing bars in concrete. LS-DYNA provides a wide range of beam element formulations

(*SECTION_BEAM), such as the Hughes-Liu element with cross section integration, the Belytschoko-Schwer element with full cross section integration; as well as truss, discrete beam/cable, and many other elements. Hughes-Liu element with the cross-section integration beam element formulation, having 2×2 Gauss integration points has been adopted in the current investigation to model the circular cross section of rebars. The element has three translational and three rotational degrees of freedom at each node. The main advantage of the Hughes-Liu beam formulation is its compatibility with the solid concrete elements. It is generated from the 8-node solid element, as shown in Figure 3-7, which makes the beam element compatible with the solid element (LSTC 2006).

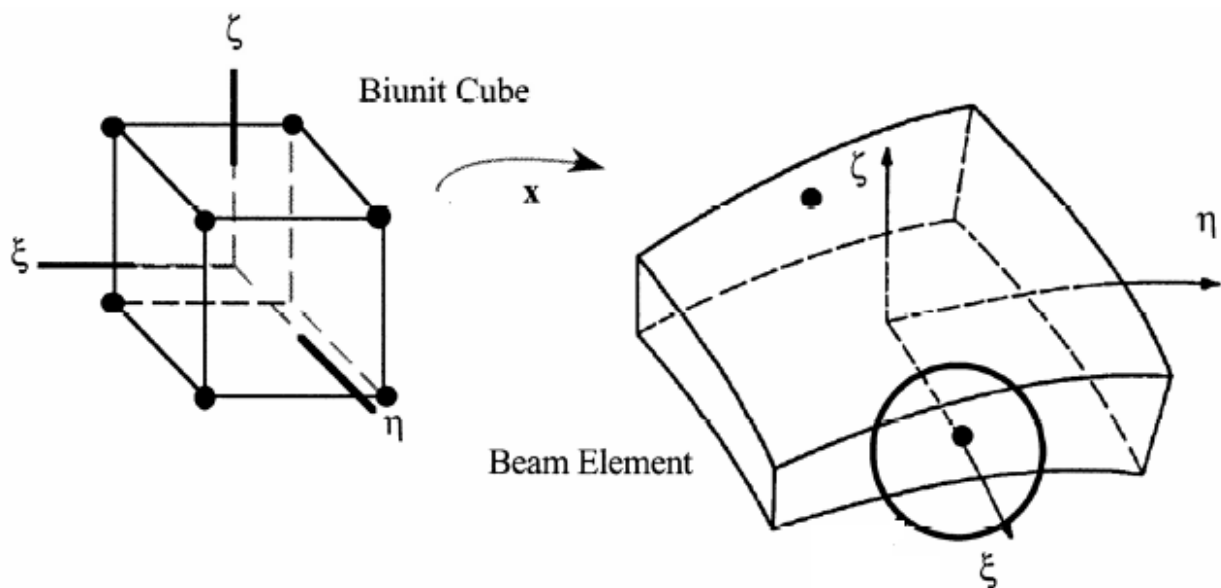


Figure 3-7: Hughes-Liu Beam Element (LSTC 2006)

3.4.2.2 Steel Material

LS-DYNA provides users with a variety of material models that can be used to model steel reinforcement bars. Materials that can be used include:

- *MAT003-PLASTIC_KINEMATIC
- *MAT024-PIECEWISE_LINEAR_PLASTICITY
- *MAT098-SIMPLIFIED_JOHNSON_COOK

The material used in this study is ***MAT003-PLASTIC_KINEMATIC**, which is an elasto-plastic material model that was developed with either isotropic hardening, kinematic hardening or combination of both (LSTC 2015). Kinematic hardening takes Bauschinger effect into account when compared with isotropic hardening, making it useful when a structural component is subjected to lateral cyclic loading. The hardening effect can be taken into account using a parameter called β . When $\beta = 0$, kinematic hardening is activated, and when $\beta = 1$, isotropic hardening is activated as shown in Figure 3-8.

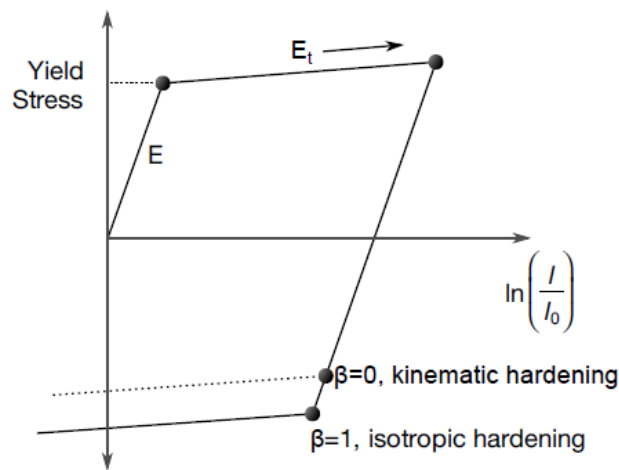


Figure 3-8: *MAT003-PLASTIC_KINEMATIC Material Model (LSTC 2015)

Strain rates are user options. The strain rate effect is based on Cowper-Symonds model that was explained in Section 2.5.2. Input requirements for this model are; material density, Poisson's ratio, elastic modulus, yield stress, tangential modulus, and rupture strain, representing a typical bi-linear stress-strain relationship that can be used to model the steel reinforcing bar.

3.4.3 Coupling of concrete and steel

To transmit forces and strains between two different element types (solid and beam), there has to be full bond between concrete and reinforcement. There are three methods available in LS-DYNA, as explained by Len Schwer (2014), that can be implemented in defining the coupling algorithm between concrete and steel. The methods are; i) smeared reinforcement, ii) shared nodes, and iii) constraint method. Smeared reinforcement method works only if the structural component stays within the elastic range, for small deformations. Shared nodes method works if the nodes of the

concrete and steel elements are at the exact same location. This limits the use of shared nodes method to only a few cases, as it can't be used if the concrete and steel nodes do not coincide. Lastly, the constraint method overcomes the limitation of the shared node method by allowing the user to construct the reinforcement and concrete mesh elements freely with different number of nodes. The constraint method restricts the concrete and steel mesh movement to be the same, by constructing a system of constraints internally (Schwer 2014). In the constraint method both acceleration and velocity are constrained at the nodes.

CONSTRAINED_LAGRANGE_IN_SOLID** keyword allows the coupling process by selecting concrete as the Master and the reinforcement as the Slave. Constraining acceleration and velocity at the nodes are done by choosing the constraint method **CTYPE=2**. Recently, a new keyword has been added to LS-DYNA (CONSTRAINED_BEAM_IN_SOLID**) to overcome some limitations that has been found using the old algorithm (LSTC 2015). In ***CONSTRAINED_BEAM_IN_SOLID**, both the master and the slave are defined as previously. **CTYPE=2** is included internally within the keyword. The coupling algorithm can be done at the beam nodes only, or within a couple of different points in the beam element.

3.5 Boundary Conditions

In LS-DYNA, boundary conditions can be applied to a certain node or group of nodes using the keyword ***BOUNDARY_SPC**. There are three translational degrees of freedom and three rotational degrees of freedom, along x-axis, y-axis and z-axis. Fixed support condition can be utilized by setting all the degrees of freedoms to 1. Simple support condition can be specified by setting only the translational degree of freedom to 1. One can choose a certain node or a set of nodes to move in only one direction. This is a useful tool that is used to model the effect of axial load on a column.

3.6 Axial Load Application

Axial load can be applied as a pressure segment in LS-DYNA on top of a column using dynamic relaxation. ***LOAD_SEGMENT_SET** keyword is used to apply pressure on the top segment of a column. Therefore, the top column nodes are allowed to move in the longitudinal direction to allow

axial movement associated with axial loading. The pressure is defined using ***DEFINE-CURVE** keyword. The axial load can be applied on the column prior to the application of blast loading or prior and during the application of blast loading. Axial load in this study is applied linearly, i.e., the axial load is equal to zero initially, and reaches its maximum value after 15 ms. It remains constant for 65 ms. The columns are ensured to remain elastic before the application of blast loads. Then, the axial load is held constantly during blast loading

3.7 Blast Load Simulation

***LOAD-BLAST_ENHANCED** keyword is available in LS-DYNA to activate the Conventional Weapons Effects Program (CONWEP) data to generate pressure-time history associated with charge weight and stand-off distance. The blast load is then applied on the structure using this time history. As mentioned earlier CONWEP data is the basis of empirical charts presented in UFC 03-0340-02. The empirical charts use blast curves that relate blast wave parameters and the distance between the explosive source and the structure. The empirical equations used in CONWEP are valid for scaled standoff distances of (LSTC 2015):

- $0.147 \text{ m/kg}^{1/3} < Z < 40 \text{ m/kg}^{1/3}$ for spherical air burst explosion
- $0.178 \text{ m/kg}^{1/3} < Z < 40 \text{ m/kg}^{1/3}$ for hemispherical air burst explosion

***LOAD-BLAST_ENHANCED** keyword requires the input of the mass of the explosive in TNT equivalent, standoff distance in three dimensions, and the blast type. There are four blast types available in LS-DYNA:

Type 1) Hemispherical surface burst where the initial shockwave is reflected by the ground.

Type 2) Spherical air burst where the explosive is free in the air, and the shockwave doesn't reflect (from ground surface) until the wave interacts with a structure.

Type 3) Air burst is just like the spherical air burst but moves with a non-spherical warhead.

Type 4) Air burst just like the spherical air burst which can get reflected from the ground surface.

For scaled standoff distances less than the above, it is recommended to perform computational fluid dynamic (CFD) analysis. This method is known to be very computationally intensive because it requires the use of a very small mesh element size for air and explosive, which are modelled explicitly in the CFD analysis. The main advantage of CFD analysis is that it can capture the blast wave clearing around the structure, hence giving more accurate predictions of blast loading parameters. Besides, the shape and the triggering point within the explosive can be done in LS-DYNA, which helps in determining the exact behaviour of the shock wave instead of assuming a spherical or hemispherical shape as is done in most of the cases.

***LOAD-BLAST_ENHANCED** keyword is used in this thesis. Because, wave clearing around the structure would not cause a huge impact on structural response of exterior reinforced concrete columns on building façade. Scaled standoff distances within the limits above will be used in this study.

3.8 Hourglass Control

Zero energy modes or hourglassing is a nonphysical deformational behaviour that occurs in elements without generating any stresses. As mentioned earlier, it happens in elements that are not fully integrated. Figure 3-9 illustrates typical hourglass deformation in solid elements. To control hourglass energy modes; hourglass viscosity type and hourglass coefficient can be added to the model. These additions help in applying additional internal energy to resist the hourglass energy developed during analysis. There are two hourglass viscosity forms available in LS-DYNA; i) viscous and ii) stiffness form. The viscous form generates forces proportional to nodal velocities, and the stiffness form generates forces (i.e. internal energy) proportional to nodal displacements. There are three types of viscous forms; i) type 1 (standard), ii) type 2 (Flanagan-Belyschko integration) and iii) type 3 (Flanagan-Belyschko with exact volume integration). There are two types for the stiffness form; i) type 4 (Flanagan-Belyschko) and ii) type 5 (Flanagan-Belyschko with exact volume integration).

After selecting the hourglass viscous type, one needs to select an appropriate value for the hourglass coefficient. This can be done by performing a parametric study based on the LD-DYNA recommendations. The LS-DYNA theory manual recommends that the hourglass energy at any

time in the simulation to be less than 10% of the peak internal energy globally, and for each part. To include the hourglass energy in the energy balance equation during the run, HGEN should be set to 2 in the *CONTROL_ENERGY keyword. The global hourglass energy is the hourglass energy for the whole system and can be viewed from the GLSTAT output file. The hourglass energy for each type of element can be viewed from MATSUM output files.

In this thesis, hourglassing control was done on all the simulations by either using fully integrated elements or by using hourglass stiffness form, which is able to reduce the accumulation of hourglass deformations.

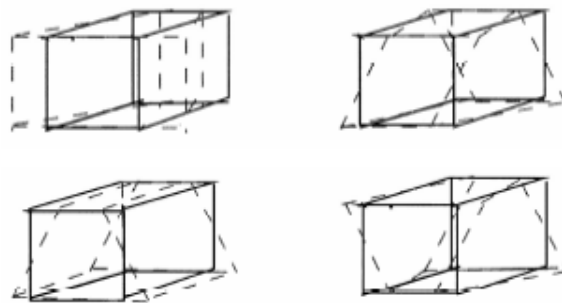


Figure 3-9: Hourglassing in Solid Elements (LSTC 2006)

3.9 Numerical Validation

3.9.1 General

In order to confirm the reliability of material models, blast loading, and the overall performance of the numerical model selected for determining the response of reinforced concrete columns to blast loading, validation of the numerical model was done with experimental and numerical results provided by Woodson and Baylot (1999). Limited published data is available for reinforced concrete columns under extreme external blast loading case, especially at closer distances. The closest possible experiment to confirm the reliability of the numerical model is by Woodson and Baylot (1999).

3.9.2 Experimental Procedure and Results

As mentioned in the literature review, Woodson and Baylot (1999) investigated response of reinforced concrete columns in buildings under blast loading experimentally. Four two-storey, quarter-scale models were tested with explosives at different standoff distances. The height of the test frames was 2000 mm. Slabs in the frame building were 41 mm thick. Exterior columns were square with cross sectional dimensions of 89 mm x 89 mm. Concrete density and unconfined compressive strength were 2225 kg/m³ and 43.0 MPa, respectively. Aggregate size and concrete cover were 9.5 mm. The exterior columns were reinforced with 8 longitudinal reinforcement steel bars with a diameter of 6.4 mm. Deformed steel wires with diameter of 2.9 mm and smooth steel wires with diameter of 2.0 mm were used as transverse reinforcements. Transverse reinforcement was spaced at 89 mm. Table 3-1 illustrates mechanical properties for the reinforcement used.

Table 3-1: Mechanical Properties of Steel Used in Woodson and Baylot Experiment (1999)

Reinforcement	Area, (mm²)	Yield Stress, (MPa)	Ultimate Stress, (MPa)	Rupture Strain, (%)	Calculated Tangent Modulus, (MPa)
Longitudinal Bars	32.3	450	513	0.18	354
Transverse Deformed Wires	6.5	400	609	0.18	1174
Transverse Smooth Wires	3.2	440	513	0.18	410

A total of five experiments were carried out to evaluate the exterior column response of the building under blast loading. The explosive type that was used was C4 with a mass of 7.1 kg at a height of 0.229 m from the ground surface in front of the middle column (shown in Figure 2-19). The first model was tested with a standoff distance of 1.5 m which resulted in very small cracks and deformations. Therefore, the same quarter scale model was used for the second experiment with a standoff distance of 1.067 m. The choice of 1.0 m was done after trial and error numerical analysis conducted by the authors for various standoff distances so the column doesn't fail but experiences some damage.

The authors' numerical analysis yielded the following for lower storey exterior column:

- Peak displacement of 7.75 mm.
- Residual displacement of 5.84 mm.
- Significant damage to the lower and upper parts of the column.

Meanwhile, the actual experiment yielded the following:

- Peak displacement of 12.7 mm.
- Residual displacement of 6.35 mm.
- Lots of cracks were observed.

Figure 3-10 illustrates the displacement measurements from the experimental and numerical test models.

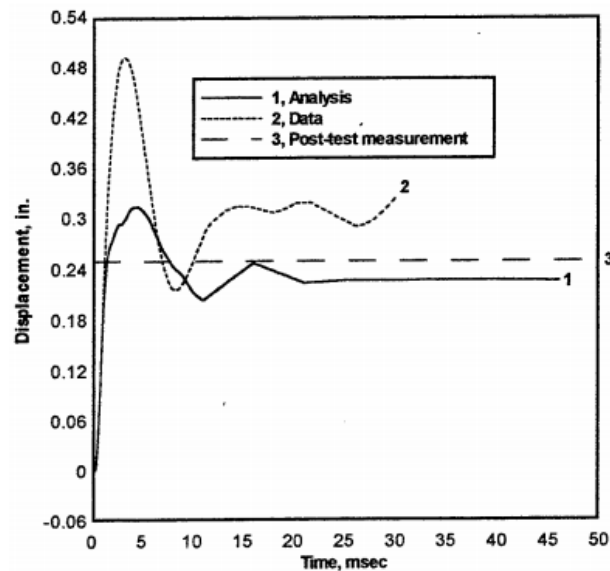


Figure 3-10: Displacement Time Histories the Experimental and Numerical Tests (Woodson and Baylot 1999)

Peak pressure and corresponding impulse were approximately 16.55 MPa and 2.86 MPa-ms obtained from the numerical analysis near the base of the column.

3.9.3 Numerical Model

Concrete was modelled with 25 mm 8-noded solid brick mesh elements (25 mm in all directions). The material model that was used for concrete was ***MAT159-CSCM_CONCRETE**. Steel

reinforcement was modeled using 25 mm 2-noded beam mesh elements. The material model that was used for steel reinforcements was ***MAT003-PLASTIC_KINEMATIC**. The material properties for concrete and steel required by LS-DYNA are presented in Table 3-2 and 3-3, respectively.

Table 3-2: Concrete Properties for Validation Stage

Property	
Density, (kg/m³)	2225
Compressive Strength, (MPa)	43.0
Aggregate Size, (mm)	9.5

Table 3-3: Steel Properties for Validation Stage

Reinforcement	Density, (kg/m³)	Yield Stress, (MPa)	Elastic Modulus, (MPa)	Tangent Modulus, (MPa)	Poisson's Ratio	Rupture Strain, (%)
Longitudinal Bars	7,800	450	200,000	354	0.3	0.18
Transverse Deformed Wires	7,800	400	200,000	1,174	0.3	0.18
Transverse Smooth Wires	7,800	440	200,000	410	0.3	0.18

The exterior middle column from the two storey building shown in Figure 2-19 was modelled. The height of the column was 2000 mm from the base to the top end of the frame. The slab and roof were not modelled to save computational time.

The bottom nodes of the column were fixed in all directions to represent foundation fixity condition. Since the slab and roof were not modelled, restraints from the slab and roof level shown in Figure 3-11 were modeled as follow:

- Displacement was restrained from x-direction motion from the sides
- Displacement was restrained from y-direction motion from the back face

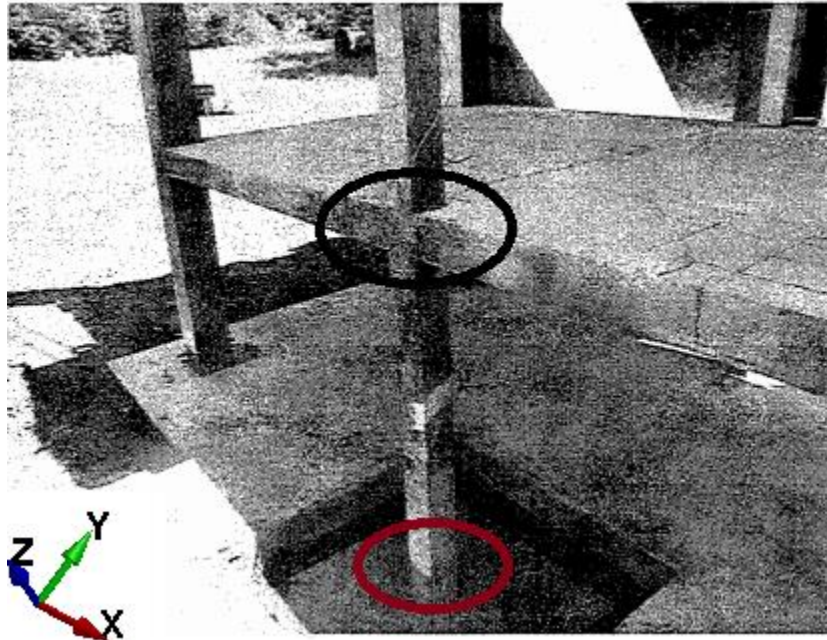


Figure 3-11: Slab and Roof Boundary Conditions (Modified from Woodson and Baylot 1999)

Axial pressure of 4.2 MPa was applied at the top of the column to simulate gravity loads. The charge was placed 1.067 m from the front face of the exterior column at a height of 0.229 m from the ground surface. Illustration of the full geometry, loading and boundary conditions is presented in Figure 3-12. The explosive type that was used in the experiment was C4. The TNT equivalent factor of 1.19 was used for the impulse to incorporate the effective charge weight. The main reason for using TNT equivalent factor for impulse instead of pressure was the fact that the structural response was reportedly more sensitive to impulse (PDC TR-06-01 Rev1 2008). The charge mass used in this model was 8.45 kg of TNT equivalent.

3.9.4 Results and Comparisons

A peak reflected pressure of 16.1 MPa, reflected impulse of 2.57 MPa-ms with a positive phase duration of 0.7 ms, were recorded near the base of the column. The pressure and impulse time histories are shown in Figure 3-13. The experimentally recorded blast parameters show slight differences when compared with the analytical values, which resulted in peak reflected pressure of 16.55 MPa and impulse of 2.86 MPa-ms. The main reasons for the differences are:

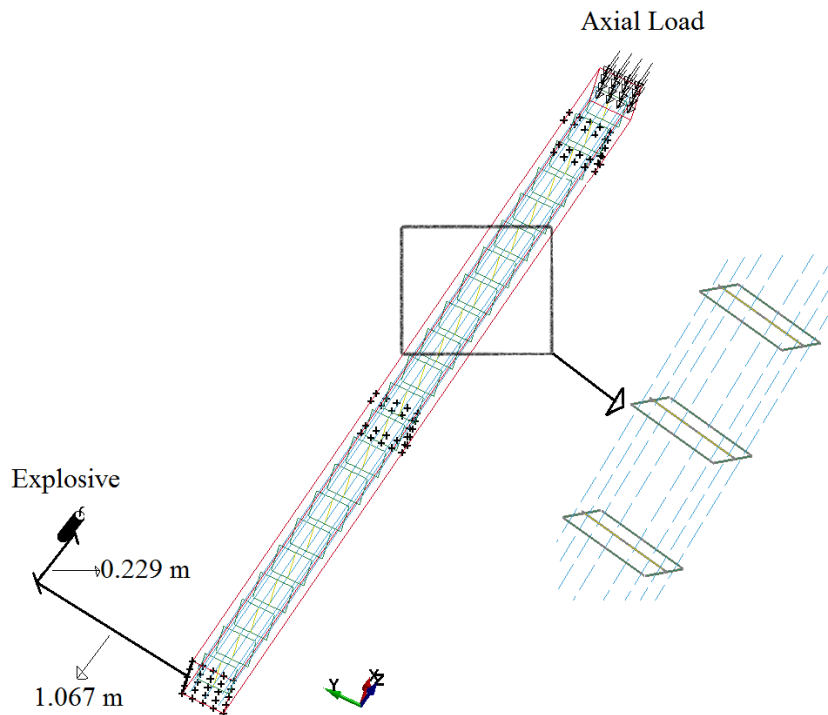


Figure 3-12: Validation Model Properties

- Explosive is not modelled explicitly. Different explosive shapes generate different shock wave shapes. The wave shape that was considered in this model was spherical because the explosive was located at 0.229 m above the ground surface.
- Wave clearing was not taking into account when using CONWEP data.

However, the two sets of values are very similar. The results show very close peak reflected pressure and impulse values experimentally and numerically. ***LOAD-BLAST_ENHANCED** blast parameter generator used in this model shows very reliable and accurate results in terms of pressure and impulse.

Displacement of 13.7 mm and residual displacement of 9.0 mm were predicted for mid-storey by the numerical model. Figure 3-14 illustrates the displacement time history for the middle exterior first storey column of the structure.

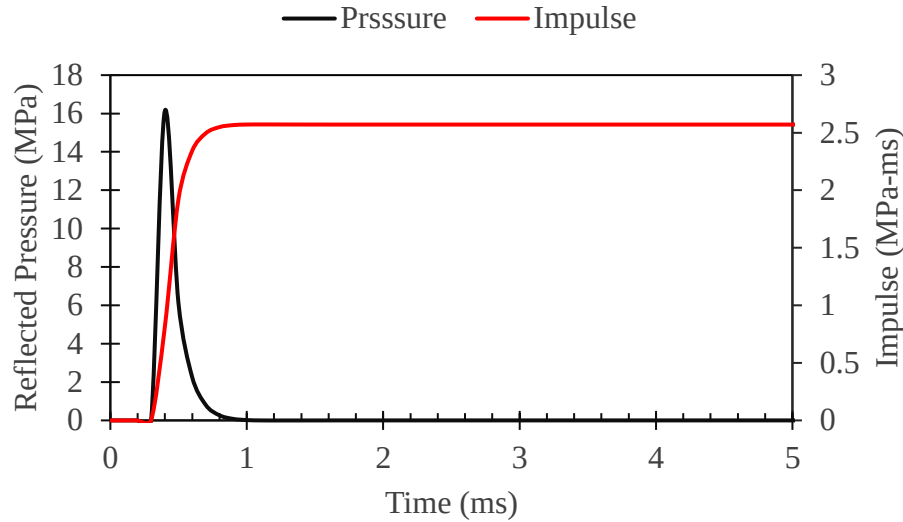


Figure 3-13: Reflected Pressure and Impulse Time Histories near Column Base for the Numerical Model

The comparison of the numerical results of 13.7 mm and 9.0 mm for peak and residual displacements with the corresponding experimental values of 12.7 mm and 6.35 mm shows reasonably good agreement, with some differences. These differences can be attributed to the following:

- The experimental set up represents an open frame structure, meaning pressure clearing around the column has an impact on the overall structural response. Pressure clearing reduces the net loading on the structure, hence this could be a possible reason for displacement variation between the numerical model and the analytical results. Also, side and rear pressures effect are not taken into consideration in this model.
- The analytical model did not include the entire structure. Additional resistance from the foundation, slab and roof are not taken into account.
- The material models used to model concrete and steel elements are not exact representatives of the materials used in the experiment.

It is noteworthy that the numerical analysis conducted by Woodson and Baylot (1999) showed less of an agreement with their test results, indicating 7.75mm and 5.84 mm for maximum and residual deflections.

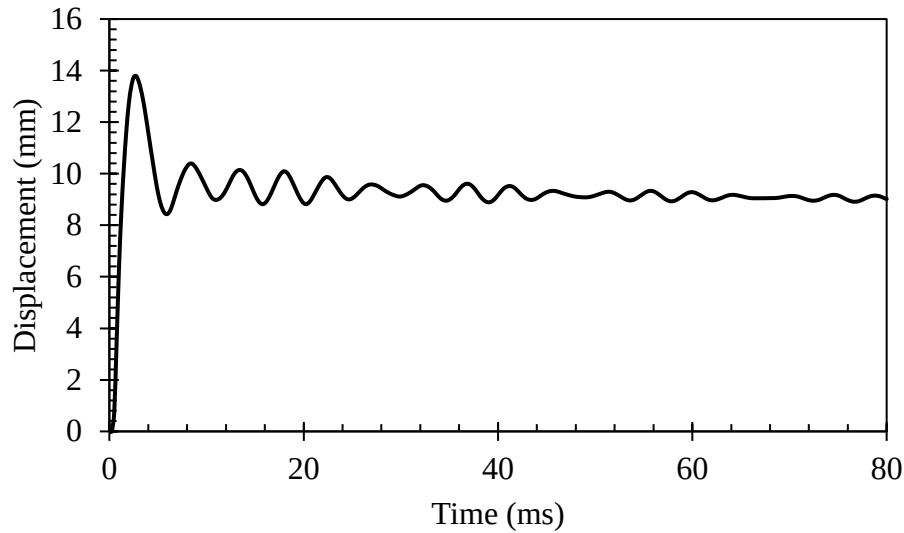


Figure 3-14: Displacement Time History from the Numerical Model

Damage levels indicated by the numerical model employed in the current investigation are illustrated in Figure 3-15, red showing high damage. The figure indicates that the damage concentrated near the mid height of lower column. This damage is due to the fact that the column displaced laterally under the blast load and attained its flexural capacity. This damage correlates well with the large number of wide cracks that was observed within the same vicinity of the test column in the experiment.

Since the façade behaviour under external extreme blast loading scenario is the objective of current research, pressure clearing effect would not be an issue in the analysis of the first storey columns forming part of a façade system. The pressure applied from the rear of the column (wraparound effect) can be ignored since the structure is enclosed by non-structural elements around the column. Furthermore, since the damage produced by the material models gave an accurate representation of the failure mode experienced by the column, the reliability of the material model to produce damage or failure modes under blast loading can be assured.

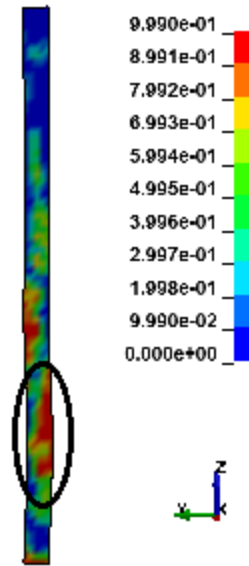


Figure 3-15: Damage Levels from the Numerical Model

4 Chapter 4

Building and Threat Selection

4.1 Introduction

A six-storey reinforced concrete building was selected to examine damage levels induced by various blast loading conditions on the external columns of the building. Different bomb sizes and standoff distances are selected to investigate failure mode possibilities associated with structural response. The prototype six-storey reinforced concrete moment resisting building was designed by Al-Sadoon (2016). The building represents a typical seismically deficient medium rise building built in the 1960's, located in Ottawa and Vancouver. It was designed according to the National Building Code of Canada (NBCC 1965) that didn't differentiate the seismic effect between eastern and western regions of Canada. Figure 4-1 illustrates the floor plan. Figures 4-2 and 4-3 illustrate the N-S and E-W elevation views, respectively. Structural and material properties are tabulated in Table 4-1.

Table 4-1: Structural and Material Properties for the Selected Building

Slab Thickness	110 mm
Main Beams	600 mm x 400 mm
Secondary Beams	300 mm x 350 mm
Interior Columns	500 mm x 500 mm
Exterior Columns	450 mm x 450 mm
Concrete Compressive Strength	30 MPa
Yield Strength of Reinforcements	400 MPa

4.2 Threat Scenarios

Explosive charges are placed outside the building as shown in Figure 4-4. The exterior frame of the building on gridline 8-(A-D) would be the most vulnerable frame to the explosive. The elements of first storey are exposed to the highest loading conditions. Hence, the first storey columns of frame 8-(A-D) are considered for the investigation. Specifically, the exterior corner

column 8-A and exterior edge column 8-C are examined to assess damage levels induced by the explosives as shown in Figure 4-5.

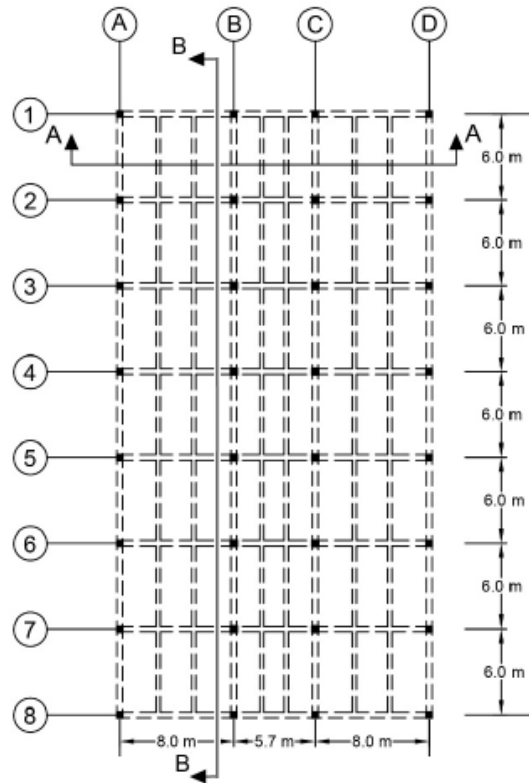


Figure 4-1: Floor Plan View (Adapted from Al-Sadoon 2016)

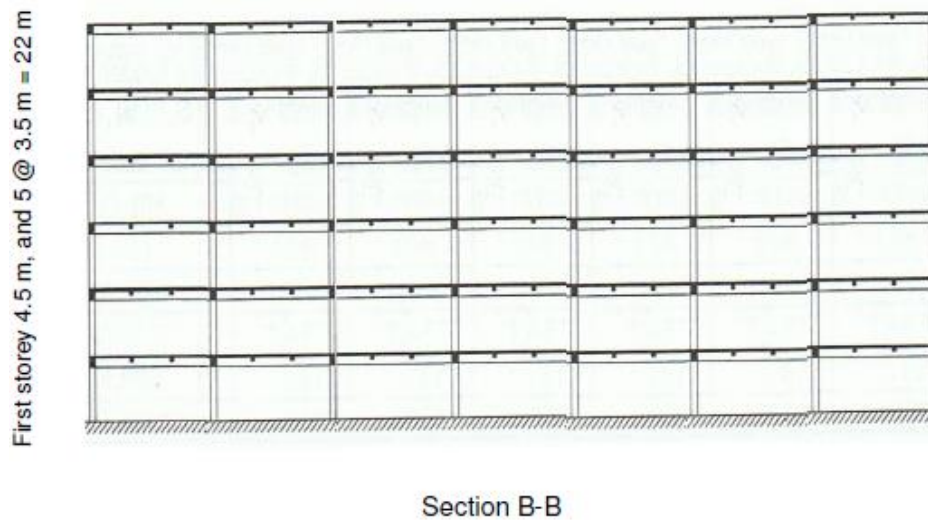


Figure 4-2: N-S View (Al-Sadoon 2016)

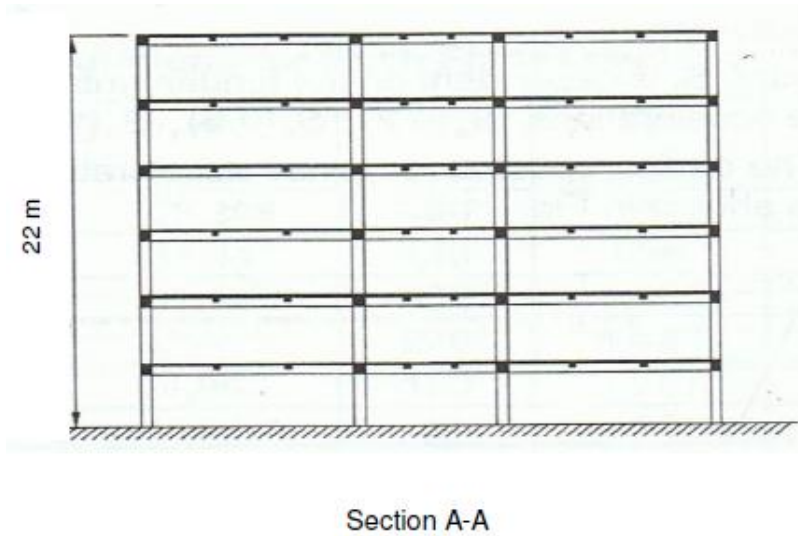


Figure 4-3: E-W View (Al-Sadoon 2016)

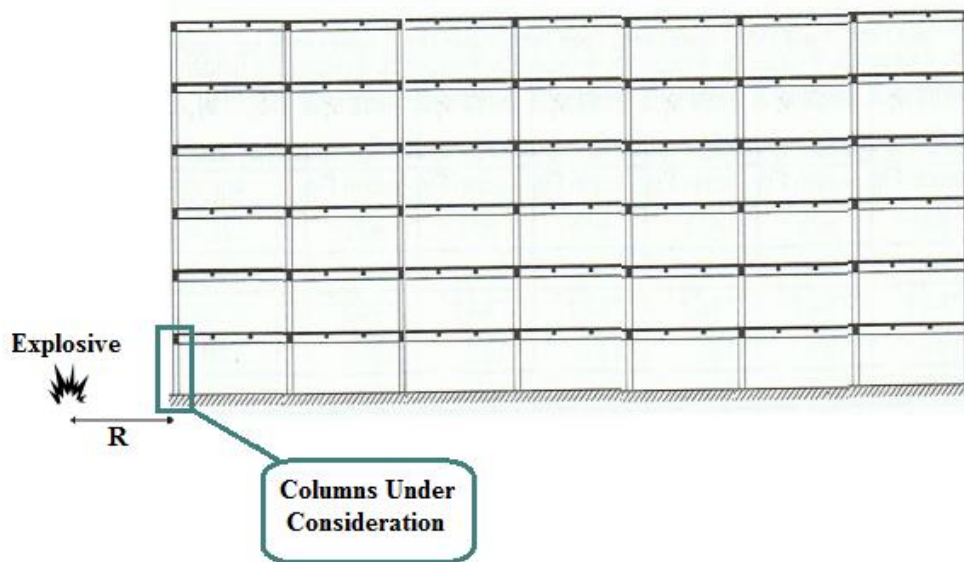


Figure 4-4: Explosive Location (Modified from Al-Sadoon 2016)

Three different explosive charge masses are used in this study to evaluate the columns' dynamic response under blast loading.

- 1) Backpack bomb with 22.67 kg of equivalent TNT mass
- 2) Compact Sedan car bomb with 227 kg of equivalent TNT mass
- 3) Sedan car bomb with 454 kg of equivalent TNT mass

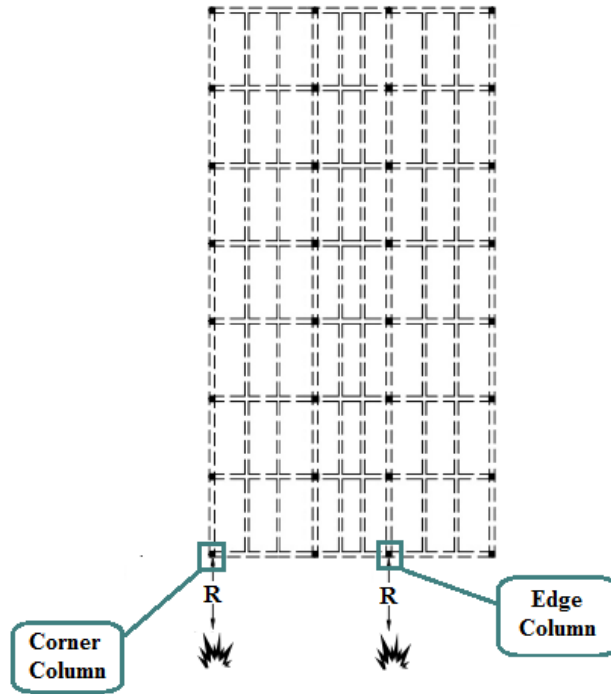


Figure 4-5: Columns under Consideration

Backpack bomb is placed at standoff distances of 0.5 m, 0.75 m, 1.5 m and 3.0 m. Compact sedan and sedan bomb vehicles are placed at standoff distances of 1.5 m, 3.0 m, 4.5 m, 6.0 m and 7.5 m. Table 4-2 shows the blast designation along with the scaled standoff distance, that are used in this study.

Table 4-2: Threat Designation

Blast Designation	Mass, (kg)	Standoff Distance, (m)	Scaled Standoff Distance, (m/kg ^{1/3})
BP1	22.67	0.5	0.177
BP2	22.67	0.75	0.265
BP3	22.67	1.5	0.530
BP4	22.67	3.0	1.060
CS1	227	1.5	0.246
CS2	227	3.0	0.492
CS3	227	4.5	0.738
CS4	227	6.0	0.984
CS5	227	7.5	1.229
S1	454	1.5	0.195
S2	454	3.0	0.390
S3	454	4.5	0.586
S4	454	6.0	0.781
S5	454	7.5	0.976

Figure 4.6 shows the cross-sectional geometry and the reinforcement arrangements for the columns investigated. Exterior corner column had 8 – 25M longitudinal steel re-bars, which corresponds to 1.98 % reinforcement ratio. Exterior edge column had 12 – 25M longitudinal steel re-bars which corresponds to 2.96 % reinforcement ratio. Both columns are square with cross section dimension of 450 mm x 450 mm. Nominal axial load of 597 kN for the corner column and 1194 kN for the edge column were considered for design.

The transverse reinforcement for both columns consisted of 10M Ties. The spacing of transverse reinforcement was chosen as another parameter for this study. Three spacing were considered; 450 mm, 225 mm and 112.5 mm, representing old construction, conventional newer construction and seismically detailed columns conforming to modern seismic codes of post-1985 era, respectively. Table 4-3 shows the column designs and their designations considered in this study. Each one of the six columns is subjected to the fourteen threat cases listed in Table 4-2.

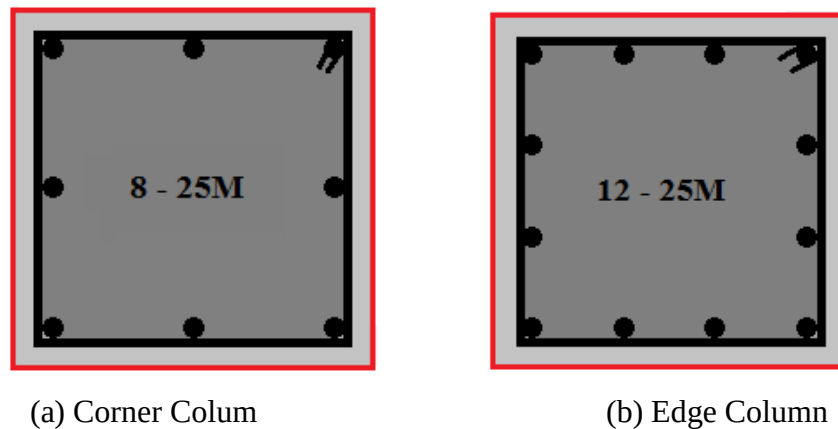


Figure 4-6: Exterior Columns Configurations

Table 4-3: Exterior Columns Properties

Column Designation	Number of Longitudinal Bars	Longitudinal Reinforcement Ratio, (%)	Transverse Reinforcement Spacing, (mm)	Transverse Reinforcement Ratio, (%)	Axial Load, (kN)
CC-TS1	8 – 25M	1.98	450	0.229	597
CC-TS2	8 – 25M	1.98	225	0.457	597
CC-TS3	8 – 25M	1.98	112.5	0.915	597
EC-TS1	12 – 25M	2.96	450	0.229	1194
EC-TS2	12 – 25M	2.96	225	0.457	1194
EC-TS3	12 – 25M	2.96	112.5	0.915	1194

4.3 The Analytical Model

Only the first storey column with its full height was modelled for analysis. Clear column height was 4.0 m and the cross-sectional properties were as presented earlier in Section 4.2. Figure 4-7 illustrates the meshed model configuration for corner and edge columns. Figure 4-8 illustrates the three transverse reinforcement spacings used in the models.

Concrete was modelled with solid cubical elements, consisting of ***MAT159-CSCM_CONCRETE** as the material model. Steel reinforcement was modeled using beam elements with ***MAT003-PLASTIC_KINEMATIC** as the material model. The mesh size for both concrete and steel was 25 mm, and the constraint method was implemented. The material properties for concrete and steel required by LS-DYNA are presented in Tables 4-4 and 4-5, respectively.

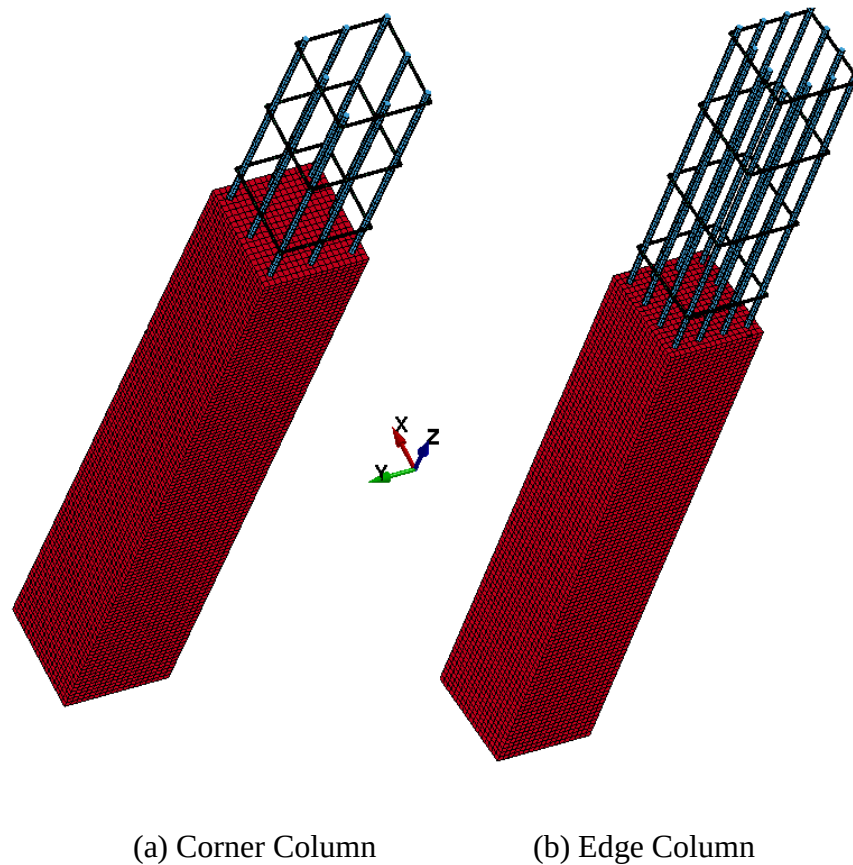
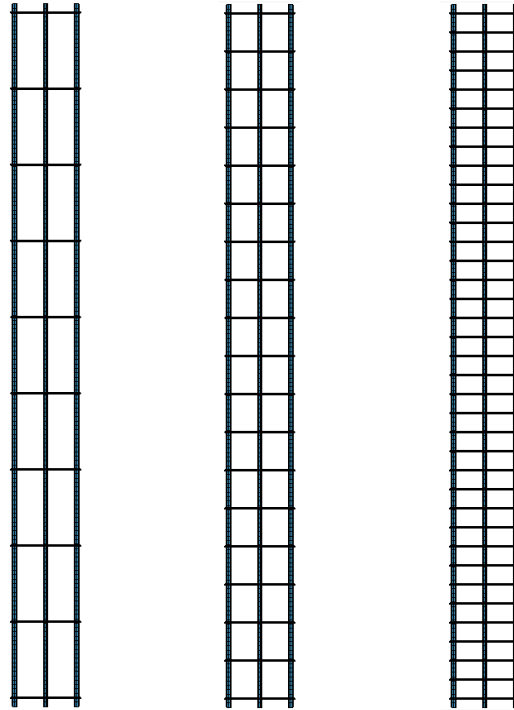


Figure 4-7: Mesh for Corner and Edge Columns



(a) 450 mm (b) 225 mm (c) 112.5 mm

Figure 4-8: Transverse Reinforcement Spacing

Table 4-4: Concrete Properties Used in LS-DYNA

Property	
Density, (kg/m ³)	2,400
Compressive Strength, (MPa)	30
Aggregate Size, (mm)	19

Table 4-5: Steel Properties Longitudinal and Transverse Steel

Property	
Density, (kg/m ³)	7,800
Yield Stress, (MPa)	400
Elastic Modules, (MPa)	200,000
Tangent Modules, (MPa)	190
Poisson's Ratio	0.3
Rapture Strain, (%)	0.15

Bottom nodes of the column were fixed in all directions. Top nodes were allowed to move vertically only to simulate gravity loads as explained in Section 3.6. The other degrees of freedoms at the top of the column were fixed in all directions to replicate the fixity condition coming from the framing girders. Axial loads of 597 kN and 1194 kN were applied on the top segment of the corner and edge columns, respectively. The charge was placed on the ground surface with mass/standoff combinations specified in Table 4-2. Hemispherical surface burst properties were assumed to be generated by the explosives. Illustrations of the full geometry, loading and boundary conditions are shown in Figure 4-9.

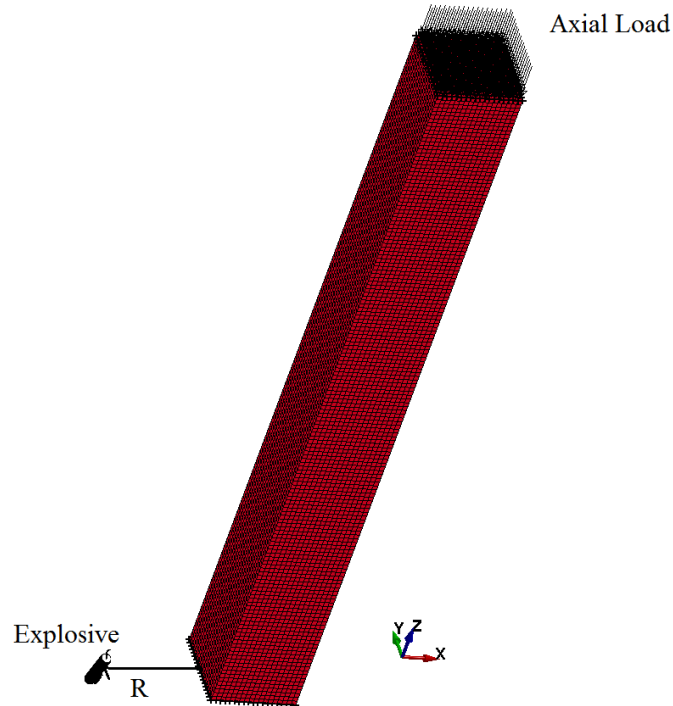


Figure 4-9: Related Model Properties

4.4 Mesh Sensitivity Analysis

Mesh sensitivity analysis was carried out to ensure the accuracy and efficiency of the finite element model. The analysis was conducted based on the maximum displacement of the column. Solid elements used to model the concrete section were cube (i.e. all dimensions are the same). Meanwhile, beam elements that were used to model the reinforcement section were of the same dimension as the solid elements. The analysis was conducted on the corner column exposed to a sedan car bomb placed at 7.5 m with transverse reinforcement spacing of 112.5 mm. The mesh size decreased and the maximum displacements were compared to optimize the results. Figure 4-10 illustrates the element mesh size used in the mesh sensitivity analysis.

The first mesh size used in the analysis was 75 mm, and generated 4,108 elements. It resulted in a maximum displacement of 58.4 mm. The second mesh size used in the analysis was 50 mm, generating 8,156 elements. It resulted in a maximum displacement of 54.7 mm. The third mesh size used in the analysis was 25 mm, generated 55,340 elements. It resulted in a maximum displacement of 46.7 mm. The fourth mesh size used was 12.5 mm, and created 421,868 elements. It resulted in a maximum displacement of 42.8 mm. Figure 4-11 illustrates maximum displacements of columns when different element mesh sizes were used.

Based on the mesh sensitivity analysis conducted above, 25 mm mesh was selected and used to discretize the finite model for the reinforced concrete column considered. This mesh size resulted in a similar accuracy in terms of maximum displacement and failure mode when compared with the finest mesh (12.5 mm). In addition, this mesh size consumed less computational time of 18 hours, in comparison with 120 hours for the smallest mesh size.

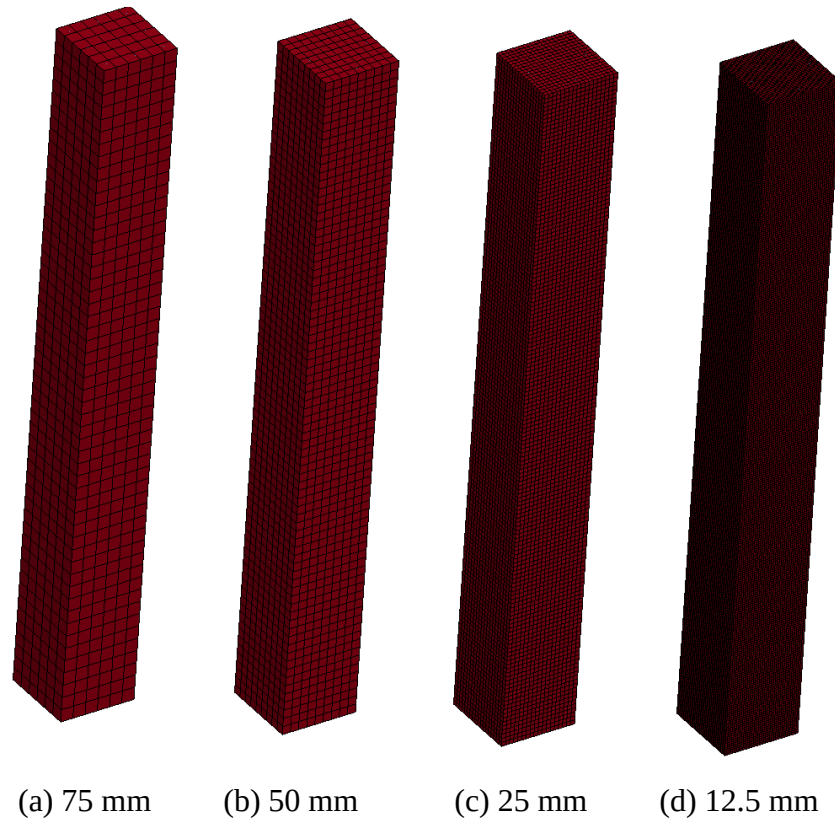


Figure 4-10: Mesh Element Size for Mesh Sensitivity Analysis

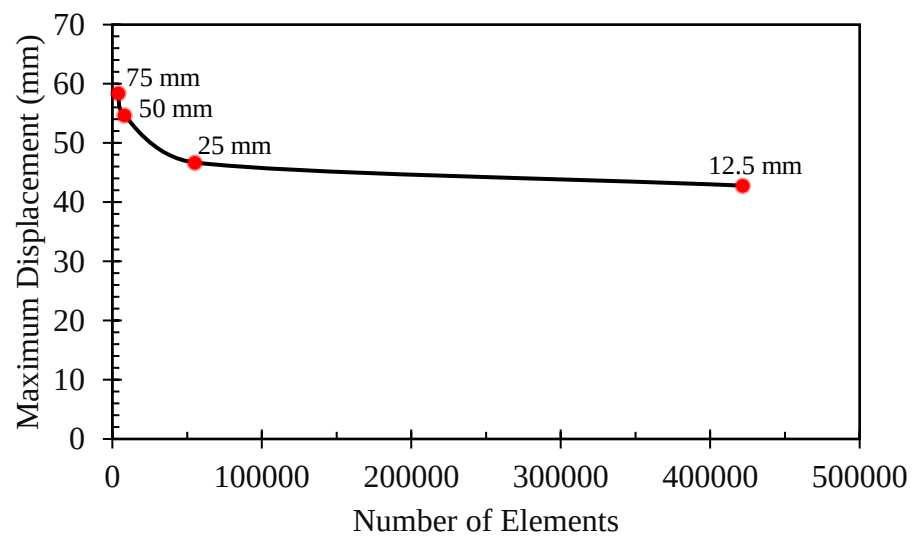


Figure 4-11: Number of Elements versus Maximum Displacement for Mesh Sensitivity Analysis

5 Chapter 5

Numerical Results

5.1 General

A total of 84 reinforced concrete column models were created in LS-PrePost user interface supplied by LS-DYNA. They were subjected to different charge/standoff distance combinations to simulate blast loading, as explained in Section 4.2. The simulations were terminated after 80 ms. Damage and failure patterns presented in the next section are up to the failure time. Displacement histories presented in the next section are for elements in the maximum regions of columns and up to the failure time of the chosen element.

Damage in concrete is evaluated using the scalar damage parameter provided with the material model for concrete. Damage level explanation is provided in Section 3.4.1.2. Damage evolution is provided for all the simulations. Performance of steel reinforcement is based on the strain values recorded with the rupturing strain in tension marking the failure of steel. Therefore, maximum strain values for longitudinal and transverse reinforcements are provided for all simulations. Any value less than 0.15 mm/mm indicate that the steel has not ruptured. The damage for concrete and the axis orientation for all simulations are provided in Figure 5-1. Blast loading is applied along the positive x-axis.

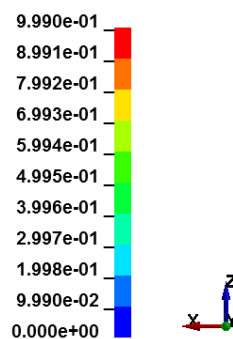


Figure 5-1: Damage for Concrete

The progression of damage in all simulations is presented in Appendix A. The blast load parameters and the stress state of the axially loaded columns (corner and edge) prior to blast load application are presented in Appendix B.

5.2 Numerical Results

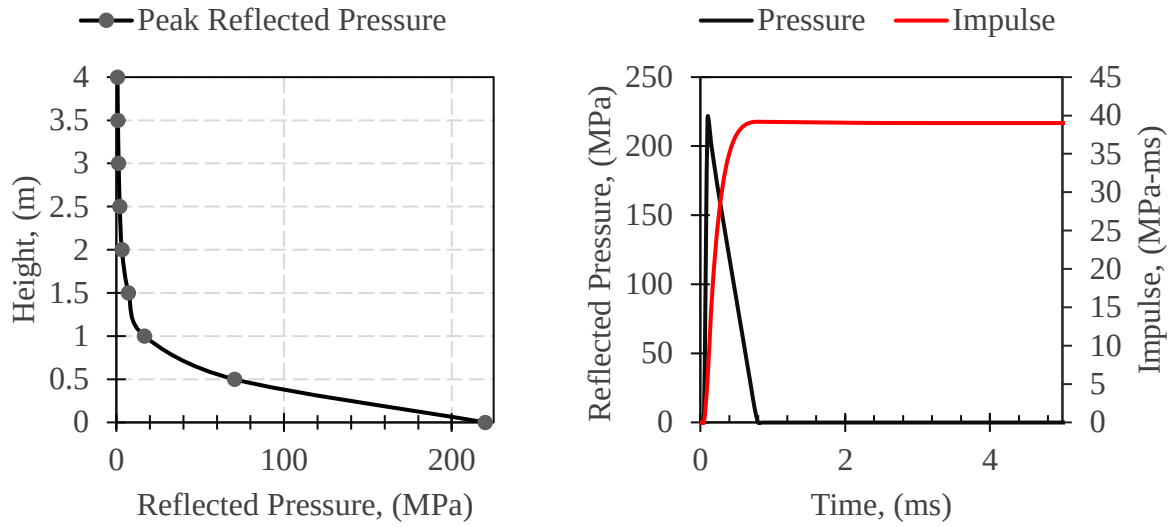
5.2.1 Response of Corner Columns CC-TS1, CC-TS2 and CC-TS3 to Backpack Bomb at 0.5 m (BP1 Blast Load)

Three corner columns with three transverse reinforcement spacing were analyzed under a 22.67 kg of backpack load placed at 0.5 m of standoff distance. The axial load was applied linearly first, with a rate of 39.8 kN/ms and held constant beyond the maximum intended magnitude of 597 kN for 65 ms. Then, blast load was applied on the front face of the columns while keeping the axial load at a constant value of 597 kN. A non-uniformly distributed load was generated from TNT charge and was applied on the front face of the columns. The peak reflected pressure distribution along the height of the column was recorded at different times depending on the arrival time of the shock wave, and is shown in Figure 5-2 (a). The peak reflected pressure of 220 MPa, reflected impulse of 39 MPa-ms, and positive phase duration of 0.75 ms were recorded at the bases of the columns. The time of arrival of the shock wave (measured from the center of the charge to the base of the columns) was 0.05 ms. Pressure and impulse time histories are shown in Figure 5-2 (b).

5.2.1.1 Corner Column with Tie Spacing Equal to Column Section Dimension (CC-TS1 under BP1)

There is no maximum displacement or maximum strain measurements for this column as it experienced severe damage. The displacement time history is shown in Figure 5-3.

The damage evolution of the column is presented in Figure A-1 (a). Concrete scabbing was initiated at time 2 ms from the start of explosive detonation near the base of the column. Continuous loss of core concrete as can be seen at times 5 ms and 8 ms which caused a complete column breaching as shown at time 12 ms.



(a) Peak Reflected Pressure along the Height (b) Pressure/Impulse Time History at the Base

Figure 5-2: BP1 Blast Load Parameters

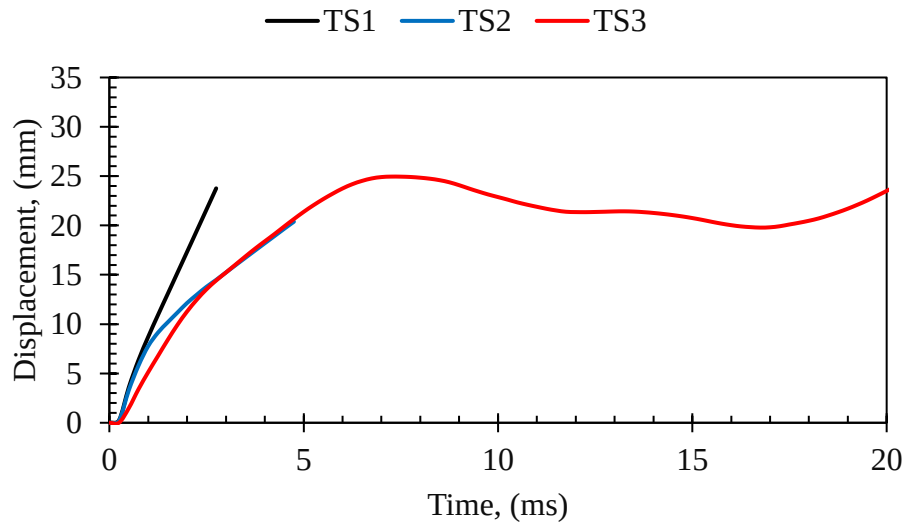


Figure 5-3: Displacement Time History for CC Columns under BP1 Loading

5.2.1.2 Corner Column with Tie Spacing Equal to Half the Column Sectional Dimension (CC-TS2 under BP1)

The column experienced severe damage. The displacement time history is shown in Figure 5-3. The damage evolution of the column is presented in Figure A-1 (b). Extensive localized shear damage was observed on the tensile side of the column near the base at time 2 ms. Scabbing initiated at time 2 ms and caused complete breaching of the column at time 14 ms. Formation of plastic hinges at the supports and along the column height led to the failure of the column.

5.2.1.3 Corner Column with Tie Spacing Equal to Quarter of the Column Sectional Dimension (CC-TS3 under BP1)

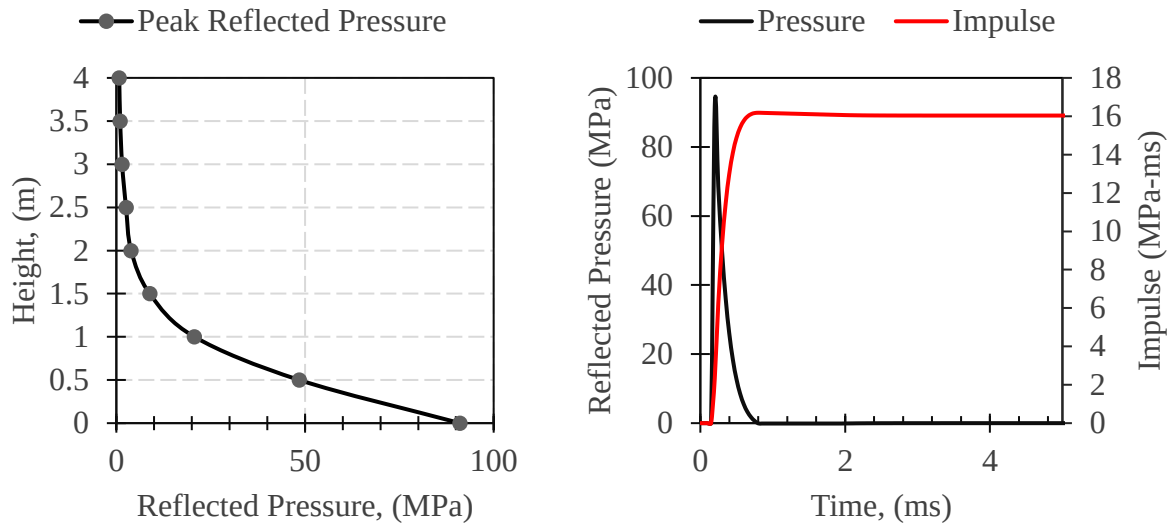
This column experienced severe damage. The displacement time history is shown in Figure 5-3. The damage evolution of the column is presented in Figure A-1 (c). Localized damage was observed at time 2 ms near the column base. Scabbing was observed at time 5 ms. Localized shear damage led to complete breaching near the base at time 18 ms. After losing core concrete, the bars buckled as shown at time 60 ms and caused column failure. Height of the damaged region was 825 mm.

5.2.2 Response of Corner Columns CC-TS1, CC-TS2 and CC-TS3 to Backpack Bomb at 0.75 m (BP2 Blast Load)

The column was loaded by following the same procedure applied on the previous three corner columns with the axial load first applied linearly with a rate of 39.8 kN/ms, and held constant at a magnitude of 597 kN for 65 ms. Once the axial load reached its maximum level, the blast load was applied while maintaining the axial load at the maximum level. The blast load consisted of a backpack bomb with a charge weight of 22.67 kg at a standoff distance of 0.75 m. A non-uniformly distributed load was generated on the front face of the columns from the application of the TNT charge. The peak reflected pressure distribution along the height of the column was recorded, and is shown in Figure 5-4 (a). The peak reflected pressure of 91.1 MPa, reflected impulse of 16.0 MPa-ms, and positive phase duration of 0.65 ms were recorded at the base of each column. The time of arrival of the shock wave (measured from the center of the charge to the base of the columns) was 0.15 ms. Pressure and impulse time histories are shown in Figure 5-4 (b).

5.2.2.1 Corner Column with Tie Spacing Equal to Column Section Dimension (CC-TS1 under BP2)

The column experienced severe damage. The displacement time history is shown in Figure 5-5. The damage evolution of the column is presented in Figure A-2 (a). Visible local shear damage was observed at the back face of the column near the base at time 1 ms. Loss of concrete elements in the damaged region occurred at time 2 ms (scabbing) with a wide diagonal tension crack. At time 15 ms, complete column breaching occurred that led to the column failure. Extensive cracking and concrete crushing observed long the height of the column at the onset of beaching indicated the formation of plastic hinges along the length of the column.



(a) Peak Reflected Pressure along the Height (b) Pressure/Impulse Time History at the Base

Figure 5-4: BP2 Blast Load Parameters

5.2.2.2 Corner Column with Tie Spacing Equal to Half the Column Sectional Dimension (CC-TS2 under BP2)

The column experienced severe damage and failed at about 9 ms. The displacement time history is shown in Figure 5-5. The evolution of damage is illustrated in Figure A-2 (b). Noticeable local shear damage was observed on the tension side of the column near the base, at time 1 ms. Scabbing of the concrete initiated at time 2 ms. At time 10 ms, partial breaching was observed with an

inclined shear failure plane clearly observed. At time 18 ms, the column was fully breached and failed with damage extending towards the height of the column.

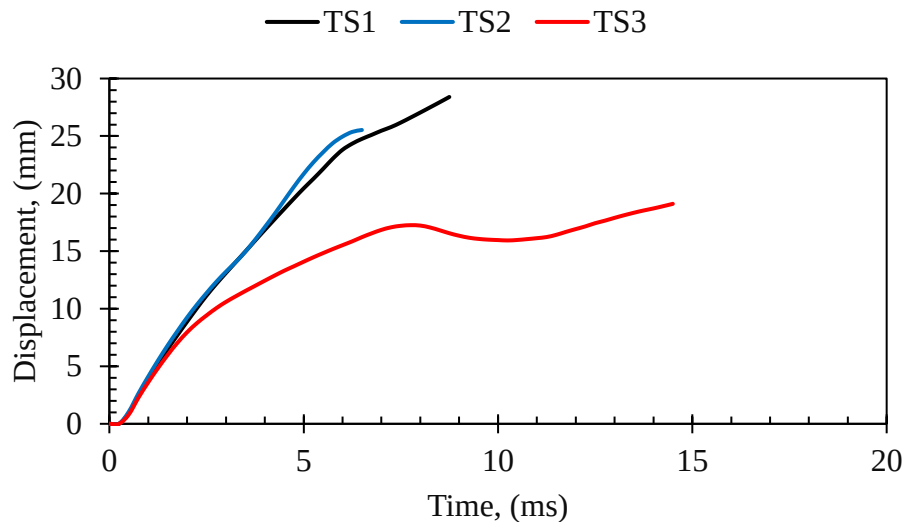


Figure 5-5: Displacement Time History for CC Columns under BP2 Loading

5.2.2.3 Corner Column with Tie Spacing Equal to Quarter of the Column Sectional Dimension (CC-TS3 under BP2)

The column experienced severe damage and failed at about 10 ms. The displacement time history is shown in Figure 5-5. The evolution of damage in the column is presented in Figure A-2 (c). Clear local damage was observed at the back face of the column near the base, after the arrival of the shock wave. Scabbing at the back face near the base of the column was observed at time 5 ms. The complete breaching of the column occurred between 18 to 20 ms.

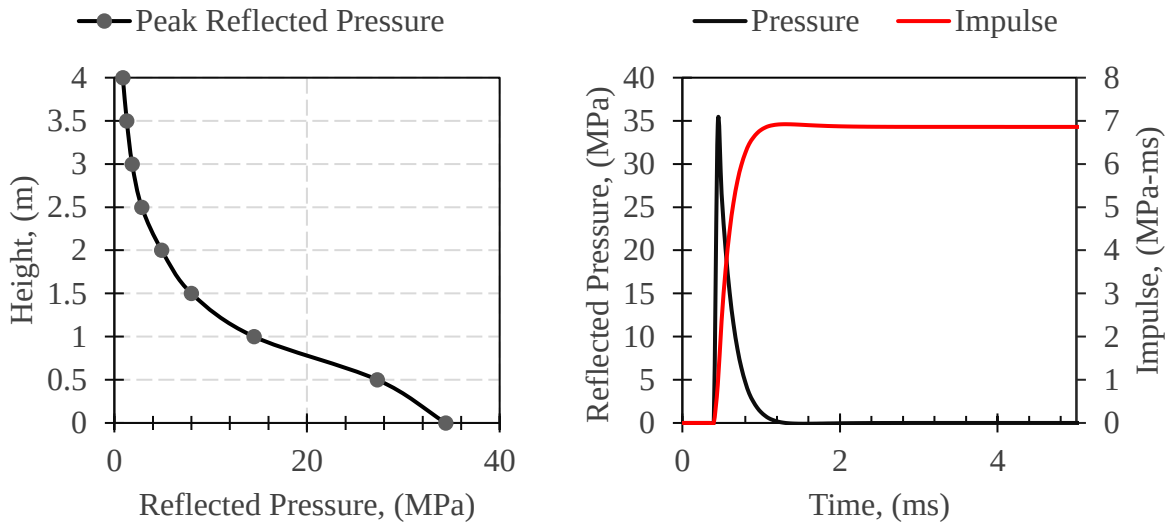
5.2.3 Response of Corner Columns CC-TS1, CC-TS2 and CC-TS3 to Backpack Bomb at 1.5 m (BP3 Blast Load)

Axial load was applied on the columns with a rate of 39.8 kN/ms and held constant for 65 ms at a magnitude of 597 kN prior to the application of blast loading. The blast load was applied while the axial load was constant at 597 kN. The blast corresponded to the detonation of 22.67 kg of TNT on the ground surface at a standoff distance of 1.5 m, which corresponds to a scaled standoff distance of $0.530 \text{ m/kg}^{1/3}$. This resulted in a non-uniformly distributed load on the front face of the

columns. The peak reflected pressure distribution along the height of the column was recorded, and is shown in Figure 5-6 (a). The peak reflected pressure at the base of each column was 34.4 MPa with a reflected impulse of 3.92 MPa-ms and a positive phase duration of 0.95 ms. The time of arrival of the shock wave (measured from the center of the charge to the base of the columns) was 0.40 ms. Pressure and impulse time histories are shown in Figure 5-6 (b).

5.2.3.1 Corner Column with Tie Spacing Equal to Column Section Dimension (CC-TS1 under BP3)

The column experienced severe damage. The displacement time history recorded is shown in Figure 5-7. The progression of damage in the column is presented in Figure A-3 (a). The formation of diagonal tension cracks associated with shear in the bottom support region can be observed in the figure with progressively increasing damage at times 1 ms, 2 ms and 5 ms and 10 ms. Extensive diagonal compression crushing was observed in the same region at time 20 ms, while flexural crushing was observed near the mid-height. This resulted in the collapse of the column.



(a) Peak Reflected Pressure along the Height (b) Pressure/Impulse Time History at the Base

Figure 5-6: BP3 Blast Load Parameters

5.2.3.2 Corner Column with Tie Spacing Equal to Half the Column Sectional Dimension (CC-TS2 under BP3)

The column experienced severe damage. The displacement time history recorded is shown in Figure 5-7. The progression of damage in the column is presented in Figure A-3 (b). Diagonal shear crack formation near the bottom support region of the column was observed at times 1 ms, 2 ms, 5 ms and 10 ms. Extensive flexural cracking was experienced in this column. The column failed due to extensive crushing of concrete under combined diagonal and flexural compressive stresses at 80 ms.

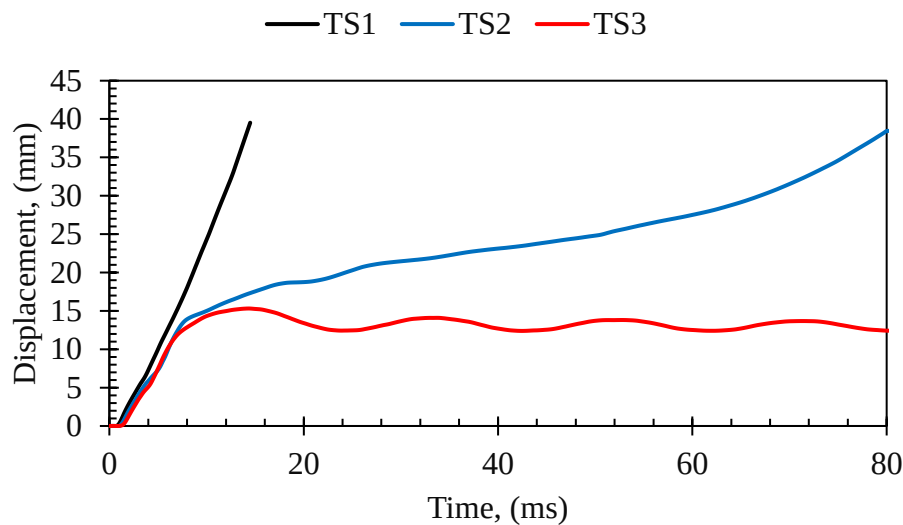


Figure 5-7: Displacement Time History for CC Columns under BP3 Loading

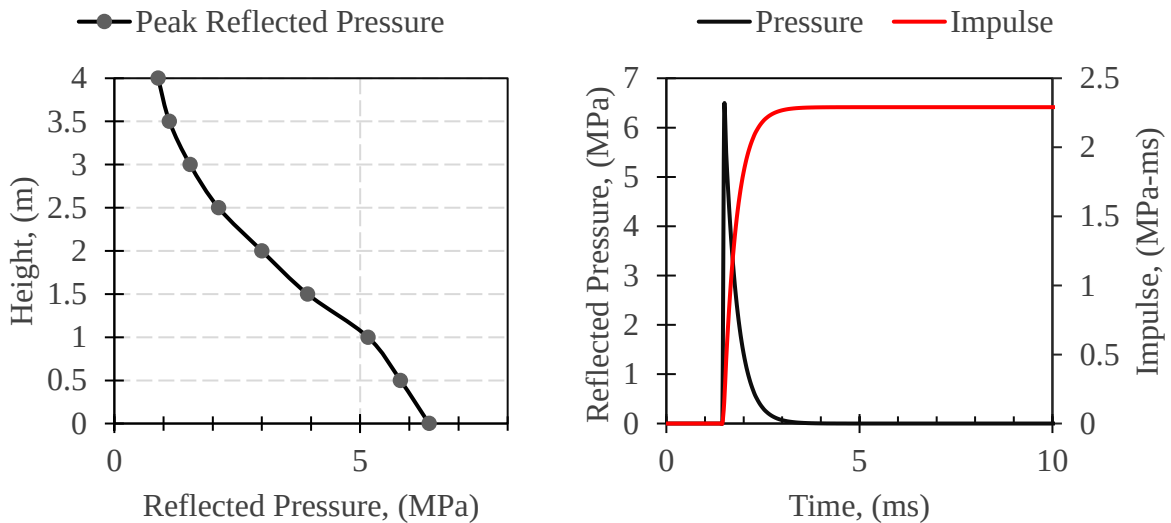
5.2.3.3 Corner Column with Tie Spacing Equal to Quarter of the Column Sectional Dimension (CC-TS3 under BP3)

The column behaved predominantly in the flexure mode, due to the increased tie spacing and a relatively distant detonation of the explosive. Maximum displacement of 15.3 mm was recorded at 14.2 ms, at a height of 2050 mm above the ground surface. Maximum support rotation was 0.45° (calculated from the top). The displacement time history is shown in Figure 5-7. The maximum strain experienced by the longitudinal reinforcements was 0.009 mm/mm. The maximum strain experienced by the transverse reinforcements was 0.0227 mm/mm.

The progression of damage is presented in Figure A-3 (c). The concrete side cover was observed to be lost during the blast event. Flexural cracks occurred at 5 ms near the column mid span. The column was dominated by flexure and showed relatively good performance in comparison to all the columns discussed earlier, developing approximately 12 mm of residual displacement.

5.2.4 Response of Corner Columns CC-TS1, CC-TS2 and CC-TS3 to Backpack Bomb at 3.0 m (BP4 Blast Load)

The columns were loaded by axial load with a rate of 39.8 kN/ms prior to the application of the blast loads. The axial load remained constant at a magnitude of 597 kN for 65 ms prior to during the blast load application. The blast load was triggered by 22.67 kg of TNT placed on the ground surface at a standoff distance of 3.0 m, which corresponds to a scaled standoff distance of 1.060 m/kg^{1/3}. A non-uniform load was generated by the TNT charge, and was applied on the front face of the columns. The peak reflected pressure distribution along the height is shown in Figure 5-8 (a). The peak reflected pressure of 6.40 MPa was generated with reflected impulse of 2.75 MPa-ms and positive phase duration of 5.6 ms. The time of arrival of the shock wave (measured from the center of the charge to the base of the columns) was 1.45 ms. Pressure and impulse time histories are shown in Figure 5-8 (b).



(a) Peak Reflected Pressure along the Height (b) Pressure/Impulse Time History at the Base

Figure 5-8: BP4 Blast Load Parameters

5.2.4.1 Corner Column with Tie Spacing Equal to Column Section Dimension (CC-TS1 under BP4)

The column showed predominantly flexural response with a maximum displacement 5.04 mm at 8.25 ms, observed at a height of 2000 mm above the ground surface. Maximum support rotation was 0.14° (calculated from the top). The displacement time history is shown in Figure 5-9. The maximum strain experienced by the longitudinal reinforcements was less than 0.002 mm/mm, signifying elastic behaviour. The maximum strain experienced by the transverse reinforcements was less than 0.002 mm/mm.

The progression of damage is presented in Figure A-4 (a). The column developed hairline flexural and inclined shear cracking, but otherwise remained virtually elastic with about 2.5 mm permanent deflection.

5.2.4.2 Corner Column with Tie Spacing Equal to Half the Column Sectional Dimension (CC-TS2 under BP4)

The column showed similar behaviour to that obtained from the earlier column (CC-TS1) with twice the tie spacing. Maximum displacement of 5.09 mm was recorded at 8.00 ms, at a height of 2000 mm above the ground surface. The maximum support rotation was 0.15° (calculated from the top). The displacement time history is shown in Figure 5-9. The maximum strain experienced by the longitudinal reinforcements was less than 0.002 mm/mm, indicating elastic response. The maximum strain experienced by the transverse reinforcements was less than 0.002 mm/mm.

The progression of damage is presented in Figure A-4 (b). Hairline cracks were observed under flexure and shear. There was no concrete crushing, and the column remained essentially elastic with about 1.8 mm permanent deflection.

5.2.4.3 Corner Column with Tie Spacing Equal to Quarter of the Column Sectional Dimension (CC-TS3 under BP4)

The column performed well, demonstrating primarily flexural response with a maximum displacement of 4.86 mm occurring at 8.5 ms at a height of 2000 mm above the ground surface. The maximum support rotation was measured to be 0.14° (calculated from the top). The displacement time history is shown in Figure 5-9. The maximum strain experienced by the

longitudinal reinforcements was less than 0.002 mm/mm, remaining elastic. The maximum strain experienced by the transverse reinforcements was less than 0.002 mm/mm.

The progression of damage in the column is presented in Figure A-4 (c). Some hairline flexural and diagonal tension cracking was observed. No crushing of concrete was observed and the column remained intact with about 2.0 mm of permanent deflection.

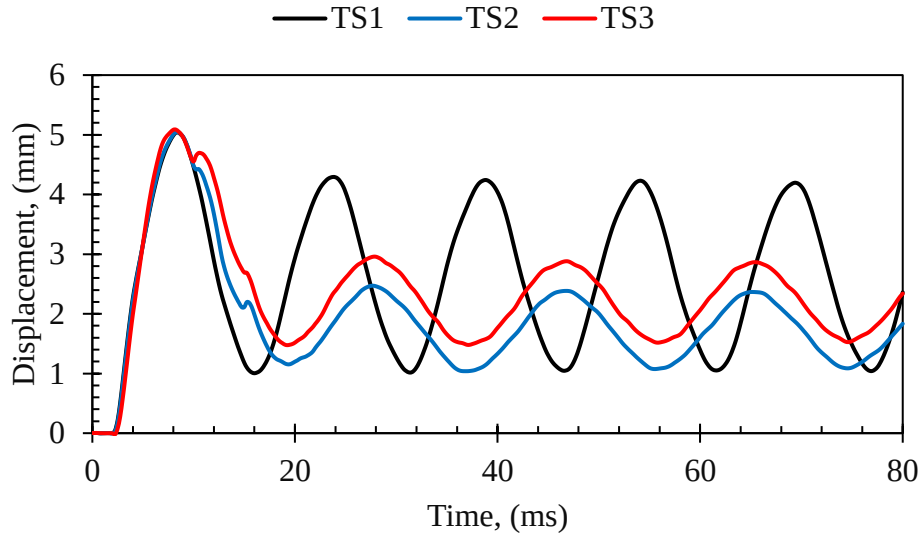


Figure 5-9: Displacement Time History for CC Columns under BP4 Loading

5.2.5 Response of Corner Columns CC-TS1, CC-TS2 and CC-TS3 to Compact Sedan Car Bomb at 1.5 m (CS1 Blast Load)

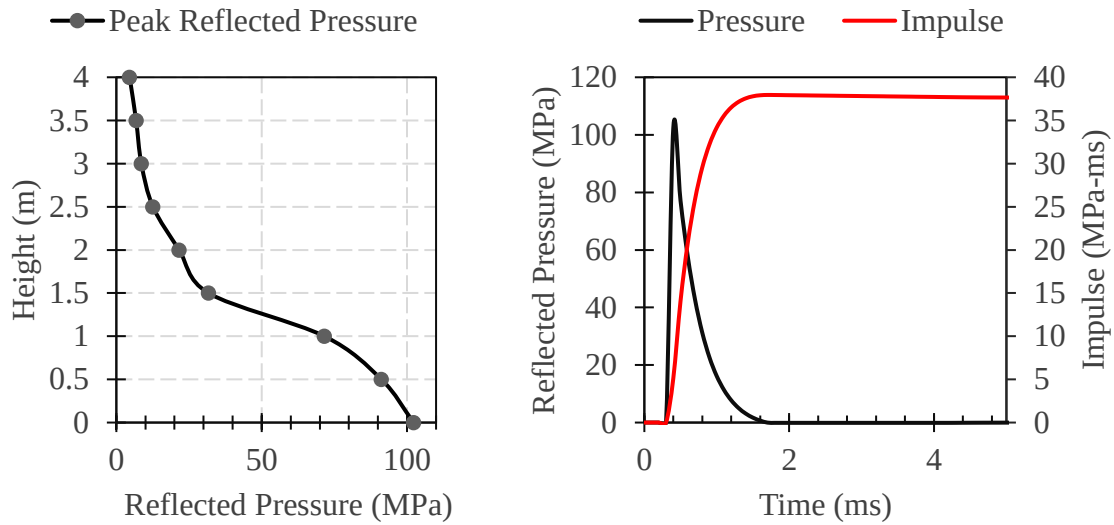
The columns were loaded axially first at a rate of 39.8 kN/ms up to a magnitude of 597 kN prior to the application of the blast loading. The axial load was kept constant for 65 ms prior the application of the blast loading and during the blast load stage. The blast loading was generated by 227 kg of TNT, placed on the ground surface at a standoff distance of 1.5 m which corresponds to a scaled standoff distance of $0.246 \text{ m/kg}^{1/3}$. A non-uniform distributed load was generated by the TNT and was applied on the front face of the columns. The peak reflected pressure distribution along the height of the columns is shown in Figure 5-10 (a). The peak reflected pressure of 102.3 MPa was applied at the base of the column with a reflected impulse of 37.6 MPa-ms and a positive phase duration of 1.4 ms. The arrival time of the shock wave (measured from the center of the

charge to the base of the columns) was 0.3 ms. Pressure and impulse time histories are shown in Figure 5-10 (b).

5.2.5.1 Corner Column with Tie Spacing Equal to Column Section Dimension (CC-TS1 under CS1)

The column was damaged excessively. Maximum displacement and maximum strain measurements could not be recorded because of the severe damage experienced. The displacement time history is shown in Figure 5-11.

The progression of damage is presented in Figure A-5 (a). Extensive local damage near the base of the column was observed on the back face at 1 ms. Direct-shear failure was detected at the bottom support at 3 ms, which led to column failure. The damage propagated towards the mid height, as column core deteriorated completely at 11 ms.



(a) Peak Reflected Pressure along the Height (b) Pressure/Impulse Time History at the Base

Figure 5-10: CS1 Blast Load Parameters

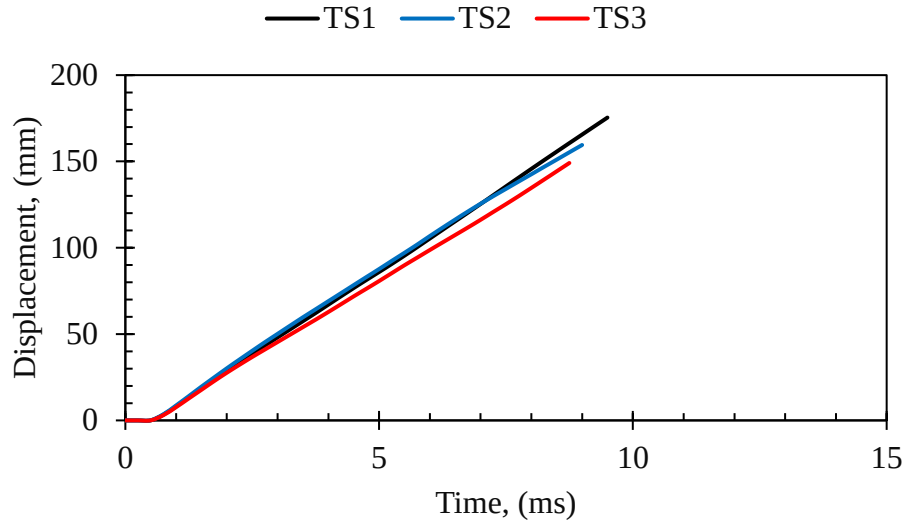


Figure 5-11: Displacement Time History for CC Columns under CS1 Loading

5.2.5.2 Corner Column with Tie Spacing Equal to Half the Column Sectional Dimension (CC-TS2 under CS1)

The column behaved similar to the companion column discussed above with wider spacing of transverse reinforcement. Because it was severely damaged no data could be recorded for maximum displacement and maximum strains. The displacement time history is shown in Figure 5-11.

The progression of damage is depicted in Figure A-5 (b). Heavily stressed local region was detected near the column base at 1 ms. Direct shear failure was observed at the bottom support at 3 ms, which was followed by the disintegration of core concrete between 6 ms near the base and 11 ms covering the bottom half of the column.

5.2.5.3 Corner Column with Tie Spacing Equal to Quarter of the Column Sectional Dimension (CC-TS3 under CS1)

This column behaved similar to the previous two companion columns with wider spacing of tie steel, developing excessive damage. Maximum displacement and maximum strain measurements could not be recorded due to this severe damage. The displacement time history is shown in Figure 5-11.

The column damage is presented in Figure A-5 (c) as observed at different times during response. Heavily stressed region was observed near the base at 1 ms. Direct shear failure initiated at 3 ms, followed by the disintegration of core concrete within the bottom half. The closely spaced transverse reinforcement was not effective in preventing the type of failure observed because of the close standoff distance.

5.2.6 Response of Corner Columns CC-TS1, CC-TS2 and CC-TS3 to Compact Sedan Car Bomb at 3.0 m (CS2 Blast Load)

The columns were axially loaded first, prior to the application of the blast load at a rate of 39.8 kN/ms, up to a magnitude of 597 kN, and kept constant for 65 ms. While maintaining the same axial load level, 227 kg of TNT was placed on the ground surface at a standoff distance of 3.0 m which corresponded to a scaled standoff distance of $0.492 \text{ m/kg}^{1/3}$ to simulate the blast load. A non-uniform distributed load was generated from the TNT charge and was applied on the front face of the three columns investigated in this section. The peak reflected pressure distribution along the height of the column was recorded and is shown in Figure 5-12 (a). A peak reflected pressure of 36.1 MPa, reflected impulse of 13.7 MPa-ms with positive phase duration of 1.7 ms was recorded at the bases of the columns. The time of arrival of the shock wave (measured from the center of the charge to the base of the columns) was 0.85 ms. Pressure and impulse time histories are shown in Figure 5-12 (b).

5.2.6.1 Corner Column with Tie Spacing Equal to Column Section Dimension (CC-TS1 under CS2)

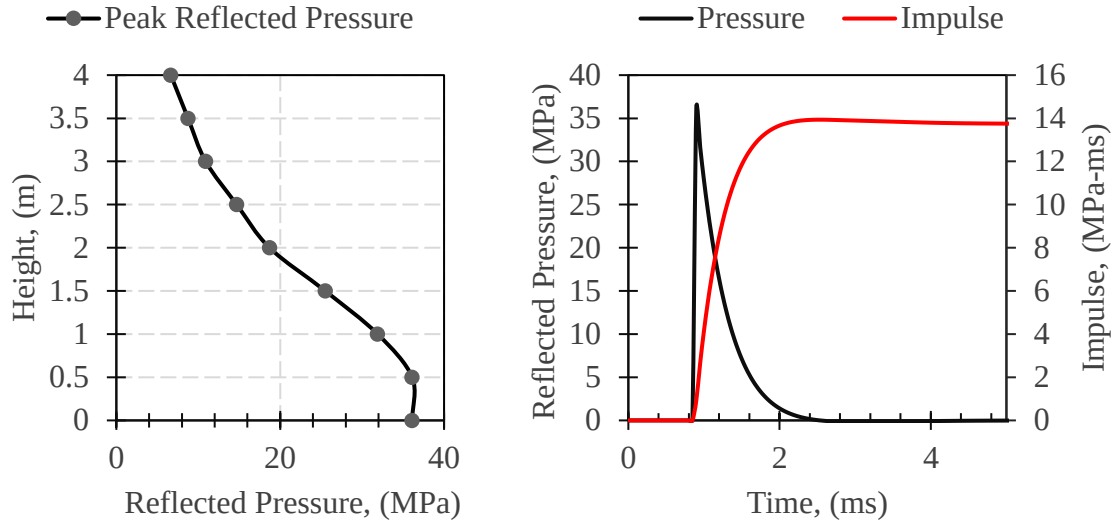
The column experienced severe damage. Therefore, maximum displacement and maximum strain measurements could not be recorded. The displacement time history is shown in Figure 5-13. The progression of damage is presented in Figure A-6 (a). A large diagonal tension crack formed at time 3 ms that led to diagonal shear failure. Concrete deterioration progressed towards the mid-height, causing the crushing of core concrete under high levels of diagonal compressive and flexural compressive stresses. The column suffered complete failure at about 12 ms.

5.2.6.2 Corner Column with Tie Spacing Equal to Half the Column Sectional Dimension (CC-TS2 under CS2)

The column suffered similar damage to the previous column, with twice the tie spacing. However, the diagonal tension crack was better controlled in this column with the wide inclined crack taking place at about 5 ms. Eventually the column suffered a similarly severe damage. Hence, no maximum displacement or maximum strain measurements were taken for this column. The displacement time history is shown in Figure 5-13. The progression of damage is presented in Figure A-6 (b). Local damage near the bottom support was observed in the early stages of the loading. Combination of shear and flexural cracks occurred at time 3 ms. Formation of the diagonal shear crack occurred at time 3 ms and progressed as shown at time 5 ms and 7 ms. Formation of flexural cracks near mid span observed clearly at time 5 ms. The column failed through the deterioration of concrete as shown in Fig. A-6 (b) at time 13 ms.

5.2.6.3 Corner Column with Tie Spacing Equal to Quarter of the Column Sectional Dimension (CC-TS3 under CS2)

The column behaved similar to the previous two columns in this group, with much better diagonal tension control due to the close spacing of column ties. Eventually the failure was quite severe, with complete deterioration of concrete within the bottom half of the column. There was no maximum displacement or maximum strain measured for this column as it experienced severe damage. The displacement time history is shown in Figure 5-13. The progression of damage is shown in Figure A-6 (c). Many flexural cracks were initiated near the column mid-height at time 3 ms. Diagonal shear crack became visible at about 5 ms. Loss of cover and some of the core concrete on the tensile side near the mid-height was observed as shown in the figure at time 10 ms. Column was dominated by a combined flexure and shear responses. The deterioration of column core concrete caused total failure of the column at time 16 ms.



(a) Peak Reflected Pressure along the Height (b) Pressure/Impulse Time History at the Base

Figure 5-12: CS2 Blast Load Parameters

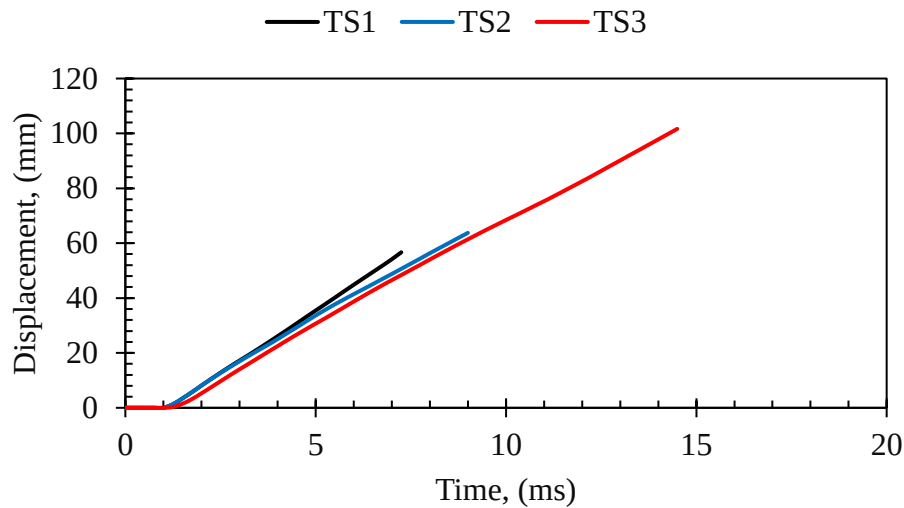


Figure 5-13: Displacement Time History for CC Columns under CS2 Loading

5.2.7 Response of Corner Columns CC-TS1, CC-TS2 and CC-TS3 to Compact Sedan Car Bomb at 4.5 m (CS3 Blast Load)

The three columns analysed in this part of the investigation was first loaded by axial compression, similar to the previous columns, with the axial load applied a rate of 39.8 kN/ms. The load was held constant at a magnitude of 597 kN prior to during the application of the blast load. The blast

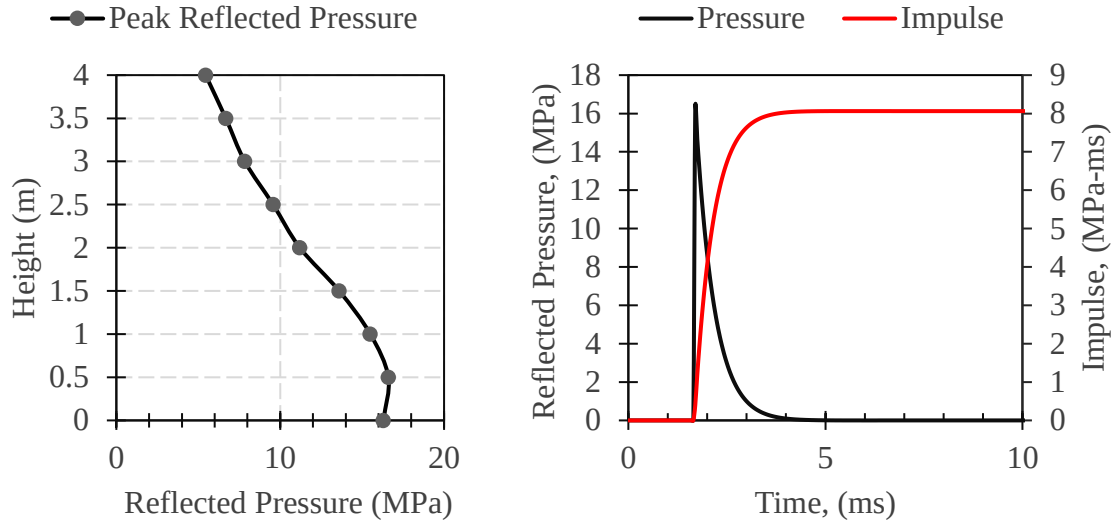
load consisted of 227 kg of TNT, placed on the ground surface at a standoff distance of 4.5 m, which corresponded to a scaled standoff distance of $0.738 \text{ m/kg}^{1/3}$. A non-uniform load was generated by the TNT, which was applied on the front face of the columns. The peak reflected pressure was distributed along the height of the column, and is shown in Figure 5-14 (a). A peak reflected pressure of 16.3 MPa, reflected impulse of 8.1 MPa-ms with positive phase duration of 3.9 ms was recorded at the bases of all three columns. The time of arrival of the shock wave (measured from the center of the charge to the base of the columns) was 1.65 ms. Pressure and impulse time histories are shown in Figure 5-14 (b).

5.2.7.1 Corner Column with Tie Spacing Equal to Column Section Dimension (CC-TS1 under CS3)

The column developed wide diagonal tension cracks, followed by the deterioration of the core concrete, causing severe damage. Therefore, maximum displacement and maximum strain measurements could not be recorded at failure. The displacement time history is shown in Figure 5-15. The progression of damage in the column is presented in Figure A-7 (a). Diagonal tension and flexural cracks started forming at about 3 ms, but did not widen until after 4 ms. Diagonal compression and flexural compression increased to critical levels starting at 10 ms and eventually led to the crushing of column core at about 20 ms. The concrete crushing propagated towards the mid-height at failure.

5.2.7.2 Corner Column with Tie Spacing Equal to Half the Column Sectional Dimension (CC-TS2 under CS3)

The column performed better than the previous column with twice the tie spacing. The ties at one half the cross-sectional dimension controlled diagonal tension near the bottom support and changed the failure mode to flexure. Eventually the column suffered from severe failure, which did not permit the recording of maximum displacement and maximum strain measurements at failure. The displacement time history is shown in Figure 5-15. The progression of damage is presented in Figure A-7 (b). The column experienced flexural cracks in the initial stages of the loading as shown at times 3 ms, 4 ms and 10 ms. Formation of plastic hinge near the column mid span region led to the loss of 100% of the core concrete and associated column failure at 35 ms.



(a) Peak Reflected Pressure along the Height (b) Pressure/Impulse Time History at the Base

Figure 5-14: CS3 Blast Load Parameters

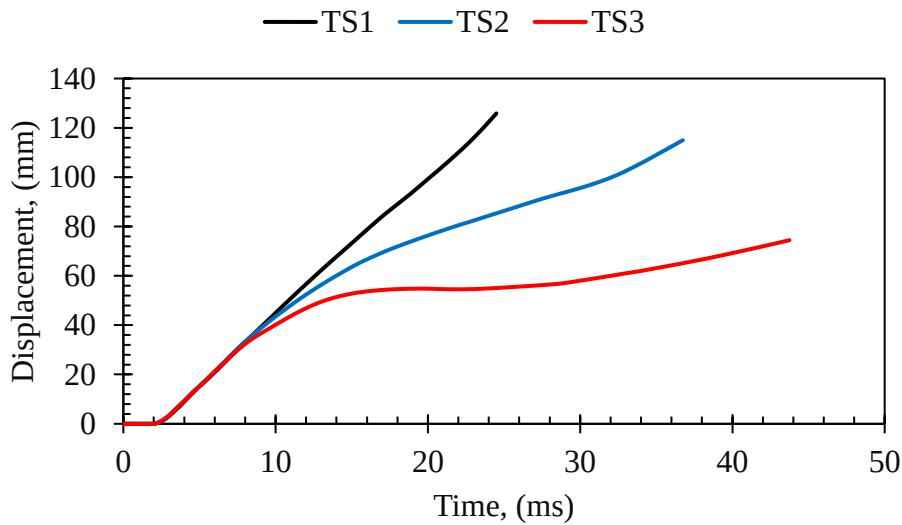


Figure 5-15: Displacement Time History for CC Columns under CS3 Loading

5.2.7.3 Corner Column with Tie Spacing Equal to Quarter of the Column Sectional Dimension (CC-TS3 under CS3)

The reduced tie spacing in this column shifted the mode of behaviour completely from shear to flexure. The eventual failure was brittle and did not permit the recording of maximum

displacement and maximum strain measurements. The displacement time history is shown in Figure 5-15. The progression of damage is presented in Figure A-7 (c). The column experienced only flexural cracking in the initial stages of loading as shown at times 3 ms, 4 ms and 10 ms. Excessive flexural tension cracking was observed near the base and mid-height regions, followed by the onset of concrete crushing at about 15 ms. Deterioration of concrete, especially near the bottom support region, leading to the formation of a plastic hinge was observed at about 40 ms. This resulted in a sudden failure of the column.

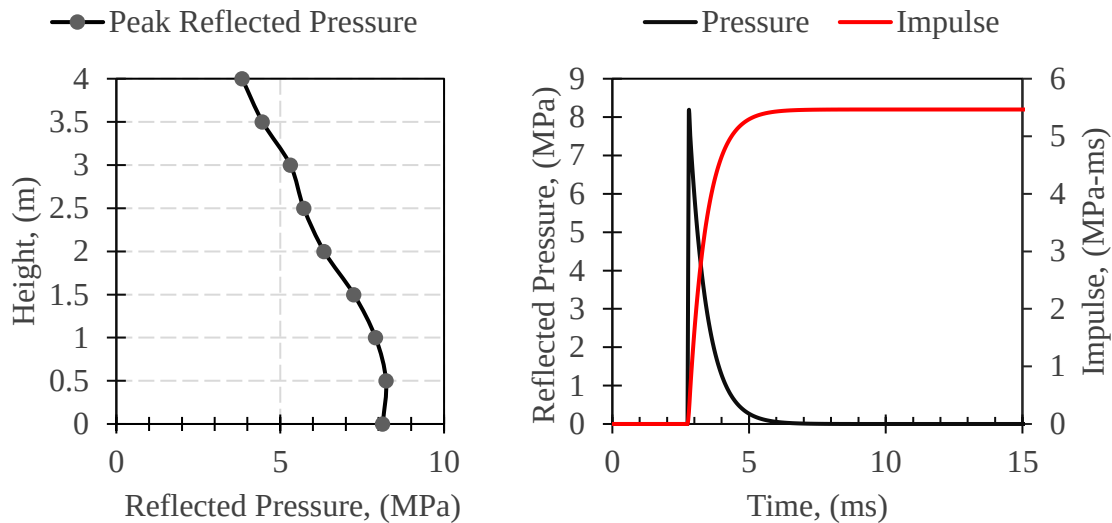
5.2.8 Response of Corner Columns CC-TS1, CC-TS2 and CC-TS3 to Compact Sedan Car Bomb at 6.0 m (CS4 Blast Load)

The columns in this series were analysed under combined axial and blast loading as before. The axial load was applied gradually with a rate of 39.8 kN/ms, and held constant at a magnitude of 597 kN for 65 ms prior to the application of the blast load and kept constant during the blast load stage. The blast loading was generated by 227 kg of TNT, placed on the ground surface at a standoff distance of 6.0 m, which corresponded to a scaled standoff distance of $0.984 \text{ m/kg}^{1/3}$. A non-uniform distributed load was generated from the TNT charge, and was applied on the front face of the columns. The peak reflected pressure distribution along the height of the column was recorded, and is shown in Figure 5-16 (a). A peak reflected pressure of 8.13 MPa, reflected impulse of 5.47 MPa-ms with positive phase duration of 10.05 ms was recorded at the base of each column. The time of arrival of the shock wave (measured from the center of the charge to the base of the columns) was 2.75 ms. Pressure and impulse time histories are shown in Figure 5-16 (b).

5.2.8.1 Corner Column with Tie Spacing Equal to Column Section Dimension (CC-TS1 under CS4)

The column developed a combined diagonal tension and flexural failures. The failure was sudden and sever, hence no maximum displacement and maximum strain measurements were recorded at failure. The displacement time history is shown in Figure 5-17. The progression of damage is presented in Figure A-8 (a). Many flexural cracks were initiated near the column mid-height at time 8 ms. Also, increased tensile and compressive stresses were observed at 20 ms, resulting in the spalling of the cover concrete near the bottom support and excessive cracking near the mid-height. Significant damage was experienced due to flexural response. A large number of concrete

elements were damaged near the base during the column rebound observed at time 20 ms. The column kept displacing near the end of the simulation and deemed to have no reserve capacity at 80 ms.



(a) Peak Reflected Pressure along the Height (b) Pressure/Impulse Time History at the Base

Figure 5-16: CS4 Blast Load Parameters

5.2.8.2 Corner Column with Tie Spacing Equal to Half the Column Sectional Dimension (CC-TS2 under CS4)

The column performed reasonably well with reduced ties spacing. The maximum displacement was 31.7 mm, which occurred at 15.2 ms at a height of 2500 mm above the ground surface. The maximum support rotation was 1.21° (calculated from the top). The displacement time history is shown in Figure 5-17. The maximum strain experienced by the longitudinal reinforcement was 0.0203 mm/mm. The maximum strain experienced by the transverse reinforcements was 0.0105 mm/mm. The progression of damage is presented in Figure A-8 (b). The column responded primarily in flexure with increased flexural cracks near the column mid-height at time 8 ms. At 10 ms, damage accumulation was observed on the front face of the column near the base. Partial loss of concrete cover was observed on the back face at time 20 ms near mid-height. Minor damage on the column back face was caused by flexure, without losing core concrete. This is illustrated in the figure at time 80 ms.

5.2.8.3 Corner Column with Tie Spacing Equal to Quarter of the Column Sectional Dimension (CC-TS3 under CS4)

This column had further reduced tie spacing relative to the previous column. It performed predominantly in flexure mode and developed a maximum displacement of 29.9 mm, which occurred at 14.5 ms at a height of 2470 mm above the ground surface. Maximum support rotation was 1.12° (calculated from the top). The displacement time history is shown in Figure 5-17. The maximum strain experienced by the longitudinal reinforcements was 0.0213 mm/mm. The maximum strain experienced by the transverse reinforcements was 0.0064 mm/mm.

The progression of damage is presented in Figure A-8 (c). Significant flexural cracking initiated near the column mid-height and bottom support regions at time 8 ms. At 10 ms, the damage level in these regions increased. Light damage on the column tension regions occurred at time 80 ms. There was no significant crushing of concrete.

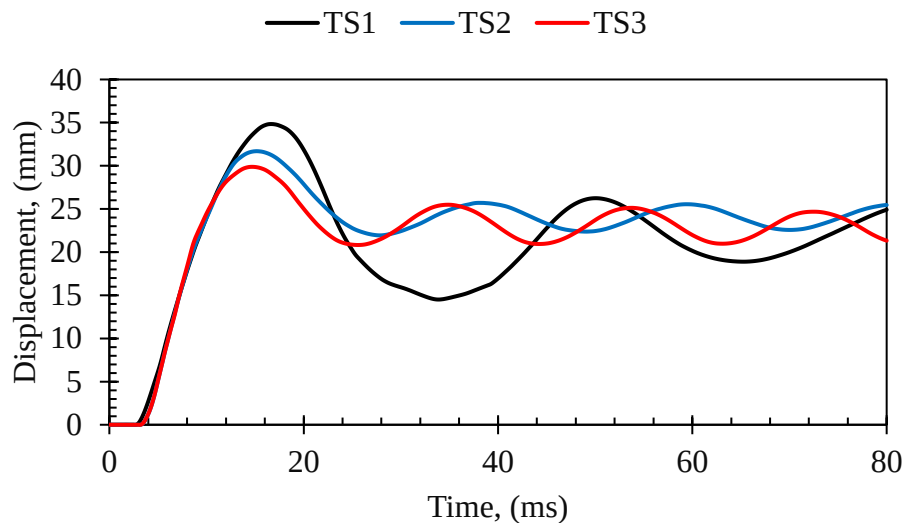


Figure 5-17: Displacement Time History for CC Columns under CS4 Loading

5.2.9 **Response of Corner Columns CC-TS1, CC-TS2 and CC-TS3 to Compact Sedan Car Bomb at 7.5 m (CS5 Blast Load)**

The columns in this series were subjected to axial compressive force of 597 kN, applied as a linearly increasing load at a rate of 39.8 kN/ms. The load was kept at this level for 65 ms prior the application of blast loading and during the blast loading stage. The blast load was triggered by the

detonation of 227 kg of TNT placed on the ground surface at a standoff distance of 7.5 m, which corresponded to a scaled standoff distance of $1.229 \text{ m/kg}^{1/3}$. A non-uniform distributed load was generated from the TNT charge, and was applied on the front face of all the columns. The peak reflected pressure distribution along the height of the columns was recorded as shown in Figure 5-18 (a). A peak reflected pressure of 4.39 MPa, reflected impulse of 4.08 MPa-ms with positive phase duration of 13.35 ms was recorded at the base of each column. The time of arrival of the shock wave (measured from the center of the charge to the base of the columns) was 4.15 ms. Pressure and impulse time histories are shown in Figure 5-18 (b).

5.2.9.1 Corner Column with Tie Spacing Equal to Column Section Dimension (CC-TS1 under CS5)

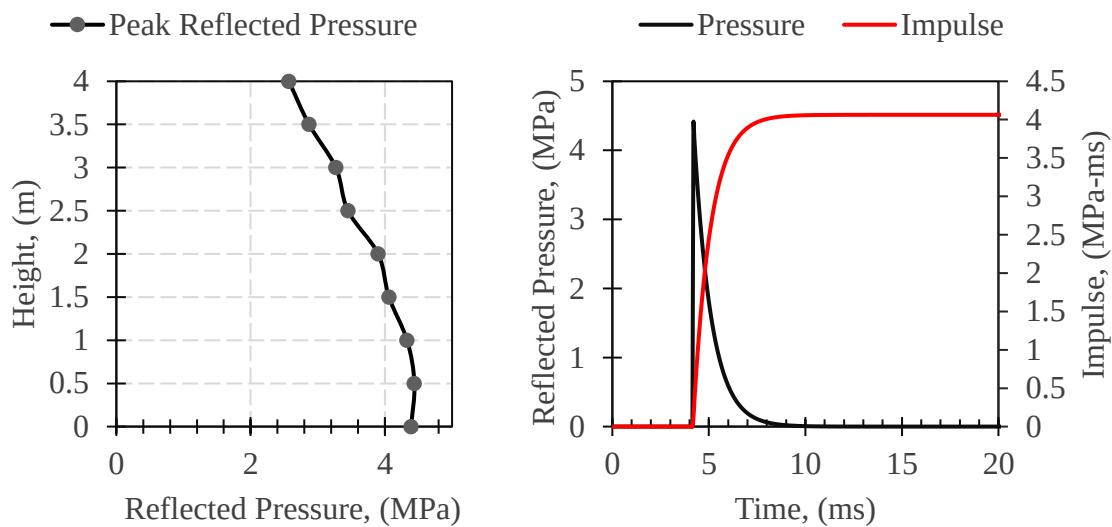
Because of the relatively long standoff distance of CS5 loading the column performed in the flexure mode even with the wide tie spacing equal to the column cross-sectional dimension. Maximum displacement of 22.3 mm was recorded at 16.5 ms at a height of 2400 mm above the ground surface. Maximum support rotation was 0.80° (calculated from the top). The displacement time history is shown in Figure 5-19. The maximum strain experienced by the longitudinal reinforcements was 0.0262 mm/mm. The maximum strain experienced by the transverse reinforcements was 0.0092 mm/mm.

The progression of damage is presented in Figure A-9 (a). Many flexural cracks were initiated near the column mid-span and bottom support regions at time 8 ms. Partial spalling of concrete cover was observed on the back face of the column at time 20 ms near the mid-height. Significant damage was detected on the column back face due to flexure with the loss of 25 % of core concrete at time 80 ms. The column remained intact with significant residual strength under this loading condition.

5.2.9.2 Corner Column with Tie Spacing Equal to Half the Section Dimension (CC-TS2 under CS5)

The column performed predominantly in flexure mode with significant residual capacity at the end of loading. A maximum displacement of 20.1 mm was recorded at 15.0 ms at a height of 2220 mm above the ground surface. Maximum support rotation was 0.65° (calculated from the top). The displacement time history is shown in Figure 5-19. The maximum strain experienced by the longitudinal reinforcements was 0.0224 mm/mm. The maximum strain experienced by the

transverse reinforcements was 0.0076 mm/mm. The progression of damage is presented in Figure A-9 (b). Significant flexural cracking occurred near the column mid-height at time 8 ms. Damage accumulation was observed on the front face of the column near the base. Partial loss of concrete cover was also observed on the back face near the mid-height region at time 20 ms. Minor concrete damage was observed on the column back face due to flexure, without losing any core concrete. This is shown in the figure at time 80 ms.



(a) Peak Reflected Pressure along the Height (b) Pressure/Impulse Time History at the Base

Figure 5-18: CS5 Blast Load Parameters

5.2.9.3 Corner Column with Tie Spacing Equal to Quarter of the Column Sectional Dimension (CC-TS3 under CS5)

The column performed in the flexure mode with significant flexural cracking in the positive and negative moment regions at column mid-height and near the bottom support region, respectively. Maximum displacement of 19.4 mm was detected at 14.0 ms at a height of 2220 mm above the ground surface. Maximum support rotation was 0.62° (calculated from the top). The displacement time history is shown in Figure 5-19. The maximum strain experienced by the longitudinal reinforcement was 0.0139 mm/mm. The maximum strain experienced by the transverse reinforcements was 0.0038 mm/mm.

The progression of damage is presented in Figure A-9 (c). Significant flexural cracks formed in the column mid-span region, as well as the bottom support region at time 8 ms. At 10 ms, the cracks have become wider. The damage was limited to the tension regions, without the crushing of concrete, leaving column with significant residual strength at time 80 ms.

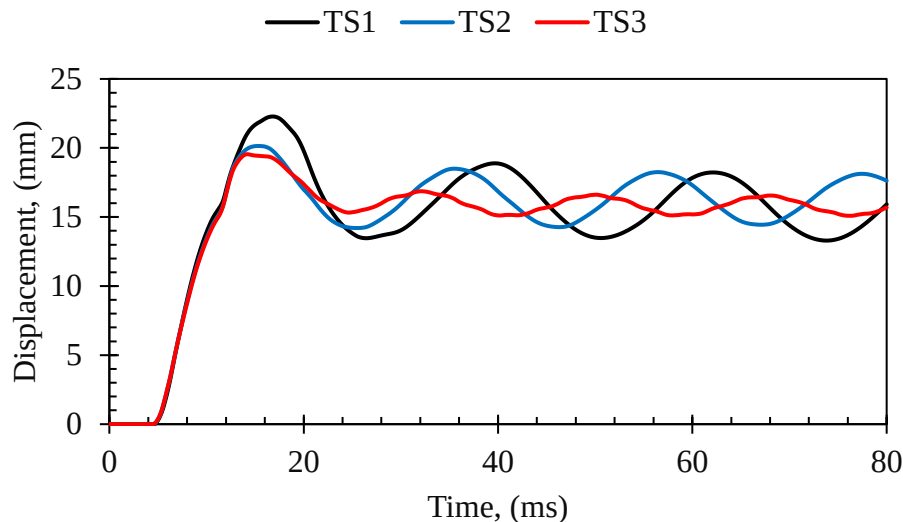


Figure 5-19: Displacement Time History for CC Columns under CS5 Loading

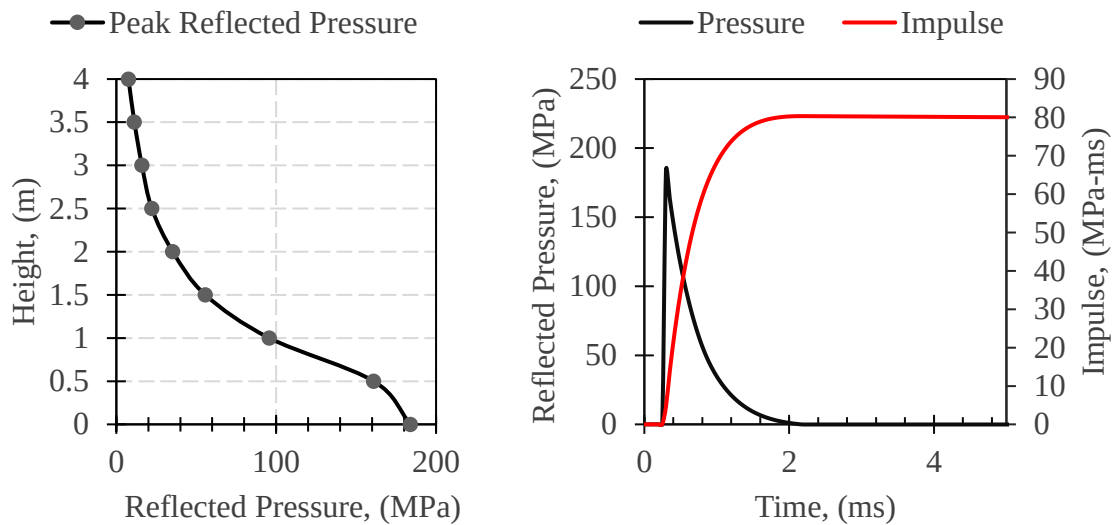
5.2.10 Response of Corner Columns CC-TS1, CC-TS2 and CC-TS3 to Sedan Car Bomb at 1.5 m (S1 Blast Load)

The columns were subjected to the same axial compression as the previous series of columns prior to blast loading. Axial load was applied linearly with a rate of 39.8 kN/ms and held constantly at a magnitude of 597 kN for 65 ms prior to the blast load application. The axial load was kept constant during blast load application. The blast loading was generated by 454 kg of TNT placed on the ground surface at a standoff distance of 1.5 m, which corresponded to a scaled standoff distance of $0.195 \text{ m/kg}^{1/3}$. A non-uniform distributed load was generated from the TNT charge, and was applied on the front face of the columns. The peak reflected pressure distribution along the height of the column was recorded and is shown in Figure 5-20 (a). A peak reflected pressure of 184 MPa, reflected impulse of 79.9 MPa-ms with positive phase duration of 1.95 ms was recorded at the bases of each column. The time of arrival of the shock wave (measured from the

center of the charge to the base of the columns) was 0.25 ms. Pressure and impulse time histories are shown in Figure 5-20 (b).

5.2.10.1 Corner Column with Tie Spacing Equal to Column Section Dimension (CC-TS1 under S1)

The column suffered excessively, hence there was no measurements of maximum displacement and maximum strain. The displacement time history is shown in Figure 5-21. The progression of damage is presented in Figure A-10 (a). Heavy local damage was observed near the base of the column on the tension side at time 1 ms. Direct shearing was observed at the base of the column taking place at time 2 ms that led to the column failure. Subsequent damage observed within the column height is illustrated in Figure A-10 (a) at 4 ms and 10 ms.



(a) Peak Reflected Pressure along the Height (b) Pressure/Impulse Time History at the Base

Figure 5-20: S1 Blast Load Parameters

5.2.10.2 Corner Column with Tie Spacing Equal to Half the Column Section Dimension (CC-TS2 under S1)

The column experienced severe damage once again, because of the close proximity of the charge. Hence, no maximum displacement or maximum strain measurements could be recorded. The displacement time history is shown in Figure 5-21. The progression of damage is presented in Figure A-10 (b). Severe damage at the column base occurred at time 1 ms. Direct shear failure

initiated at the base at time 2 ms that eventually led to the collapse of the column. The figure illustrates the extent of damage observed at 4 ms and 10 ms.

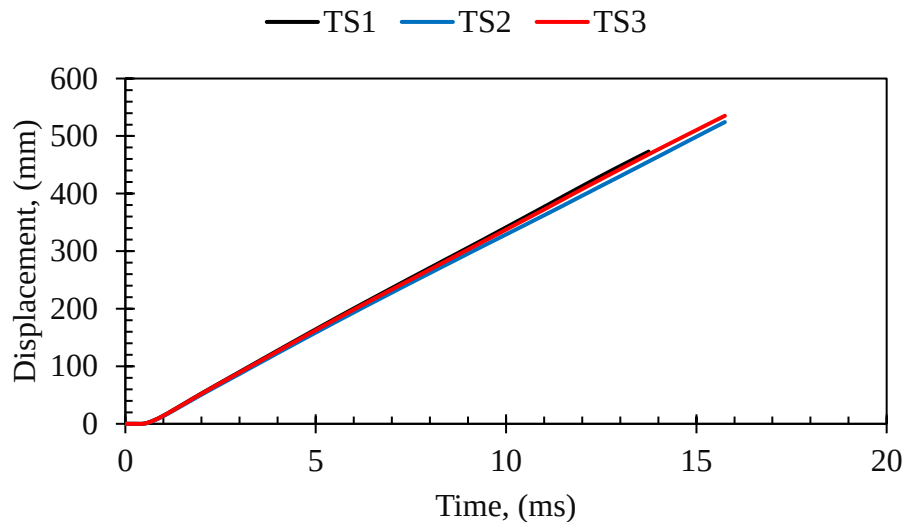


Figure 5-21: Displacement Time History for CC Columns under S1 Loading

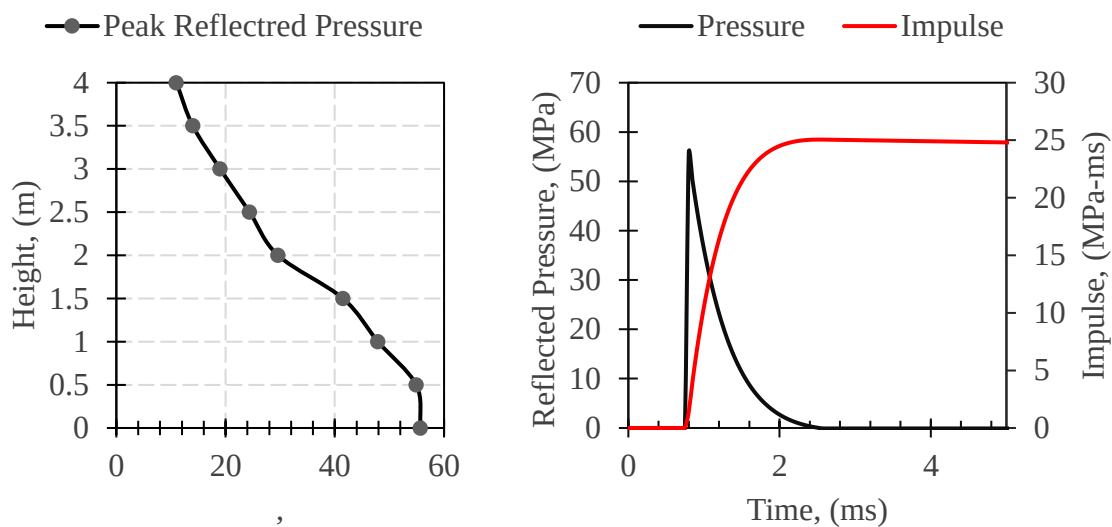
5.2.10.3 Corner Column with Tie Spacing Equal to Quarter of the Column Section Dimension (CC-TS3 under S1)

Further reduction in tie spacing did not change the failure mode of the column, relative to the previous two columns, as the standoff distance was too close to the column and the failure mode was governed by direct shear. The damage was severe and hence, no maximum displacement or maximum strain could be measured for this column. The displacement time history is shown in Figure 5-21. The progression of damage is presented in Figure A-10 (c). Severe damage at the column base occurred at time 1 ms. Direct shear failure initiated at the bottom support of the column at time 2 ms, which eventually led to the collapse of the column. The figure illustrates the extent of damage observed at 4 ms and 10 ms.

5.2.11 Response of Corner Columns CC-TS1, CC-TS2 and CC-TS3 to Sedan Car Bomb at 3.0 m (S2 Blast Load)

The columns were first subjected to axial load, which was applied linearly at a rate of 39.8 kN/ms. The load was then kept constant at a magnitude of 597 kN for 65 ms prior and during the time which blast load was applied. The blast loading was triggered by a 454 kg of TNT, placed on the

ground surface at a standoff distance of 3.0 m, which corresponded to a scaled standoff distance of $0.390 \text{ m/kg}^{1/3}$. A non-uniform distributed load was generated by the TNT charge, and was applied on the front face of the columns. The peak reflected pressure distribution along the height of each column was recorded and is shown in Figure 5-22 (a). A peak reflected pressure of 55.7 MPa, reflected impulse of 24.7 MPa-ms with positive phase duration of 1.8 ms was recorded at the bases of the columns. The time of arrival of the shock wave (measured from the center of the charge to the base of the columns) was 0.75 ms. Pressure and impulse time histories are shown in Figure 5-22 (b).



(a) Peak Reflected Pressure along the Height (b) Pressure/Impulse Time History at the Base

Figure 5-22: S2 Blast Load Parameters

5.2.11.1 Corner Column with Tie Spacing Equal to Column Section Dimension (CC-TS1 under S2)

The column suffered severe damage caused mostly by direct shear, and therefore the maximum displacement and maximum strain measurements could not be obtained for this column. The displacement time history is shown in Figure 5-23. The progression of damage is presented in Figure A-11 (a). Severe damage near the column base was detected at time 2 ms. Flexural cracks and diagonal shear crack were also observed at time 5 ms. The column resistance was mostly

governed by direct shear at the bottom support, which initiated at time 3 ms and eventually resulted in the total collapse to the column.

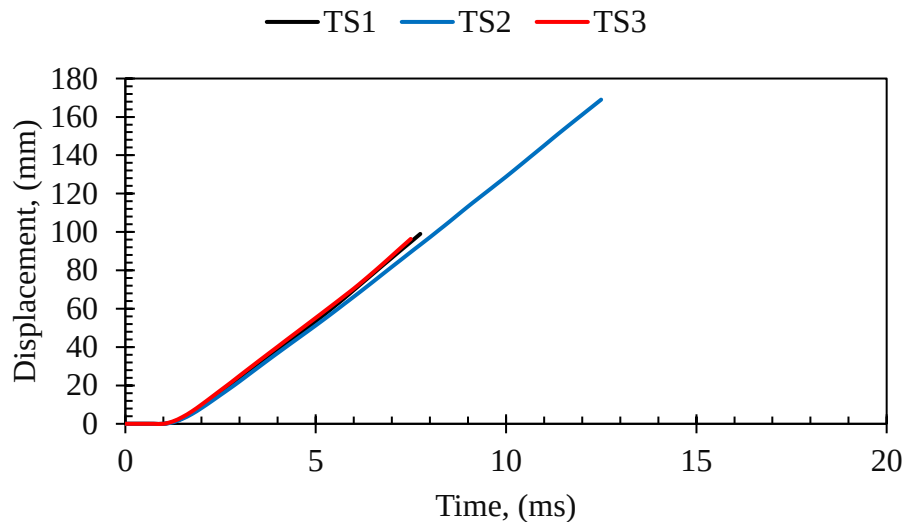


Figure 5-23: Displacement Time History for CC Columns under S2 Loading

5.2.11.2 Corner Column with Tie Spacing Equal to Half the Column Section Dimension (CC-TS2 under S2)

This column had half the tie spacing as the previous column, but because of the proximity of the explosive to the column this did not help the direct shear failure mode. Hence the damage was excessive and no maximum displacement or maximum strain could be measured. The displacement time history is shown in Figure 5-23. The progression of damage is presented in Figure A-11 (b). Heavy damage near the column base occurred at time 5 ms. Direct shearing of the column at its bottom support region was observed at 7 ms. This was accompanied by the formation of flexural and inclined shear cracks. Column failed due to direct shear at the base as shown in Figure A-11(b) at time 12 ms.

5.2.11.3 Corner Column with Tie Spacing Equal to Quarter of the Column Section Dimension (CC-TS3 under S2)

This column had closely spaced ties, which reduced the damage along the height, but because of the close standoff distance the dominant failure mode remained as direct shear, causing severe damage. For this reason, maximum displacement and maximum strain measurements could not be

recorded. The displacement time history is shown in Figure 5-23. The progression of damage is presented in Figure A-11 (c). The column was highly stressed near the bottom support at 2.0 ms while also developing some flexural cracks within the bottom 1/3rd. Extensive flexural cracking occurred at time 5 ms. The column response was mostly dominated by direct shear at the bottom support. This led to the initiation of failure at 7 ms, followed by significant deterioration of concrete within the bottom half of the column at time 12 ms.

5.2.12 Response of Corner Columns CC-TS1, CC-TS2 and CC-TS3 to Sedan Car Bomb at 4.5 m (S3 Blast Load)

The columns were first subjected to axial load, which was applied linearly at a rate of 39.8 kN/ms. The load was then kept constant at a magnitude of 597 kN for 65 ms prior and during the time which blast load was applied. The blast loading was triggered by a 454 kg of TNT, placed on the ground surface at a standoff distance of 4.5 m, which corresponded to a scaled standoff distance of $0.586 \text{ m/kg}^{1/3}$. A non-uniform distributed load was generated by the TNT charge, and was applied on the front face of the columns. The peak reflected pressure distribution along the height of the column was recorded and is shown in Figure 5-24 (a). A peak reflected pressure of 27.4 MPa, reflected impulse of 14.3 MPa-ms with positive phase duration of 2.8 ms was recorded at the base of each column. The time of arrival of the shock wave (measured from the center of the charge to the base of the columns) was 1.4 ms. Pressure and impulse time histories are shown in Figure 5-24 (b).

5.2.12.1 Corner Column with Tie Spacing Equal to Column Section Dimension (CC-TS1 under S3)

The increase in the standoff distance to 4.5 meters changed the mode of behaviour in this series of columns from direct shear to a combination of diagonal tension and flexure. The column continued to suffer severe damage and hence no maximum displacement or maximum strain measurements were possible. The displacement time history recorded for the column is shown in Figure 5-25. The progression of damage is presented in Figure A-12 (a). Clear diagonal shear crack initiation was observed at time 2.5 ms. Progression of diagonal shear crack near the bottom support at time 3.5 ms led to the column failure. Flexural damage was also observed, resulting in the crushing of

column core concrete at failure. The column damage at different stages of loading is illustrated in Figure A-12.

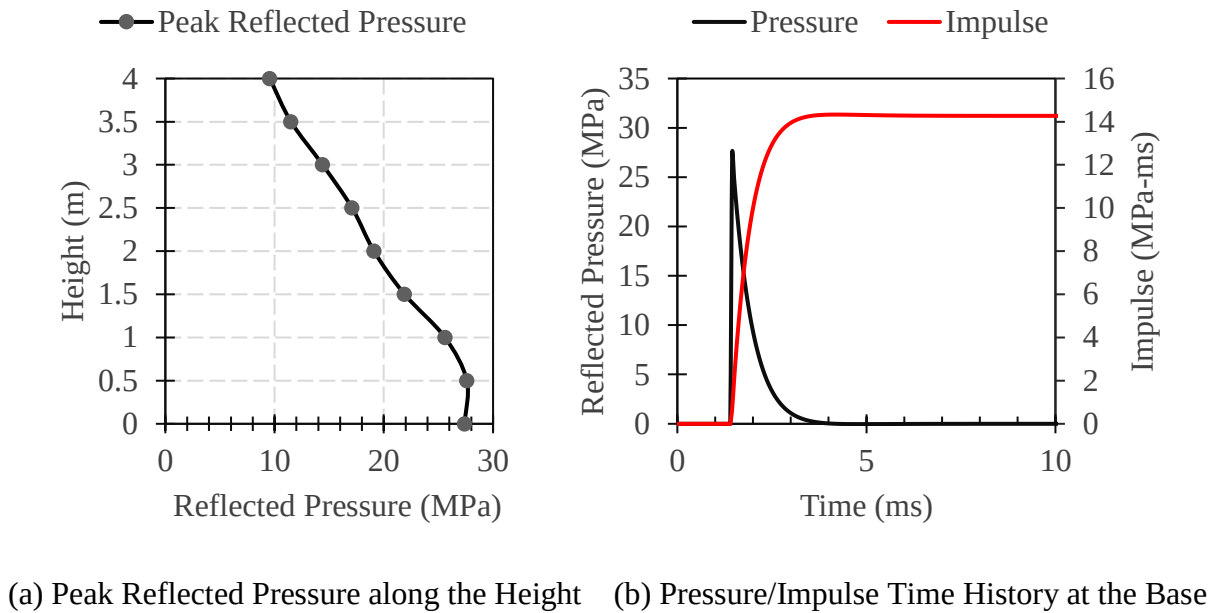


Figure 5-24: S3 Blast Load Parameters

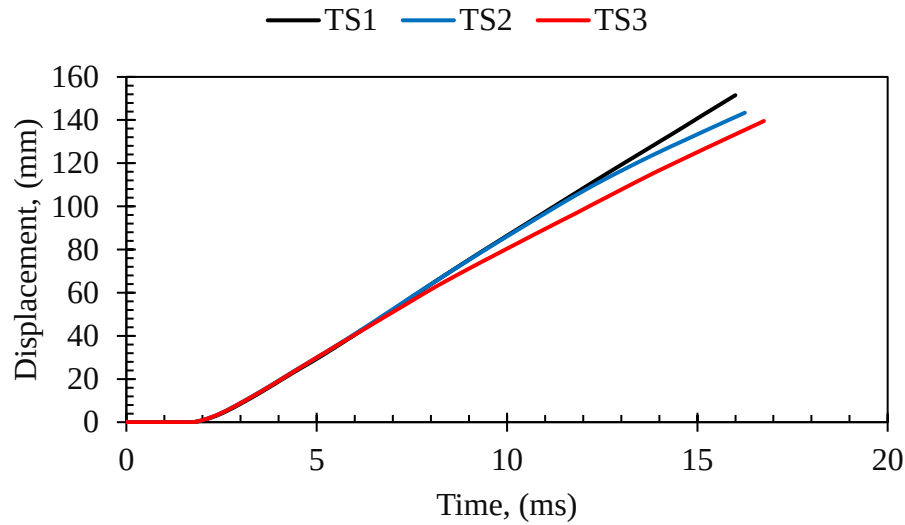


Figure 5-25: Displacement Time History for CC Columns under S3 Loading

5.2.12.2 Corner Column with Tie Spacing Equal to Half the Column Section Dimension (CC-TS2 under S3)

This column had half the tie spacing as the previous column, which resulted in better diagonal shear crack control. The increase in the standoff distance to 4.5 metres eliminated direct shear failure. Nevertheless, the column suffered from severe damage with no readings for maximum displacement and maximum strain measurements. The displacement time history is shown in Figure 5-25. The progression of damage is presented in Figure A-12 (b). The formation of diagonal shear cracks can be observed at times 3.5 ms, 6 ms and 10 ms. The combination of diagonal tension and diagonal compression resulted in the failure of the column at time 14 ms.

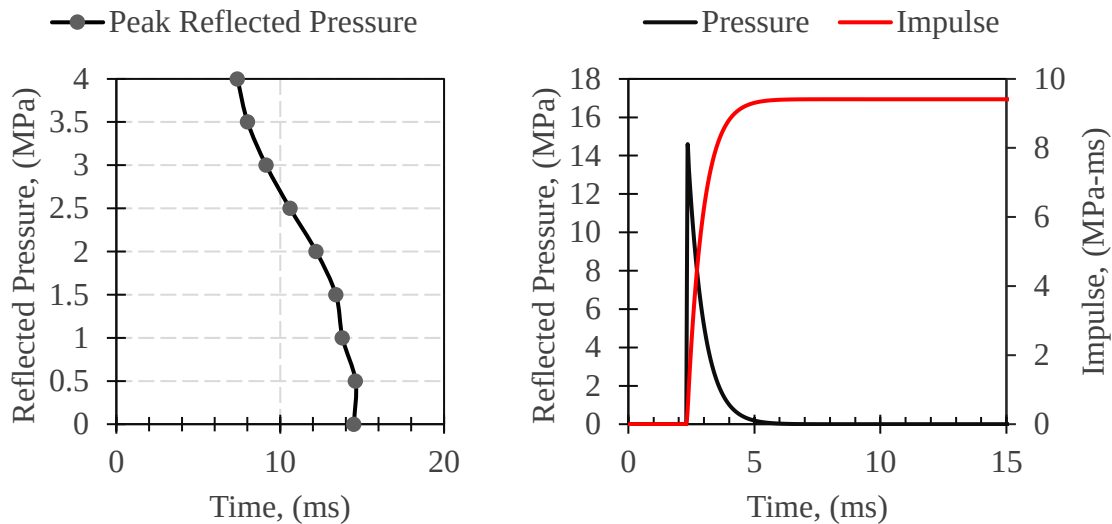
5.2.12.3 Corner Column with Tie Spacing Equal to Quarter of the Column Section Dimension (CC-TS3 under S3)

This column had sufficient high transverse reinforcement ratio that controlled diagonal shear cracking, resulting in flexural failure. However, the column experienced severe damage. Therefore, no readings could be taken for maximum displacement or maximum strains. The displacement time history is shown in Figure 5-25. The progression of damage is presented in Figure A-12 (c). Extensive flexural cracking initiated within the column mid-height region at times 3.5 ms and 6 ms. This resulted in extensive concrete crushing on the front face of the column. The column experienced heavy flexural damage with significant loss of cover and core concrete on the back side of the column.

5.2.13 Response of Corner Columns CC-TS1, CC-TS2 and CC-TS3 to Sedan Car Bomb at 6.0 m (S4 Blast Load)

The columns were subjected to the same axial compression as the previous series of columns prior to blast loading. Axial load was applied linearly with a rate of 39.8 kN/ms and held constantly at a magnitude of 597 kN for 65 ms prior to the blast load application. The axial load was kept constant during blast load application. The blast loading was generated by 454 kg of TNT placed on the ground surface at a standoff distance of 6.0 m which corresponds to a scaled standoff distance of $0.781 \text{ m/kg}^{1/3}$ to simulate the blast load. A non-uniformly distributed load was generated from TNT charge and was applied on the front face of the three columns. The peak reflected pressure distribution along the height of the column was recorded and is shown in Figure

5-26 (a). A peak reflected pressure of 14.5 MPa, reflected impulse of 9.41 MPa-ms with a positive phase duration of 5.85 ms was recorded at the bases of the columns. The time of arrival of the shock wave (measured from the center of the charge to the base of the columns) was 2.3 ms. Pressure and impulse time histories are shown in Figure 5-26 (b).



(a) Peak Reflected Pressure along the Height (b) Pressure/Impulse Time History at the Base

Figure 5-26: S4 Blast Load Parameters

5.2.13.1 Corner Column with Tie Spacing Equal to Column Section Dimension (CC-TS1 under S4)

The column failed due to a combined diagonal tension and flexural behaviours at time 20 ms as shown in Figure A-13 (a). The failure was sudden and severe, hence no maximum displacement and maximum strain measurements were recorded at failure. The displacement time history is shown in Figure 5-27. The progression of damage is presented in Figure A-13 (a). Many flexural cracks were initiated near the column mid-height at times 6 ms and 8 ms. The increase in tensile and compressive stresses resulted in the loss of cover and core concrete near the bottom support and near the mid-height, and hence complete sudden failure.

5.2.13.2 Corner Column with Tie Spacing Equal to Half the Column Section Dimension (CC-TS2 under S4)

The reduced tie spacing in this column shifted the mode of behaviour completely to flexure. The eventual failure was brittle and did not permit the recording of maximum displacement and maximum strain measurements. The displacement time history is shown in Figure 5-27. The progression of damage is presented in Figure A-13 (b). Flexural cracking was observed in the initial stages of loading as shown at time 5 ms. Extensive flexural tension cracking was observed near the base and mid-height regions, followed by the onset of concrete crushing at about 14 ms. Deterioration of concrete, near the bottom support region, leading to the formation of a plastic hinge was observed at about 20 ms. In conjunction with loss of core concrete near mid-span, resulting in severe and sudden column failure.

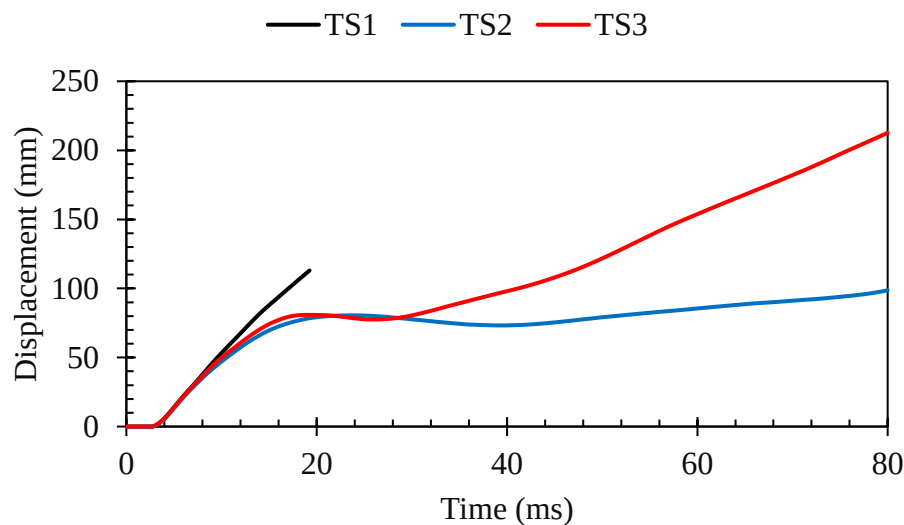


Figure 5-27: Displacement Time History for CC Columns under S4 Loading

5.2.13.3 Corner Column with Tie Spacing Equal to Quarter of the Column Section Dimension (CC-TS3 under S4)

Further reduction in tie spacing did not change the failure mode of the column, relative to the previous column. The failure was brittle and did not permit the recording of maximum displacement and maximum strain measurements. The displacement time history is shown in Figure 5-27. The progression of damage is presented in Figure A-13 (c). The column experienced

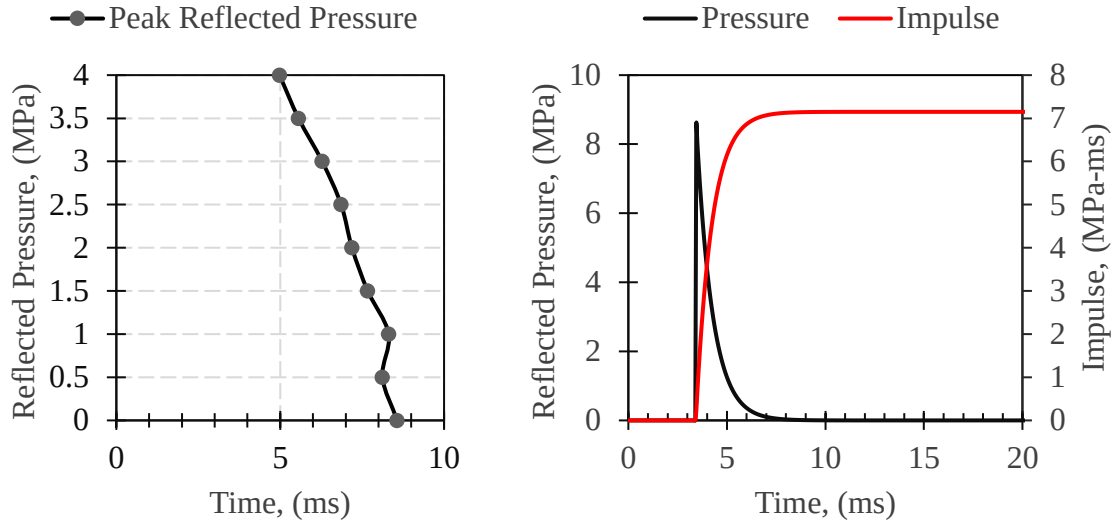
only flexural cracking in the initial stages of loading as shown at time 5 ms. Excessive flexural tension cracking was observed near the base and mid-height regions, followed by the onset of concrete crushing at about 14 ms. Deterioration of concrete, especially near the bottom support region, leading to the formation of a plastic hinge was observed at about 20 ms. This resulted in a sudden failure of the column.

5.2.14 Response of Corner Columns CC-TS1, CC-TS2 and CC-TS3 to Sedan Car Bomb at 7.5 m (S5 Blast Load)

The columns in this series were analysed under combined axial and blast loading as explained before. The axial load was applied gradually with a rate of 39.8 kN/ms, and held constant at a magnitude of 597 kN for 65 ms prior to the application of the blast load and kept constant during the blast load stage. The blast loading was generated by 454 kg of TNT, placed on the ground surface at a standoff distance of 7.5 m which corresponds to a scaled standoff distance of $0.976 \text{ m/kg}^{1/3}$ to simulate the blast load. A non-uniform load was generated from the TNT charge, and was applied on the front face of the columns. The peak reflected pressure distribution along the height of the column was recorded and is shown in Figure 5-28 (a). A peak reflected pressure of 8.57 MPa, reflected impulse of 7.15 MPa-ms with positive phase duration of 12.3 ms were recorded at the base of each column. The time of arrival of the shock wave (measured from the center of the charge to the base of the columns) was 3.4 ms. Pressure and impulse time histories are shown in Figure 5-28 (b).

5.2.14.1 Corner Column with Tie Spacing Equal to Column Section Dimension (CC-TS1 under S5)

The increase of standoff distance to 7.5 m caused a combination of failure modes in the form of diagonal tension and flexure. The column suffered severe damage, and therefore the maximum displacement and maximum strain measurements could not be obtained for this column. The displacement time history recorded for the column is shown in Figure 5-29. The progression of damage is presented in Figure A-14 (a). Clear diagonal shear crack initiation was observed at time 10 ms. Progression of diagonal shear cracks near the top and bottom support at time 20 ms led to the column failure. Flexural damage was also observed, resulting in the crushing of column core concrete at failure. The column damage at different stages of loading is illustrated in Figure A-14.



(a) Peak Reflected Pressure along the Height (b) Pressure/Impulse Time History at the Base

Figure 5-28: S5 Blast Load Parameters

5.2.14.2 Corner Column with Tie Spacing Equal to Half the Column Section Dimension (CC-TS2 under S5)

The reduced ties spacing in this column resulted in flexural behaviour with the control of diagonal shear cracks. The maximum displacement was 49.8 mm, which occurred at 17.5 ms at a height of 2400 mm above the ground surface. The maximum support rotation was 1.78° (calculated from the top). The displacement time history is shown in Figure 5-29. The maximum strain experienced by the longitudinal reinforcement was 0.0295 mm/mm. The maximum strain experienced by the transverse reinforcements was 0.0043 mm/mm. The progression of damage is presented in Figure A-14 (b). The column responded primarily in flexure with increased flexural cracks near the column mid-height at time 8 ms. At 10 ms, damage accumulation was observed on the front face of the column near the base. Partial loss of concrete cover was observed on the back face at time 20 ms near mid-height. Deterioration of concrete, especially near the bottom support region was the major damage experienced by this column as it remained in-place at time 80 ms.

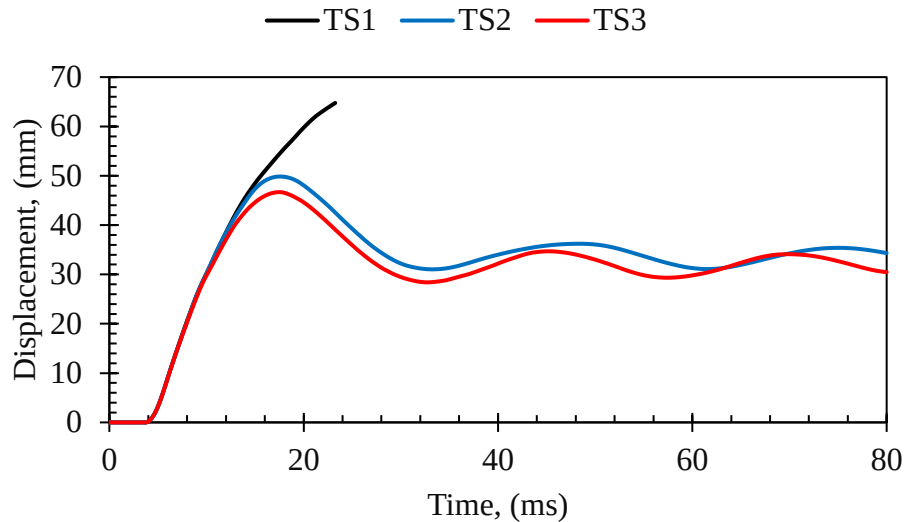


Figure 5-29: Displacement Time History for CC Columns under S5 Loading

5.2.14.3 Corner Column with Tie Spacing Equal to Quarter of the Column Section Dimension (CC-TS3 under S5)

This column had closely spaced ties which reduced the overall damage experienced by the column relative to the previous columns. The maximum displacement was 46.7 mm, which occurred at 17.5 ms at a height of 2400 m above the ground surface. The maximum support rotation was 1.67° (calculated from the top). The displacement time history is shown in Figure 5-29. The maximum strain experienced by the longitudinal reinforcement was 0.0341 mm/mm. The maximum strain experienced by the transverse reinforcements was 0.0068 mm/mm. The progression of damage is presented in Figure A-14 (c). The column showed flexural behaviour with cracks occurring near the column mid-height at times 6 ms, 8 ms and 10 ms. At 10 ms. Damage accumulation was observed on the front face of the column near the base. Partial loss of concrete cover was observed on the back face at time 20 ms near the mid-height.

5.2.15 Response of Edge Columns EC-TS1, EC-TS2 and EC-TS3 to Backpack Bomb at 0.5 m (BP1 Blast Load)

Three edge columns with three transverse reinforcement spacing were analyzed under a 22.67 kg backpack bomb placed at 0.5 m as the standoff distance. The axial load was applied linearly first, with a rate of 79.6 kN/ms and held constant beyond the maximum intended magnitude of 1194 kN

for 65 ms. Then, the blast load was applied on the front face of the columns while keeping the axial load at a constant value of 1194 kN. The blast loading parameters are the same as those described in Section 5.2.1.

5.2.15.1 Edge Column with Tie Spacing Equal to Column Section Dimension (EC-TS1 under BP1)

The column experienced severe damage. The displacement time history is shown in Figure 5-30. The progression of damage is presented in Figure A-15 (a). Concrete scabbing was initiated at time 2 ms on the tension side of the column near the base. Continuous loss of core concrete can be seen at times 5 ms, 8 ms and 12 ms which caused complete column breaching as shown at time 12 ms.

5.2.15.2 Edge Column with Tie Spacing Equal to Half the Column Sectional Dimension (EC-TS2 under BP1)

The column experienced severe damage. The displacement time history is shown in Figure 5-30. The progression of damage is presented in Figure A-15 (b). Concrete scabbing was observed at the back face of the column near the base at time 2 ms. At time 12 ms, more concrete loss was observed in the localized base region. The column failed at time 24 ms due to the breaching of column.

5.2.15.3 Edge Column with Tie Spacing Equal to Quarter of the Column Sectional Dimension (EC-TS3 under BP1)

This column had the closest spacing of ties, which helped its performance. Maximum displacement was 18.8 mm, which occurred at 7.5 ms at a height of 775 mm above the ground surface. Maximum support rotation was 1.40° (calculated from the bottom). The displacement time history is shown in Figure 5-30. The maximum strain experienced by the longitudinal reinforcement was 0.0484 mm/mm. The maximum strain experienced by the transverse reinforcement was 0.0333 mm/mm.

The progression of damage is presented in Figure A-15 (c). Localized damage was detected at time 0.5 ms near the column base. Loss of concrete elements can be observed at time 2 ms. Shear localized damage led to the scabbing of concrete on the back face as shown at time 80 ms. The column experienced some damage but behaved well under the specified blast load.

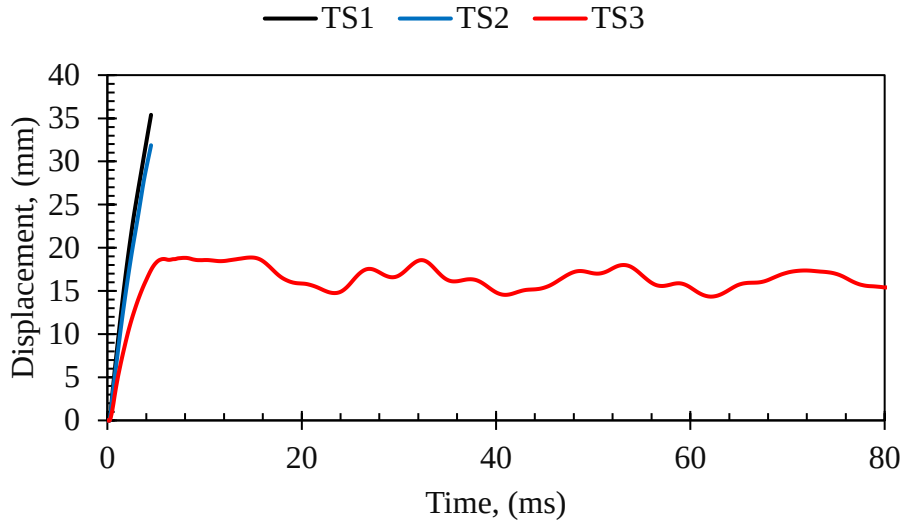


Figure 5-30: Displacement Time History for EC Columns under BP1 Loading

5.2.16 Response of Edge Columns EC-TS1, EC-TS2 and EC-TS3 to Backpack Bomb at 0.75 m (BP2 Blast Load)

The column was loaded by following the same procedure applied on the previous three corner columns with the axial load first applied linearly with a rate of 79.6 kN/ms, and held constant at a magnitude of 1194 kN for 65 ms. Once the axial load reached its maximum level, the blast load was applied while maintaining the axial load at the maximum level. The blast load consisted of a backpack bomb with a charge weight of 22.67 kg at a standoff distance of 0.75 m. The blast loading parameters are the same as described in Section 5.2.2.

5.2.16.1 Edge Column with Tie Spacing Equal to Column Section Dimension (EC-TS1 under BP2)

The column experienced severe damage. The displacement time history is shown in Figure 5-31. The damage evolution of the column is presented in Figure A-16 (a). Clear local shear damage at the back face of the column near the base at time 1 ms. Complete column breaching occurred at time 15 ms. Extensive cracking and concrete crushing observed along the height of the column at the onset of beaching indicated significant inelasticity in flexure along the length of the column.

5.2.16.2 Edge Column with Tie Spacing Equal to Half the Column Sectional Dimension (EC-TS2 under BP2)

The column experienced severe damage. The displacement time history is shown in Figure 5-31. The progression of damage is presented in Figure A-16 (b). Noticeable local damage on the back face of the column near the base was observed at 2 ms. Scabbing occurred on the tension side of the column and progressed to complete breaching. After losing core concrete, the bars buckled as shown at time 60 ms and caused column failure.

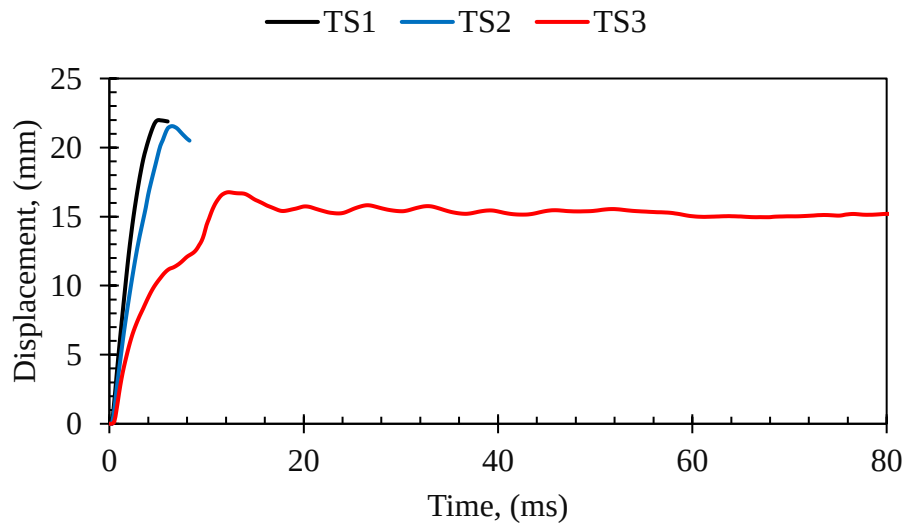


Figure 5-31: Displacement Time History for or EC Columns under BP2 Loading

5.2.16.3 Edge Column with Tie Spacing Equal to Quarter of the Column Sectional Dimension (EC-TS3 under BP2)

This column had closely spaced ties. The maximum displacement of 16.7 mm was measured at 12 ms, at a height of 1070 mm above the ground surface. The maximum support rotation was 0.89° (calculated from the bottom). The displacement time history is shown in Figure 5-31. The maximum strain experienced by the longitudinal reinforcement was 0.0115 mm/mm. The maximum strain experienced by the transverse reinforcement was 0.0254 mm/mm.

The progression of damage is presented in Figure A-16 (c). Local damage at the back face of the column at time 2 ms was observed. The column showed very good performance and survived the

blast loading with only scabbing of concrete elements at the back face. This was the major damage experienced by the column during blast response.

5.2.17 Response of Edge Columns EC-TS1, EC-TS2 and EC-TS3 to Backpack Bomb at 1.5 m (BP3 Blast Load)

The columns were first loaded with axial compression. The axial load was applied at a rate of 79.6 kN/ms and held constant for 65 ms at a magnitude of 1194 kN prior to the application of blast loading. The blast load was applied while the axial load was constant at 1194 kN. The blast corresponded to the detonation of 22.67 kg of TNT on the ground surface at a standoff distance of 1.5 m. The blast loading parameters are the same as described in Section 5.2.3.

5.2.17.1 Edge Column with Tie Spacing Equal to Column Section Dimension (EC-TS1 under BP3)

The column experienced severe damage. The displacement time history recorded is shown in Figure 5-32. The progression of damage in the column is presented in Figure A-17 (a). The formation of diagonal tension cracks associated with shear in the bottom support region can be observed in the figure with progressively increasing damage at times 1 ms, 2 ms and 5 ms and 10 ms. This resulted in the collapse of the column as shown at time 35 ms.

5.2.17.2 Edge Column with Tie Spacing Equal to Half the Column Sectional Dimension (EC-TS2 under BP3)

The maximum displacement recorded during blast response was 13.7 mm, which occurred at 10 ms at a height of 2000 mm above the ground surface. The maximum support rotation was 0.39° (calculated from the bottom). The displacement time history is shown in Figure 5-32. The maximum strain experienced by the longitudinal reinforcement was 0.0068 mm/mm. The maximum strain experienced by the transverse reinforcement was 0.0416 mm/mm.

The column experienced severe damage. The displacement time history recorded is shown in Figure 5-32. The progression of damage is presented in Figure A-17 (b). Diagonal shear crack formation near the bottom support region was observed at times 2 ms, 5 ms, 10 ms and 80 ms. Extensive flexural cracking was experienced in the column near the mid-height. However, the dominant failure mode for this column was diagonal shear cracking, as shown at time 80 ms.

5.2.17.3 Edge Column with Tie Spacing Equal to Quarter of the Column Sectional Dimension (EC-TS3 under BP3)

This column had closely spaced ties. The detonation took place at a relatively distant location. Hence, the column behaved predominantly in the flexure mode. The maximum displacement of 11.5 mm was recorded at 9.5 ms at a height of 2000 mm above the ground surface. The maximum support rotation was 0.32° (calculated from the bottom). The displacement time history is shown in Figure 5-32. The maximum strain experienced by the longitudinal reinforcement was 0.0102 mm/mm. The maximum strain experienced by the transverse reinforcement was 0.0136 mm/mm.

The progression of damage is presented in Figure A-17 (c). The concrete side cover was observed to be lost during the blast event. Flexural cracks occurred at 5 ms near the column mid span. The column was dominated by flexure and showed relatively good performance in comparison to all the columns discussed earlier, developing approximately 9.8 mm of residual displacement.

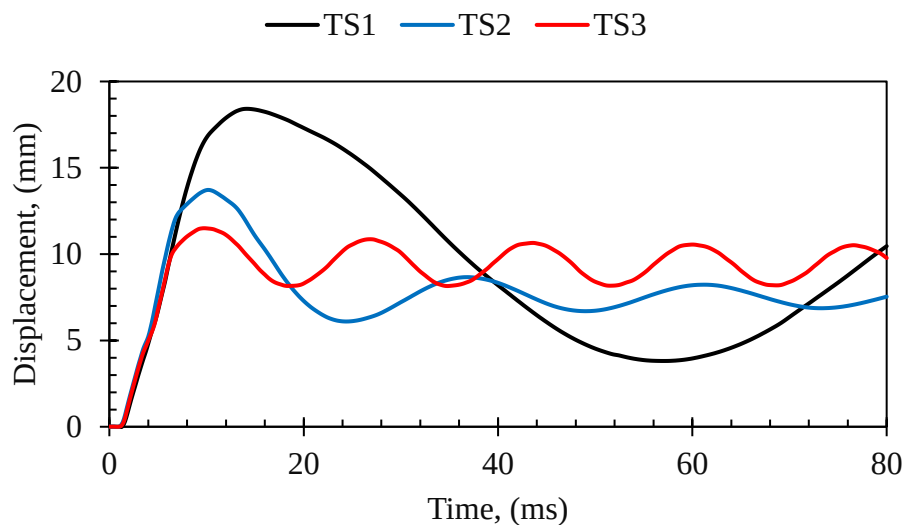


Figure 5-32: Displacement Time History for or EC Columns under BP3 Loading

5.2.18 Response of Edge Columns EC-TS1, EC-TS2 and EC-TS3 to Backpack Bomb at 3.0 m (BP4 Blast Load)

The columns were loaded by axial load with a rate of 79.6 kN/ms prior to the application of blast loads. The axial load remained constant at a magnitude of 1194 kN for 65 ms prior to during the blast load application. The blast load was triggered by 22.67 kg of TNT placed on the ground

surface at a standoff distance of 3.0 m. The blast loading parameters are the same as described in Section 5.2.4.

5.2.18.1 Edge Column with Tie Spacing Equal to Column Section Dimension (EC-TS1 under BP4)

The column showed predominantly flexural response with a maximum displacement 4.69 mm, which occurred at 8 ms at a height of 2000 mm above the ground surface. The maximum support rotation was 0.13° (calculated from the bottom). The displacement time history is shown in Figure 5-33. The maximum strain experienced by the longitudinal reinforcement was less than 0.002 mm/mm. The maximum strain experienced by the transverse reinforcement was less than 0.002 mm/mm.

The progression of damage is presented in Figure A-18 (a). The column developed hairline flexural and inclined shear cracking, but otherwise remained virtually elastic with about 0 mm permanent deflection.

5.2.18.2 Edge Column with Tie Spacing Equal to Half the Column Sectional Dimension (EC-TS2 under BP4)

The column showed similar behaviour to that obtained from the earlier column (EC-TS1) with twice the Tie spacing. The maximum displacement of 4.35 mm was recorded, which occurred at 7 ms at a height of 2000 mm above the ground surface. The maximum support rotation was 0.12° (calculated from the bottom). The displacement time history is shown in Figure 5-33. The maximum strain experienced by the longitudinal reinforcement was less than 0.002 mm/mm. The maximum strain experienced by the transverse reinforcement was less than 0.002 mm/mm.

The progression of damage is presented in Figure A-18 (b). Hairline cracks were observed under flexure and shear. There was no concrete crushing, and the column remained essentially elastic with about 1.4 mm permanent deflection.

5.2.18.3 Edge Column with Tie Spacing Equal to Quarter of the Column Sectional Dimension (EC-TS3 under BP4)

The column performed well, demonstrating primarily flexural response with a maximum displacement of 4.28 mm occurring at 7.5 ms at a height of 2000 mm above the ground surface.

The maximum support rotation was measured to be 0.12° (calculated from the bottom). The displacement time history is shown in Figure 5-33. The maximum strain experienced by the longitudinal reinforcement was less than 0.002 mm/mm , remaining elastic. The maximum strain experienced by the transverse reinforcement was less than 0.002 mm/mm .

The progression of damage in the column is presented in Figure A-18 (c). Some hairline flexural and diagonal tension cracking was observed. No crushing of concrete was observed and the column remained intact with about 1.4 mm of permanent deflection.

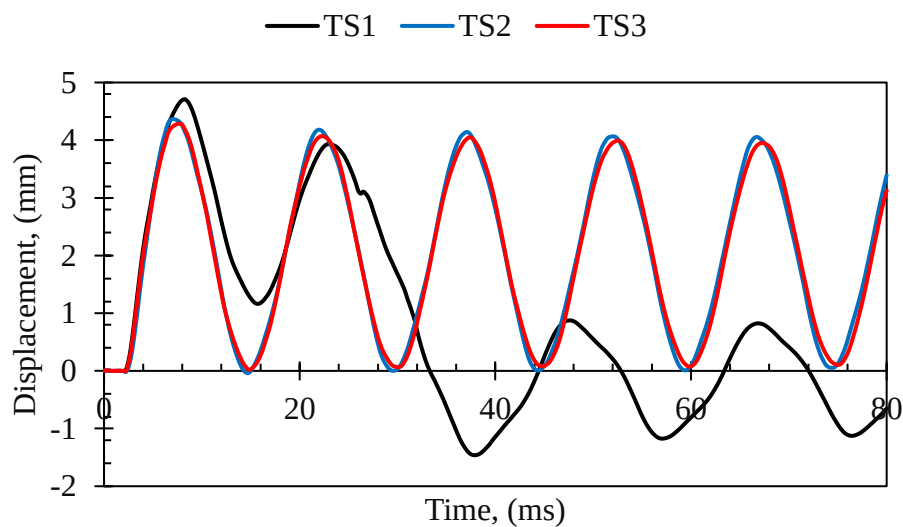


Figure 5-33: Displacement Time History for EC Columns under BP4 Loading

5.2.19 Response of Edge Columns EC-TS1, EC-TS2 and EC-TS3 to Compact Sedan Car Bomb at 1.5 m (CS1 Blast Load)

The columns were loaded axially first at a rate of 79.6 kN/ms up to a magnitude of 1194 kN prior to the application of the blast loading. The axial load was kept constant for 65 ms prior the application of the blast loading and during the blast load stage. The blast loading was generated by 227 kg of TNT, placed on the ground surface at a standoff distance of 1.5 m . The blast loading parameters are the same as described in Section 5.2.5.

5.2.19.1 Edge Column with Tie Spacing Equal to Column Section Dimension (EC-TS1 under CS1)

The column was damaged excessively. Maximum displacement and maximum strain measurements could not be recorded because of the severe damage experienced. The displacement time history is shown in Figure 5-34.

The progression of damage is presented in Figure A-19 (a). Extensive local damage near the base of the column was observed on the back face at 1 ms. Direct-shear failure was detected at the bottom support at 3 ms, which led to column failure. The damage propagated towards the mid height, as column core deteriorated completely at 11 ms.

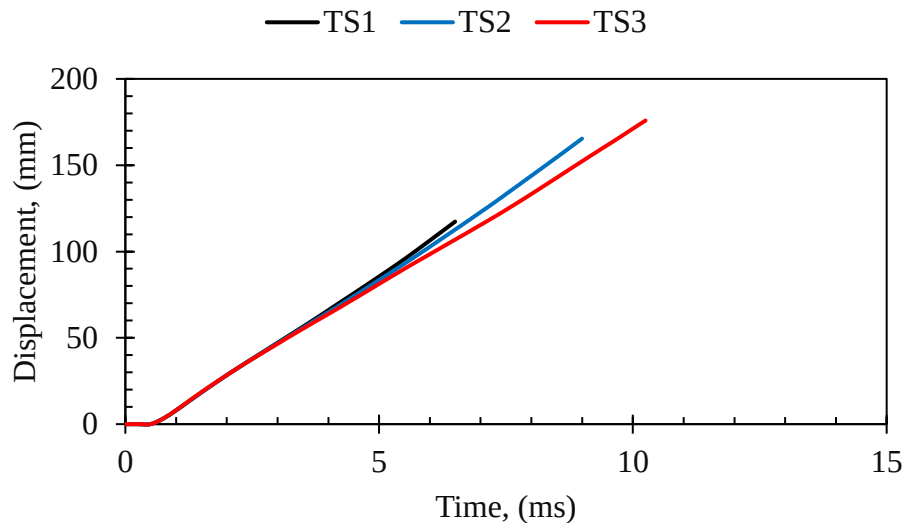


Figure 5-34: Displacement Time History for EC Columns under CS1 Loading

5.2.19.2 Edge Column with Tie Spacing Equal to Half the Column Sectional Dimension (EC-TS2 under CS1)

The column behaved similar to the companion column discussed above with wider spacing of transverse reinforcement. Because it was severely damaged no data could be recorded for maximum displacement and maximum strains. The displacement time history is shown in Figure 5-34.

The progression of damage is depicted in Figure A-19 (b). Heavily stressed local region was detected near the column base at 1 ms. Direct shear failure was observed at the bottom support at 3 ms, which was followed by the disintegration of core concrete between 6 ms near the base and 11 ms covering the bottom half of the column.

5.2.19.3 Edge Column with Tie Spacing Equal to Quarter of the Column Sectional Dimension (EC-TS3 under CS1)

This column behaved similar to the previous two companion columns with wider spacing of tie steel, developing excessive damage. Maximum displacement and maximum strain measurements could not be recorded due to this severe damage. The displacement time history is shown in Figure 5-34.

The column damage is presented in Figure A-19 (c) as observed at different times during response. Heavily stressed region was observed near the base at 1 ms. Direct shear failure initiated at 3 ms, followed by the disintegration of core concrete within the bottom half. The closely spaced transverse reinforcement was not effective in preventing the type of failure observed because of the close standoff distance.

5.2.20 Response of Edge Columns EC-TS1, EC-TS2 and EC-TS3 to Compact Sedan Car Bomb at 3.0 m (CS2 Blast Load)

The columns were axially loaded first prior to the application of the blast load at a rate of 79.6 kN/ms up to a magnitude of 1194 kN, and kept constant for 65 ms. While maintaining the same axial load level, 227 kg of TNT was detonated on the ground surface at a standoff distance of 3.0 m. The blast loading parameters are the same as described in Section 5.2.6.

5.2.20.1 Edge Column with Tie Spacing Equal to Column Section Dimension (EC-TS1 under CS2)

The column experienced severe damage. Therefore, maximum displacement and maximum strain measurements could not be recorded. The displacement time history is shown in Figure 5-35. The progression of damage is presented in Figure A-20 (a). A large diagonal tension crack formed at time 3 ms that led to diagonal shear failure. Concrete deterioration progressed towards the mid-

height, causing the crushing of core concrete under high levels of diagonal compressive and flexural compressive stresses. The column suffered complete failure at about 12 ms.

5.2.20.2 Edge Column with Tie Spacing Equal to Half the Column Sectional Dimension (EC-TS2 under CS2)

The column suffered similar damage to the previous column, with twice the tie spacing. However, the diagonal tension crack was better controlled in this column with wide inclined crack taking place at about 5 ms. Eventually the column suffered a similarly severe damage. Hence, no maximum displacement or maximum strain measurements were taken for this column. The displacement time history is shown in Figure 5-35. The progression of damage is presented in Figure A-20 (b). Local damage near the bottom support was observed in the early stages of loading. Combination of shear and flexural cracks occurred at time 3 ms. Formation of diagonal shear crack occurred at time 3 ms and progressed as shown in the figure at times 5 ms and 7 ms. The formation of flexural cracks near mid span was observed clearly at time 5 ms. The column failed through deterioration of concrete as shown in Fig. A-20 (b) at time 13 ms.

5.2.20.3 Edge Column with Tie Spacing Equal to Quarter of the Column Sectional Dimension (EC-TS3 under CS2)

The column behaved similar to the previous two columns in this group, with much better diagonal tension control due to the close spacing of column ties. Eventually the failure was quite severe with complete deterioration of concrete within the bottom half of the column. There was no maximum displacement or maximum strain measured for this column as it experienced severe damage. The displacement time history is shown in Figure 5-35. The progression of damage is shown in Figure A-20 (c). Many flexural cracks were initiated near the column mid-height at time 3 ms. Diagonal shear crack became visible at about 5 ms. Loss of cover and some of the core concrete on the tension side near the mid-height was observed as shown in the figure at time 10 ms. The column was dominated by a combination of flexure and shear responses. The deterioration of column core concrete caused total failure of the column at time 18 ms.

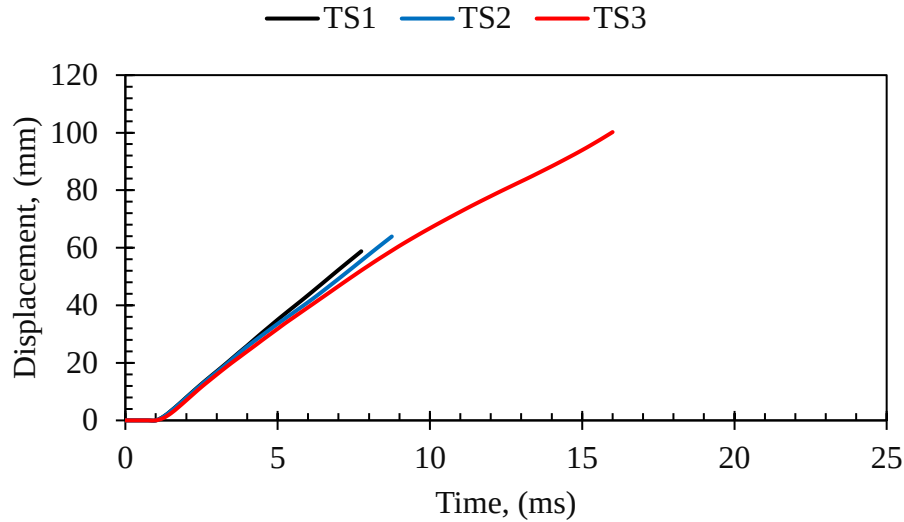


Figure 5-35: Displacement Time History for EC Columns under CS2 Loading

5.2.21 Response of Edge Columns EC-TS1, EC-TS2 and EC-TS3 to Compact Sedan Car Bomb at 4.5 m (CS3 Blast Load)

The three columns analysed in this part of the investigation was first loaded by axial compression, similar to the previous columns, with the axial load applied a rate of 79.6 kN/ms. The load was held constant at a magnitude of 1194 kN during the application of the blast load. The blast load consisted of 227 kg of TNT, placed on the ground surface at a standoff distance of 4.5 m. The blast loading parameters are the same as described in Section 5.2.7.

5.2.21.1 Edge Column with Tie Spacing Equal to Column Section Dimension (EC-TS1 under CS3)

The column developed wide diagonal tension cracks, followed by the deterioration of the core concrete, causing severe damage. Therefore, maximum displacement and maximum strain measurements could not be recorded at failure. The displacement time history is shown in Figure 5-36. The progression of damage in the column is presented in Figure A-21 (a). Diagonal tension and flexural cracks started forming at about 6 ms. Diagonal compression and flexural compression increased to critical levels starting at 10 ms and eventually led to the crushing of column core at about 40 ms. The concrete crushing propagated towards the mid-height at failure.

5.2.21.2 Edge Column with Tie Spacing Equal to Half the Column Sectional Dimension (EC-TS2 under CS3)

The reduced tie spacing in this column shifted the mode of behaviour completely to flexure. The failure was brittle and did not permit the recording of maximum displacement and maximum strain measurements. The displacement time history is shown in Figure 5-36. The progression of damage is presented in Figure A-21 (b). The column experienced only flexural cracking in the initial stages of loading, as shown at time 6 ms. Excessive flexural tension cracking was observed near the base and mid-height regions followed by the onset of concrete crushing at about 20 ms. Deterioration of concrete, especially near the bottom support region, lead to failure at time 80 ms as the column kept displacing beyond that time.

5.2.21.3 Edge Column with Tie Spacing Equal to Quarter of the Column Sectional Dimension (EC-TS3 under CS3)

This column had further reduced tie spacing relative to the previous column. It performed predominantly in flexure mode and developed a maximum displacement of 38.5 mm, which occurred at 12.7 ms at a height of 2300 mm above the ground surface. Maximum support rotation was 1.30° (calculated from the top). The displacement time history is shown in Figure 5-36. The maximum strain experienced by the longitudinal reinforcement was 0.0261 mm/mm. The maximum strain experienced by the transverse reinforcement was 0.0048 mm/mm.

The progression of damage is presented in Figure A-21 (c). Flexural cracking initiated near the column mid-height and bottom support regions at time 6 ms. At 20 ms, the damage level in these regions increased. Damage on the column tension regions occurred at time 80 ms, which led to loss of some core concrete. Crushing of concrete was detected at the bottom support region.

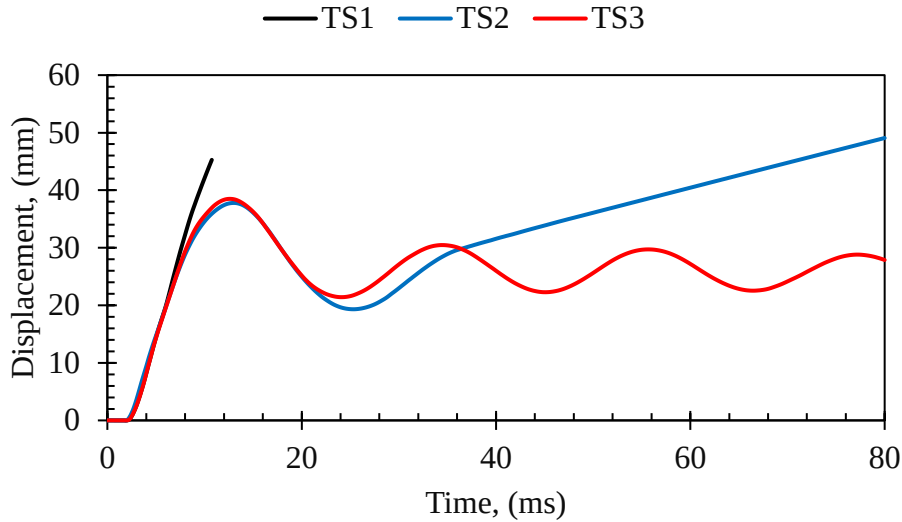


Figure 5-36: Displacement Time History for EC Columns under CS3 Loading

5.2.22 Response of Edge Columns EC-TS1, EC-TS2 and EC-TS3 to Compact Sedan Car Bomb at 6.0 m (CS4 Blast Load)

The columns in this series were analysed under combined axial and blast loading as before. The axial load was applied gradually with a rate of 79.6 kN/ms, and held constant at a magnitude of 1194 kN for 65 ms prior to the application of the blast load and kept constant during the blast loading stage. The blast loading was generated by 227 kg of TNT, placed on the ground surface at a standoff distance of 6.0 m. The blast loading is the same as described in Section 5.2.8.

5.2.22.1 Edge Column with Tie Spacing Equal to Column Section Dimension (EC-TS1 under CS4)

The column performed in the flexure mode with significant flexural cracking at column mid-height and near the bottom support region. The maximum displacement of 26.0 mm was detected at 14.0 ms at a height of 2520 mm above the ground surface. The maximum support rotation was 1.01° (calculated from the top). The displacement time history is shown in Figure 5-37. The maximum strain experienced by the longitudinal reinforcement was 0.0143 mm/mm. The maximum strain experienced by the transverse reinforcement was 0.0205 mm/mm.

The progression of damage is presented in Figure A-22 (a). Significant flexural cracks formed in the column mid-span region, as well as the bottom support region at time 8 ms. Partial spalling of

concrete cover was observed on the back face of the column at time 20 ms near the mid-height. The column had a significant residual strength at time 80 ms.

5.2.22.2 Edge Column with Tie Spacing Equal to Half the Column Sectional Dimension (EC-TS2 under CS4)

The column performed reasonably well with reduced ties spacing. The maximum displacement was 20.5 mm, which occurred at 11.2 ms at a height of 2520 mm above the ground surface. The maximum support rotation was 0.78° (calculated from the top). The displacement time history is shown in Figure 5-37. The maximum strain experienced by the longitudinal reinforcement was 0.0141 mm/mm. The maximum strain experienced by the transverse reinforcement was 0.0233 mm/mm. The progression of damage is presented in Figure A-22 (b). The column responded primarily in flexure with increased flexural cracks near the column mid-height at time 8 ms. At 10 ms, damage accumulation was observed on the front face of the column near the base. Partial loss of concrete cover was observed on the back face at time 20 ms near mid-height. Minor damage on the column back face was caused by flexure, without losing core concrete. This is illustrated in the figure at time 80 ms.

5.2.22.3 Edge Column with Tie Spacing Equal to Quarter of the Column Sectional Dimension (EC-TS3 under CS4)

This column had further reduced tie spacing relative to the previous column. It performed predominantly in the flexure mode and developed a maximum displacement of 20.4 mm, which occurred at 11.0 ms at a height of 2450 mm above the ground surface. The maximum support rotation was 0.79° (calculated from the top). The displacement time history is shown in Figure 5-37. The maximum strain experienced by the longitudinal reinforcement was 0.0202 mm/mm. The maximum strain experienced by the transverse reinforcement was 0.0049 mm/mm.

The progression of damage is presented in Figure A-22 (c). Significant flexural cracking was initiated near the column mid-height and bottom support regions at time 8 ms. At 10 ms, the damage level in these regions increased. Light damage on the column tension regions occurred at time 80 ms. There was no significant crushing of concrete.

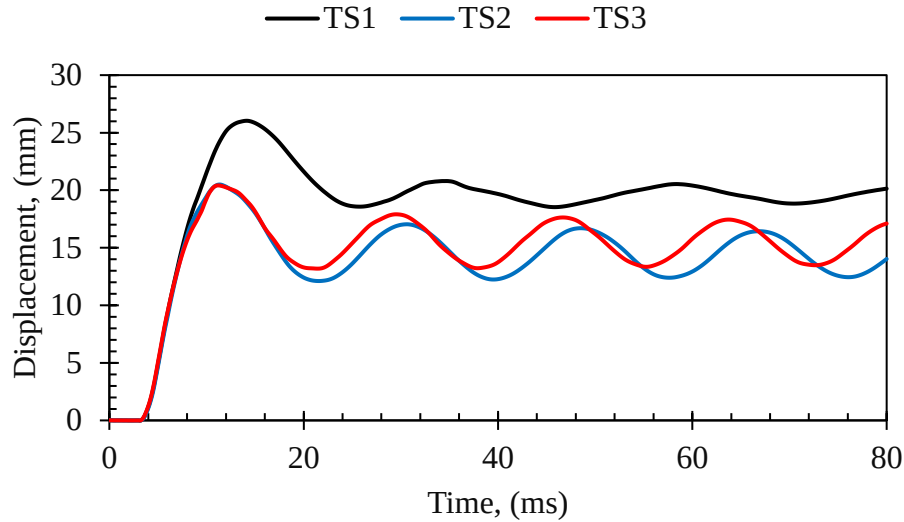


Figure 5-37: Displacement Time History for EC Columns under CS4 Loading

5.2.23 Response of Edge Columns EC-TS1, EC-TS2 and EC-TS3 to Compact Sedan Car Bomb at 7.5 m (CS5 Blast Load)

The columns in this series were subjected to axial compressive force of 1194 kN, applied as a linearly increasing load at a rate of 79.6 kN/ms. The load was kept at this level for 65 ms prior the application of blast loading and during the blast load application. The blast load was triggered by the detonation of 227 kg of TNT placed on the ground surface at a standoff distance of 7.5 m. The blast loading parameters are the same as described in Section 5.2.9.

5.2.23.1 Edge Column with Tie Spacing Equal to Column Section Dimension (EC-TS1 under CS5)

The maximum displacement of 18.7 mm was recorded at 14.2 ms at a height of 2300 mm above the ground surface. The maximum support rotation was 0.63° (calculated from the top). The displacement time history is shown in Figure 5-38. The maximum strain experienced by the longitudinal reinforcement was 0.0118 mm/mm. The maximum strain experienced by the transverse reinforcement was 0.012 mm/mm.

The progression of damage is presented in Figure A-23 (a). Many flexural cracks were observed near the column mid-span and bottom support regions at time 8 ms. Spalling of concrete cover was observed on the back face of the column at time 20 ms near mid-height. Significant damage

was detected on the column front face near the bottom support region at time 80 ms. The column remained intact with significant residual strength under this loading condition.

5.2.23.2 Edge Column with Tie Spacing Equal to Half the Column Sectional Dimension (EC-TS2 under CS5)

The column performed predominantly in flexure mode with significant residual capacity at the end of loading. The maximum displacement of 15.8 mm was recorded at 12.5 ms at a height of 2320 mm above the ground surface. The maximum support rotation was 0.54° (calculated from the top). The displacement time history is shown in Figure 5-38. The maximum strain experienced by the longitudinal reinforcement was 0.0077 mm/mm. The maximum strain experienced by the transverse reinforcement was 0.0093 mm/mm. The progression of damage is presented in Figure A-23 (b). Flexural cracking took place near column mid-height at time 8 ms. Damage accumulation was observed on the front face of the column near the base. Minor concrete damage was observed on the column back face due to flexure, without losing any core concrete. This is shown in the figure at time 80 ms.

5.2.23.3 Edge Column with Tie Spacing Equal to Quarter of the Column Sectional Dimension (EC-TS3 under CS5)

The column performed in the flexure mode with significant flexural cracking in the positive and negative moment regions at column mid-height and near the bottom support region, respectively. The maximum displacement of 14.4 mm was detected at 12.2 ms at a height of 2320 mm above the ground surface. The maximum support rotation was 0.49° (calculated from the top). The displacement time history is shown in Figure 5-38. The maximum strain experienced by the longitudinal reinforcement was 0.0076 mm/mm. The maximum strain experienced by the transverse reinforcement was 0.0022 mm/mm.

The progression of damage is presented in Figure A-23 (c). Flexural cracks formed in the column mid-span region, as well as the bottom support region at time 8 ms. No significant crushing of concrete was detected, leaving the column with significant residual strength at time 80 ms.

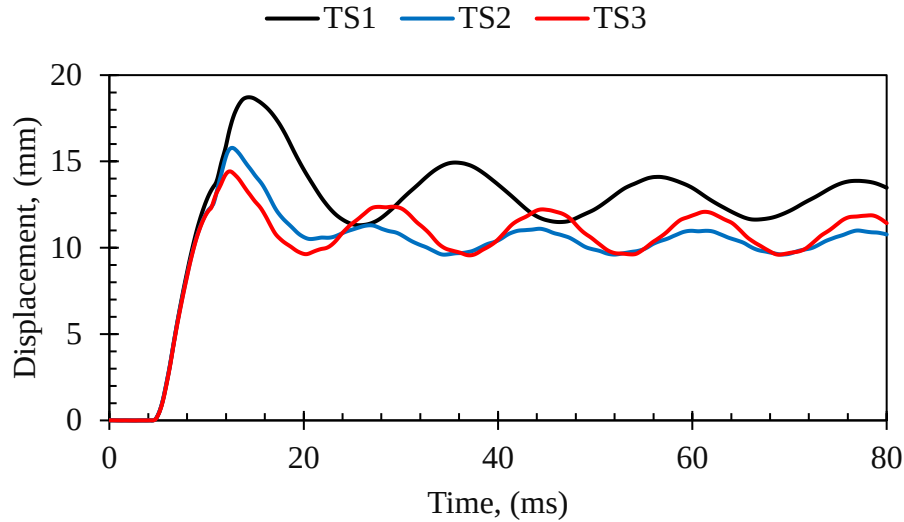


Figure 5-38: Displacement Time History for EC Columns under CS5 Loading

5.2.24 Response of Edge Columns EC-TS1, EC-TS2 and EC-TS3 to Sedan Car Bomb at 1.5 m (S1 Blast Load)

The columns in this series were subjected to the same axial compression as the previous series of columns prior to blast loading. Axial load was applied linearly with a rate of 79.6 kN/ms and held constant at a magnitude of 1194 kN for 65 ms prior to the blast load application. The axial load was kept constant during the blast load application. The blast loading was generated by 454 kg of TNT placed on the ground surface at a standoff distance of 1.5 m. The blast loading parameters are the same as described in Section 5.2.10.

5.2.24.1 Edge Column with Tie Spacing Equal to Column Section Dimension (EC-TS1 under S1)

The column suffered excessively, hence there was no measurements of maximum displacement and maximum strain. The displacement time history is shown in Figure 5-39. The progression of damage is presented in Figure A-24 (a). Heavy local damage was observed near the base of the column on the tension side at time 1 ms. Direct shearing was observed at the base of the column taking place at time 2 ms that led to the column failure. Subsequent damage observed within the column height is illustrated in Figure A-24 (a) at 4 ms and 10 ms.

5.2.24.2 Edge Column with Tie Spacing Equal to Half the Column Sectional Dimension (EC-TS2 under S1)

The column experienced severe damage once again, because of the close proximity of the charge. Hence, no maximum displacement or maximum strain measurements could be recorded. The displacement time history is shown in Figure 5-39. The progression of damage is presented in Figure A-24 (b). Severe damage at the column base occurred at time 1 ms. Direct shear failure occurred at the base at time 2 ms, eventually leading to the collapse of the column. The figure illustrates the extent of damage observed at 4 ms and 10 ms.

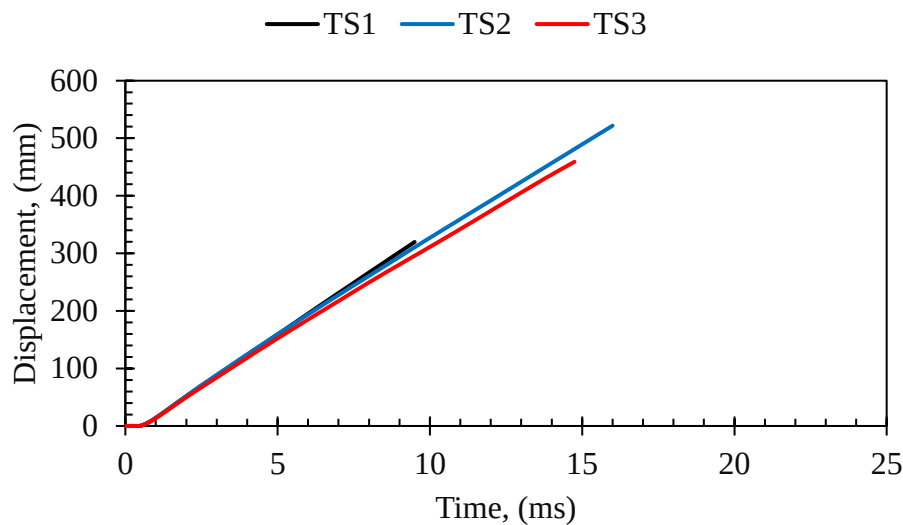


Figure 5-39: Displacement Time History for EC Columns under S1 Loading

5.2.24.3 Edge Column with Tie Spacing Equal to Quarter of the Column Sectional Dimension (EC-TS3 under S1)

Further reduction in tie spacing did not change the failure mode of the column, relative to the previous two columns, as the standoff distance was too close to the column and the failure mode was governed by direct shear. The damage was severe and hence, no maximum displacement or maximum strain could be measured for this column at failure. The displacement time history is shown in Figure 5-39. The progression of damage is presented in Figure A-24 (c). Severe damage at the column base occurred at time 1 ms. Direct shear failure initiated at the bottom support of the

column at time 2 ms, which eventually led to the collapse of the column. The figure illustrates the extent of damage observed at 4 ms and 10 ms.

5.2.25 Response of Edge Columns EC-TS1, EC-TS2 and EC-TS3 to Sedan Car Bomb at 3.0 m (S2 Blast Load)

The columns were first subjected to axial load, which was applied linearly at a rate of 79.6 kN/ms. The load was then kept constant at a magnitude of 1194 kN for 65 ms prior and during the time of blast load application. The blast loading was triggered by a 454 kg of TNT, placed on the ground surface at a standoff distance of 3.0 m. The blast loading parameters are the same as described in Section 5.2.11.

5.2.25.1 Edge Column with Tie Spacing Equal to Column Section Dimension (EC-TS1 under S2)

The column suffered severe damage caused mostly by direct shear, and therefore the maximum displacement and maximum strain measurements at failure could not be obtained. The displacement time history is shown in Figure 5-40. The progression of damage is presented in Figure A-25 (a). Severe damage near the column base was detected at time 2 ms. Flexural cracks and diagonal shear crack were also observed at time 5 ms. The column resistance was mostly governed by direct shear at the bottom support, which initiated at time 3 ms and eventually resulted in the total collapse to the column.

5.2.25.2 Edge Column with Tie Spacing Equal to Half the Column Sectional Dimension (EC-TS2 under S2)

This column had half the tie spacing as the previous column, but because of the proximity of the explosive to the column, direct shear failure was observed. Hence the damage was excessive and no maximum displacement or maximum strain could be measured at failure. The displacement time history is shown in Figure 5-40. The progression of damage is presented in Figure A-25 (b). Heavy damage near the column base occurred at time 5 ms. Direct shearing of the column at its bottom support region was observed at 7 ms. This was accompanied by the formation of flexural and inclined shear cracks. Column failed due to direct shear at the base as shown in Figure A-25 (b) at time 12 ms.

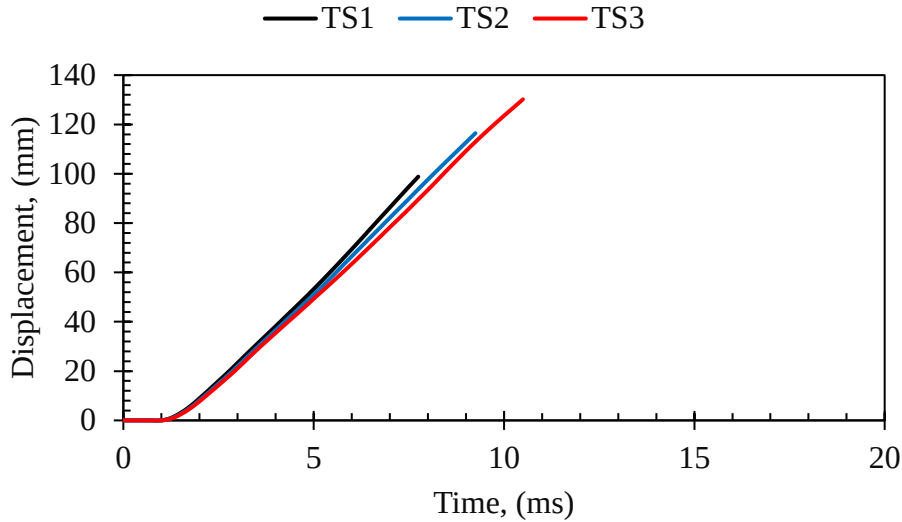


Figure 5-40: Displacement Time History for EC Columns under S2 Loading

5.2.25.3 Edge Column with Tie Spacing Equal to Quarter of the Column Sectional Dimension (EC-TS3 under S2)

This column had closely spaced ties, which reduced the damage along the height, but because of the close standoff distance the dominant failure mode remained as direct shear, causing severe damage. For this reason, maximum displacement and maximum strain measurements could not be recorded at failure. The displacement time history is shown in Figure 5-40. The progression of damage is presented in Figure A-25 (c). The column was highly stressed near the bottom support at 2 ms while also developing some flexural cracks within the bottom 1/3rd. Extensive flexural cracking occurred at time 5 ms. The column response was mostly dominated by direct shear at the bottom support. This led to the initiation of failure at 7 ms, followed by significant deterioration of concrete within the bottom half of the column at time 12 ms.

5.2.26 Response of Edge Columns EC-TS1, EC-TS2 and EC-TS3 to Sedan Car Bomb at 4.5 m (S3 Blast Load)

The columns were first subjected to axial load, which was applied linearly at a rate of 79.6 kN/ms. The load was then kept constant at a magnitude of 1194 kN for 65 ms prior and during the application of blast loading. The blast loading was triggered by a 454 kg of TNT, placed on the

ground surface at a standoff distance of 4.5 m. The blast loading parameters are the same as described in Section 5.2.12.

5.2.26.1 Edge Column with Tie Spacing Equal to Column Section Dimension (EC-TS1 under S3)

The increase in the standoff distance to 4.5 meters changed the mode of behaviour in this series of columns from direct shear to a combination of diagonal tension and flexure. The column continued to suffer severe damage and hence no maximum displacement or maximum strain measurements were possible at failure. The displacement time history recorded for the column is shown in Figure 5-41. The progression of damage is presented in Figure A-26 (a). Clear diagonal shear crack initiation was observed at time 3.5 ms. Progression of diagonal shear crack near the bottom support at time 6 ms led to the column failure. Flexural damage was also observed, resulting in the crushing of column core concrete at failure. The column damage at different stages of loading is illustrated in Figure A-26.

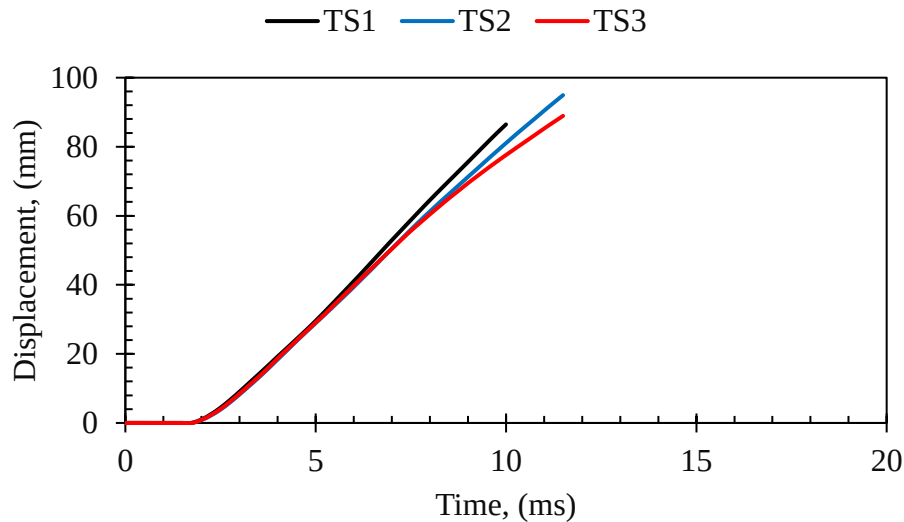


Figure 5-41: Displacement Time History for EC Columns under S3 Loading

5.2.26.2 Edge Column with Tie Spacing Equal to Half the Column Sectional Dimension (EC-TS2 under S3)

This column had half the tie spacing as the previous column, which resulted in better diagonal shear crack control. The increase in the standoff distance to 4.5 metres eliminated direct shear

failure. Nevertheless, the column suffered from severe damage with no readings for maximum displacement and maximum strain measurements. The displacement time history is shown in Figure 5-41. The progression of damage is presented in Figure A-26 (b). The formation of diagonal shear cracks can be observed at times 3.5 ms, 6 ms and 10 ms. The combination of diagonal tension and diagonal compression resulted in the failure of the column at time 14 ms.

5.2.26.3 Edge Column with Tie Spacing Equal to Quarter of the Column Sectional Dimension (EC-TS3 under S3)

This column had sufficiently high transverse reinforcement ratio that controlled diagonal shear cracking, resulting in flexural failure. However, the column experienced severe damage. Therefore, no readings could be taken for maximum displacement or maximum strains at failure. The displacement time history is shown in Figure 5-41. The progression of damage is presented in Figure A-26 (c). Extensive flexural cracking initiated within the column mid-height region at times 3.5 ms and 6 ms. This resulted in extensive concrete crushing on the front face of the column. The column experienced heavy flexural damage with significant loss of cover and core concrete on the back side of the column.

5.2.27 Response of Edge Columns EC-TS1, EC-TS2 and EC-TS3 to Sedan Car Bomb at 6.0 m (S4 Blast Load)

The columns were subjected to the same axial compression as the previous series of columns prior to blast loading. Axial load was applied linearly with a rate of 79.6 kN/ms and held constant at a magnitude of 1194 kN for 65 ms prior to the blast load application. The axial load was also kept constant during the blast load application. The blast loading was generated by 454 kg of TNT placed on the ground surface at a standoff distance of 6.0 m. The blast loading parameters are the same as described in Section 5.2.13.

5.2.27.1 Edge Column with Tie Spacing Equal to Column Section Dimension (EC-TS1 under S4)

The column failed due to a combined diagonal tension and flexural behaviours at time 30 ms as shown in Figure A-27 (a). The failure was sudden and severe. Hence no maximum displacement and maximum strain measurements were recorded at failure. The displacement time history is

shown in Figure 5-42. The progression of damage is presented in Figure A-27 (a). Many flexural cracks were initiated near the column mid-height at time 5 ms. The increase in tensile and compressive stresses resulted in the loss of cover and core concrete near the bottom support and near the mid-height regions.

5.2.27.2 Edge Column with Tie Spacing Equal to Half the Column Sectional Dimension (EC-TS2 under S4)

The reduced tie spacing in this column shifted the mode of behaviour completely to flexure. The eventual failure was brittle and did not permit the recording of maximum displacement and maximum strain measurements at failure. The displacement time history is shown in Figure 5-42. The progression of damage is presented in Figure A-27 (b). Flexural cracking was observed in the initial stages of loading as shown at time 5 ms. Extensive flexural tension cracking was observed near the base and mid-height regions, followed by the onset of concrete crushing at about 15 ms. Deterioration of concrete, near the bottom support region, leading to the formation of a plastic hinge was observed at about 20 ms. In conjunction with the loss of core concrete near mid-height, the crushing of concrete resulted in a sudden failure of the column.

5.2.27.3 Edge Column with Tie Spacing Equal to Quarter of the Column Sectional Dimension (EC-TS3 under S4)

Further reduction in tie spacing did not change the failure mode of the column, relative to the previous column. The failure was brittle and did not permit the recording of maximum displacement and maximum strain measurements at failure. The displacement time history is shown in Figure 5-42. The progression of damage is presented in Figure A-27 (c). The column experienced only flexural cracking in the initial stages of loading as shown at time 5 ms. Excessive flexural tension cracking was observed near the base and mid-height regions, followed by the onset of concrete crushing at about 15 ms. Deterioration of concrete, especially near the bottom support region, leading to the formation of a plastic hinge was observed at about 20 ms. This resulted in a sudden failure of the column.

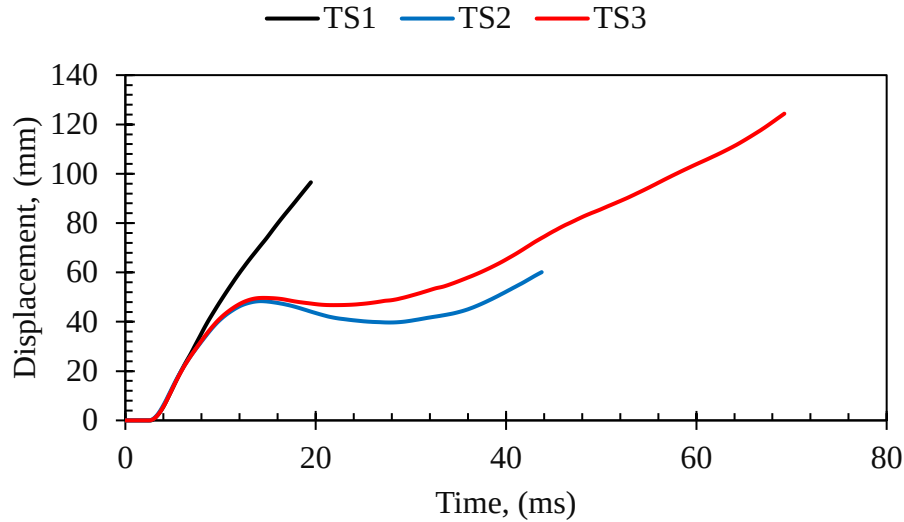


Figure 5-42: Displacement Time History for EC Columns under S4 Loading

5.2.28 Response of Edge Columns EC-TS1, EC-TS2 and EC-TS3 to Sedan Car Bomb at 7.5 m (S5 Blast Load)

The columns in this series were analysed under combined axial and blast loading as explained before for the earlier columns. The axial load was applied gradually with a rate of 79.6 kN/ms, and held constant at a magnitude of 1194 kN for 65 ms prior to the application of the blast load and kept constant during the blast load stage. The blast loading was generated by 454 kg of TNT, placed on the ground surface at a standoff distance of 7.5 m. The blast loading parameters are the same as described in Section 5.2.14.

5.2.28.1 Edge Column with Tie Spacing Equal to Column Section Dimension (EC-TS1 under S5)

The increase of standoff distance to 7.5 m caused a combination of failure in the form of diagonal tension and flexure. The column suffered severe damage, and therefore the maximum displacement and maximum strain measurements could not be obtained for this column. The displacement time history recorded for the column is shown in Figure 5-43. The progression of damage is presented in Figure A-28 (a). Clear diagonal shear crack initiation was observed at time 10 ms. Progression of diagonal shear crack near the top and bottom support regions at time 20 ms

led to column failure. Flexural damage was also observed, resulting in the crushing of column core concrete at failure. The column damage at different stages of loading is illustrated in Figure A-28.

5.2.28.2 Edge Column with Tie Spacing Equal to Half the Column Sectional Dimension (EC-TS2 under S5)

This column had half the tie spacing as the previous column, and showed flexural response. The eventual failure was brittle and did not permit the recording of maximum displacement and maximum strain measurements at failure. The displacement time history is shown in Figure 5-43. The progression of damage is presented in Figure A-28 (b). Flexural cracking was observed during the initial stages of loading as shown at time 8 ms. Extensive flexural tension cracking was observed near the base and mid-height regions. A plastic hinge formed near the bottom support region due to the weakening of concrete, as illustrated at time 20 ms. This resulted in a sudden failure of the column at time 80 ms.

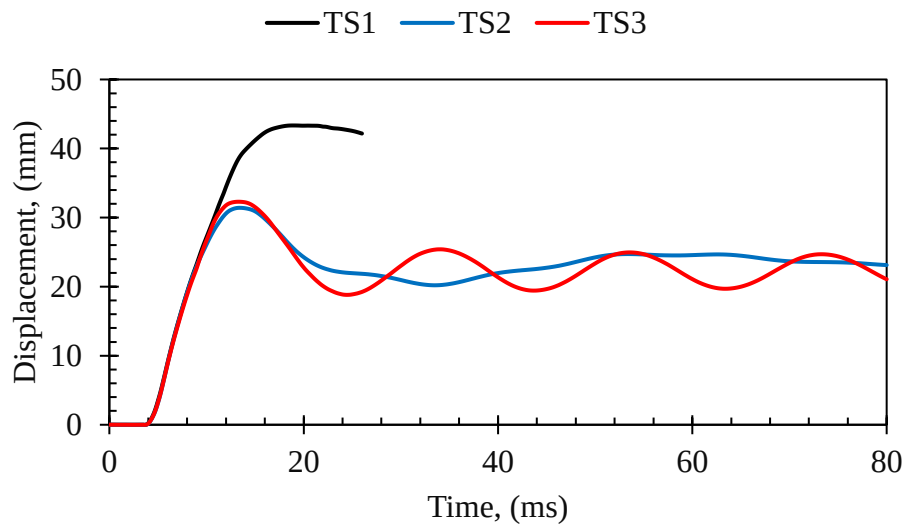


Figure 5-43: Displacement Time History for EC Columns under S5 Loading

5.2.28.3 Edge Column with Tie Spacing Equal to Quarter of the Column Sectional Dimension (EC-TS3 under S5)

The column performed predominantly in flexure mode with significant residual capacity at the end of loading. The maximum displacement of 32.3 mm was recorded at 13.2 ms at a height of 2370 mm above the ground surface. The maximum support rotation was 1.14° (calculated from the top).

The displacement time history is shown in Figure 5-43. The maximum strain experienced by the longitudinal reinforcement was 0.0272 mm/mm. The maximum strain experienced by the transverse reinforcement was 0.0103 mm/mm. The progression of damage is presented in Figure A-28 (c). Flexural cracking occurred near the column mid-height at time 8 ms. Damage accumulation was observed on the front face of the column near the base. Minor concrete damage was observed on the column back face due to flexure, without losing any core concrete. This is shown in the figure at time 80 ms.

5.3 Discussion of Numerical Results

The effects of test variables are presented in this section. The failure modes are discussed first as affected by the test variables. The effects of individual test variables, i.e., i) standoff distance, ii) charge weight, iii) column ties spacing, and iv) column location (corner versus edge column) are then discussed separately.

5.3.1 Failure Modes

5.3.1.1 Backpack Bomb

Both corner and edge columns under the backpack loading suffered localized shear damage near the column base irrespective of the transverse reinforcement spacing when the standoff distance was 0.5 m or 0.75 m. The failure was due to either concrete scabbing at the back face or complete breaching. The peak reflected pressures at the base of the column for 0.5 m and 0.75 m standoff distances were 220.0 MPa and 91.1 MPa, respectively. These dynamic loadings corresponded to positive time durations of 0.75 ms and 0.65 ms, respectively. Because the standoff distance was very small, very high pressures were generated over very short durations. The displacement associated with response of these columns was very small because of local damage, which prevented the formation of global column response. Concrete scabbing at the back face occurred because the tensile stress generated from the loading exceeded the low tensile strength of concrete. This led to partial or complete column breaching in some cases. All the corner columns experienced complete breaching when the standoff distance was 0.5 m or 0.75 m. The edge columns with a transverse reinforcement spacing of 450 mm or 225 mm also failed due to breaching. Only the edge columns with 112.5 mm spacing survived the loading condition with concrete scabbing at the back face.

The mode of failure for corner and edge columns was changed to global failures when the standoff distance was increased to 1.5 m. These columns were able to attain their diagonal tension (caused by shear) or flexural capacities. The columns with a transverse reinforcement spacing of 450 mm or 225 mm failed due to diagonal tension. Decreasing the transverse reinforcement spacing to 112.5 mm controlled diagonal tension, and promoted flexural failure in both corner and edge columns. When the standoff distance increased to 3.0 m, the peak reflected pressure generated at

the base of the column was 6.40 MPa with positive time duration of 2.75 ms. All the corner and edge columns survived this blast loading condition without shear or flexural failures.

5.3.1.2 Compact Sedan Car Bomb

When the compact sedan car bomb was detonated at a standoff distance of 1.5 m, both corner and edge columns showed extensive local damage on the back side of the column due to the close-in blast effects. The blast loading exceeded the shear capacity of the columns, resulting in the direct shear failure mode at the bottom support. This standoff distance corresponded to a reflected pressure of 102.2 MPa with positive time duration of 1.4 ms. The arrival of the shock wave was observed to be at 0.3 ms. The failure occurred within the first three milliseconds of the explosion. This caused a catastrophic direct shear failure of the columns at the base.

At a standoff distance of 3.0 m, local damage due to the close-in effect was reduced. The columns with 450 mm and 225 mm transverse reinforcement spacing suffered diagonal tension failures caused by shear near the bottom support region. The columns with 112.5 mm transverse reinforcement spacing improved both diagonal tension and diagonal compression capacities, changing the failure mode to primarily flexure.

With 4.5 m, 6.0 m and 7.5 m standoff distance, all corner and edge columns were performed in diagonal tension and flexure modes. No localized shear damage was observed. All these columns had an accumulated damage on the compression face near the base. As the standoff distance increased the shape of the shock front became planar. Reflected pressure along the height of the columns generated was almost constant over a one-meter strip measured from column base. This is shown in Figures 5-14 (a), 5-16 (a), 5-18 (a).

At 4.5 m standoff distance, corner column with 450 mm transverse reinforcement spacing failed due to the formation of diagonal shear cracks, while the edge column failed due to the combination of diagonal shear and flexural cracks. With 225 mm transverse reinforcement spacing, corner column failed due to flexure and edge column is deemed to have no more residual capacity. With 112.5 mm spacing, the edge column attained its flexural capacity and showed a very good dynamic response, while the corner column behaved well and attained its flexural capacity. The resistance of the corner column near the base had already reduced in the inbound stage, and suffered failure

during rebound. At 6.0 m and 7.5 m standoff distances, only the corner column with 450 mm spacing failed during rebound at the base. The rest of the columns showed very good response without failure, though they all developed diagonal tension and flexure cracks.

5.3.1.3 Sedan Car Bomb

At a close standoff distance of 1.5 m the mode of behaviour of columns was the same as those subjected to compact sedan car bomb, developing direct shear failure at the bottom support of the columns. The same failure mode was also observed when the standoff distance was increased to 3.0 m, unlike those subjected to the compact car bomb, but the columns did develop significant diagonal tension and flexural cracking prior to failing due to direct shear. This standoff distance corresponded to a peak reflected pressure of 55.7 MPa and reflected impulse of 24.7 MPa-ms, which was considerably high, and caused failure within the first seven milliseconds of detonation. At a standoff distance of 4.5 m, the governing mode of failure changed from direct shear to diagonal tension and flexure, depending on tie spacing. Both the corner and edge columns with 450 mm and 225 mm transverse reinforcement spacing failed due to lack of diagonal tension capacity, exhibiting wide diagonal shear cracks. Columns with 112.5 mm spacing failed in flexure. At a standoff distance of 6.0 m, both corner and edge columns failed in diagonal tension when the tie spacing was 450 mm, having developed noticeable diagonal shear cracks. When the tie spacing decreased to 225 mm and 112.5 mm, the column behaviour was dominated by flexure. They experienced heavy flexural damage on the tension side, followed by concrete deterioration on the compression side, but failed due to the damage accumulation within the bottom half of the column as the applied pressure was more uniform within this region. At a standoff distance of 7.5 m, the columns with 450 mm failed due to lack of diagonal shear capacity associated with the formation of diagonal shear cracks (top and bottom). The edge column with 225 mm transverse reinforcement spacing failed in rebound near the column base. The other three columns performed predominantly in flexure mode.

5.3.2 Effect of Standoff Distance

The influence of standoff distance on column performance is discussed in this sub-section by examining the behaviour of columns with the same transverse reinforcement ratio, exposed to the same TNT charge weight. First, the effect of standoff distance is examined under a compact car

bomb (227 kg of TNT), which generated reflected pressures of 102.3 MPa, 36.1 MPa, 16.3 MPa, 8.1 MPa and 4.4 MPa at the base of the column when the standoff distances were 1.5 m, 3.0 m, 4.5 m, 6.0 m and 7.5 m, respectively. Figure 5-44 shows the damage level as affected by standoff distance on a corner column. At 1.5 m distance the explosion caused direct shear failure with column breaching. When the distance is increased to 3.0 m the failure mode shifted to diagonal tension caused by shear, while the column remained intact in flexure. Further increase to 4.5 m triggered flexural response and flexural failure in the lower half of the column where the pressure was the highest and applied almost uniformly. The tie spacing of $h/2$ (one half of the cross-sectional dimension) was able to provide sufficient transverse reinforcement to control diagonal cracking. As the standoff distance increased to 6.0 m and 7.5 m, the damage was reduced and the column maintained its integrity. Another comparison is presented in Figure 5-45, this time for a corner column reinforced with closely spaced ties ($s = h/4$), exposed to a backpack bomb. The standoff distances considered are 0.75 m and 1.5 m. The damage level (indicated by red) and loss of concrete decreased significantly with the increase in standoff distance by only 0.75 m. The column with 0.75 m standoff distance was breached, while the column with 1.5 m standoff distance didn't experience any failure. This can be explained by the decrease in reflected pressure and impulse by one third when the backpack bomb was placed at 1.5 m. At 1.5 m, the mode of behaviour was flexure, with light damage.

The displacement response time histories for a corner and an edge column, exposed to backpack, compact sedan car and sedan car bombs placed at different standoff distances are shown in Figures 5-46 through 5-48. The columns were reinforced with different percentages of transverse reinforcement, having different tie spacing. Under closer standoff distances the columns suffered complete failure, before responding to shock waves while retaining some level of residual strength. Columns with closely spaced ties maintained their shear capacities if governed by diagonal tension, and responded in the flexural mode.

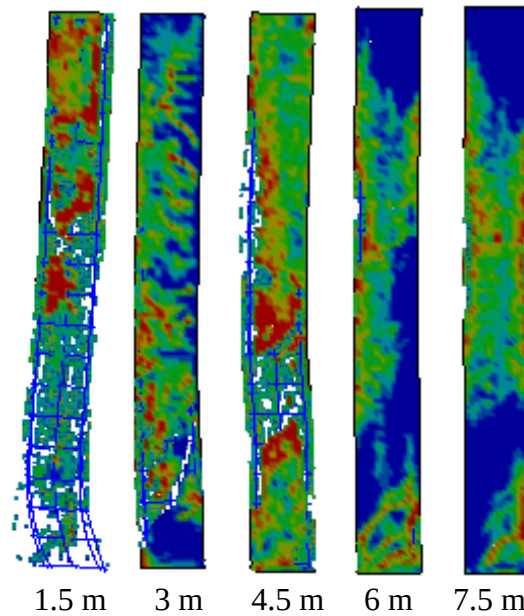


Figure 5-44: Damage to Corner Column with Tie Spacing Equal to Half the Column Section Dimension Under Compact Sedan Bomb (CS1 to CS5)

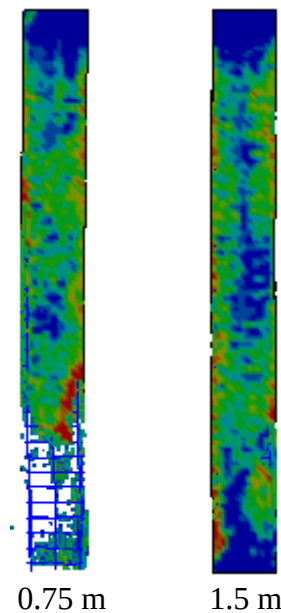


Figure 5-45: Damage to Corner Column with Ties Spacing Equal to Quarter of the Column Section Dimension Under Back Pack Bomb (BP2 and BP3)

Another important observation that can be made from Figures 5-46 through 5-48 is the variation in the initial slope of the response curves. The initial slope of the displacement time history decreases with the increase in standoff distance. The slope represents the initial velocity of the

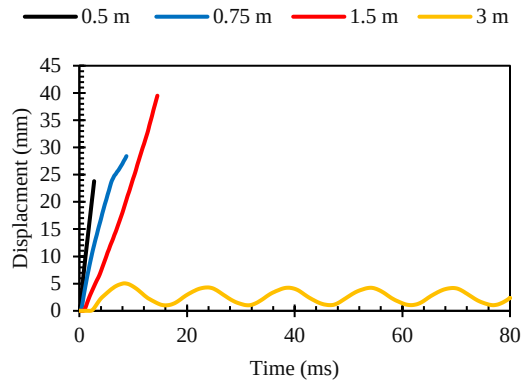
column and the speed of the potential flying debris resulting from the material/member failure. This information is significant as the speed of flying debris is directly linked with minimize potential injuries caused by damaged columns. All columns showed consistent response with the change in standoff distance in terms of failure modes and peak displacements.

5.3.3 Effect of Charge Mass

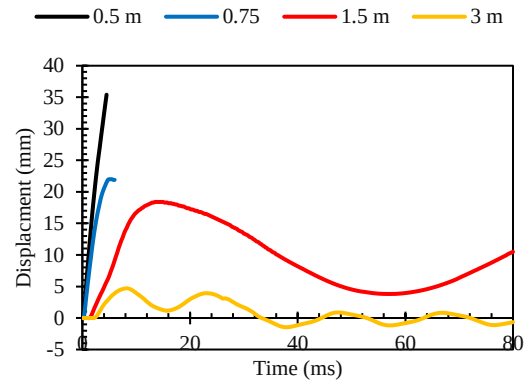
Two scenarios were considered to examine the effect of explosive charge mass on column response; i) high charge masses at the same standoff distance ii) small and high charge masses with the same scaled standoff distance.

5.3.3.1 Compact Sedan versus Sedan Car Bombs at Equal Standoff Distance

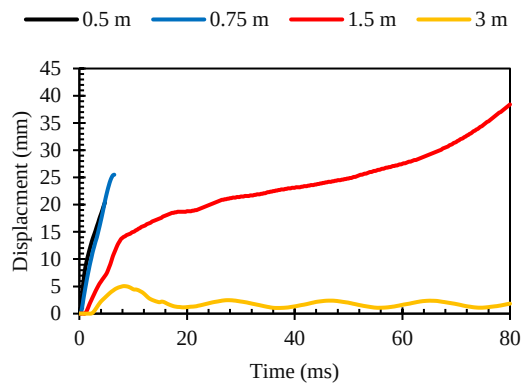
In this sub section, columns having the same transverse reinforcement ratio, exposed to different TNT charge mass (compact sedan and sedan cars) placed at the same standoff distance are compared to investigate the effect of charge mass on column response. Figure 5-49 shows column damage for an edge column (with the three tie spacing) subjected to blast loading generated from compact sedan and sedan car bombs located at 6.0 m. The performance of columns indicates that the bomb with higher charge weight generated more damage as expected. The sedan car bomb (with a charge mass of 454 kg) resulted in the complete failure of the column with a tie spacing equal to the section dimension both in diagonal tension and flexure. The same column when subjected to a compact sedan car bomb (with 227 kg charge weight) survived the blast, though developed high tensile and compressive stresses. When the columns with transverse reinforcement spacing of 225 mm and 112.5 mm are compared under the two levels of bomb, it can be observed that the columns suffered shear and flexure failures in rebound, with first peak displacements of approximately 48 mm and 20 mm when subjected to sedan car bombs (with a charge mass of 454 kg). The same columns survived the blast and developed approximately 20 mm peak displacement under compact sedan car bombs (when the charge mass was 227 kg). When the other companion columns are compared under two different bombs with two different charge weights, it is consistently observed that the higher charge weight results in more damage.



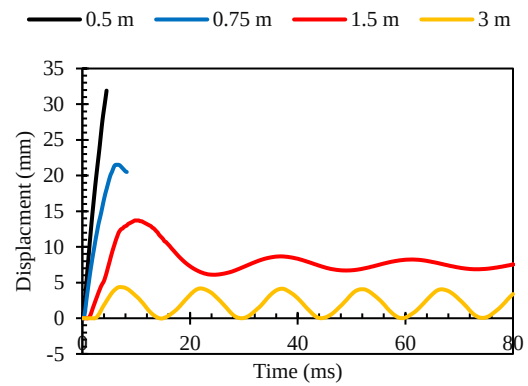
(a) CC-TS1-(BP1 to BP4)



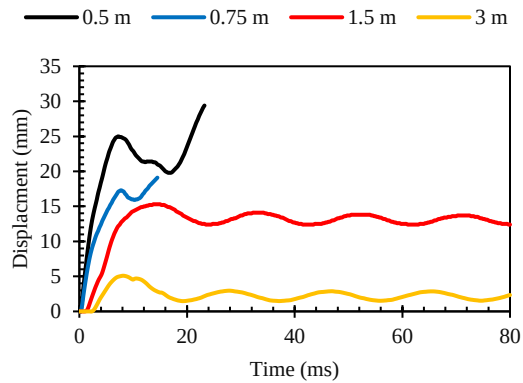
(b) EC-TS1-(BP1 to BP4)



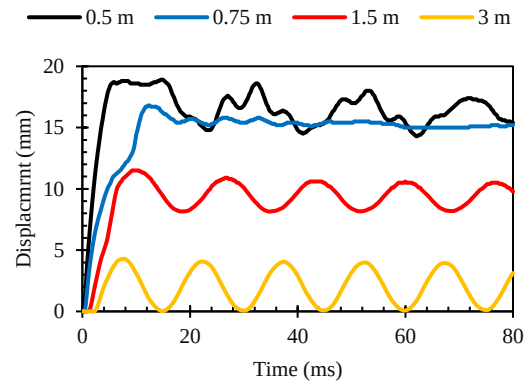
(c) CC-TS2-(BP1 to BP4)



(d) EC-TS2-(BP1 to BP4)

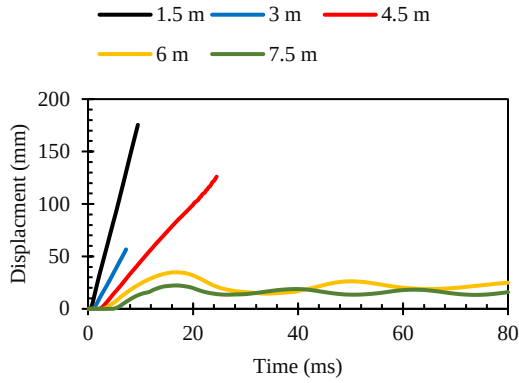


(e) CC-TS3-(BP1 to BP4)

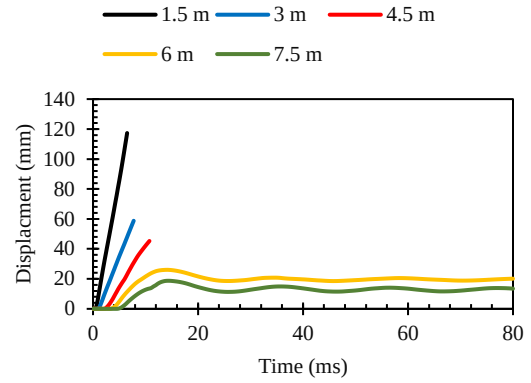


(f) EC-TS3-(BP1 to BP4)

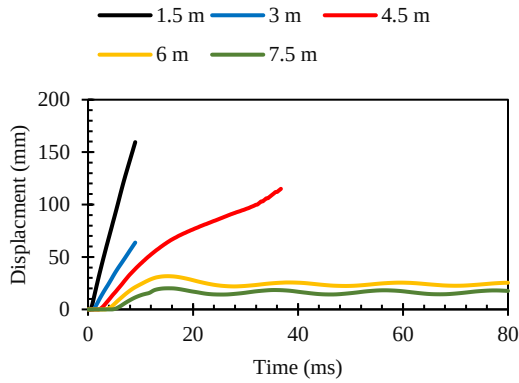
Figure 5-46: Effect of Standoff Distance on Displacement Response of Corner and Edge Columns with Different Transverse Reinforcement Spacing under a Backpack Bomb



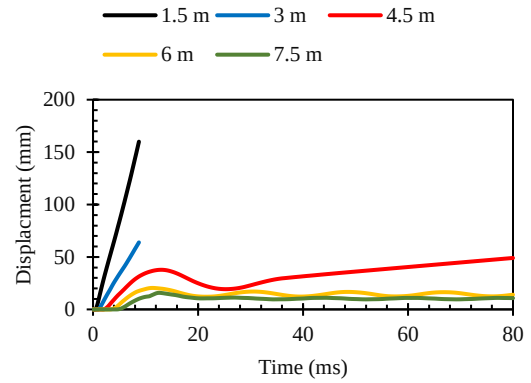
(a) CC-TS1-(CS1 to CS5)



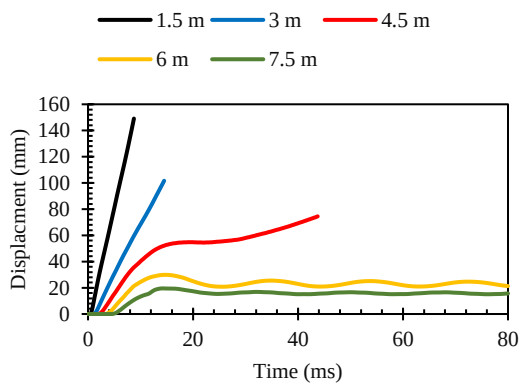
(b) EC-TS1-(CS1 to CS5)



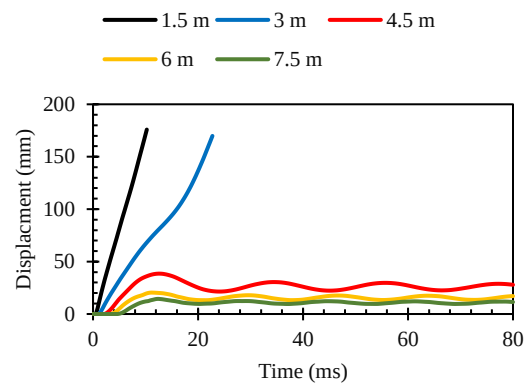
(c) CC-TS2-(CS1 to CS5)



(d) EC-TS2-(CS1 to CS5)

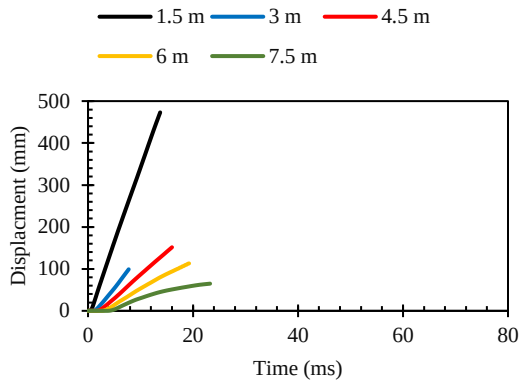


(e) CC-TS3-(CS1 to CS5)

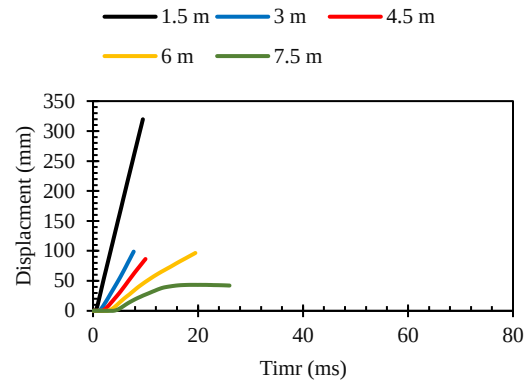


(f) EC-TS3-(CS1 to CS5)

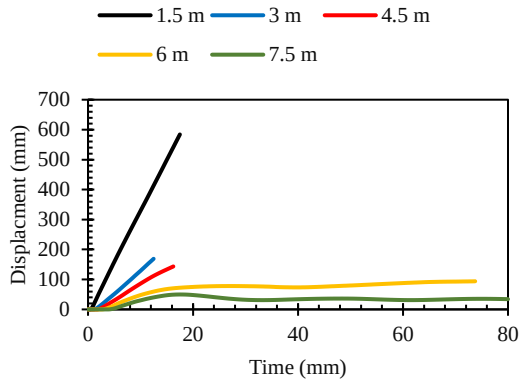
Figure 5-47: Effect of Standoff Distance on Displacement Response of Corner and Edge Columns with Different Transverse Reinforcement Spacing under a Compact Sedan Bomb



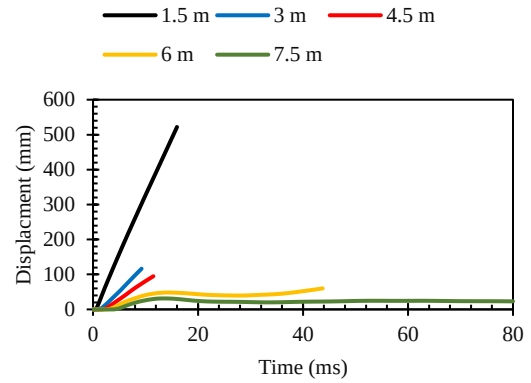
(c) CC-TS1-(S1 to S5)



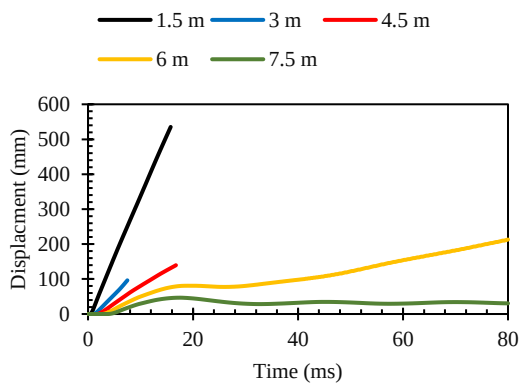
(d) EC-TS1-(S1 to B5)



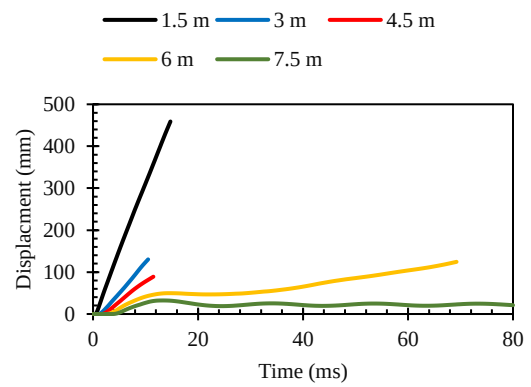
(c) CC-TS2-(S1 to S5)



(d) EC-TS2-(S1 to S5)



(c) CC-TS3-(S1 to S5)



(d) EC-TS3-(S1 to S5)

Figure 5-48: Effect of Standoff Distance on Displacement Response of Corner and Edge Columns with Different Transverse Reinforcement Spacing under a Sedan Bomb

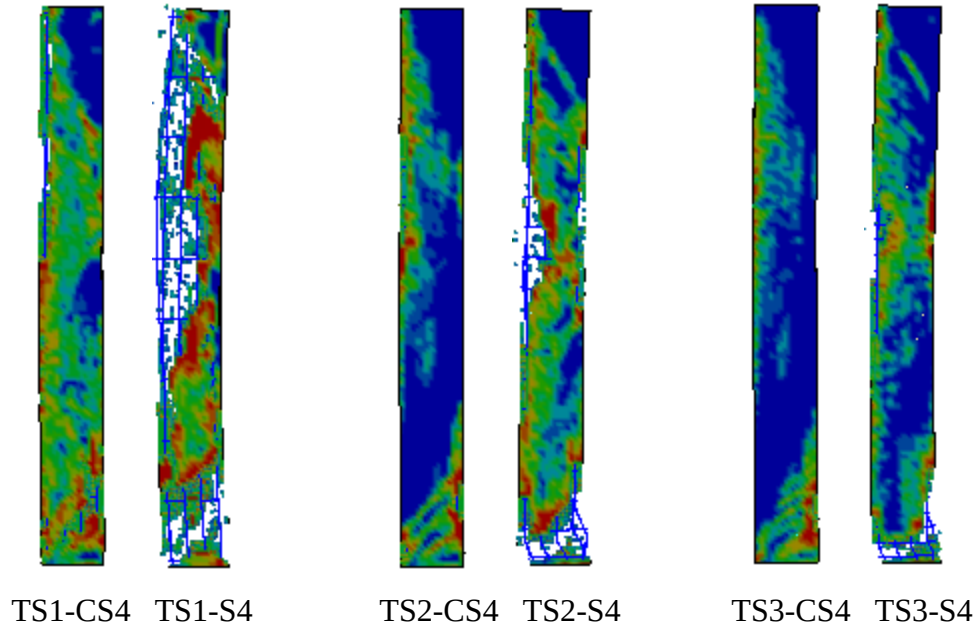


Figure 5-49: Damage to Edge Column Pairs with the Same Tie Spacing Caused by Compact Sedan Car Bomb (CS4) and Sedan Car Bomb (S4) Located at 6.0 m

5.3.3.2 Same Scaled Standoff Distance (backpack versus compact sedan)

The effect of charge weight on columns having the same scaled distance is investigated using backpack bomb and compact sedan car bomb. The scaled standoff distance for the backpack bomb at 0.5 m is approximately $0.178 \text{ m/kg}^{1/3}$. Another simulation with the compact sedan vehicle as the explosive was carried out with a standoff distance of 1.086 m to produce the same scaled standoff distance as that for the backpack, i.e., $0.178 \text{ m/kg}^{1/3}$. Pressure/impulse time history for the new simulation is presented in Figure 5-50. The peak reflected pressure of 221 MPa was obtained near the column base with an impulse of 81.8 MPa-ms and positive time duration of 1.4 ms. The reflected pressure obtained is the same as that for the backpack bomb having the same scaled distance. The simulation was carried out on a corner column with transverse reinforcement spacing of 225 mm. The progression of damage for this column is presented in Figure 5-51(a), while also compared with that for a companion column exposed to a backpack bomb with lower charge weight but the same scaled distance shown in Fig. 5-51(b). The column under compact sedan car bomb failed due to direct shear at the bottom support within 2 ms from the onset of detonation.

Significant deterioration of column concrete followed near column mid-height at 5 ms. The failure mode with the backpack was governed by full breaching near the base.

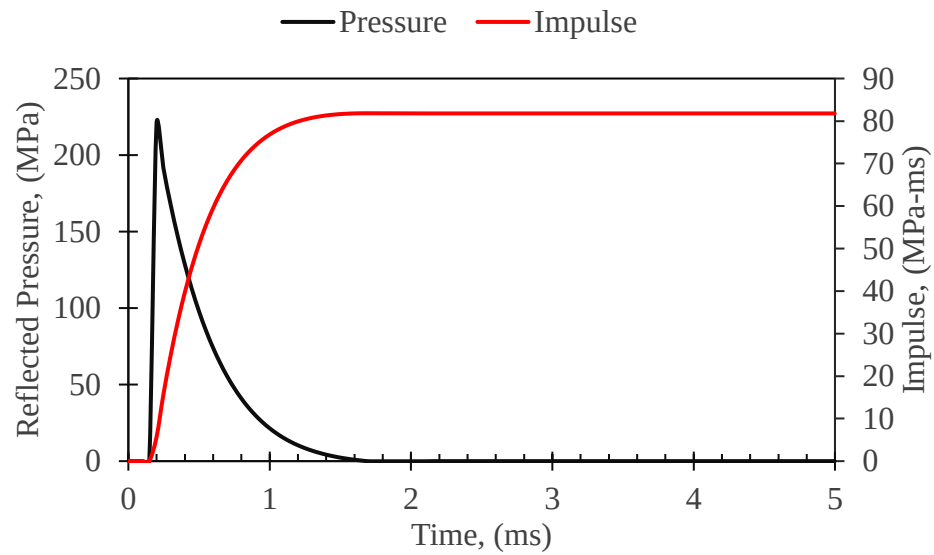


Figure 5-50 : Pressure/Impulse Time Histories near the Base of Column

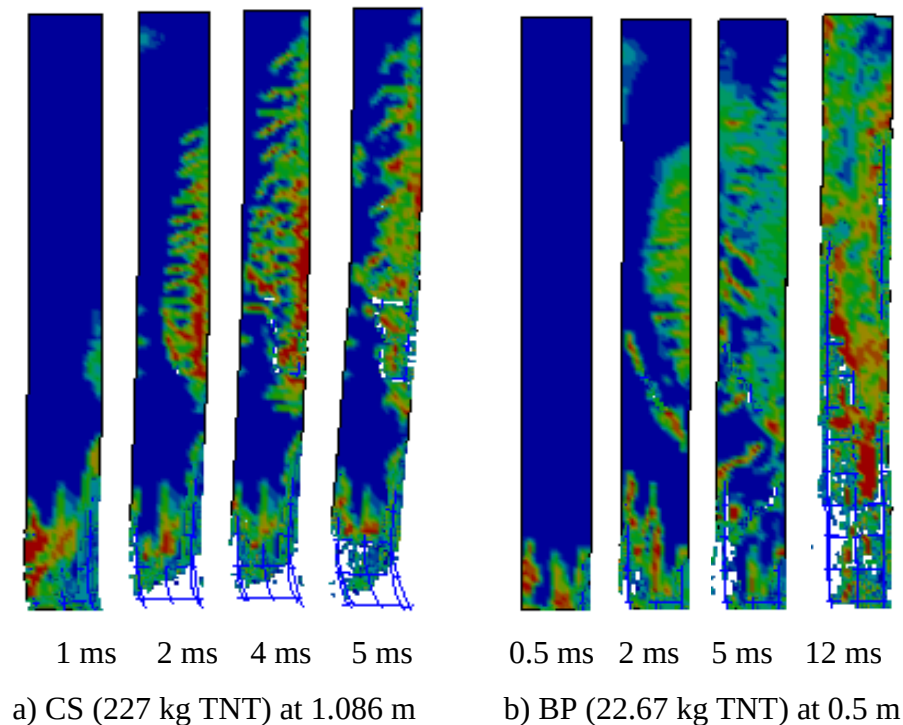


Figure 5-51: Progression of Damage in Column CC-TS2 under Different Charge Weights but the Same Scaled Distance of $Z = 0.178 \text{ m/kg}^{1/3}$

The reflected pressures generated by the two bombs and the scaled standoff distances were approximately the same with magnitudes of 220 MPa and $0.178 \text{ m/kg}^{1/3}$, respectively. However, the impulse produced by the higher and smaller charge masses were 81.6 MPa-ms and 39 MPa-ms, respectively. The impulse generated with higher mass explosive (compact sedan car bomb) was almost doubled the impulse generated by the backpack bomb. This indicates that column response to blast loading is heavily dependent on the impulse. Higher values of impulse produce more global damage in column.

Figure 5-52 illustrates the displacement response for both threat scenarios with the same scaled distance. The displacement is very low under the backpack load when compared with that under the compact sedan due to the differences in impulse.

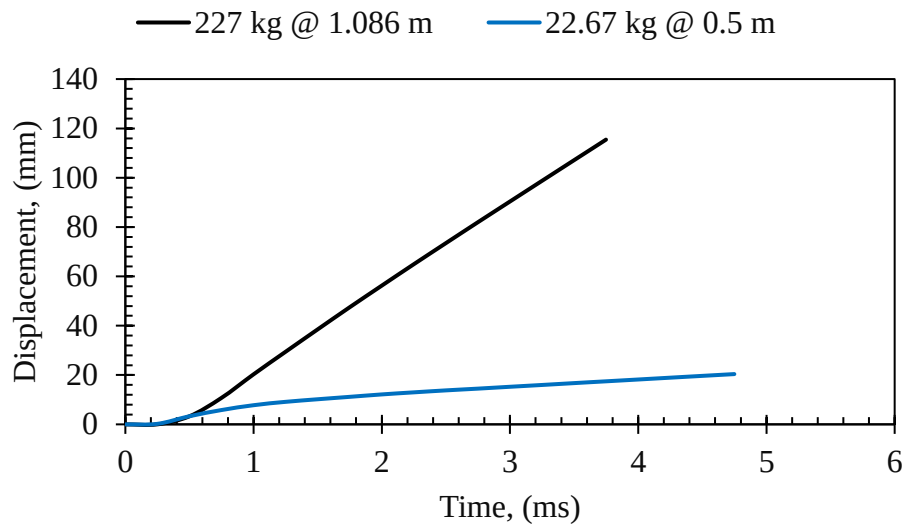


Figure 5-52: Displacement Time History Comparison for Backpack and Compact Sedan Bombs

5.3.4 Effect of Transverse Reinforcement Spacing

In this sub section, columns with different transverse reinforcement spacing that are exposed to the same TNT charge mass, detonated at the same standoff distance are compared to investigate the effect of tie spacing and percentage of transverse reinforcement on columns response. The columns investigated had ties of the same size but different tie spacing. This translated into different amounts of shear reinforcement, as well as concrete confinement. Confinement of the

core concrete enhanced concrete compressive strength, while shear reinforcement helped control diagonal cracking associated with diagonal tension caused by shear, promoting flexural response. Figure 5-53 shows the effects of transverse reinforcement on column response.

Figure 5-53 (a) shows edge columns with different transverse reinforcement spacing subjected to backpack bomb located at 0.5 m. The columns with tie spacing of 450 mm and 225 mm were completely breached due to local effects. The column with a tie spacing of 112.5 mm suffered only from concrete scabbing on the back face. Reducing the transverse reinforcement spacing and improving the confinement of core concrete significantly decreased the localized damage near the bottom support. The same effect was observed when the standoff distance was increased under the backpack charge weight.

Figure 5-53 (b) shows corner columns with different transverse reinforcement spacing under a sedan car bomb located at 4.5 m. The columns with transverse reinforcement spacing of 450 mm and 225 mm failed with the formation of large diagonal shear cracks. Decreasing the tie spacing to 112.5 mm changed the mode of failure from diagonal shear to flexure.

Figure 5-53 (c) shows the corner column with different tie spacing under a compact sedan car bomb located at 7.5 m. The column with 450 mm spacing experienced significant flexural cracking and loss of concrete cover. Furthermore, 25 % of the core concrete on the back face, near mid-height was lost. Reducing the tie spacing to 225 mm resulted in moderate but reduced damage in due to flexure. Further reduction in ties spacing down to 112.5 mm provided excellent concrete confinement and further reduced the damage to column concrete. The flexural damage is directly related to peak displacement experienced by the columns. Peak displacements of 22.3 mm, 20.1 mm and 19.4 mm were recorded for columns with tie spacing of 450 mm, 225 mm and 112.5 mm, respectively. The decrease in peak displacement with the increase in concrete confinement was noticed in columns that did not fail by breaching or scabbing. The confinement effect on peak displacement was more pronounced when the flexural response governed the behaviour. Also, confinement helps in keeping damaged concrete elements in place without too much shattering. This helps minimize fragmentation and flying debris that are potential sources of injuries.

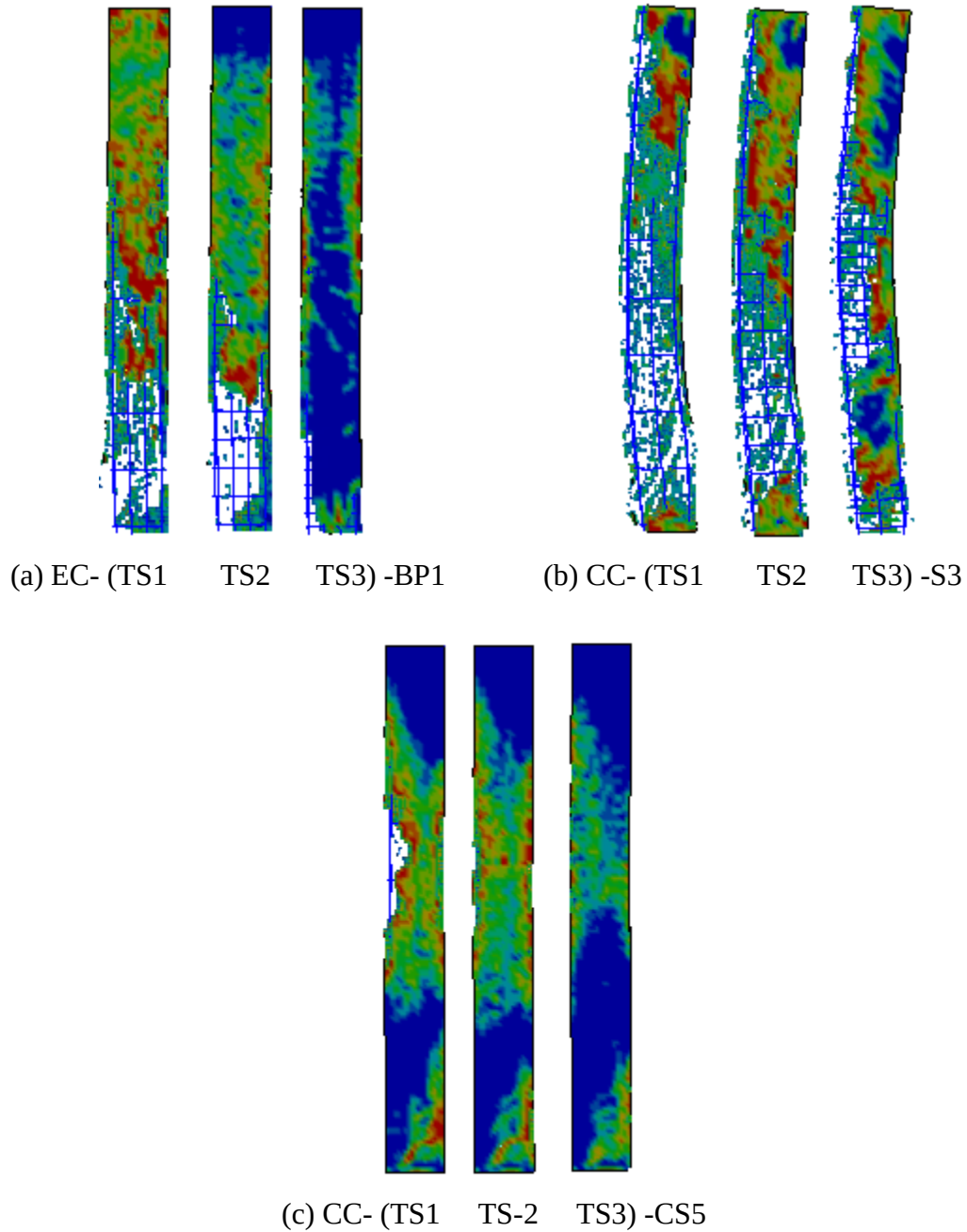


Figure 5-53: Effect of Transverse Reinforcement on Blast Response of Columns

5.3.5 Effect of Column Location (Corner versus Edge)

The effect of column location to blast loading was investigated as it affects column design, rather than their locations relative to shock waves and the presence of potential wrap-around effects. The current research project did not utilize the computational fluid dynamics (CFD) option of Software

LS-DYNA to model the shock wave, which would be required to investigate other parameters of column location associated with the formation and progression of shock waves.

The edge column was designed with 12 – 25M longitudinal bars under 1194 kN axial load. The corner column was designed with 8 – 25M longitudinal bars under 597 kN axial load. The damage levels induced from the different threat mass/standoff combinations differed between the two columns. Generally, the edge columns behaved better in terms of damage levels and peak displacement than the corner columns due to the increase in reinforcement ratio.

Figure 5-54 illustrates the damage induced on a corner and an edge column with transverse reinforcement spacing of 450 mm when exposed to a backpack bomb located at 0.5. The corner column failed due to column breaching. Meanwhile, the edge column developed concrete scabbing at the back face, but did not breach. This is attributed to the closer spacing of 25M longitudinal reinforcement for the edge column which helped in confining the concrete in the critical base region. Moreover, the higher axial compression load on the edge column may significantly contributed to not cause complete column breaching.

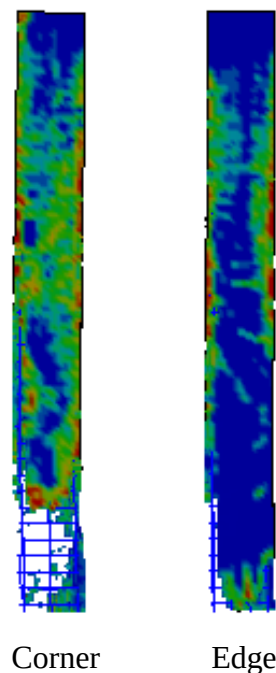


Figure 5-54: Corner versus Edge Columns subjected to Backpack Located at 0.5 m

The effect of the column location within the building layout on the global response of the columns was observed on the columns that were flexural dominant. All flexural dominant edge columns experienced peak displacements that were less than corner columns. This is reflected with flexural capacity enhancement associated with the increase in reinforcement ratio.

6 Chapter 6

Summary and Conclusions

6.1 Summary

Limited research has been conducted in the past on behaviour of reinforced columns subjected to close-in explosions. The available research is mostly limited to blast response under distant explosions. Therefore, the objective of the current research project was selected to investigate the dynamic response of reinforced concrete columns under close-in blast loading numerically, using FEM software LS-DYNA. A six-storey reinforced concrete building was selected to investigate the effects of blast loading on first storey exterior columns. The failure modes were investigated under different standoff distance/charge mass combinations for a corner and an edge column. Three different charge masses were selected for this purpose: i) backpack bomb having 22.67 kg (50 lbs) of TNT, ii) compact sedan car bomb with 227 kg (500 lbs) of TNT and iii) sedan car bomb with 454 kg (1000 lbs) of TNT. The bombs were placed at different standoff distances away from the building, resulting in different failure modes. The columns had a 450 mm square cross-section, designed for appropriate tributary areas for a corner and an edge column with three different transverse reinforcement ratios and spacing.

Computer simulations were carried out using CONWEP data embedded in LS-DYNA software to compute blast pressure-duration combinations from different charge weight and standoff distance scenarios. The material that was used to model the concrete was *MAT159-CSCM_CONCRETE, which is a plasticity-based damage model. The damage was calculated from strain softening due to strength degradation beyond the peak strength, and reductions in material modulus due to loading and unloading. Steel reinforcement was modelled with MAT003-PLASTIC_KINEMATIC, which is an elasto-plastic model. Verification of the material models and the blast loading simulations were carried out by comparing the numerical results with an experiment conducted by Woodson and Baylot (1999) on a 2-storey, quarter scale reinforced concrete building.

An analytical parametric investigation was conducted to assess the significance of the blast parameters considered. The following section presents the conclusions attained from the research project.

6.2 Conclusions

The main conclusions that can be drawn from the analytical investigation conducted in the current research project are presented below:

- Blast response of reinforced concrete columns due to small charge mass explosives, as in the case of backpack bombs, located at a close distance to the column may be governed by local behaviour. The local failure mode is initiated by the scabbing of concrete at the back face, followed by full breaching of concrete. Peak displacements associated with local behaviour are usually small. The 22.67 kg (50 lbs) TNT backpack bomb considered in the current investigation resulted in the breaching failure near the column base when placed at 0.5 m or 0.75 m away from a 450 mm column. The failure mode shifted to diagonal tension when the standoff distance was increased to 1.5 m while having insufficient transverse reinforcement to control inclined shear cracking. The use of sufficient shear reinforcement further shifted the mode of behaviour to flexure.
- Response of reinforced concrete columns due to large charge mass explosives located at a close distances is governed by direct shear failure at the supports, followed by global column response. In such cases the failure occurs within the first few milliseconds of the explosions due to the excessive concentration of shear stresses associated with blast shock wave at support reactions. This failure mode subsequently results in global failure as the deterioration of concrete propagates towards the column mid-height under high impulses associated with larger charge weights. The compact sedan car bomb considered in the current investigation, with 227 kg TNT charge weight produced a direct shear failure when placed at 1.5 m away from a 450 mm concrete column, followed by a global failure with continued concrete deterioration above the support region. The same failure mode occurred under a sedan car bomb with 454 kg TNT charge weight at 1.5 m and 3.0 m standoff distances.

- At higher standoff distances, the response of reinforced concrete columns is governed by global damage. The possible failure modes include i) diagonal shear failure due to lack of shear capacity, ii) flexural failure due to lack of sufficient flexural capacity, or iii) failure in rebound motion due to the concentration of shock wave stresses near supports. In such columns, the amount and spacing of transverse reinforcement dictate whether the failure is governed by diagonal tension or flexure.
- The same scaled distance under different charge weight results in the same reflected pressure on the column face. However, the impulse generated by each explosive can be substantially different, resulting in significant differences in column damage. The comparison of a backpack bomb with 22.67 kg (50 lbs) TNT and a compact sedan car bomb with 227 kg TNT (10 times higher charge weight), both having the same scaled distance of $0.178 \text{ m/kg}^{1/3}$ indicates that the backpack bomb resulted in column breaching, whereas the car bomb resulted in direct shear failure, even though they both experienced the same peak reflected pressure.
- Confinement of concrete with closely spaced transverse reinforcement of sufficient reinforcement ratio helps reduce the local damage associated with breaching due to the increased strength of confined core concrete. In such columns with close-in explosions the damage may be limited to scabbing of the back face, while the confinement of the core concrete helps minimize flying concrete fragments.
- Increased amount and reduced spacing of transverse column reinforcement controls diagonal tension, as well as diagonal compression (due to confinement), promoting more ductile flexural response. Confined concrete also reduces column deformations at large standoff distances. This reduction is more prevalent at smaller standoff distances if the column survives the blast. The confinement also helps prevent rebound motion failure by enhancing the flexural capacity associated with higher compressive strength of confined concrete.
- Increasing standoff distance helps reduce the initial velocity of columns during response to blast loads, hence lowering the traveling velocity of flying debris, and thereby minimizing the lethal effects of fragmentation.

- Columns with higher percentage of longitudinal reinforcement, well distributed along the perimeter of the column section, controls scabbing and breaching more so than columns with low percentage of reinforcement. The edge columns of the current investigation, with higher number and percentage of longitudinal re-bars showed better performance than the corner columns with smaller percentage of reinforcement when subjected to close-in explosions.

6.3 Recommendations

It is a practical requirement to increase standoff distance as much as possible for a given building layout and location for protection against bomb blasts. Increasing standoff distance can be achieved by altering the building layout perimeter. However, this may not be possible in certain applications. When the standoff distance is limited, obstructions can be provided around the building perimeter to keep intruders at sufficiently far distances. Obstructions such as bollards, barriers, fences, gates and trees help limit the detrimental effects of blast loads, especially if the charge weight associated specified in threat assessment is high, as in the case of car or truck bombs. Obstructions are not effective against intruders with backpack or hand-held bombs. Hence, other security measures need to be employed for bombs that can be carried by intruders. Security screening and surveillance may be used as effective blast risk mitigation techniques.

When other measures as described above cannot be implemented, structural hardening of the building may become necessary. The vulnerability of the first storey columns should be assessed through blast analysis, like the approach employed in the current investigation, and if needed, should be hardened and retrofitted.

6.4 Recommendations for Future Research

The following areas of research are recommended for future investigations:

- Investigate the effect of axial load on the failure modes of reinforced concrete column under close-in explosions.

- Study the effect of column cross section shape and dimension on the response of reinforced concrete columns.
- Explore the effect of blast loading on columns by modelling the whole building façade frame to evaluate progressive collapse possibilities.
- Investigate the effect of using different material models that works well for high loading rate problems.
- Perform computational fluid dynamic (CFD) analysis to model the effect of shock wave clearing of open framed structures.
- Conduct CFD analysis to assess blast load characteristics on columns with non-structural infill walls on either one or both sides of columns.

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Appendix A : Damage Evolution

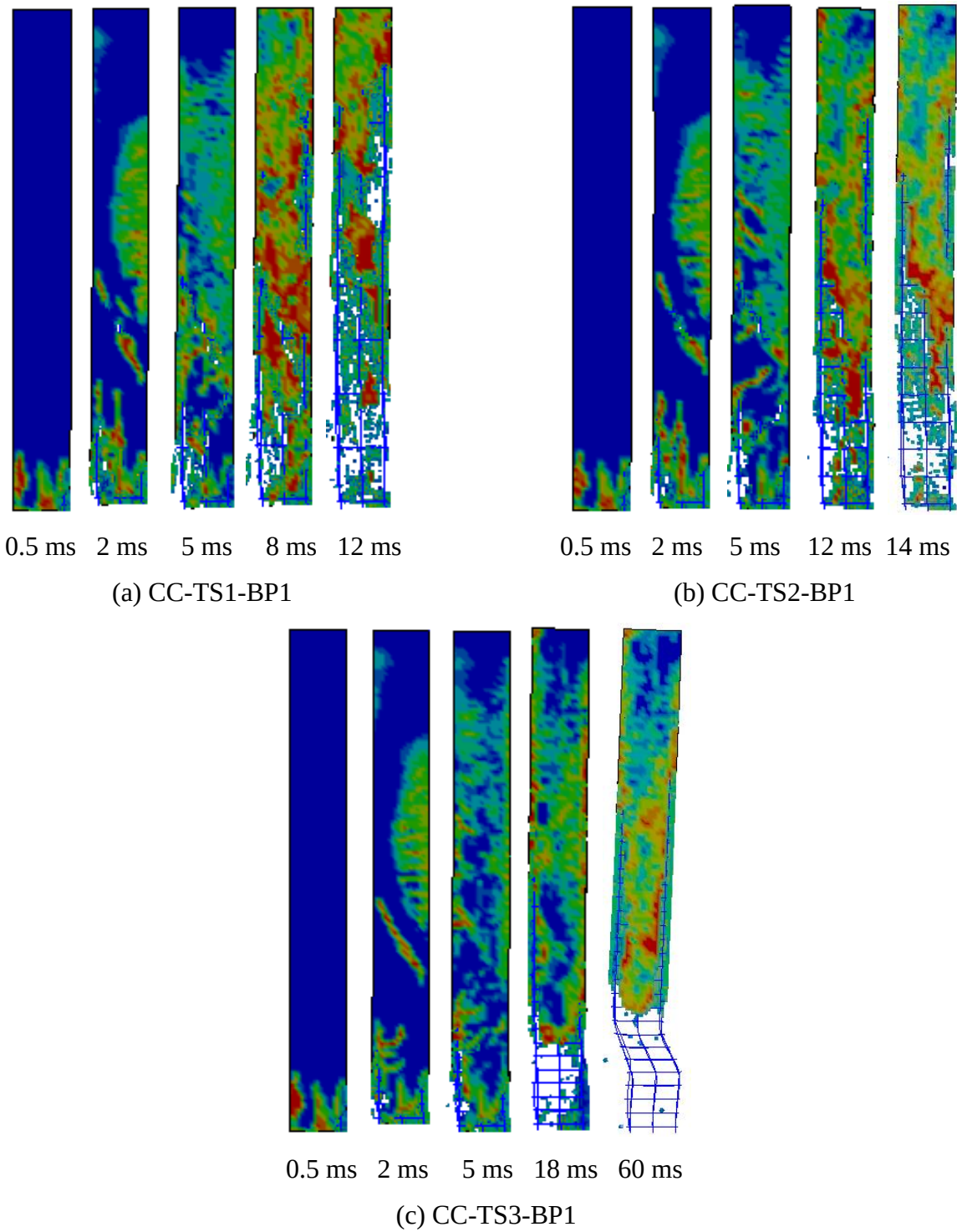


Figure A-1: Damage Evolution for CC Columns under BP1 Loading

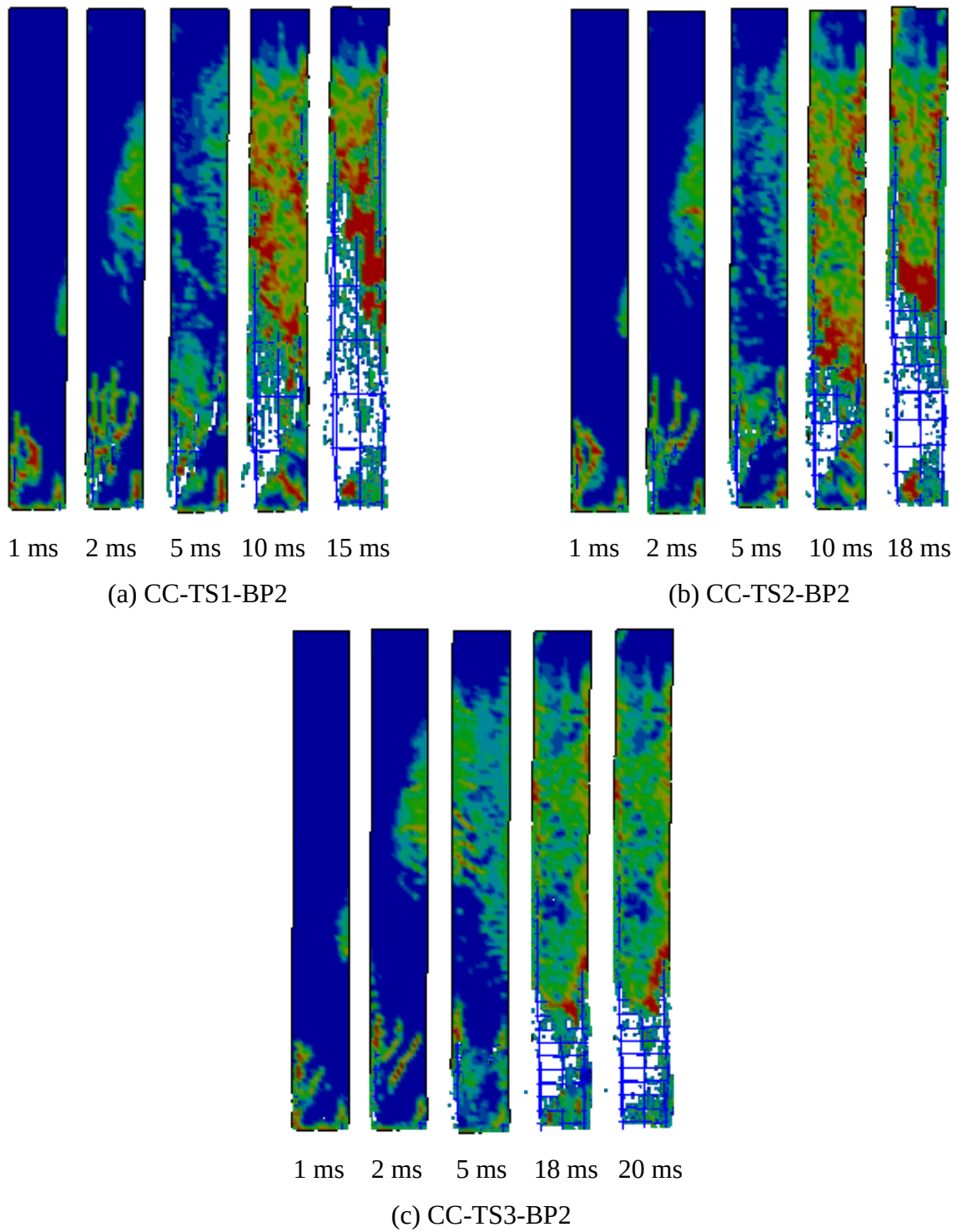


Figure A-2: Damage Evolution for CC Columns under BP2 Loading

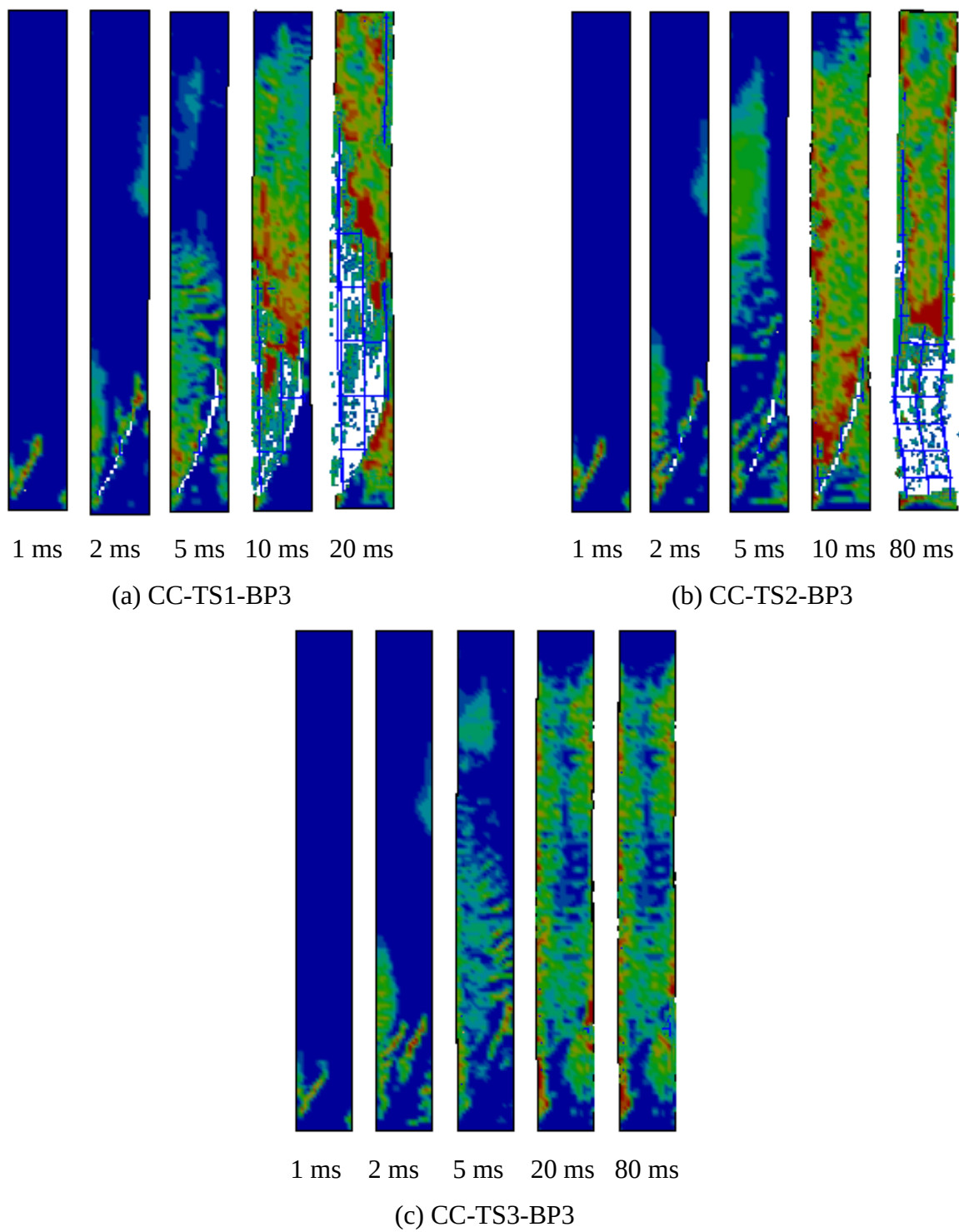


Figure A-3: Damage Evolution for CC Columns under BP3 Loading

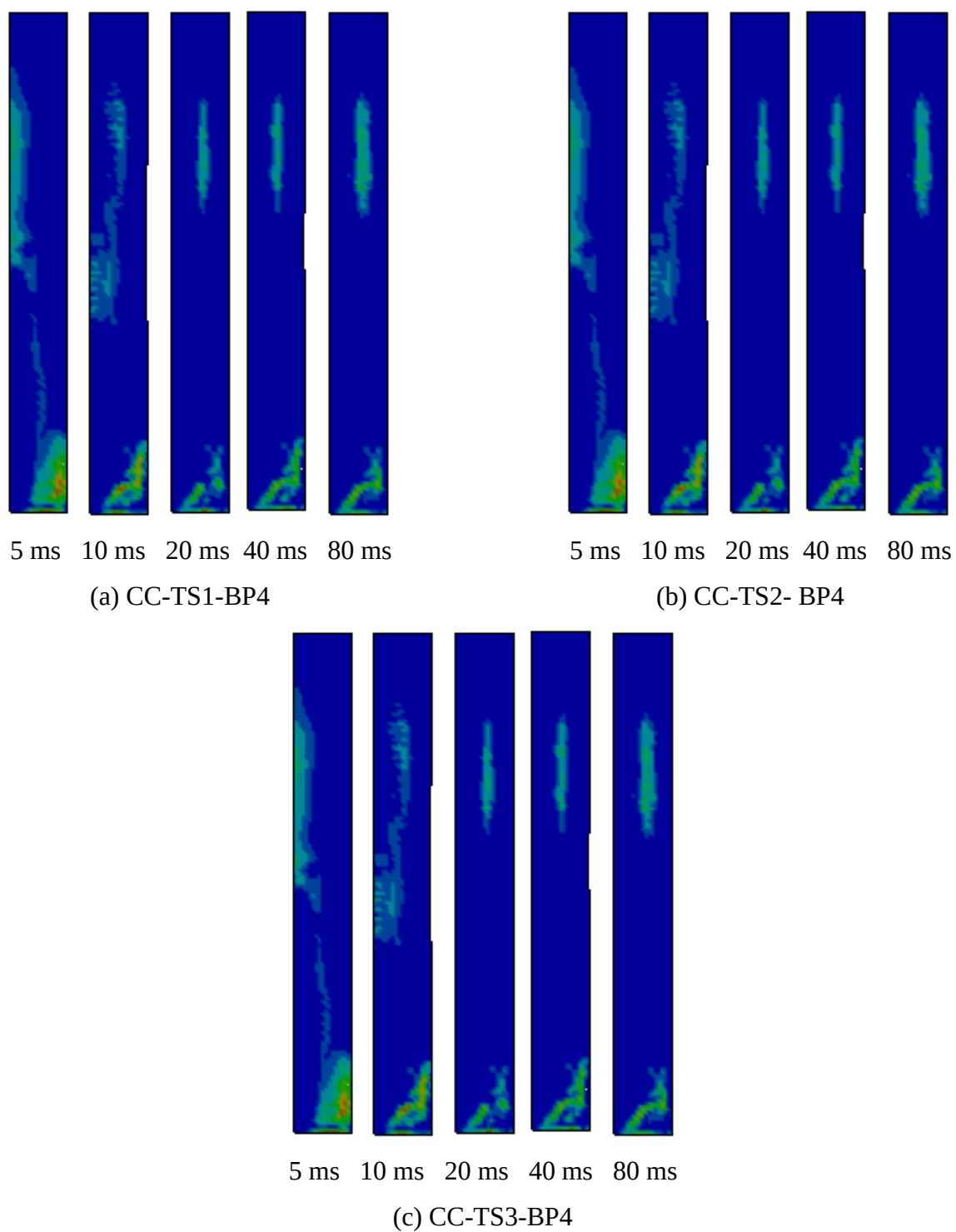


Figure A-4: Damage Evolution for CC Columns under BP4 Loading

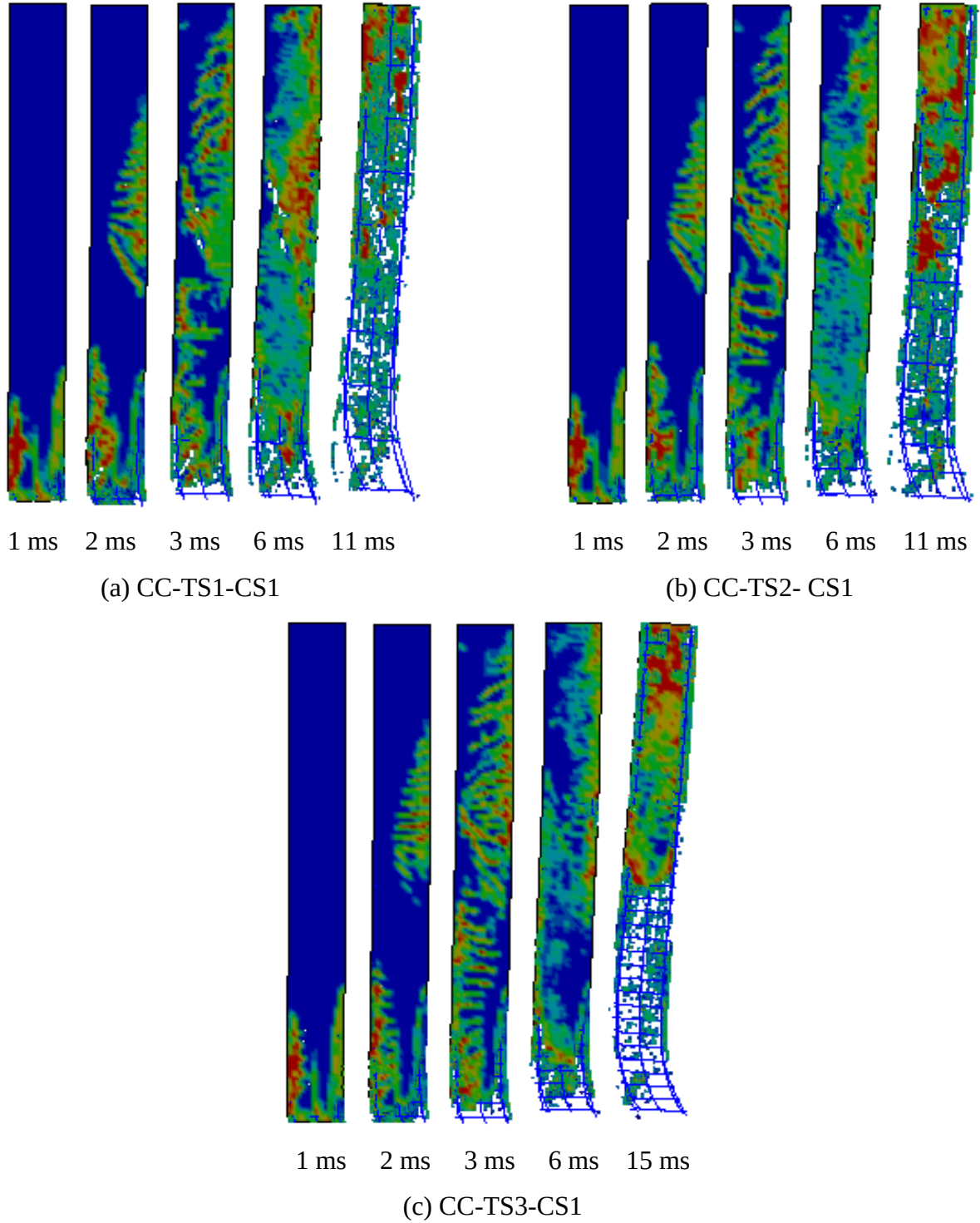


Figure A-5: Damage Evolution for CC Columns under CS1 Loading

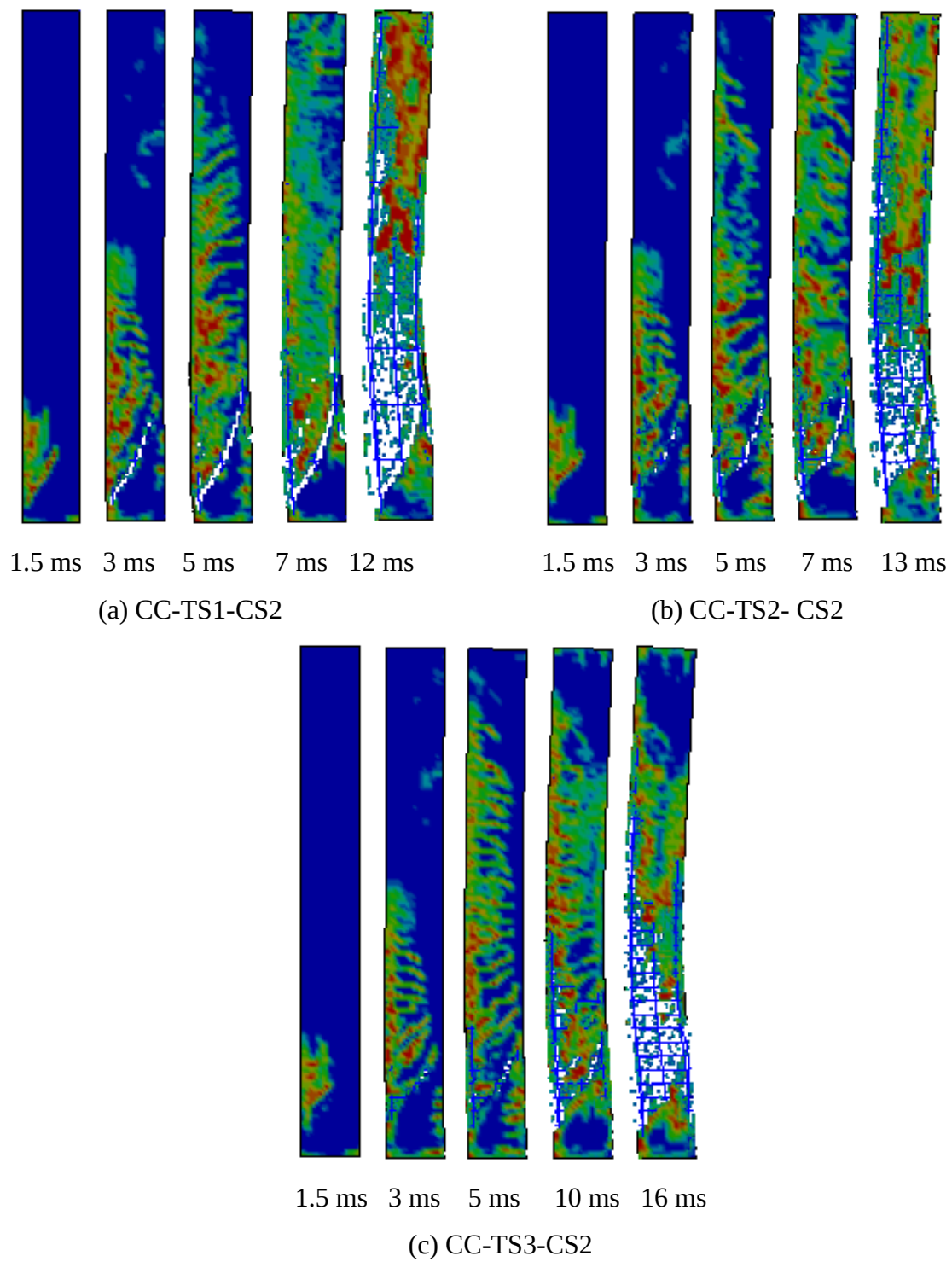


Figure A-6: Damage Evolution for CC Columns under CS2 Loading

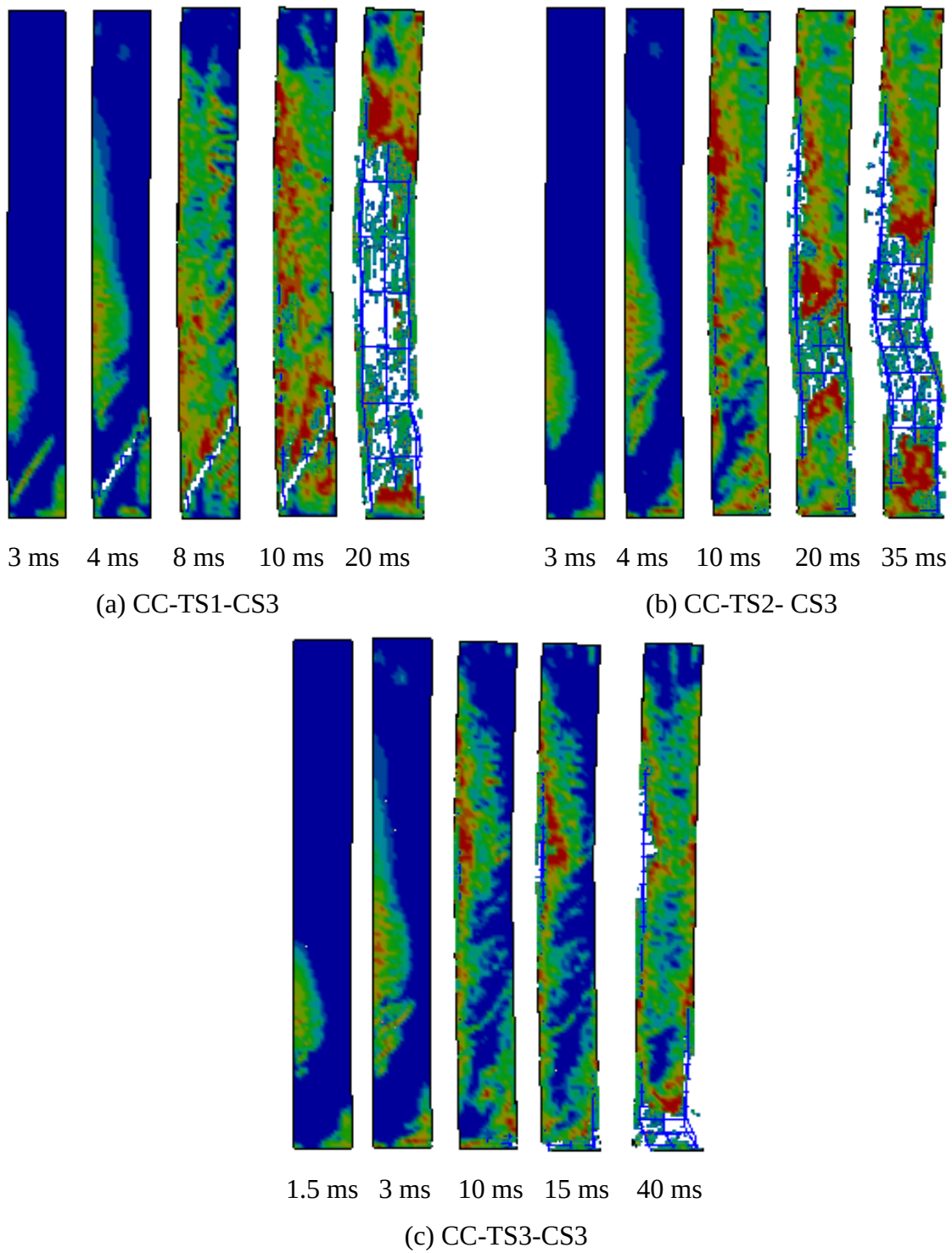


Figure A-7: Damage Evolution for CC Columns under CS3 Loading

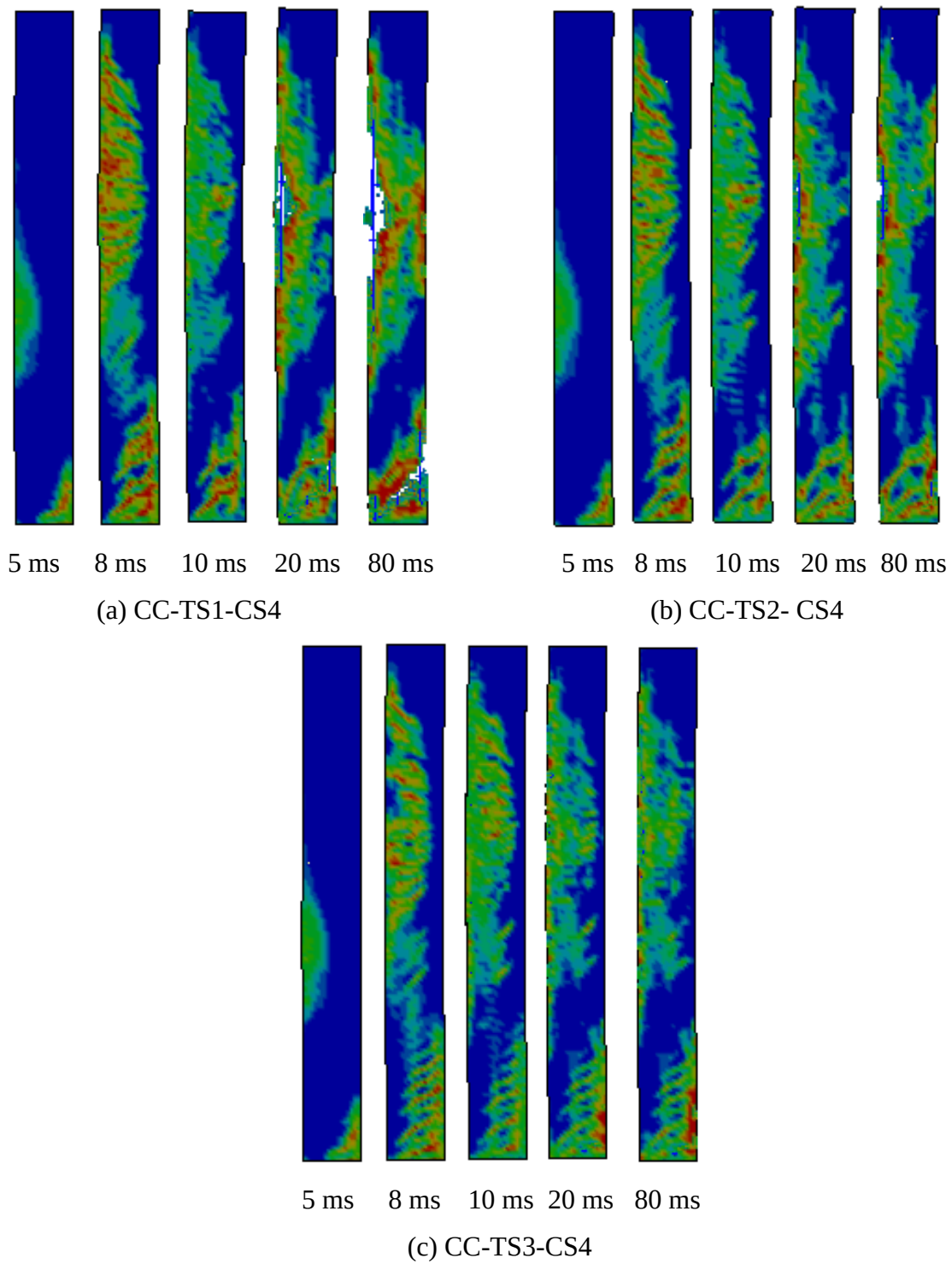


Figure A-8: Damage Evolution for CC Columns under CS4 Loading

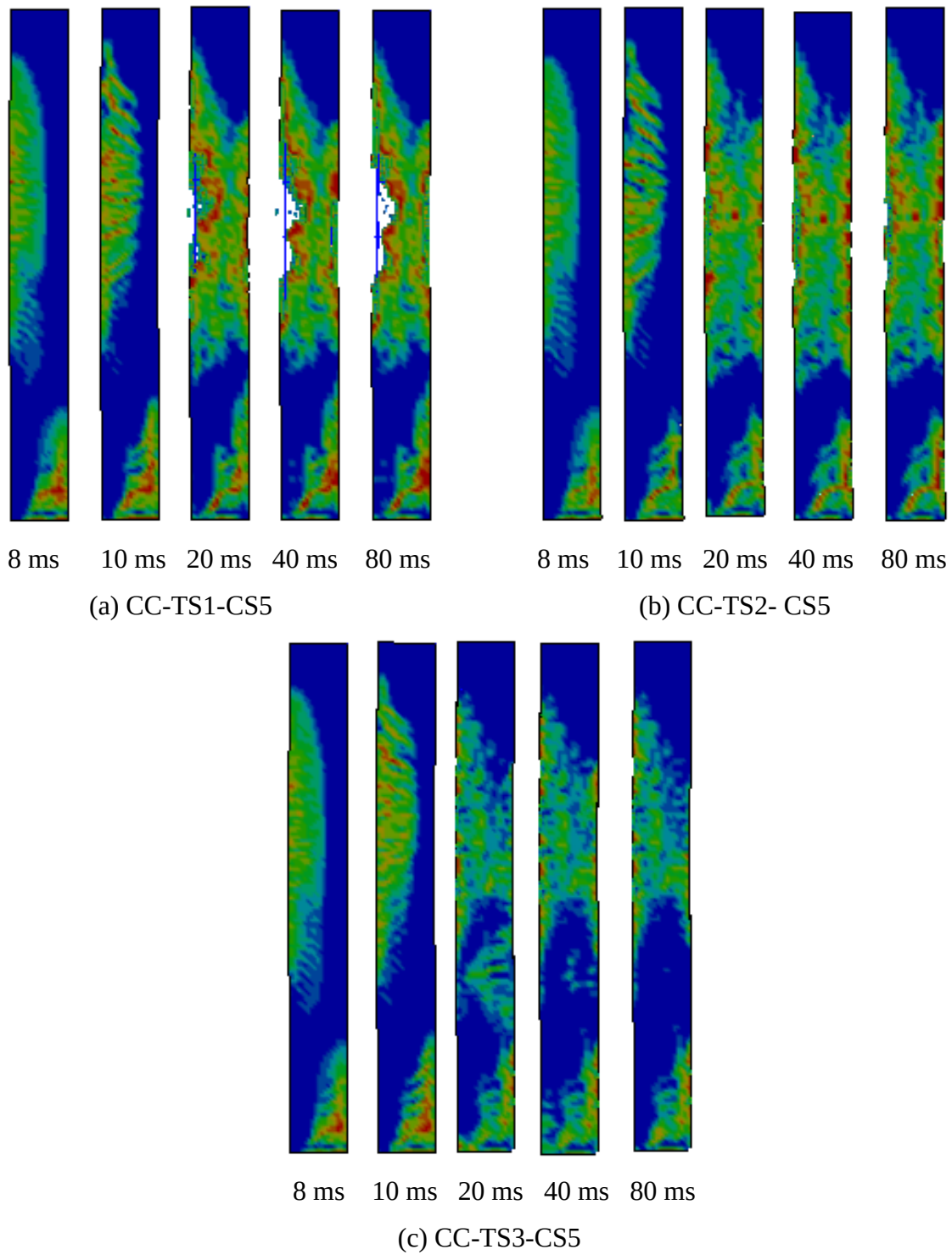


Figure A-9: Damage Evolution for CC Columns under CS5 Loading

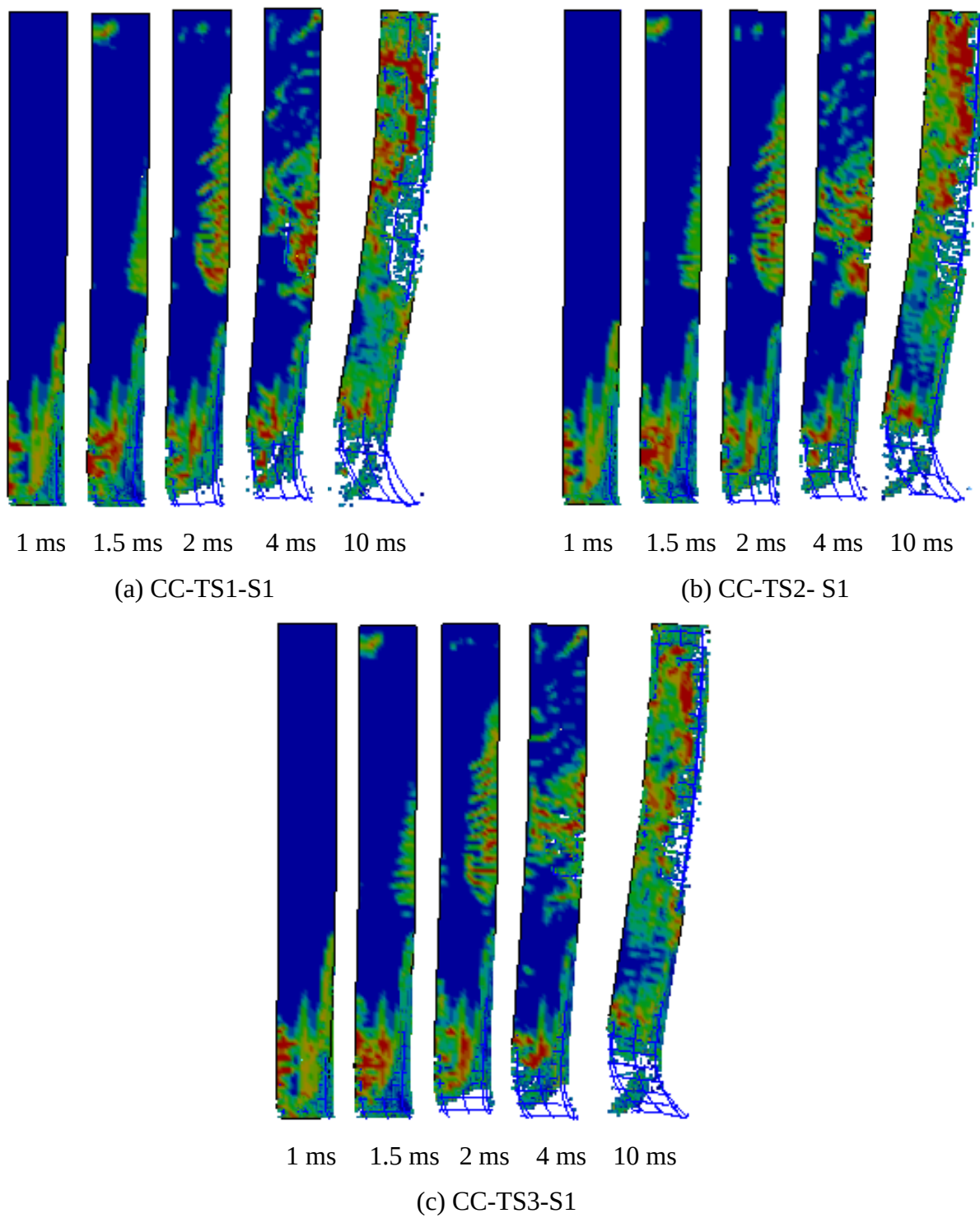


Figure A-10: Damage Evolution for CC Columns under S1 Loading

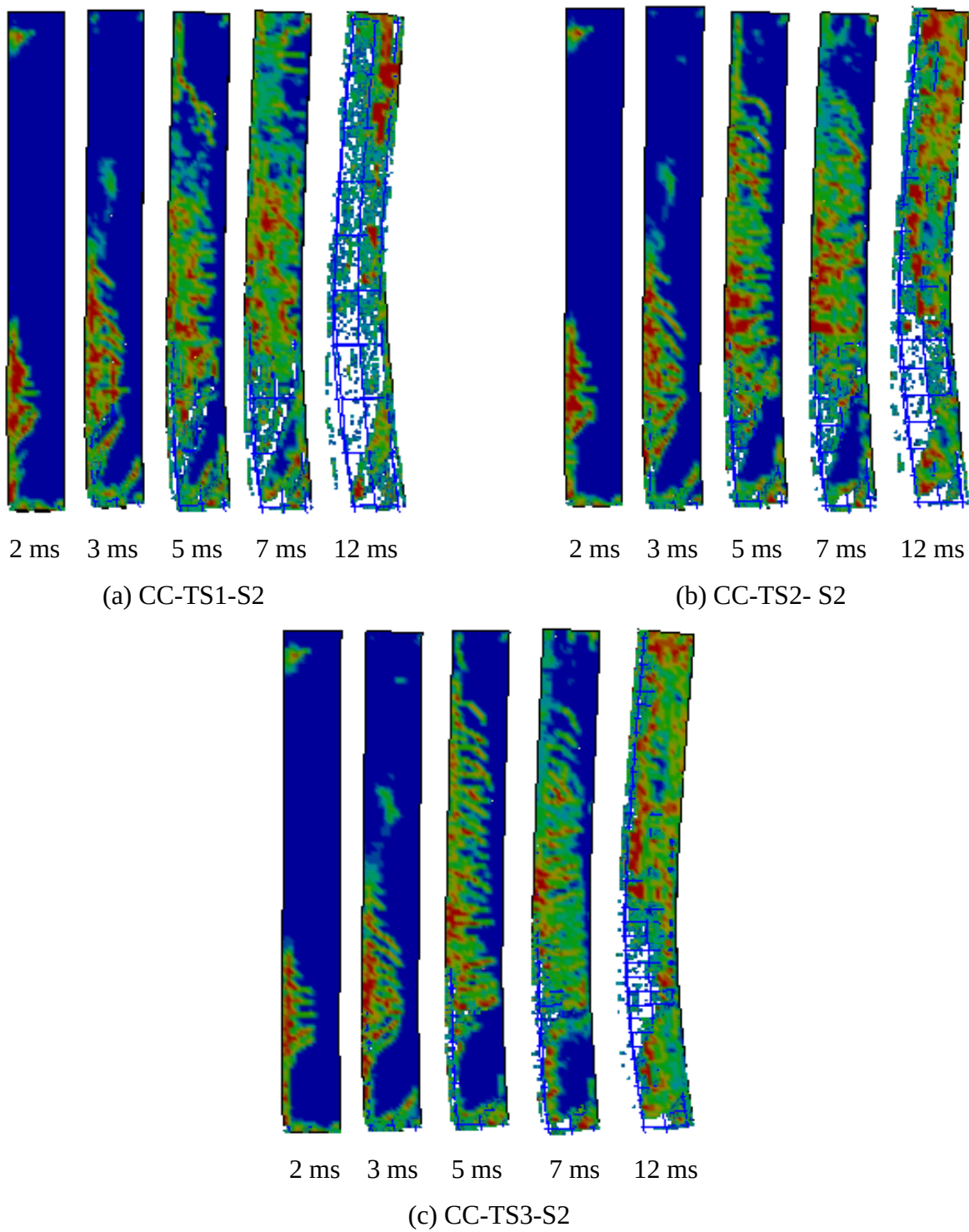


Figure A-11: Damage Evolution for CC Columns under S2 Loading

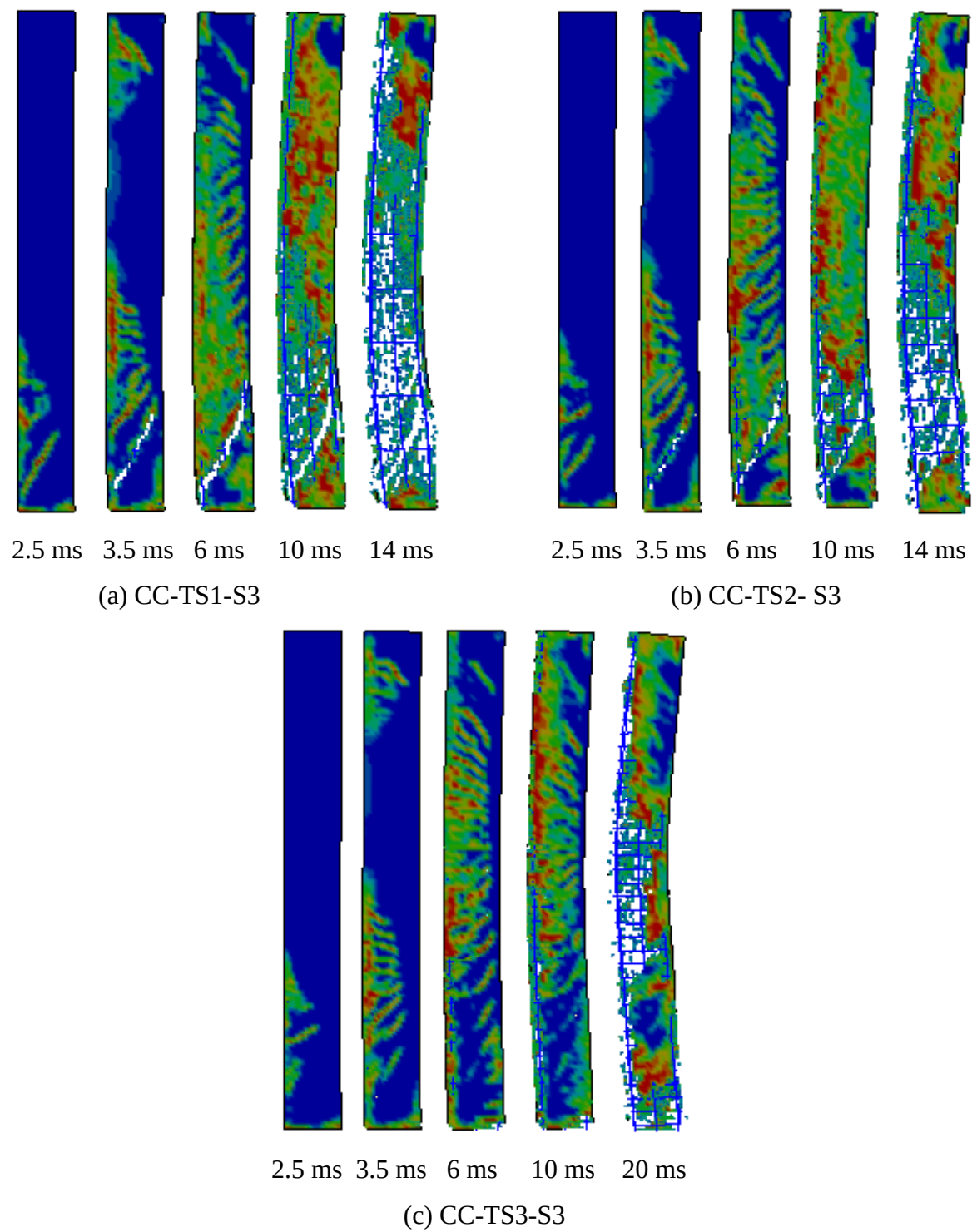


Figure A-12: Damage Evolution for CC Columns under S3 Loading

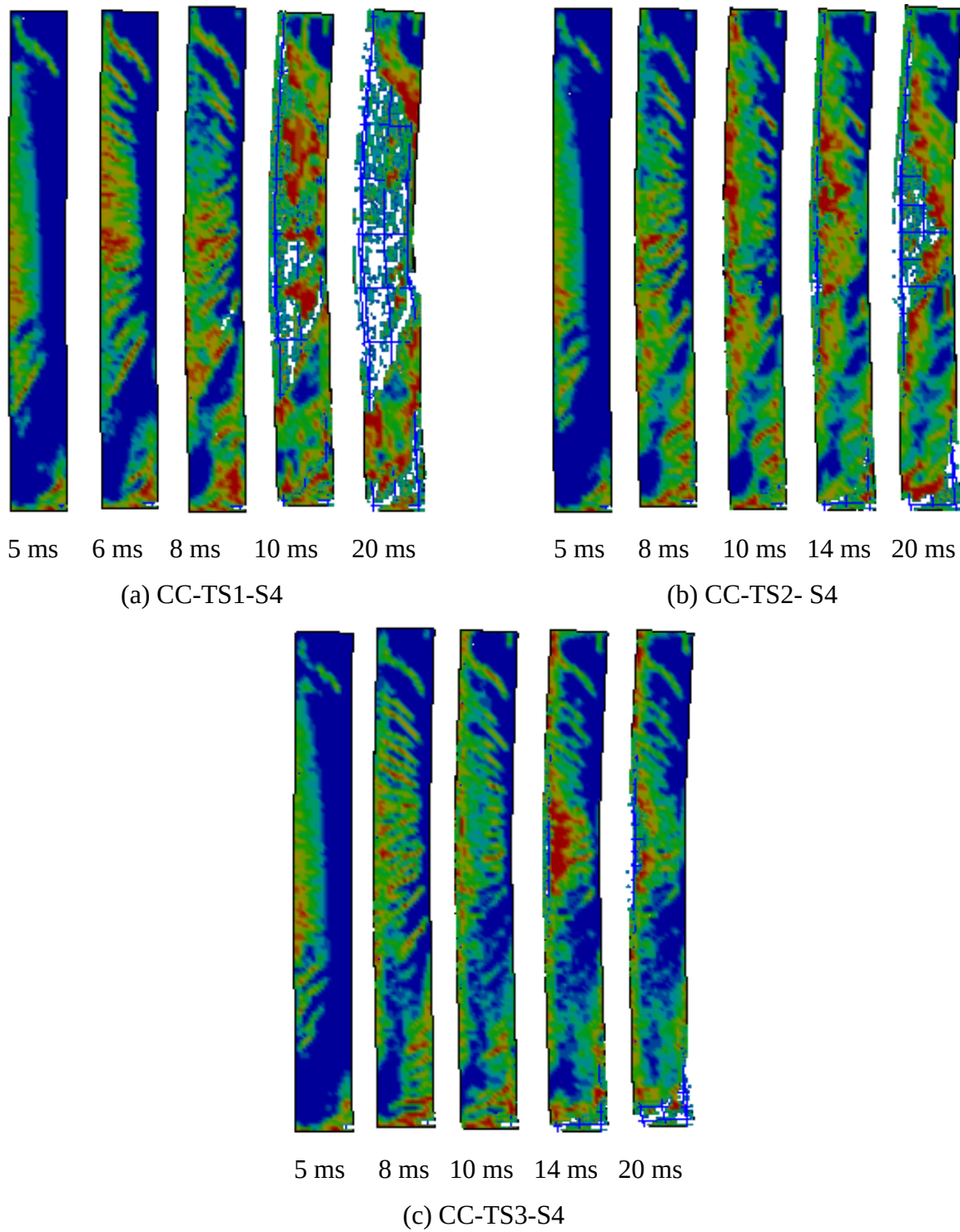


Figure A-13: Damage Evolution for CC Columns under S4 Loading

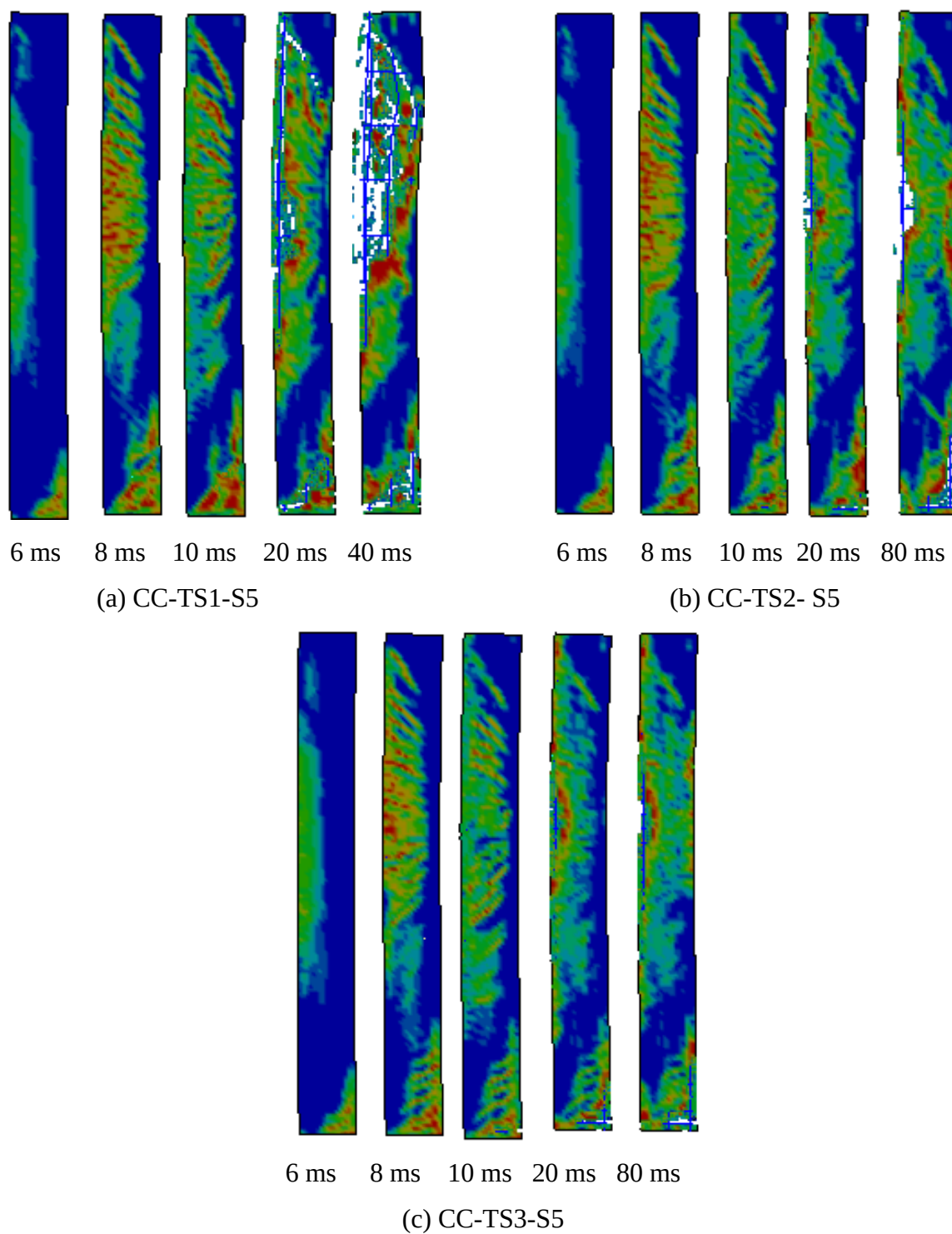


Figure A-14: Damage Evolution for CC Columns under S5 Loading

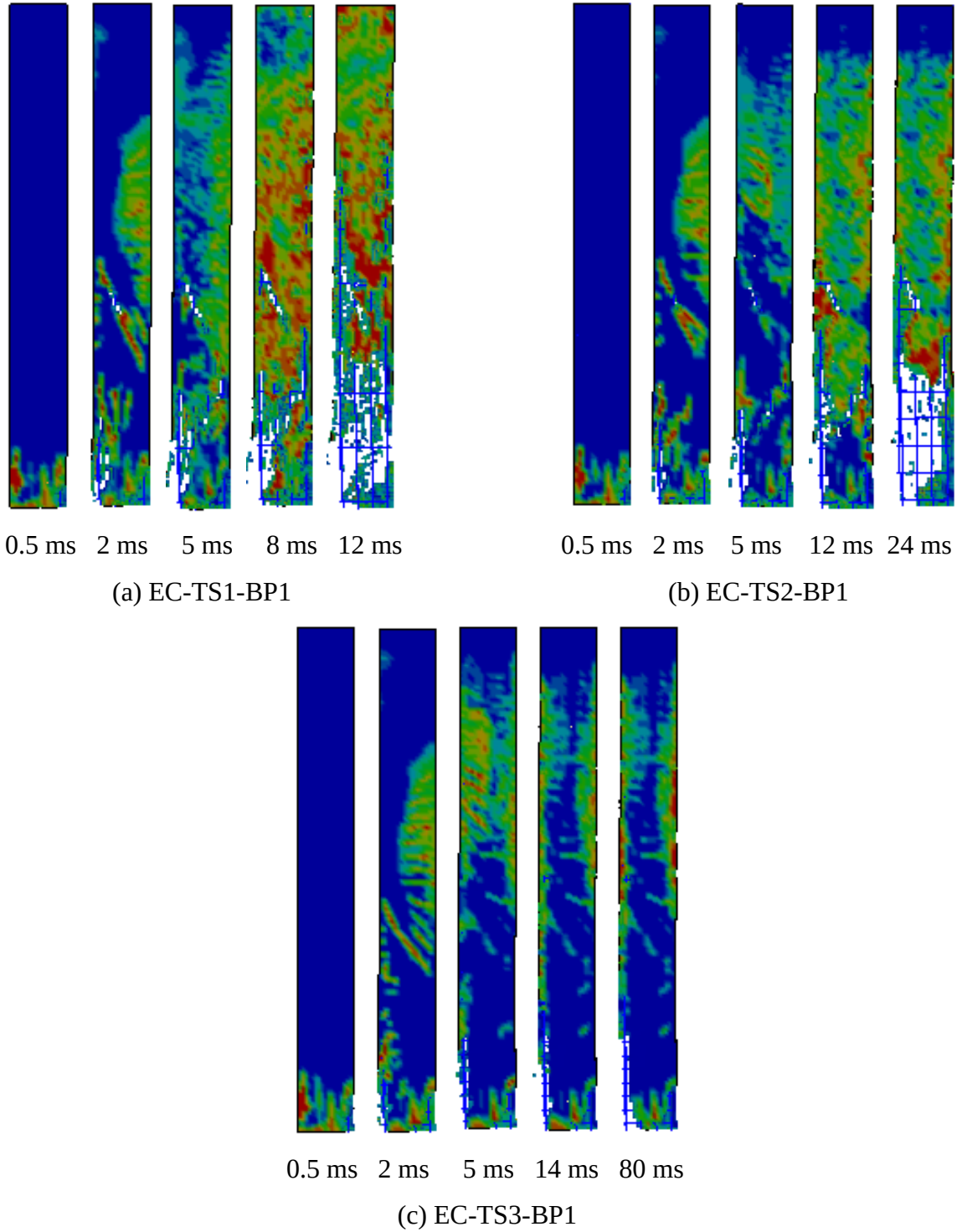


Figure A-15: Damage Evolution for EC Columns under BP1 Loading

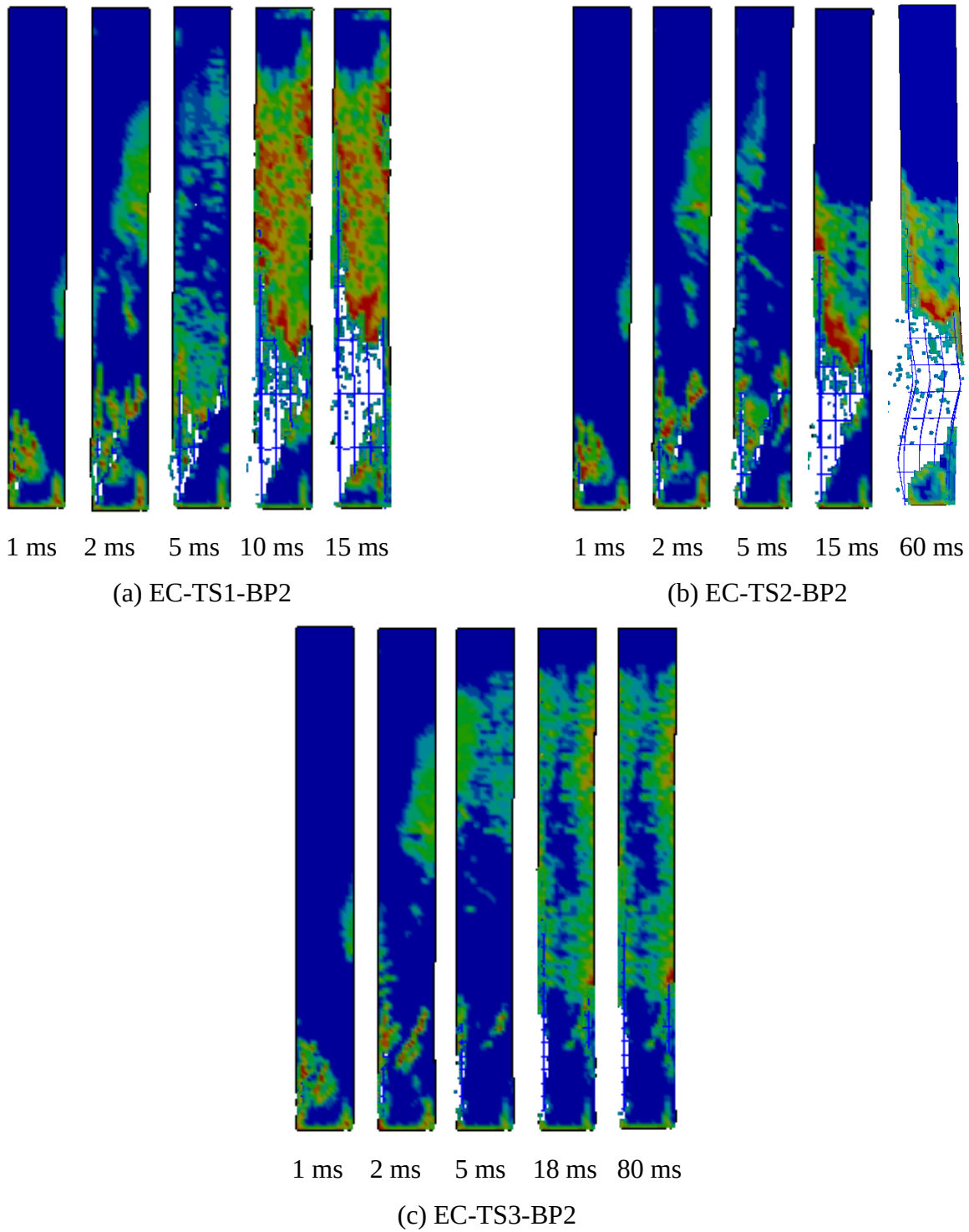


Figure A-16: Damage Evolution for EC Columns under BP2 Loading

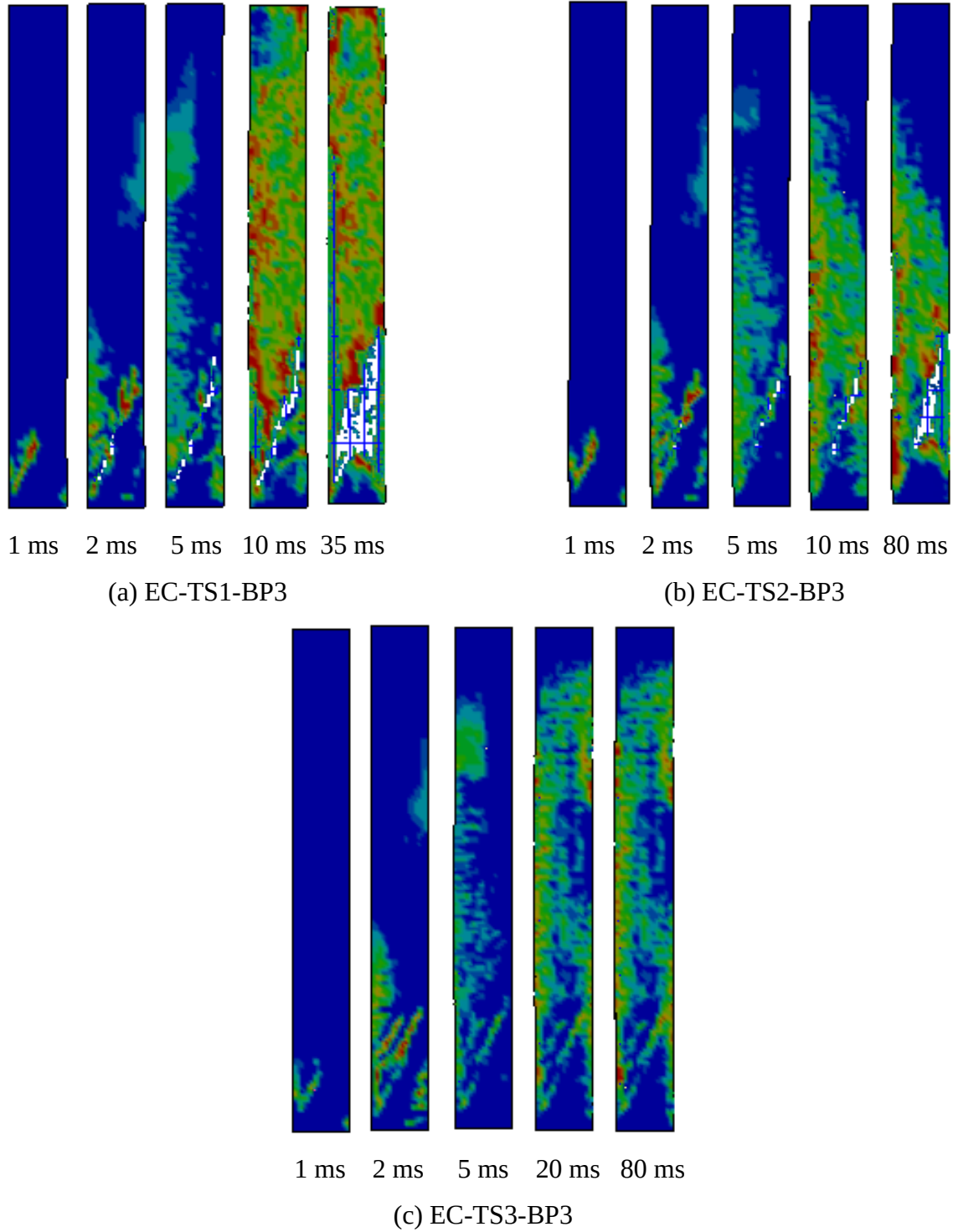


Figure A-17: Damage Evolution for EC Columns under BP3 Loading

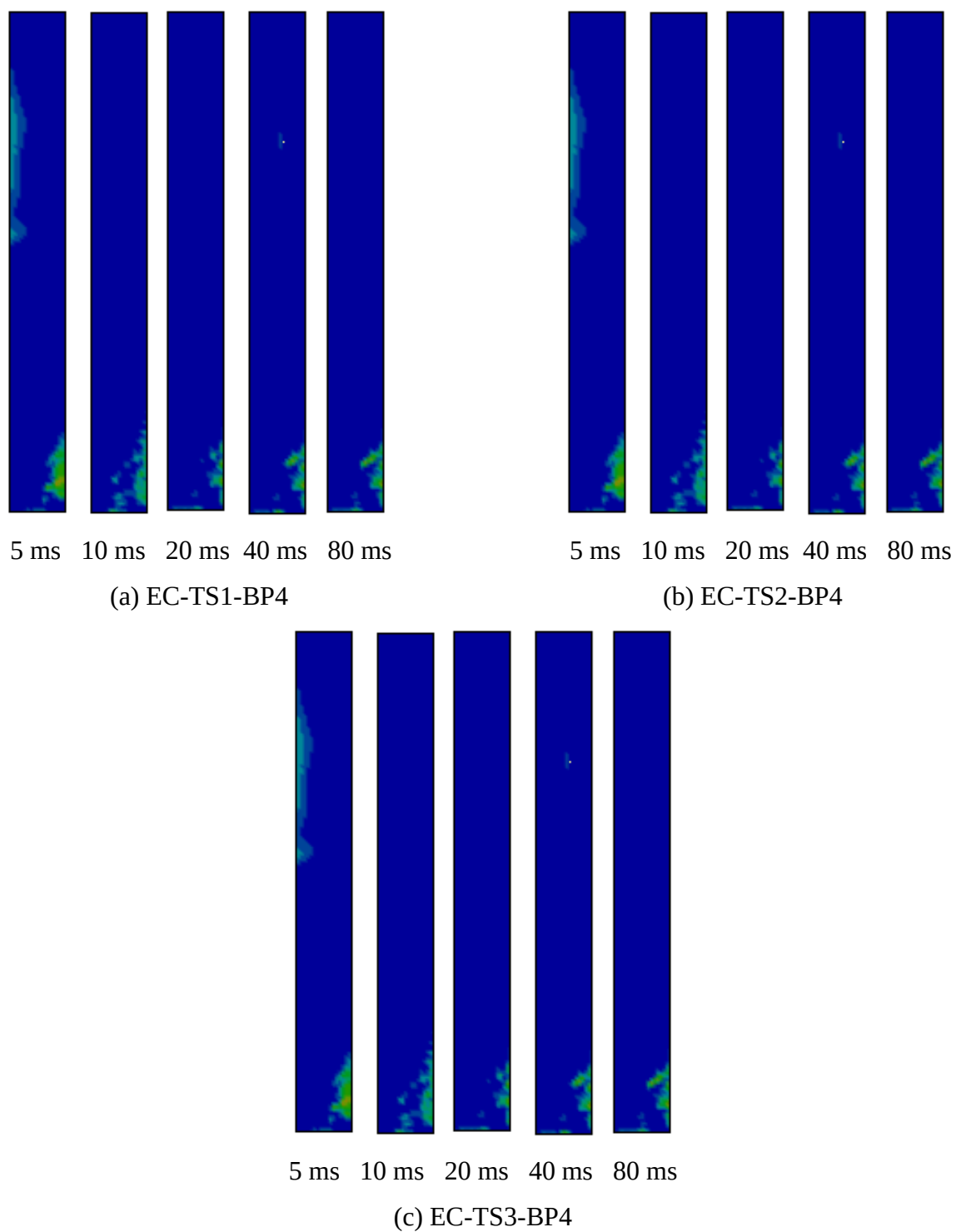


Figure A-18: Damage Evolution for EC Columns under BP4 Loading

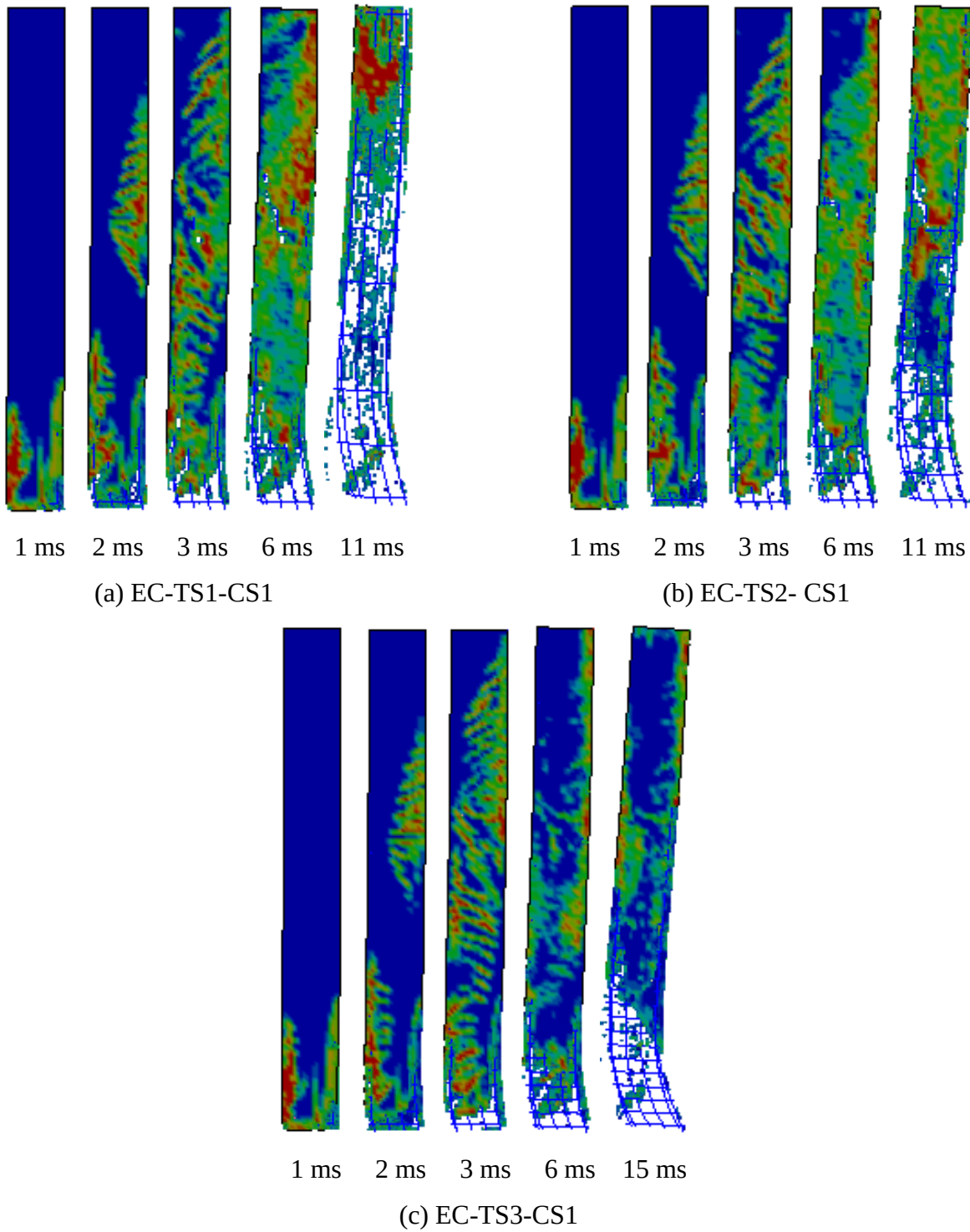


Figure A-19: Damage Evolution for EC Columns under CS1 Loading

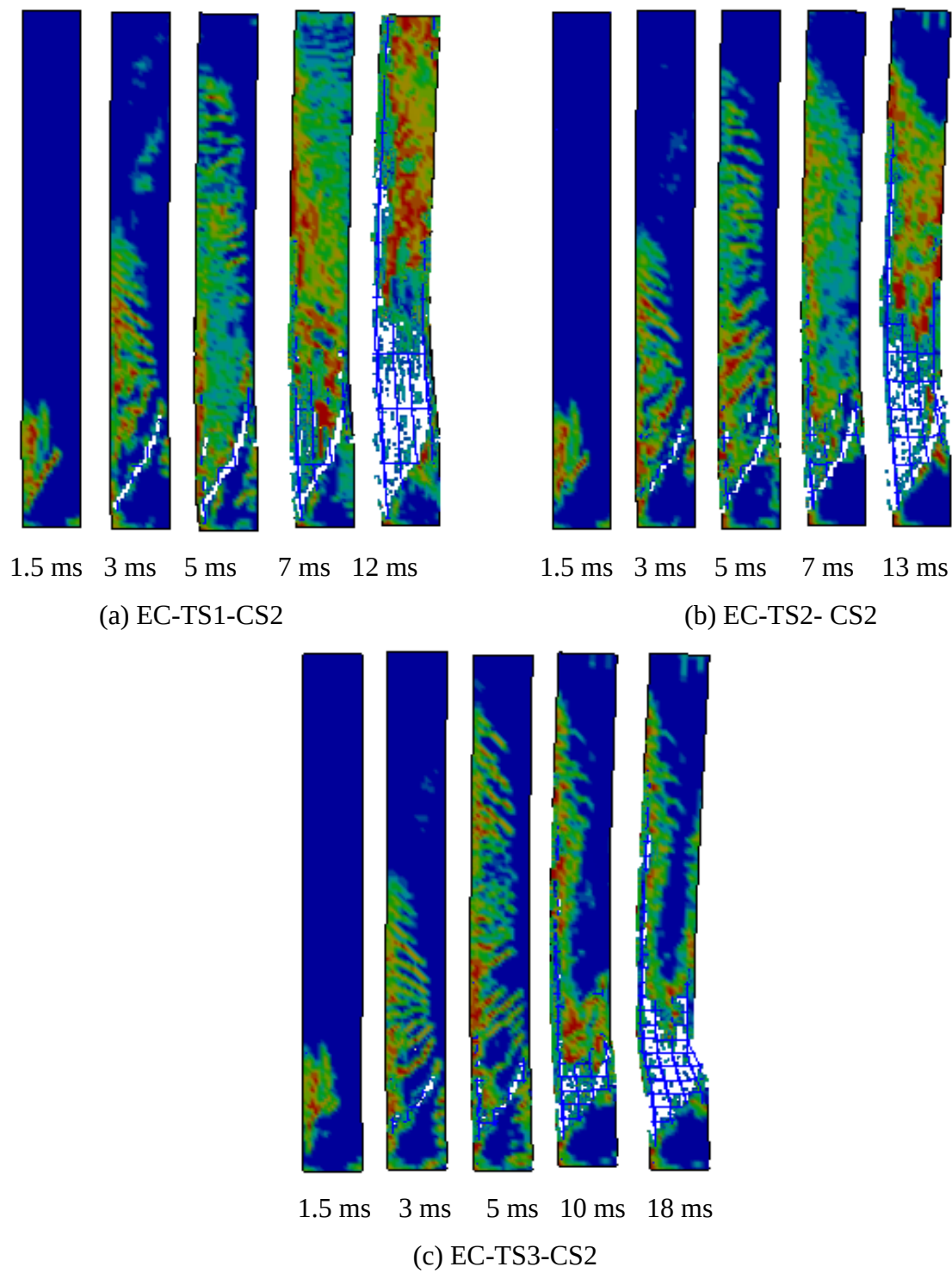


Figure A-20: Damage Evolution for EC Columns under CS2 Loading

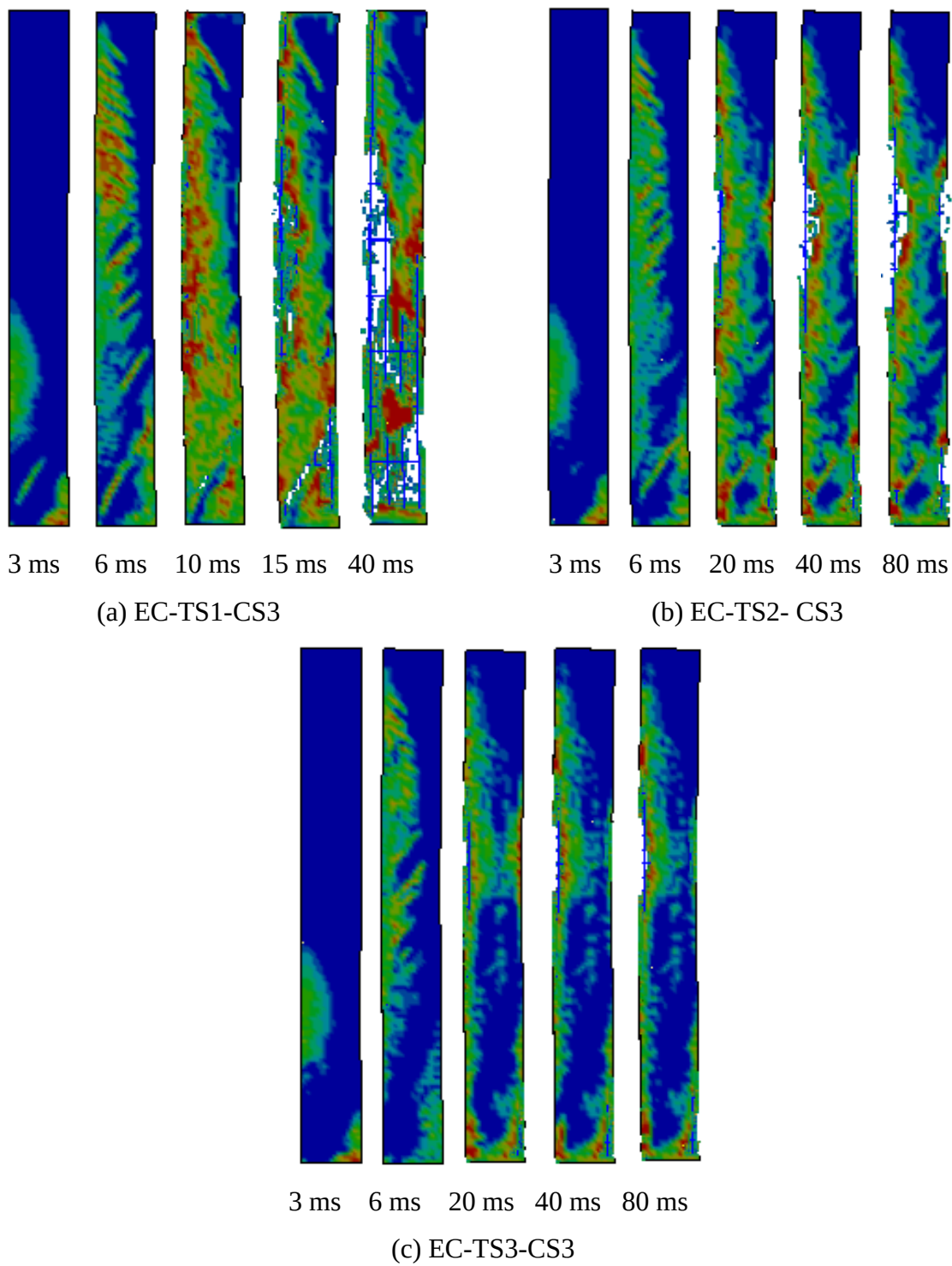


Figure A-21: Damage Evolution for EC Columns under CS3 Loading

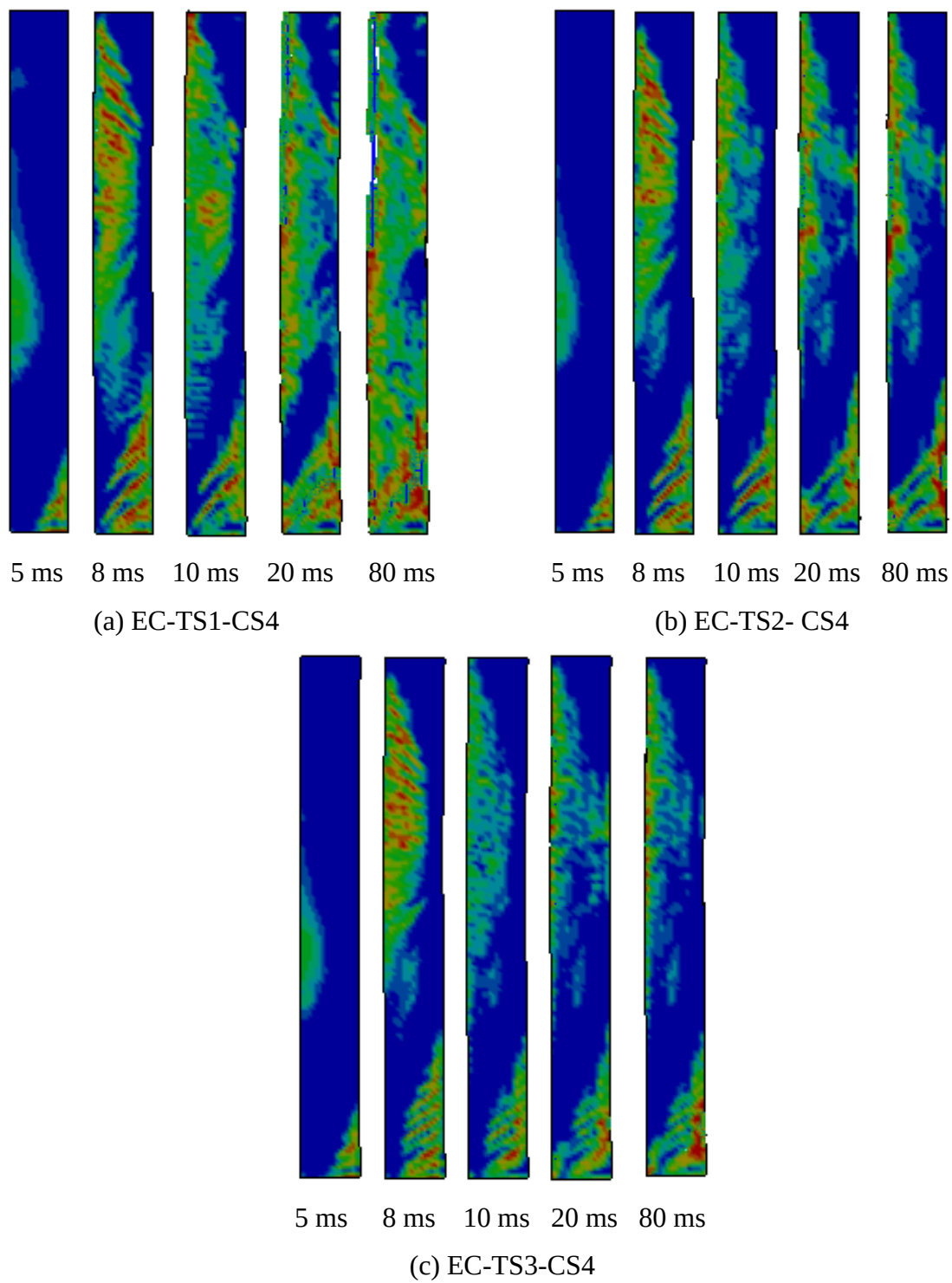


Figure A-22: Damage Evolution for EC Columns under CS4 Loading

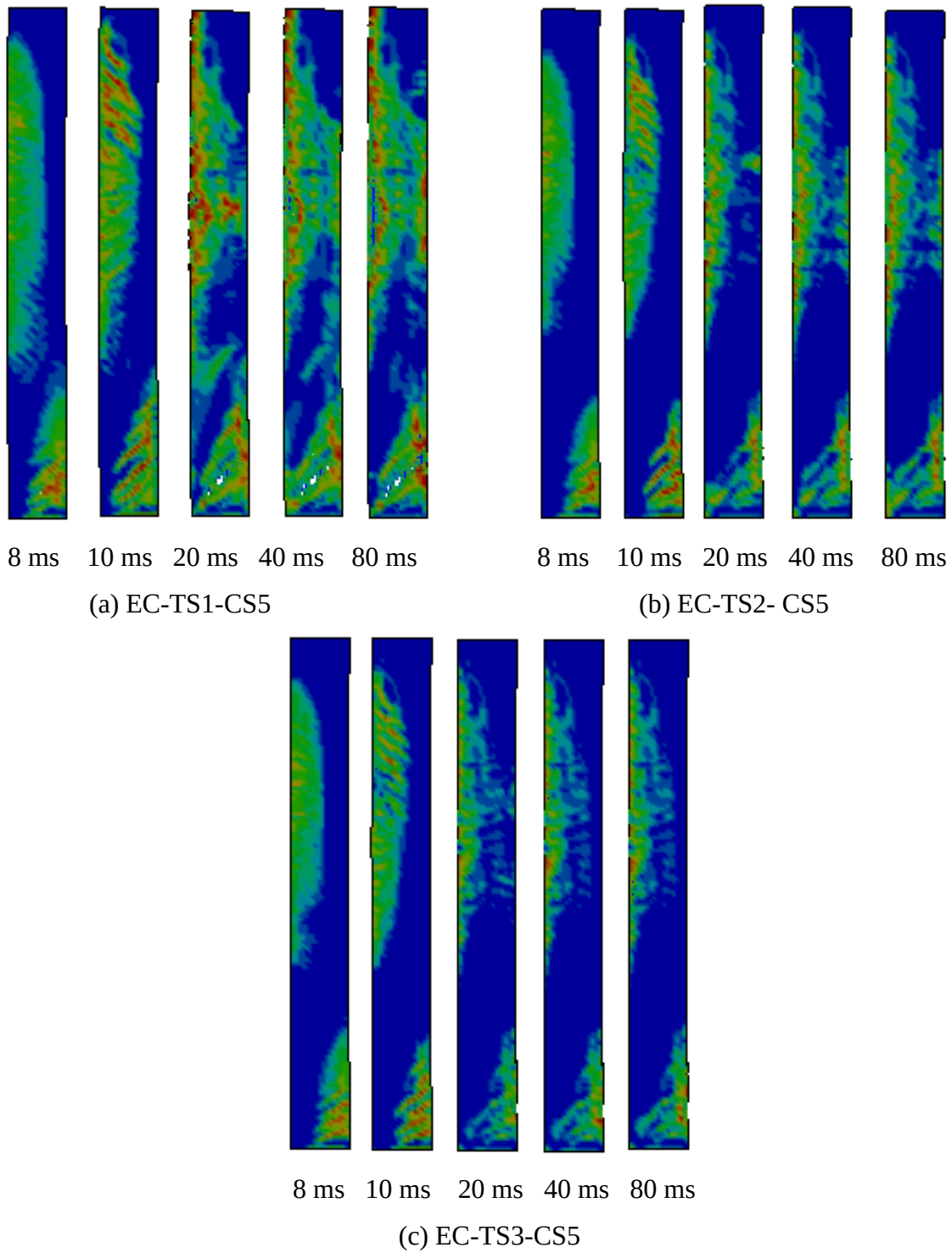


Figure A-23: Damage Evolution for EC Columns under CS5 Loading

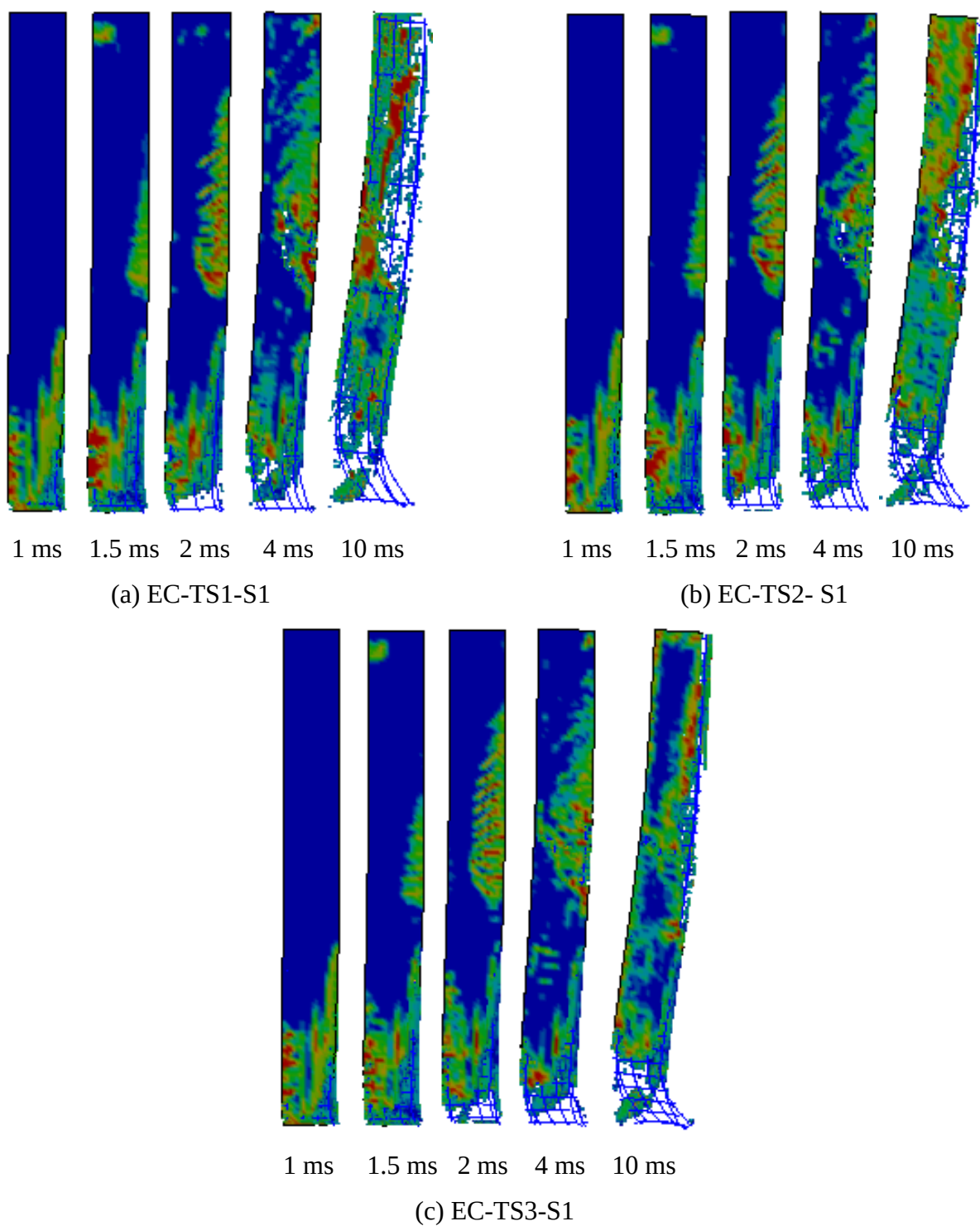


Figure A-24: Damage Evolution for EC Columns under S1 Loading

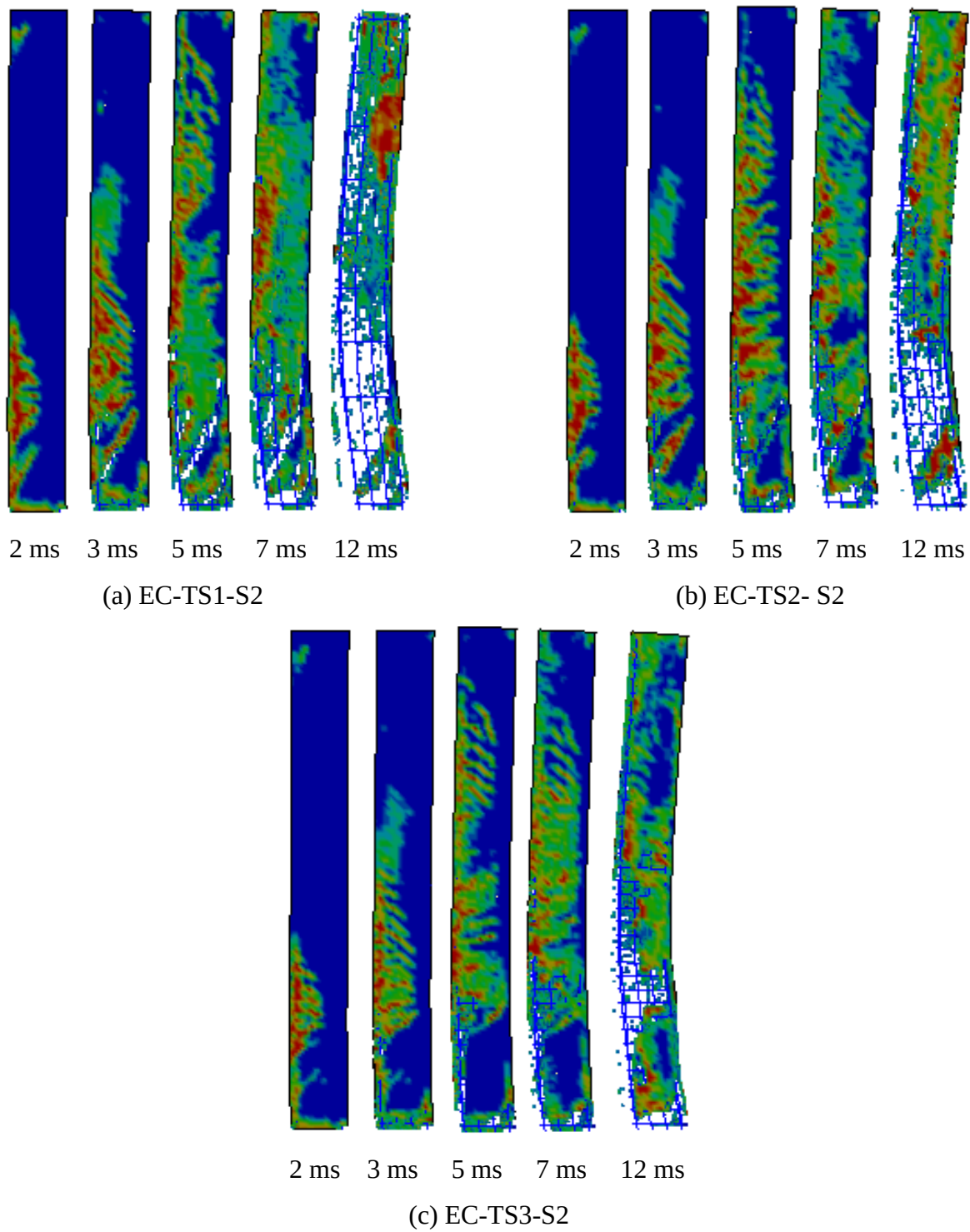


Figure A-25: Damage Evolution for EC Columns under S2 Loading

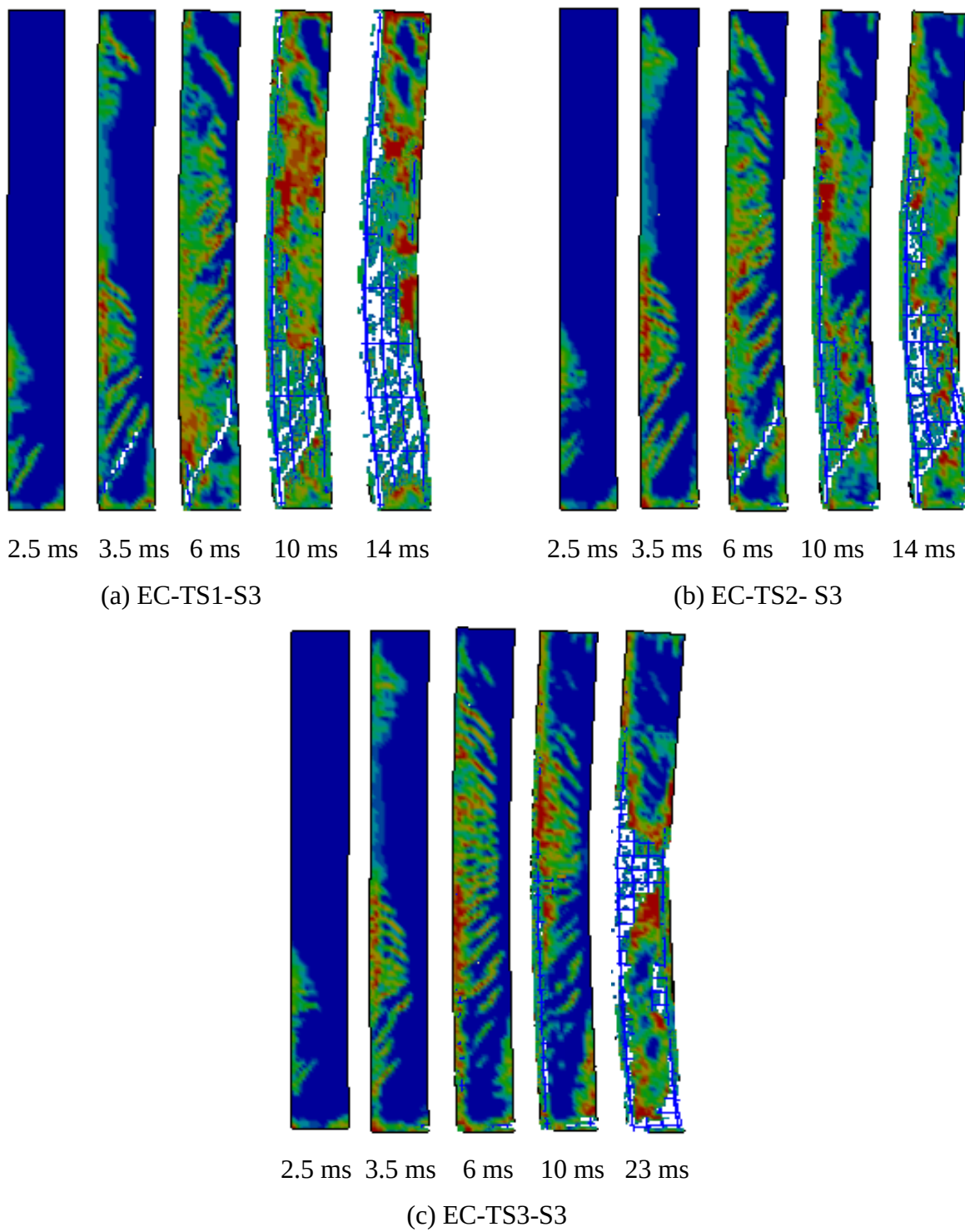


Figure A-26: Damage Evolution for EC Columns under S3 Loading

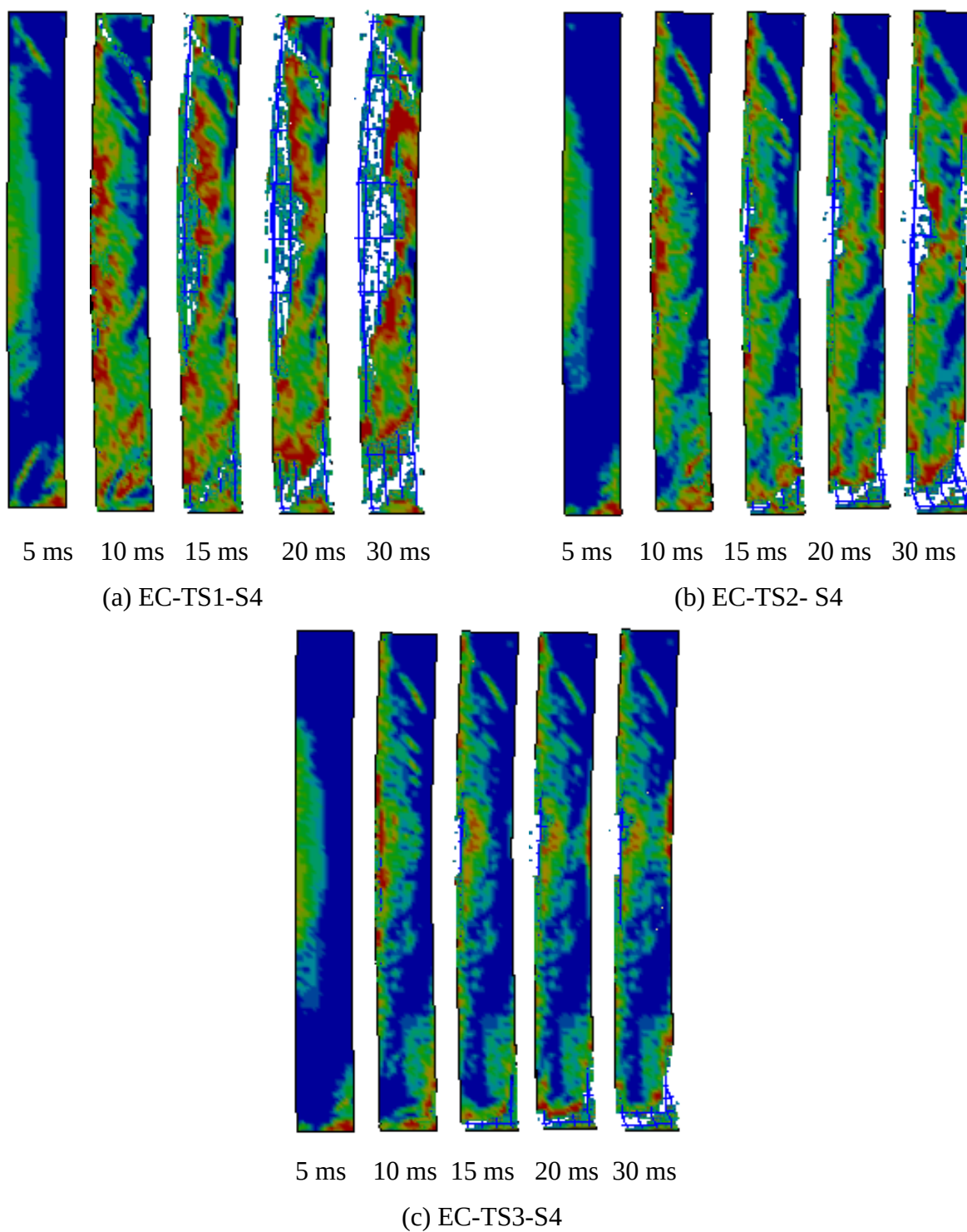


Figure A-27: Damage Evolution for EC Columns under S4 Loading

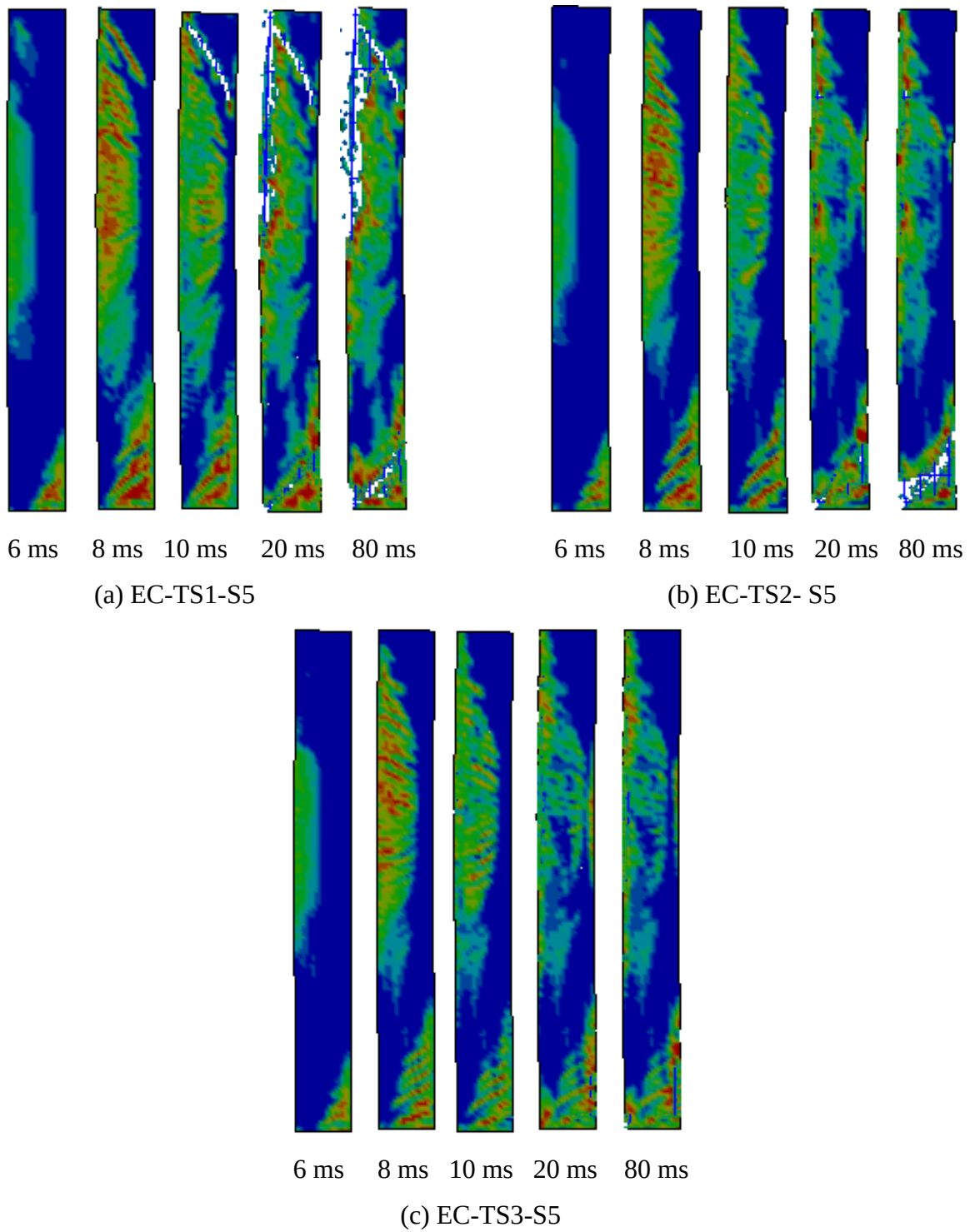


Figure A-28: Damage Evolution for EC Columns under S5 Loading

Appendix B : Blast and Axial Load Parameters

Table B-1: BP1 Blast Parameters

Height, (m)	Peak Reflected Pressure, (MPa)	Impulse, (MPa-ms)
0	220	39
0.5	70.5	12
1	16.8	2.54
1.5	7.24	1.22
2	3.3	0.784
2.5	2.02	0.784
3	1.38	0.764
3.5	0.935	0.656
4	0.712	0.599

Table B-2: BP2 Blast Parameters

Height, (m)	Peak Reflected Pressure, (MPa)	Impulse, (MPa-ms)
0	91.1	16
0.5	48.5	8.19
1	20.7	3.49
1.5	8.85	1.68
2	3.91	1.29
2.5	2.57	0.99
3	1.52	0.83
3.5	1.01	0.7
4	0.76	0.63

Table B-3: BP3 Blast Parameters

Height, (m)	Peak Reflected Pressure, (MPa)	Impulse, (MPa-ms)
0	34.4	3.92
0.5	27.3	5.51
1	14.5	3.12
1.5	8.01	1.99
2	4.92	1.56
2.5	2.85	1.21
3	1.85	1.01
3.5	1.29	0.88
4	0.88	0.73

Table B-4: BP4 Blast Parameters

Height, (m)	Peak Reflected Pressure, (MPa)	Impulse, (MPa-ms)
0	6.4	2.29
0.5	5.82	2.14
1	5.16	2.03
1.5	3.93	1.71
2	3	1.5
2.5	2.12	1.23
3	1.54	1.06
3.5	1.12	0.94
4	0.89	0.82

Table B-5: CS1 Blast Parameters

Height, (m)	Peak Reflected Pressure, (MPa)	Impulse, (MPa-ms)
0	102.3	37.6
0.5	91.2	33.9
1	71.6	26.4
1.5	31.7	11.2
2	21.6	7.58
2.5	12.6	4.5
3	8.6	3.27
3.5	6.8	2.84
4	4.57	2.17

Table B-6: CS2 Blast Parameters

Height, (m)	Peak Reflected Pressure, (MPa)	Impulse, (MPa-ms)
0	36.1	13.7
0.5	36.1	13.9
1	31.9	12.4
1.5	25.5	10.2
2	18.7	7.73
2.5	14.7	6.38
3	10.9	5.09
3.5	8.75	4.47
4	6.65	3.81

Table B-7: CS3 Blast Parameters

Height, (m)	Peak Reflected Pressure, (MPa)	Impulse, (MPa-ms)
0	16.3	8.06
0.5	16.6	8.23
1	15.5	7.87
1.5	13.6	7.08
2	11.2	6.07
2.5	9.57	5.49
3	7.84	4.82
3.5	6.68	4.49
4	5.45	4.01

Table B-8: CS4 Blast Parameters

Height, (m)	Peak Reflected Pressure, (MPa)	Impulse, (MPa-ms)
0	8.23	5.57
0.5	7.91	5.45
1	7.25	5.13
1.5	6.34	4.67
2	5.72	4.41
2.5	5.31	4.32
3	4.46	3.87
3.5	3.84	3.56
4	8.23	5.57

Table B-9: CS5 Blast Parameters

Height, (m)	Peak Reflected Pressure, (MPa)	Impulse, (MPa-ms)
0	4.39	4.08
0.5	4.44	4.12
1	4.33	4.07
1.5	4.06	3.91
2	3.9	3.86
2.5	3.45	3.54
3	3.27	3.5
3.5	2.87	3.23
4	2.57	3.04

Table B-10: S1 Blast Parameters

Height, (m)	Peak Reflected Pressure, (MPa)	Impulse, (MPa-ms)
0	184	79.9
0.5	161	72.4
1	95.7	41.3
1.5	55.7	22.9
2	35.3	13.8
2.5	22.2	8.4
3	16	6.02
3.5	11.3	4.39
4	7.58	3.17

Table B-11: S2 Blast Parameters

Height, (m)	Peak Reflected Pressure, (MPa)	Impulse, (MPa-ms)
0	55.7	24.7
0.5	54.9	24.5
1	47.9	21.4
1.5	41.5	18.8
2	29.6	13.4
2.5	24.4	11.4
3	19	9.14
3.5	14	7.08
4	11	5.88

Table B-12: S3 Blast Parameters

Height, (m)	Peak Reflected Pressure, (MPa)	Impulse, (MPa-ms)
0	27.4	14.3
0.5	27.6	14.4
1	25.6	13.5
1.5	21.9	11.7
2	19.1	10.5
2.5	17.1	9.69
3	14.4	8.53
3.5	11.5	7.19
4	9.56	6.23

Table B-13: S4 Blast Parameters

Height, (m)	Peak Reflected Pressure, (MPa)	Impulse, (MPa-ms)
0	14.5	9.41
0.5	14.6	9.54
1	13.8	9.15
1.5	13.4	9.01
2	12.2	8.44
2.5	10.6	7.56
3	9.14	7.01
3.5	8.01	6.27
4	7.39	6.14

Table B-14: S5 Blast Parameters

Height, (m)	Peak Reflected Pressure, (MPa)	Impulse, (MPa-ms)
0	8.57	7.15
0.5	8.12	6.8
1	8.31	7.04
1.5	7.67	6.61
2	7.19	6.36
2.5	6.86	6.25
3	6.28	5.93
3.5	5.56	5.47
4	4.98	5.13

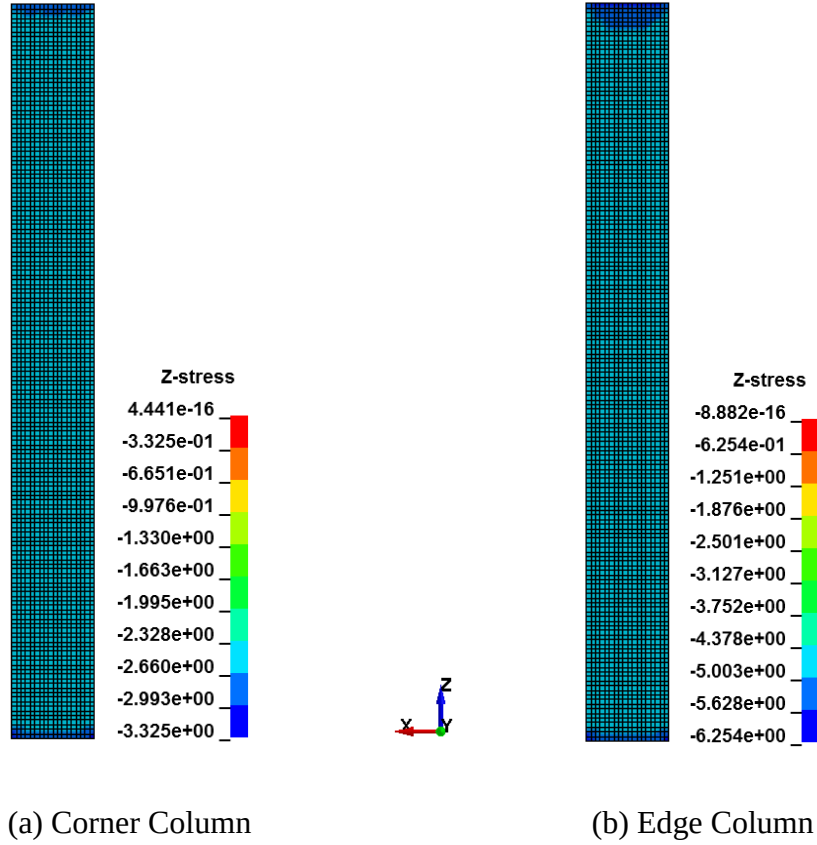


Figure B-1: Axial Load Effect on the Corner and Edge Columns*

* The z-stress values are along the length of the column and it is presented in MPa