

Improved Connections for Blast-Resistant Curtain Walls

by

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ABSTRACT

Curtain walls provide exterior façade to modern buildings. When subjected to blast shock waves, curtain walls may suffer significant damage, potentially causing serious injuries and casualties to building occupants. Protective films, laminated glass and strengthening of mullions and transoms are used to protect curtain wall components against blast loads. Limited research is available on blast protection of curtain wall components. On the other hand, connections of curtain wall mullions with the supporting substrate, as well as mullion-transom connections form potentially vulnerable locations under blast loads. Research on these connections is lacking in the literature. Therefore, a comprehensive research project has been undertaken in this thesis to address the behavior, analysis, and design of curtain wall connections, both between the mullions and supporting concrete slabs/beams and the mullions and transoms. The research project consists of three phases: i) Experimental research using the University of Ottawa Shock Tube as blast simulator, ii) Numerical investigation based on three-dimensional finite element method (FEM) using LS-DYNA, and iii) Non-linear dynamic analysis of curtain wall systems based on a single-degree-of-freedom (SDOF) to develop a connection design procedure.

The experimental phase consisted of tests of three full-size curtain walls mounted on steel HSS sections of the Shock Tube to investigate mullion-to-transom connections and nine single mullions connected to concrete beams to investigate mullion-to-concrete substrate connection. The single mullions either represented floor-to-floor mullions or continuous mullions over the supporting slab. They were connected to concrete beams (representing floor slabs) by means of brackets, which provided high degree of rotational restraints and full translational restraints or connected through damping materials (springs or HRD rubber pads), which allowed translational movements as they dampened the effects of the shock wave.

The numerical investigation involved FEM analysis and modeling of all the test specimens. The first step involved the validation of numerical models against test data. The analysis was then extended to conduct a parametric investigation to cover cases that have not

been covered in the experiments. This resulted in the investigation of six different design parameters used in connection design. The numerical outcomes illustrated the importance of blast effects on connection design parameters, support reactions, curtain wall response, force and stress distributions on curtain wall components.

The information gathered through experimental and numerical research on connection performance led to the formulation of a connection design procedure. Single-degree-of-freedom (SDOF) dynamic analysis technique was adopted to curtain wall analysis as a tool to compute connection design forces. Both the Uniform Facilities Criteria (UFC) charted solution (manual calculations) and two computer software developed at the University of Ottawa (RC-Blast and CW-Blast) were used to conduct SDOF analysis to validate the procedure against experimental and numerical results before they were recommended as design tools. Finally, the details of connection design are provided for different types of connections.

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NOTATIONS AND ACRONYMS

CW	-	Curtain wall
P_r	-	Peak reflected pressure
I_r	-	Peak reflected impulse
f_y	-	Yield strength of material
E_A	-	Modulus of elasticity of Aluminum
E_s	-	Modulus of elasticity of Steel
E_f	-	Modulus of elasticity of protective Film
K_s	-	Stiffness for simply supported beam
K_f	-	Stiffness for fixed supported beam
R_{y1}	-	Initial yield resistance
U_{y1}	-	Initial yield displacement
k_f	-	Fixed support stiffness
R_{y2}	-	Final yield resistance
U_{y2}	-	Final yield displacement
k_s	-	Simple support stiffness
R_m	-	Elastic yield resistance
U_y	-	Equivalent yield resistance
K_{eff}	-	Equivalent stiffness
K_{LM}	-	Load mass factor
T_s	-	Time period of structure
F_1	-	Blast load acting on tributary area
U_m	-	Maximum inelastic displacement
P_o	-	Membrane resistance

M	-	Moment capacity of section
I	-	Moment of Inertia
y	-	Maximum distance of tension fibers from neutral axis
m	-	Mass of structure
μ	-	the ratio of maximum deformation to the same type of deformation at yield
θ	-	Support rotation
DLF	-	Dynamic load factor
SDOF	-	Single Degree of Freedom
MDOF	-	Multiple Degrees of Freedom
FEA	-	Finite Element Analysis
DAS	-	Data Acquisition System
LVDT	-	Linear Variable Displacement Transducer
GAC	-	Global Affairs Canada
IGU	-	Insulated Glass Unit
TNT	-	Trinitrotoluene
UFC	-	Unified Facilities Criteria
FSI	-	Fluid Structure Interaction
SSCW	-	Steel Stud Curtain Wall
FE/BR	-	Forced Entry / Ballistic Resistance
ALE	-	Arbitrary Lagrangian Method
VE	-	Visco-elastic Damper
ADAS	-	Additional Damping and Stiffness Device
HSS	-	Hallow Steel Section

Part I

Introduction & State of the Art

CHAPTER 1. INTRODUCTION

1.1 General

The Global Affairs Canada (GAC) funded a research project to assess the performance of curtain walls under blast loads and to develop blast-resistant retrofit techniques to enhance Canada's preparedness and prevention capabilities against blast threats. The project includes research and development on retrofitting curtain wall components with an emphasis on strengthening mullion connections between horizontal mullions (transoms) and vertical mullions, as well as the connections of vertical mullions with the structural system of the building. The connections to buildings would consist of mullion-concrete slab connections between any two floors. One of the important objectives of the project was to generate research data on performance of blast-resistant curtain walls. The research consists of experimental and analytical components, which form the driving force for the current doctoral investigation.

1.2 Background

Curtain walls provide exterior façade to modern buildings. They are non-structural elements, essentially used as part of the building envelop, providing protection to the building against exterior environmental conditions, such as the effects of weather (hot or cold), and safety against intrusions while fulfilling other architectural functions. They often consist of lightweight materials usually designed and fabricated as architectural components. As such, their lateral load design is commonly dictated by prescribed wind loads. Recent blast incidences by maliciously intended acts of terror all-around the world demonstrated that these building components do not have the capacity to sustain extreme blast loads. Failure of building facades and window systems under blast loads results in injuries and loss of lives as evidenced by bombing occurrences near buildings, such as the 1995 Oklahoma City Bombing (Norville et al. 1999), the 2011 Oslo Regjeringskvartalet bombing, and the 2015 Mexico City Hospital gas explosion. It was observed that the majority of injuries and casualties in such instances are caused by flying debris and fragmentation of building components, including broken glass shards. Several types of glass assemblies are used to form building envelop, such as single or

multi-light punched windows, insulated glass units (IGU), monolithic or laminated glass units, and curtain walls. Punched windows are widely used in old and new residential, commercial, and public buildings. Curtain walls are commonly used in new and modern office, commercial, and public buildings.

A curtain wall is a thin vertical element usually consisting of aluminum-framed in-fill glass panes, metal panels, or thin stone. The framing is attached to the building structure and does not carry the floor or roof loads of the building. The gravity and wind loads acting on the curtain wall are transferred to the building structure, typically at the floor levels. Curtain walls can be classified by their method of fabrication and installation as stick systems, and unitized (also known as modular) systems. In the stick system, the curtain wall frame is constructed mainly on site with mullions and transoms supporting glass, spandrel panels, metal panels and brise-soleils, connected piece by piece. Each mullion is usually supported by the floor or perimeter beams. Stick curtain walling systems are versatile and allow for the integration of other systems, such as sliding doors and windows. They tend to be less specialised and can be built by all types of fabricators as they are not dependent on having a large factory. However, stick systems do require multiple steps to erect and seal the wall, which means more time is spent on site – an approximate rule of thumb would be that 70% of the work is carried out on site, with 30% in the factory. This incurs further costs, such as those associated with labour time. In the unitised system, on the other hand, the curtain wall is composed of large units that are assembled in the factory, shipped to the site, and erected on the building. Aluminium profiles are used to form the frame, which is normally one-storey high. Opening vents, glazing and infill panels are built into the units before being transported to site. As with stick construction, each unit is usually supported by the floor or perimeter beams. Typical units are 1.5 m to 2.0 m (5.0 ft to 6.0 ft) wide. For unitised curtain walling, about 30% of the work is done on site, while 70% is carried out in the factory. Another advantage of unitised systems is that, unlike stick systems, they do not transfer impact noises from floor to floor or horizontally from room to room. This is because the unitised panels are separated by linking gaskets. The use of different materials (i.e., gasket to aluminium), becomes acoustically excited to different

levels, thus creating full separation at the junctions between units. In addition, point fixed glazing systems (spider glazing) can be used to create a near transparent glazed façade or sloped roof with a flush glass appearance. Point fixed glazing systems are manufactured from sheets of toughened glass and/or toughened laminated glass. Manufactured to the required design with either a selection of laminated glass structural fins or by a series of either 304 or 316 stainless steel spider fittings and stainless-steel bolts, which are generally supported by a secondary steel structure. The point fixed structural glass wall can be designed to suit both single-glazed and double-glazed requirements. Normal detailing will require the glass to terminate at the building interface by a series of stainless U-shaped steel channels.

1.3 Problem Definition

Curtain walls have traditionally been considered architectural elements, whose lateral design is commonly dictated by prescribed wind loads. However, the increase in terrorist actions directed towards civilian targets has raised concerns about the meaning of a ‘low-risk’ structure and has encouraged designers and building owners to assess the blast resistance of curtain walls. The behaviour of blast-resistant curtain wall has not been deeply understood and comprehensive research is needed to elaborate on the effect of high impulse loads on full-size curtain wall and their connections. Although there has been some research on window glazing and overall performance of blast-resistant window systems, the behaviour of large walls like curtain walls, especially the performance of their mullions has not been well-established. It is uncertain how the mullion forces would be transferred to the structural members like slabs, and if the existing connections are capable of resisting the applied loads. It is paramount to translate the information generated into a reliable design procedure. Likewise, for existing buildings it is unclear how the resistance of curtain wall systems is characterized by different retrofit approaches. Challenges associated with the use of different retrofitting techniques with a combination of different materials, like aluminum and steel, on vertical mullions and horizontal transoms/stick joints, with significantly different mechanical properties augments the complexity of the problem. Further complexities arise due to nonlinear dynamic response of curtain wall systems having different characteristics for connecting vertical and horizontal

mullions, screw patterns on retrofit plates, internal U-angle connections, glass panes and window frames. The post-breakage performance of glazed panes with the resulting membrane action compounds the problem, creating additional questions in ascertaining applied forces. Lack of sufficient knowledge on connections between curtain wall sections and structural members (beams/slabs) is compensated by empirical design procedures which can be excessively conservative, while sometimes resulting in unsafe practices. Therefore, these types of connections merit significant attention since any connection failure would result in overall failure of the curtain wall. In addition, the connection design will benefit from innovative ideas, such as incorporating damper materials at the connections. Research gaps exist in the current body of knowledge specifically on blast resistant curtain wall connections. It is clear that both experimental and analytical research is needed to generate design information and design procedures for curtain wall connections.

1.4 Objective and Scope

The primary objective of the research project is to develop analysis and design information for blast-resistant curtain wall connections. The connections that are investigated include those that connect horizontal mullions to vertical mullions, as well as vertical mullions to concrete building slabs/beams. . The objective includes generation of much needed test data on performance of blast-resistant curtain wall connection details, and analysis techniques suitable for use in design of new curtain walls, as well as retrofitting existing curtain walls.

The above objectives are planned to be achieved by following the steps given below, which form the scope of the investigation:

- Review to existing literature and the earlier phases of the same overall research program at the University of Ottawa.
- Construction, instrumentation, and testing of 12 full-size curtain wall components under simulated blast loading using the University of Ottawa Shock Tube. Three of the specimens consist of full-size curtain wall segments, consisting of mullions supporting steel sheets as representatives of glass panes. The remaining 9 specimens consist of full-size single mullions with different connection details. Four of the mullions are to be tested

to investigate rigid bolted connections to concrete slabs, and the remaining 5 are to be tested with different supplementary damping materials at the connections between the mullions and the supporting concrete substrate.

- Evaluation of test data and the assessment of mullion-to-transom connection retrofit performance, as well as the specific behavior of vertical and horizontal connections, while also investigating the effects of test parameters.
- Selection of a general-purpose computational fluid dynamics (CFD) and finite element (FE) software LS-DYNA for numerical analysis of curtain walls with emphasis on the effect of mullion retrofitting on the behavior of curtain walls, effect of connection retrofitting by steel brackets, and the bracket arrangements at the connections.
- Construction of analytical models for curtain walls using the available modelling techniques in LS-DYNA and the validation of the analytical models against test data generated during the experimental phase of the investigation.
- Analytical parametric investigation using LS_DYNA to analyze the importance of design parameters comprising of selected connection types and details between vertical and horizontal mullions, as well as between mullions and the supporting structure for various blast threat levels.
- Development of a non-linear SDOF assessment approach for blast-resistant curtain wall systems and the construction of curtain wall resistance functions to compute mid-span displacements and support reactions by using UFC-charted solution, RC-Blast software, and CW-Blast software.
- Development of connection design procedures for mullion-to-transom and mullion-to-concrete substrate connections.
- Presentation of results in the form of a manuscript-based thesis.

1.5 Research Methodology

The proposed research consists of experimental and analytical components. The experimental component consists of three phases, i) tests of curtain walls with emphasis on mullion-to-transom connections, with and without the hardening of the mullions/transoms, ii) tests of individual mullions with emphasis on connections to structural substrates using reinforced concrete beams/slabs as common building prototype, and iii) tests of vertical mullions connected to structural substrates by means of supplementary damping materials. The analytical component consists of two phases; i) FEM analysis using software LS-DYNA to develop numerical models validated against test data, followed by a numerical parametric investigation to examine the effects of curtain wall parameters on mullion-to-transom and mullion-to-substrate connection designs, ii) SDOF dynamic analysis of individual mullions and transoms for correlation with test data and LS-DYNA analysis results, followed by the development of a connection design procedure based on the SDOF dynamic analysis results.

In the first phase of the experimental component (Phase I), three full size curtain wall specimens were assembled with retrofitted internal connections between the vertical mullions and horizontal transoms. Different connection strengthening techniques were used and the specimens had either unretrofitted or retrofitted mullions and transoms, tested under various levels of uniform blast loads applied on the exposed wall surface. These tests were followed by 4 single mullion tests (Phase II) to assess the performance of their connections to concrete slabs. Blast loads were applied through a load application device that would collect the pressure induced at the end of the shock tube and apply it uniformly to the flange surface of the mullions along their heights. These tests consisted of 4 rigidly bolted connections to concrete slabs. Phase III tests included 5 connections equipped with damping materials (either XRD rubber or steel springs) placed at the mullion-concrete substrate connections. The response limits given in CSA-S850 were considered to define the failure of specimens. The specimens were instrumented to measure deflections and strains. High-speed cameras were used to record the position of the curtain wall components during response.

The numerical investigation involved FEM modelling and analysis of curtain walls and single mullions using software LS-DYNA. The FEM models were first validated against the test data collected in the experimental phase. The models were then used to conduct an analytical parametric investigation. The parameters in this research included connection retrofit techniques, mullion rigidity, type of supports, connection fixity level, replacing the glass and film with steel sheet, the effectiveness of side brackets, bracket locations, bracket thickness, improving the level of protection by dampers, and the applied blast pressure-impulse combination. The results are expected to provide sufficient information to develop retrofit design procedures for connections between horizontal and vertical mullions, as well as the connections between vertical mullions and structural slabs/beams of buildings.

The analytical research is expanded to develop a simplified SDOF method of analysis for curtain wall systems. The analysis procedure is expected to provide strength demands for each curtain wall component (glass panes, protective film/laminate, vertical and horizontal mullions, connections between the mullions and their anchorage to structural slabs/beams). The support reactions obtained from the analysis are used to develop a connection design procedure.

1.6 Organization of Thesis

This thesis consists of six chapters as presented below:

- Chapter 1 presents research needs, background information, research objectives and scope, as well as a summary of research methodology proposed to be followed in three main phases of experimental research and analytical investigation.
- Chapter 2 provides review of available literature and previous research on blast-resistant curtain walls, both experimental and analytical.
- Chapters 3 to 5 are presented in manuscript-based thesis format
 - Chapter 3 (Article 1) "**Experimental Investigation of Improved and Retrofitted Connections for Blast-Resistant Curtain Walls**". This chapter presents an overview of the experimental research with further details included in Appendices A of the thesis.

- Chapter 4 (Article 2) "**Numerical Investigation of Improved and Retrofitted Connections for Blast-Resistant Curtain Walls**". This chapter provides a comprehensive numerical analysis including result validation and parametric study using LS-DYNA software and the FEM modeling implemented, with further details included in Appendix B of the thesis
- Chapter 5 (Article 3) "**Analysis and Design of Curtain Wall Connections**". This chapter elaborates on non-linear SDOF analysis for curtain walls and single mullions considering the effect of retrofit plates, level of fixity, and inserted damper stiffness on non-linear response. The analysis included the construction of mullion resistance functions, transom resistance function, UFC-charted solution, displacement time history, and transom dynamic support loads. UFC-charted solution, RC-Blast, and CW-Blast software were used with further details included in Appendix C and D of this thesis. Analysis outcome provides mullion and transom support forces for use in connection design. Appendix E presents the comparison of mullion deflection at mid-height as well as the relevant PI diagrams.
- Chapter 6 provides conclusions and a summary of findings of the research project.

CHAPTER 2. STATE OF THE ART

2.1 General

Curtain walls and large multi-pane office windows are quite common in modern construction, not only in new buildings but also for rehabilitated and remodeled existing buildings. While these non-structural elements perform adequately under gravity and wind loads, their performance under extreme blast loads is less than adequate as evidenced by the catastrophic failures of such elements during previous bomb attacks and accidental explosions. Previous research on the behaviour of curtain walls is restricted to individual mullions and small-size discontinuous curtain wall segments under static loads. This resulted in limited understanding of large size and continuous curtain walls under blast loads. Moreover, there is limited data available on hardening of curtain wall mullions and their connections. The limited number of books that have recently become available on blast-resistant buildings provide insufficient information on curtain wall design and performance, with even more limited guidance provided for retrofit and hardening of curtain wall components/connections (Mays et al. 2003; Bangash et al. 2006; Krauthammer 2008; Dussenberry 2010; Uddin 2010). This chapter provides a review of previous research on blast resistant curtain wall systems. It consists of an overview of recent research at the University of Ottawa as part of the earlier phases of the current research program, as well as experimental and analytical research conducted by others.

2.2 Research at the University of Ottawa

2.2.1 Curtain Wall Tests and Analysis

A number of large-scale curtain wall specimens, consisting of two double-pane glass panels separated by a horizontal mullion and supported by two vertical mullions on either side and two horizontal mullions at top and bottom were tested using the University of Ottawa Shock Tube (Saatcioglu et al. 2017a). The tests were conducted under different blast pressure-impulse combinations. They were representative of an office building curtain wall consisting of two insulated glass units (IGU). The specimens were manufactured to simulate the actual curtain wall components used in the field. The outer pane of each IGU had a 4.7 mm thick

tempered glass, which was not protected against blast pressures. The inner pane consisted of a laminated glass, having two 4.0 mm tempered glass with a 3.0 mm laminate in between. Figure 2.1 shows the details of the curtain wall specimens tested. The prototype building had a daylight film application on the inner pane consisting of 7 mil protective film. This was applied on the inner pane prior to testing. Because the blast load applied was anticipated to exceed the capacity of the curtain wall, it was protected by a Cascade coil catchment system placed at 300 mm on the protected side to catch potential debris generated. The specimen was subjected to reflected pressure and impulse combination of 85.5 kPa and 1338 kPa-ms. Figure 2.2 shows the damage occurred in the curtain wall at the end of the test. The horizontal transom was completely detached, and debris generated were partially controlled by the catchment system.

The performance of IGU panes, and especially the behavior of horizontal mullion (transom) during Test 1 indicated that a blast retrofit would improve curtain wall performance. Therefore, the curtain wall specimen used in the second test was retrofitted. The retrofit to the second specimen consisted of improvements in the connection between the horizontal and vertical mullions, while also providing additional window security film, mechanically anchored to the mullions (mechanical glazing). The horizontal mullion was attached to the vertical mullions by means of two aluminum angles of 6.35 mm thickness and 63.5 mm flange width, bolted on the vertical mullions using 15.9 mm through bolts. This is illustrated in Figure 2.3. The interior laminated IGU panes were mechanically glazed using 7 mil security films. One ply of security film was applied to each of the two interior panes. The film was anchored to the mullions by means of 25 mm square aluminum hollow sections, bolted on to the mullions. The test was conducted under reflected pressure and impulse combination of 85.6 kPa and 1923 kPa-ms. This level of blast load caused the failure of vertical mullions as illustrated in Figure 2.4. The failure of the vertical mullions was attributed to the holes drilled into them at the critical maximum moment locations to bolt the aluminum angles which were placed to increase the connection strength with the anticipation that the detachment of the horizontal mullion observed in the previous test would not occur.

The third specimen involved a companion curtain wall with improved connections of the mullions by using the same aluminum angles, except this time glued on the vertical mullion by a marine epoxy adhesive known as “cold weld,” rather than bolted mechanical connections (Saatcioglu et al. 2017b). In addition, the interior glass pane was retrofitted by adding an additional 7-mil protective film, mechanically glazed. The film was anchored to the vertical mullions only, promoting one-way action by means of 25 mm square aluminum hollow sections that were bolted to the mullions with 6.35 mm diameter, 100 mm long, drill-through bolts. The bolts were provided at a spacing of 100 mm. Figure 2.5 illustrates the retrofit method implemented. The curtain wall was subjected to a blast load consisting of 80 kPa reflected pressure and 448 kPa-ms reflected impulse. This retrofit technique resulted in successful performance of the curtain wall as depicted in Figure 2.6. Though the horizontal mullion bent and developed permanent deflection, the curtain wall remained intact, without failure.

The fourth and the last test in this series of curtain wall tests included an enhanced retrofit strategy for higher blast loads, which included strengthening of the mullions. This test was labelled as Test 4. The retrofit for the curtain wall consisted of strengthening both vertical mullions and the middle horizontal mullion by epoxy gluing two 3.18 mm aluminum plates on both sides of each mullion (Saatcioglu et al. 2017b). This method of strengthening the vertical mullions resulted in an increase of a factor of 1.5 in the elastic flexural capacity of vertical mullions (assuming perfect bond between the added plates and the existing mullions). This increase in strength was believed to provide additional capacity against higher threat levels while also account for the fact that in the prototype building the blast pressures would be applied to glass units on both sides of vertical mullions, unlike the test condition. Furthermore, the connections between the middle horizontal mullion and the vertical mullions on both sides were enhanced by means of two aluminum angles at each end, epoxy glued and later also screwed using three 9.5 mm self-tapping screws to the mullions, one at the top and the other at the bottom of each connection. Figures 2.7 illustrate the retrofit technique used. The curtain wall was further retrofitted by mechanically glazing the inner panes. This involved the installation of an additional ply of 7-mil protective film using aluminum angles along the sides.

The angles were mechanically anchored to vertical mullions to secure the film for one-way action using 6.35 mm self-tapping screws. This is illustrated in Figure 2.8. This curtain wall was tested under 84 kPa reflected pressure and 1630 MPa reflected impulse. This reflected impulse was approximately a factor of 4.0 higher than the previous impulses applied. It was observed that the curtain wall did not perform well during Test 4. The horizontal mullion flew in towards the witness panel along with all four panes of glass. While the vertical mullions remained connected to the Shock Tube frame, three of the four vertical aluminum plates used for strengthening the vertical mullions became loose and detached, indicating that the chemical bond between the existing aluminum mullions and the newly added plates was not sufficient to develop full composite action. The angles used to connect the horizontal mullion to the vertical mullions were detached from the vertical mullions, flying in along with the horizontal mullion. The epoxy glue and the screws used to secure the angles to the vertical mullions sheared off at the ends. Large mass of glass shards penetrated towards the witness panel along with other curtain wall components. The mechanical glazing angles were detached from the left vertical mullion and flew in, whereas those on the right mullion remained connected. Figure 2.9 shows the damage inflicted on the curtain wall.

The behaviour of the curtain wall specimens tested was analyzed using a single-degree-of freedom (SDOF) analysis (Saatcioglu 2019). Excellent agreement was obtained between the test data and the analysis results. The test specimens had two vertical mullions resisting blast loads applied on a single panel between the two vertical mullions. However, in field applications the blast pressure would be applied to panels on both sides of each vertical mullion with twice the tributary area. This case was analytically investigated using the model shown in Figure 2.10. The results indicated failure when the mullions were simply supported and developed excessive inelastic deformations when they were fixed. Both cases improved when the mullions were hardened with L-shaped steel plates. The results of SDOF analyses with and without the hardening of the mullions are shown in Figure 2.11.

In addition to the four tests reported above, samples of the curtain walls used in the Singapore Airport Terminal were prepared and tested using the University of Ottawa Shock

Tube (Saatcioglu 2016). Two laminated glass panels, each 2.1 m high and 1.0 m wide, were tested side by side under the same blast shock wave. Each panel consisted of a multi-layered laminated glass having a total panel thickness of 26 mm as illustrated in Figure 2.12. The panels were anchored to top and bottom steel frames, which represented concrete slabs of the prototype building, by extending the interlayers into the supports (too be cast in concrete in the prototype building). The reflected pressure and impulse applied on the curtain wall system was 93 kPa and 1060 kPa-ms, respectively, with a positive phase duration of 23 msec. Both panels remained intact during the test as illustrated in Figure 2.13, proving the effectiveness of the unique anchorage system employed. They both reached a maximum displacement of about 660 mm. Immediately after the test the residual maximum deflection of the north panel was 355 mm and that of the south panel was 200 mm.

2.2.2 Tests of Curtain Wall Mullions

The previous set of tests reported in the preceding section showed vulnerabilities of curtain wall mullions when the blast loads exceeded their capacity. This is particularly true for vertical mullions which transfer the blast loads to the supporting building. Therefore, the next phase of research consisted of investigating the hardening options for vertical mullions by means of steel plates. Tests were conducted using the University of Ottawa Shock Tube to develop hardening options for mullions using steel plates of different configurations (Saatcioglu et al. 2020). Three different sizes of aluminum mullions were considered in the test program, consisting of; i) 6 in x 2 in x 1/8 in (152 mm x 51 mm x 3.0 mm); ii) 6 in x 2 in x 3/16 in (152 mm x 51 mm x 4.8 mm) and iii) 5 in x 2 in x 1/8 in (127 mm 51 mm x 3.0 mm). Each of these mullions was subjected to progressively increasing blast loads until a significant inelastic deformation was observed, marking the load capacity of the mullions. Three companion mullions of the same size were hardened using three different steel plate/section assemblies. They consisted of i) steel side plates, ii) steel L-shaped plates covering the web and the tension flange, and iii) addition of a square steel HSS profile attached to the tension flange with steel side plates. All the steel pieces had 3.0 mm wall thickness. Figure 2.14 provides cross-sectional views of all the mullions tested. Figure 2.15 illustrates the mullions with the

three configurations of steel plates before and after each test. The results are shown in Figure 2.16 and indicate significant improvements in strength and stiffness of aluminum mullions when hardened with steel plates.

2.3 Literature Review of Blast-Resistant Curtain Wall Systems Reported by Other Researchers

The following sections present previous research conducted by other researchers, in addition to those conducted at the University of Ottawa, reported in the preceding section.

2.3.1 Kirk, Carrie and Peter (2016)

The authors investigated the combined performance of glass and mullion response for conventional curtain walls subjected to blast loads. Five shock tube tests were conducted under increasing blast pressure using a full-scale curtain wall specimen (Test 1 was repeated twice). The test curtain wall is illustrated in Fig 2.17. The blast loads covered a range of responses from no glass breakage up to glass fracture with significant Polyvinyl Butyral (PVB) and mullion response. The curtain wall consisted of mullions and insulated glass units (IGUs) connected along top and bottom supports to simulate one-way bending response. The IGUs had a nominal dimension of 1.68 m wide $\times$ 3.15 m tall. A 219 mm deep (178mm behind the IGU), 6063-T6 aluminum unit frame was selected to support the units. Mullion cross sections are shown in Figure 2.18. The reflected pressure was gradually increased for each test from low to high. The measured blast impulse for each test was 152, 152, 262, 407, 531, and 738 kPa-ms, respectively. In addition to the test program, FEM analysis was conducted using software LS-DYNA. The FEM model and the mullion details are illustrated in Figure 2.19. The breakage of glass is depicted in Figure 2.20. For numerical analysis, design-level simplification was deemed appropriate and Timoshenko-type beam elements were used to represent the aluminum sections.

The results indicated a coupled system response, where the maximum glazing response was observed at roughly the same time as the maximum mullion displacement. The specimen with ordinary insulating glass developed variety of responses from no-breakage to glass breakage with PVB response. Furthermore, the maximum window response (pre-bite failure) happened near the maximum mullion response. In the initial tests (Test 1 and 2), the vertical

mullions and jambs showed elastic reactions (with a parabolic deformed shape) and showed no yielding or residual displacement because of the low magnitude of the applied blast loads. Between Tests 3 and 5 the vertical mullions and jambs responded elastically (parabolic deformed shape), which subsequently yielded and developed web buckling. A plastic hinge was observed at mid-span during Test 5. The researchers reported that during the tests, the internal horizontal mullion, as well as sill moved as rigid bodies along with the system and had slight local deflection without any yielding. The numerical results illustrated that simplified modeling strategy for aluminum mullions is viable and computationally efficient approach. Comparison of all no-break simulation results with mullion laser gauge data showed the ability of the current mullion modeling strategy to precisely distinguish peak mullion response. Likewise, the mullion displacement was exactly predicted for Tests 3 and 4, which had moderate blast pressures.

2.3.2 Benjamin, David, Keanini (2013)

A comprehensive research project was carried out at the University of North Carolina Charlotte Structures Laboratory. The project included a series of five full-scale curtain wall specimens and were tested under blast loads. The specimen before and after the test are presented in Fig 2.21. The primary objective of the tests was to provide comprehensive resistance curve, inclusive of the nonlinear behaviour of the specimens up to ultimate failure. The authors defined the ultimate failure as mullion fracture or conspicuous breach of the curtain wall system when observed as the protective barrier between building occupants and the external blast load. It was indicated that the blast energy was absorbed by the curtain wall system over three different restriction stages, including: i) cracking of glass, ii) yielding of mullions and transoms, iii) and fracture/breach of the system (ultimate failure). Moreover, authors investigated the ultimate energy absorption capability, the recoverable linear strain energy and the nonlinear energy that resulted in the formation of damage mechanisms and maximum mullion end rotations for reliable and economical design of blast resistant curtain walls. A conventional curtain wall was selected for investigation, with use of laminated glass lites that were structurally glazed (wet-glazed) to a conventional framing system with structural

silicone sealant – to enhance performance to extreme out-of-plane loading. Subsequently, a simplified methodology was provided for analytically predicting curtain wall resistance performances that could be used for nonlinear structural dynamic behaviour under an impulsive loading. This study utilized three curtain wall systems including: System 1 with dry glazed monolithic glass lites, System 2 with dry-glazed laminated glass lites in place of the monolithic glass lites, System 3 with wet-glazes laminated glass lites. Researchers instrumented the specimens with 13 strain gages (SG1–SG13) and 10 wire potentiometers (D1–D10) to measure deflections. In the end, test results were presented by way of representative local load-deflection and load-strain resistance curves. Global load-deflection resistance curves were created from the local load-deflection resistance curves so that energy absorption capacities could be computed. The comparison of experiment and analytical resistant curves is shown in the Fig 2.22. The results indicated that the addition of laminated glass (System 2) resulted in improved performance relative to fully conventional System 1. In fact, it developed over four times the ultimate energy absorption capacity, which resulted in more than twice the capacity of the conventional system. The application of wet glazing of the laminated glass lites produced further enhancement of the system. Impressively, System 3 demonstrated roughly 14 times the ultimate energy absorption capacity and nearly four times the blast resistance of the conventional system.

2.3.3 Anderson and Maj. Dov Dover (2003)

The authors summarized some of the early pioneering work at US Airforce Research Laboratories (AFRL) involving a new damping system for sealed blast-resistant windows for protection against terrorist threats. They also included in their review recent developments in the evolution of "membrane" windows. It was indicated that the AFRL windows can resist blast pressures a full order of magnitude greater than typical blast windows and remain sealed against subsequent bio-chem attack. Several factors played significant roles in increasing the blast resistance of windows, including: i) polymer membranes, ii) damping chamber, and iii) special anchoring methodologies. Two significant concepts: "The damping chamber" and "The membrane" were scrutinized in the study with an emphasis on substituting an otherwise rigid

structural element with a structural element of similar structural capability. They also used the damping chamber as an air gap between the front and back panels of a double panel window that enhanced blast resistance by damping and mitigating the blast waves. The “Flex” and “Super-Flex” window systems were developed as part of the research project. The schematic of the Flex window is illustrated in Figure 2.23, which is the same as Super-Flex except for the film anchoring system. The blast tests were performed on nominal peak reflected pressure of 690 kPa and showed excellent performance. The improvement in performance was attributed to the damping chambers provided. The overall strength capacity of the anchorage system was increased by adding elastic deformation to the potential of the window system. The Flex and Super-Flex systems were reported to be sufficient to prevent spreading of the glass shards inside the structure even after the second blast. The researchers highlighted that the windows might lose their lucidity because of a destructive blast circumstance, nevertheless, they remain totally clear in any other situations.

2.3.4 Adam, David, Matthew, and Hongbing (2015)

The authors provided an overview of the experimental and numerical investigations of glass curtain walls subjected to the low-level blast loads. In this research, three full-scale curtain walls were tested in the open arena. The explosion was created by the detonation of high explosives and the specimen was subjected to a low-level blast. Authors performed low-level blast loads that were representative of a satchel charge at short-to-moderate standoffs, giving in overpressures which were adjacent to the acoustic limit (peak reflected overpressures are roughly twice the peak incident overpressures). All three specimens were nominally 3.66 m (12 ft) tall, spanning vertically, and 2.44 m (8 ft) wide. Four nominal 1.22 m × 1.83 m (4 ft × 6 ft) laminated glass lites were attached to the aluminum extrusions with a 12.7 mm × 6.4 mm (0.50 in × 0.25 in). Most of the blast energy was reduced through the cracking and the making large deformations of the laminated glass lites which is shown in Fig 2.24. During the loading, the laminated glass lites firstly demonstrated plate-bending behavior up to fracture. After the cracking, the glass transitioned into membrane behavior with the tensioned PVB interlayer giving the majority of the remaining lateral stiffness. Permanent deformations of the

bottom halves of the lower glass lites were also observed. Blast test results showed the effectiveness of utilizing laminated (annealed) glass lites connected to conventional mullions with structural silicone glue to resist low-level blast loads. This study was complimented by finite element modeling which both horizontal and vertical aluminum mullions were modeled using Hughes–Liu beams with the hollow box-shape cross-section option. The dynamic responses of the glass and the mullions were utilized to validate the FE analysis. After validation of the FE simulations with the open-arena blast test, it was observed that the deflected profile of the glass predicted by the FE model indicated excellent agreement with the actual deflected profile captured by high-speed video.

To conclude, the tests demonstrated that curtain wall system provided adequate protection, almost corresponding to the medium level of protection established by the department of defense, Washington (minimum antiterrorism standards for buildings), when subjected to blast loads from a satchel-sized charge. Furthermore, the numerical simulation was utilized to assess the reliability of an ordinary nonlinear design expression to approximate the blast resistance of a curtain wall system. Besides, the tear-out was the controlling limit state when the curtain wall was impulsively loaded. Tear-out lengths from the FE simulations were established for applied impulses and the FE predicted failure impulse. It was compared to the maximum impulse predicted by the design expectation.

2.3.5 Timothy, Kenneth and John (2015)

This study was a cooperative project between Karagozian & Case Inc. (K&C), developed curtainwall technology for US government infrastructure overseas capable of resisting the loads predicted from massive explosive circumstances such as Vehicle-Borne Improvised Explosive Device (VBIED) at adjacent proximity. The analysis conducted by High Fidelity Physics Based (HFPB) calculations, which was comprised of Computational Structural Dynamics (CSD), and Computational Fluid Dynamics (CFD) methods. A complicated full-scale explosive was done to validate the simulation and analysis. The Steel Stud Curtain Wall (SSCW) test specimens was included of a nominal 3.7 m floor to floor span, and was installed on a steel framed reaction structure. In addition, for the window curtain wall configuration,

ballistic stops were needed spanning between the window frame and curtain wall connection to provide continuity of the BR at this interface. In total, approximately 60 mm glazing thickness was considered. The pretest external view as well as the effect of positive and negative of blast is shown in Fig 2.25.

The author indicated that steel stud curtain wall design required effective and practical design to successfully resist against the blast hazards. So, new retrofitting techniques are vital to enhance the level of blast resistant curtain wall. Besides, the structural response of the SSCW can be achieved and captured key details of structural response and air-blast loading, as well as high speed video coverage of the exterior and interior of the structure. Basically, the SSCW performed well with the basic mullion framing, and cassettes reacted as they were expected (within intended design restrictions). The approximately 7 kPa internal pressures was measured by interior blast pressures gauges. In terms of comparing the test results and computed results, the analytical models properly predicted the real blast test. Therefore, it can present reliable outcomes and thus can provide a reliable means to help in the adaption of this design to an actual building.

2.3.6 Chiara and Claudio (2012)

The researchers developed a viscoelastic device to protect curtain walls against blast shock waves through a numerical investigation. The curtain wall that was simulated for numerical analysis consisted of a laminated glass panel spanning floor to floor, a continuous bead of structural silicone sealant, a split screw spline frame and four rigid brackets. The idealized connection with glass panes and a scheme of possible viscoelastic devices are shown in the Fig 2.26. Initially, a linear elastic FE-model (M01) was conducted to examine dynamic stresses and deformations under the applied blast loads by considering geometrical nonlinearities. Because of the expected cracking of the glass lites a second model (M02) was also built incorporating the effects of glass cracking. In addition, a new model (M03) was proposed that contained viscoelastic (VE) devices at the frame corners to mitigate the effects of blast loads on main components of the curtain wall, especially the bead of silicone. Authors managed to have the mullions fixed to the structural substrate by means of bolted steel brackets.

Therefore, only rotations were permitted to the frame corners of the modular unit. The idealized modular unit and scheme of a conventional bracket is illustrated in Fig 2.27.

In conclusion, the M01 FE-model could merely capture general curtain wall behaviors because the cracking of glass lites and the plastification of aluminum mullions affected dynamic response. Authors indicated that the M02 model provided acceptable results for capturing the cracking of glass panes and mullion yielding realistically. They emphasized the importance of structural silicone sealant on response, because silicone is affected by a sudden and noticeable rise of shear stresses soon after the application of blast loads. In addition, it was indicated that the vertical mullions should be designed such that the axial stresses in them should be minimized too ensure the stability of the glazing system. They concluded that the use of viscoelastic devices can lower maximum stresses in glass lites, silicone joints and mullions, reducing deflections and support reactions transferred to structural substrates.

2.4 Standards and Functional Practice Guidelines

2.4.1 Testing Standards

2.4.1.1 ISO 16933 and ISO 16934 (2016):

The International Organization for Standardization (ISO) provided two bomb blast standards for blast testing, which are known as ISO 16933 and ISO 16934. ISO 16933 defines a structured procedure to determine the air-blast resistance of glazing and sets forth the required apparatus, procedures, specimens, other requirements and guidelines for conducting arena air-blast tests of security glazing. Table 2.1 presents nominal charge sizes and standoff distances for typical test frames and large facade for the satchel-bomb classifications. In addition, nominal charge sizes and standoff distances calculated to equate to the blast values for the vehicle-bomb classifications are illustrated in Table 2.2. Likewise, ISO 16933 asserts criteria for assessing the level of damage and threat to the glazing and the hazard consequences to the area behind the glazing. This evaluation is match with Tables 2.1 and 2.2. The hazard rating criteria is shown in Table 2.3. In a similar way to ISO 16933, the ISO 16934 tabulates classification for blast-resistant glazing systems according to shock tube test. Furthermore, the ISO 16934 asserts a shock tube test method and classification requirements for explosion

pressure resistant glazing, including glazing fabricated from glass, plastic, glass-clad plastics, laminated glass, glass/plastic glazing materials, and film-backed. The shock tube blast pressure and impulse presented in Table 2.4.

2.4.1.2 BS EN 13123-1: Standard (2001):

This European Standard is applicable to blast overpressure generated in a shock tube test facility used to simulate a high explosive detonation on windows, doors and shutters, complete with their frames and infills, for use in both internal and external locations in buildings. This European Standard concerns a method of test against blast waves generated by using a shock tube facility to simulate a high explosive detonation in the order of 100 kg to 2 500 kg TNT at distances from about 35 m to 50 m. It gives no information on the explosion resistance capacity of the wall or other surrounding structure.

2.4.1.3 ISO/FDIS 22497 Standard:

ISO (the International Organization for Standardization) is a worldwide federation of national standards bodies (ISO member bodies). ISO/FDIS 22497 states the relevant requirements about doors, windows and curtain walling against impulse. Standardization in the field of windows and curtain wall including hardware, manufactured from any good-quality material covering the specific performance requirements, terminology, manufacturing sizes and dimensions, and methods of test.

2.4.1.4 CEN - EN 13119 Standard (2007, 2020):

This European standard was released on 2007 and describes terminology used in documents, drawings, specifications etc., when referring to the detailed elements of curtain walling and provides a proper, though not total, list of regular terms. It does not set out to repeat those physical definitions properly included within individual curtain walling standards related to performance requirements and associated test methods, but it provides general recommendation for strengthening as well as retrofitting the existing curtain walls. It is stipulated in EN 13124-2 standard that Bomb blast net curtains and catchers are intended to protect people inside buildings from exterior explosions. The nets cover curtain walls inside buildings and are aimed to catch and retain flying shards of glass, preventing the whole glass

panels from being dislodged by blast wave, and it is illustrated in Fig 2.28. In addition, EN 13124-2 standard shows that some ‘classic windows’ fulfilling the requirements for standard window class (wind classes 0-5), and it recognizes five different classes of blast resistant windows, with the highest-pressure level equal to 6.30 bar.

2.4.1.5 GSA Standard (2003):

The US General Services Administration (GSA) presented standards for large Window Glazing Analysis Response and Design (WINGARD), which was developed by Applied Research Associates (ARA). This standard is basically based on ASTM 1642. The main goal of this document was to develop a practical but precise analytical model of the response of large windows to the effects of an explosion. This test standard is intended to ensure an adequate measure of standardization and quality assurance in the testing of window systems including but not limited to glazing, sealants, seats and seals, frames, anchorages and all attachments and/or secondary catcher or restraint mechanisms designed to mitigate the hazards from flying glass and debris. Design codes such as GSA TS-01 (2003), categorizes glass fragments threat based on their splash distances into a given occupied area. The air-blast loading is created either from an open arena or a blast simulator (shock tube). The criteria of fragments threat, in accordance with GSA TS-01 is illustrated in Fig 2.29. According to the GSA code, large glass windows which do not break or break but managed to keep fragments within frame members are rated as “no threat”. Provided glass fragments are expected to break down within 1 m distance from the opening, the threat is rated as “very low”; when the debris fly longer than 0.6 at 3 m distance, the hazard level is rated as “high”. Table 2.5 indicates the GSA rating of the window hazards. Basically, the blast loading ought to be commensurate with the specification of the building usage. Table 2.6 demonstrates the GSA protection level.

2.4.1.6 ASTM F 1642-96 Standard:

This test method sets forth procedures for the evaluation of the resistance of glazing or glazing systems against air-blast loadings. Besides, it provides the flexibility for glazing to be tested with or without framing systems. This test method is organized to test all glazing and glazing systems, like those fabricated from glass, plastic, glass-clad plastics, laminated glass,

glass/plastic glazing materials, and film-backed glass. In this standard, the air-blast is created by either a live explosive testing arena or a shock tube. The blast-resistant curtain wall was evaluated based on performance of the window during the test. If the glazing failed and permitted the pressure to spread inside the building, or the fragments passed through the aluminum foil located at 305 mm away from the window, the glazing was considered to have failed.

2.4.1.7 ASTM F 1642-12 Standard (2012):

This test method sets forth procedures for the evaluation of hazards of glazing or glazing systems against air-blast loadings. The specifying authority shall provide the air-blast loading parameters. Glazing systems shall be as defined in Specification F2912. The hazard levels after a blast test are exhibited in Fig 2.30. This standard concentrates on the position of the post-test window fragments and debris compared to its pre-test location. This standard does not intend to solve all of the safety issues, if any, associated with its use. It is the responsibility of the user to provide appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.

2.4.2 Design Standards

2.4.2.1 Canadian Standards Association (CSA) S850-12:

This standard presents an overview of the requirements of design and assessment of buildings subjected to blast loads. CSA S850 standard is designed to specify the requirements for analysis and design of new buildings and assessment of existing buildings to resist blast loading. The main objective of this standard is to decrease human casualties and damage to assets due to an air blast. Likewise, it acknowledges that the extent and severity of an explosion cannot be predicted with certainty. The behavior of the building and the safety of the residents, are dependent on the level of protection and the design basis threat (DBT). Table 2.7 presents the level of building protection based on CSA performance criteria. The test results are expected to follow the requirements of either one of ASTM F1642, GSA-TS01 or ISO 16933. Therefore, if testing was not available, dynamic analysis of the glazed window would be needed. According to the CSA standard, level of protection varies between B1 to B4 responses

as they are illustrated in Figure 2.31. Response limits shall be determined for the building component in order to establish the level of damage. It can be specified by considering support rotation, and the ratio of maximum deformation to the same type of deformation at yield. Table 2.8 presents response limits for open web steel joists, cold-formed steel elements, and steel glazing system framing in flexure. Based on this standard, the mullions and transoms ought to be designed using dynamic analysis or be analyzed experimentally to examine their potentials. Basically, utilized materials are comprised of steel, aluminum, wood or other usual substances. The curtain wall system should be designed such that the mullion has more capacity than the glazing, and anchors are expected to be stronger than the curtain wall mullion and transom. In addition, it follows the window requirement which the substrate should be 50% stronger than the window frame. Therefore, the curtain wall mullion and transoms ought to be 50% more rigid than the substrates.

2.4.2.2 The Unified Facilities Criteria (UFC) 2018:

The UFC Standard was developed for Department of Defense (DoD) facilities. The main reason of this standard is to provide minimum engineering standards that incorporate antiterrorism (AT) based mitigating measures where no identified threat or level of protection has been determined in commensurate with UFC 4-020-01. An interagency security Committee standard is applicable to all structures and facilities in the United States occupied by Federal employees for non-military activities. These comprise of existing and new buildings, or major modernizations. The UFC establish standards that provide minimum levels of protection against terrorist attacks for the residents of all DoD inhabited buildings. It establishes a foundation for additional protective measures in higher threat environments. When confronted with design basis threats (explosive weights) more severe than outlined by UFC 4-010-01, the designer should consult UFC 4-020-01. These UFCs are intended to be used by security and antiterrorism personnel and design teams to identify the minimum requirements that must be incorporated into the design of all new construction and major renovations of inhabited DoD buildings. UFC 4-010-01 recommends specific standoffs for conventional construction

standoffs, which are summarized in Table 2.9. The design load values considered in UFC 4-010-01 based on the ASTM F2248 are presented in Figure 2.32.

According to UFC, for glazing in exterior building elements such as windows, curtain walls, and skylights provide no less than 1/4 in. (6 mm) nominal polycarbonate or laminated glass. The 1/4 in. (6 mm) laminated glass comprise of two nominal 1/8 in. (3 mm) glass panes bonded together with a minimum of a 0.030 in. (0.75 mm) interlayer of a material that has typically been utilized in blast resistant window applications. In addition, for curtain wall system, primary mullions that span between points of structural support should be deemed supporting frame members and may be designed dynamically or statically. If designed by the static approach, the moment and shear capacities of framing members should be designed to withstand 2 times the load resistance of the glazing applied to the framing members only from the tributary area of the curtain wall, and deflection should be limited to 1/60 of the members' span lengths between points of structural support. Furthermore, intermediate mullions ought to be assessed for deflection with the supported edge length taken as equal to the longest span of a single glass panel and the deflection should be measured according to simple support conditions for that length.

2.4.2.3 ASTM F2248 and ASTM E1300:

This standard sets forth a method to define an equivalent 3-second design loading appropriate to use with Practice E1300 to select the thickness and type of blast resistant glazing fabricated with laminated glass to glaze a fenestration. Glass plies used to construct laminated glass are recommended to be either annealed or heat strengthened glass. This standard applies to blast resistant glazing fabricated using laminated glass only, including single laminated glass and insulating glass fabricated with laminated glass. As a minimum, insulating glass shall use laminated glass for the inboard (protected side) lite. The ASTM F2248 presumes that blast resistant glazing should be connected to its supporting frame utilizing structural silicone sealant or adhesive glazing tape. Figure 2.33 depicts the particular chart to obtain the design loads based on variety of charge weights and standoff distances combinations. The width of the structural silicone sealant bead shall be at least equal to the larger of 10-mm (3/8-in.) or the

thickness selection of the glass to that it connected, nevertheless, not bigger than two times the thickness selection of the glass to which it adheres. The minimum thickness of the structural silicone bead ought to be 5-mm (3/16-in.). The width of glazing tape shall be at least alike to 2 times but not more than 4 times the thickness selection of the glass to which it complies.

ASTM E1300, specifies the approach to determine the load resistance of glass types, including combinations of glass types utilized in a sealed insulating glass (IG) unit, exposed to a uniform lateral load of short or long duration, for a specified possibility of breakage. This practice is applicable merely to monolithic and laminated glass constructions of rectangular shape which have continuous lateral support alongside of one, two, three, or four edges. The non-factored load chart for 6mm annealed glass with four sides simply supported is shown in Fig 2.34. This standard includes three assumptions, including: i) the supported glass edges for two, three, and four-sided support conditions should be simply supported as well as free to slip in-plane; ii) glass supported on two sides behaves as a simply supported beam; and iii) glass supported on one side acts as a cantilever. For insulating glass units, this Standards just applies to insulating glass units which have four-sided edge support.

2.4.2.4 Technical Reference for Office Building Design; Public Service and Procurement Canada (2017)

This document states the general solutions to the design of Public Service and Procurement Canada (PSPC) custodial office buildings. In addition, it elaborates the technical concepts that apply to each important principle related to the design. The technical reference for curtain walls and office windows should be applied to new buildings in their entirety and to renovations of existing buildings, given their constraints, to the greatest extent possible. It includes a section for exterior curtain wall assemblies and components which recommends general solutions for security concerns like, blast threats as well as progressive collapse prevention. PSPC stipulates that there are currently no standards relating to curtain wall systems that require assemblies to resist any level of bomb blast. Therefore, it recommends that a security expert designer should define the level of hazards and threats in terms of expected

overpressure at wall surface. Furthermore, it asserts that structural silicone glazing systems provides the best resistance to blasts.

2.4.2.5 Graham Architectural Series W2700 Window Wall Specifications (2015)

The Graham architectural products specifications provide manufacturing description on glazed aluminum curtain walls and blast mitigation (Section 08 44 13, 2012b). This guide specification was developed using the Construction Specifications Institute (CSI) “Manual of Practice,” including the recommendations for the CSI 3 Part Section Format and the CSI Page Format. The maximum deflection in the midspan is restricted to 1/60 frame length in the same direction of the deflection. Generally, the mentioned specifications are in line with American architectural manufacturing association, American society of testing materials, and GSA instructions. It recommends inside glazed or outside glazed system with 1-1/4" double glazed insulating glass. Also, it states that glass in the curtain wall should be inside glazed or outside glazed and held in place with extruded aluminum glass stops.

2.4.2.6 CERL (2015):

CERL (Canadian Explosives Research Laboratory) within the Explosives Safety and Security Branch of the Lands and Minerals Sector at National Research of Canada (NRC) is known as the only authorized Canadian government laboratory which is in charge of providing guidelines for explosion and blast testing methodology. Based on CERL, the best practice for making curtain walls or office windows blast resistant is to utilize PVB layers as well as film protective. It clarifies the structures based on their hazard level, and requirements are met by utilizing a protective film on the inner side of the large windows and curtain wall. Nevertheless, the film protective layer must be anchored to the mullions and notwithstanding the mullions should be entirely fixed to the slabs and related substrates. The substrates are expected to have at least 193 MPa tensile strength, because the substrate capacity should be more than the maximum transferred strength from film protective layer.

Table 2.1: Nominal charge sizes and standoff distances equate to the blast values for the SATCHEL-BOMB classifications (ISO 16933, 2007)

Classification Code	Classification criteria blast values		Nominal charge mass and standoffs for a small test frame		Calculated nominal equivalents to a large facade	
	Pressure (kPa)	Impulse (kPa-ms)	TNT equivalent charge size (kg)	Standoff distance (m)	TNT equivalent charge size (kg)	Standoff distance (m)
SB 1(X)	70	150	3	9	3.25	9.00
SB 2(X)	110	200	3	7	3.5	7.26
SB 3(X)	250	300	3	5	3.50	5.07
SB 4(X)	800	500	3	3	3.70	3.40
SB 5(X)	700	700	12	5.5	12.00	5.26
SB 6(X)	1600	1000	12	4	12.6	4.06
SB 7(X)	2800	1500	20	4	21.00	4.00

Table 2.2: Nominal charge sizes and standoff distances equate to the blast values for the VEHICLE-BOMB classifications (ISO 16933, 2007)

Classification Code	Classification criteria blast values		Nominal standoffs of approx. 100 kg TNT for a small test frame (m)	Calculated nominal equivalents to a large facade	
	Pressure (kPa)	Impulse (kPa-ms)		TNT equivalent charge size (kg)	Standoff distance (m)
EXV 45(X)	30	180	45	30	32
EXV 33(X)	50	250	33	30	23
EXV 25(X)	80	380	25	40	19
EXV 19(X)	140	600	19	64	17
EXV 15(X)	250	850	15	80	14.4
EXV 12(X)	450	1200	12	100	12.4
EXV 10(X)	800	1600	10	125	11

Table 2.3: The hazard rating criteria and classification for arena tests (ISO 16933, 2007)

Hazard rating	Hazard-rating description	Definition
A	No break	The glazing is observed not to fracture and there is no visible damage to the glazing system
B	No hazard	The glazing is observed to fracture but the inner, rear face leaf is fully retained in the facility test frame or glazing system frame with no breach and no material is lost from the interior surface; outer leaves from the attack face may be sacrificed and may fall or be projected out
C	Minimal hazard	The glazing is observed to fracture; outer leaves from the attack face may be sacrificed and may fall or be projected out; the inner, rear face leaf shall be substantially retained, with the total length of tears plus the total length of pull-out from the edge of the frame less than 50% of the glazing sight perimeter. Also, there are no more than three ratable perforations or indents anywhere in the witness panel and any fragments on the floor between 1m and 3m from the interior face of the specimen have a sum total united dimension of 250mm or less; glazing dust and slivers are not accounted for in the hazard rating. If by design intent there is more than 50% pull-out but the glazing remains firmly anchored by purpose-designed fittings, a rating of C (minimal hazard) may be awarded, provided that the other fragment limitations are met; the survival condition and anchoring provisions shall be described in the test report
D	Very low hazard	The glazing is observed to fracture and significant parts are located no further than 1m behind the original location of the rear face; parts are projected any distance from the attack face towards the blast source. Also, there are no more than three ratable perforations or indents anywhere in the witness panel, and any fragments on the floor between 1m and 3m from the interior face of the specimen have a sum total united dimension of 250mm or less; glazing dust and slivers are not accounted for in the rating
E	Low hazard	The glazing is observed to fracture, and glazing fragments or the whole of the glazing falls between 1m and 3m behind the interior face of the specimen and not more than 0.5m above the floor at the vertical witness panel. Also, there are 10 or fewer ratable perforations in the area of the vertical witness panel higher than 0.5m above the floor and none of the perforations penetrate more than 12mm
F	High hazard	Glazing is observed to fracture and there are more than 10 ratable perforations in the area of the vertical witness panel higher than 0.5m above the floor, or there is one or more perforations in the same witness panel area with fragment penetration more than 12mm

Table 2.4: Classification of and impulse for glazed window based on ISO 16934, 2007

Classification Code	Minimum values	
	Peak Pressure <i>kPa</i>	Impulse kPa-ms
ER 30(X)	30	170
ER 50(X)	50	370
ER 70(X)	70	550
ER 100(X)	100	900
ER 150(X)	150	1500
ER 200(X)	200	2200

Table 2.5: GSA Ratings of window hazard (2003)

Performance condition	Protection level	Hazard level	Description of Window Glazing Response
1	Safe	None	Glazing does not break. No visible damage to glazing or frame.
2	Very high	None	Glazing cracks but is retained by the frame. Dusting or very small fragments near sill or on floor acceptable
3a	High	Very low	Glazing cracks. Fragments enter space and land on floor no further than 3.3 ft. from the window
3b	High	Low	Glazing cracks. Fragments enter space and land on floor no further than 10 ft. from the window
4	Medium	Medium	Glazing cracks. Fragments enter space and land on floor and impact a vertical witness panel at a distance of no more than 10 ft. from the window at a height no greater than 2 ft. above the floor
5	Low	High	Glazing cracks and window system fails catastrophically. Fragments enter space impacting a vertical witness panel at a distance of no more than 10 ft. from the window at a height greater than 2 ft. above the floor

Table 2.6: GSA Protection levels (2001)

Building Description				Maximum Over-pressure	Maximum impulse
Type	Occupants	Square Feet	Tenants		
A	<10	<25k	Recruiting Office	0	0
B	<150	<80k	Social Sec. Admin.	0	0
C	<450	<150k	Law Enforcement	4 psi	30 psi-msec
D	>450	>150k	Courts, Justice	10 psi	90 psi-msec
E	>450	>150k	Pentagon, CIA	Classified	Classified

Table 2.7: Comparison of LOP and hazard for various glazing standards CSA-S580

Level of protection	BUILDING PERFORMANCE
Very low	Collapse Prevention: Able to evacuate; damage up to onset of collapse; re-entry unlikely; contents not intact.
Low	Life Safety: Able to evacuate; temporary re-entry; repair not economically viable.
Medium	Immediate Occupancy: Able to evacuate; operational after cleanup or repair; repair economically viable.
High	Operational: Uninterrupted occupancy; contents fully functional; minimal local damage only.

Table 2.8: Response limits for open web steel joists, cold-formed steel elements, and steel glazing system framing in flexure (CSA-S580)

Element type			B1		B2		B3		B4	
			$\mu_{\max}$	$\theta_{\max}$	$\mu_{\max}$	$\theta_{\max}$	$\mu_{\max}$	$\theta_{\max}$	$\mu_{\max}$	$\theta_{\max}$
Flexure	Open web steel joist	Downward loading†	1	—	—	3°	—	6°	—	10°
		Upward loading‡	1	—	1.5	—	2	—	3	—
		Shear response§	0.7	—	0.8	—	0.9	—	1	—
	Cold-formed steel	Girt or purlin	1	—	—	3°	—	10°	—	20°
		Stud with sliding connection at top	0.5	—	0.8	—	0.9	—	1	—
		Stud connected at top and bottom**	0.5	—	1	—	2	—	3	—
		Stud with tension membrane††	0.5	—	1	0.5°	2	2°	5	5°
		Corrugated panel (1-way) with full tension membrane‡‡	1	—	3	3°	6	6°	10	12°
		Corrugated panel (1-way) with some tension membrane§§	1	—	—	1°	—	4°	—	8°
		Corrugated panel (1-way) with limited tension membrane***	1	—	1.8	1.3°	3	2°	6	4°
	Glazing system framing	Steel	1	—	—	3°	—	6°	—	10°

Table 2.9: Conventional construction standoffs (UFC 4-010-01)

Standoff		Facility A (Low level of protection)	Facility A (Very low level of protection)
To control perimeter	Conventional Construction	148 feet	82 feet
	Minimum	82 feet	33 feet
To parking and roadways within the controlled perimeter	Conventional Construction	82 feet	33 feet
	Minimum	33 feet	33 feet

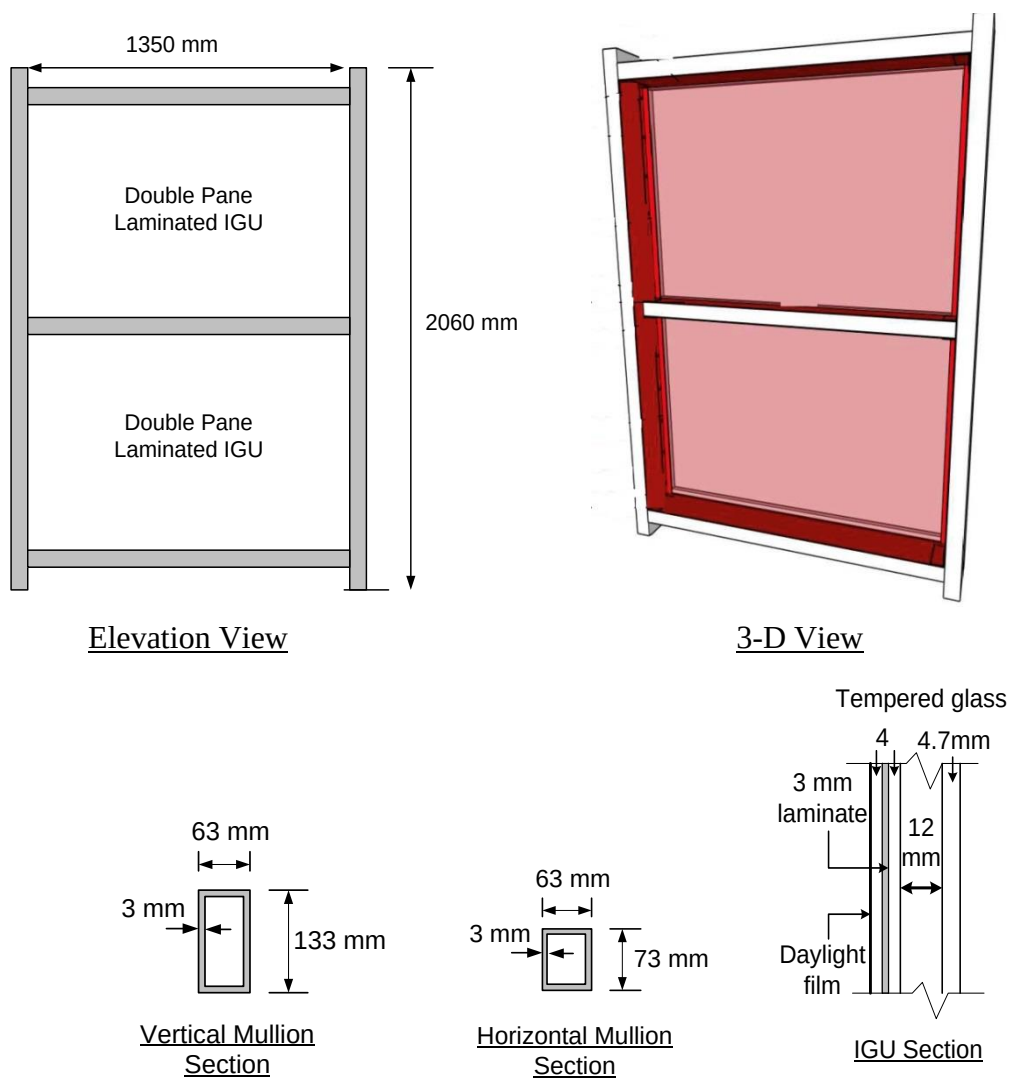


Figure 2.1: Geometric properties of curtain wall test specimens



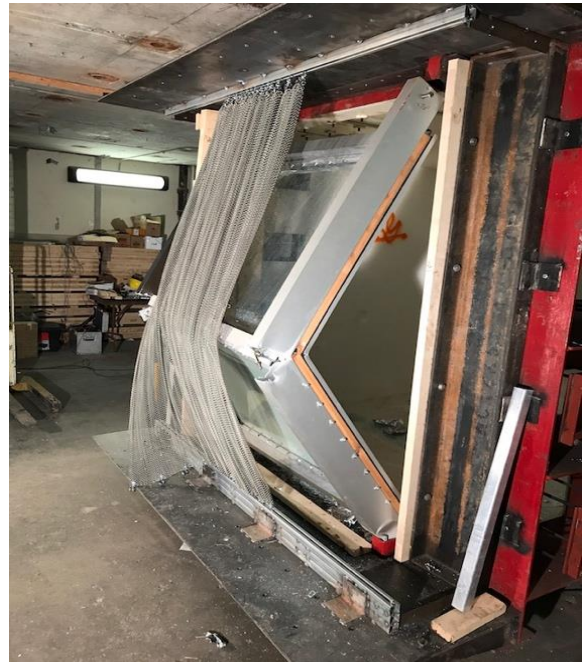
Figure 2.2: Behaviour of the curtain wall during the first test



Figure 2.3: Curtain wall specimen used in Test 2 prior to retrofit (on left) and the aluminum angles used to enhance the connection between horizontal and vertical mullions (on right).



a) Front view at the end of Test 2



b) Side view after moving the drapery to expose the vertical mullion

Figure 2.4: Retrofitted curtain wall after Test 2



(a) Mechanical glazing



(b) Strengthening mullion connection

Figure 2.5: Blast retrofit implemented for Test 3



(a) Bending of vertical mullion



(b) Bending of horizontal mullion

Figure 2.6: Performance of curtain wall during Test 3



Figure 2.7: Application of aluminum plates and angles with epoxy adhesive for strength enhancement



Figure 2.8: Mechanical glazing and mullion connection strengthening



Figure 2.9: Performance of curtain wall during Test 4 (none of the glass panes and horizontal components remained attached to the vertical mullions)

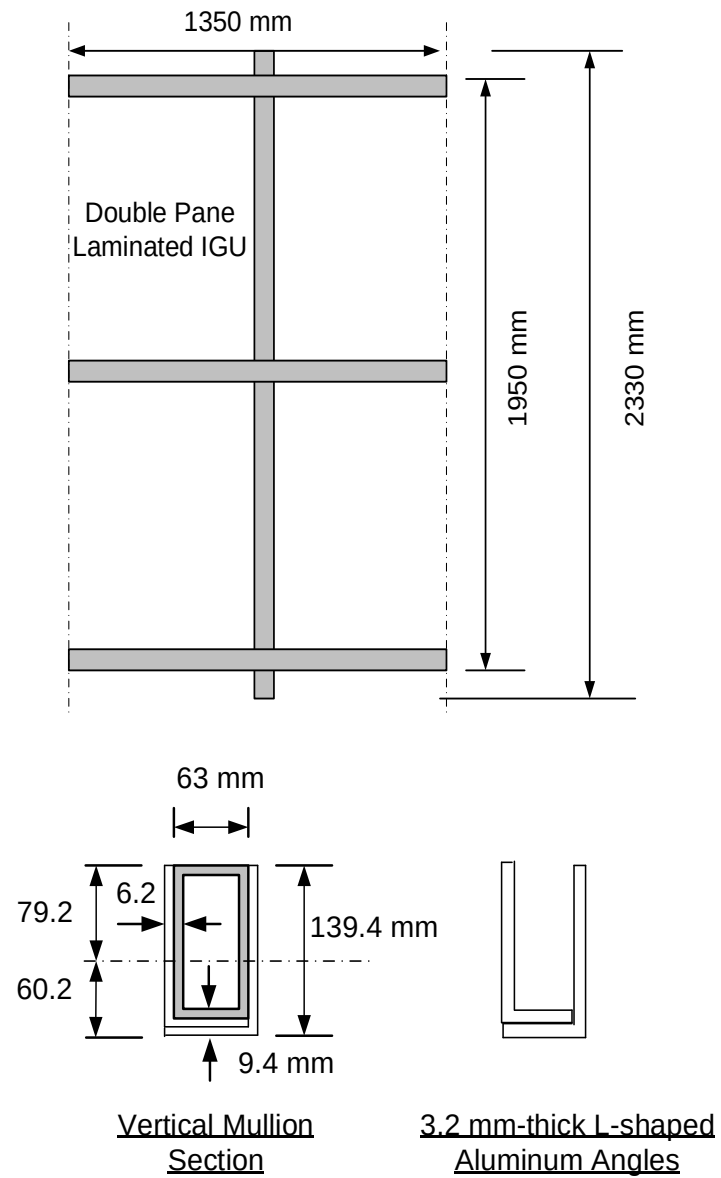
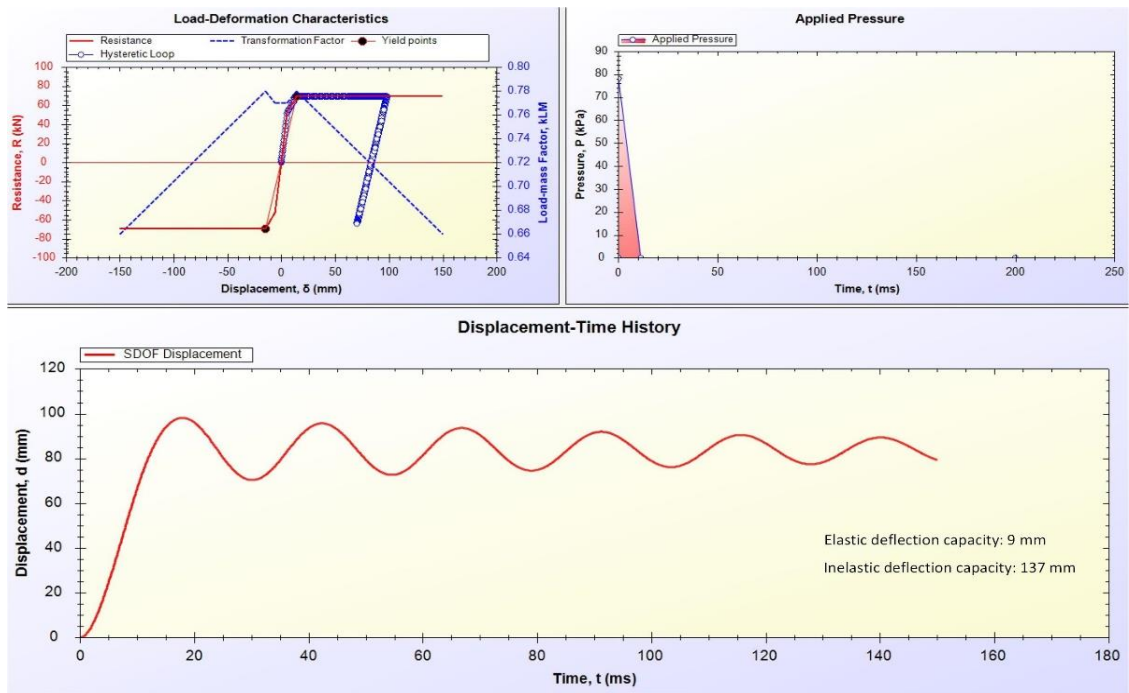
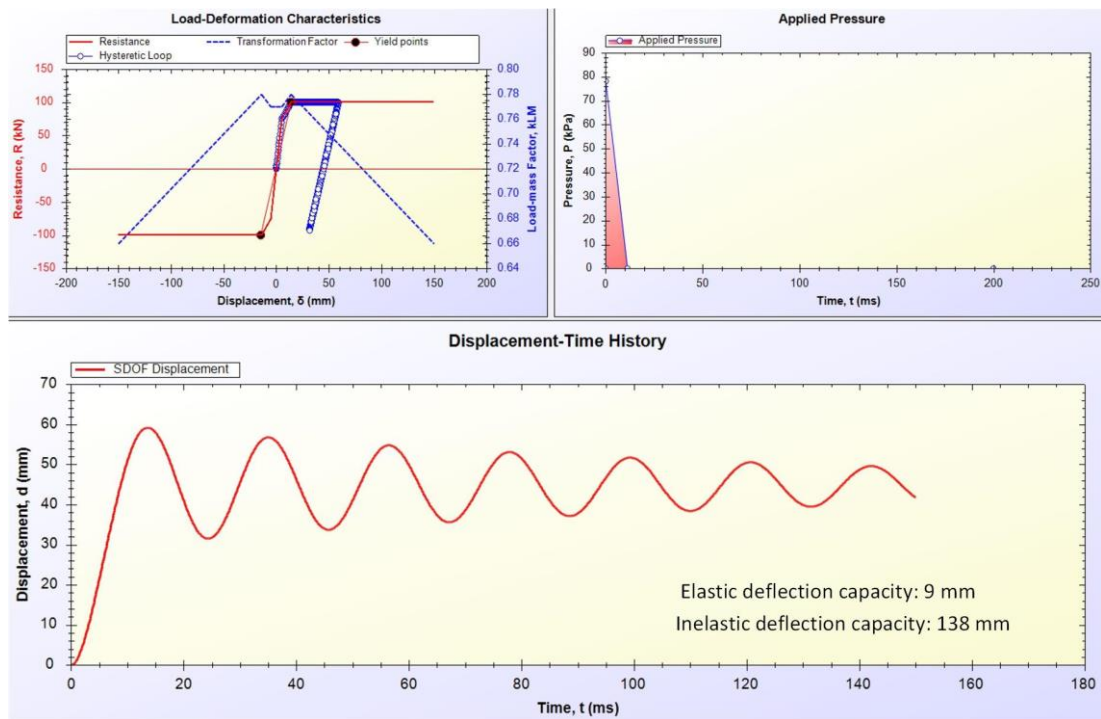


Figure 2.10: Hardened mullions considered in SDOF analysis



a: SDOF response of unhardened mullion with simply end supports



(b) SDOF response of hardened mullion with fixed end supports.

Figure 2.11: Mullion displacement-time history Mullion displacement-time history

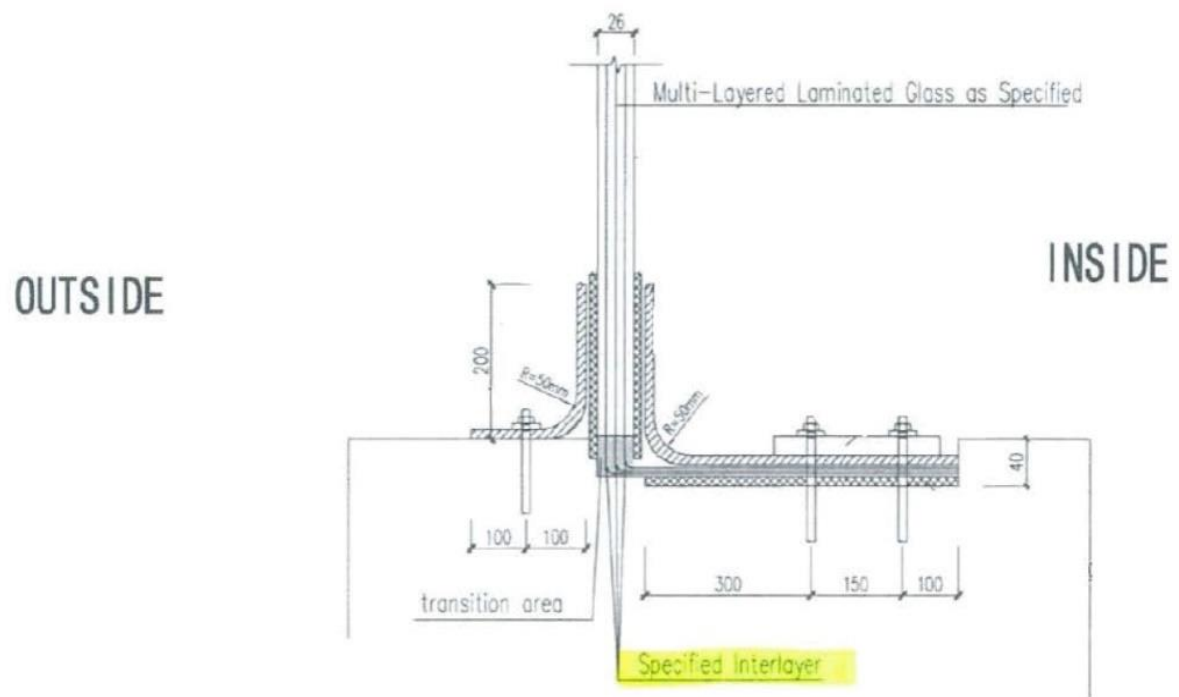


Figure 2.12: Details of the curtain wall sample tested



Figure 2.13: Performance of curtain walls anchored to top and bottom floors

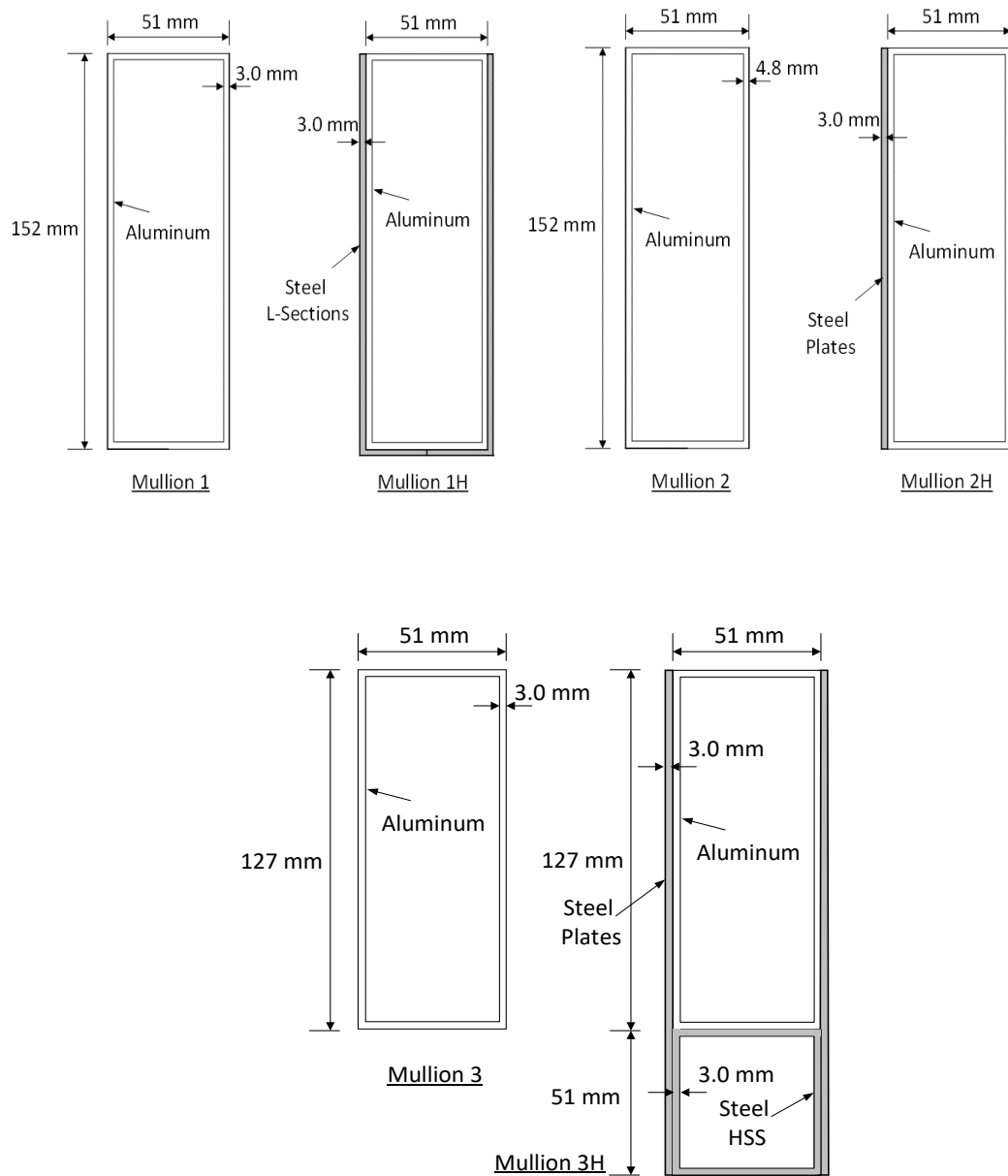
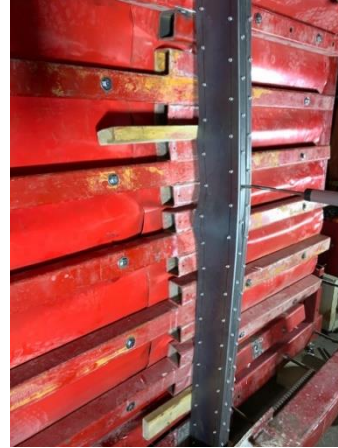


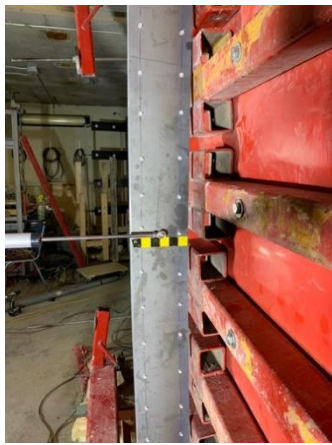
Figure 2.14: Hardening of mullions with different configurations of steel plates



(a) Mullion 1H before test



(b) Mullion 1H after test



(c) Mullion 2H before test



(d) Mullion 2H after test

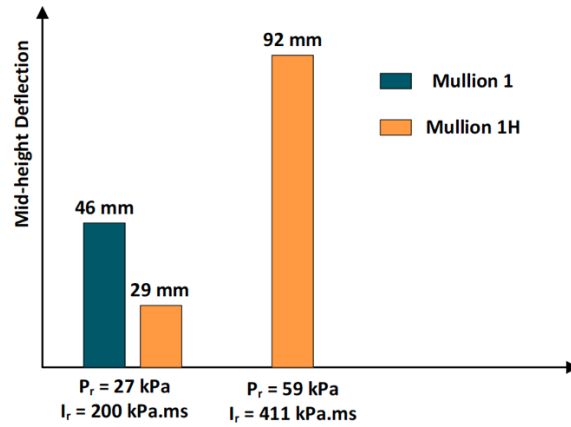


(e) Mullion 3H before test

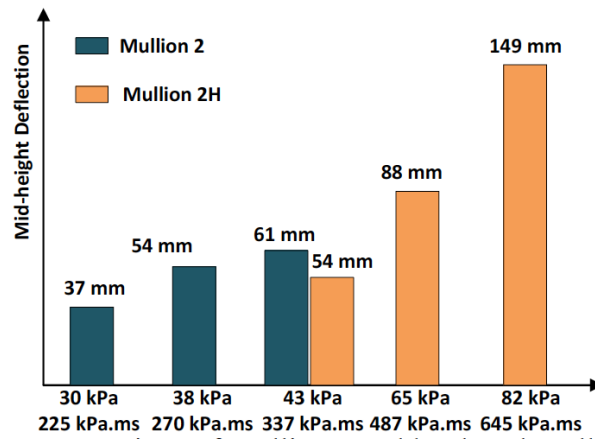


(f) Mullion 3H after test

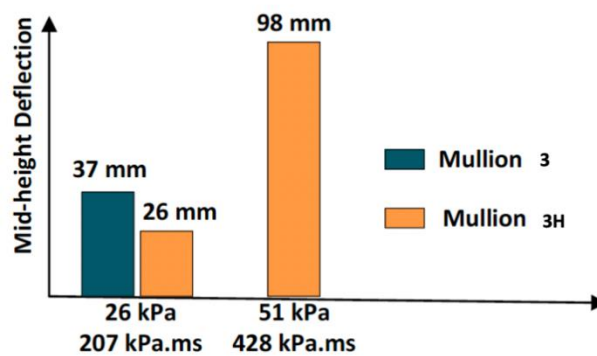
Figure 2.15: Hardened mullion before and after blast tests



(a) Unhardened Mullion 1 versus hardened Mullion 1H



(b) Unhardened Mullion 2 versus hardened Mullion 2H



(c) Unhardened Mullion 3 versus hardened Mullion 3H

Figure 2.16: Hardened mullion before and after blast tests

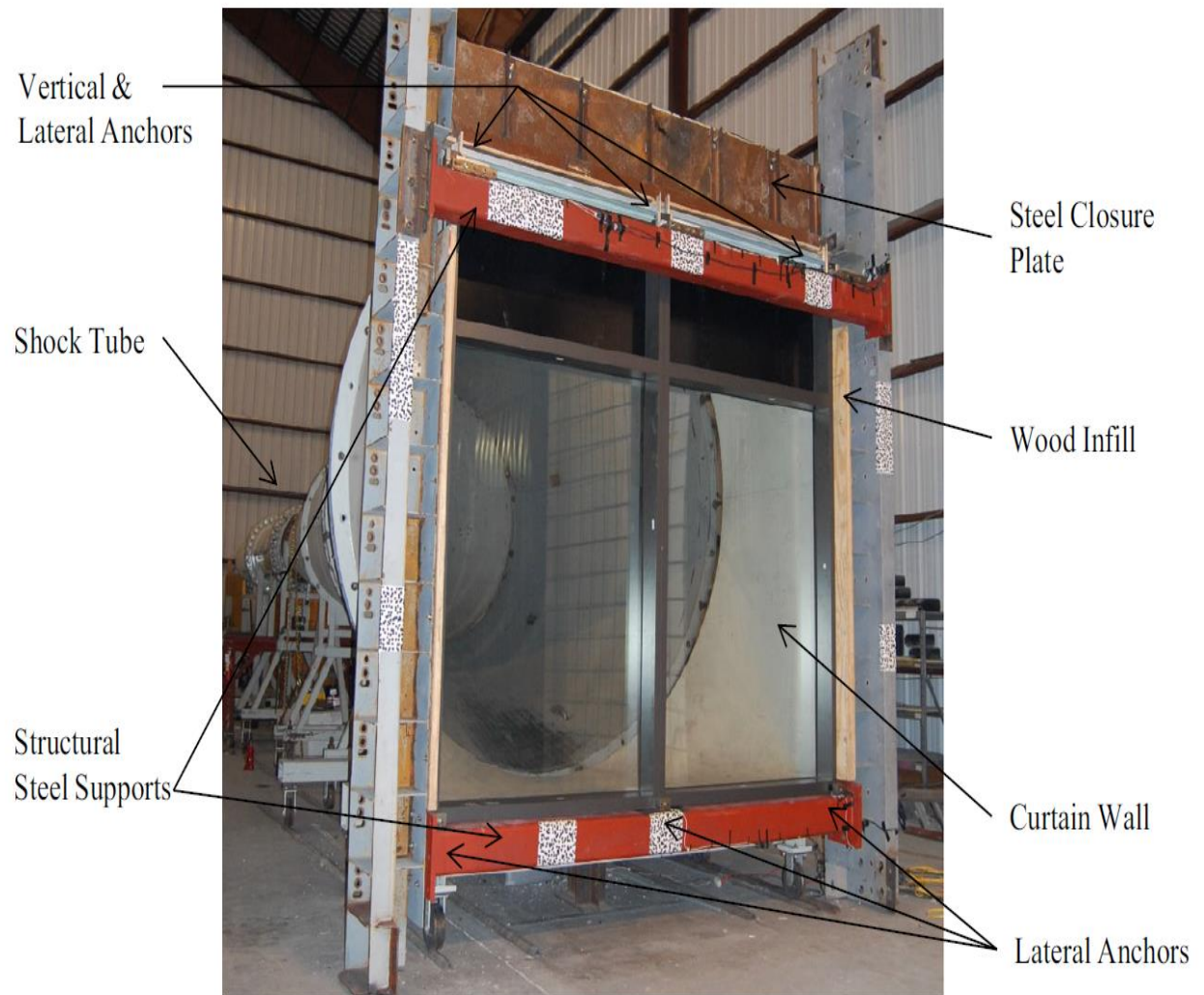


Figure 2.17: Full size curtain wall assembly and Shock-tube opening (Kirk et al. 2016)

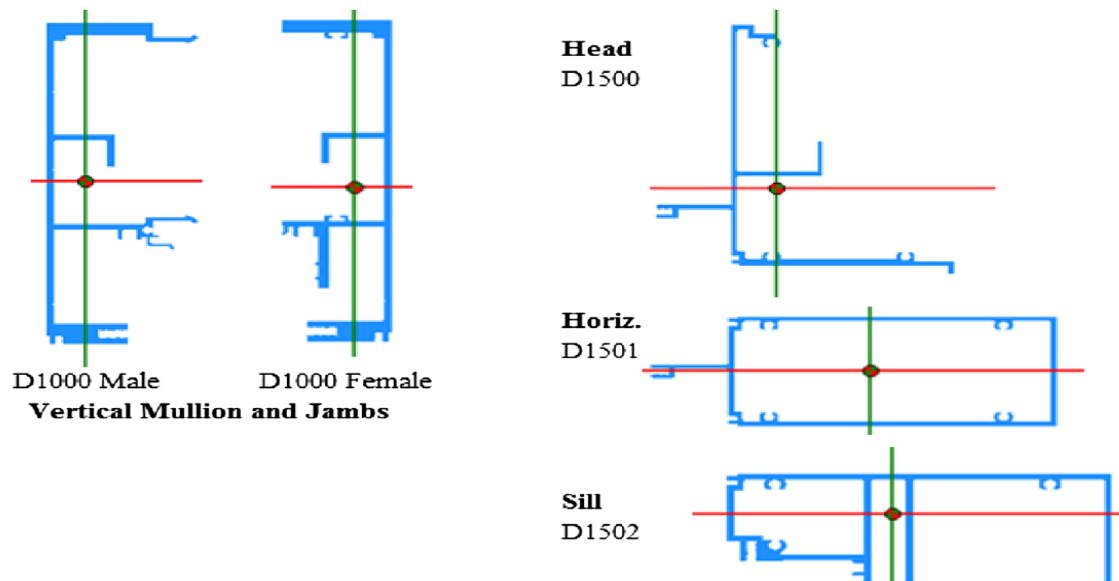
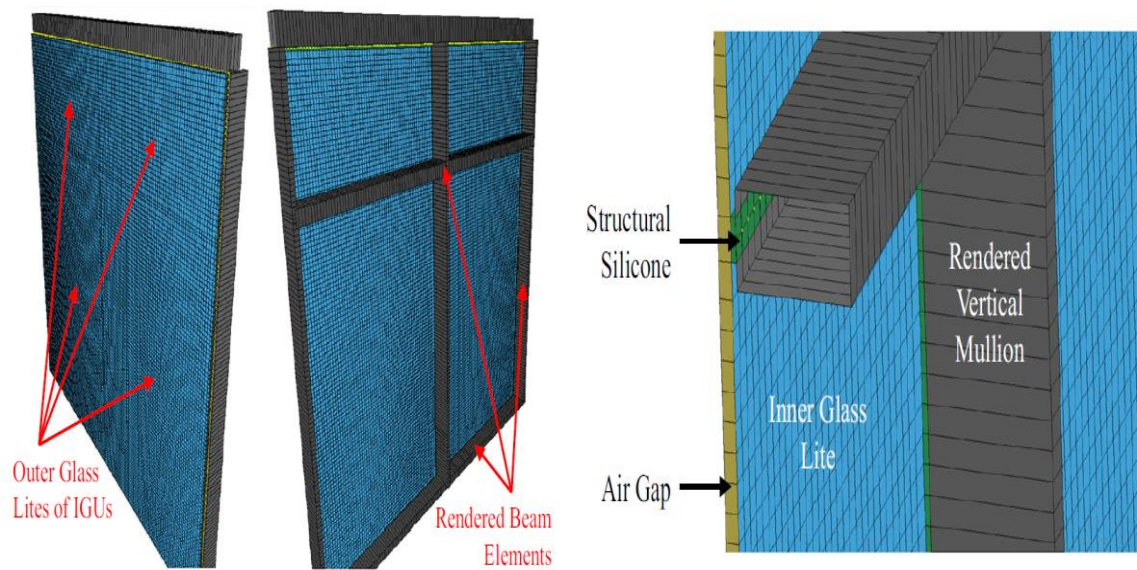


Figure 2.18: Aluminum mullion cross section (Kirk et al. 2016)



a) Blast-loaded face and protected face

b) Details along cross-section through centerline

Figure 2.19: FEM representation of curtain wall specimen IGUs (Kirk et al. 2016)



Figure 2.20: FEA representation of glass-break methodology (Kirk et al. 2016)



a) Unloaded specimen SPS-2



b) Loaded specimen just after mullion fracture

Figure 2.21: Curtain wall specimen in the reaction frame (Benjamin et al. 2013)

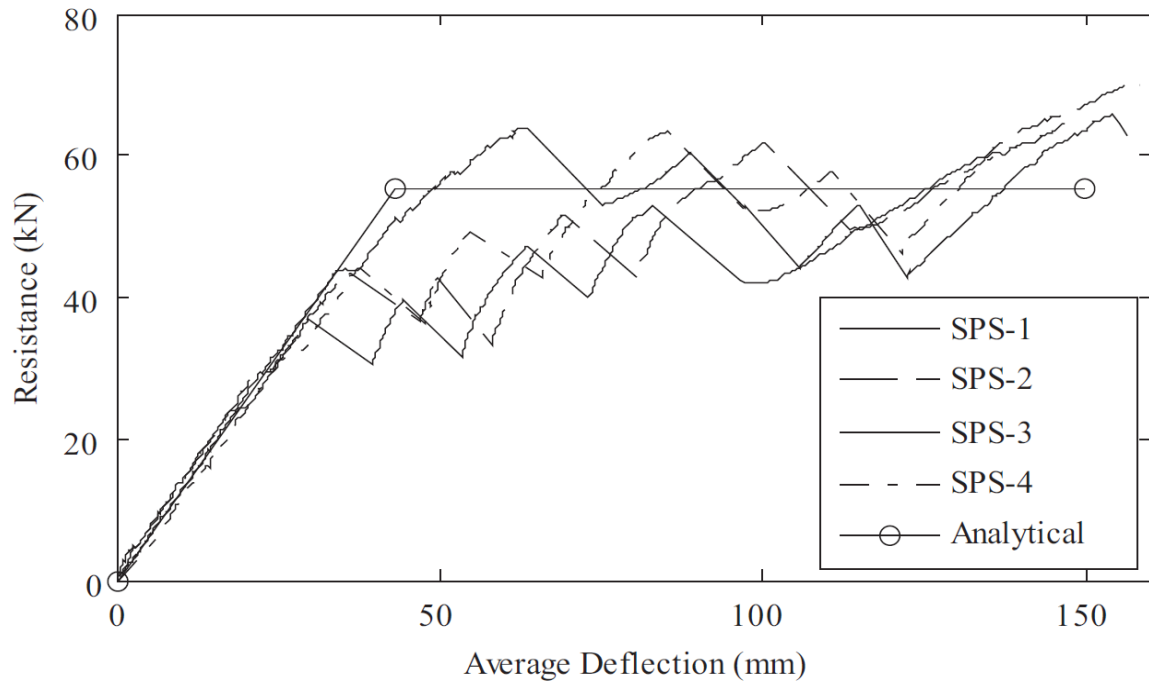


Figure 2.22: Comparison of experimental resistance curves to the analytical resistance function (Benjamin et al. 2013)

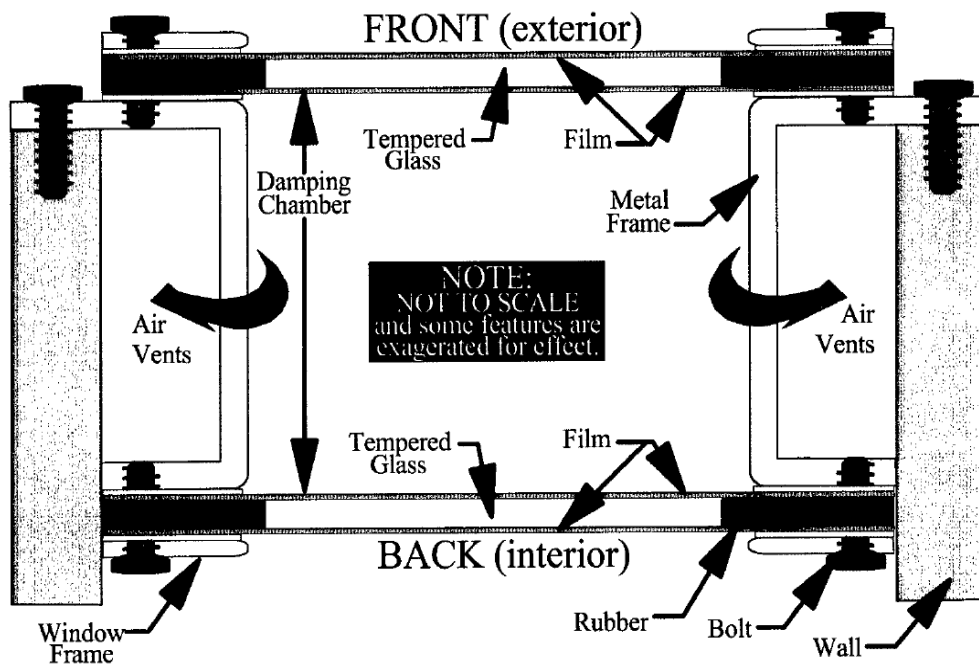


Figure 2.23: Schematic of sealed blast-resistant Flex window (Anderson et al. 2003)

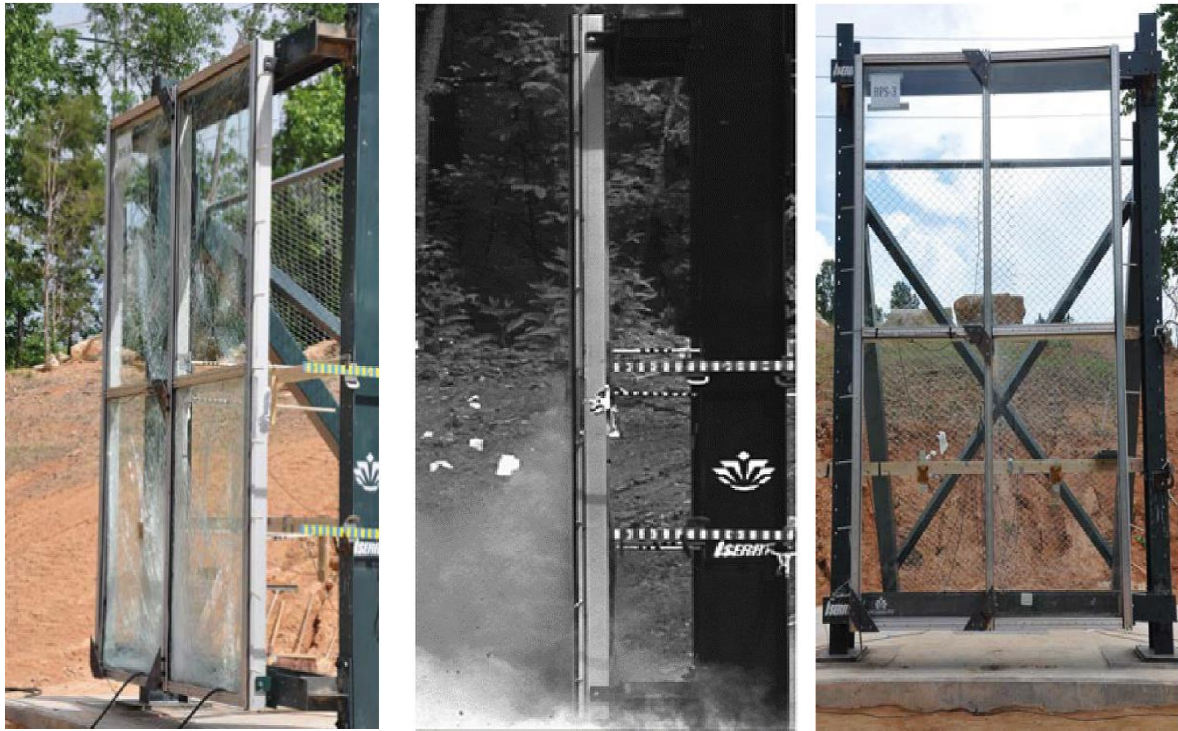
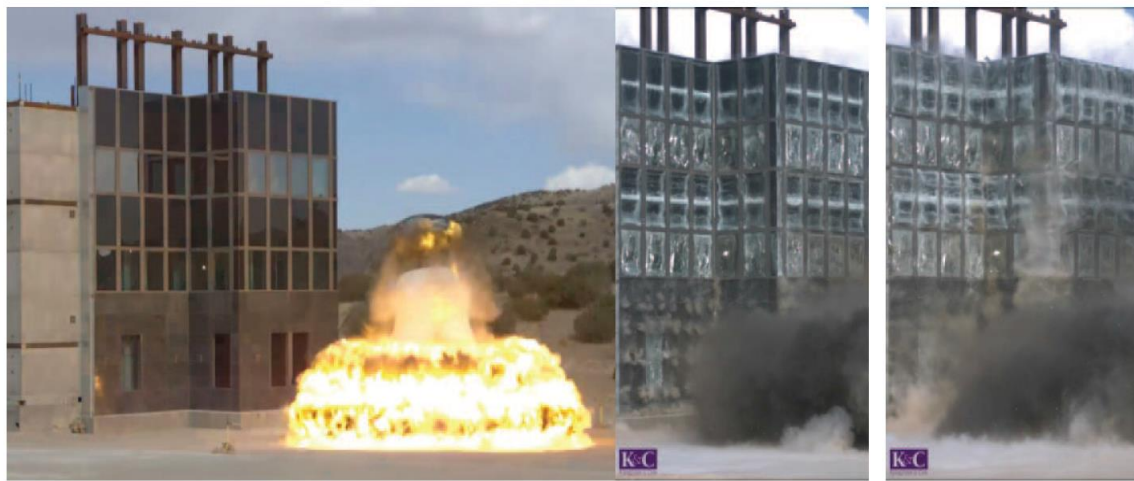


Figure 2.24: Curtain wall specimen before and after the test (Adam et al. 2015)



Detonation

Positive Phase

Negative Phase

Figure 2.25: Pretest and post-test exterior views of SSCW (Timothy et al. 2015)

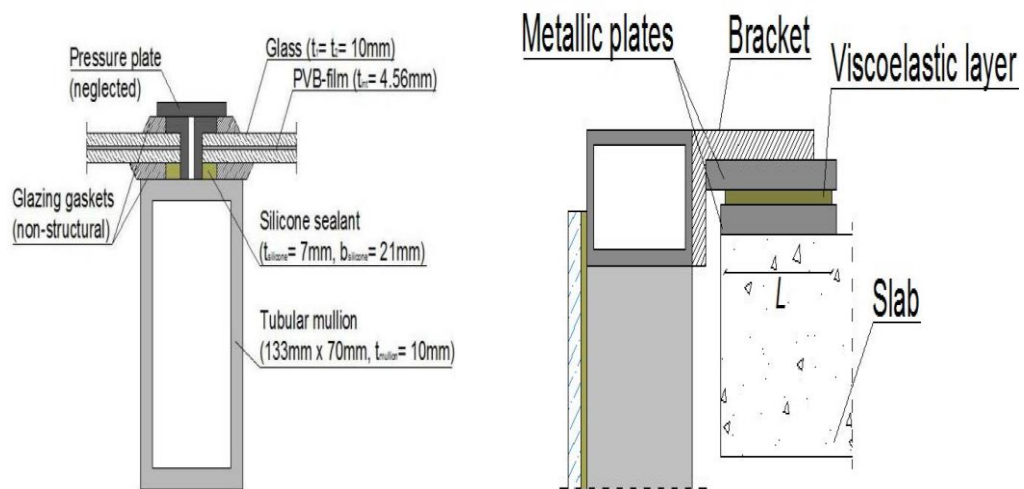


Figure 2.26: An example of idealized connection with glass panes, and scheme of a possible viscoelastic device (Rong et al. 2009)

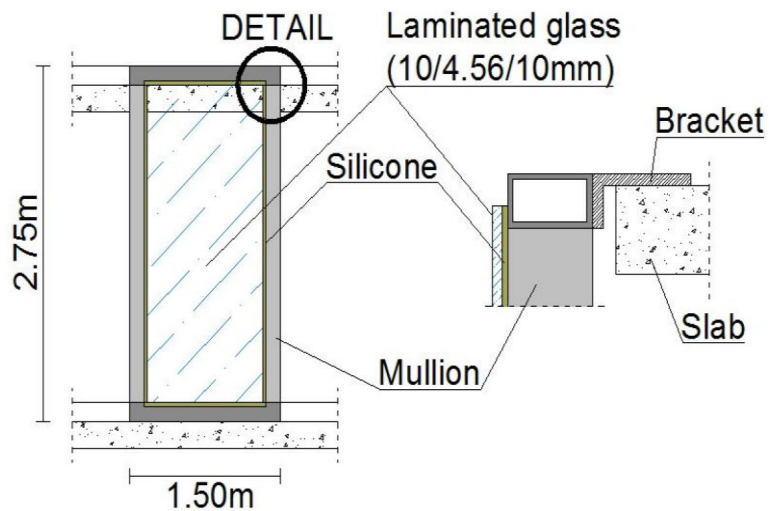


Figure 2.27: Idealized unit and scheme of a conventional bracket (Rong et al. 2009)

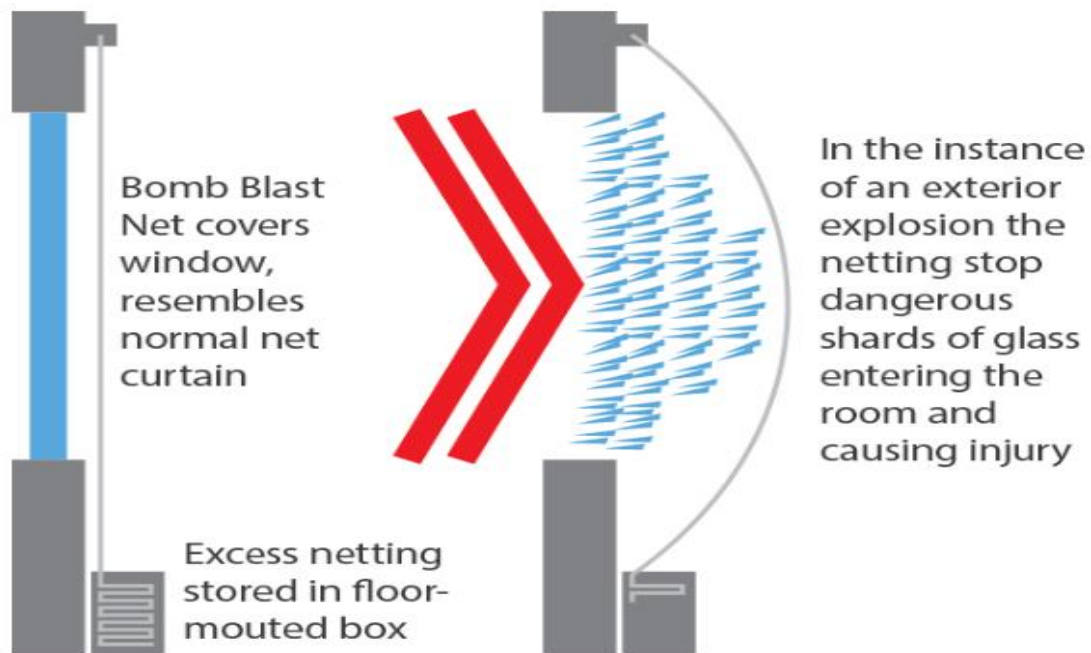


Figure 2.28: Bomb blast net curtains; schematic overview of the general concept

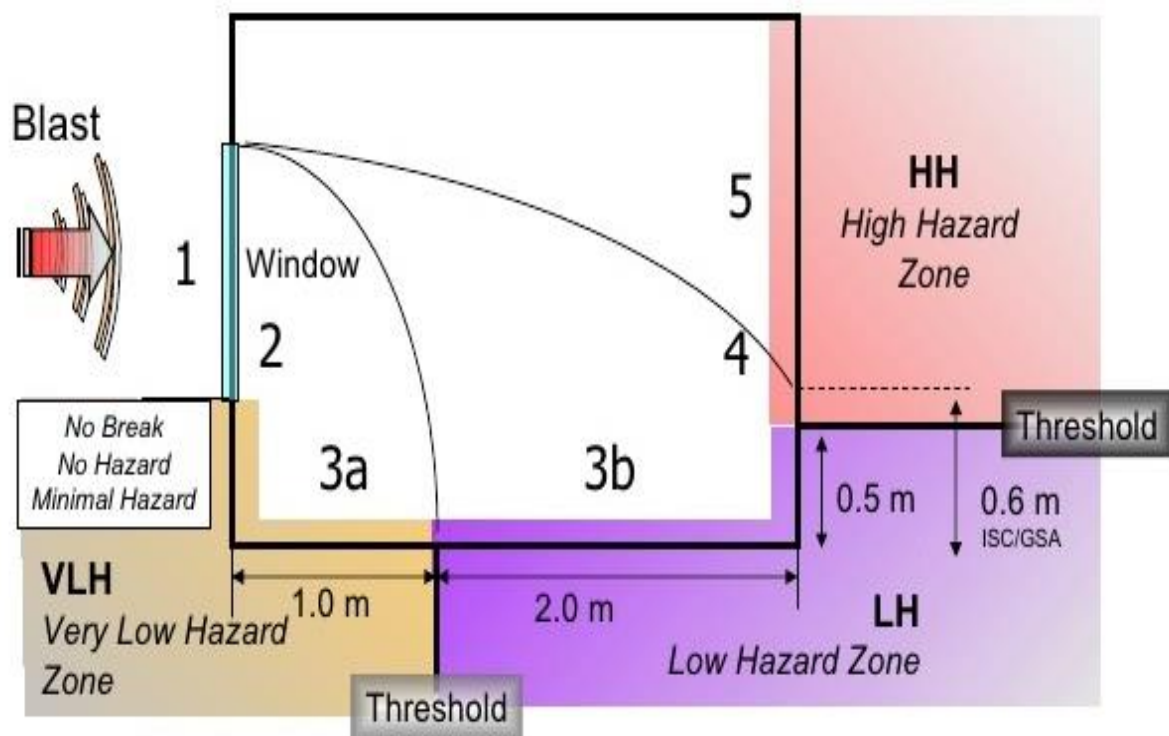


Figure 2.29: Criteria of fragments threat, in accordance with GSA TS-01 (2003)

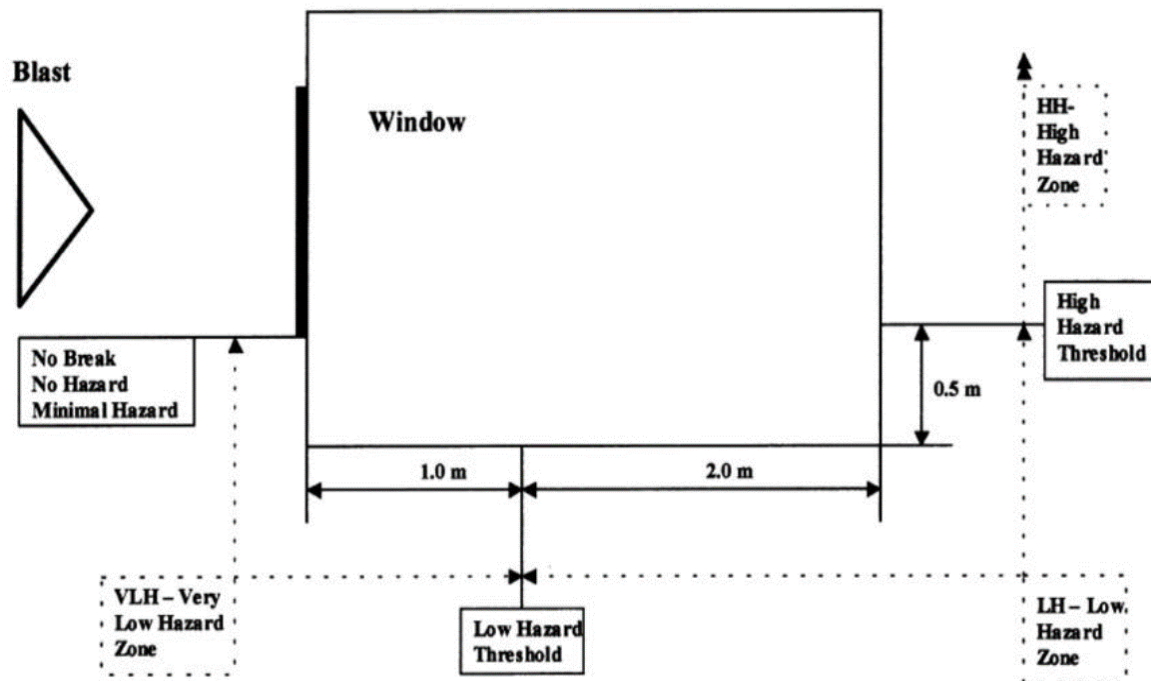


Figure 2.30: Window test set-up and hazard levels (ASTM F 1642-12)

Levels of Protection

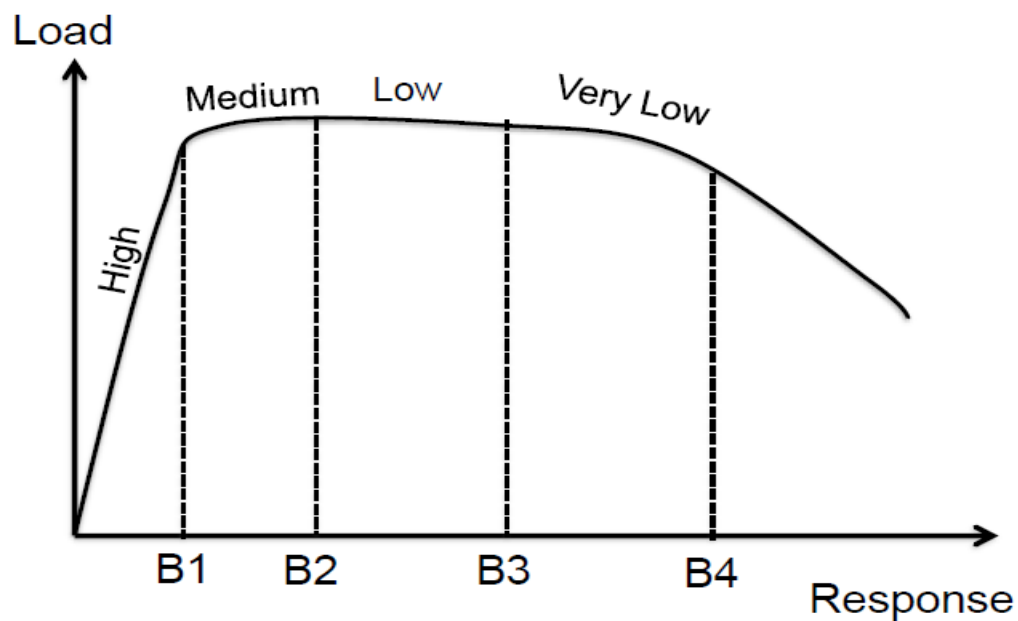


Figure 2.31: Performance criteria for level of protection based on CSA S850-12

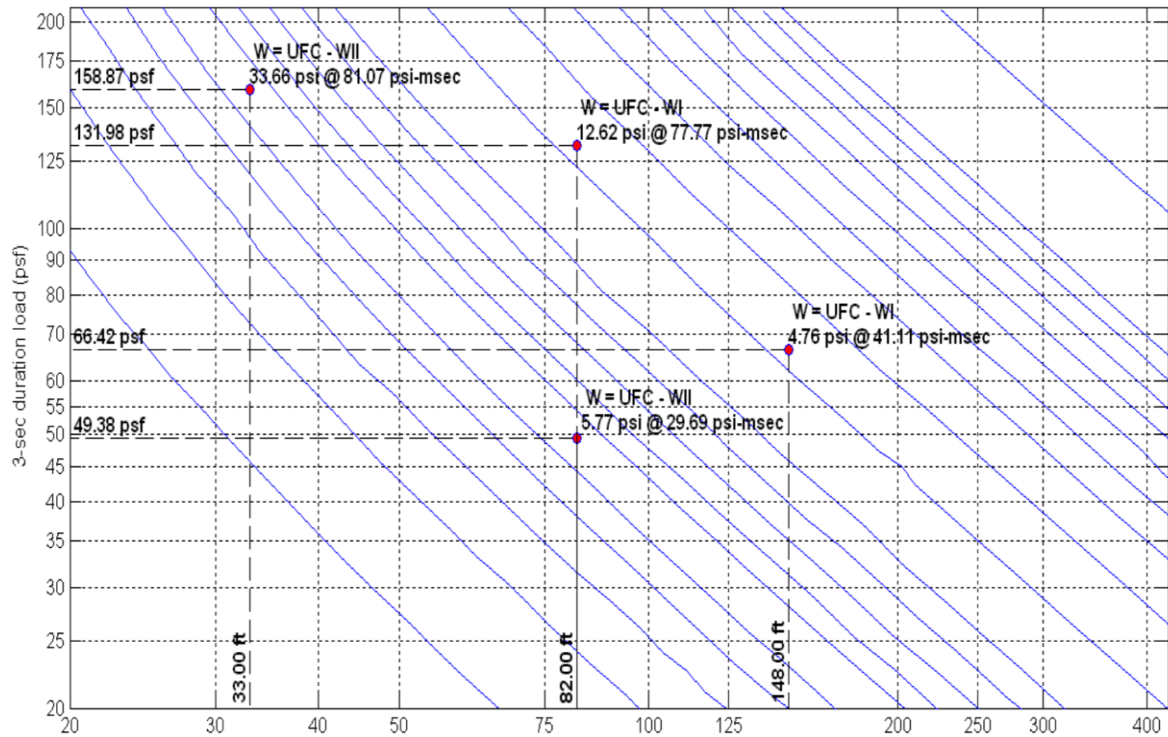


Figure 2.32: The design load values (UFC 4-010-01)

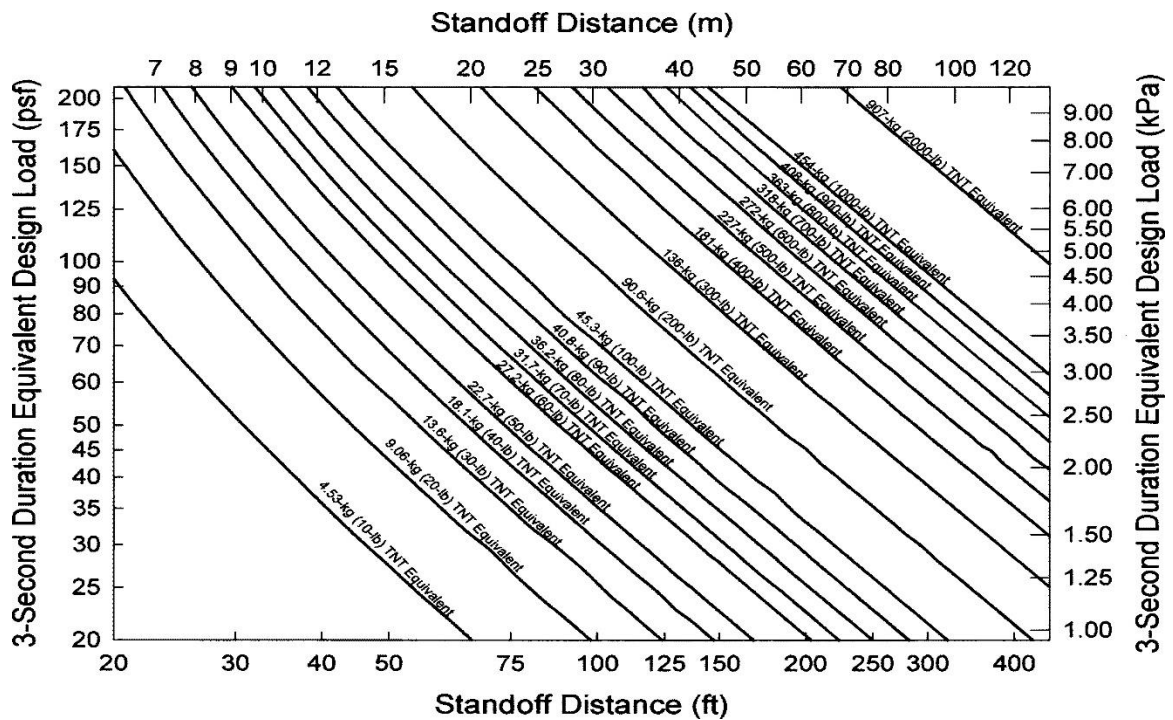


Figure 2.33: Graphical Relationship Between Standoff Distance, TNT Charge Mass, and 3-Second Equivalent Design Load (ASTM F2248-12)

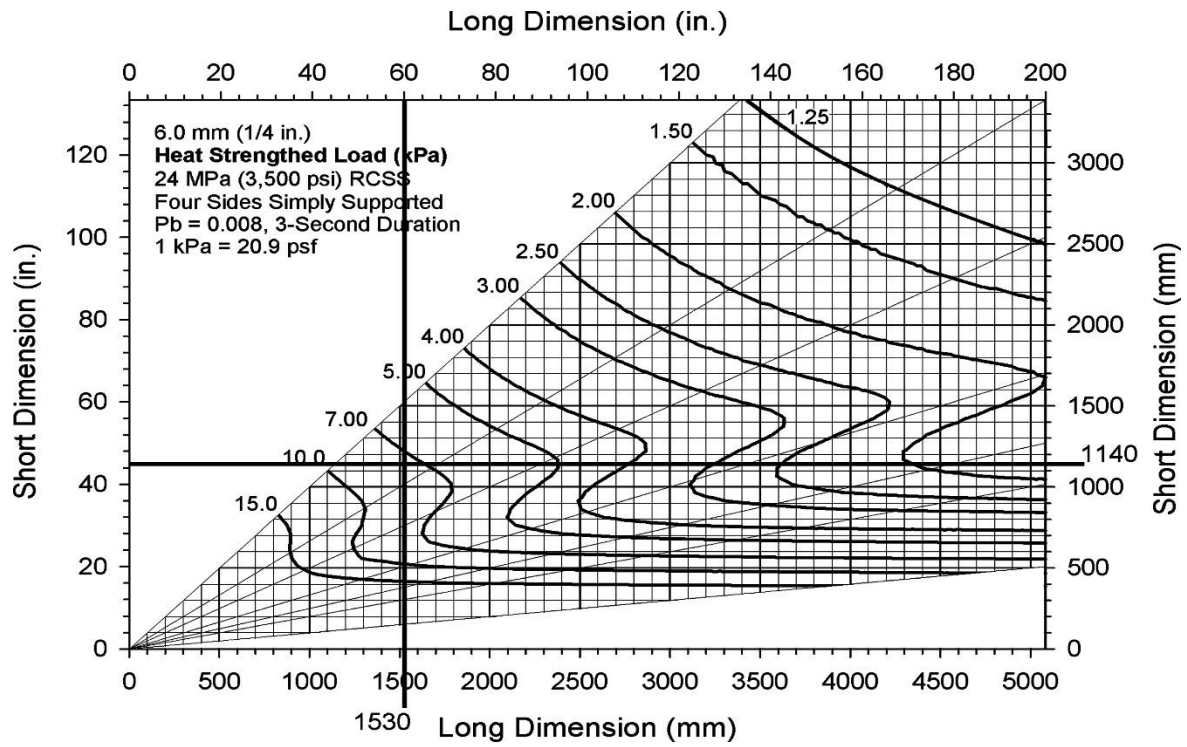


Figure 2.34: Non-factored load chart for 6mm annealed glass with four sides simply supported (ASTM 1300)

Part II

Experimental research

CHAPTER 3. Experimental Investigation of Improved and Retrofitted Connections for Blast-Resistant Curtain Walls

Abstract

A comprehensive experimental research program was carried out on blast-resistant curtain wall connections using a shock tube. The first phase consisted of three simulated curtain wall specimens with retrofitted connections between the vertical mullions and horizontal transoms. Different connection strengthening techniques were used. Some may be utilized in existing buildings and some may be applicable to new structures. The specimens had either hardened or unhardened mullions and transoms. They were tested under different levels of uniform blast loads applied on the exposed wall surface. Use of steel U-brackets connected to the mullions, inserted into the transoms, strengthened the mullion-transom connection and eliminated connection failure. The use of additional side brackets had little effect on the connections and led to the challenges of external connections at flexure critical locations of mullions. However, they performed well when used on hardened mullions, with proper connection to the steel hardening plates, which resulted in 52% reduction of mullion deflections. These tests were followed by eight single mullions tested to assess the performance of their connections to concrete slabs. Four of the mullions were connected to concrete slabs by means of brackets. The remaining four were connected to concrete by means of dampers that consisted of either rubber pads or spring systems. Blast loads were applied through a load application device that would collect the pressure induced at the end of the shock tube and transfer the load uniformly to the flange surface of the mullions along their heights. The curtain walls and single mullions were instrumented to measure mid-height deflection. The concrete beams were equipped with strain gauges placed on steel anchor bolts to assess the reactive connection forces. Mullion to concrete connections by means of steel brackets resulted in satisfactory connection strength, especially when brackets were placed on the side of the mullion. It was also observed that the use of dampers at the mullion-concrete substrate

connections mitigated damage to the curtain wall, with better performance achieved when a spring system was used.

Keywords: Blast loads, blast resistant curtain wall, curtain wall connection, mullion-to-transom connection, dampers, shock tube testing.

3.1 Introduction

Maliciously intended terrorist bomb attacks and accidental explosions have raised awareness for the need to design civilian structures to resist blast load effects. Structural and non-structural elements of building envelopes, including glass windows and curtain walls are high-risk elements when subjected to blast shock waves. Building façade is the most vulnerable component of a building when exposed to spherical or semi-spherical external blast shock waves and deserve particular attention. The focus in the current research project is placed on curtain wall response to such dynamic loads, especially their connections, which tend to be critical. Critical connections in a curtain wall are located either between the vertical elements (mullions) and the horizontal elements (transoms), or at the mullion-to-substrate connection regions. Unlike the conventional structural design loads for which design codes and standards have been developed, blast loads on curtain walls and their response to such loading are not widely available in the literature, especially as they affect their connections.

Significant research has been conducted over the past four decades to determine methods of predicting the characteristics of blast loads and analysis of structures under such loads (Mays et al. 2003; Bangash et al. 2006; Krauthammer 2008; Dussenberry 2010; Uddin 2010). However, research on blast-resistant curtain walls is scarce in the literature. A limited number of experimental and analytical research projects have been conducted providing limited information (Kirk et al. 2016, Benjamin et al. 2013, Anderson and Dover 2003, Adam et al. 2015, Timothy et al. 2015, Chiara and Claudio 2012). A comprehensive research program was undertaken at the University of Ottawa on curtain wall performance under blast loads, including tests of curtain wall components using a Shock Tube (Saatcioglu 2016, Saatcioglu et al. 2017a, 2017b, Saatcioglu et al. 2020 and Saatcioglu 2020). The research program involved

tests of two different types of large-scale curtain wall components. The first type consisted of two double-pane glass panels separated by a horizontal mullion and supported by two vertical mullions on either side and two horizontal mullions at the top and bottom. The objective was to establish blast hardening requirements for existing curtain walls, as well as to generate design information for new curtain walls. The first series of tests was conducted on curtain walls protected by externally placed Guardian Coil as a protective material (Saatcioglu et al. 2017a). One of the curtain walls was hardened by protective security film, mechanically glazed, while the connections between the mullions and the transoms were strengthened by means of aluminum angles (Saatcioglu et al. 2017b). The tests demonstrated that the use of Guardian Coil can be partially successful and had the disadvantage of reduced esthetic appearance and functionality. They further exhibited vulnerability of the connections. The first attempt to strengthen the connections involved the use of drill through bolts, which damaged the mullions at their flexure-critical mid-height regions, leading to the pre-mature failure of the mullions under blast loads. Therefore, in the second series of tests the angles were epoxy glued on the mullions and transoms at the connections using marine epoxy (cold weld). This resulted in satisfactory performance without failure (Saatcioglu et al. 2017b) for the threat level considered. The tests exhibited weaknesses in the connections. Another series of tests were conducted on laminated blast-resistant curtain walls to investigate the suitability of connecting the wall panels to top and bottom concrete slabs by extending and casting the interlaminates in the concrete at the top and bottom (Saatcioglu 2016). The second type of test specimens consisted of individual mullions. They were tested to investigate hardening options for existing aluminum mullions by means of steel plates. Three different sizes of aluminum mullions with three different hardening techniques were tested. They consisted of i) steel side plates, ii) steel L-shaped plates covering the web and the tension flange, and iii) addition of a square steel HSS profile attached to the tension flange with steel side plates (Saatcioglu et al. 2020). The results indicated an increase in mullion load capacity and a reduction in deflections. It was shown that the steel hardening components developed full composite action with the existing aluminum sections, indicating the effectiveness of the hardening technologies employed.

There is limited guidance provided on design of blast-resistant curtain walls and their connections in current standards and guidelines. ASCE 59-11(2011) and CSA S850-12 (2012) provide requirements for glazing systems, including glass details, frames and connections. Satisfactory performance is required to be shown either by providing independent third-party test results or analysis. When tests are performed, they are required to conform to ASTM F1642, GSA-TS01 or ISO 16933. The analysis is to be performed using established methods that take into account the dynamic nature of the blast loading. Similar requirements are outlined in the DoD Unified Facilities Criteria (UFC3-340 2008). In addition, the Canadian Explosives Research Laboratory (CERL) provides guidance for blast testing methodologies and hardening of curtain walls (CERL 2015). It is asserted that the best practice to strengthen the curtain walls and office windows is by utilizing PVB layers as well as film protected glass. Furthermore, vertical and horizontal sections are recommended to be hardened and retrofitted by increasing their rigidity.

Both mullion to transom, and mullion to substrate connections were investigated experimentally. The first phase (Phase I) of the experiments focused on improving the connections between the vertical mullions and horizontal transoms. They consisted of tests of three curtain wall assemblies with two glazed glass panes, simulated by an equivalent steel sheet, separated by two mullions and three transoms. The blast load was applied on the entire surface of the walls. Each curtain wall specimen had a different connection strengthening technique. These tests were followed by 9 single mullion tests (Phase II) to assess the performance of their connections to the substrate represented by concrete slab edges/beams. Blast loads were applied through a load application device that would collect the pressure induced at the end of the shock tube and transfer it uniformly to the flange surface of the mullions. These tests consist of 4 mullions that were rigidly bolted on to the concrete beams, and 5 mullions connected to the beams at their ends by using dampening materials. All specimens were instrumented to measure deflections and strains. High-speed cameras were used to record the position of the curtain wall and mullion components during response.

3.2 Testing Facility

The blast testing of curtain walls was conducted at the University of Ottawa's Blast Research Laboratory. This research laboratory provides a pneumatically driven shock-tube to simulate blast shock waves. The shock waves have a maximum reflected pressure capacity of 150 kPa and durations ranging between 5 ms to 50 ms. Figure 3.1 illustrates the shock-tube facility at the University of Ottawa. The driver section of the shock tube has six steel pipe sections having an inner diameter of 597 mm and a steel wall thickness of 6.35 mm. Two sections have a length of 1525 mm, one with 915 mm length, one with 610 mm length and the last two have 305 mm length. The driver length can be adjusted from 305 mm to 5185 mm with increments of 305 mm to control the reflected pressure and impulse. A shock wave inside a shock tube can be generated by the buildup of compressed air charged by a compressor, which bursts aluminum diaphragm(s) provided between the driver and expansion section. Further details of the Shock Tube are provided elsewhere (Lloyd, 2009).

3.3 Data Acquisition and Instrumentation

The instrumentation consisted of pressure sensors, Linear Variable Displacement Transducers (LVDTs), two high-speed cameras with image processing software, and a Data Acquisition System (DAS) to collect the data. Figure 3.2 exhibits the LVDTs and the high-speed cameras used. The pressure sensors were installed on the interior face of the shock tube near the end of the expansion section close to the test area. They were triggered by the shock wave, which activated the DAS for data collection. Similarly, the high-speed cameras were triggered based on the signals received from the pressure sensors. The pressure and displacement transducers were connected to a computer located away from the shock tube for remote operation. The reflected pressure captured by the pressure sensors and the displacements measured by the LVDTs were transferred to DAS for processing. In addition, the strain readings measured on the steel bolts cast in concrete beams were recorded by means of 120-ohm strain gauges.

3.4 Curtain Wall Test Specimens

3.4.1 Specimen Description

A total of three full-size curtain walls were prepared for testing in the first phase of experimental research. Figure 3.3 shows the geometric characteristics of each curtain wall. The specimens were built using hollow aluminum sections (6061-T6). The two vertical mullions had 127 mm $\times$ 50 mm (5" $\times$ 2") cross-sectional dimensions and 2.44 m (96") height. The transom's section size was 76 mm $\times$ 50 mm (3" $\times$ 2") with 1.93 m (76") length. The thickness of aluminum sections was 3 mm. Figure 3.4 shows a typical curtain wall specimen during construction. They were built and assembled in the Laboratory. The hollow aluminum sections were first cut to required sizes, and then mullions and transoms were connected to each other by means of steel U-brackets, which were first bolted on vertical mullions by means of 4 self-drilling screws tapped and then inserted on horizontal transoms at each of the top and bottom ends. It should be noted that the use of steel U-brackets was one of the retrofit techniques considered to strengthen the connections. Figure 3.5 exhibits the steel U-brackets used. The glass panes between the mullions were simulated by gauge #22 steel sheets having 0.9 mm thickness representing blast resistant glass (glass + protective film). The steel sheet had an equivalent mass to a glazed blast resistant IGU. Two steel sheets having 2.03 m $\times$ 1.02 m surface area were used for this purpose. They were anchored to the mullions and transoms using rivets with a 100 mm spacing. It should be noted that the pre-cracking behavior of glass was not simulated as the resistance of glazed glass in the post-cracking region governed response for the curtain walls considered is insignificant. For the middle transom, which had a common area for top and bottom steel sheets, a zigzag pattern was employed for anchoring. A specific type of rivet (12-24 x 1-1/4" Strong-Drive® Self-Drilling X Metal Screw, 5/16" Hex Washer Head, #5 Point) was used for all anchorage systems during the experiment. The test specimens were fixed to the end of the shock tube expansion section by means of four hollow structural steel sections (HSS) having a 203 mm by 203 mm (8" $\times$ 8") cross section. Table 3.1 summarizes the geometric properties of all the test specimens.

The mullion-to-transom connections for Specimen CW1 were strengthened using the same technique used in one of the curtain walls tested in an earlier phase of the same research

program (Saatcioglu et al. 2017a) but was subjected to a higher pressure-impulse combination. The strengthening involved the use of aluminum L-angles ($51 \times 51 \times 3$ mm) that were epoxy glued on the joints. This was achieved by using 25-mL of LePage Marine Epoxy Adhesive. Figure 3.6 shows Specimen CW1 and the glued L-shape aluminum angles. Specimen CW2 consisted of hardened mullions, connected to the transoms by means of L-shaped aluminum angles bolted on the mullions and the middle transom. The mullions were hardened by two L-shaped steel plates with longer side of the “L” fixed on mullion webs as depicted by Figure 3.7. Specimen CW3 consisted of hardened mullions and transoms with strengthened connections. The mullions and transoms were strengthened with two L-shape steel plates riveted on each element. They were connected to the hardened mullions by means of six rivets. Figure 3.8 exhibits the retrofitting plates and the L-angles used to strengthen the third specimen. The curtain wall properties were selected to be representatives of those used in practice. The specimens were prepared and assembled prior to being secured on the shock-tube frame for testing. They were clamped on the shock tube frame by means of 203 mm by 203 mm (8"×8") HSS sections at the top and bottom. Steel angles having 51 mm by 51 mm (2" x 2") cross-sectional dimensions and 400 mm length were used between the shock tube frame and the HSS sections to provide edge support at the top and bottom. Figure 3.9 illustrates the simple supports for the mullions. All three curtain walls were instrumented with two Linear Variable Differential Transducers (LVDT), one placed at mid-height of left mullion, and the other at mid-length of middle transom to monitor deflections in the direction of blast pressure. For the full-size curtain walls, the blast pressure impulse combination followed the preceding curtain wall tests conducted at the University of Ottawa (Saatcioglu et al. 2017a). The pressure impulse combination for single mullions was adopted from the previous phase of the same overall test program as per the Design Basis Threat (DBT) defined by the project sponsors.

3.4.2 Test Results

The first curtain wall (CW1) was subjected to a single shot with a pressure-impulse combination of 50 kPa – 495 kPa-ms as shown in Figure 3.10. The blast shot resulted in excessive deformations in each member, causing the vertical mullions to rupture at their critical

mid-height locations. During the test, the mid-height vertical mullion LVDT was damaged due to excessive deflections, and the second LVDT resting on the horizontal transom could not capture the deflections accurately. Therefore, the deflections were obtained from the high-speed cameras. The maximum deflection at mid-height of vertical mullion and the mid-length of horizontal transom was measured to be 351 mm. Figure 3.11 shows the damage induced on CW1 at the end of the test. It was observed that the mullion-transom connections were not damaged. However, the epoxy-glued angles on the connections were detached and flew away during the test. The test video indicated that the side brackets got detached early in response and had little effect on the connection strength. The favorable performance of the connections was attributed to the U-shaped steel connectors inserted inside the connection during the manufacturing process in the laboratory. This type of connection is believed to be stronger than the commercially used connectors with small size rivets that suffered damage in earlier tests.

The second curtain wall (CW2) was subjected to a single blast shot with a pressure-impulse combination of 36 kPa – 270 kPa-ms as shown in Figure 3.12. This time the vertical mullions, as well as the mullion-transom connections performed well because of the steel hardened mullions. However, the lack of strengthening in the middle transom made this component vulnerable, resulting the mid-length rupturing of the transom after developing 18 mm of maximum deflection. Figure 3.13 exhibits CW2 after the blast test. The failure of the transom led to increased membrane action with large in-plane deformations in the steel sheets, which resulted in the detachment of the steel sheets from the middle transom. This is evident by the deformed shape observed in Fig. 3.13.

The third curtain wall (CW3) was hardened with the L-shaped steel plates used for CW2, except that this time both the vertical mullions and the horizontal transoms were strengthened. The mullion-transom connections were also strengthened by means of steel angles (side brackets) riveted on the connections. This curtain wall was subjected to incrementally increasing three shots. The first shot had a pressure-impulse combination of 46 kPa – 483 kPa-ms. It resulted in a maximum displacement of 167 mm as recorded by the LVDTs. Figure 3.14 depicts the pressure-time and deflection-time graphs. The steel hardening

increased the capacity and the curtain wall performed well. The steel plates showed buckling on the critical compression side. The inner aluminum core (original mullions) also suffered from the buckling of the compression flange. In fact, at this level of load, the damage was limited to local buckling of the aluminum mullion and the steel plates at mid-height. Otherwise, the hardening technique performed well, without any distress in the connecting angles. The steel corner angles performed well in strengthening the connections. Figure 3.15 shows the third curtain wall at the end of the first shot. The specimen was then subjected to increased blast load as the second shot. The gaps between the specimen and shock-tube that resulted from the bending of the specimen during the first shot were covered by pieces of wood to prevent leakage of pressure in subsequent shots. The second blast shot was more intense with a pressure-impulse combination of 50 kPa – 575 kPa-ms. The additional maximum deflection induced at mid-height of mullions was 133 mm. Figure 3.16 depicts the pressure-time and deflection-time graphs after second shot. This shot caused cracking of the steel plates in the maximum positive bending region at mid-height, and the L-shaped hardening plates were slightly fractured. Figure 3.17 shows CW3 after the second shot. The third shot was applied as the final test on CW3. This load was intended to force the curtain wall to observe the complete failure mode. The third blast shot had a pressure-impulse combination of 61 kPa – 671 kPa-ms. Figure 3.18 depicts the pressure time history of the third shot. This was sufficient to cause the failure of the vertical mullions by rupturing and buckling of the steel plates on tension and compression sides, respectively. The vertical mullions pulled out of the supports and deflected excessively as indicated in Figure 3.19. The crack width in the mullion was measured to be 85 mm. The LVDT that was placed on the mullion slipped away and could not record deflections. Therefore, the high-speed camera was used to capture the additional deflection relative to that experienced after the second shot as 132 mm. In addition, rivet failures were observed in the top-right and mid-length segments of the middle transom. There was no damage observed near the support regions. The maximum in-plane displacement was reduced by 50% by retrofitting all components of the curtain wall.

3.5 Single Mullion Test Specimens

3.5.1 Specimen Description

The second phase of the experimental research involved tests of individual aluminum mullions, connected to reinforced concrete beams representing the edges of supporting slabs. The mullions were connected to reinforced concrete slabs either at the front edge of the slab with brackets on either side of the mullion web, or at the top and bottom of the slab attached to the mullion flange. The connections were used either at the ends of a floor-to-floor high vertical mullions (single mullion configuration) or at mullion mid-height to simulate continuous mullions over a slab, extending towards the floors above and below (continuous mullion configuration). All the mullions had a hollow rectangular cross-section with outside dimensions of 127 mm $\times$ 51 mm (5 in by 2 in), aluminum wall thickness of 3 mm and a length of 2.4 m. They were placed in front of the shock tube with a test area of 2.0 m by 2.0 m, covered by the load application device that collected the applied blast pressure and distributed over the height of the mullion as a uniformly distributed linear load. Two reinforced concrete beams were cast as representatives of slabs of a given storey. These beams had holes matching the holes on the shock tube frame to bolt them on to the shock tube frame. The concrete beam section size was 200 mm $\times$ 150 mm (8" $\times$ 6") with 2.54 m (100") length. Six longitudinal rebars were used in each beam, consisting of four M-25 and two M-20 bars with 10M stirrups placed at 150 mm spacing. Figure 3.20 illustrates the reinforcement cage for the concrete beams. Ready-mix concrete was used to cast the beams. The average concrete strength was 35 MPa as established by standard cylinder tests. Grade 8 bolts with 10 mm diameter, 110 mm length, 1000 MPa tensile capacity and 0.0047 yield strain were cast in the beams to be able to secure the steel brackets for mullion connection. These bolts were positioned to permit "slab-end" connections, as well as top and bottom slab connections. Each bolt was instrumented by 120-ohm strain gauges to provide strain data during the test. One flange of each bracket was connected to the concrete beam by using nuts and washers while the other flange was anchored to the mullion by means of rivets. Figure 3.21 shows the positions of the concrete beams schematically. Four of the nine mullions tested were rigidly connected to the beams with fixed

end conditions. The remaining five mullions had dampers at the connections and permitted deformations perpendicular to the beams. The specifics of each mullion tested and their support conditions are described below:

M-C1: Mullion M-C1 was connected at the ends using brackets attached to the front end of the concrete beam (slab) and the two opposite sides (web) of the mullion as shown in Figure 3.22. It represented a curtain wall mullion spanning over a storey height. The brackets were bolted to the slabs by means of two 10-mm bolts that had been cast in concrete. Two brackets were used at each end of the mullion, one on either side. They were attached to the mullions by means of 8 self-tapping metal screw rivets.

M-C2: Mullion M-C2 was connected at the ends using brackets attached to the top and bottom of concrete beam (slab) and the narrow side (flange) of the mullion as shown in Figure 3.23. This specimen also represented a curtain wall mullion spanning over a storey height. The brackets were bolted on the slab by means of 10-mm bolts that had been cast in concrete; hence two brackets were used at each end. They were attached to the aluminum mullion by means of 6 self-tapping metal screw rivets.

M-C3: Mullion M-C3 was connected at mid-height over the concrete beam (slab) and connected using brackets attached to the top and bottom of the concrete beam and the narrow side (flange) of the mullion as shown in Figure 3.24. This was a single vertical mullion simulating continuous mullion over a reinforced concrete slab. The brackets were bolted on the slab by means of 10-mm bolts that had been cast in concrete. They were attached to the aluminum mullion by means of 6 self-tapping metal screw rivets. The mullion was clamped by 150 mm HSS sections at the top and bottom, which created full fixity. Hence, the mullion consisted of two fully fixed spans over the height.

M-C4: Mullion M-C4 connected at mid-height over a concrete beam (slab) and connected using brackets attached to the front end of the concrete beam and two opposite sides (web) of the mullion as shown in Figure 3.25. MC-4 was a single vertical mullion simulating a continuous mullion over a reinforced concrete slab like M-C3. However, this time the connection details were different. It was connected to the beam by means of steel brackets

using 10-mm bolts that had been cast in concrete, where the brackets were attached to the web of the aluminum mullion by means of 6 self-tapping metal screw rivets. The mullion had fixed supports at the top and bottom ends, clamped between 150 mm square steel HSS steel sections and the shock tube frame.

M-C5: Mullion M-C5 simulated a continuous mullion over a supporting concrete slab, similar to MC-4, except it was equipped with an XRD rubber between the mullion and the concrete beam as a damping material. The intent was to investigate the possibility of reducing the effects of blast loads on the supporting connection. The rubber dimensions were 216 mm × 279 mm (8.5 in × 11 in) with 12.7 mm (0.5 in) thickness. The mass density of XRD rubber was 320 kg/m³ (20 lb/ft³). The XRD rubbers were compressed during the assembly to 10 mm thickness to maintain tightness of the connections. Four holes were drilled in the rubber to accommodate the bolts that connected the brackets to the concrete beam. Figure 3.26 shows M-C5 with inserted XRD rubber between the mullion and the concrete beam. Each bracket was connected to the concrete beam and the mullion by means of two bolts and eight rivets on either side, respectively. Two LVDTs were installed to record mullion deflection at mid-height of top and bottom spans.

M-C6: Mullion M-C6 was similar to M-C3 with brackets attached to the concrete beam on the top and bottom, with XRD rubber inserted between the mullion and the supports. This specimen needed more pads compared to the M-C5 due to the larger support area. The same instrumentation as the previous tests was implemented. Both brackets were connected to the narrow side of the mullion (parallel side) by using six rivets. Figure 3.27 illustrates M-C6. The inserted rubber allowed M-C6 with 10 mm space to further compress during the application of blast loads in the direction of the applied load.

M-C7: Mullion M-C7 consisted of a single mullion connected to the concrete beams at the ends through dampers. Each spring system consisted of two 10 mm steel plates welded to the ends of a spiral spring. The spring was manufactured in the laboratory using an 11.3 mm diameter reinforcing steel with 4 successive turns, resulting in 220 mm initial length, which was reduced to 160 mm when compressed to have a tight connection during the installation.

The spring stiffness was measured to be 8 N/mm for each spring. Two brackets were connected to the body of the shock-tube, one on each side of the mullion to guide the mullion as it moves in the direction of the load without any tilting. There was a gap of approximately 1.0 mm between the mullion ends and the side brackets to prevent friction. Figure 3.28 illustrates M-C7 prior to testing.

M-C8: Mullion M-C8 was the same as M-C7, except this time the spring stiffness at the supports was increased 30 N/mm. This stiffness was obtained by adding an additional smaller diameter spring (with a stiffness of 22 N/mm) inside the previously used spring (having stiffness of 8 N/mm). The total net length of the springs prior to testing was 160 mm. The same setup and instrumentation as those used for M-C7 was also used for this mullion. Figure 3.29 b shows the end springs.

M-C9: Mullion M-C9 was similar to M-C8 with two springs provided on each end. A smaller diameter spring with a stiffness of 15 N/mm was enclosed in the larger diameter spring used in the previous tests. The total stiffness was in between those used for M-C7 and M-C8, with a stiffness value of 23 N/mm (8 N/mm + 15 N/mm). The available spring length was increased to 200 mm in an anticipation of a longer travel distance needed during the blast load. Figure 3.29c shows the end springs for M-C9. Figure 3.30 indicates the location of LVDTs for each phase of the experiment based on the specific test configurations.

3.5.2 Test Results

The first mullion (M-C1) was subjected to 22 kPa reflected pressure and 248 kPa-ms reflected impulse. The specimen failed by rupturing of the mullion at its critical mid-height location. The failure of the mullion at mid-height location also resulted in the rupturing of the instrumentation wire attached to this location for LVDT measurement. Therefore, no deflection reading was taken for this mullion. However, image processing of the side video taken during the test allowed for the estimation of maximum deflection as 85 mm and residual deflection as 45 mm. Figure 3.31 illustrates the pressure-time relationship and the deflection measured during the test. The mullion to concrete connections remained in tack during the test, without any sign of distress. Figure 3.32 shows the failure of the mullion. The strain gauges on the bolts

embedded in concrete recorded a maximum strain of 0.1%, which is significantly below the yield strain of 0.47%. The mullion-to-slab connection used for this specimen was judged to have sufficient strength and may be considered to be oversized because of the relatively low flexural capacity of the mullion.

The second mullion (M-C2) was subjected to a single shot with a pressure-impulse combination of 28 kPa – 285 kPa-ms, which produced a maximum deflection of 37 mm. The pressure and deflection time histories are shown in Figure 3.33. The mullion suffered from local buckling of the compression flange at mid-height. This is shown in Fig. 3.34. The connections remained intact, although slight bending of the brackets was observed at the bottom of the top slab and at the top of the bottom slab, where the mullions were bearing against the brackets. The top two bolts embedded in concrete of the slab above were instrumented with strain gauges to assess stresses in the bolts. They recorded extremely low readings, indicating that they remained elastic.

The third mullion (M-C3) was subjected to one blast shot with a pressure-impulse combination of 72 kPa – 905 kPa-ms. The LVDT placed at mid-height of the upper extension showed 33 mm deflection as depicted in Figure 3.35. It should be noted that M-C3 suffered from connection failure when the brackets were bent excessively, reducing the original 90-degree angle to approximately 45 degrees. The concrete slab bared against the mullion flange, crushing the aluminum mullion as illustrated in Figure 3.36. Clearly, this kind of mullion-to-concrete slab connection did not fulfill its function as expected. Bending of the brackets strained the bolts that were used to connect the mullion, excessively. The strain gauges placed on the bolts indicated yielding of the bolts, developing 1.67% and 2.7% strains, far in excess of the yield strain of 0.47%. This level of straining indicated axial tension resistance of the connecting bolts to be approximately equal to 78.5 kN each. The mullion experienced slight compression buckling at mid-height of the upper extension. Upper extension developed higher deflection than the lower extension, with a maximum displacement value recorded as 33 mm at mid-height. It should be noted that this displacement value includes the effect of the movement of the mullion as a result of the crushing of the aluminum at the concrete connection

level, which was measured to be approximately 44 mm. The effect of this displacement on mid-height deflection measurements of the upper and lower segments can be taken as half of the value recorded at the support, i.e., $44/2 = 22$ mm.

The fourth mullion (M-C4) was subjected to two blast shots. The initial shot was intended to be the same as that for M-C3. However, there was a leak in the system and the pressure could not be increased above 60 kPa during the first shot. The first shot was fired at 60 kPa reflected pressure and 750 kPa-ms reflected impulse. It did not generate significant damage, neither in the mullion nor in the connection brackets. The maximum mid-height mullion deflection was recorded to be 8 mm in the lower segment of the mullion (below the concrete slab connection) with a residual deflection of 4 mm. Fig. 3.37 and Fig. 3.38 present the recorded data and the performance of M-C4 after the first shot. The bolts connecting the brackets to the concrete slab remained elastic. Strain gauge readings indicated the maximum strains of 0.07% and 0.16%, below the yield strain of 0.47%.

The second shot was applied to Mullion MC-4 at the intended higher pressure-impulse range. The second shot generated 70 kPa reflected pressure and 812 kPa-ms reflected impulse, and the LVDT reading on the lower segment below the slab indicated a maximum deflection of 13 mm at mid-height. Figure 3.39 depicts the reflected pressure and deflection time history. There was no damage induced on the connections. A slight loosening of the bracket was observed on the left bracket. Figure 3.40 illustrates views of the mullion and its connections after the second shot. According to the strain gauge readings, the bolt strains remained elastic with a maximum strain of 0.15%. The comparison of the connections used for Mullions MC-3 and MC-4 indicates that the connection for MC-4 showed superior performance, without any damage to the mullion and the bracket, unlike the connection for MC-3, which suffered from bearing failure. In the current test, the bracket leg attached to the end of the slab increased the bearing surface area, preventing crushing of the aluminum. Furthermore, the bracket leg attached to the mullion sides strengthened the mullion in the web, which also helped prevent the crushing of aluminum under bearing stresses. The brackets maintained their rigidity as the rigid concrete slab prevented the bending of the brackets, unlike what was observed in the

connection for MC-3. It can be concluded that this type of connection can be recommended for blast resistance.

The fifth mullion (M-C5) was tested with side brackets while XRD rubber was inserted between the mullion with attached brackets and the concrete beam. M-C5 was subjected to a single shot with a pressure-impulse combination of 71 kPa – 830 kPa-ms. The LVDT reading on the lower segment below the slab indicated a maximum deflection of 21 mm. Figure 3.41 illustrates the reflected pressure and deflection time history. By inserting dampers between the components, better performance was observed in M-C5 compared to the rigidly connected companion mullion (M-C4). However, the thickness of the rubber was not sufficient to significantly reduce the damage on the specimen. There was no damage induced on the connections, and the compression buckling of the mullion was reduced. Figure 3.42 shows M-C5 after the test. Unlike the rigidly connected companion test, loosening of the bracket was not observed on any of the brackets because of the reduced stress transfer to the brackets enabled by the presence of the rubber. As in the case of rigidly connected side brackets (Mullion MC4), the bracket leg attached to the end of the slab increased the bearing area, preventing the crushing of aluminum. As before, the bracket leg attached to the mullion sides strengthened the mullion in the web, which also helped prevent the crushing of aluminum under bearing stresses. The strain gauge readings proved that the bolt strains remained elastic, with a maximum strain of 0.16%. No failure was observed at the support; however, web buckling occurred at mid-height of the mullions above and below.

The sixth mullion (M-C6) was tested with inserted XRD rubber and brackets on top and bottom of the concrete beam. It was subjected to one shot with a pressure-impulse combination of 73 kPa – 912 kPa-ms, and the LVDT reading on the lower segment below the slab indicated a maximum deflection of 36 mm. Figure 3.43 illustrates the reflected pressure and deflection time history. No significant improvement was observed in M-C6 by inserting thin layers of rubber. Like the companion test for rigidly connected mullion, M-C6 suffered from connection failure when the brackets were bent, reducing the original 90-degree angle to approximately 50 degrees. The concrete slab bared against the mullion flange, crushing the

aluminum mullion as illustrated in Figure 3.44. Bending of the brackets strained the bolts that were used to connect the mullion. Furthermore, the mullion experienced slight compression buckling at mid-height of the upper and lower extensions with a maximum displacement value recorded as 36 mm at mid-height. It should be noted that this displacement value includes the effect of the movement of the mullion as a result of the crushing of the aluminum at the concrete connection level. No failure was observed in the supports adjacent to the HSS sections.

The seventh mullion (M-C7) with springs on each end (having a stiffness of 8 N/mm) was subjected to a pressure-impulse combination of 24 kPa – 245 kPa-ms. The mullion displaced horizontally up to 134 mm as springs were compressed elastically while the mullion also remained elastic without any damage. Figure 3.45 shows the plotted experimental data. Figure 3.46 shows M-C7 after the blast loading. It demonstrated the benefits of using dampers at curtain wall-substrate connections. It is worth recalling that M-C1 (single mullion without the dampers) suffered failure under the same reflected pressure; however, M-C7 did not suffer any permanent deflection or failure even after was subjected to two similar blast shots.

The eighth mullion (M-C8) with double springs (with increased stiffness of 30 N/mm) was subjected to a higher magnitude of a blast shot with a pressure-impulse combination of 31 kPa – 312 kPa-ms. The mullion experienced 172 mm and 114 mm maximum and residual deflections, respectively. This is indicated in Figure 3.47. The spring travel length and stiffness were not sufficient to protect the mullion against the higher blast load. Consequently, a significant compression buckling was observed at mid-height. During the blast loading the springs at the supports were compressed completely resulting in a pin support during the subsequent response. Therefore, excessive deformations were developed in the mullion, which led to failure. This is presented in Fig. 3.48. No damage was observed at the support connections. M-C8 showed the importance of spring length and stiffness for damper design.

The ninth mullion (M-C9) was intended to test the end damper concept with springs having an intermediate value of stiffness (23 N/mm), while blast loads were gradually increased, using the experience gained from the previous two tests. The first shot was at a pressure-impulse combination of 22 kPa – 192 kPa-ms. This shot compressed springs at the

supports up to 192 mm, while the mullion remained elastic. The recorded test data are plotted in Fig. 3.49. After the first shot it was observed that the welds that connected the smaller diameter inner springs broke at the end plate and became unstable. Therefore, no subsequent shots were applied, and the effects of spring stiffness and length were decided to be investigated analytically. Figure 3.50 shows M-C9 and the end springs after the test.

3.6 Discussion of Results and Conclusions

3.6.1 Mullion-to-Transom Connections

The main objective of the first phase of testing was to investigate mullion-to-transom connections of curtain walls for blast resistance. Earlier research of the same overall experimental program involved tests of ordinary curtain walls, primarily designed for wind load effects, glazed by protective film afterwards for blast resistance (Saatcioglu et al. 2017a). These curtain walls suffered from premature failure of the mullion-to-transom connections. Their connections were built using aluminum U-brackets, riveted to the mullions, which were then inserted into the transoms at the connection locations. They were proven to be the weak points in resisting blast loads. The transoms separated from the rest of the curtain walls during blast loading and flew away prematurely prior to any damage induced on the rest of the curtain wall elements after the cracking of the glazed glass panes. In subsequent tests, L-shaped aluminum angles were provided as a connection strengthening technique (Saatcioglu et al. 2017b). The angles were either bolted or epoxy glued on the mullions and transom flanges at their connections. When bolted, they damaged the mullions at their critical mid-height tension locations, resulting in the rupturing of the mullions at these locations. When epoxy glued at the connections, they performed well under low magnitudes of pressure-impulse combinations. When bolted on the transom and mullion web, the connections failed. The curtain walls tested in the current research (Phase I) were manufactured in the Blast Research Laboratory of the University of Ottawa as discussed in Section 3.4.1 and had stronger steel U-brackets bolted to the mullions and transoms as illustrated in Fig. 3.5. These connections were stronger than the riveted aluminum brackets typically used in the industry for wind resistance. Curtain wall specimen CW1 had the steel U-brackets as well as externally placed aluminum angles epoxy glued on the connection. The results indicated that the connection survived the applied blast

load, though epoxy glued angles detached during response to blast loads, indicating very little contribution to resistance. However, internally provided steel U-brackets provided sufficient connection strength, hence are recommended for blast resistant mullion-to-transom connections.

Specimen CW1 connections survived the applied blast load, but shifted the weak spot to the mullions, causing the rupturing and buckling of the mullion flanges under increased blast loads immediately above the connections. Therefore, Curtain Wall CW2 was hardened by covering the mullions with L-shaped steel plates. The transoms were not hardened because they did not develop excessive bending during the previous test and followed the mullions as they acted as primary load resisting elements. The connections were strengthened by means of externally bolted aluminum angles as illustrated in Fig. 3.7. This wall failed through the rupturing of the middle transom, indicating the need for hardening of this element as well if the curtain wall was to resist higher loads.

Curtain Wall CW3 mullions and transoms were hardened by means of L-shaped steel plates. The connections were also strengthened by steel angles bolted on mullions and transoms at their connections, as illustrated in Fig. 3.8. The connections of CW3 survived the highest blast loads applied in Phase I tests ($P_r = 61$ kPa and $I_r = 671$ kPa-ms) until the wall failure took place in the mullions.

The comparison of the three curtain wall specimens tested indicates that aluminum U-brackets typically provided for wind resistance is not sufficient to resist blast loads, making mullion-to-transom connections the weakest link in the chain. When possible, it is recommended that these brackets be replaced with steel U-brackets securely bolted on the adjoining members, i.e., mullions and transom. Enhanced blast resistance can be obtained by hardening the mullions and transoms, while also strengthening their connections by means of bolted angles.

3.6.2 Mullion-to-Substrate Connections

The objective of Phase II tests was to investigate the performance of mullion to concrete slab/beam connections under blast loads. Single mullions forming part of a curtain wall system

as the main structural element were tested using the shock tube. Two types of mullions were considered, either single mullions extending between the floor slabs or spandrel beams, or continuous mullions spanning over a floor slab, extending above and below. Reinforced concrete beams were used to simulate slab edge conditions. The mullions were connected to the concrete beams by means of two L-brackets at each end. The brackets were positioned either at the front face of the beams and connected to the flange of the mullion or secured on the top and bottom of the beams connected to the web of the mullion. Those bolted on the slab edge and mullion web (M-C1 and MC4) performed well without any damage to the connections and the failure was governed by the failure of the mullions (rupturing in tension and buckling in compression in their flexure-critical regions). When the brackets were attached to the front face of the beams and the flange of the mullions, the brackets bent under the blast loads but survived the test until the failure of the mullions (M-C2 and M-C3). However, M-C3 that was continuous over the floor slab/spandrel beam suffered from the crushing of aluminum under bearing stresses as the excessive bending of the brackets placed the burden on the bearing area of the mullion on concrete. Therefore, it may be concluded that, when possible, the connections should preferably be made with the brackets placed on the front face of the supporting concrete element.

The second half of the mullion-to-substrate connection tests were performed using flexible material between the mullion and the bearing area (in between the aluminum and the concrete). This was believed to dampen the blast load effects and reduce the burden on connections. The first two tests were conducted with XRD rubber material using continuous mullions over the concrete support (M-C5 and MC-6). These were companion to M-C4 and M-C3 that had rigid connections. While the presence of rubber damped the effects of the blast loads, the stiffness and the thickness of the XRD material used were not suitable for such applications. XRD rubber does not show perfectly elastic material under higher stresses and may offer the functionality as an energy absorbing material, the effects of which require further research. The current research indicates that thicker rubber with higher stiffness may be used as dampers in the connections of curtain wall systems. The behavior of the brackets in these

tests was similar to those observed in rigidly connected brackets with bearing failure of the aluminum over the supports reduced due to the presence of the rubber pads.

The following three mullion tests included elastic springs as support dampers. M-C7 through M-C9 were equipped with springs at the end supports having a range of spring stiffness between 8 N/mm and 30 N/mm and a spring length of 160 mm to 200 mm. The results showed favorable behavior of the use of springs. When the blast pressure was low, and there was sufficient spring length, neither the connection nor the mullion suffered damage (M-C7). However, when the blast pressure was high enough to compress the springs completely, the connection performed similar to those observed in rigidly connected brackets beyond this point. Nevertheless, the use of the spring dampers increased the overall resistance of the mullions. These tests proved that the concept of providing properly designed dampers at mullion-to-substrate connections offer improvements in response. This point is further researched analytically in the subsequent phase of research.

3.6.3 Summary of Conclusions

The following conclusions can be drawn from the experimental research conducted on curtain wall connections.

- Commercially produced curtain walls designed for wind load effects may not have sufficient mullion-to-transom connections when subjected to blast loads. Blast-resistant connections can be produced with the use internal steel U-brackets bolted on to the adjoining elements (mullions and transoms).
- The connections of curtain walls without considerations given to blast loads can be strengthened with the use of externally placed L-brackets at the connections to improve their blast resistance. These brackets, preferably made from steel, should not be bolted on the mullions at flexure critical locations to avoid premature rupturing of the mullions. The use of epoxy adhesion provides strengthening of the connections for low levels of blast loads. Under higher blast loads, mechanical connections should be used along with hardened mullions and transoms. Curtain walls hardened with steel plates,

with strengthened connections using bolted L-brackets provide superior blast protection.

- Steel plates secured on the aluminum sections can develop fully composite behavior until after significant inelastic deformations are developed, followed by the buckling of the plates between the self-tapping screws used for securing the retrofit plates. The self-drilling metal screws used for retrofitting, including the attachment of the corner angles performed well during the tests. No yielding and failure were observed until after the curtain wall was excessively damaged due to other causes, and even then, the damage to the screws was limited to a few locations.
- Vertical mullions of blast-resistant curtain walls can be connected to reinforce concrete floor slabs (or spandrel beams) by means of steel brackets. The brackets may be oriented to be bolted on the front edge of the concrete member where the bolts primarily provide tensile resistance, or they can be bolted on top and/or bottom of the concrete element, in which case the bolts primarily provide shear resistance. The orientation of brackets depends on the location of vertical mullion (intermediate or edge) and the accessibility of concrete elements.
- The brackets must have sufficient rigidity so that they do not bend in the process of transferring support reactions to the concrete element. When the support reactions are high and the connection is made to the top and bottom of the slab edge, as was observed in Mullion M-C3 of the experimental program, the brackets may bend and place the burden on direct bearing of the aluminum on concrete. This may result in bearing failure of the mullion. In such cases, it is advisable to support the mullions from face of the slab with brackets bolted on to the sides (web) of the mullion.
- The use of elastic dampers at mullion-substrate connections can reduce blast load effects and control damage to curtain walls. Selection of the stiffness and geometry of the dampers are paramount for proper functioning of the connections. The damper stiffness and the travel distance during response should be considered in design. The

XRD rubber pads selected for connection protection offered limited protection because of insufficient stiffness and thickness.

- The use of elastic springs as dampers at the connections of curtain walls with the substrate offer blast protection. They should be designed for the design basis threat to possess sufficiently high stiffness and travel distance.

Table 3. 1: Properties of curtain wall and mullion sections

Model	First phase: full-size curtain wall						Maximum Reflected Pressure (kPa)
	Mullions (mm)			Transoms (mm)			
	Section	Thickness	Length	Section	Thickness	Length	
CW1	127 × 51	3	2032	76 × 51	3	1930	50
CW2	127 × 51	3	2032	76 × 51	3	1930	42
CW3	127 × 51	3	2032	76 × 51	3	1930	42
							50
							62
Second phase: full-size single mullion							
Model	Mullions (mm)			L-shape bracket (mm)			
	Section	Thickness	Length	Section	Thickness		
M-C1	127 × 51	3	2032	100 × 100	3	22	
M-C2	127 × 51	3	2032	100 × 100	3	28	
M-C3	127 × 51	3	2032	100 × 100	3	72	
M-C4	127 × 51	3	2032	100 × 100	3	71	
Third phase: full-size mullion with Poron XRD rubber							
Model	Mullion (mm)			Poron XRD rubber			
	Section	Thickness	Length	Section (mm)	Density (kg/m ³)		
M-C5	127 × 51	3	2032	216 × 279 × 13	240	71	
M-C6	127 × 51	3	2032	216 × 279 × 13	240	73	
Third phase: full-size mullion with spiral spring							
Model	Mullion (mm)			Spring			
	Section	Thickness	Length	Net length (mm)	Stiffness (N/mm)		
M-C7	127 × 51	3	2032	160	8	24	
M-C8	127 × 51	3	2032	160	30	31	
M-C8	127 × 51	3	2032	200	23	22	



Figure 3.1: Shock-tube assembly; driver section and frame of test area

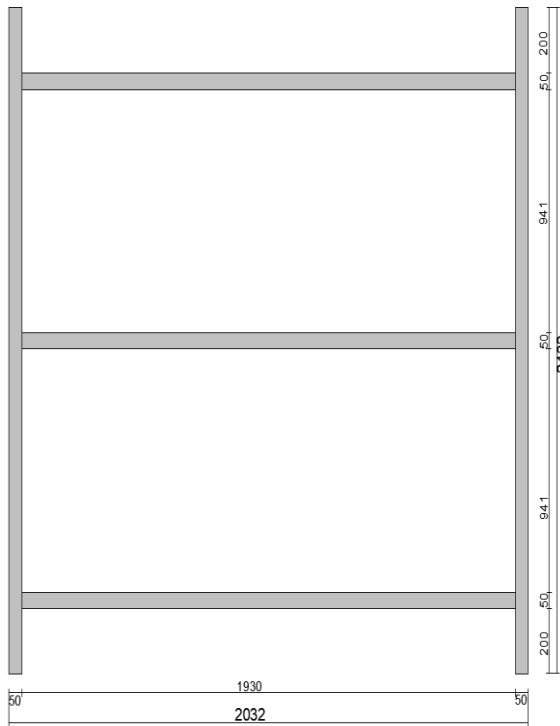


a) LVDT(s)

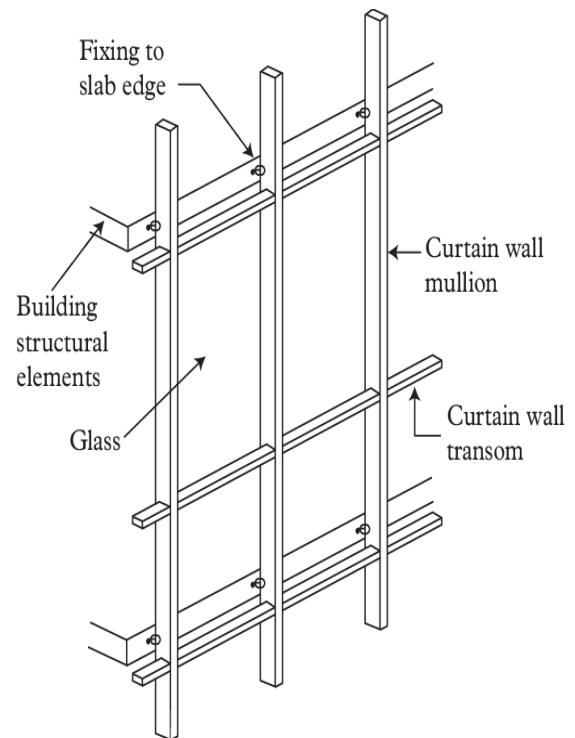


b) High-speed cameras

Figure 3.2: Instrumentation



a) The tested curtain wall sketch at uOttawa



b) An ordinary curtain wall in real life

Figure 3.3: The glossematics schematic characteristics of the curtain wall system



Figure 3.4: Typical curtain wall specimen, interior (left) and exterior (right)



Figure 3.5: U-angle connections between mullions and transoms



Figure 3.6: First curtain wall; without hardening of mullions, but with connections strengthened by epoxy glued aluminum brackets



Figure 3.7: Second specimen with retrofitted mullions and anchored aluminum brackets



Figure 3.8: Third specimen with retrofitted mullions and transoms, as well as strengthened connections by anchored steel brackets



a) Knife-edge support welded on HSS sections



b) Prepared support

Figure 3.9: Simple edge supports for curtain wall mullions, prepared by using steel angles

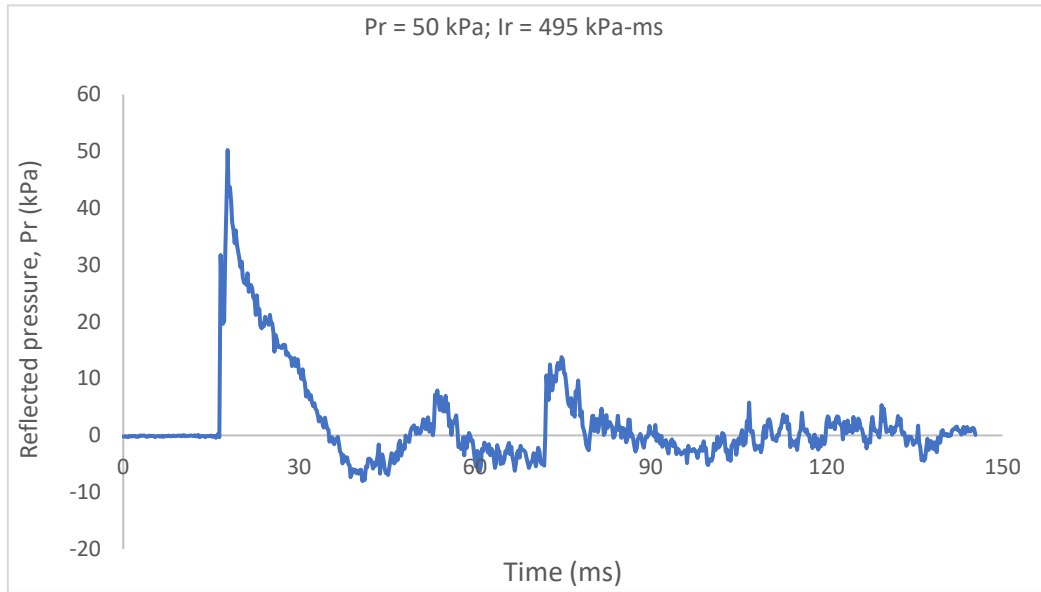


Figure 3.10: First curtain wall pressure-time history



Figure 3.11: First curtain wall after blast test

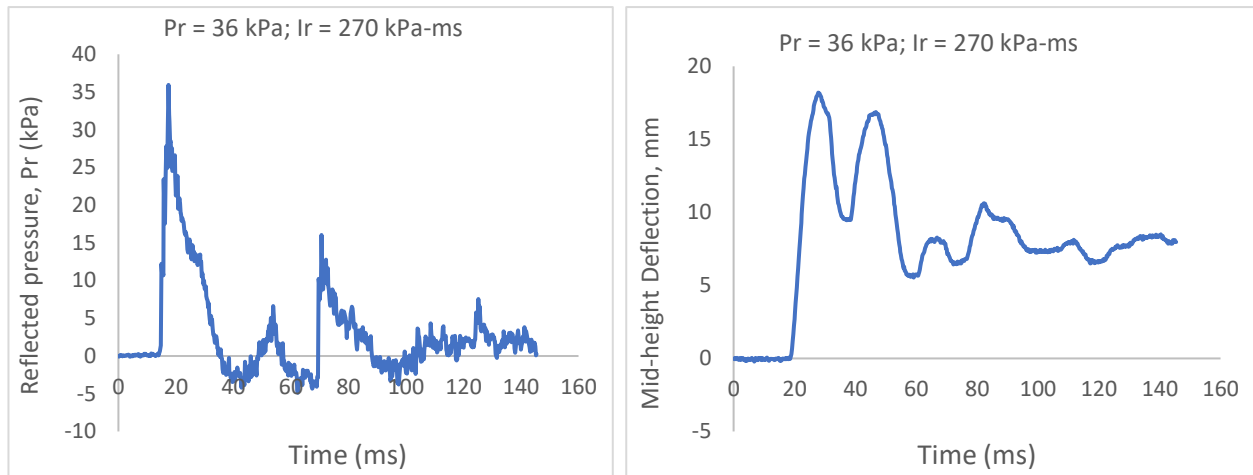


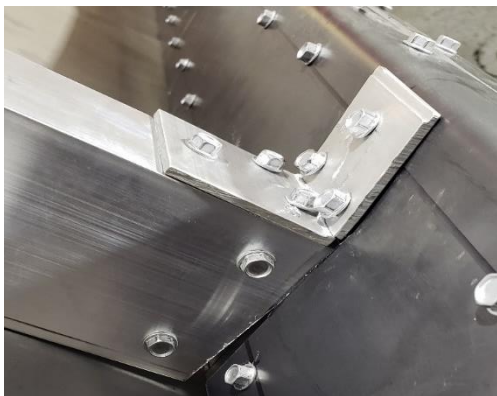
Figure 3.12: Second curtain wall pressure-time history (left), and mid-height mullion deflection-time history (right)



a) Overall curtain wall view



b) Membrane action and its effect on mullions



c) Anchored aluminum brackets

Figure 3.13: Second curtain wall after blast test

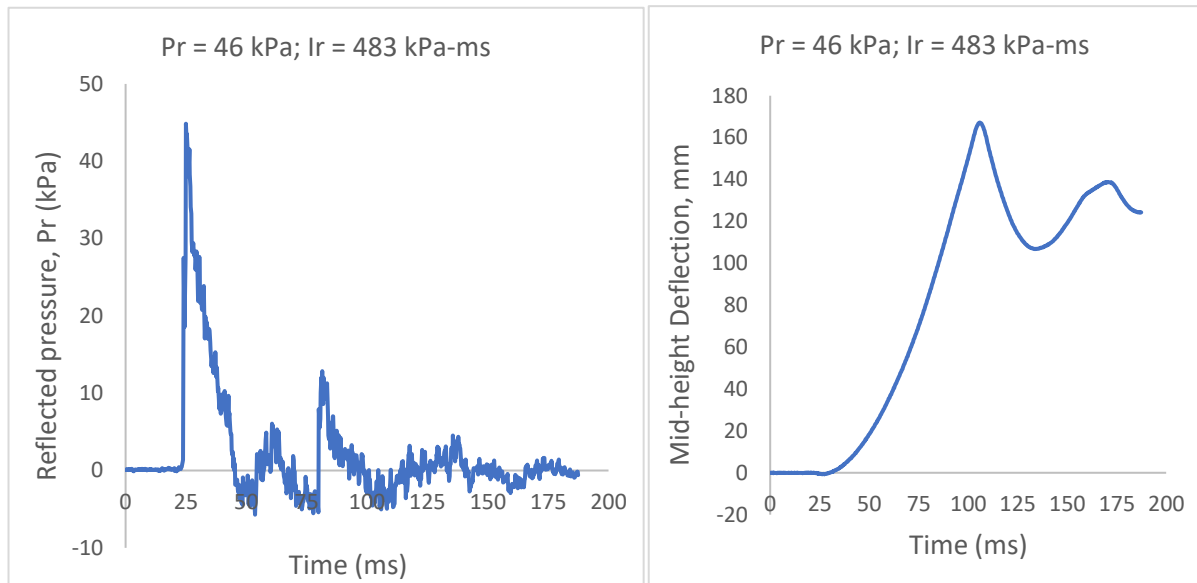


Figure 3.14: Third curtain wall; 1st shot pressure-time history (left), mid-height mullion deflection-time history (right)



Figure 3.15: Third curtain wall 1st shot; after blast test

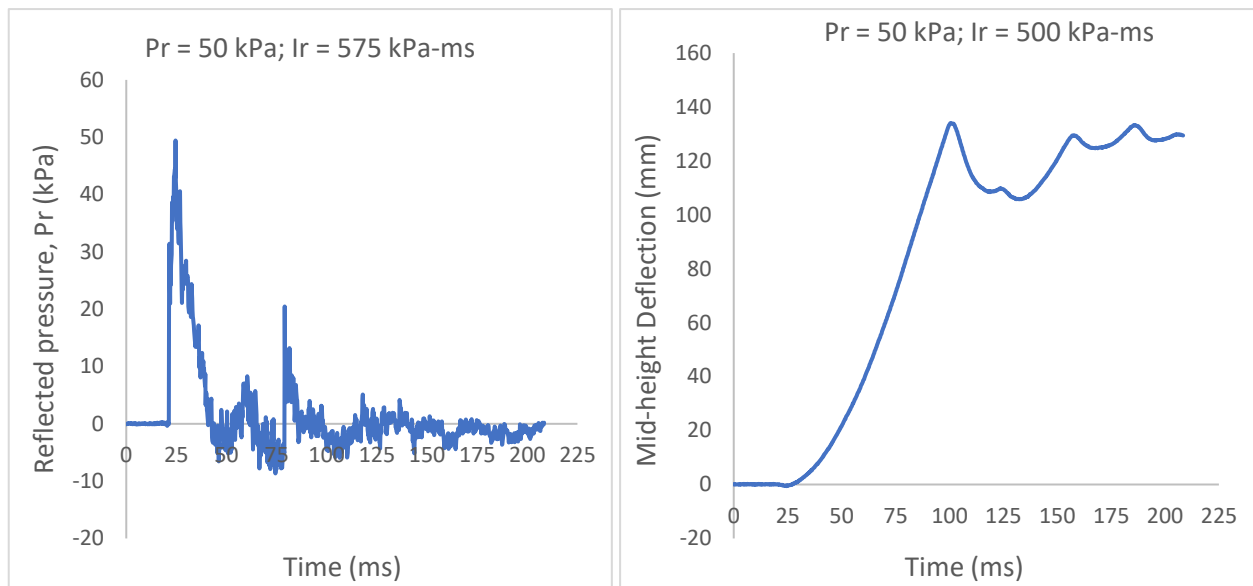


Figure 3.16: Third curtain wall; 2nd shot pressure-time history (left), mid-height mullion deflection-time history (right)



Figure 3.17: Third curtain wall, 2nd shot; after blast test.

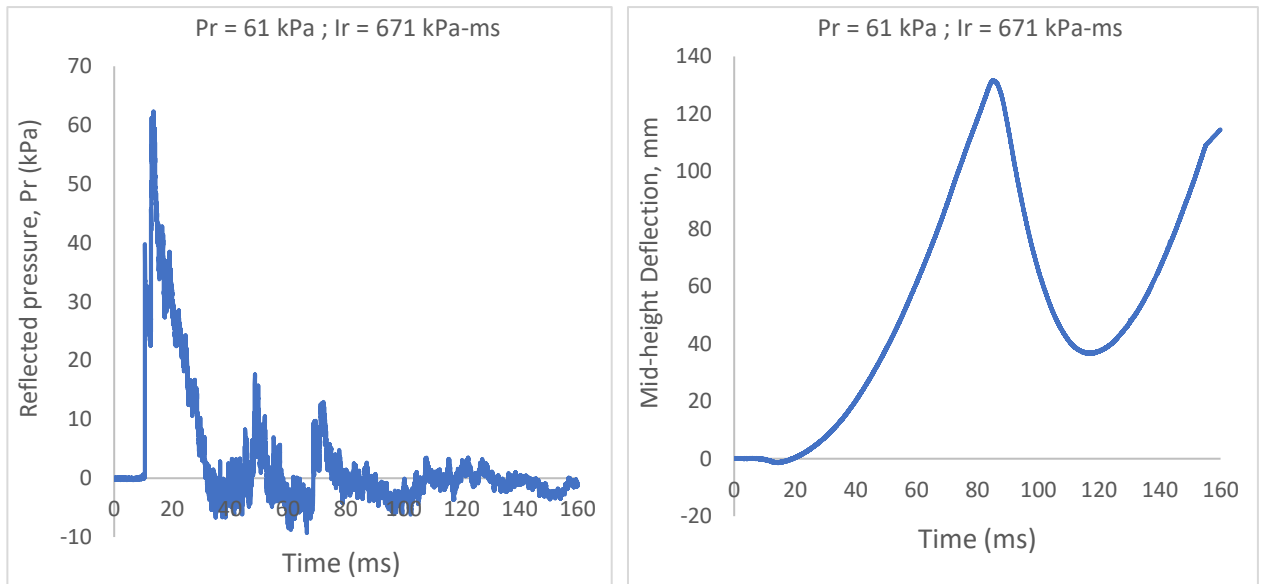


Figure 3.18: Third curtain wall; 3rd shot pressure-time history (left), mid-height mullion deflection-time history (right)



Figure 3.19: Third curtain wall 3rd shot; after blast test

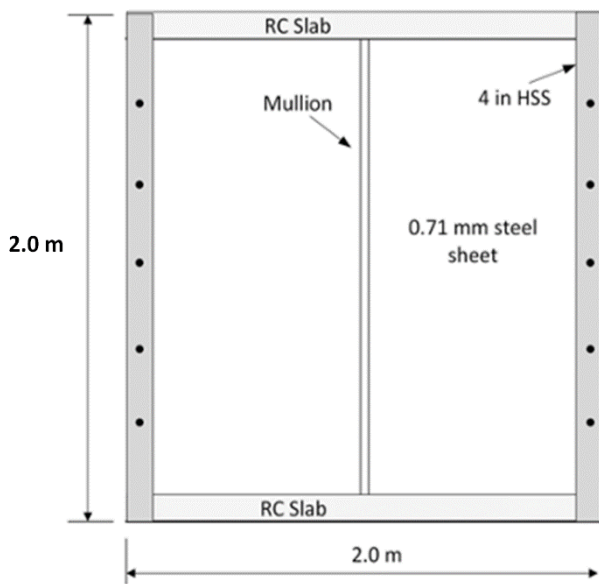


(a) Reinforcement

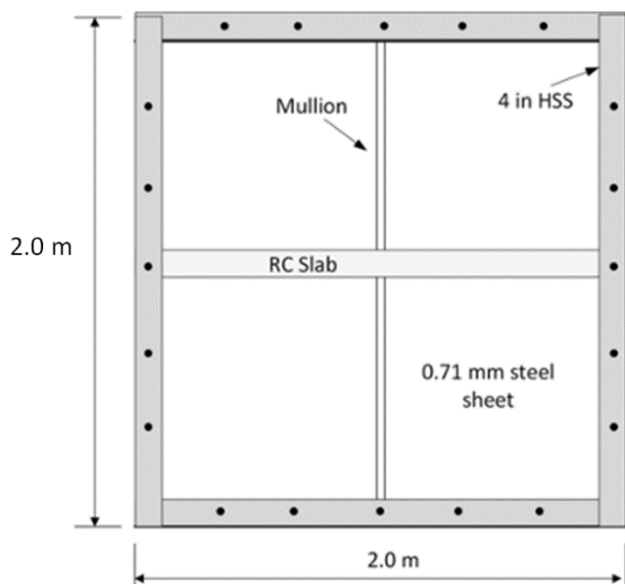


b) Hole locations

Figure 3.20: Reinforced concrete beam details



(a) End connection



(b) Continuous mullion over slab

Figure 3.21: Mullions supported by the slabs at the ends, and mullions extending over slabs



Top connection



Bottom connection

Figure 3.22: M-C1 with brackets at the ends, connected through the mullion sides (web)



Top connection



Bottom connection

Figure 3.23: M-C2 with brackets at the ends, connected through the mullion end (flange)



a) Overall view of test configuration



b) Top of slab



c) Bottom of slab

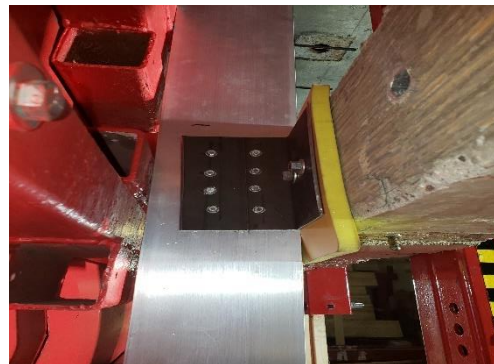
Figure 3.24: M-C3 continuous over the concrete slab connected on the flange of the mullion



Figure 3.25: M-C4 continuous over the concrete slab, connected on the web of the mullion



a) Overall test configuration



b) Side brackets with damper

Figure 3.26: M-C5 with inserted XRD rubber between mullion and concrete beam



a) Overall view of test configuration (top and bottom bracket)



b) Inserted XRD rubber between mullion, concrete beam, and brackets

Figure 3.27: M-C6 with inserted rubber and top and bottom brackets



a) Single mullion + spring test setup



b) Typical spring instrumentation



c) Stiffness: 8 N/mm; net length: 160mm

Figure 3.28: M-C7 with inserted springs between the structural members and mullion



a) Typical single mullion + spring configuration with supporting structural members



b) M-C8 Stiffness: 30 N/mm
Net length: 160 mm

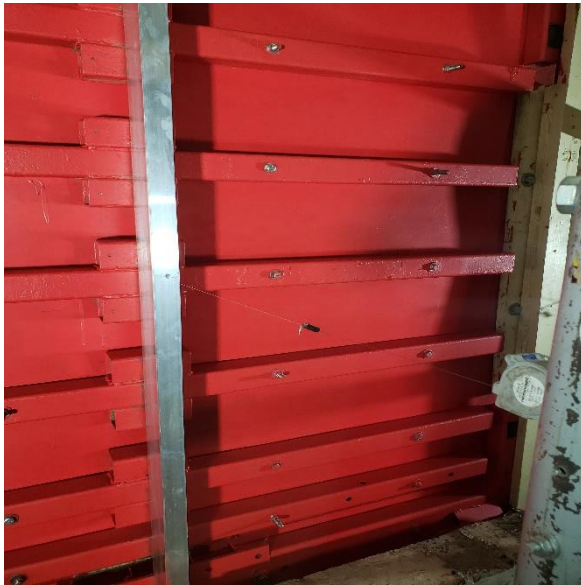


c) M-C9 Stiffness: 23 N/mm
Net length: 200 mm

Figure 3.29: M-C8 and M-C9 mullions with different spring characteristics



a) Full size curtain wall with two LVDT



b) Single mullion setup with one LVDT



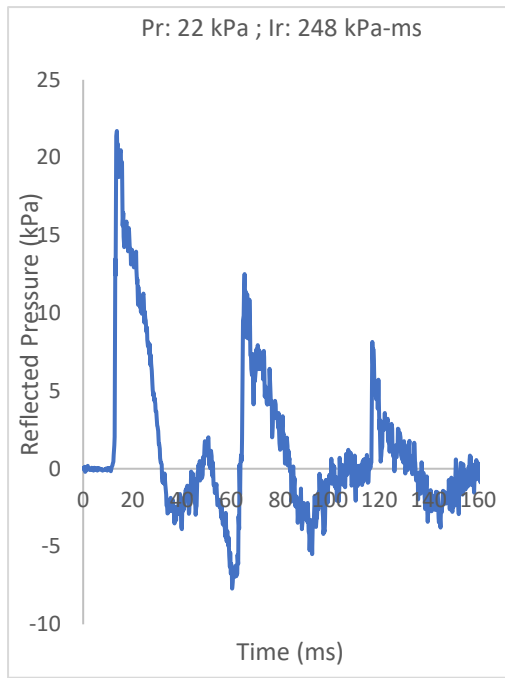
c) Continuous mullion setup with two LVDT



d) Mullion with inserted springs using



Figure 3.30: Typical LVDT locations on the sections during all three phases



a) Reflected pressure



b) Max deflection



c) Residual deflection

Figure 3.31: M-C1 pressure-time history (left), and deflection-time history (right)



a) Overall view of single mullion



b) Observation at failure



c) In tack side steel bracket

Figure 3.32: Specimen MC-1 after the test (single mullion configuration)

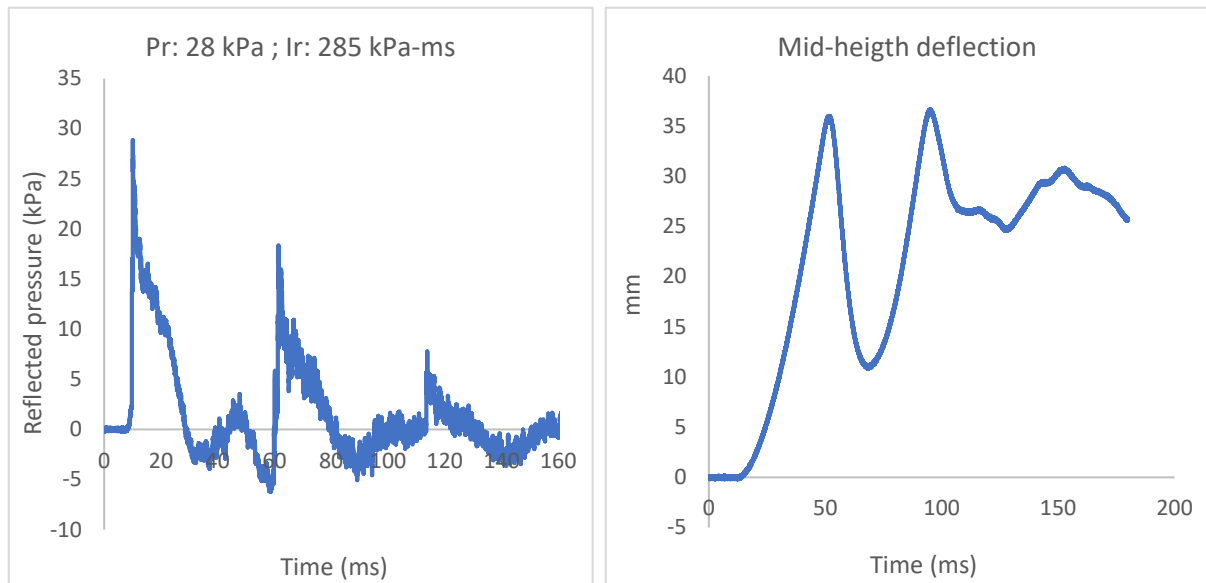


Figure 3.33: M-C2 pressure-time history (left), and deflection-time history (right)



a) Overall view of mullion

b) observations at failure

Figure 3.34: Mullion M-C2 after blast shot (single mullion configuration)

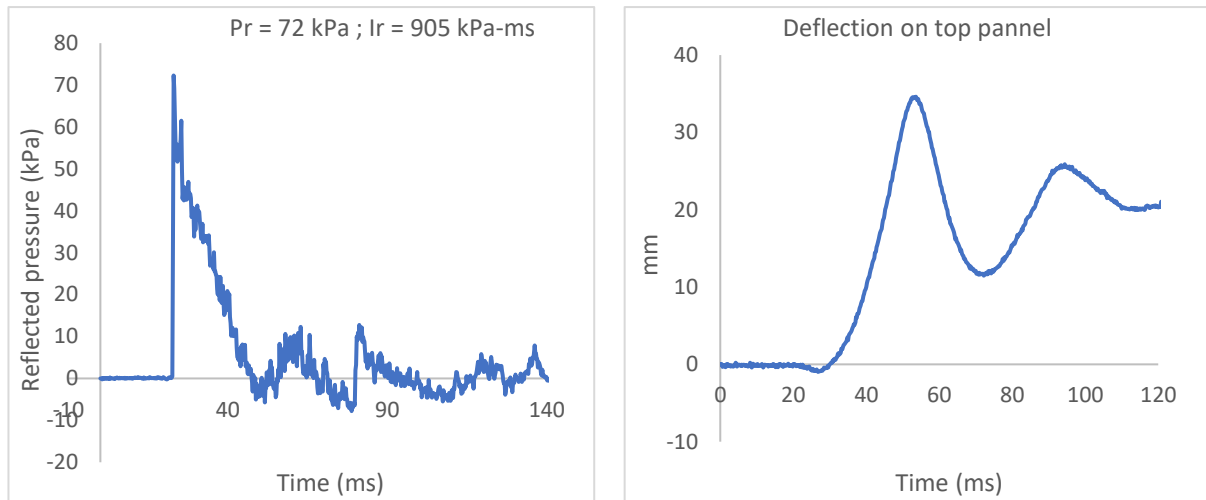
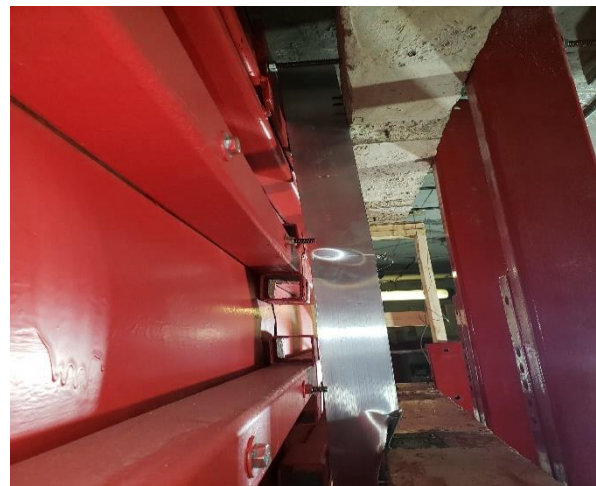


Figure 3.35: M-C3 pressure-time history (left), and deflection-time history (right)



a) Failure observation at the middle concrete



b) Compression buckling at upper extension



c) Bottom bracket



d) Top bracket

Figure 3.36: Mullion M-C3 after blast shot (continuous mullion configuration)

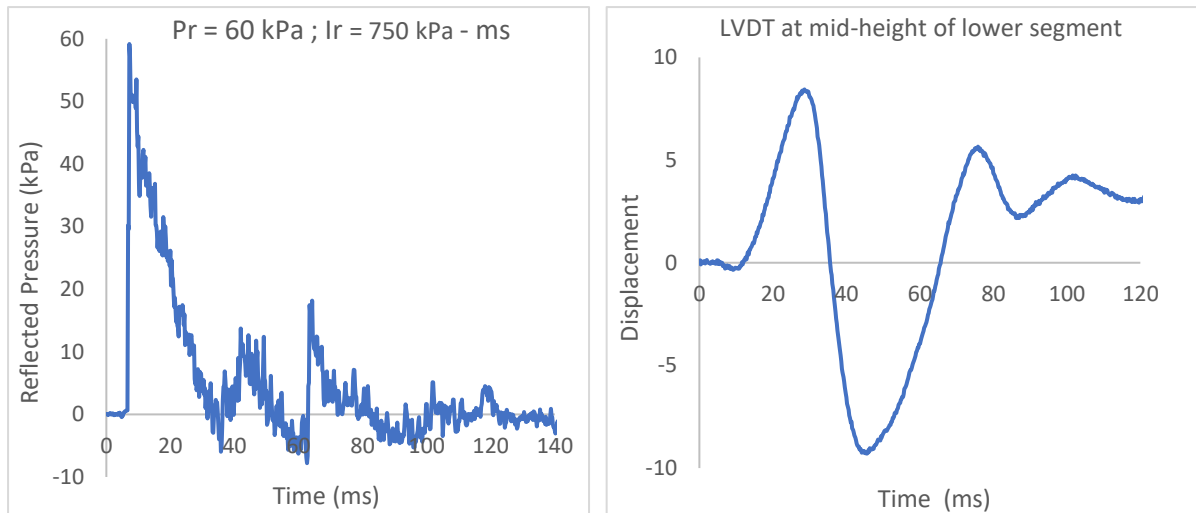


Figure 3.37: M-C4 pressure-time, and deflection-time history (1st shot)



a) Mullion without any damage



b) In tack side brackets

Figure 3.38: Mullion M-C4 after 1st blast shot (continuous mullion configuration)

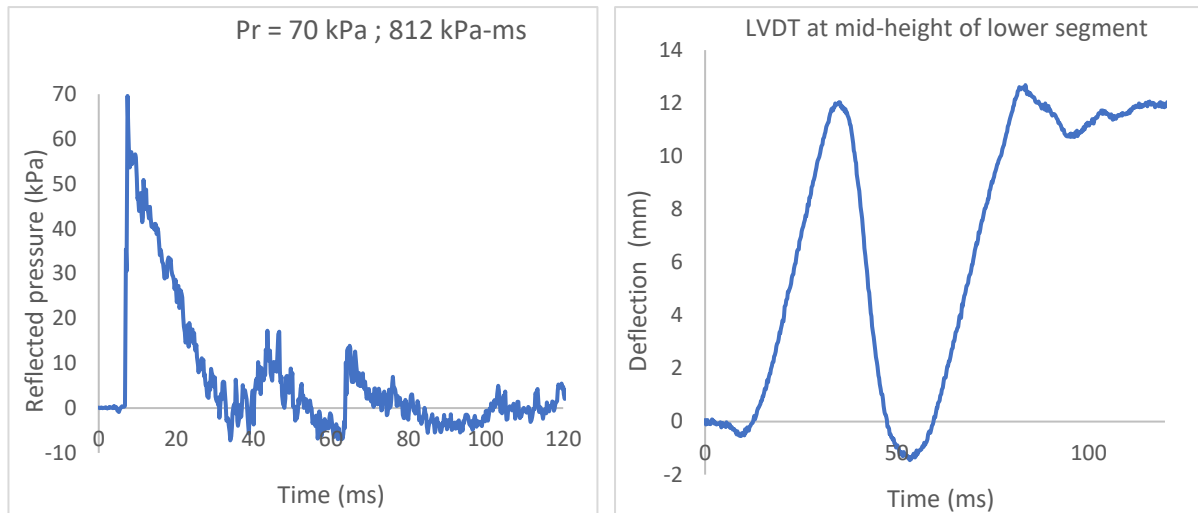


Figure 3.39: M-C4 pressure-time, and deflection-time history (2nd shot)



a) Overall view

b) Compression buckling

c) Undamaged side brackets

Figure 3.40: Mullion M-C4 after 2nd blast shot (continuous mullion configuration)

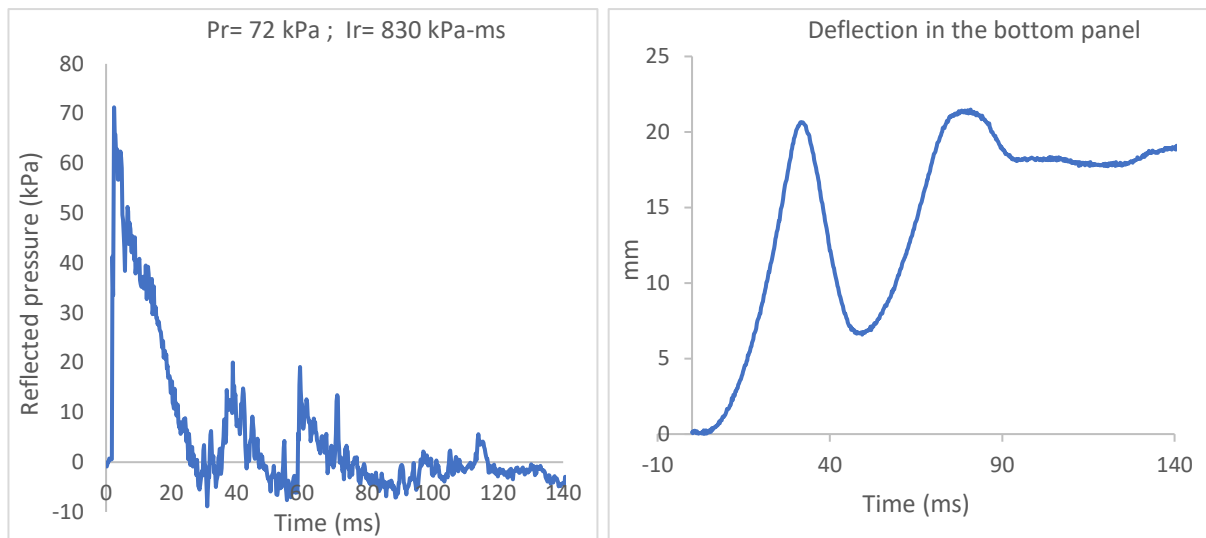
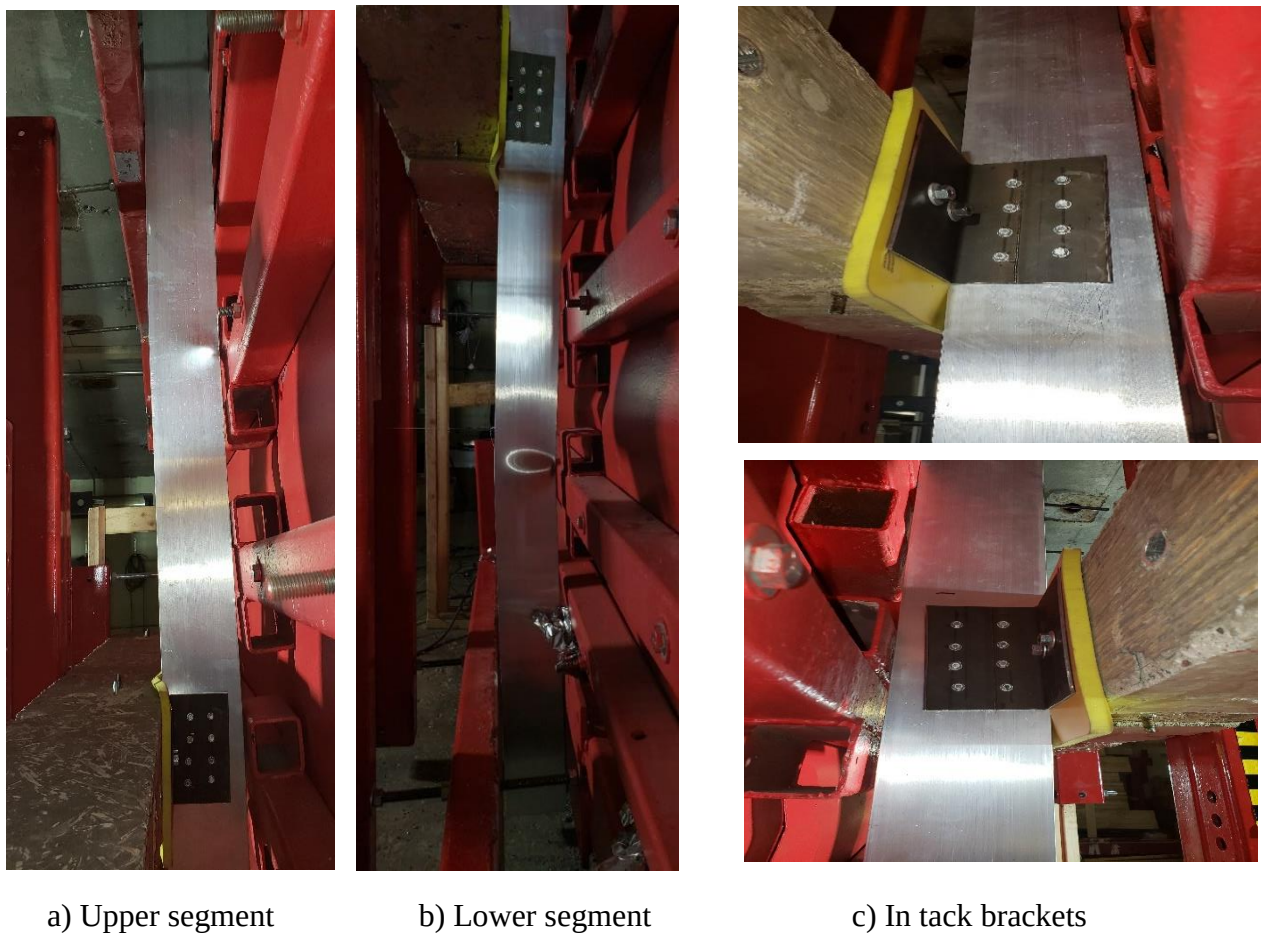


Figure 3.41: M-C5 pressure-time history (left), and deflection-time history (right)



a) Upper segment

b) Lower segment

c) In tack brackets

Figure 3.42: Mullion M-C5 after blast shot (continuous mullion configuration + rubber)

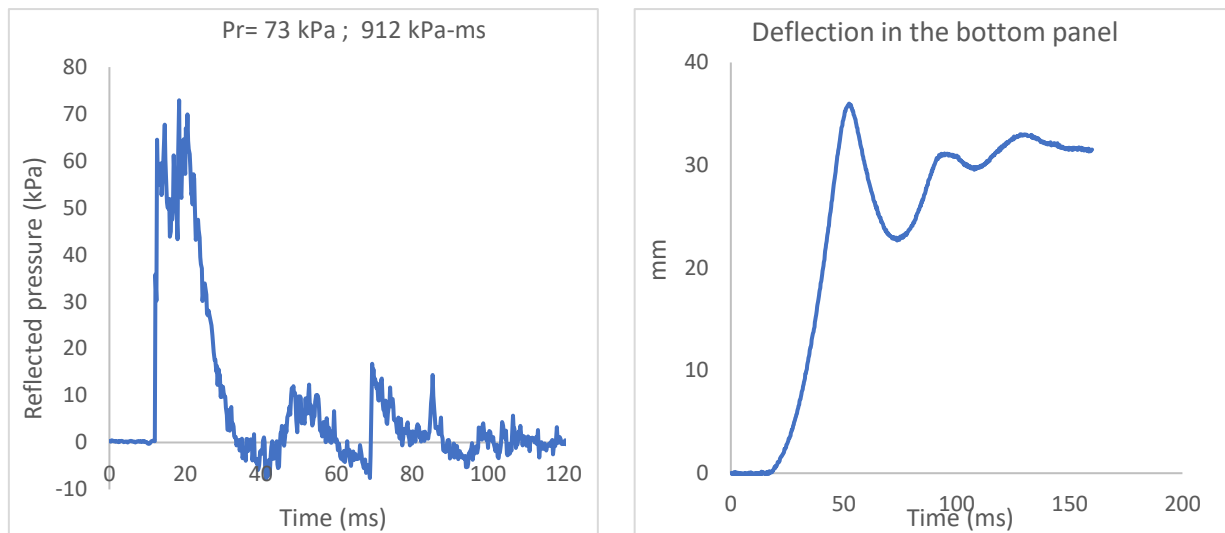
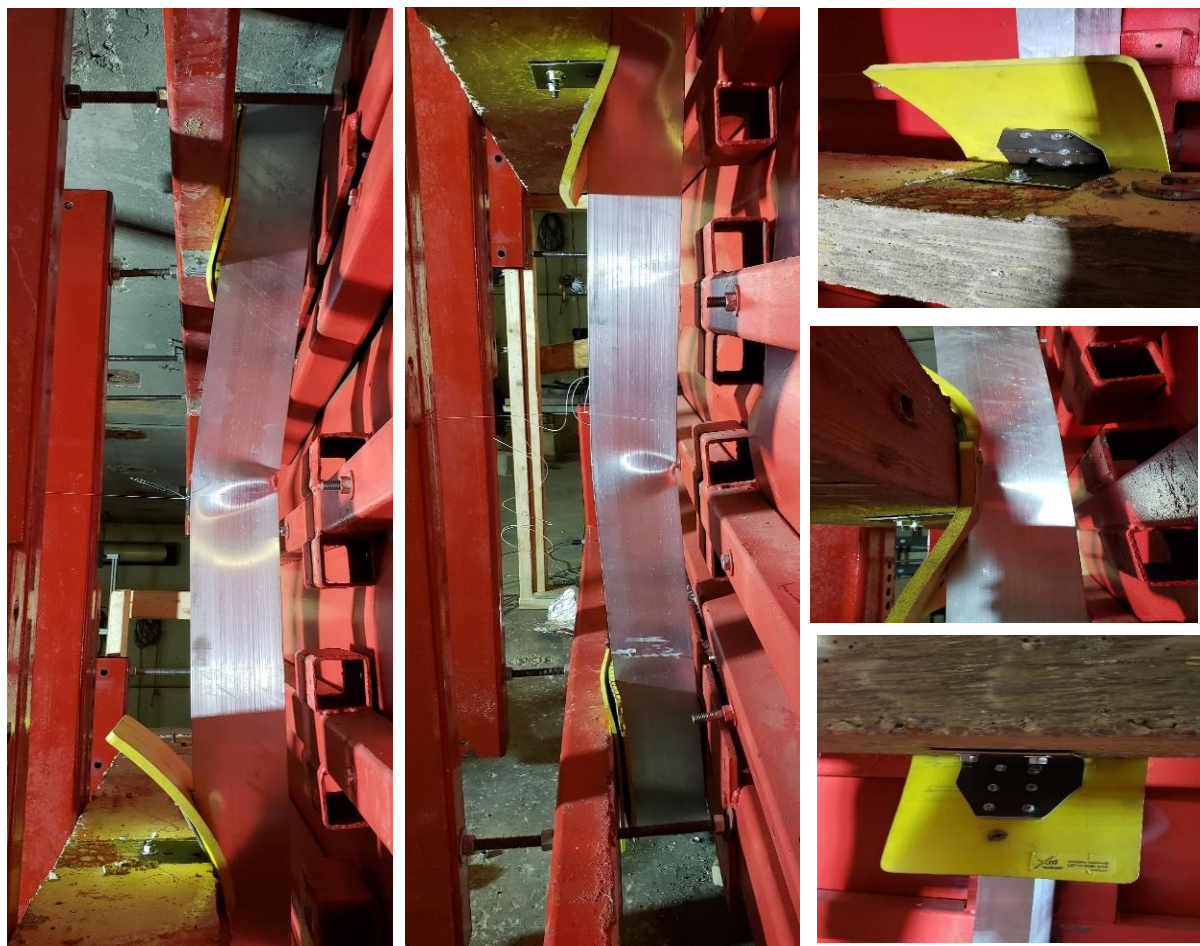


Figure 3.43: M-C6 pressure-time history (left), and deflection-time history (right)



a) Upper segment

b) Lower segment

c) Bent brackets with rubber

Figure 3.44: Mullion M-C6 after blast shot (continuous mullion configuration + rubber)

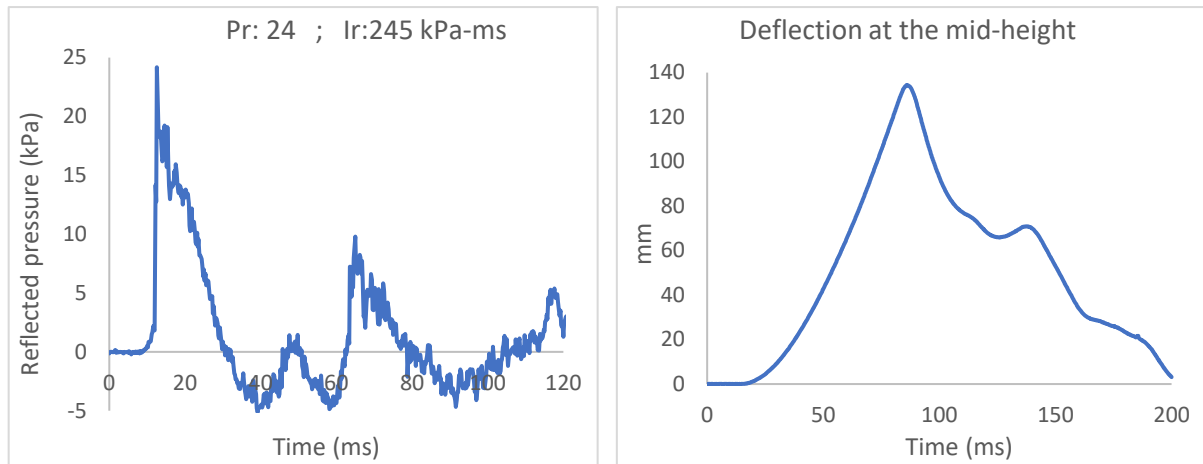


Figure 3.45: M-C7 pressure-time history (left), and deflection-time history (right)



a) In tack mullion



b) top and bottom springs

Figure 3.46: Mullion M-C7 after blast shot (single mullion configuration + spring 8 N/mm)

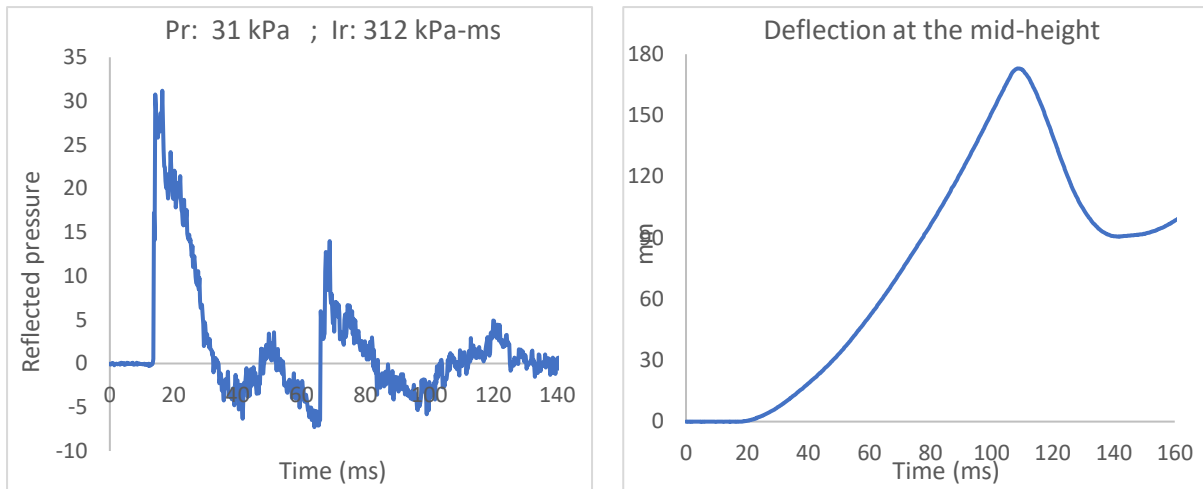


Figure 3.47: M-C8 pressure-time history (left), and deflection-time history (right)



Figure 3.48: Mullion M-C8 after blast shot (single mullion configuration + spring 8 N/mm)

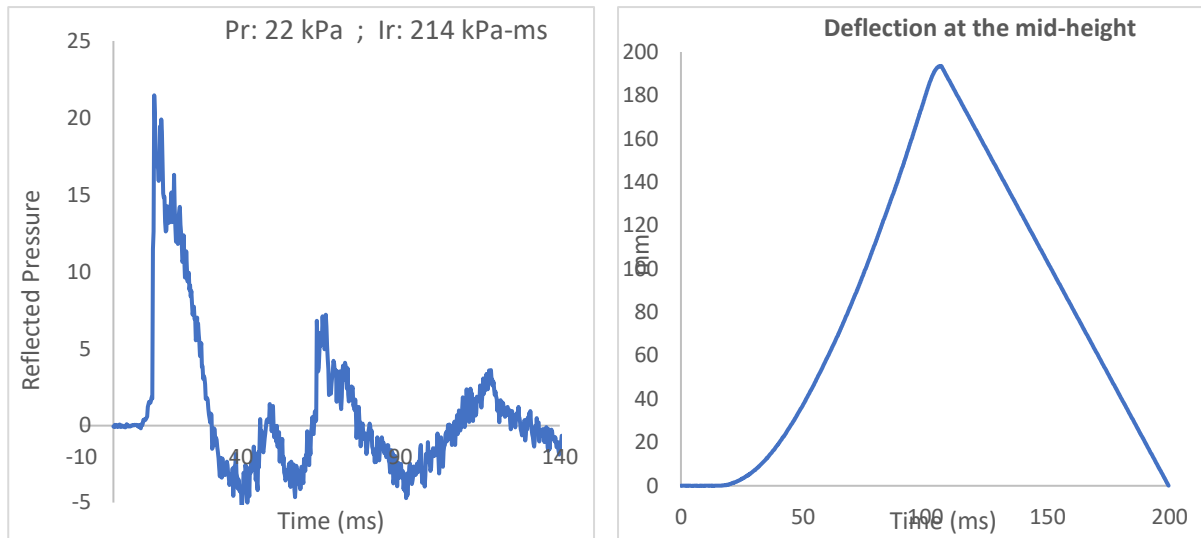


Figure 3.49: M-C9 pressure-time history (left), and deflection-time history (right)



a) In tack mullion



b) Top springs



c) Bottom springs

Figure 3.50: Mullion M-C9 after blast shot (single mullion configuration + spring 23 N/mm)

Part III

Analytical Research

CHAPTER 4. Numerical Investigation of Improved and Retrofitted Connections for Blast-Resistant Curtain Walls

Abstract:

The performance of connections for blast-resistant curtain walls was investigated numerically. Both the mullion to transom connections and mullion to concrete substrate connections were examined. The numerical research consisted of two phases; the validation of the analytical models against the curtain wall components tested in the experimental part of the same overall research project, and a parametric investigation to assess the significance of connection design parameters. LS-DYNA advanced commercial Multiphysics simulation software was used to conduct the numerical investigation. The experimental data used for the validation of analytical models included shock tube tests of three large-scale curtain wall specimens and nine single mullions with different connection properties. Four of the nine mullion tests had rigid connections, whereas the remaining five had damping materials attached to the connections for improved performance. The large-scale curtain wall tests focused on the mullion-to-transom connections with different connection strengthening techniques implemented on un-hardened and hardened mullions and transoms. The single mullion tests focused on the mullion to concrete substrate connections, simulating curtain wall to concrete slab/beam connections at floor levels. These twelve test specimens were modeled for validating the numerical approach employed. The correlation between the test data and the numerical simulations was found to be excellent. Additional nine analytical models were generated to conduct a parametric investigation. The parameters included connection details, material thickness and the characteristics/stiffness of the damper materials. The significance of each parameter was assessed by the observed damage, maximum and residual displacements, support rotations, stress levels and connection forces. The numerical results provided extensive design data, highlighting the importance of parameters that contributed to connection strength. Stresses, strains and force distributions obtained numerically provided recommendations for use in design.

Keywords: Blast-resistant curtain walls, blast load analysis, connection design, curtain walls, curtain wall hardening, dynamic inelastic analysis, finite element method, mullions, and transoms.

4.1 Introduction

Glass curtain walls are commonly used in modern construction for buildings, such as offices, hospitals, school, and commercial buildings. They are deemed to be architectural elements, whose lateral design is often dictated by wind loads. Unless specifically designed for blast load effects, they have little resistance against blast induced shock waves. Failure of curtain wall facades and window systems in buildings results in a serious threat to life safety. As examples, significant number of injuries and fatalities were reported after the 1995 bombing of the Murrah Federal Building in Oklahoma, the 2011 Oslo Regjeringskvartalet Bombing, and the 2015 Mexico City Hospital gas explosion (Norville et al. 1999). Most of the existing curtain wall buildings remain vulnerable to blast threats and hazards. Blast retrofits of curtain walls offer feasible solutions to blast risk mitigation. Retrofitting such façade elements pose challenges in terms of strengthening vertical mullions and horizontal transoms, which are typically made out of aluminum, their connection, as well as the glazing of glass elements with protective film.

Several design guidelines and standards have been developed for blast-resistant curtain walls. The International Organization for Standardization (ISO) provides two standards for blast testing (ISO 16933 and ISO 16934). ISO 16933 provides a procedure to establish the air-blast resistance of glazing and sets forth the required apparatus, procedures, specimens, and other requirements for conducting arena air-blast tests. ISO/FDIS 22497 provides requirements for windows and curtain walls against blast impulse. Canadian Standards Association CSA S850-12 presents analysis and design requirements for blast-resistant buildings and their components, including the requirements for curtain walls and windows. The US General Services Administration (GSA) provides guidelines and software for Window Glazing Analysis Response and Design (WINGARD) which was developed by Applied Research

Associates (ARA) based on ASTM 1642. The Unified Facilities Criteria (UFC) standard developed by the Department of Defense (DoD) of the US Federal Government is perhaps the most comprehensive design document for blast protection of critical facilities, including guidelines on window and curtain wall glazing (UFC 4-020-01). Despite the availability of aforementioned documents, there is lack of complete design guidance supported by research, especially for the connections of mullions to transoms and the anchorage to the supporting substrate.

Kirk, Carrie, and Peter (2016) investigated coupled response of glazed glass and the framing around them in conventional curtain walls subjected to blast loads. Their numerical results indicated that curtain walls developed coupled system response with maximum glazing response occurring at roughly the time as maximum mullion displacement. It was reported that simplified modeling strategy for aluminum mullions is a viable and computationally efficient approach. Comparison of no-break simulation results with mullion laser gauge data demonstrated the ability of the current mullion modeling strategy to predict peak mullion response. In addition, no failure was reported in the connections. Timothy, et al. (2015) developed a curtain wall technology for the US government through High Fidelity Physics Based (HFPB) analysis which comprised of Computational Structural Dynamics (CSD), and Computational Fluid Dynamics (CFD) methods. According to the result, the analytical models properly captured the actual blast test observations. The researchers attested that the numerical results could generate proper predictions of the actual test results and thus can provide reliable guidelines for the connection design. Hongbin et al. (2015) provided an overview of the experimental and numerical investigations of glass curtain walls subjected to low-level blast loads. The numerical simulation was deemed to be satisfactory for assessing the efficacy of an ordinary nonlinear design procedure to establish the level of blast resistance of connections in curtain walls. The researchers observed that tear-out was the controlling limit state when the curtain wall was impulsively loaded. Tear-out lengths from the finite element method (FEM) of simulations were established. FEM simulations were able to predict observed failures.

Chiara et al. (2018) elaborated on the numerical assessment of vibration control systems for multi-hazard design and mitigation of glass curtain walls. The feasibility and potentials for special mechanical joints interposed at the interface between a given multi-storey primary building and a traditional Unitized Glazing Curtain Wall (UGCW) was investigated via Finite-Element numerical models under different impact scenarios. Results indicated that fasteners could have more flexibility when vibration control systems are utilized. The researchers showed that UGCW can act as distributed Tuned Mass Damper (TMD) in multi-storey buildings under blast hazard. They further claimed that the numerical models they used had appropriate precision when compared with actual test data. Rong-Bing et al. (2010) numerically investigated the authentic simulation method for assessing blast-induced flow propagation around an intricate curtain wall. The authors shed light on the complex flow of air blasts around structures and predicted blast loads acting on the connections. The researchers reported that the simulated models of pressure history had good agreement with experimental data, and numerical results accurately showed pressure propagation in curtain wall panels. It was further reported that the multi-material finite element method mixed with parallel computing is a suitable and adequate method for simulating fluid–structure interaction. Chiara et al. (2012) numerically proposed the use of viscoelastic dissipative devices (dampers) to enhance curtain wall joint performance against blast waves. The simulation of the curtain wall, spanning floor to floor, included a laminated glass panel, a continuous bead of structural silicone sealant, a split screw spline frame and four rigid brackets. Numerical results indicated that the FEM model could only capture general curtain wall behavior because of the cracking of glass lites and the plasticization of aluminum mullions, which changed the dynamic movement. It was concluded that the use of viscoelastic devices can lower maximum stresses in the glass lites, silicone joint, and in mullions, thereby cutting down the deflection and the support reactions at the structural substrate. Martin et al. (2011) carried out a study on the simulation of laminated glass curtain wall loaded by air blast waves. Shell elements with special failure criteria were used to model aluminum sections, as they were the most efficient layered elements for the application. Moreover, an elastic-plastic material law was used for the

Polyvinyl Butyral (PVB) protective layer in the glass. Results also showed that a smeared model can only be utilized for cases where the interlayer is not broken after the loading.

The aforementioned literature review indicates limited numerical research on blast-resistant curtain walls, particularly the research on connection performance and the required strengthening techniques for mullion to transom and mullion to supporting substrate connections is scarce in the literature. Furthermore, limited numerical research was conducted on the use of dampers at the connections. The objective of the current numerical research project is to fill the existing research gaps on blast resistant curtain walls with a view of generating connection design information. The scope includes numerical investigation of curtain walls to assess their performance under different variables, including those affecting the connection design. The connections considered were mullion-to-transom connections in blast hardened and un-hardened curtain walls, as well as mullion to concrete slab/beam connections with and without damping materials. The first phase of research involved building numerical models and validating them against the 12 shock tube tests conducted in the experimental phase of current research (Nasseralshariati and Saatcioglu, 2023). The first three tests and the representative numerical models involved large-scale specimens of curtain walls consisting of two vertical mullions, three horizontal transoms and a steel sheet representing the glazed glass attached to them. The steel sheet was designed to represent the mass of glazed glass panels, and its use as representative glazing was validated numerically by comparing the blast performance of companion curtain walls with glazed glass and steel sheets, as discussed later in the paper. The test units had different connection strengthening techniques between the mullions and transoms, either with or without hardened mullions and transoms. The subsequent validation was done for the 9 single mullions tested. The numerical simulations showed excellent agreement with the test results both in terms of the measured data and the failure modes. The second phase of the numerical research consisted of parametric investigations, where the parameters were selected to help generate design information. More specifically, the effects of strengthening the connections between mullions and supporting structural members (concrete beam/slab) were numerically investigated. Besides the parametric investigation

additional numerical analysis were conducted to assess: i) authenticity of results obtained from using steel sheet instead of glass glazed with protective film, ii) effectiveness of retrofitting existing connections designed for wind loads; iii) the effect of bracket thickness at the mullion-concrete substrate connection; v) effectiveness of using damper materials at mullion supports and connections.

The numerical investigation was conducted using the advanced multiphysics commercial simulation software LS-DYNA. This software has two aspects that makes it industry standard for blast load analysis; i) computational fluid dynamics to simulate the blast phenomenon and the resulting pressures and duration on structures, and ii) finite element method (FEM) of analysis to simulate the behavior of structural and non-structural components. The current research involves FEM analysis of curtain walls, with blast shock wave generated by external surface explosion and resulting semi-spherical shock wave applied as uniformly distributed pressure on FEM models, as recorded during the tests. All the test specimens were modeled based on the actual material properties established experimentally. The validation was done in terms of out-of-plane deflections of vertical mullions and the middle horizontal transom for which experimental data was available. The LS-DYNA output also provided in-plane displacements that resulted from the membrane action of simulated glass panes in the form of equivalent steel sheets, as well as support reactions, stress distributions, and Von Mises stresses. Additional data, like support reactions, in-plane and out-of-plane displacements, stress distribution and Von Mises stresses were obtained to compliment the experimental data.

4.2 Numerical Models

4.2.1. Models for Curtain Wall and Mullion Test Specimens

Material and geometric properties implemented in FEM models were representative of the specimens tested in the experimental phase of current research, reported elsewhere (Nasseralshariati and Saatcioglu 2023). The large-scale curtain wall specimens had 2440 mm by 2030 mm (96" × 80") dimensions, simply supported at the top and bottom ends. Each curtain wall model included two vertical mullions with 2.44 m (96") height, and three

horizontal transoms with 1.93 m (76") length. Each curtain wall had two 22-gauge steel sheets, representing two glazed panels (glass + protective film). The overall geometry of the numerical models and boundary locations are illustrated in Figs. 4.1 and 4.2. Different techniques were used to improve the mullion-to-transom connections for hardened and un-hardened curtain walls. The models for each of these additional strengthening elements are illustrated in Fig. 4.3. They consist of: i) internally provided steel U-bracket to assemble mullions and transoms; ii) L-shaped steel plates to harden transoms and/or mullions; and iii) externally placed side brackets at the connections of mullions and transoms.

The first Curtain Wall (CW1) did not have any hardened mullions or transoms. Strengthening was implemented at their connections. This was done by first providing internal steel U-brackets to connect the mullions to transoms and by attaching aluminum L-brackets externally at the connections using an epoxy adhesive. Figure 4.4 shows the numerical model for CW1. The second curtain wall (CW2) model also had the inner U-brackets and external aluminum L-brackets, but this time the mullions were hardened by means of L-shaped steel plates and the external aluminum brackets were mechanically connected by means of rivets. Figure 4.5 indicates the numerical model for CW2. The third curtain wall (CW3) had hardened mullions and transoms with internal steel U-brackets and external steel L-brackets mechanically connected at the joints. Fig. 4.6 illustrates the numerical model for CW3. The details and section sizes for each component used in the three curtain wall specimens are listed in Table 4.2.

The validation of the numerical simulation procedure extended to single mullion tests with emphasis on the mullion-to-concrete substrate connection. The mullions were connected to reinforced concrete beams/slabs with angle brackets. The brackets were connected either to the front edge of the slab with the other side of the angle attached to the mullion web, or on the top and bottom of the slab attached to the flange of the mullion. Each type of connection was used at the ends of a floor-to-floor high vertical mullions as well as mullions that are continuous over the slab extending towards the floors above and below. Figure 4.7 illustrates both configurations. When the continuous mullions were tested, the ends were clamped to the shock

tube with fully fixed end condition. The aluminum mullions all had a hollow rectangular cross-section with outside dimensions of 127 mm x 51 mm and wall thickness of 3 mm. Each mullion had a length of 2.4 m to be placed in front of the load application device of the shock tube, which consisted of a 3 mm thick steel sheet that would collect the pressure applied by the shock tube and transfer it as uniformly distributed load on the mullions, resulting in a test area of 2.0 m by 2.0 m. The use of flexible steel sheet eliminated any possible resistance to blast pressure provided by the load application device. Furthermore, it was designed to simulate the mass of the double glass pane (107 kg) on both sides of the mullion within the tributary area of load distribution in the prototype curtain wall. The mullions were connected to the reinforced concrete beams with 200 mm by 150 mm cross-sectional dimensions and 2.5 m length representing supporting structural elements. The same reinforced concrete beam was modeled to conduct the numerical simulation. The beams were reinforced with 6 bars as illustrated in Fig.4.8. The first mullion (M-C1) was connected at the ends using brackets attached to the front end of the concrete beam (slab) and two opposite sides (web) of the mullion. The second mullion (M-C2) was connected at the ends using brackets attached to the top and bottom of concrete beam (slab) and the narrow side (flange) of the mullion. Figure 4.9 depicts M-C1 and M-C2 models. The third mullion (M-C3) was connected at mid-height over a concrete beam (slab) and connected using brackets attached to the top and bottom of the concrete beam (slab) and the narrow side (flange) of the mullion. The fourth single mullion (M-C4) was connected at mid-height over a concrete beam (slab) by using brackets attached to the front end of the beam (slab) and the two opposite sides (web) of the mullion. Figure 4.10 indicates M-C3 and M-C4 models. The L-shape angle steel brackets had dimensions of 200 mm × 200 mm with 3 mm thickness.

The next 5 model validations were performed against tests of mullions with damper material placed at the connections. M-C5 and M-C6 had XRD rubber placed between the connection plates and the concrete support using the continuous mullion configuration. Figure 4.11 shows the models for these mullion tests. M-C5 had 216 mm × 279 mm XRD rubber with 12.7 mm thickness placed between the concrete beam and the mullion at mid-length with the

brackets attached on the web of the mullion. In this test there was no rubber placed at the far end supports. M-C6 had the same size XRD rubber placed at mullion mid-length, with the brackets attached to the flange of the mullion. This mullion had the same size XRD also placed at the end supports. These details are illustrated in the models shown in Fig. 4.11. The remaining three mullions were equipped with springs at the end supports. The springs had different stiffness levels and lengths as illustrated in Fig. 4.12. The springs were secured between two steel plates that were connected to the concrete beams and the mullions. The steel plates on the mullion side had dimensions of 250 mm $\times$ 75 mm with 12.7 mm thickness, and on the concrete side they had 200 mm $\times$ 200 mm dimensions with 12.7 mm plate thickness.

The curtain wall model configuration followed the details used in the test specimens, so the transom sections were secured between right and left mullions. The modeled mullion and transom section sizes are illustrated in Table 4.1. The steel sheet was modeled as a square sheet with a dimension of 1981 mm with a thickness of 1 mm. The steel sheet entirely covered the mullions and transoms, except for the 25 mm segment at the top and bottom ends of the mullions. Based on the experimental tests, four simple supports were defined for each finite element model. The location of each support was 200 mm from the top and bottom ends of the mullions. The white crosses marked in Figure 4.1 exhibit the location of simple supports in all models. The two L-shape retrofitting steel plates per mullion were modeled, forming a U-shape plate on top of the aluminum mullions (and transoms when used). Table 4.2 presents the geometry of the retrofit technique used. The second model had retrofitting steel plates only on the mullions, whereas the third model also had the transoms retrofitted with steel plates.

For the analysis of the single mullions, they were secured between the concrete beams and HSS sections. The blast load was transferred to the mullion models through a load application device having 2.0 m $\times$ 2.0 m surface area and 3.5 mm thickness, with 107 kg of total mass. The supporting concrete beams and the HSS sections were specified as boundary conditions. Table 4.1 indicates the section sizes and material characteristics of the models generated for individual mullion tests.

4.2.2 Finite Element Discretization and Material Constitutive Modelling

The most appropriate material models, element types and mesh sizes were selected based on previous research. The mullions, transoms, U-brackets, side brackets, and retrofit plates were modeled using two-dimensional shell elements (***SECTION_SHELL**) because the sections were hollow and/or thin. In addition, the protective film layer was simulated using the shell section due to its thinness. All the shell elements included capabilities to simulate the membrane, bending and shear deformation. The fully integrated shell element formulation modified for higher accuracy was used in this study, which is the most precise option and should be used for such elements with featured particular formulation. Although this formulation is computationally expensive, it prevents Hourglass effects due to using default Belytschko-Tsay formulation (Belytschko et al. 1989; 1992). Triangular shells can be mixed with quadrilateral shells within the same material property set, provided that the element sorting is flagged. Three-dimensional plane stress constitutive subroutines are implemented for the shell elements which update the stress tensor such that the stress component normal to the shell mid surface is zero (LS-DYNA Manual Vol 1). The integration points are stacked vertically at the centroid of the element. To enhance the accuracy of results, a mesh-size sensitivity analysis was carried out to achieve the best element mesh size for the curtain wall sections. Eight mesh sizes were assessed between 15 mm and 50 mm with 5 mm increments. Eventually the mesh size of 25 mm was considered to be the optimum element mesh size for all sections. In addition, the Hourglass effect and the balance of developed energy in the model was checked as an additional assurance for the precision of the numerical results; however, the fully integrated formulation used for shell elements was sufficient to prevent the Hourglass effect.

The concrete beam, XRD rubber, and glass were modeled by using (***SECTION_SOLID**) because of their volume. Three-dimensional solid mesh elements are commonly deemed to be suitable for modelling elements having effective thickness. The 3-D mesh elements can have four or more nodes. Eight-node hexahedron solid mesh element was selected for use in the current investigation. Figure 4.13 illustrates the hexahedron solid mesh

element. There is a wide range of solid element formulations available in LS-DYNA for modelling concrete, rubber, glass etc. These elements 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. These solid elements work well for problems involving excessive deformations as in the case of blast events. The selection of a material model depends on the fundamental behavior 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).

Two-dimensional beam mesh elements (***SECTION_BEAM**) were used to model the rebars in the reinforced concrete beams. Hughes-Liu element with the cross-section integration beam element formulation, having 2 x 2 Gauss integration points has been adopted in the current investigation to model the circular cross section of rebars (Hughes et al 1981a; 1981b) having the same rigidity (stiffness) like the actual rebars. LS-DYNA provides a wide range of beam element formulations for beam sections, 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 others in its library. The Hughes-Liu element has three translational and three rotational degrees of freedom at each node. The main advantage of this beam formulation is its compatibility with solid concrete elements. It is generated from the 8-node solid element, as shown in Figure 4.14, which makes the beam element compatible with the solid element (LSTC 2006).

One-dimensional (***SECTION_DISCRETE**) was used to model the springs inserted on the ends of single mullions and connected to the concrete beams. The main purpose of this discrete section is to define spring and damper elements for translation and rotation. Resultant forces and moments of a discrete beam are output in the local coordinate system. There is a clearance option in the discrete section, which defines a compressive displacement that the spring sustains before beginning the force-displacement relationship given by the load curve defined in the material

selection. If a non-zero clearance is defined, the spring is compressive only. The deflection limit in compression and tension is restricted in its application to no more than one spring per node subject to this limit, and it is intended for deformable bodies only. When the limiting deflection is reached, momentum conservation calculations are performed, and a common acceleration is computed in the appropriate direction (LSDYNA Manual 2011).

***PIECEWISE_LINEAR_PLASTICITY** (*MAT_024) was used to model the aluminum and steel components including: mullions, transoms, side brackets, U-brackets, L-shaped retrofit plates, and steel sheets. Piecewise is an elasto-plastic material with an arbitrary stress versus strain curve while arbitrary strain rate dependency can be defined. In addition, failure based on a plastic strain or a minimum time step size can be defined (LS-DYNA manual 2007). This material model is able to consider the elastic-plastic behavior of the sections. Hence, the material behavior can be treated as a bi-linear stress-strain curve or can be treated as a multi-linear curve by defining a set of stress-strain points. The multi-linear behavior can also be treated by defining a load curve in terms of effective stress vs. effective plastic strain. Bi-linear stress-strain curve is the simplest form of representation, but the multi-linear curve captures the behavior more accurately. The effective stress versus effective plastic strain curve for aluminum (T-6061), and steel (hot rolled AISI 1020) was defined. Figure 4.15 and Figure 4.16 depict the effective stress versus effective plastic strain curve for the materials, respectively. Table 4.3 represents their material properties. The basic input material parameters for this model are mass density, yield stress, Poisson's ratio, Young's modulus and tangent modulus.

*** PLASTIC_KINEMATIC** (*MAT_003), was used to specify material properties for glass, protective film layer and reinforcement. This is an elasto-plastic material model that was developed with either isotropic hardening, kinematic hardening or the combination of both (LSTC 2015). There is controversy in the literature on the significance of strain rate effects on glass. (Zhang 2013) and (Holmquist et al. 1995) adopted Johnson Holmquist Ceramic constitutive model (JH2) which considers the effect of high strain rate on glass. The use of (JH2) and other dynamic glass material models have not been validated against experiments

and the effect of high strain rate on glass was not well-justified. Kinematic hardening due to Bauschinger effect was incorporated into 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 4-17. Strain rates are user options and the strain rate effects are based on Cowper-Symonds model. Input parameters to be defined for this material are: material density, Poisson's ratio, elastic modulus, yield stress, tangential modulus, and rupture strain, representing a typical bi-linear stress-strain relationship to model the steel reinforcing bar. Table 4.3 indicates the material properties of glass, film layer and rebars.

The concrete beam was modeled using * **CSCM_CONCRETE** (*MAT159). The CSCM (continuous surface cap model) was first derived for examining 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 essential 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 are 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). Concrete is known to be a nonlinear material that exhibits inelastic behavior, especially under extreme loading. The model requires only the input of concrete density, unconfined compressive strength and aggregate size. The material properties of concrete are presented in Table 4.3. Concrete constitutive material models can be categorized into elastic models, plastic models, elasto-plastic models, continuum models, endochronic models and fracture models. It has three invariant yield surfaces as shown in Figure 4.18. They define yield surfaces at the onset of plastic stage. 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, as well as when the plastic strain increases, as shown in Figure 4.19.

***CRUSHABLE FOAM (*63)** was used to model Poron XRD rubber as a polymer material. This material in LS-DYNA is dedicated to model crushable foam with optional damping and tension cut-off values. Furthermore, unloading is fully elastic and tension is treated as elastic-perfectly-plastic material at the tension cut-off value. Basically, the crushable foam plasticity models are intended for the analysis of crushable foams that are typically used as energy absorption structures. It can be used to model the enhanced ability of a foam material to deform in compression due to the cell wall buckling processes (it is assumed that the resulting deformation is not recoverable instantaneously and can, thus, be idealized as being plastic for short duration events including blast). The elastic part of the response is specified as described in linear elastic behavior. So, only linear isotropic elasticity can be used. For the plastic part of the behavior, the yield surface is a Mises circle in the deviatoric stress plane and an ellipse in the meridional (p - q) stress plane. This is indicated in Figure 4.20. The hardening curve must describe the uniaxial compression yield stress as a function of the corresponding plastic strain. In defining this dependence at finite strains, stress and logarithmic strain values should be given. Both models predict similar behavior for compression-dominated loading. However, for hydrostatic tension loading, the volumetric hardening model assumes a perfectly plastic behavior, while the isotropic hardening model predicts the same behavior in both hydrostatic tension and hydrostatic compression. The essential parameters for this material model are as follows: Mass density, Young's modulus, Poisson's ratio, and yield stress versus volumetric strain curve. Table 4.3 present the material properties entered for XRD rubber material. Figure 4.21 illustrates the material properties and the curve of yield stress versus volumetric strain relationship for XRD rubber, respectively.

The springs were modeled by considering ***SPRING_ ELASTOPLASTIC (*S03)** material model. This spring is specified for (***ELEMENT_DISCRETE**). It provides an elastoplastic translational or rotational spring with isotropic hardening located between two

nodes. Only one degree of freedom is connected. In this regard, the elastic stiffness (force/displacement) or (moment/rotation) is required to be defined in this material model.

4.2.3. Modelling of Contacts

Defining a correct and suitable contact between parts and elements plays an important role in the accuracy of results. In blast analysis, the deformations can be very large and the determination of the location and the nature of contacts may be difficult or impossible prior to the analysis. For this reason, the automatic contact options are recommended as these contacts are non-oriented, meaning they can detect penetration coming from either side of a shell element (LS-DYNA Manual 2011). Basically, a contact is defined by identifying (via parts, part sets, segment sets, and/or node sets) the locations to be checked for potential penetration of a slave node through a master segment. In finite element analysis, it is recommended that redundant contact, i.e., two or more contacts producing forces due to the same penetration, to be avoided by the user as this can lead to numerical instabilities. The assessment of the types of connection retrofitting approaches for curtain wall and single mullion models included the transmission of applied blast loads from the steel sheet to the elements. Consequently, the selection of a variety of appropriate contact types become necessary.

It is essential to define contact points between the parts that impact each other. The contact search algorithms employed by automatic contacts make them better suited than the older contact types for handling disjoint meshes. The steel sheet and the load application device form main surfaces at which blast loads are directly applied. These elements transfer the blast loads directly to the models, and ***CONTACT_AUTOMATIC_SURFACE_TO_SURFACE** was considered for this purpose. This is a recommended contact type since in blast simulations the orientation of parts relative to each other cannot always be anticipated as the model undergoes large deformations. Contact automatic surface to surface was defined between steel sheet and curtain wall sections. The same contact was also specified between load application device and single mullion models. By selecting this contact, it is possible to either designate the master and slave surfaces or leave it up to the software to evaluate the entire model and select the master and slave automatically. In general, the surface or the part with larger size

with higher number of nodes should be selected as the master segment, and the thinner part is defined as the slave. Therefore, the load application device and steel sheet elements were assigned as master parts, whereas other single sections were selected as the slave segments.

The other application of contacts in LS-DYNA involves two parts or segments that are tightly connected to each other; or they are completely anchored to other parts. **CONTACT_TIED_SURFACE_TO_SURFACE** was defined to provide attachment between segments and solid parts in connection regions. This contact was considered for connections at anchor locations where detachment was not observed or expected. Basically, the tied contact is very similar to the automatic general contact; however, once the software detects contact between the master and the slave surfaces, it ties those two surfaces together. It should be noted that the slave nodes are constrained to move with the master surface simultaneously. At the beginning of the simulation, the nearest master segment for each slave node is located based on an orthogonal projection of the slave node to the master segment. If the slave node is considered close to the master segment based on the established criteria, the slave node is moved to the master surface. This way, the initial geometry may be slightly altered without invoking stresses. As a result, common nodes react together and do not get detached from each other unless failure/yielding occurs in one of the elements. As the simulation progresses, the iso-parametric position of the slave node with respect to its master segment is held fixed using kinematic constraint equations. Since rotational degrees of freedom for slave nodes are not constrained, the most common use of this contact can be for connecting solid parts to each other. In this project, steel plates placed on the ends of springs were connected to concrete beams and mullions by using this type of contact. Likewise, glass parts were attached to the protective film layer using tied surface to surface contact. In addition, Poron XRD rubber parts were attached to the concrete beam using this contact type by considering concrete beam as master and rubber as slave.

Special attention was paid to main contact points for simulating elements anchored to each other. ***CONTACT_TIED_SURFACE_TO_SURFACE_OFFSET** was used to simulate the connections by mechanical anchoring, like rivets and bolts. Technically, this

contact type works the same as tied surface to surface contact, but an offset distance between the master segment and the slave node is permitted. This contact type works best if the surfaces are very close, since moments that develop due to the offset are not taken into account. Offset tied contacts use a penalty-based formulation and thus can be used to tie rigid bodies. This interface uses the constraint method. Slave nodes are located within master segments during initialization, and then treated as dependent degrees of freedom during the simulation. This reduces the size of the global stiffness matrix. Furthermore, the offset option helps software remove the probable gaps between the master and the slave. Therefore, it eliminates any possible geometric errors which could be inadvertently input by users. With this option, the force and moment resultants are transferred. The numerical analysis of curtain wall specimens benefited from this type of contact, including i) steel sheet surface to curtain wall sections; and ii) U-brackets, side brackets and retrofit plates to mullions and transoms. In the curtain wall models, only one line of rivets was used. Therefore, only one line of nodes was defined as contact segment around curtain wall models. When two lines of rivet were used for the middle transom with a zigzag pattern, two sets of nodes were selected to develop the tied contact. Moreover, this contact was used to connect the retrofit brackets between single mullions and concrete beams, either with or without using dampers.

***CONTACT_TIED_SURFACE_TO_SURFACE_FAILURE** was considered to glue the side brackets to CW1 since loss of composite action was expected due the high level of stress previously observed (Saatcioglu 2020). This contact was categorized as one of the tiebreak contacts. It has the added feature of including a failure criterion. These contact types are usually penalty-based. Any contact specified for adhesion region between two elements is required to take the material properties of the adhesive glue into account. Furthermore, it should have an option to define the disconnection criteria after reaching a specific level of stress in the element. Initially, the side brackets were expected to be perfectly attached to the mullions and transoms to allow synchronous deformations. Nevertheless, disconnection was observed during the actual test indicating the lack of composite action. This was modeled by specifying tied surface to surface failure contact.

Coupling of concrete and rebars plays an important role to have an acceptable reinforced concrete model. ***CONSTRAINED_LAGRANGE_IN_SOLID** keyword allows the coupling process by selecting concrete as the Master and the reinforcement as the Slave. Perfect bond has to be defined between the two element types (solid and beam) so that forces and strains are transferred between concrete and reinforcement. 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. However, it 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. Constraining acceleration and velocity at the nodes are done by choosing the constraint method (CTYPE=2). In this study, the constrained LaGrange in solid was used to couple the concrete and steel rebars. The coupling algorithm can be implemented at the beam nodes only, or within a couple of different points in the beam element.

4.2.4. Loading

The blast load was applied directly to the top segment of outer steel sheet as well as the load application device, and all elements were included in receiving the blast load. A ***LOAD_SEGMENT_SET** was specified to give the blast command to the software on the entity section of LS-DYNA. The whole exterior surface of steel sheet and the load application device were defined as the main segment for applying blast loads. For this purpose, ***DEFINE_CURVE** keyword was employed, and the specific reflected pressure time history recorded during the test was used to define the curve. Figure 4.22 indicates pressure-time history of the blast shots applied during testing, which were used for the numerical models as well.

4.3 Numerical Model Validation

Numerical models were analyzed to validate against the experimental results. The main purpose of this research was to improve the curtain wall connections by using different retrofit techniques including: strengthening the internal U-angle connections, adding side brackets to the connections adjacent to the mullions and transoms, hardening curtain wall sections and

retrofitting the connections between mullion and structural members (concrete beam/slab) by using steel brackets. In addition, the role of damping materials (XRD rubber, and spring) was investigated when placed at mullion-concrete substrate connections beam. Therefore, the numerical modelling, their validations, as well as the subsequent parametric investigation focused on the variables that affect the connections. The validation was conducted against the experimental measurements and observations. The numerical results were also used to generate data on support reactions to assess connection forces and Von Mises stresses.

Phase I of the experimental research focused on mullion-to-transom connections. The first curtain wall (CW1) failed due to the rupturing of vertical mullions. A similar behavior was observed numerically. The experimental and numerical observations at failure are illustrated in Fig. 4.23, showing remarkable similarities in response. Both mullions suffered yielding at mid-height. The steel U-brackets used inside to attach the mullions and transoms fulfilled their functions throughout the response, providing sufficient connection strength. The externally glued L-shape angles between the mullions and the transoms detached and flew away. This behavior was also captured by the numerical model. The experimental and numerical deflections are compared in Table 4.4. Relatively smaller deflection computed by the numerical simulation was attributed to the support sliding observed during testing. Indeed, the mullions slid down at the top support after developing large deflections, causing additional mullion deflections. Because of the failure of the mullions in CW1, the second curtain wall (CW2) mullions were hardened by steel plates. This resulted in stronger mullions, which survived the blast load both in the experiment and numerical simulations. This curtain wall suffered from the failure of the middle horizontal transom which had not been hardened. The observed failure mode is illustrated in Fig. 4.24, showing acceptable agreement between the experimental and numerical results. The externally placed L-shape brackets, mechanically attached to the mullion-transom connections survived the blast load without any damage. Table 4.4 presents the in-plane and out-of-plane deflections. Considerable membrane action was observed in CW2. Specimen CW3 had both vertical mullions and the middle horizontal transom hardened by means of steel plates. This specimen also had mechanically connected steel angles placed

at mullion-transom connections. CW3 resisted high levels of blast loads. Incrementally increasing three shots were applied on the specimen. The numerical model captured the experimental observations very well. Figure 4.25 shows the comparison of numerical and experimental response for CW3 at the end of each shot. The first shot caused inelastic response, generating permanent deflections in the curtain wall. There was no tensile rupture observed, but compression buckling of the steel plates on vertical mullions was evident. The numerical model also showed buckling of the inner aluminum mullion sections. The second shot, only marginally higher than the first shot, resulted in increased inelastic deformations. The third shot resulted in excessive bending of the vertical mullions with increased buckling of steel plates on the compression side. The numerical model indicated that the steel retrofit elements were eliminated at the supports/boundaries indicating failure. Also, the web buckling observed in the hardening plates was attributed to the distress of the core aluminum elements, which led to the elimination of these elements, hence placing more burden on the steel plates. Besides, aluminum elements were eliminated that resulted in higher stress in the remained retrofit elements. Table 4.4 summarizes experimentally measured and numerically computed maximum deflections for all shots of CW3.

The second phase of the numerical validation focused on mullion-to-substrate structural element connections. The performance of the nine single mullion tests conducted was simulated to validate the numerical models and the procedure. The analysis of the first single mullion M-C1 showed similar behavior to the test and suffered tensile rupturing at mid-height on the tension side. No failure was observed at fixed boundaries; however, considerable stress was observed on the mullion around the supporting concrete beams. The lower nodes of brackets located on the top concrete beam, and the upper nodes of brackets located on the bottom concrete beam received higher stresses. Figure 4.26 illustrates M-C1 numerical observations. Only the legs of brackets attached to the mullion web were under high pressure. The second single mullion M-C2 showed similar behavior as the actual test, but the model showed more severe damage, perhaps because it was fully fixed at the supports, as opposed to the test mullions, which showed some relaxation of support rotations during the testing process.

The model failed by rupturing of the mullion at its critical mid-height location. The mullion nodes near the supporting concrete beams experienced high stresses and strains. The bracket legs attached to the mullion flange on the span side (rather than the support side) experienced excessive stresses and developed inelastic bending. Figure 4.27 illustrates observations for M-C2 during the test and the numerical simulation. The numerical simulation of the third mullion M-C3 exhibited excellent similarity to the actual test performance. The same failure pattern was observed in mullion close to the middle concrete beam. It suffered from connection failure when the brackets were bent excessively, reducing the original 90-degree angle to approximately 40 degrees. The concrete slab bared against the mullion flange, crushing the aluminum mullion severely. However, no failure was observed on the top and bottom boundaries. Figure 4.28 presents experimental and numerical comparisons and brackets' stress distribution. The legs of brackets connected to the concrete beam were in tension, whereas the other legs (attached to the mullion flange) were bent excessively due to compressive stresses induced by the mullion. The fourth mullion (M-C4), with the brackets connected to the mullion web, showed superior performance because of the increased bearing surface area, preventing the crushing of the aluminum. Furthermore, the bracket leg that was attached to the mullion web strengthened the mullion in the compression critical zone, which prevented the crushing of the aluminum under bearing stresses. The brackets maintained their rigidity as the rigid concrete slab prevented the bending of the brackets. Figure 4.29 shows the comparison of experimental and numerical observations, as well as the stress distributions in the side brackets. A significant web buckling was observed at mid-height of the mullion segments above and below the concrete support during testing on the compression side. This was not observed in the numerical model. The difference in response was attributed to the difference in loading. The numerical model was subjected to perfectly distributed blast pressure, whereas the test mullion was loaded by the load application device, which consisted of individual HSS sections that applied the lateral load at discrete points. While this was close enough to the distributed load when the supports were at the ends and the mullions were loaded along their entire height (as in the case of M-1 and M-2), M-C4 consisted of two half mullions with shorter spans above

and below the concrete beam. This amplified the effect of the load application device over shorter spans. Nevertheless, the numerical simulation captured the overall behavior of the mullion.

The second segment of single mullion tests was conducted on mullions connected to the substrate through damping materials. Mullion M-C5 had XRD rubber material inserted between the mullion and the supporting concrete beam. It was companion to M-C4, which was rigidly connected to the concrete beam without any damping material. The numerical model generated similar behavior to the actual test mullion. No failure was observed in the mullion, nor at the boundaries. The XRD rubber dampened the load transfer on the mullion and reduced stresses on the mullion. Figure 4.30 shows the performance of M-C5 experimentally and numerically. More uniform stress distribution was observed on the steel brackets. The flange connected to the mullion experienced reduced stress as well. The middle portion of the rubber, where the mullion section was placed, experienced high plastic strains. The same rubber material was used during the testing of M-C6. This time however, XRD was also placed at the far end supports, in addition to the mid-height support at the location of the concrete beam. The numerical simulation of M-C6 demonstrated excellent agreement with test observations. Figure 4.31 shows experimental and numerical observations. However, the thickness (and the stiffness) of the XRD rubber used was not high enough to save the mullion from crushing, but less damage was observed at mid-height compared to the companion test without the rubber (M-C3). No damage/failure was observed at the end boundaries, since the mullion experienced significant damage at the mid-height connection, damping the energy.

The next three single mullion tests included concrete beam supports at the ends of mullions with springs used as damping devices. Mullion M-C7, with a spring having 8 N/mm stiffness at each end showed significant improvement in response. The applied load on the mullion was high-enough to compress the springs to their ends. Beyond this point the mullion performed as a simply supported element and started bending between the supports. Nevertheless, it survived the blast load without failure. It should be noted that the companion tests without the springs (M-C1 and M-C2) suffered failure at mid-height under the similar

applied load. The numerical simulation and experiment observations correlated well with each other, as depicted in Figure 4.32. The next mullion with springs at the ends was M-C8. This specimen had double springs at each end with a stiffness of 30 N/mm. The applied load was higher than the load applied on the previous mullion (M-C7). Once again, the springs were compressed to their ends, beyond which the mullion started taking load and developed flexural deformations. It eventually suffered failure at mid-height through local buckling. The numerical simulation captured the same behavior, as seen in Fig. 4.33, except it also indicated tension rupturing at mid-height. The last mullion tested was M-C9 with double springs at each end having a spring stiffness of 23 N/mm per end. This mullion was subjected to a lower blast load, as compared to the previous two cases. The mullion essentially remained elastic. The same behavior was captured by the numerical simulation, as illustrated in Figure 4.34, which also shows the distribution of tensile and compressive stresses.

As discussed above, all twelve test specimens were modeled and examined numerically. Figures 4.23 through 4.34 show the comparisons between the experimental and numerical results. Tables 4.4 and 4.5 compare the deflections obtained numerically with those recorded during the tests. The validation of the LS-DYNA analysis results against experimental data confirms the applicability of the models for further analytical parametric investigations.

4.4 Support Reaction Assessment

Support reactions for mullions and transoms are important quantities for connection design. Therefore, the support reactions at the ends of mullions, where they are connected to the supporting substrate, and at the ends of transoms, where they are connected to vertical mullions as supporting elements were computed analytically. The analytical calculations include LS-Dyna FEM analysis, manual SDOF analysis using UFC-Charts, and computer software CW-Blast. The test specimens were not instrumented to record reaction forces. Therefore, the validation of the SDOF analysis and CW-Blast software as design tools were made against the numerical simulation obtained from the LS-Dyna software.

4.4.1 Mullion Support Reactions

The curtain walls and mullions tested in the experimental program were modelled as SDOF systems. The support reactions were computed using the UFC-charts prepared for manual SDOF dynamic analysis, and were compared with the numerical values obtained from LS-DYNA software. The UFC-charted solution is known to be a reliable SDOF approach to calculate yield resistance, yield displacement, elastic and plastic range support reactions. Appendix C presents the UFC-charted analytical solution. The results obtained are compared to those attained by LS-DYNA analysis in Table 4.6. The LS-DYNA analysis simulated the behavior of the test mullions. The numerical models included the load application device, which transferred the applied pressure up to near maximum displacements and then detached from the mullions. The SDOF analysis assumed continuous application of blast loads. This difference in modelling resulted in somewhat lower support reactions computed by LS-DYNA. However, the results indicate an acceptable level of correlation and suitability of the SDOF analysis for use in connection design. Similar comparisons were made for the mullions tested with dampers at the supports.

4.4.2 Transom Support Reactions

The support reactions for transoms provide connection design forces between the transoms and the supporting mullions. The curtain wall specimens tested in Phase I of experimental research consisted of two vertical and three horizontal transoms supporting two panes. SDOF analysis was conducted using Curtain Wall Blast (CW-Blast) software. This is illustrated in Appendix D. Table 4.7 compares the middle transom support reactions. An acceptable correlation was found between SDOF and the finite element results; however, LS-DYNA showed 20%-25% higher support reaction results. This can be still considered as an acceptable result; nonetheless, more investigations on different curtain wall systems with various aspects, ratios and material properties are needed to optimize the accuracy of the outcome.

By comparing the overall results obtained from single degree of freedom and numerical finite element analysis, a good correlation was observed. The validation of LS-DYNA analysis results against experimental and analytical data confirmed the applicability of the models for further parametric investigations.

4.5 Parametric Investigation

Following the validation of the numerical models and the procedure, a parametric investigation was undertaken to expand the experimental findings and to generate additional data for connection design. A total of six parameters were considered. This necessitated an additional 9 numerical models. The parameters considered include: i) validity of the use of steel sheets in the experimental program to simulate glazed glass panes; ii) the effect of retrofitting internal mullion-transom connectors; iii) the effect of external steel brackets to strengthen mullion-to-transom connections; iv) the effect of steel bracket thickness used at the mullion-substrate connection; v) the effect of parameters governing the dampers used at the mullion-substrate connections.

4.5.1 The Effect of Using Steel Sheets Instead of Glazed Glass Panes in CW1

The curtain wall specimens tested in the experimental program had steel sheets between the mullions and the transoms to simulate glazed glass. This was done for convenience, with due considerations given to the glass mass and ability of the sheets to simulate post glass breakage membrane action. The steel sheets were selected to be gauge #22, which had the same mass as a double pane insulated glass unit (IGU) consisting of 6 mm glass pane each. The rigidity of the steel sheet represents post-breakage glazed window behavior. The glazed window (glass + protective film) does not have any flexural rigidity, neither does the steel sheet. They both perform in the membrane mode of behavior (membrane tension). Therefore, the use of steel sheets is deemed to be a practical and cost-saving alternative for use in tests if one does not want to break glass after each test. It should be noted that the pre-cracking behavior of glass was neglected as the response was governed by the resistance of glazed glass in the post-cracking range. The numerical model that was developed for specimen CW1 was used with the same boundary conditions and configuration, but with the steel sheet replaced by

glass with protective film. The new model was labelled as CW1+G&F. The numerical model had one glass pane with a thickness of 8 mm and one film layer of 0.8 mm thickness. The glass pane and film layer were attached to each other. The film was also anchored to the curtain wall mullions and transoms. Table 4.3 represents the material properties used in CW1+G&F model. The pressure-time history recorded for CW1 (Fig 4.22a) was also applied on CW1+G&F. Figure 4.35 shows CW1+G&F before and after the blast load. The results of the two analyses showed similar behavior. The mullions suffered failure at mid-height and many nodes at boundaries were eliminated due to excessive strain. The same behavior was observed in the experiment and its validated CW1 model (both using steel sheet). Figure 4.36 shows the comparison of CW1 actual test, CW1 numerical result and CW1+G&F numerical observations. Table 4.8 presents the results for deflections and support reactions for the three cases. The transoms in CW1+G&F had more oscillations during dynamic response since the rigidity of glass + film was less than the steel sheet. It also showed 7.5 % and 5% less mullion and transom support reactions compared to CW1. It can be concluded that similar behavior and performance can be achieved by using steel sheets instead of glazed glass, confirming the validity of the use of steel sheets in the experimental program to simulate glazed glass.

4.5.2 The Effect of Internal Steel U-Brackets at Mullion-Transom Connections

Non-blast-resistant curtain wall connections are designed to resist gravity and wind loads. Mullions and transoms are connected internally through aluminum U-brackets riveted on the members. Therefore, they are not strong enough to resist blast loads. During the experiment, internal steel U-brackets were used to connect the mullions to transoms (Fig 4.3 a). The effectiveness of steel U-brackets relative to the weaker aluminum brackets was examined. A new numerical model with weak aluminum brackets was created (CW1+WC) from the previously used CW1+G&F model. They were analyzed under the same pressure-time history that was used for CW1 (Fig 4.22a). Table 4.2 and Table 4.3 present the U-bracket section size and material properties, respectively. CW1+WC demonstrated that the weak connections were not able to maintain the integrity of mullion-to-transom connections. The transoms flew away as soon as the blast load impacted CW1+WC. This observation is

consistent with earlier curtain wall tests where the transom was detached during the test and resulted in the failure of the curtain wall (Saatcioglu 2017). Figure 4.37 exhibits the performance of CW1+WC at different stages of response. In contrast, the steel U-brackets in CW1+G&F performed very well without any detachment from the curtain wall. This comparison clearly shows the superiority of internal steel brackets bolted on mullions and transoms over the aluminum brackets riveted to the members. The riveted aluminum brackets provide a weak chain in the system and fail prematurely.

4.5.3 The Effect of Retrofitting Mullion-Transom Connections with External Steel L-Brackets

The vulnerability of ordinary mullion-transom connections designed for wind loads with aluminum U-brackets riveted to the members, and the positive effects of steel U-brackets bolted on the members were already discussed in the preceding section. Strengthening the connections by replacing existing internal brackets with stronger steel alternative poses challenges, and may be regarded as being impractical. Therefore, adding external steel brackets connected to the top and bottom of the transoms and the side faces of the mullions while keeping the internal weak aluminum U-brackets was numerically investigated. These locations for the brackets would avoid placing the connection bolts through the flexure critical regions of the mullions. Furthermore, the strengthening of the connections in the way suggested would include two brackets: one on the top and the other at the bottom would result in a stronger connection with better stress distribution. A numerical model that was similar to CW3 was developed to investigate the effectiveness of externally placed steel brackets on mullions and transoms that had already been hardened with steel plates, while having ordinary weak aluminum U-brackets internally, except this time glazed panes (glass plus film) were used instead of the steel sheets. The L-shape steel brackets had the geometric dimensions of $76 \times 76 \times 3$ mm. Table 4.3 present the material properties used in the model. The new model was labelled as CW3+T&B and the pressure-time history depicted in Fig 4.22a was applied. Figure 4.38 shows CW3+T&B before and after applying the load, demonstrating the effectiveness of the external brackets for strengthening the mullion-transom connections.

The effectiveness of external steel brackets was further investigated by analyzing the previously developed model for CW3 (with steel sheets as curtain wall panels, internal steel U-brackets, and externally placed steel L-brackets bolted on the flange of the transoms, one at each end) under the same set of blast loads used for CW3+T&B (with glazed glass panels, weaker internal aluminum brackets and externally bolted steel L-brackets at the top and bottom of the transom), i.e., the pressure time history shown in Fig. 4.22a. The results obtained from these two analyses are compared in Table 4.9. The comparison indicates that CW3+T&B had higher maximum deflection at the mid-length of transoms compared to that for CW3. This can be explained by the weak U-brackets in CW3+T&B, which had negligible contribution to the rigidity of the mullion-transom connections. This increased the support rotations at the ends of transoms, resulting in higher transom deflections. CW3+T&B experienced 20% and 74% higher maximum mullion and transom deflections respectively, when compared with those for CW3. Furthermore, a significant glass elimination was observed in CW3+T&B which indicated higher out-of-plane deflection of the system, especially at the middle transom region. The type of mullion-to-transom connections used in CW3+T&B is recommended for strengthening the connections. However, the connection capacity, especially the bolt design capacity should be checked as high strain values were observed in the anchorage system.

4.5.4 The Effectiveness of External L-Brackets for Curtain Walls with Internal Steel U-Brackets.

During the experimental phase of the current investigation, connection failures were not observed at mullion-transom connections, unlike the earlier tests in which the connections had failed (Saatcioglu 2017). This was attributed to the use of stronger internal steel U-brackets at the connections in place of the weaker aluminum U-brackets riveted for resistance to wind pressures in the earlier tests. Nevertheless, the role of external steel U-brackets remained a controversial question. The use of externally-placed steel U-brackets, connected to the flange of the transom as was done in CW3 was investigated numerically. Numerical analysis of CW3 was conducted with and without the external U-brackets, while in both cases internal steel U-connectors were used. Table 4.1 and 4.3 present the section size and material properties

respectively. The model without the external brackets was labelled as CW3_NB. Both models were subjected to the three shots illustrated in Fig 4.22c,d,e, which were used to test the companion test wall (CW3). The results indicate little improvement imparted by external brackets when internal steel connectors were bolted on the members. It was observed that the connection stresses are resisted primarily by the internal U-brackets. Figure 4.39 compares CW3_NB (without external side brackets) and CW3 (with side brackets) after each shot. It should be noted that the presence of side brackets resulted in less element elimination in the core aluminum mullions underneath the hardening steel plates. Table 4.10 compares deflections and support reactions for CW3 and CW3_NB. It is illustrated that the level of stress on the side brackets is low and removing them would not impact the strength and performance of curtain wall connections considerably.

4.5.5 The Effect of L-Bracket Thickness on Mullion-Substrate Connection

During the experimental phase of research, a severe failure was observed in single mullion test M-C3, representing a continuous mullion over a concrete slab. The brackets used in the test had a 3 mm thickness and were connected on top and bottom of the concrete beam that represented the structural slab system (substrate). The failure was associated with excessive bending of the brackets, showing lack of sufficient thickness and strength. Essentially, the brackets were not able to provide sufficient bearing area and bending strength when they were connected to the flange (narrow side) of the mullion. The end result was the bearing of the concrete on the flange of the aluminum mullion, resulting in the crushing of the material under bearing stresses. It is worth mentioning that a significant improvement was observed in M-C4 when the brackets were connected to the web of the mullion (sides of the mullion), which strengthened the web of mullion while also increasing the bearing surface area on the concrete slab. Therefore, a numerical investigation was conducted to assess the effect of using thicker brackets on M-C3. Two additional models were generated for this purpose, similar to M-C3, with increased thickness to 10 mm (M-C3_10) and 20 mm (M-C3_20). Figure 4.22h shows the pressure-time history applied to the models. Table 4.1 and table 4.3 present section size and material properties of the models, respectively. The results indicated that

thickening the brackets can provide improved mullion performance. By thickening the brackets, less element elimination was observed on the tension side of the mullion. However, many elements were buckled on the compression side of the mullion. In other words, mullion deformation was controlled on the tension side because of the thickened brackets, which resulted in reduced bending. However, the blast load caused considerable local buckling on the compression side. Figure 4.40 compares bracket deflections after the blast load. When the brackets were stiffened due to increased steel thickness, a larger area (concrete + brackets) was provided as bearing surface area, reducing the stress on the mullion. Brackets on CW3 bent 40 mm, and the mullion experienced severe crushing. Brackets on M-C3_10 and M-C3_20 bent up to 23 mm and 3 mm, respectively. Figure 4.41 shows the maximum deflection on the top node of brackets. The comparison of these three bracket thicknesses indicates that increasing bracket thickness provides protection against excessive deformations in mullions within the connection region if the brackets are to be attached to the flange of the mullions. An alternative to increasing bracket thickness is to change the orientation of the brackets to connect them to the sides of the mullions (web of the mullion) and the end of the concrete slab.

4.5.6 The Effect of Using Dampers at Mullion-Substrate Connection

The use of dampers at the mullion-substrate connection was investigated as an innovative strategy to reduce connection design forces in the experimental phase. XRD rubber and elastic steel springs were used at mullion-substrate connections to investigate their effectiveness in reducing connection force demands. The results proved that the concept is feasible and dampers could reduce the damage to mullions and forces at the connections (Nasseralshariati and Saatcioglu 2023). However, it became evident that the design parameters for the dampers had to be investigated further numerically to expand the findings. The effects of two parameters were investigated numerically: i) damper stiffness and ii) available gap between the mullion and its supports before the mullion comes in full contact with the supports. The numerical work involved the use of the analytical model that was generated for M-C9 with 200 mm net gap at the supports. It was used with different spring stiffness levels and gaps. The first analysis was conducted with an increased spring stiffness of 100 N/mm. This model was

labelled as SP-100. The second analysis was conducted on the same model after replacing the springs with 150 mm by 100 mm XRD rubber. The strain-stress curve of the material was specified as crushable foam and the stiffness was modified to provide 100 N/mm stiffness. This numerical model was labelled as XRD-100. The thickness of the rubber was specified as 260 mm so that the provided net gap would be 200 mm, considering the rubber volume and its secondary stiffness hardening. Table 4.1 and Table 4.3 illustrate the geometric and material properties, respectively. Figure 4.42 exhibits the numerical models prior to the application of the load. Both SP-100 and XRD-100 were subjected to progressively increasing reflected pressure values of 30, 40, 50 and 60 kPa. These reflected pressures values were selected after reviewing the experimental results. Only the positive phase of blast with 20 ms duration was considered. It is worth recalling that the companion model M-C1 without any damper suffered failure under 24 kPa reflected pressure and developed 18.24° support rotation; but by inserting the springs at the supports (M-C7 and M-C9), mullions had no failure under the same load.

The above analysis was repeated with increased spring and XRD stiffness by a factor of 2.0. Consequently, the spring stiffness was increased to 200 N/mm (SP-200), and stress-strain curve of rubber was set to 200 N/mm stiffness (XRD-200). The applied pressure was also increased by a factor of 2.0 (60, 70, 80, and 90 kPa reflected pressure).

The results were assessed based on support rotations. The computed support rotations were compared with the response limits given in CSA S850 for different levels of protection. After the first shot on SP-100, zero support rotation was attained since the mullion did not fully compress the springs. This mullion achieved high level of protection without any bending of the mullion. The second shot on SP-100 also indicated zero support rotation with high level of protection. The mullion moved by 173 mm at the supports but did not compress the springs fully. The third shot was able to squeeze the springs completely and mullion came in contact with the concrete supports. At this stage of loading the mullion developed 3° support rotation, and medium level of protection was attained. The maximum mid-height deflection was 232 mm, consisting of the translation at the ends (at supports) and inelastic bending of the mullion. After the 4th shot, the springs were compressed completely and the mullion developed

excessive inelastic displacement. Support rotation of 7° was observed at this stage of loading, which corresponded to a very low level of protection. The mullion experienced 306 mm of mid-span displacement. Also, the mullion suffered damage at the boundaries close to the concrete beams. Figure 4.43 shows the support rotation of SP-100 after each shot. Table 4.11 compares the level of protection and mullion displacement after each shot. The results of the analysis for SP-200 showed exactly the same support rotations as those for SP-100, which indicated a linear elastic relationship between the mullion translation and spring stiffness. However, higher stresses were observed along the height of the mullion and at its boundaries.

According to the first shot on XRD-100, zero support rotation was observed since mullion did not reach to the concrete beams and a high level of protection was obtained. The mullion just translated horizontally up to 116 mm along the applied load with no failure or damage. By applying the second shot, XRD-100 again did not reach to concrete beam supports, and mullion just experienced 163 mm displacement and high level of protection was attained. The third shot was able to squeeze the springs completely and mullion came in contact with the concrete supports. It happened because rubbers resisted against higher mullion translation due to the compressed volume and secondary stiffness hardening of XRD rubber. The mullion experienced 274 mm deflection and a permanent bending was observed at its mid-height, which developed 5° support rotation and resulted in low level of protection. After the 4th shot, a significant bending was observed at mullion mid-height, however, no failure occurred on it. It developed 10° rotation at support which stood at the edge of very low level of protection. The mullion moved horizontally up to 377 mm along the applied load. Figure 4.44 and Table 4.12 compares XRD-100 model outcomes after each shot. Finally, the same analysis was conducted on the XRD-200. Almost the same support rotation and level of protection was achieved which demonstrated a linear elastic relationship between the mullion translation and spring stiffness.

It can be concluded that both the spring and XRD rubber have proper potential to dampen the applied load and notably improve the level of blast protection. Nevertheless, a few behavior differences were noticed. The elastic behavior of spring was able to push the mullion

back to the initial location after each shot, which is a desirable scenario, whereas the rubber did not act elastically and could not push the mullion back. Therefore, the mullion stayed on rubbers since XRD rubber acted plastically like a memory form. The ideal performance is what spring demonstrated because there would not be a gap between the mullion and existing façade after any blast incidence. Nonetheless, by using rubber, the façade would require adjustment, repair etc. On the other hand, it should be noted that no damage or failure was observed in the models used XRD rubber since rubber provided better stress distribution on the system, especially at the boundaries. But models that used springs experienced element deletion at support on the higher reflected loads. A linear relationship exists between mullion translation and dampers' stiffness; however, by increasing the applied load and stiffness simultaneously, significant higher stress was observed in the body of mullion.

4.6 Summary and Conclusions

Numerical investigation of blast-resistant curtain walls was conducted with emphasis on retrofitting connections. Analytical models for software LS-DYNA were generated and validated against actual test data. An analytical parametric investigation was conducted subsequently to assess the significance of different design parameters. The followings are the conclusions obtained from this numerical investigation.

- The behaviour of curtain walls and single mullions retrofitted with different techniques can be properly predicated by LS-DYNA software. All the connection retrofitting techniques considered can be modeled accurately, and the anchorage can be idealized by using tied-contacts. Furthermore, the blast load can be applied either on the curtain wall exterior surface or through a load application device, both providing similar results.
- The numerical investigation proved that the use of steel sheets to simulate glass panes provides reliable results as representatives of curtain walls with glass panes. The simulations showed insignificant differences in deflections and support reactions between the two sets of analysis. However, less deflection was observed in the model for glass + film due to the high elasticity of the film layer.

- No failure was observed when steel inner U-brackets are used, which numerically indicated the effectiveness of proposed connection retrofit approach. Using steel U-brackets anchored to mullions and transoms can keep the curtain wall components together without any detachment, whereas the transoms with weak U-brackets (as designed for wind loads) show detachment of the members indicating the vulnerability of commonly used connections in real-life practices.
- Externally placed side brackets have insignificant effect on the strength of mullion-to-transoms connections so long as the inner U-brackets maintain their integrity. In this case the side brackets move along with the other components, without attracting much forces and stresses during response. Analysis without the side brackets resulted in similar response as that with the brackets. When both inner steel U-angles and externally placed side brackets are used, the transom support reactions are mostly resisted by the interior U-brackets.
- Steel plates riveted on aluminum sections can provide fully composite performance until after significant inelastic deformations are developed. However, the aluminum mullion that forms the core suffers failure at mid-height.
- Steel L-brackets placed externally on the top and bottom of transoms, anchored to the mullions, provide significant resistance to blast loads when inner U-angles are designed for wind. When such connection hardening technique is used, it is advisable to also harden the mullions and transoms by means of steel plates so that the attachment of external L-brackets do not damage the aluminum mullions in their critical regions.
- The use of steel L-brackets for connecting mullions to concrete substrate provides sufficient connection strength under blast loads. In such applications it is recommended to secure the brackets on the slab edge (rather than top and bottom surfaces of the slab), with brackets bolted on the web of the mullion. The bracket leg attached to the edge of the slab increases the bearing surface area for the mullion, preventing the crushing of aluminum. Moreover, the bracket leg attached to the mullion web provides additional

strengthening to the mullion web, which also contributes towards the prevention of mullion crushing.

- Insufficient thickness and rigidity of L-brackets led to the bending of the bracket legs, softening the rigid connections. The use of thicker steel brackets resulted less element elimination in the analysis on the tension side of the mullion, which indicated further strengthening of this region. However, significant local buckling of the compression side of mullion was observed.
- Using springs and XRD rubber materials at mullion-substrate connections resulted in reduced support rotations and increased level of protection against blast loads. These damper materials prevented failure of the mullion when compared with companion mullions without the dampers. The XRD rubber was not able to restore support displacements and acted like a memory foam. However, they provided better stress distribution in the mullion when compared with the use of springs.

Table 4.1: Properties of curtain wall and mullion sections

Model	First phase: full-size curtain wall						Maximum Reflected Pressure (kPa)
	Mullions (mm)			Transoms (mm)			
	Section	Thickness	Length	Section	Thickness	Length	
CW1	127 × 51	3	2032	76 × 51	3	1930	50
CW2	127 × 51	3	2032	76 × 51	3	1930	42
CW3	127 × 51	3	2032	76 × 51	3	1930	42
							50
							62
Second phase: full-size single mullion							
Model	Mullions (mm)			L-shape bracket (mm)			
	Section	Thickness	Length	Section	Thickness		
M-C1	127 × 51	3	2032	100 × 100	3	22	
M-C2	127 × 51	3	2032	100 × 100	3	28	
M-C3	127 × 51	3	2032	100 × 100	3	72	
M-C4	127 × 51	3	2032	100 × 100	3	71	
Third phase: full-size mullion with Poron XRD rubber							
Model	Mullion (mm)			Poron XRD rubber			
	Section	Thickness	Length	Section (mm)	Density (kg/m ³)		
M-C5	127 × 51	3	2032	216 × 279 × 13	240	71	
M-C6	127 × 51	3	2032	216 × 279 × 13	240	73	
Third phase: full-size mullion with spiral spring							
Model	Mullion (mm)			Spring			
	Section	Thickness	Length	Net length (mm)	Stiffness (N/mm)		
M-C7	127 × 51	3	2032	160	8	24	
M-C8	127 × 51	3	2032	160	30	31	
M-C8	127 × 51	3	2032	200	23	22	

Table 4.2: Details of retrofitting techniques considered for curtain wall models

Model NO.	Two L-shape plates to harden the <u>Mullion</u> (mm)		Two L-shape plates to harden the <u>Transom</u> (mm)		Side Brackets (mm)		Steel U-angle connectors (mm)
	Section: 130×54× 3	Length: 2032	Section: 80×54× 3	Length: 1930	Size: 50 × 50 × 3		Size: 45 × 45 × 3
					Aluminum	Steel	
CW1	✘		✘		✓		✓
CW2	✓		✘		✓		✓
CW3	✓		✓			✓	✓

Table 4.3: Material properties used in the model

MAT_PLASTIC_KINEMATIC						
	Density (kg/m ³)	Yield Stress (MPa)	Young's modulus (MPa)	Tangent Modules (MPa)	Poisson's Ratio	Rapture Strain (%)
Glass	2,600	29	42,000	41,500	0.22	0.0007
Film	1,470	165	4,140	420	0.25	1.3
Reinforcement	7,800	400	200,000	190	0.3	0.14
MAT_PIECEWISE_LINEAR_PLASTICITY						
	Density (kg/m ³)	Yield Stress (MPa)	Young's modulus (MPa)	Poisson's Ratio	Failure flag	
Aluminum	2712	264	69,000	0.33	0.12	
Steel Plate	7830	350	200,000	0.29	0.31	
MAT-CSCM_CONCRETE						
	Density (kg/m ³)	Compressive Strength, (MPa)		Aggregate Size, (mm)		
Concrete	2,400	30		20		
CRUSHABLE_FOAM						
	Density (kg/m ³)	Young's modulus (MPa)		Poisson's Ratio	Rate sensitivity via damping coefficient	
XRD rubber	240	42.5		0.35	0.4	

Table 4.4: Comparisons of experimental and numerical results for curtain walls

		Maximum deflection				In-plane displacement	
		Left Mullion		Middle Transom		Right Mullion	
Model		Exp (mm)	Numerical (mm)	Exp (mm)	Numerical (mm)	Exp (mm)	Numerical (mm)
First CW		351	326	350	343	-	5
Second CW		16	10	Missed	136 *	15	11
Third CW	1 st shot	167	145	155	212	-	2
	2 nd shot	133	152	140	158	-	5
	3 rd shot	132	168	139	178	-	7

* Considering failure with complete attachment to the steel sheet

Table 4.5: Comparisons of experimental and numerical results for single mullions

Model	Maximum Mid-height deflection		Residual deflection at mid-height	
	Experimental (mm)	Numerical (mm)	Exp (mm)	Numerical (mm)
<i>M-C1</i>	85	72	45	58
<i>M-C2</i>	37	84*	26	73
<i>M-C3</i>	33	38	25	31
<i>M-C4</i>	13	5	12	3
<i>M-C5</i>	21	9	19	5
<i>M-C6</i>	36	23	31	20
<i>M-C7</i>	134	127	0	0
<i>M-C8</i>	172	153	107	79
<i>M-C9</i>	192	178	0	0

Table 4.6: Comparison of mullion support reactions; UFC-chart versus LS-DYNA

Model	UFC chart (kN)		Numerical (kN)
	<i>Elastic Range</i>	<i>Plastic Range</i>	
CW1	139.45	141.66	137.10
CW2	106.20	110.03	97.24
CW3	148.82	150.54	141.77
M-C1	66.54	68.13	57.84
M-C2	82.53	86.72	75.66
M-C3	128.30	132.54	114.10
M-C4	124.90	128.90	118.25
M-C5	126.55	137.20	117.82
M-C6	130.12	134.38	118.94
M-C7	45.97	54.00	31.72
M-C8	58.33	67.25	46.46
M-C9	45.86	51.35	33.50

Table 4.7: Comparison of middle transom support reaction; CW-Blast versus LS- DYNA

	CW-Blast (kN)		Numerical (kN)
	<i>Elastic Range</i>	<i>Plastic Range</i>	
CW1	17.49	19.08	23.50
CW2	14.41	15.47	20.46
CW3	47.13	-	33.18

Table 4.8: The deflection comparison between using steel sheet versus glass + film in the first curtain wall

	Max deflection	Residual deflection	Max deflection	Residual deflection	Residual in-plane deflection	Support reaction (kN)	
	Left mullion (mm)		Middle transom (mm)		Right mullion (mm)	Mullion	Middle transom
Experiment test	351	350	350	350	8	-	-
CW1 model	326	267	343	295	5	137.10	31.25
CW1+G&F model	320	264	306	255	7	127.50	29.72

Table 4.9: The comparison of using brackets on the narrow side, or on top and bottom of transoms in the third curtain wall

	Maximum deflection (mm)		In-plane displacement (mm)
Test	Left Mullion	Middle Transom	Right Mullion
CW3	151	213	2
CW3+T&B	188	374	8

Table 4.10: Comparison of third curtain wall outcomes with and without side brackets

	Maximum deflection (mm)						Residual displacement (mm)			Support reaction	
Test	Left Mullion			Middle Transom			Left Mullion			Mullion (kN)	Middle Transom (kN)
	1 st shot	2 nd shot	3 rd shot	1 st shot	2 nd shot	3 rd shot	1 st shot	2 nd shot	3 rd shot		
Experiment	167	133	132	155	140	139	125	130	114	-	-
CW3	145	152	168	212	158	178	131	142	153	141.77	33.16
CW3_NB	147	156	172	214	166	185	132	146	164	143.58	34.90

Table 4.11: Response limit of mullion by inserting springs between components

	Shot Number	Mullion Displacement (mm)	Support Rotation (θ)	Protection Level	Reflected Pressure (kPa)
SP-100	1 st	135	0°	High	30
	2 nd	173	0°	High	40
	3 rd	232	3°	Medium	50
	4 th	306	7°	Very Low	60

Table 4.12: Response of mullion after each shot by inserting rubber between components

	Shot Number	Mullion Displacement (mm)	Support Rotation (θ)	Protection Level	Reflected Pressure (kPa)
XRD-100	1 st	116	0°	High	30
	2 nd	163	0°	High	40
	3 rd	274	5°	Low	50
	4 th	377	10°	Very Low	60

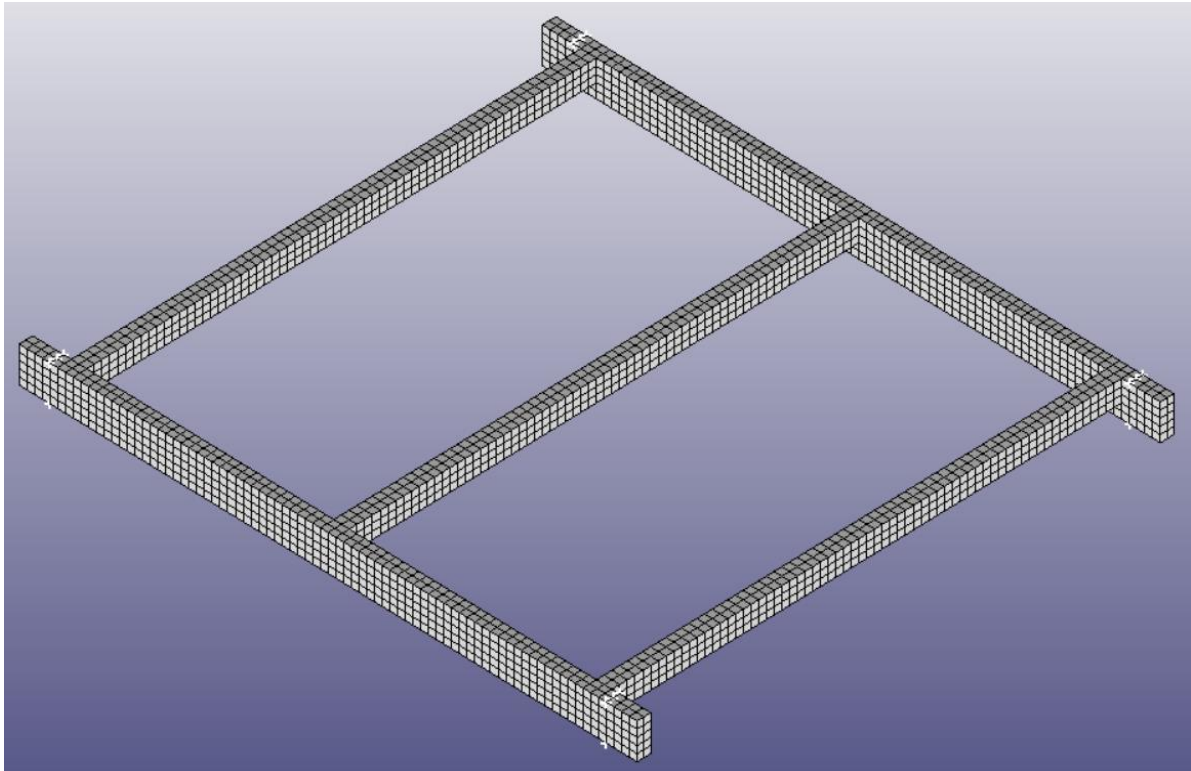


Figure 4.1: Curtain wall elements; and simple support locations (marked in white xxx)

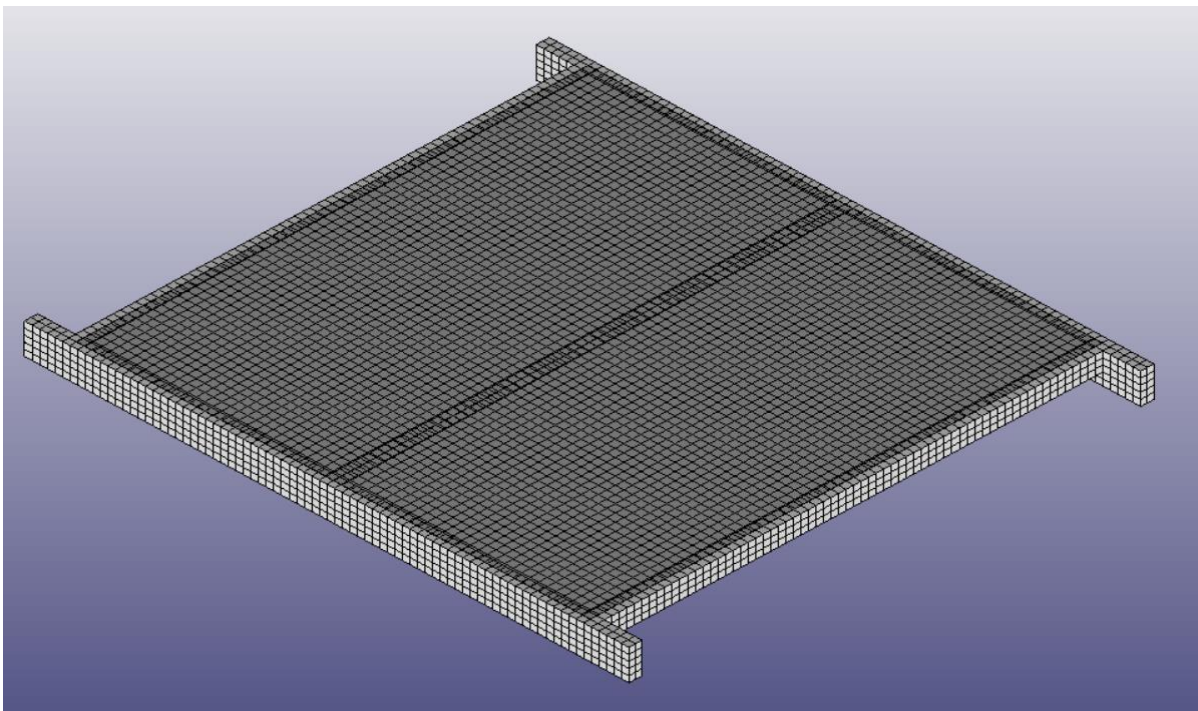
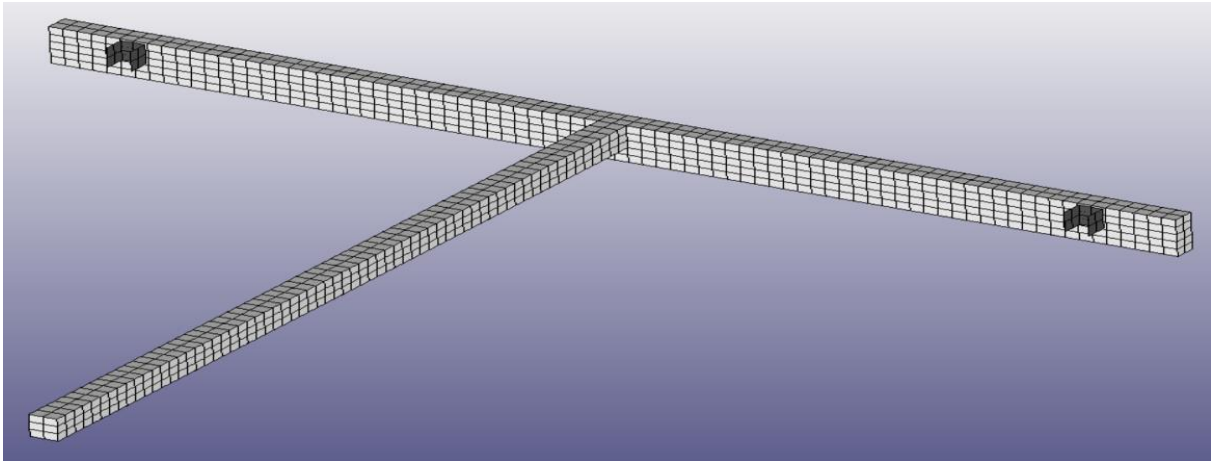
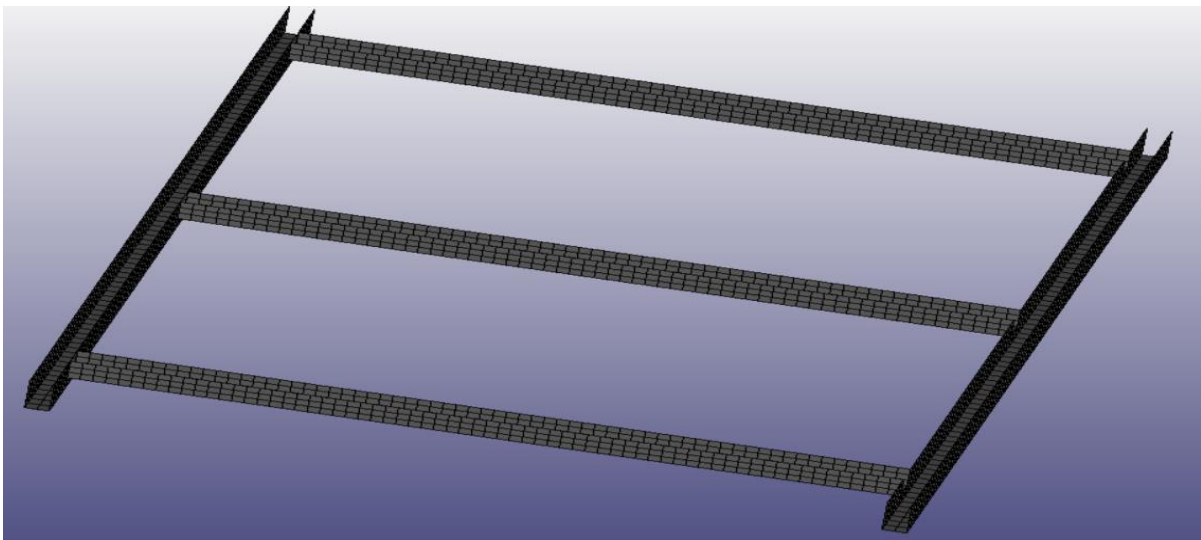


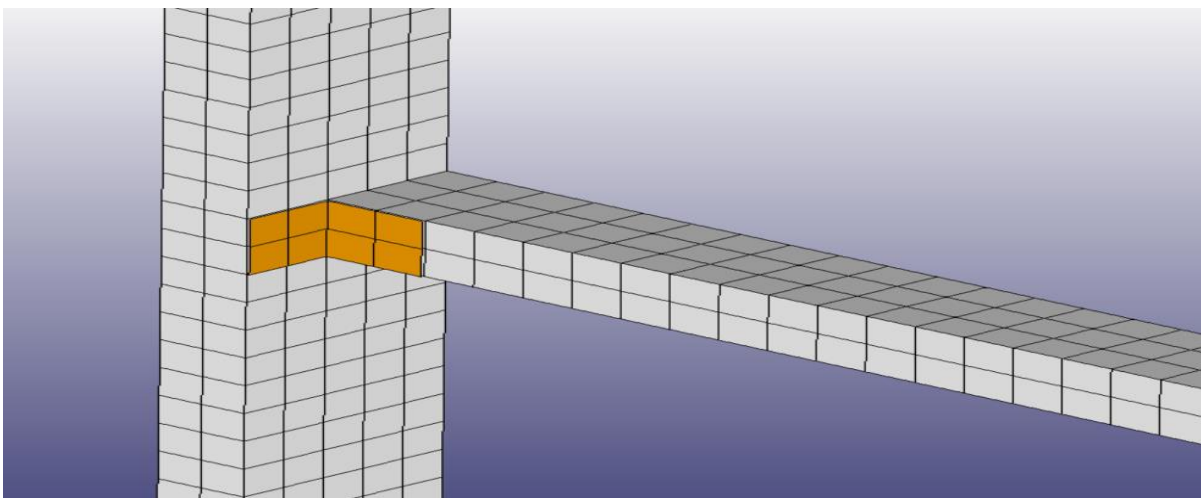
Figure 4.2: Steel sheet gauge #22 used to cover curtain wall



a) Steel U-angles for connecting the mullions and transoms (assembling curtain wall)



b) Two L-shape steel plates used to harden the intended sections



c) Side brackets to strengthen the connection between mullion and transom

Figure 4.3: Retrofit techniques used for full-size curtain walls

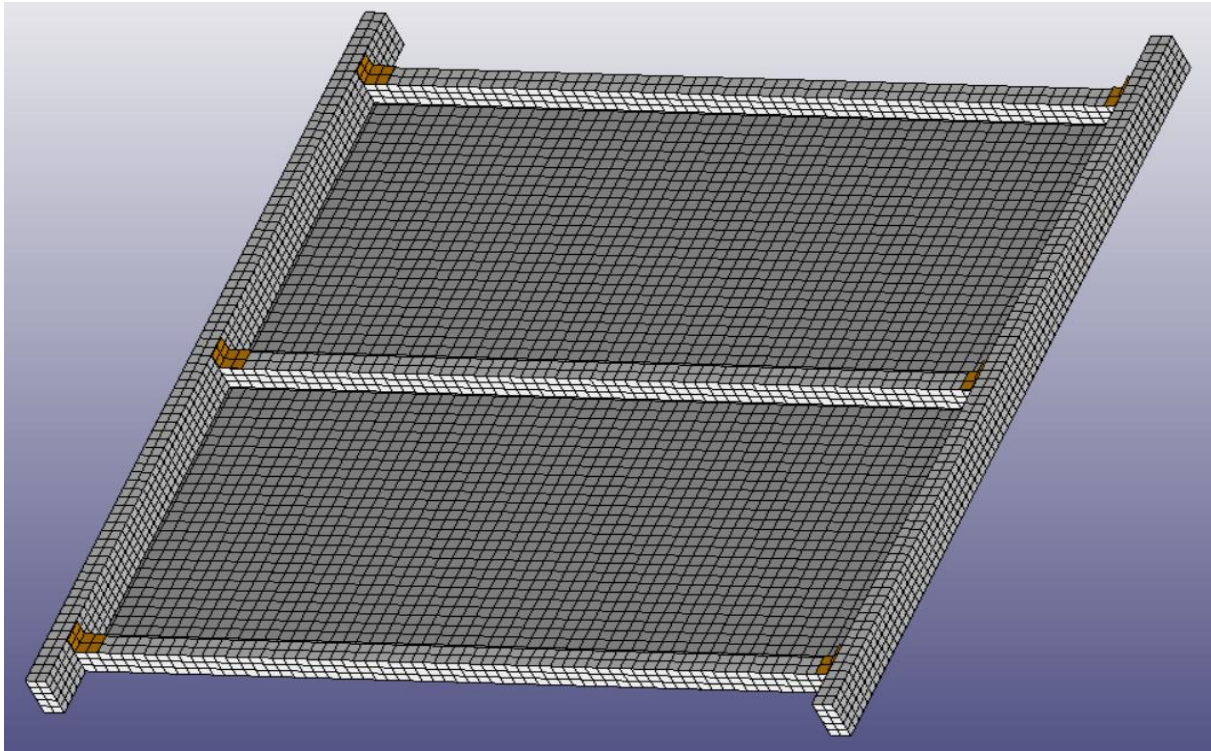


Figure 4.4: CW1 with retrofitted internal U-angles, and glued aluminum side brackets

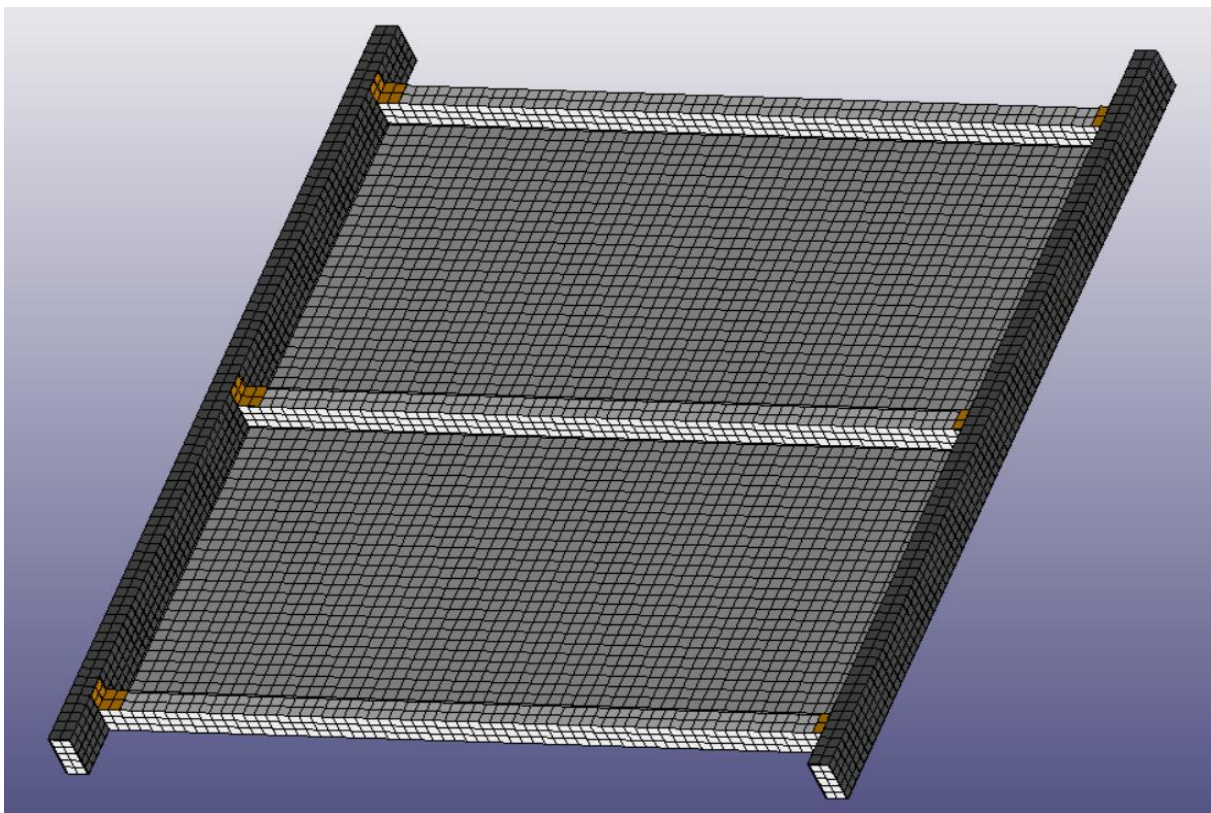


Figure 4.5: CW2 with hardened vertical mullions; retrofitted internal U-angles, and anchored aluminum side brackets

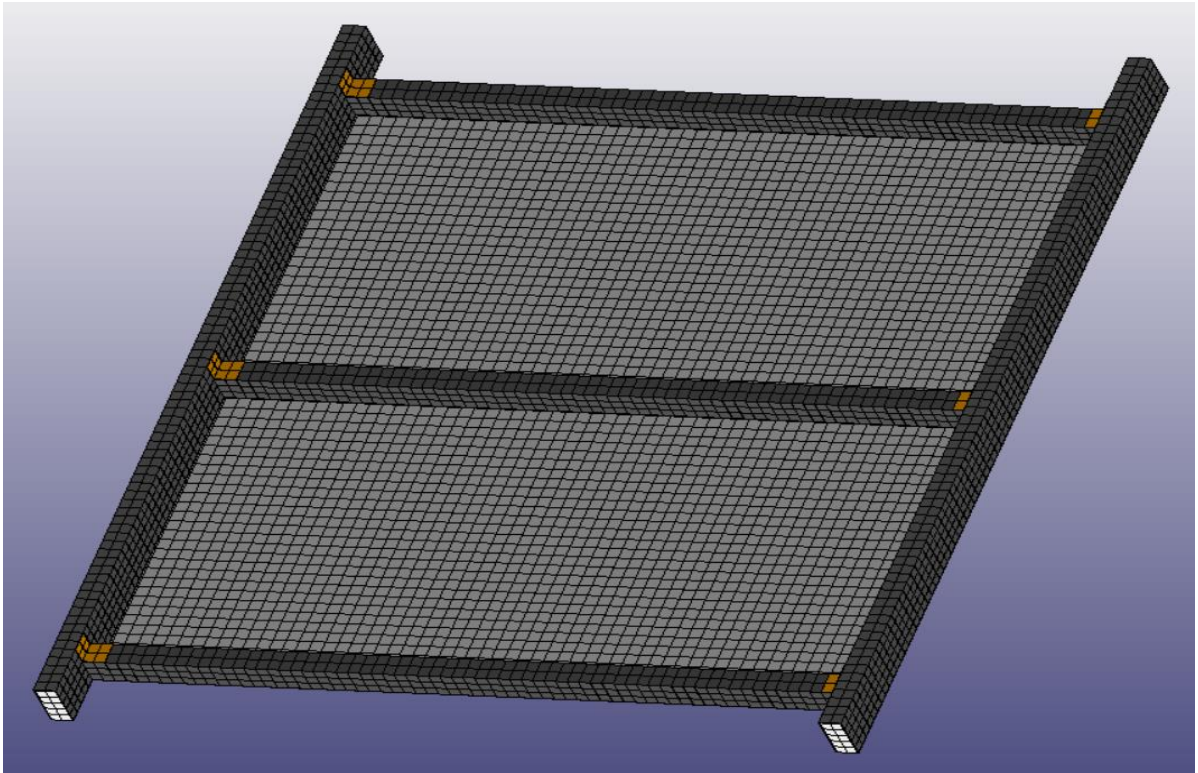
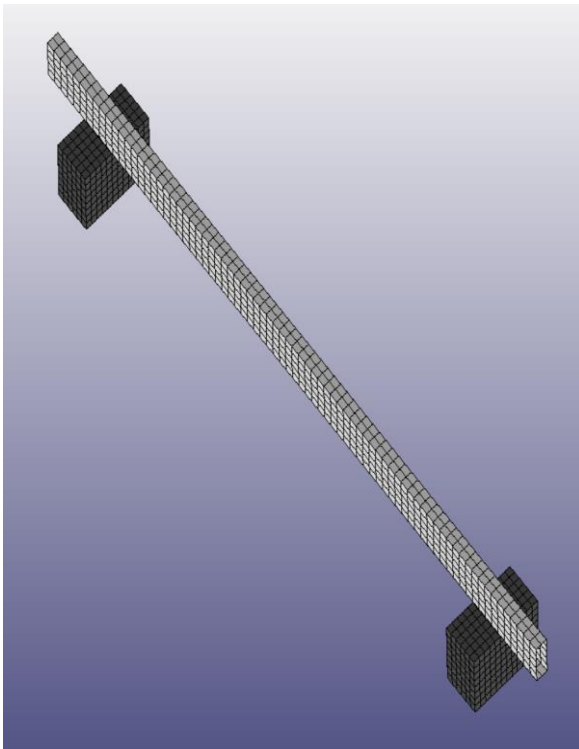
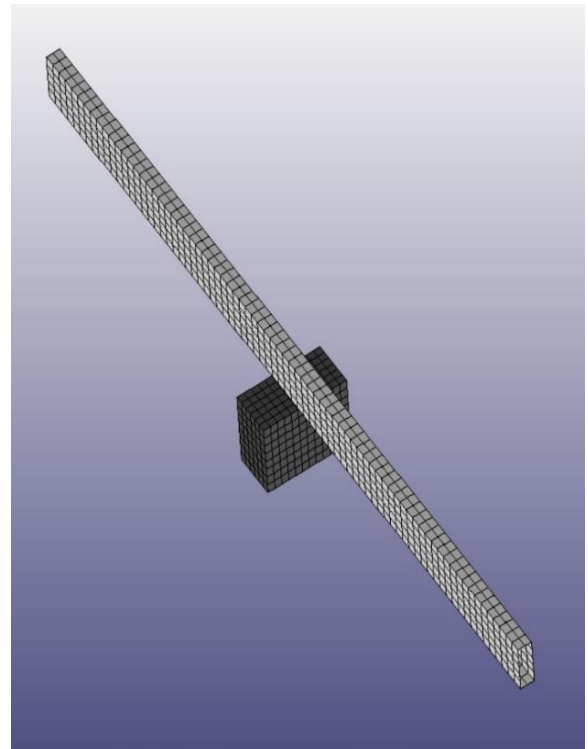


Figure 4.6: CW3 with hardened mullions and transoms; retrofitted internal U-angles, and anchored steel side brackets

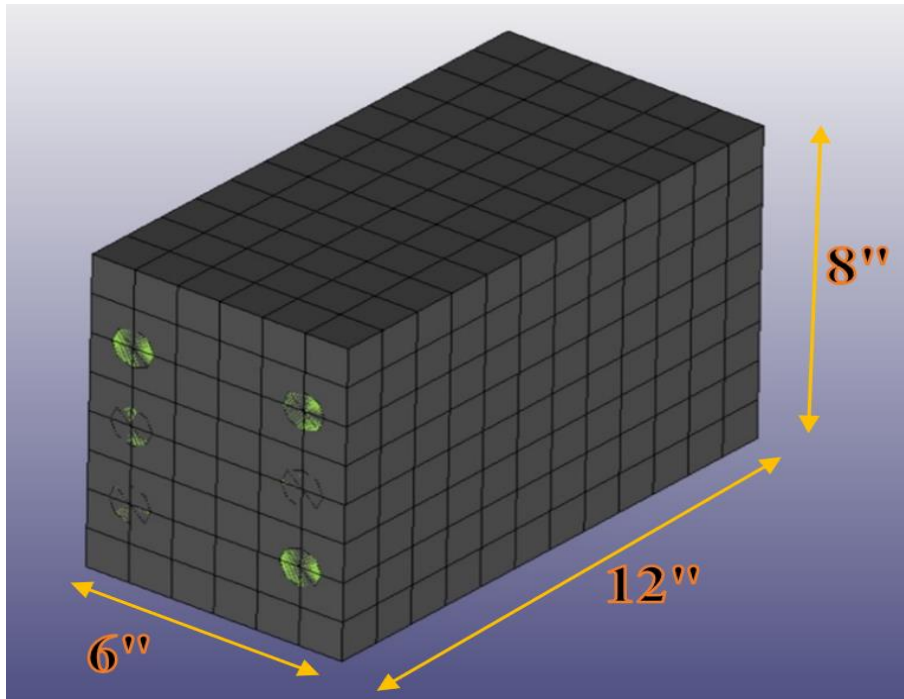


a) Single mullion configuration

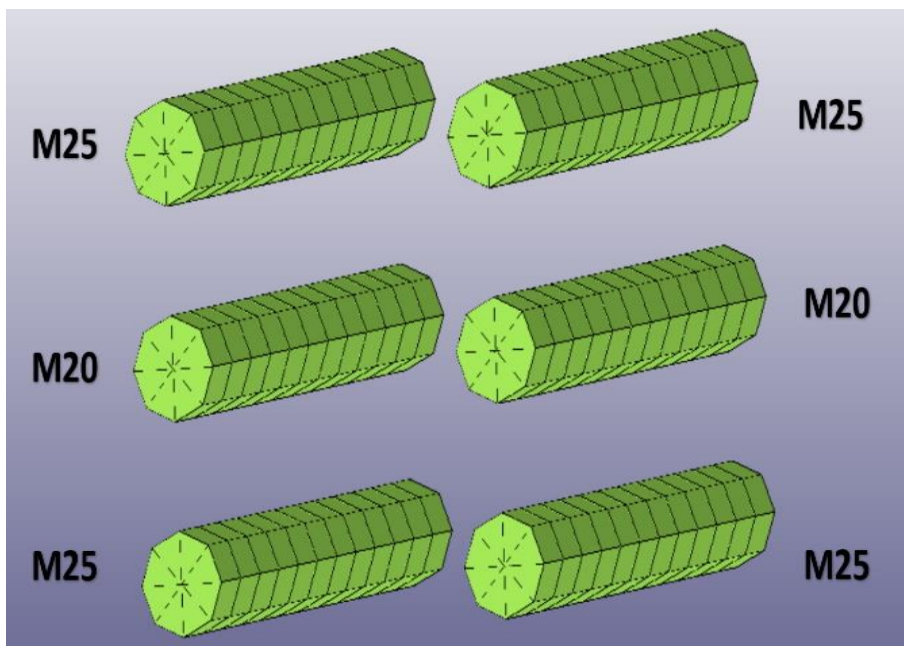


b) Continuous mullion configuration

Figure 4.7: Single mullion test configurations and the location of concrete beams

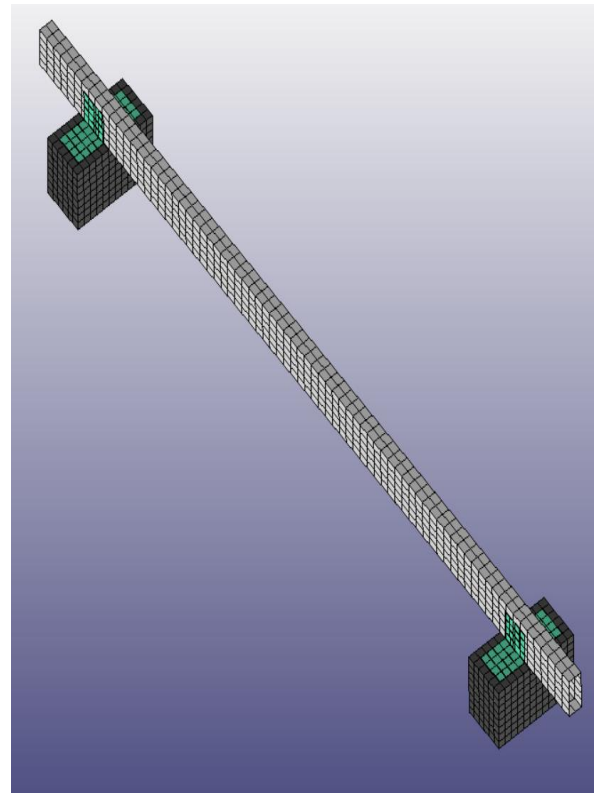
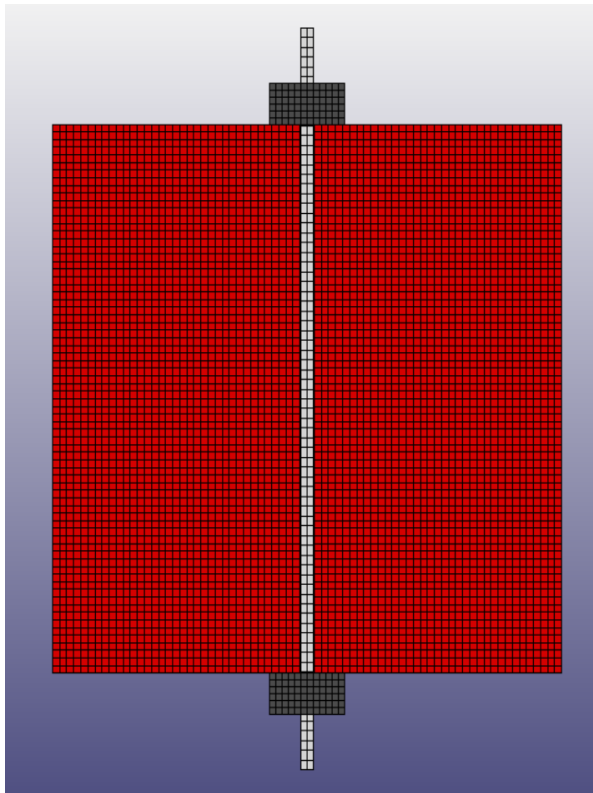


a) Concrete beam section size

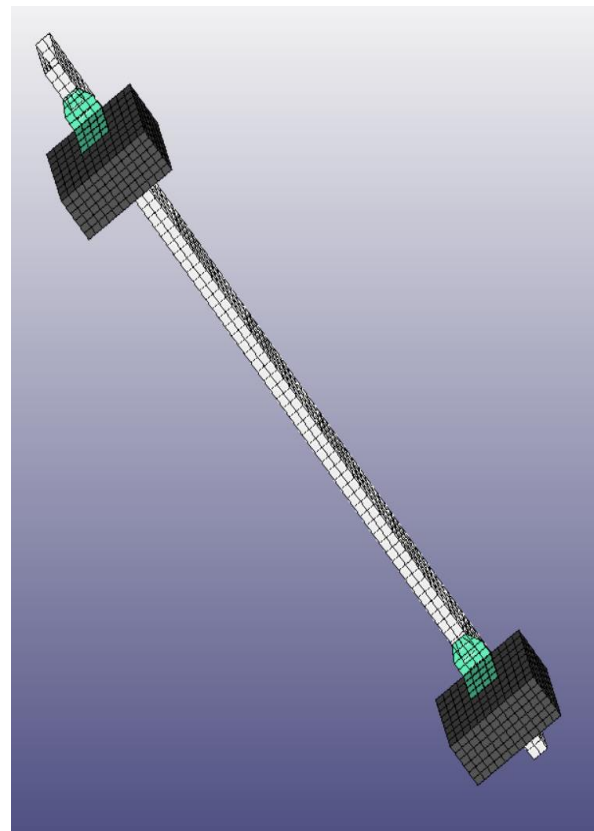
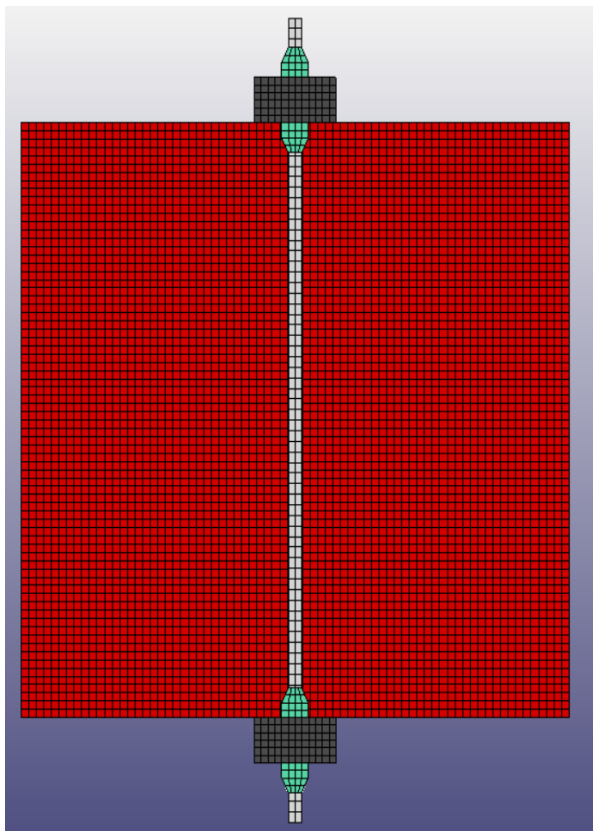


b) Reinforcement

Figure 4.8: The details of reinforced concrete beams

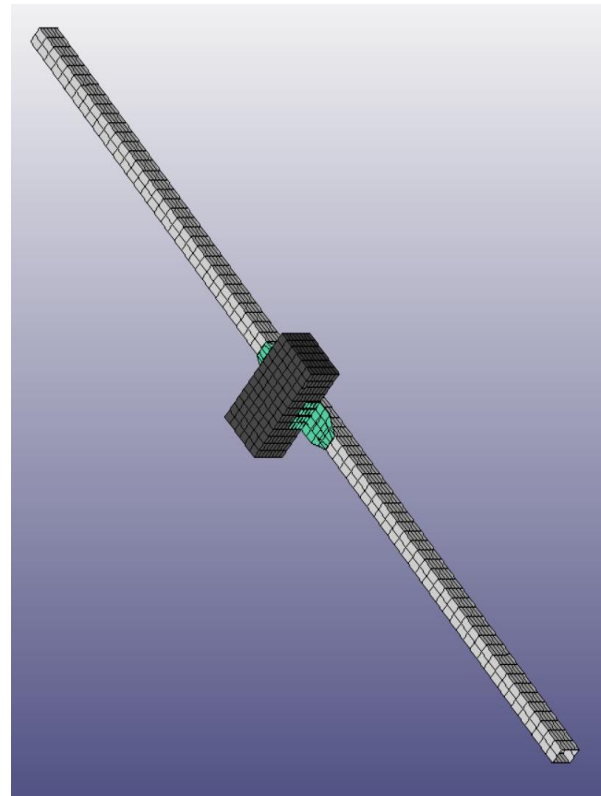
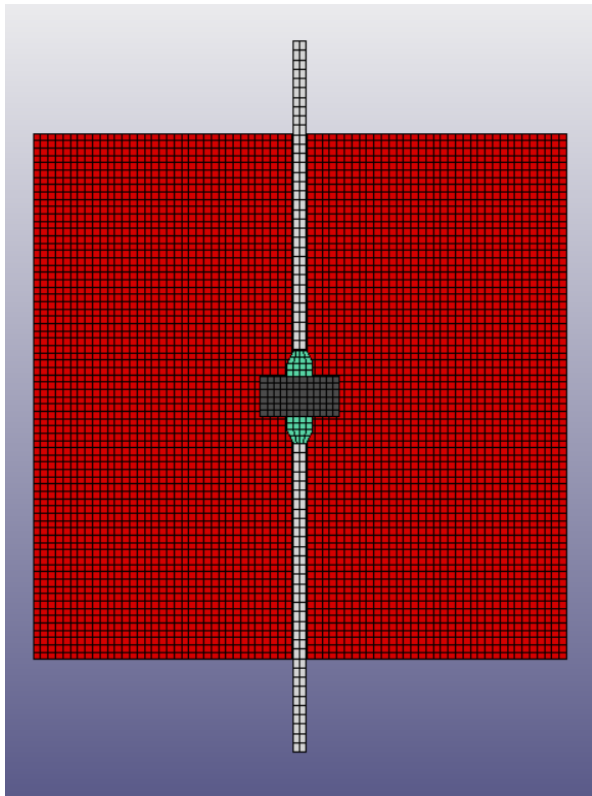


a) M-C1 model with side brackets

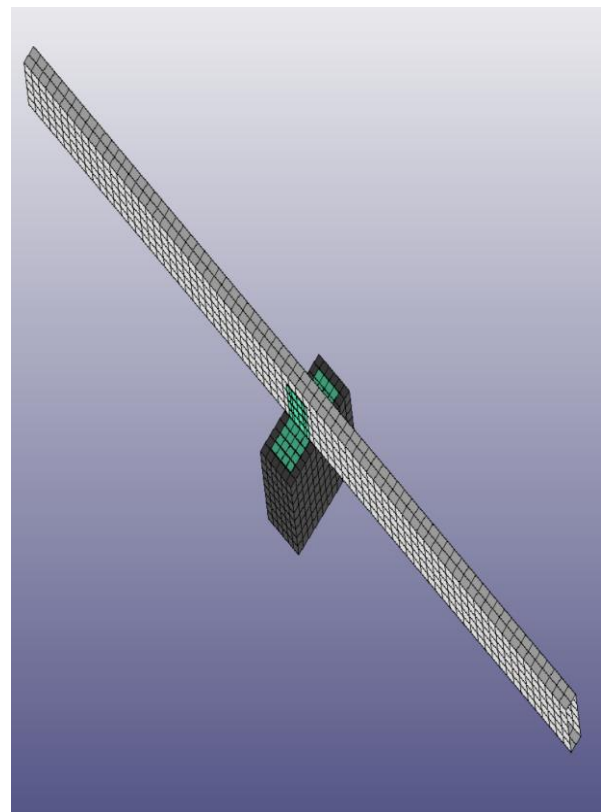
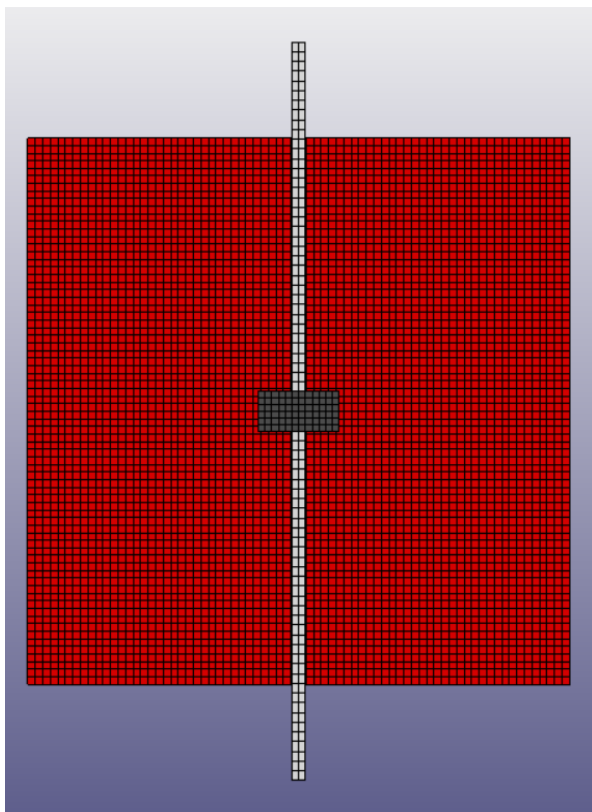


b) M-C2 model with top and bottom brackets

Figure 4.9: The 1st and 2nd mullions (single-mullion configuration)

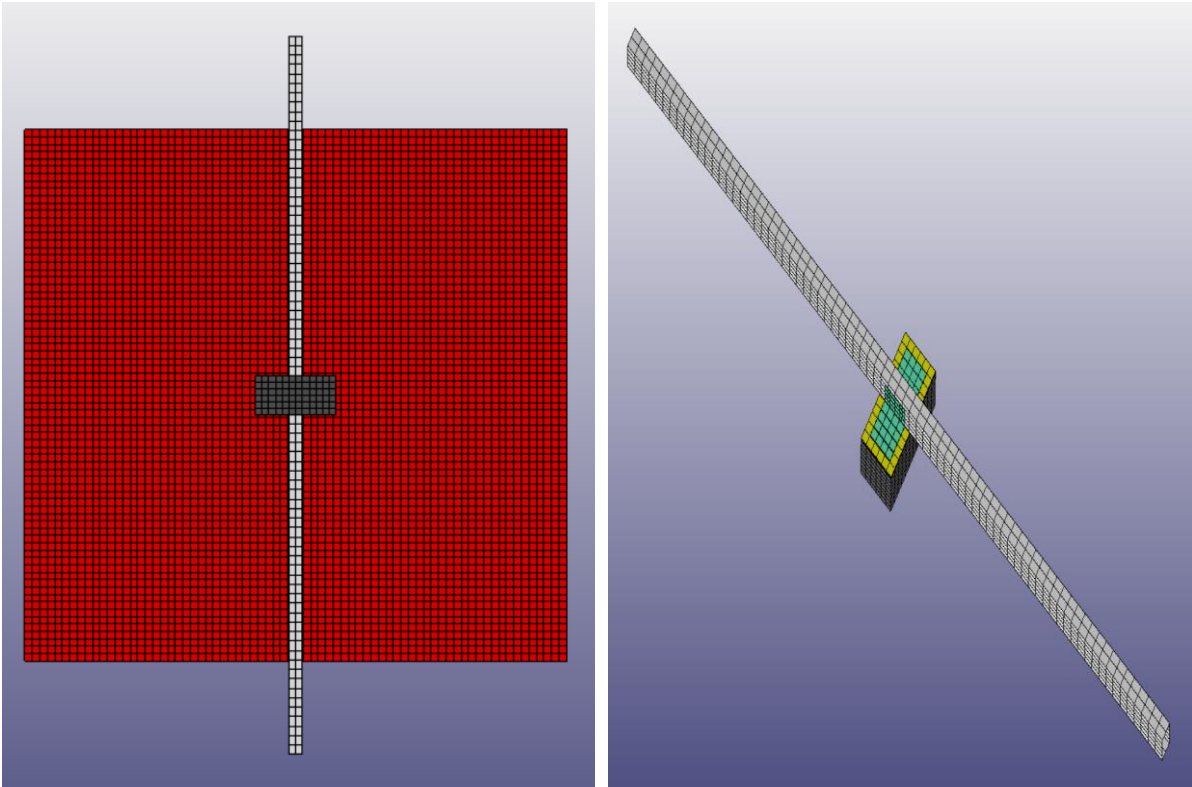


a) M-C3 model with top and bottom brackets

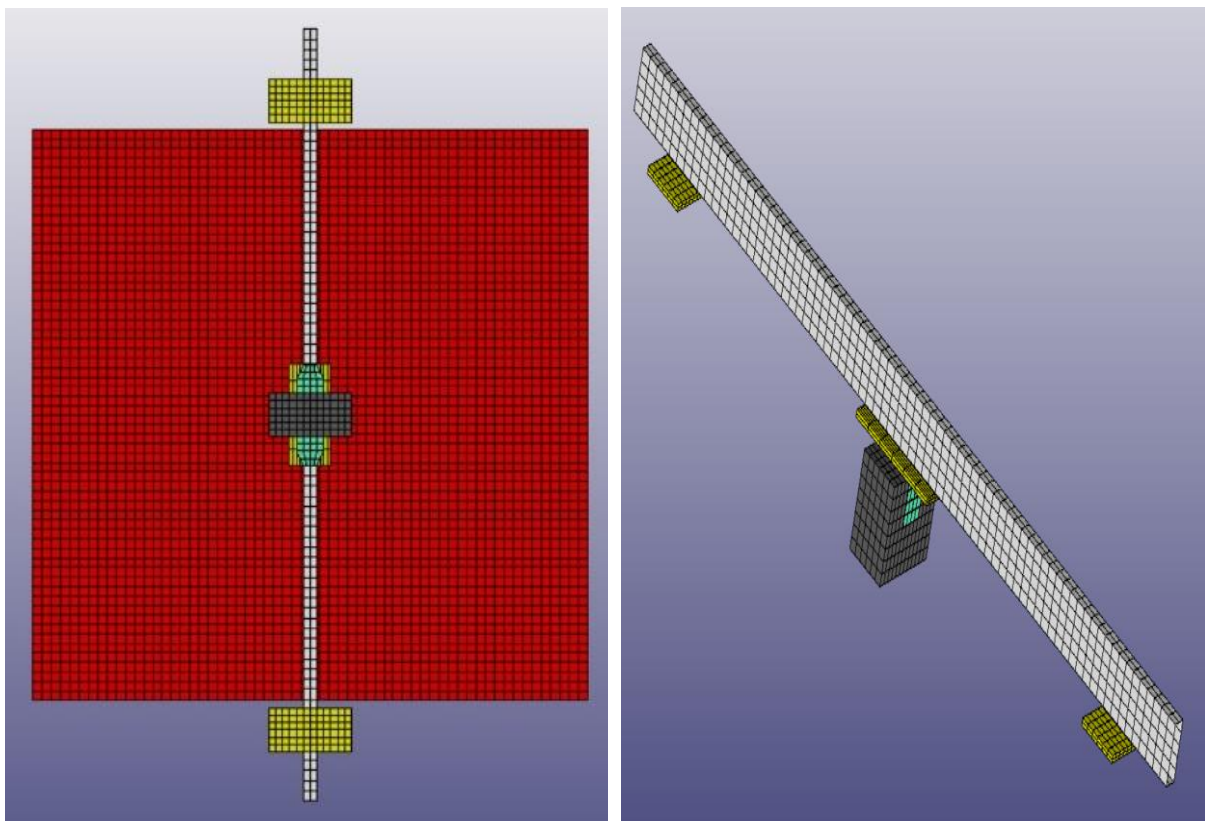


b) M-C4 model with side brackets

Figure 4.10: The 3rd and 4th mullions (continuous-mullion configuration)

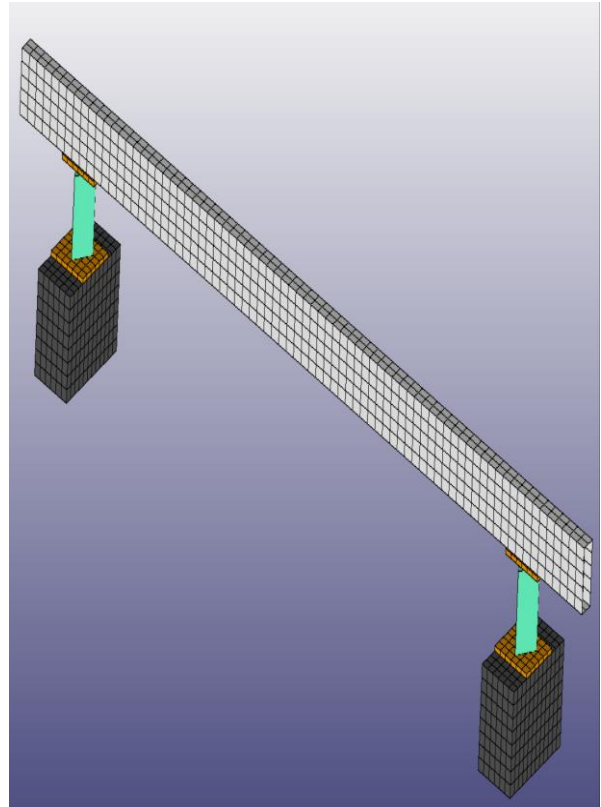
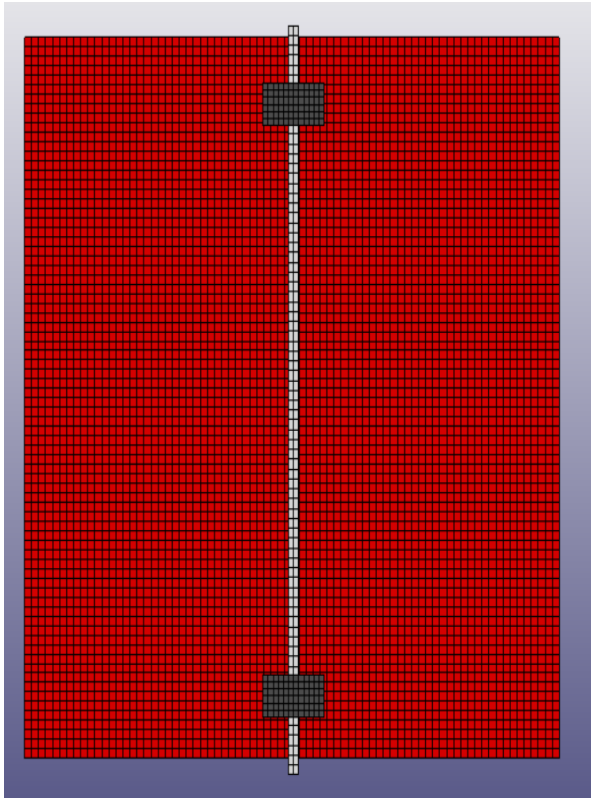


a) M-C5 model with side brackets

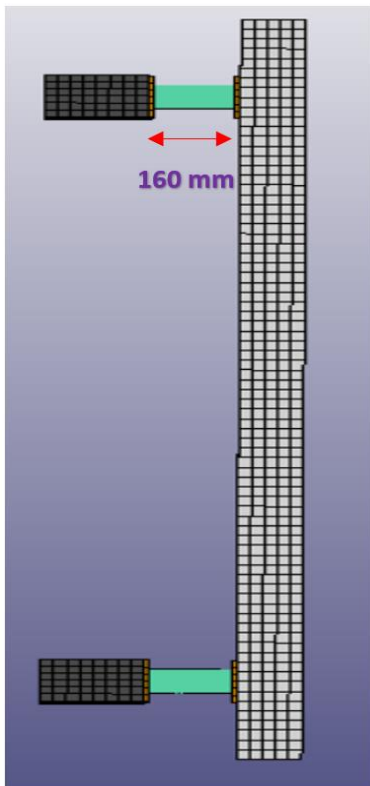


b) M-C6 model with top and bottom brackets

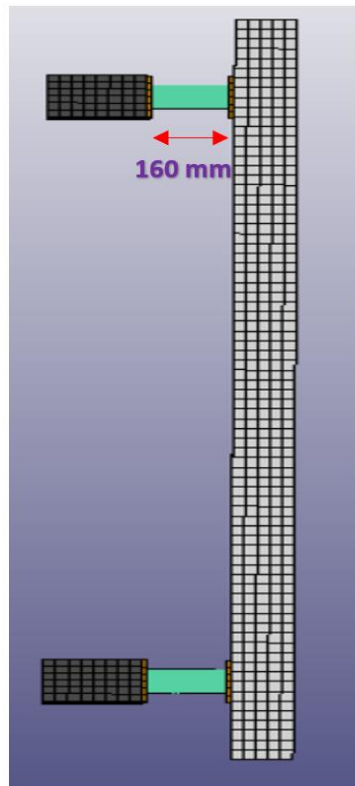
Figure 4.11: The 5th and 6th mullions with XRD rubber



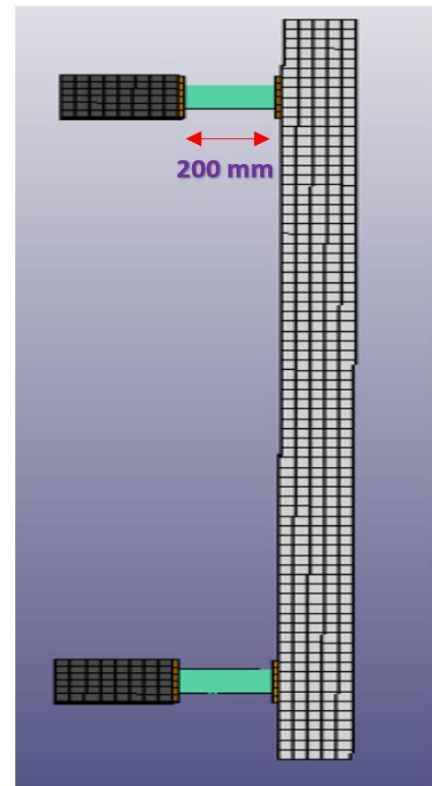
a) The overall view of models with inserting spiral springs



b) M-C7; Stiffness 8 N/mm



c) M-C8; Stiffness 30 N/mm



d) M-C9; Stiffness 23 N/mm

Figure 4.12: The 7th, 8th, and 9th mullion models with inserted spring

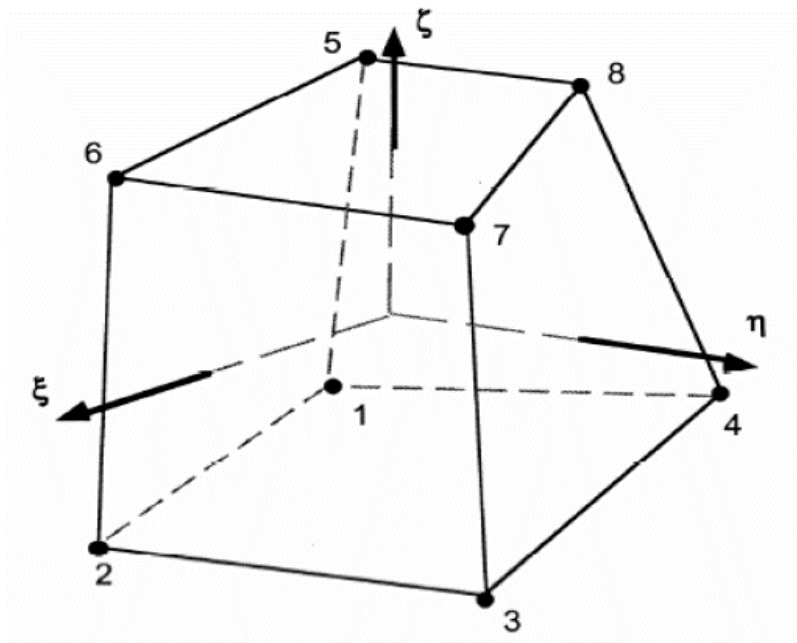


Figure 4.13: Eight-Node Solid Element (LSTC 2006)

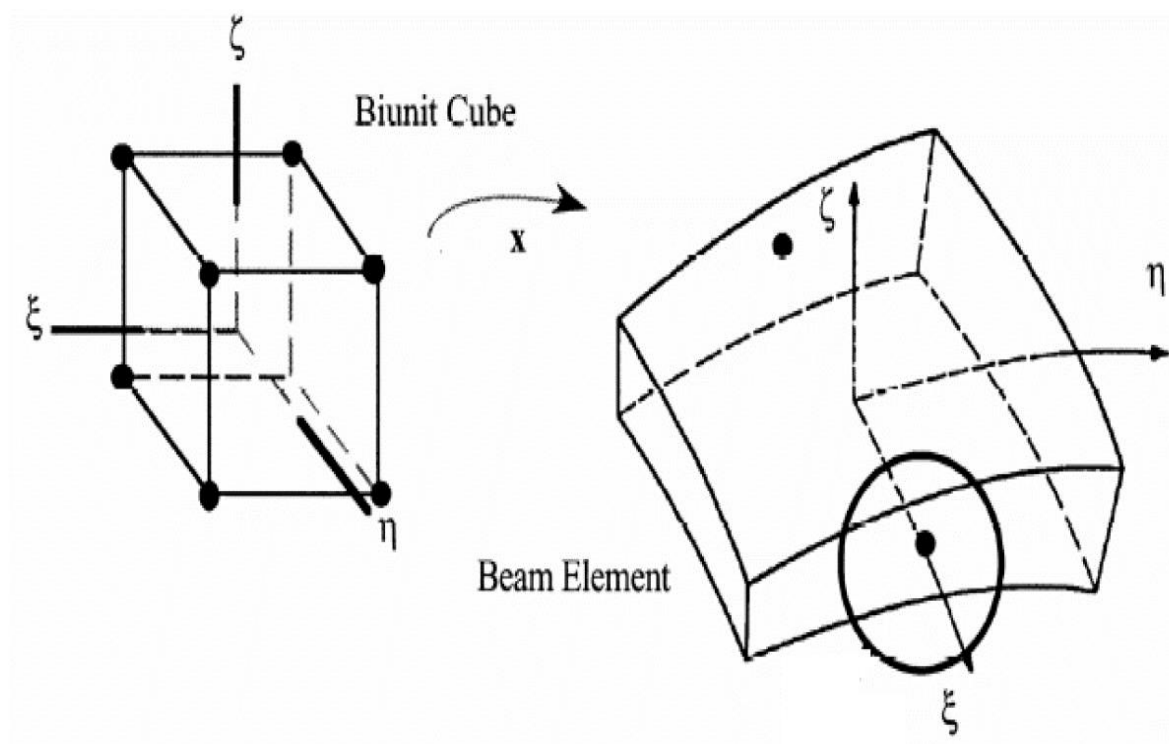


Figure 4.14: Hughes-Liu Beam Element (LSTC 2006)

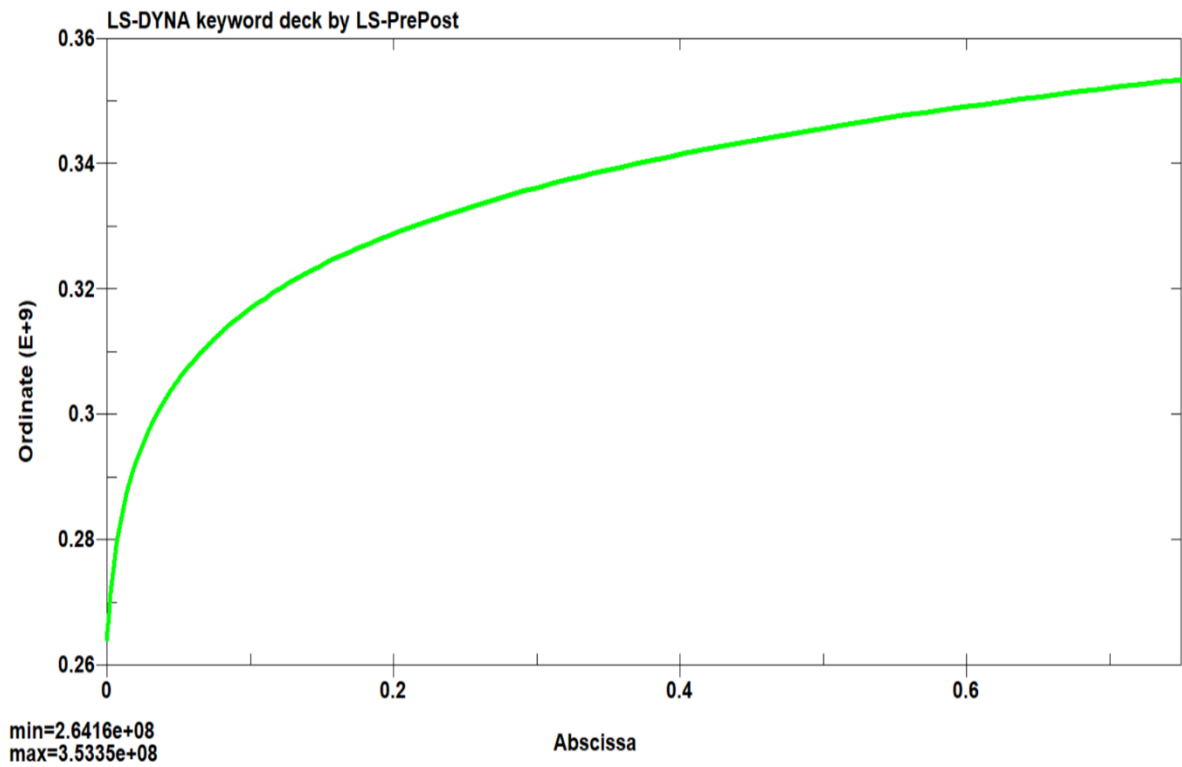


Figure 4.15: The effective stress versus effective plastic strain curve for Aluminum T-6061

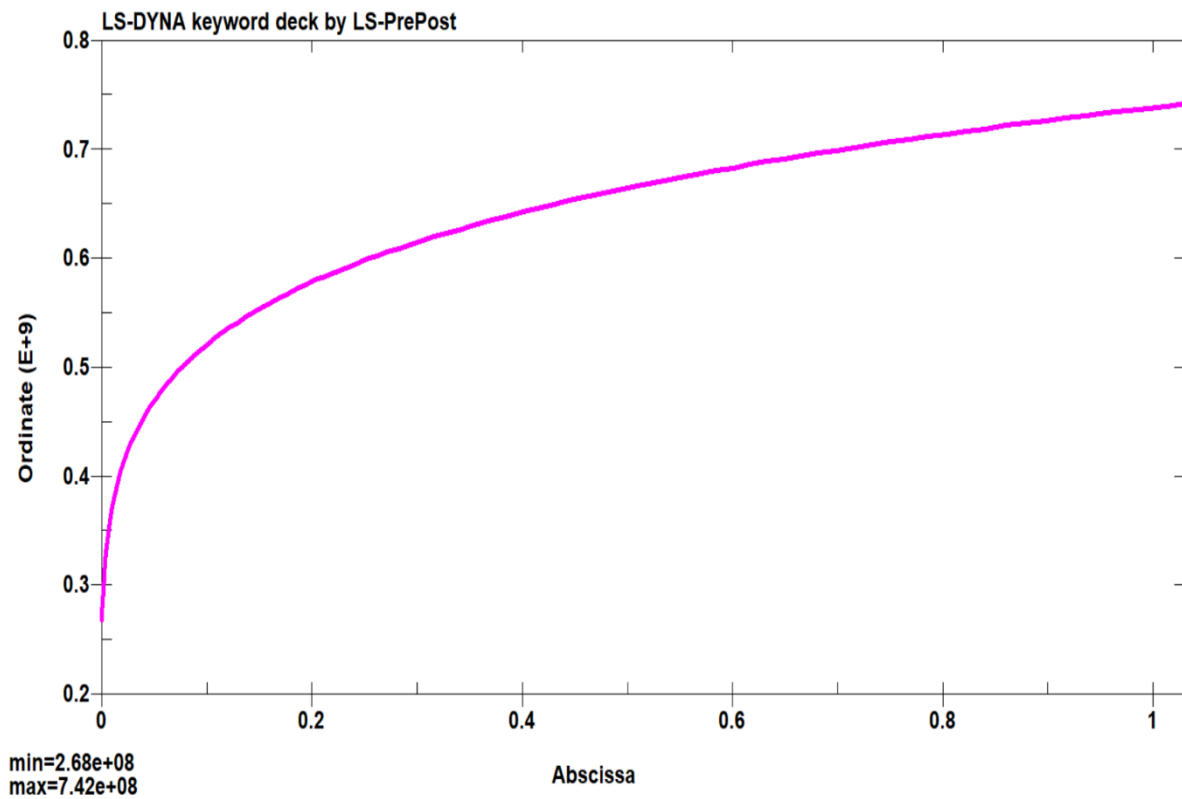


Figure 4.16: The effective stress versus effective plastic strain curve for Steel AISI 1020

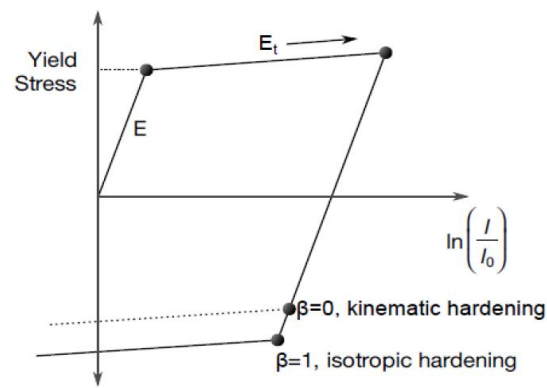


Figure 4.17: MAT003-PLASTIC_KINEMATIC Material Model (LSTC 2015)

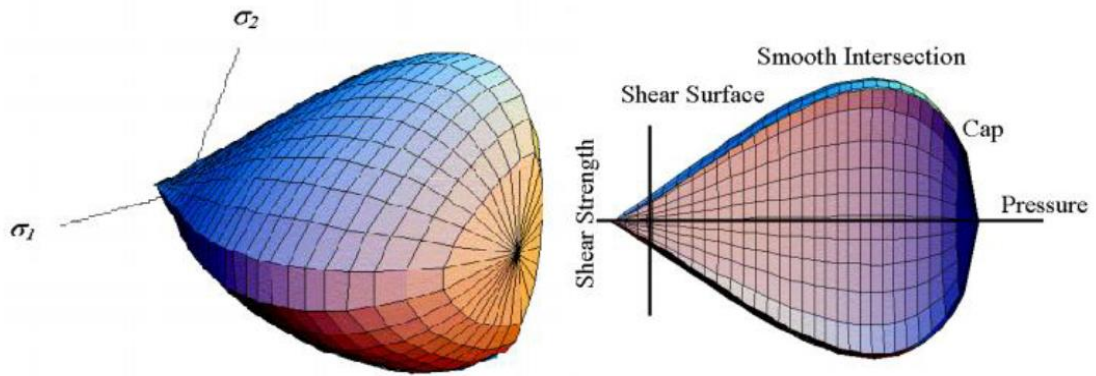


Figure 4.18: 3-D and 2-D view of the model yield surface for concrete (Murray 2007)

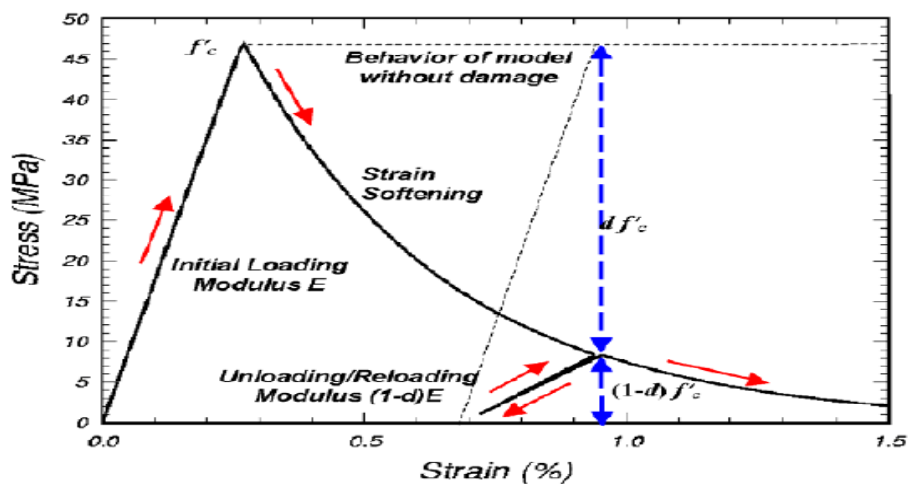


Figure 4.19: Stain softening and modulus reduction in concrete (Murray 2007)

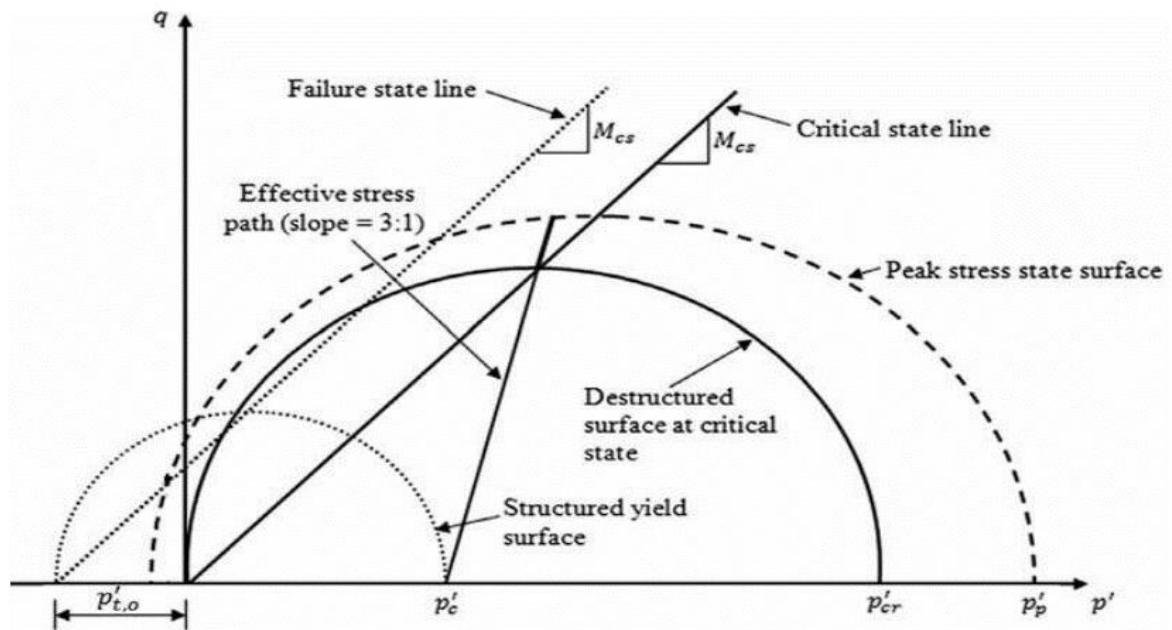


Figure 4.20: Mises circle in the deviatoric stress plane (LS-DYNA manual 2011)

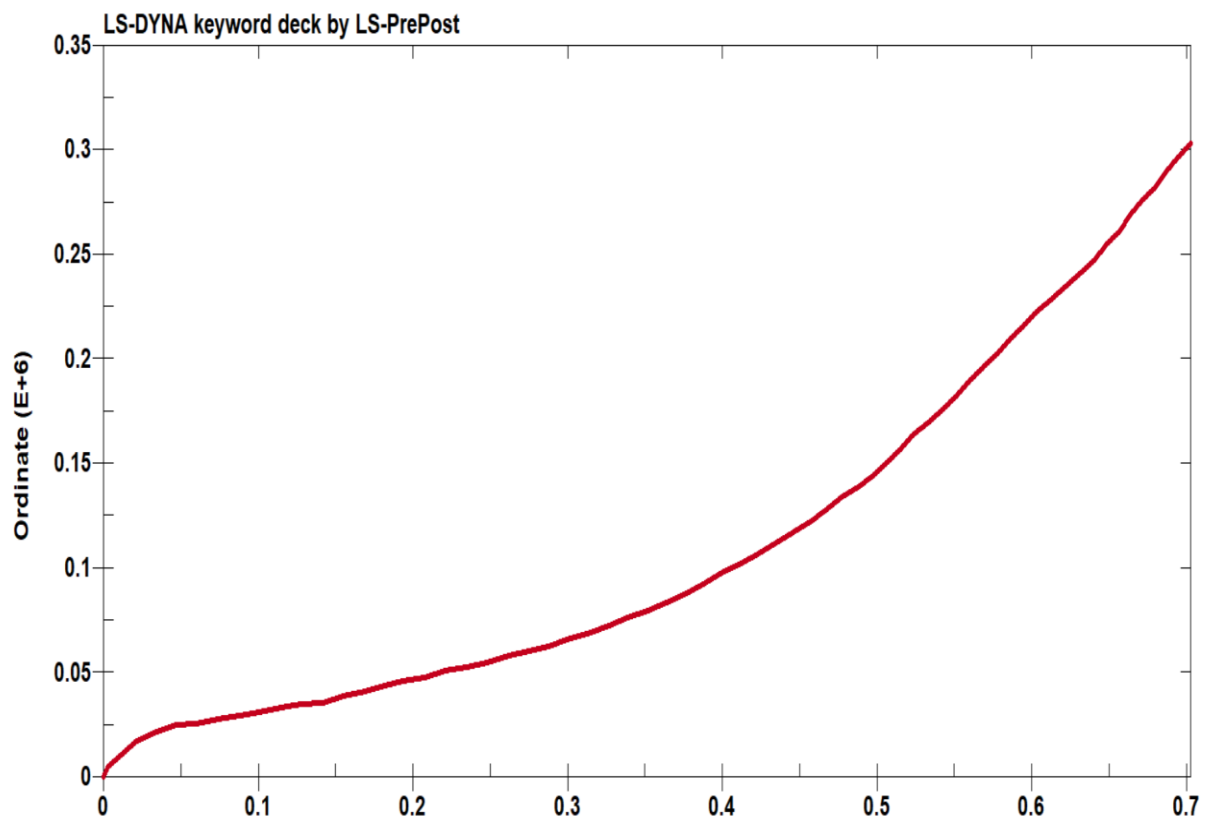
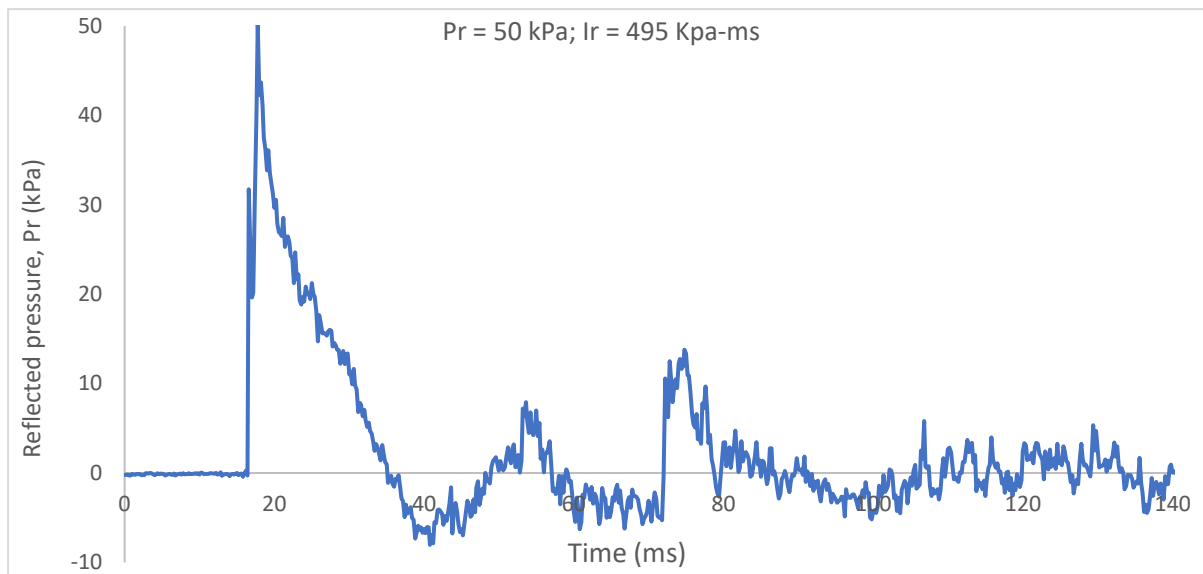
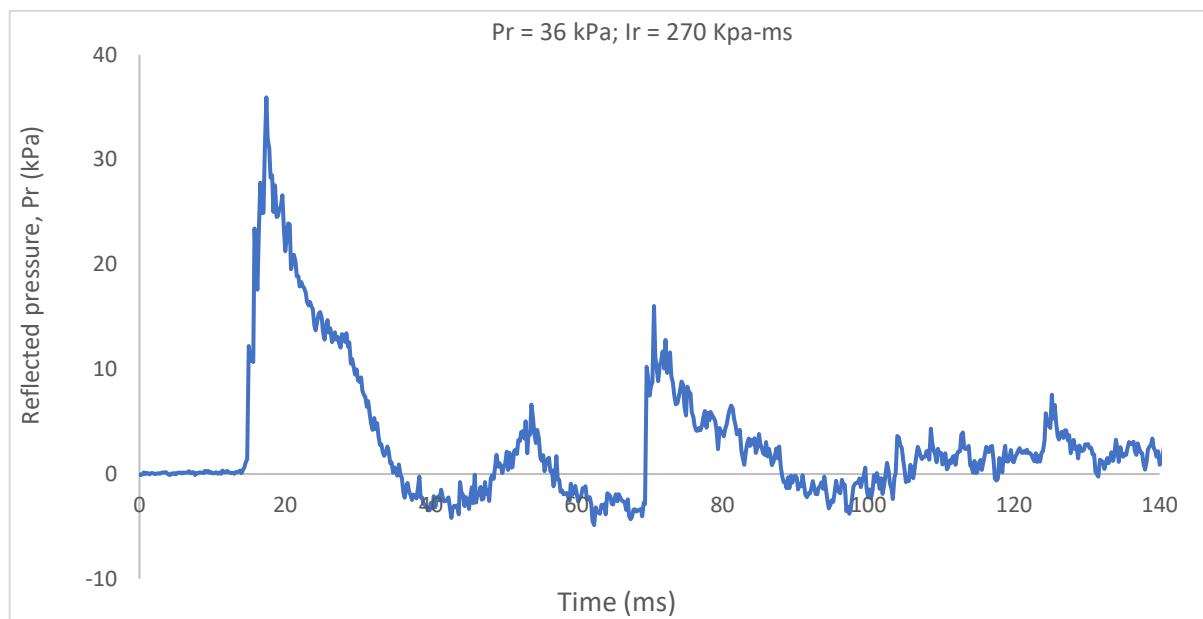


Figure 4.21: Yield stress versus volumetric strain curve defined for crushable foam

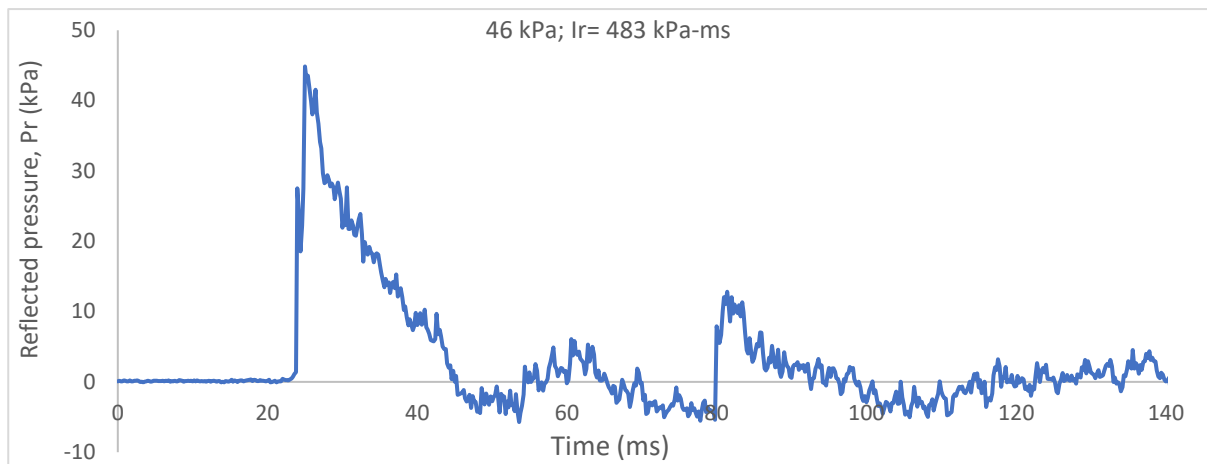


a) Pressure-time history used for first curtain wall model (CW1)

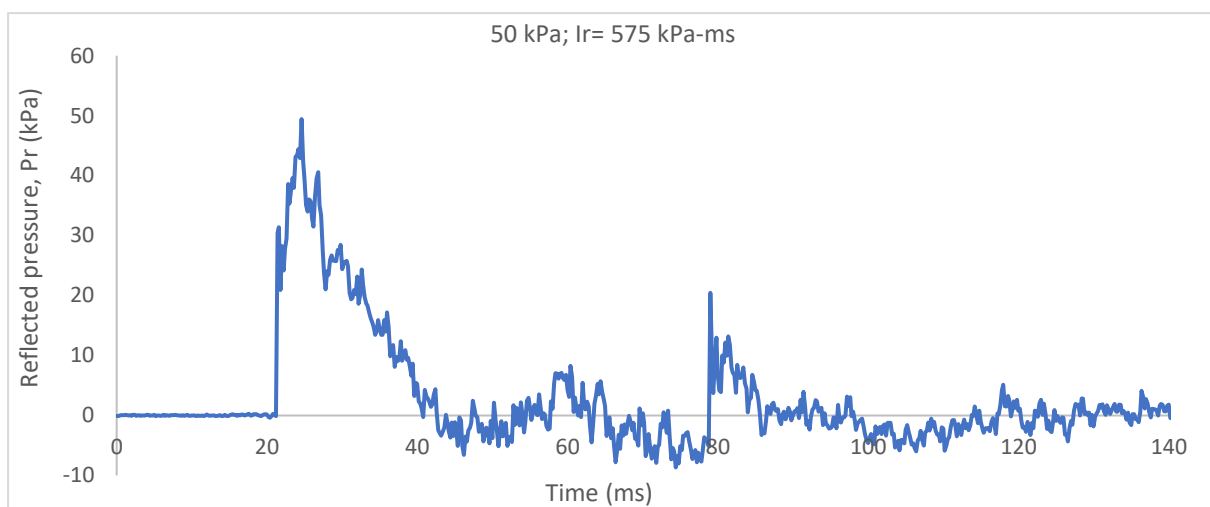


b) Pressure-time history used for second curtain wall model (CW2)

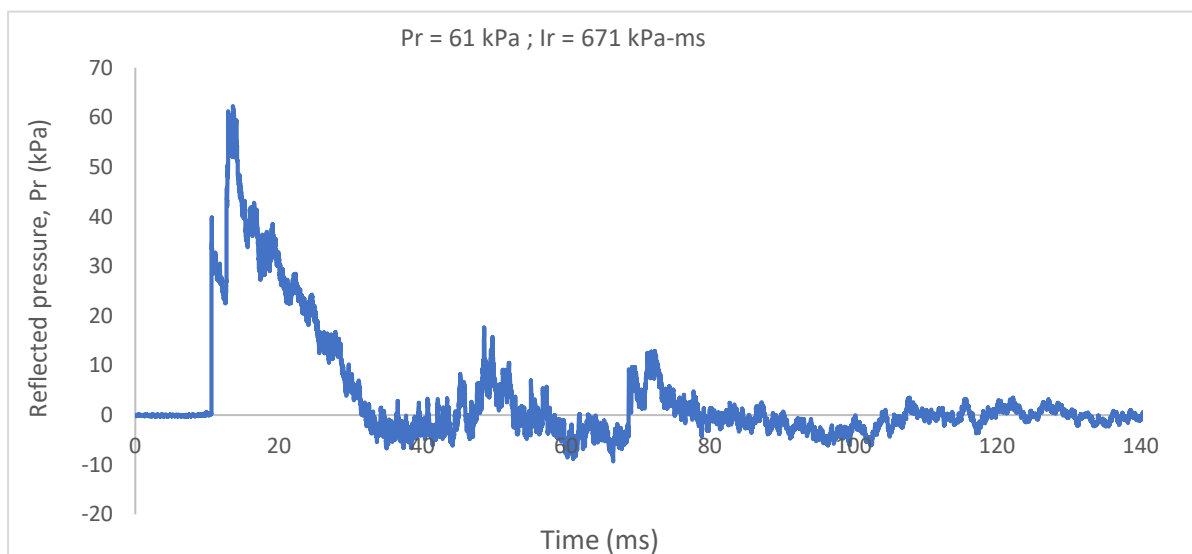
Figure 4.22 (Cont'd)



c) Pressure-time history used for third curtain wall; 1st shot (CW3)

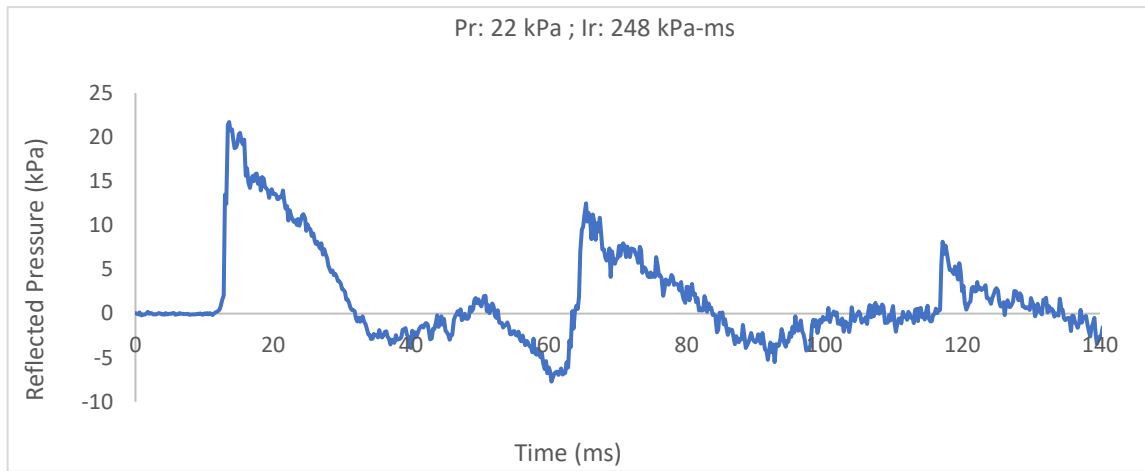


d) Pressure-time history used for third curtain wall; 2nd shot (CW3)

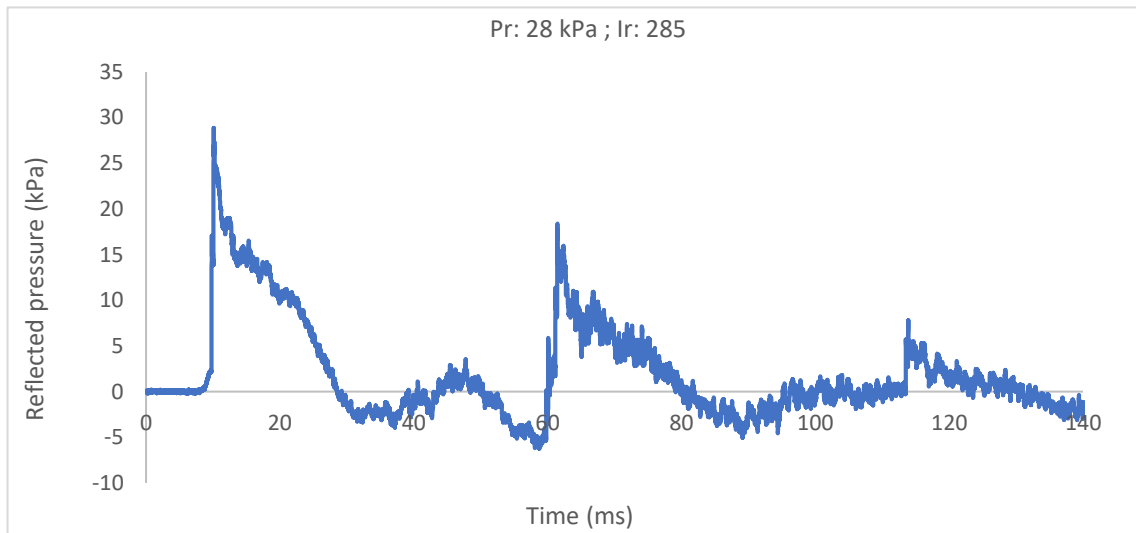


e) Pressure-time history used for third curtain wall; 3rd shot shot (CW3)

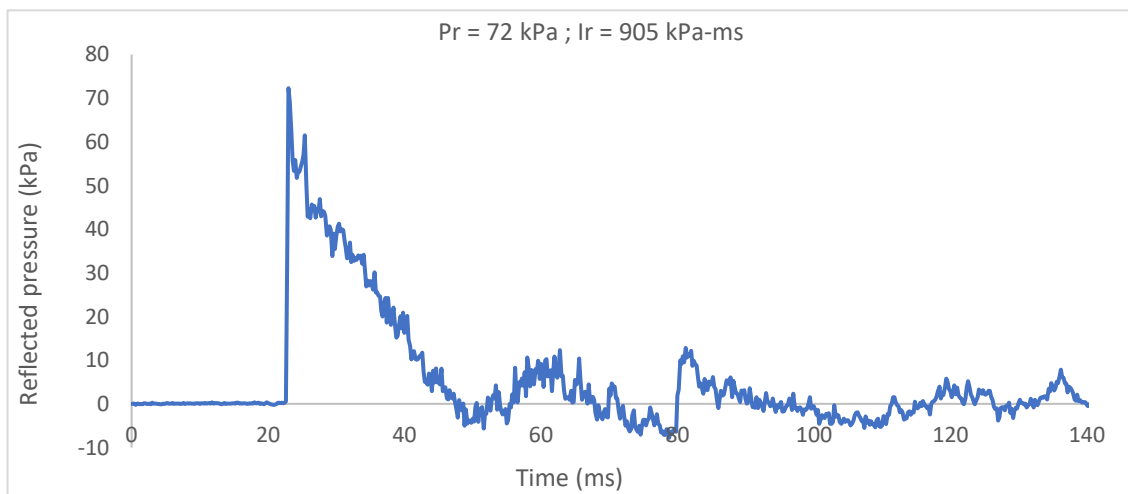
Figure 4.23 (Cont'd)



f) Pressure-time history used for first mullion model (M-C1)

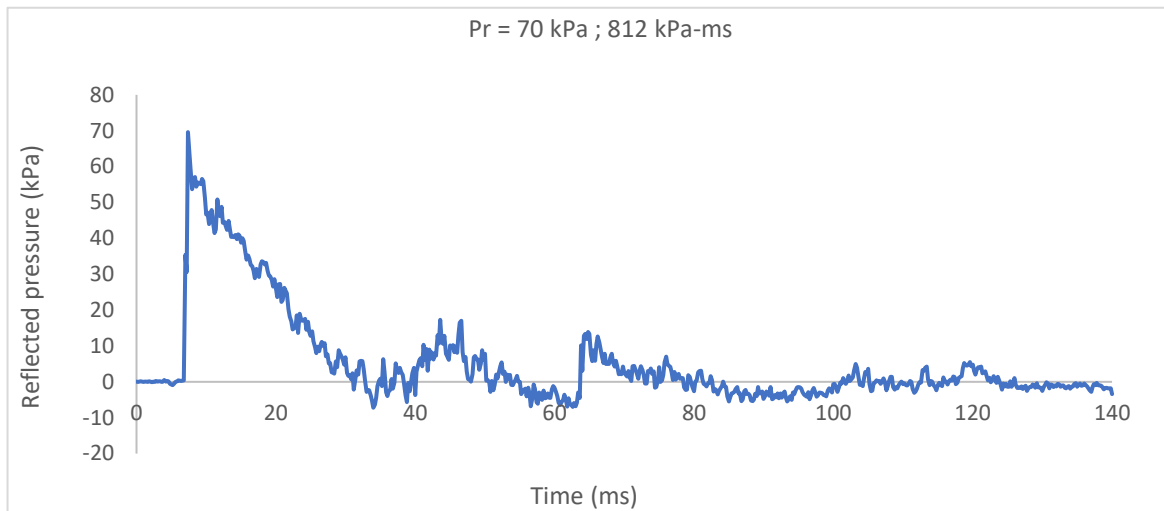


g) Pressure-time history used for second mullion model (M-C2)

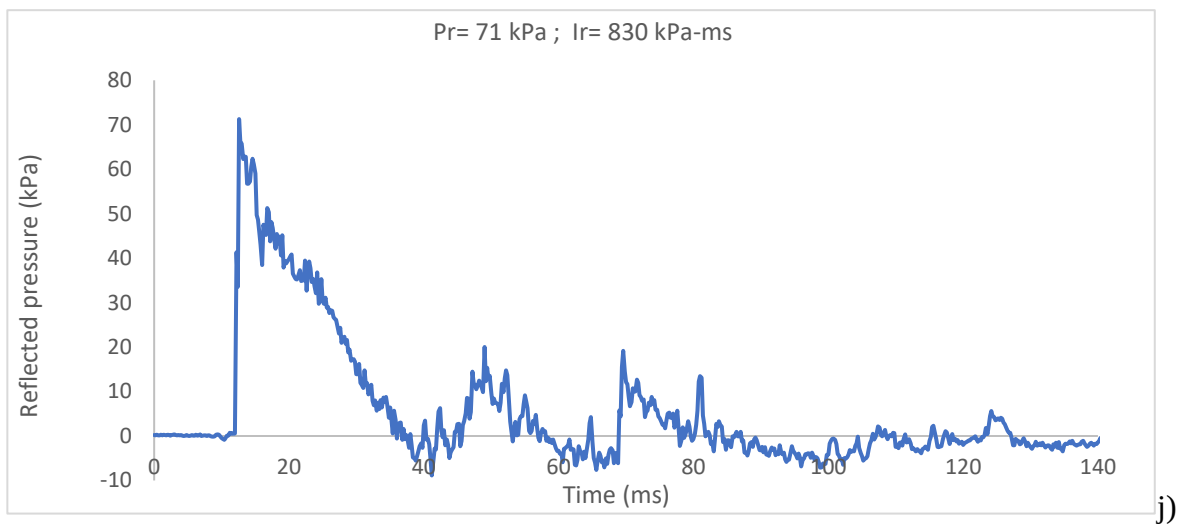


h) Pressure-time history used for third mullion model (M-C3)

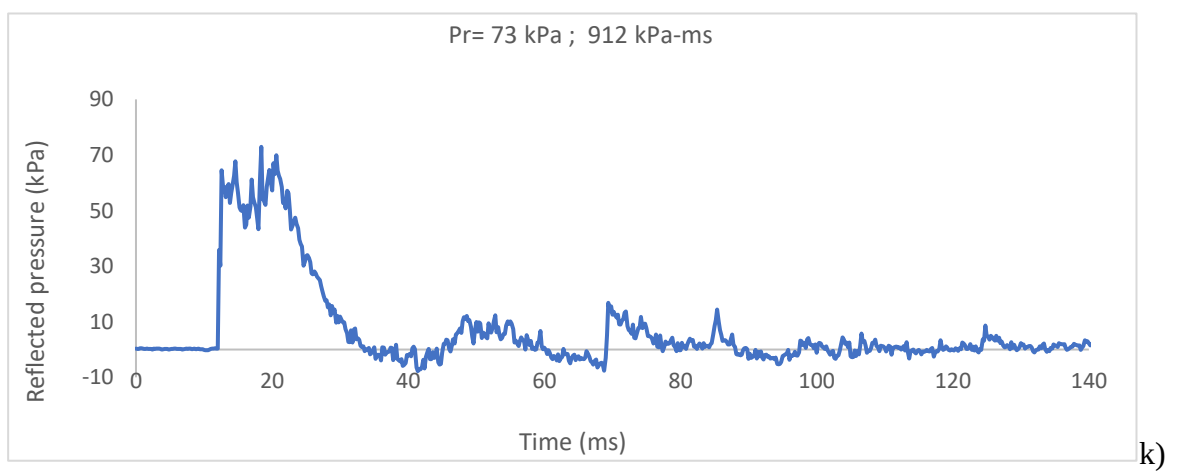
Figure 4.24 (Cont'd)



i) Pressure-time history used for forth mullion model (M-C4)

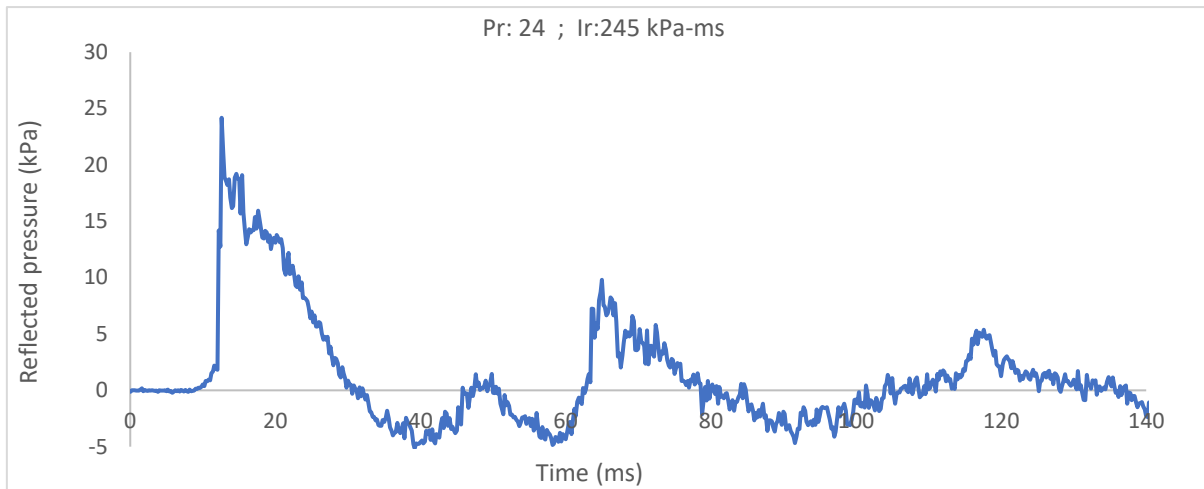


Pressure-time history used for fifth mullion model (M-C5)

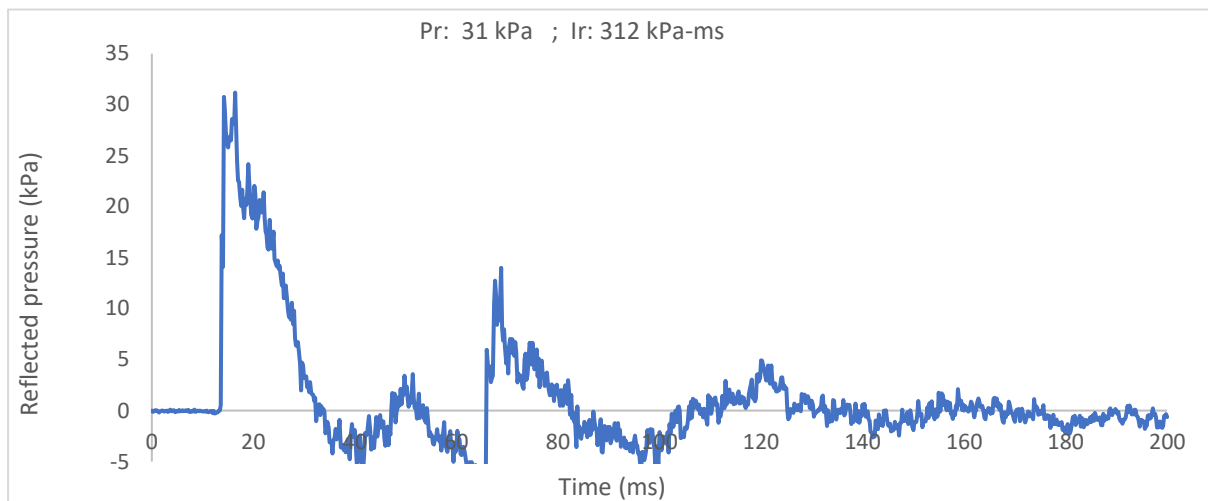


Pressure-time history used for sixth mullion model (M-C6)

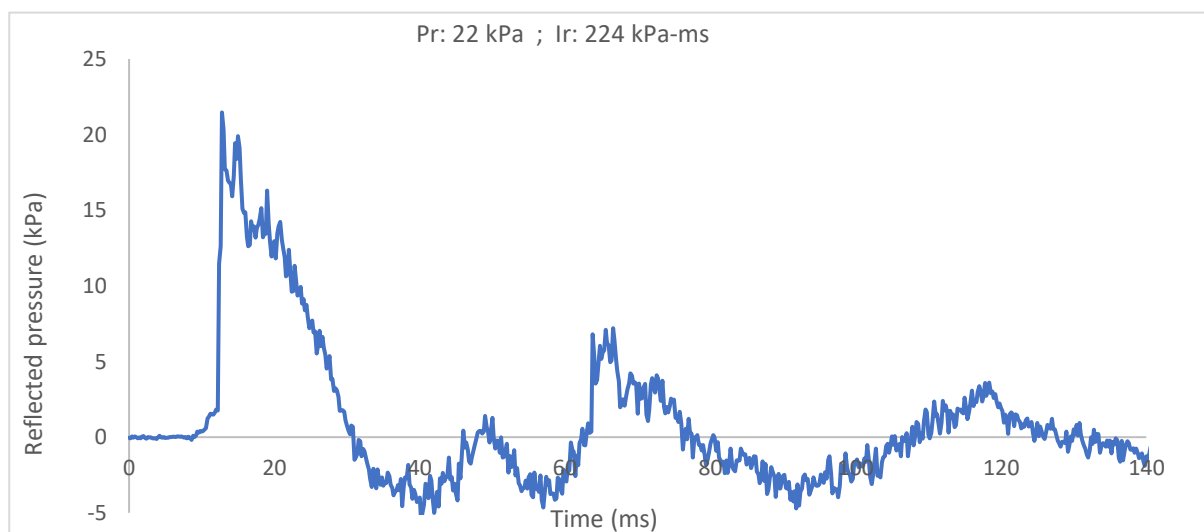
Figure 4.25 (Cont'd)



l) Pressure-time history used for seventh mullion model (M-C7)

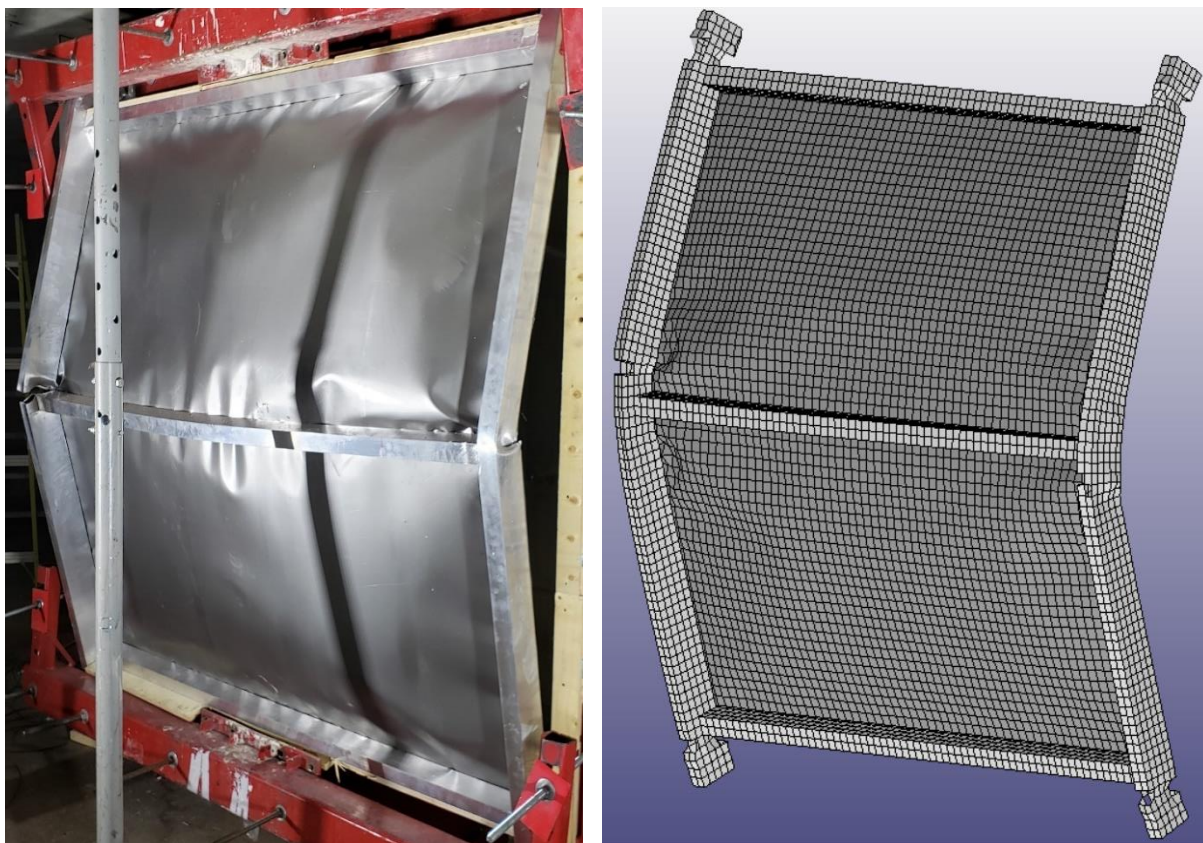


m) Pressure-time history used for eighth mullion model (M-C8)

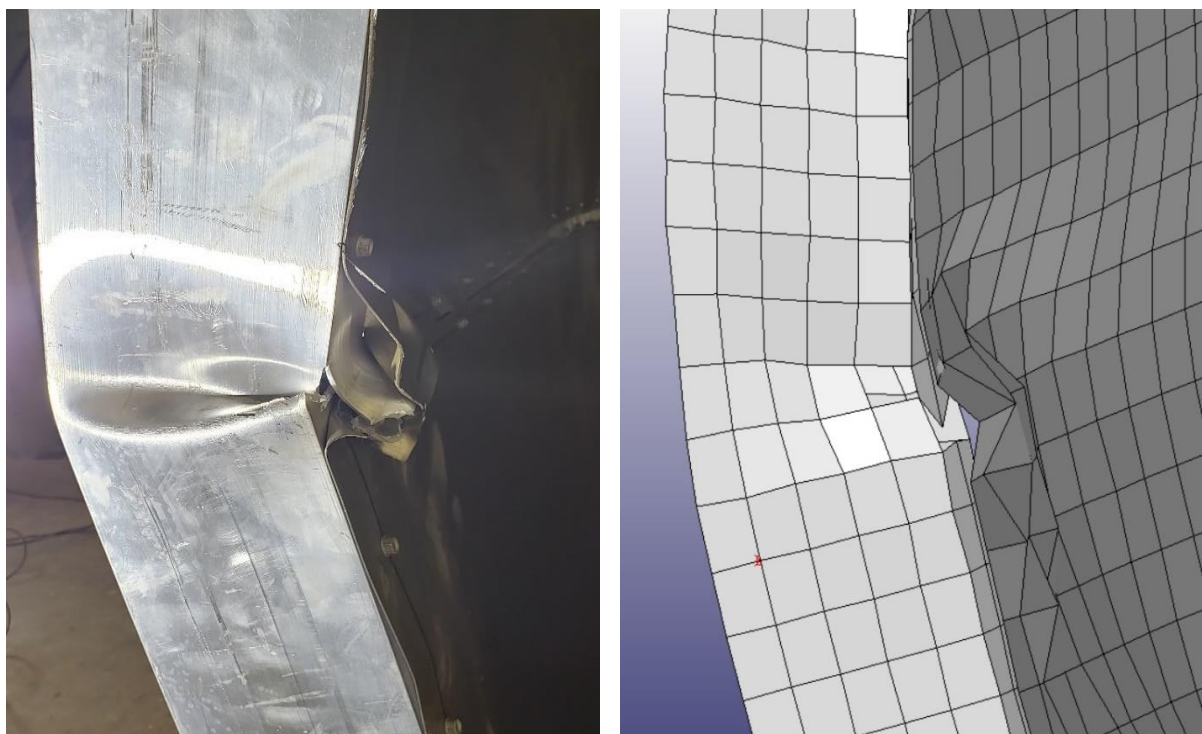


n) Pressure-time history used for ninth mullion model (M-C9)

Figure 4.26: Reflected pressure-time histories used in FEM analysis

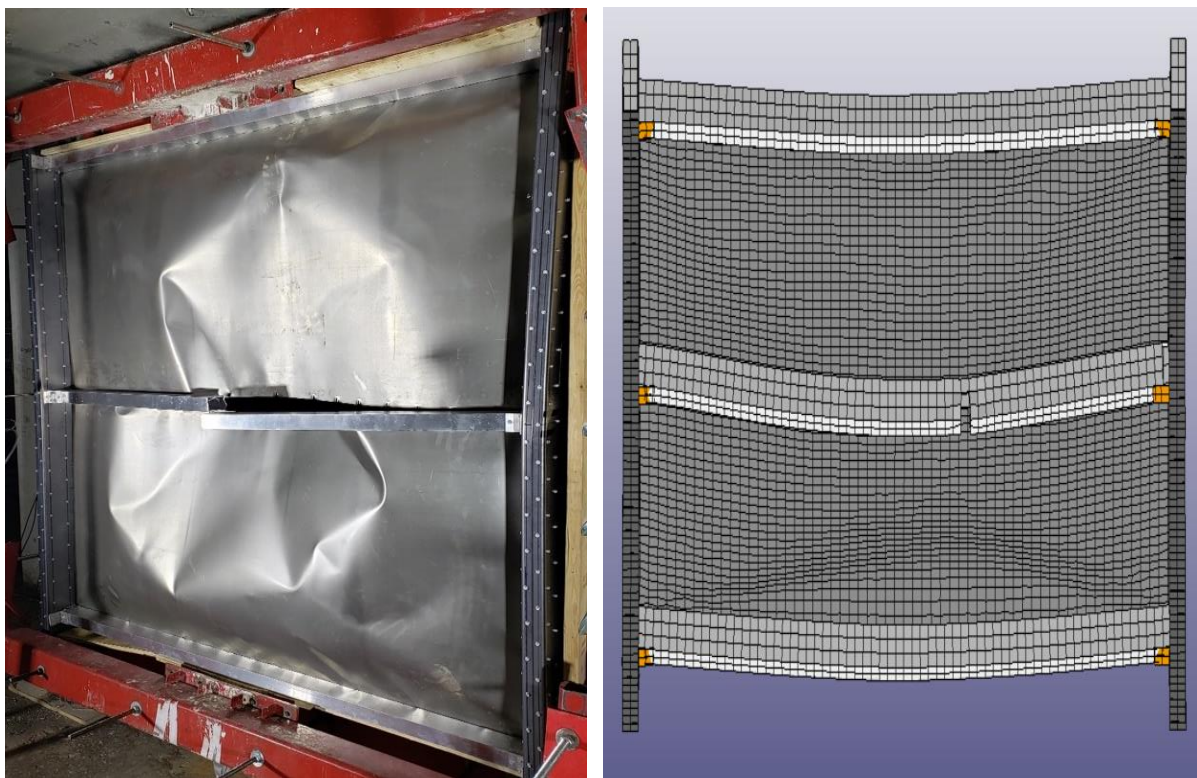


a) Overall actual experiment and numerical performance

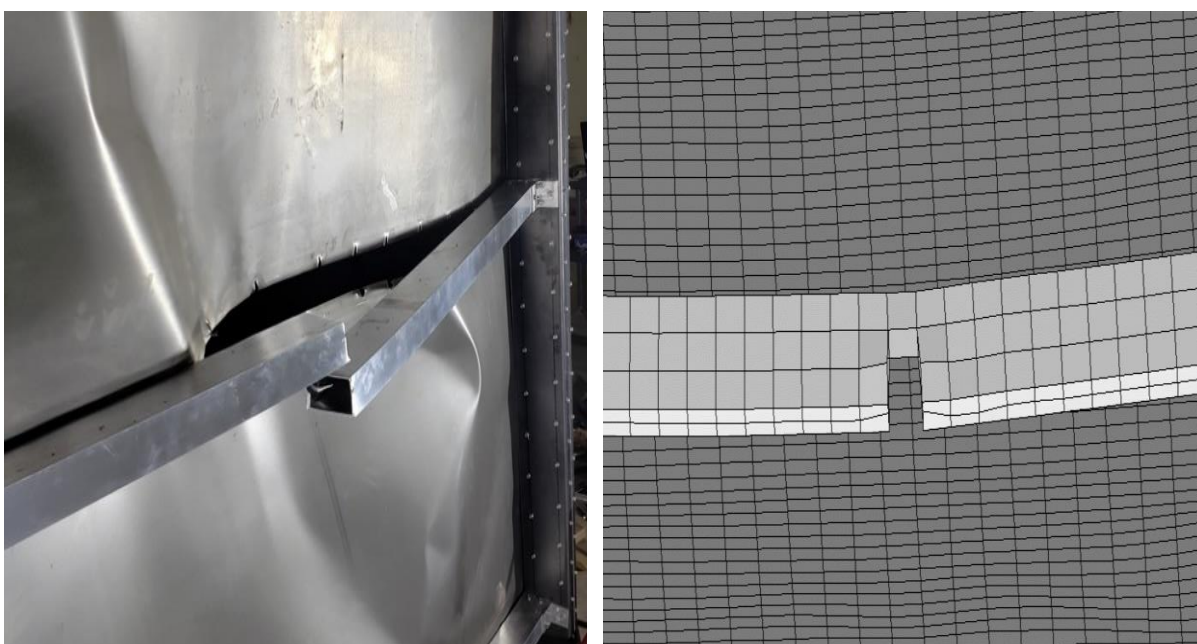


b) Close-up view of mullion and steel sheet

Figure 4.27: Comparison of experimental and numerical behavior of the first curtain wall (CW1)

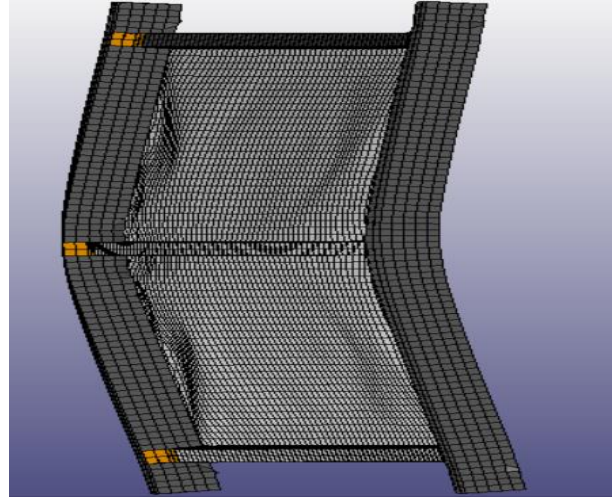


a) Overall test and numerical performance

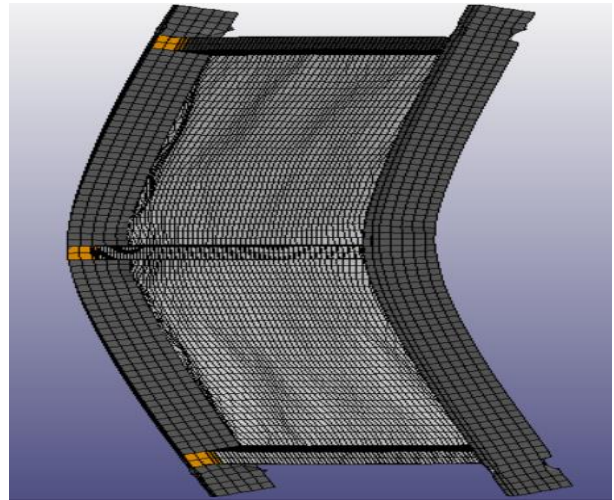


b) Close-up view of middle transom failure zone

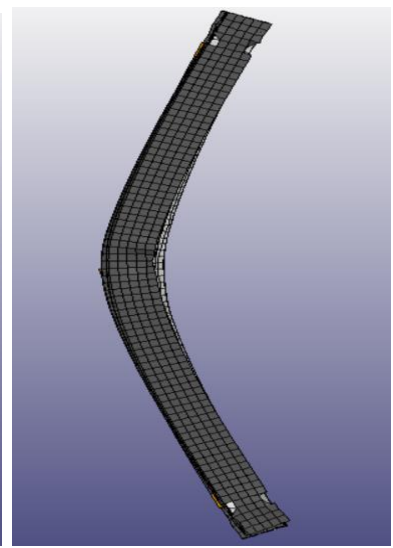
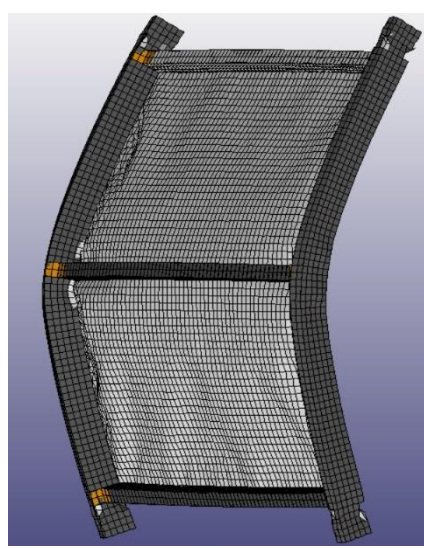
Figure 4.28: Comparison of experimental and numerical behavior of the second curtain wall (CW2)



a) Overall actual test and numerical performance after First shot

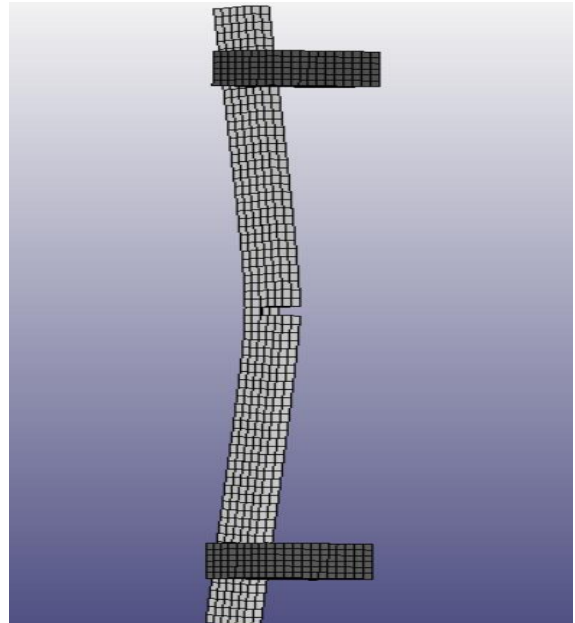


b) Overall experiment and simulation performance after Second shot

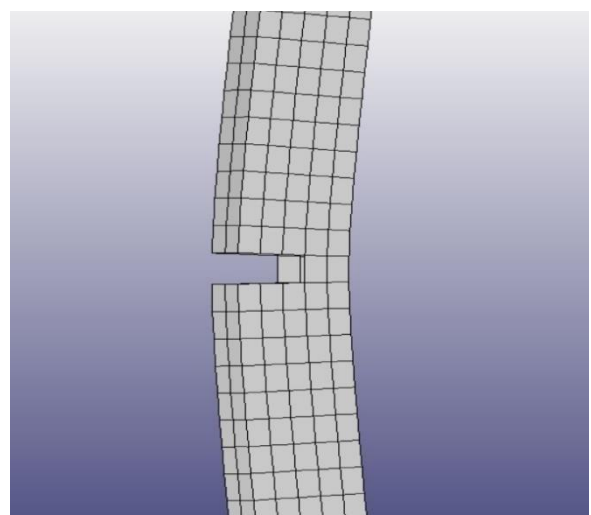


c) Overall test and numerical performance after Third shot

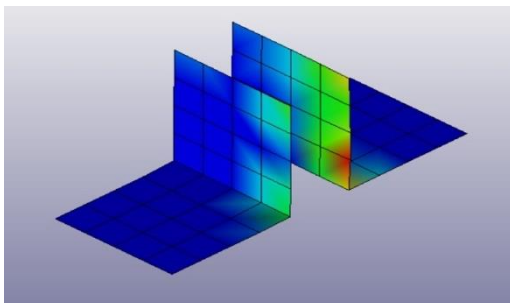
Figure 4.29: Comparison of experimental and numerical behavior of the third curtain wall (CW3)



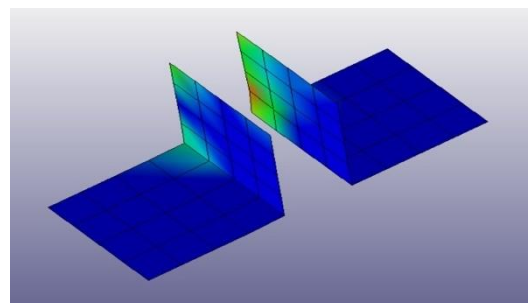
a) Overall test and numerical performance



b) Close-up view of mullion deflection at mid-height of single mullion

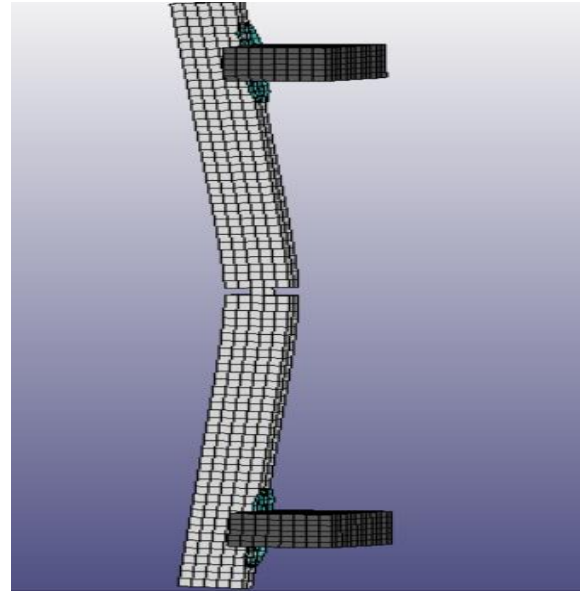


c) Brackets on the Top concrete beam

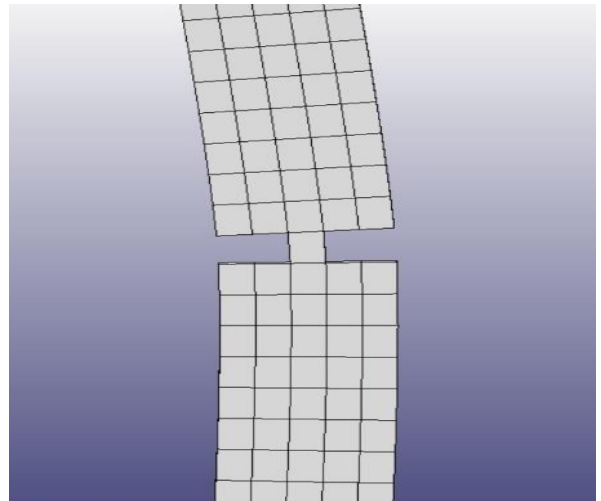


d) Brackets on the Bottom concrete beam

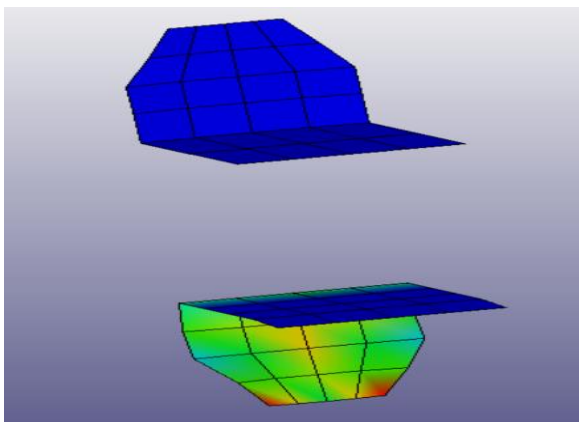
Figure 4.30: Comparison of experimental and numerical behavior of first mullion (M-C1)



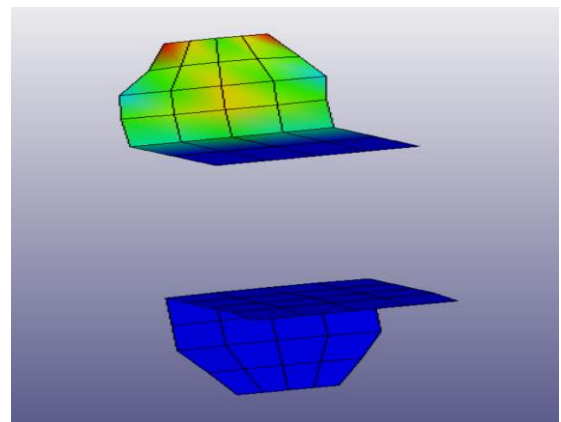
a) Overall actual test and numerical performance



b) Close-up view of mullion deflection at mid-height

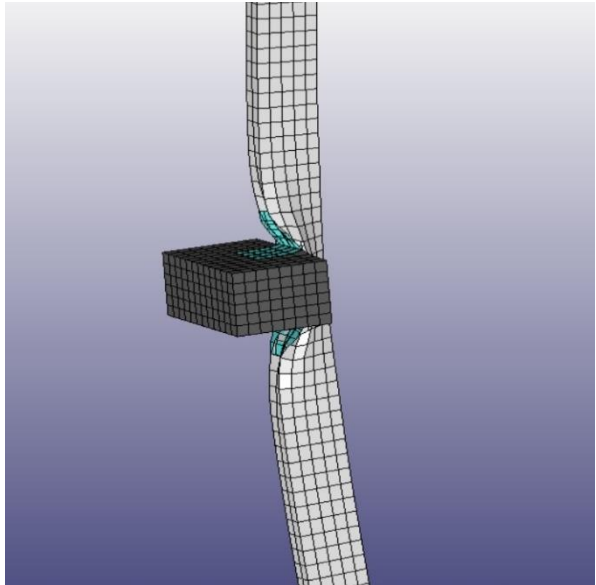


c) Brackets on Top concrete beam

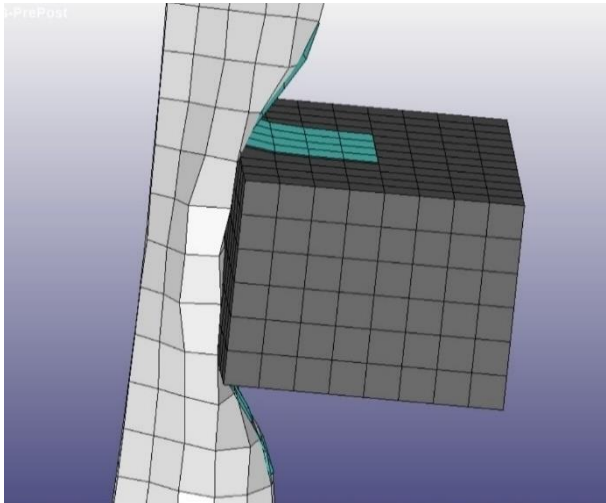


d) Brackets on Bottom concrete beam

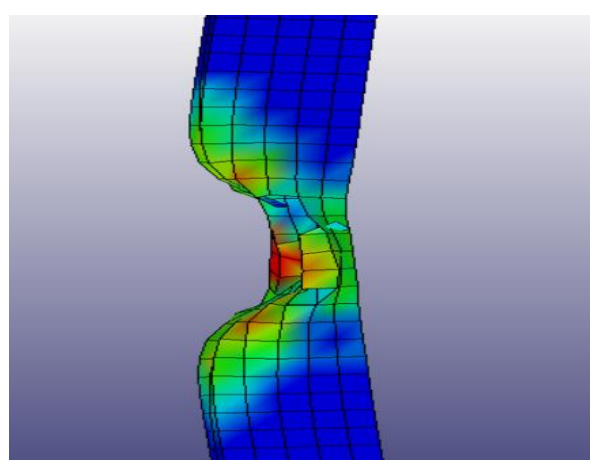
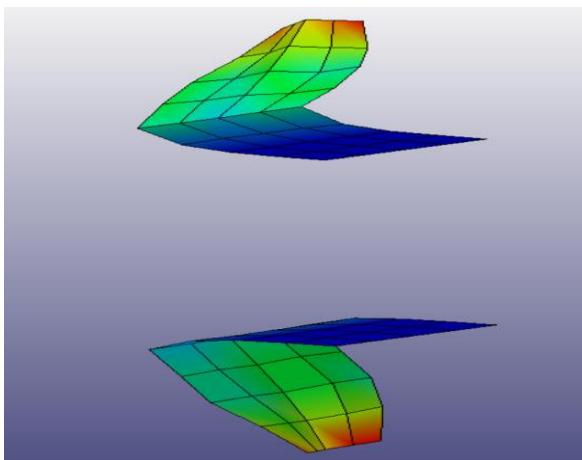
Figure 4.31: Comparison of experimental and numerical behavior of second mullion (M-C2)



a) Overall actual test and numerical performance

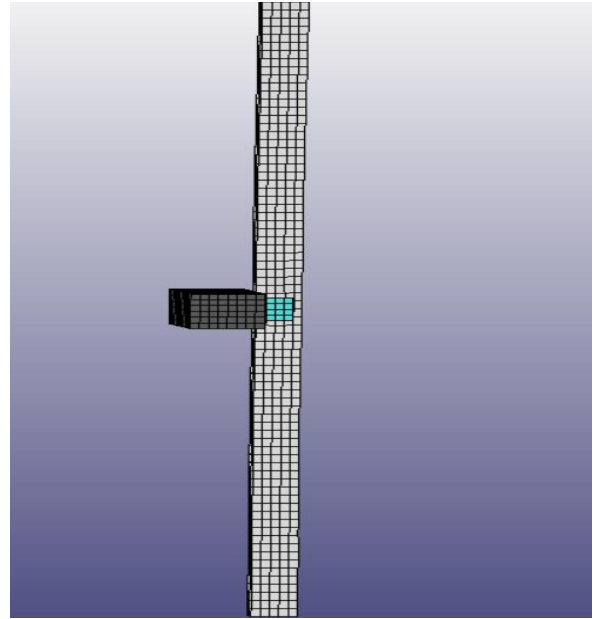


b) Close-up view of mullion failure zone at the middle concrete beam

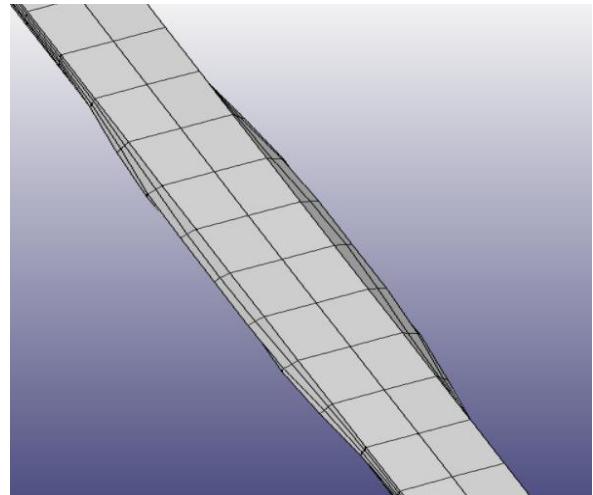


c) Stress distribution in the brackets and mullion

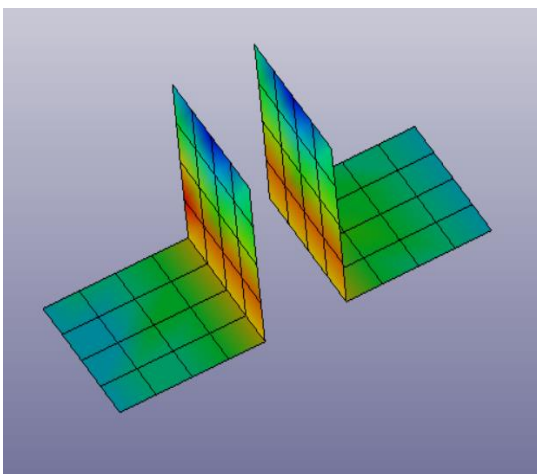
Figure 4.32: Comparison of experimental and numerical behavior of third mullion (M-C3)



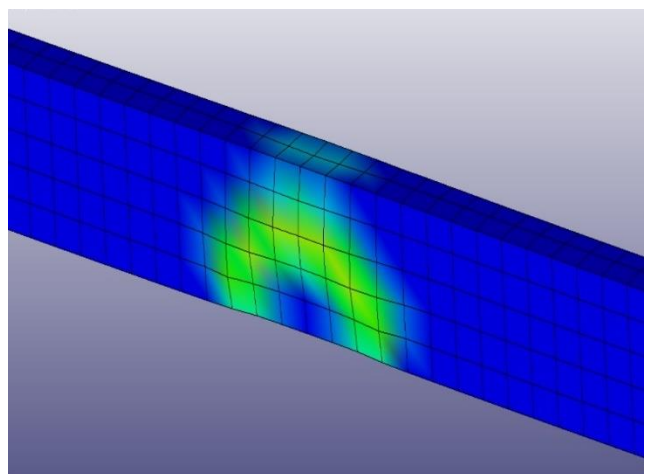
a) Overall experiment and numerical performance



b) Close-up view of buckling zone at mid-height where brackets were located

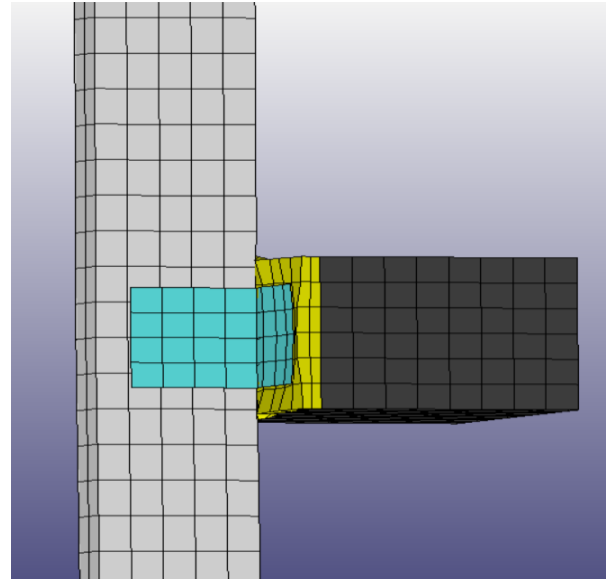


c) Pressure in the brackets

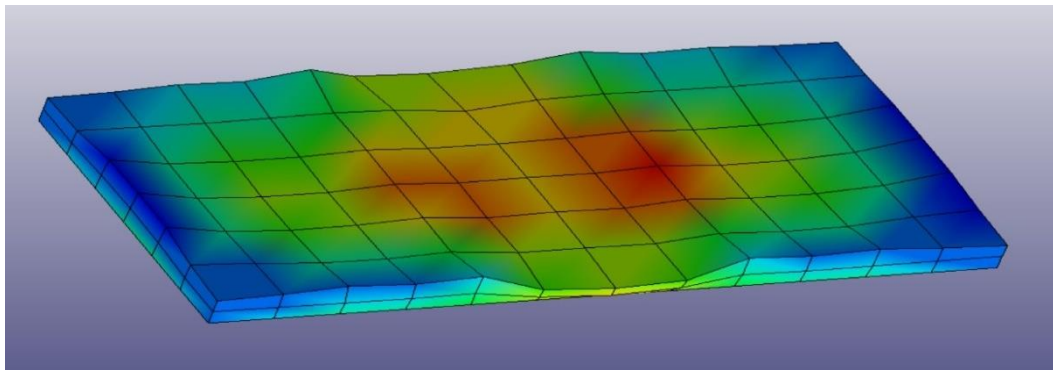


d) Effective plastic strain (brackets location)

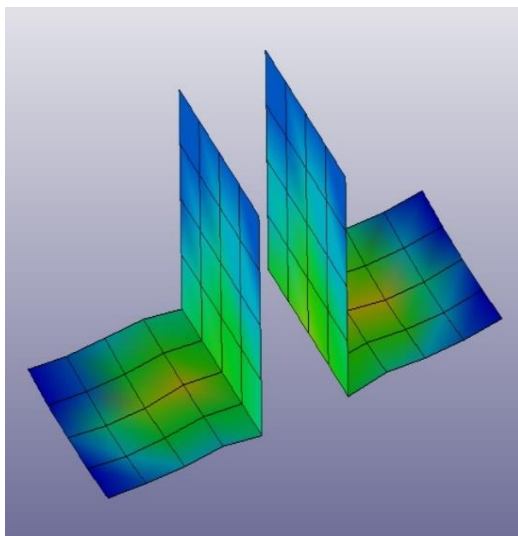
Figure 4.33: Experimental vs numerical behavior of forth mullion (M-C4)



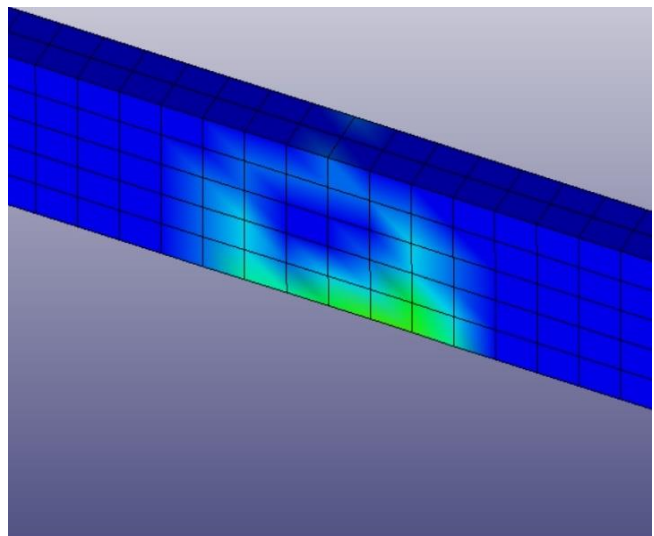
a) Experiment and numerical observation



b) XRD rubber stress distribution



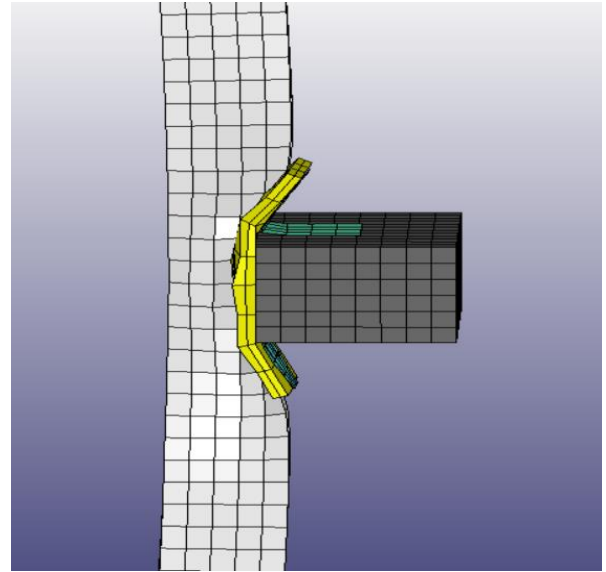
c) Effective plastic strain in brackets



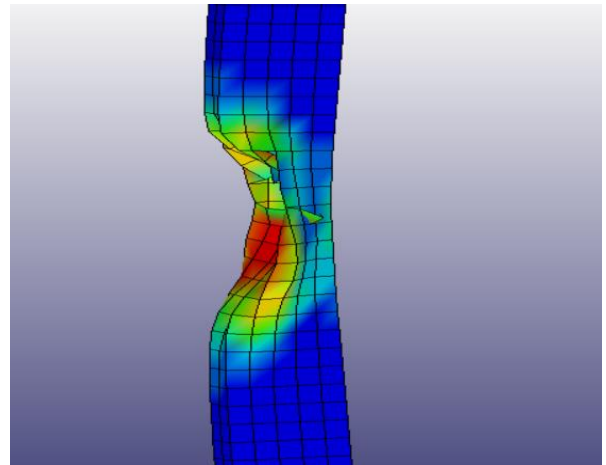
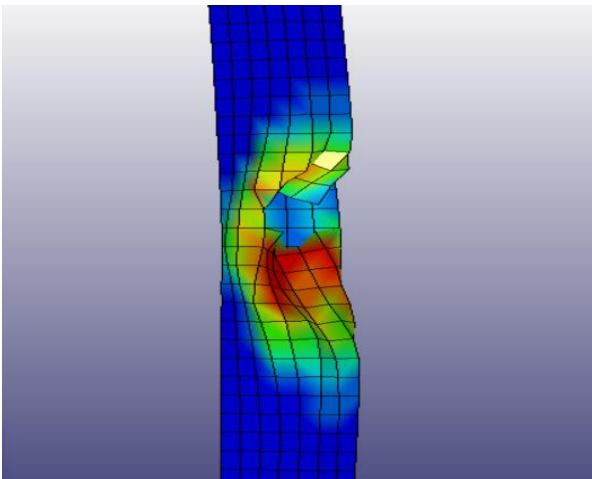
d) Mullion stress distribution (brackets location)

Figure 4.34: Experimental vs numerical behavior of fifth mullion with inserted XRD rubber

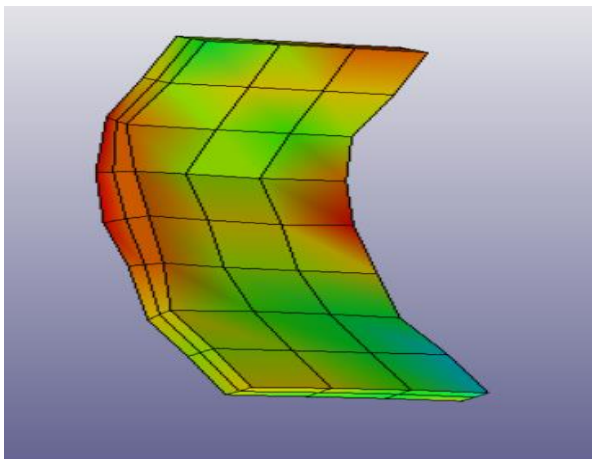
(M-C5)



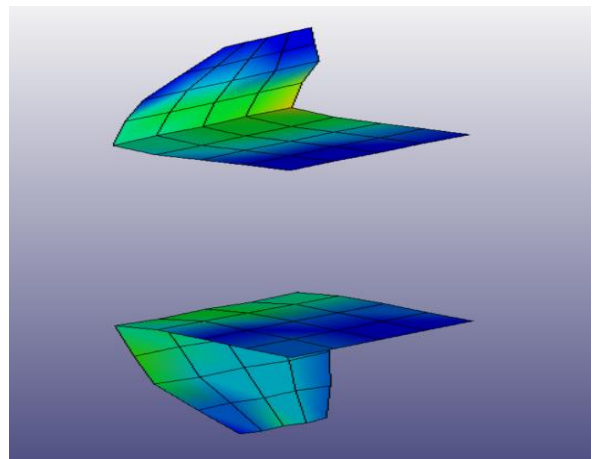
a) Experiment and numerical observation at the middle concrete beam



b) Close-up view of failure and stress distribution at the mid-height of mullion



c) Stress distribution on Poron XRD rubber



c) Effective plastic strain in brackets

Figure 4.35: Experimental vs numerical behavior of sixth mullion with inserted XRD rubber

(M-C6)

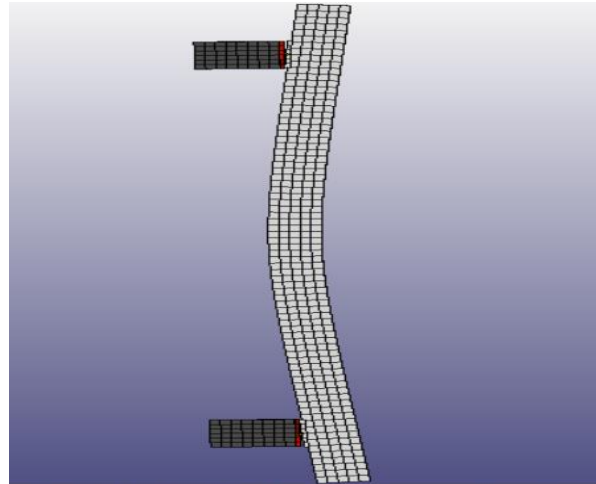
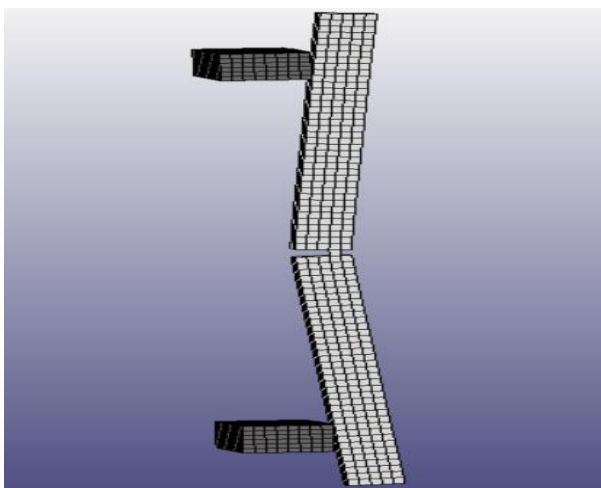
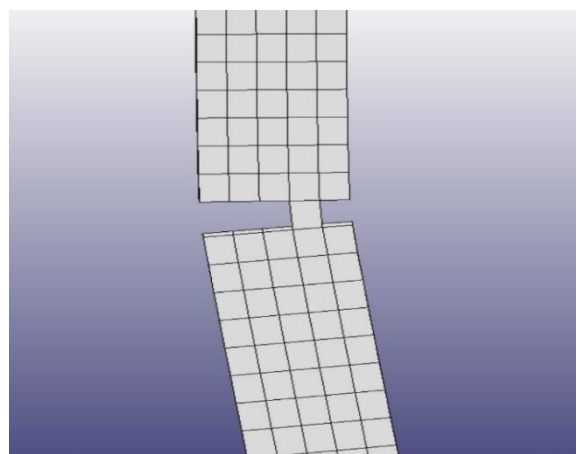


Figure 4.36: Experiment vs numerical observations of seventh mullion with inserted springs (M-C7)



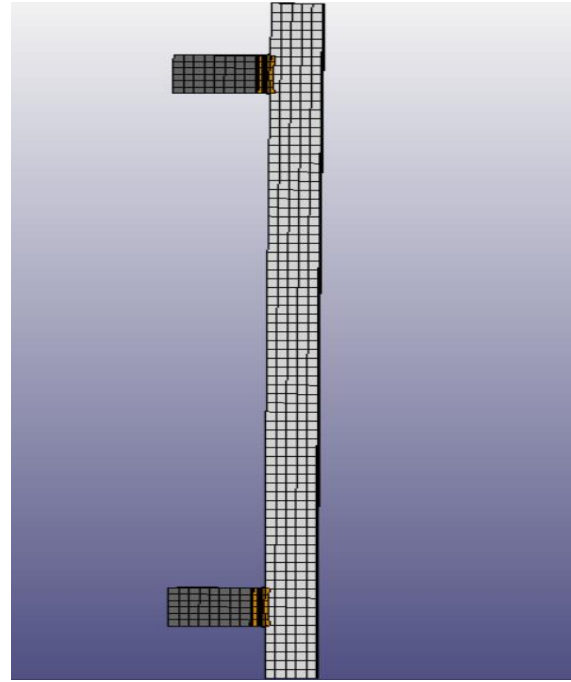
a) Overall experiment vs numerical performance of observation M-C8



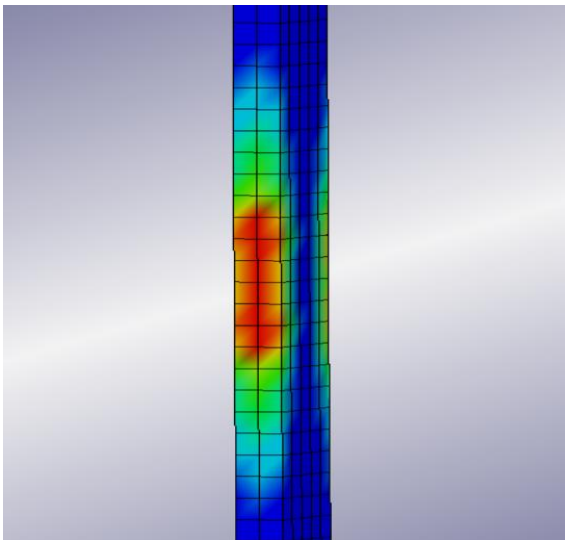
b) Close-up view of mullion failure zone at mid-height

Figure 4.37: Experiment vs numerical performance of eighth mullion with inserted springs

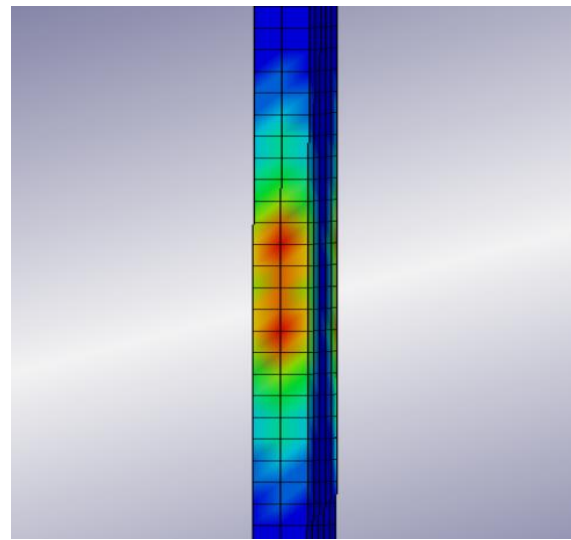
(M-C8)



a) Overall Experiment vs numerical observations (fully squeezed springs)

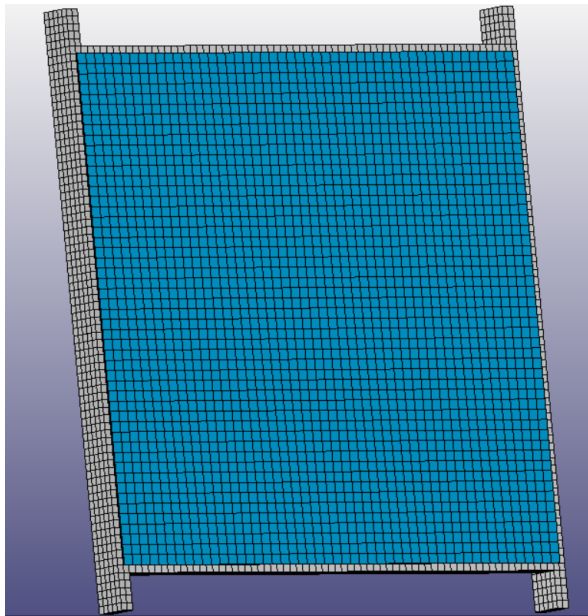


b) Stress distribution on tension side

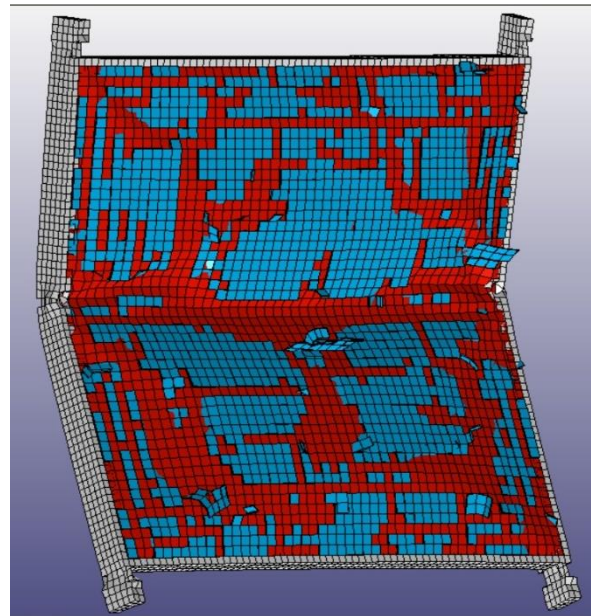


c) Stress distribution on compression side

Figure 4.38: Experiment vs numerical performance of ninth mullion with inserted springs
(M-C9)



a) CW1+G&F before test

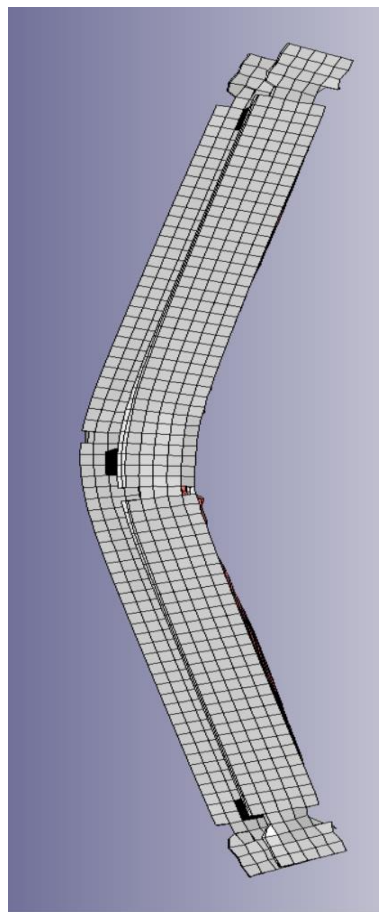


b) CW1+G&F after applying the load

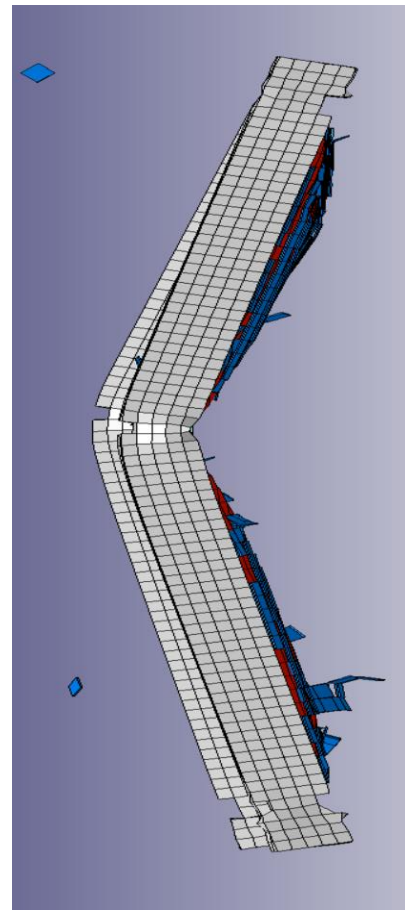
Figure 4.39: The first curtain wall model with glass + film (CW1 + G&F)



a) Experiment

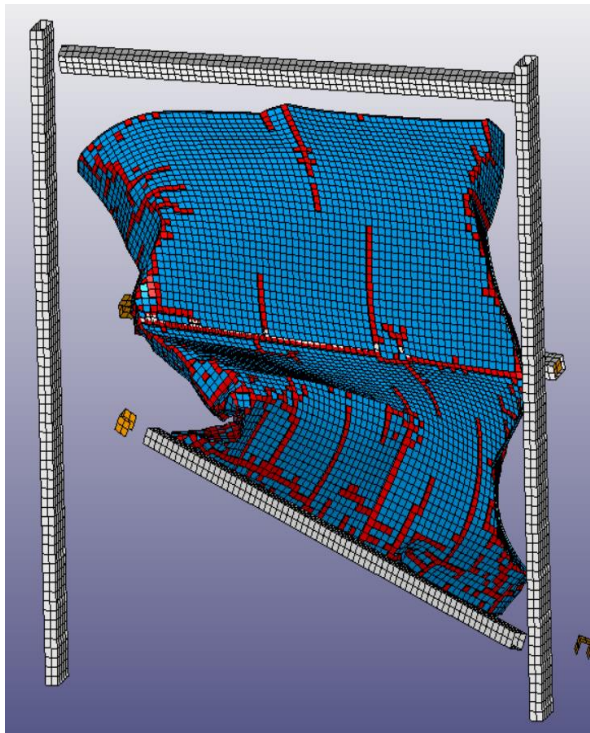


b) CW1

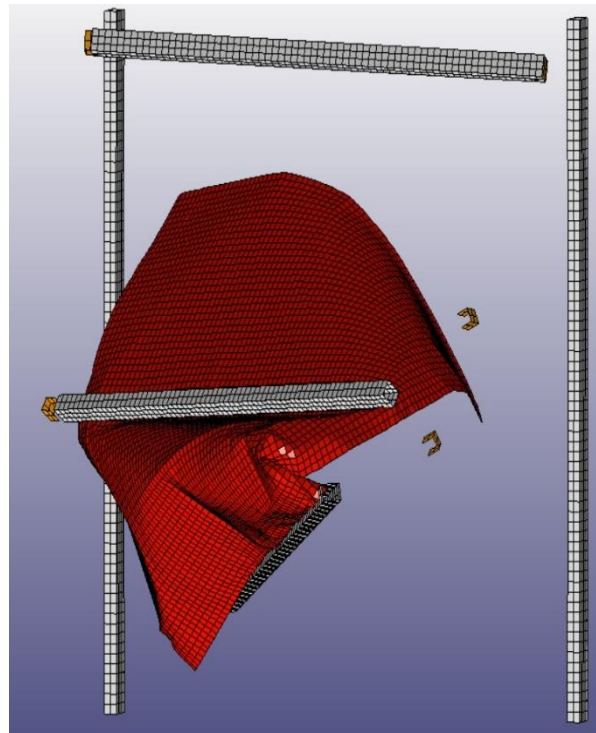


c) CW1 + G&F

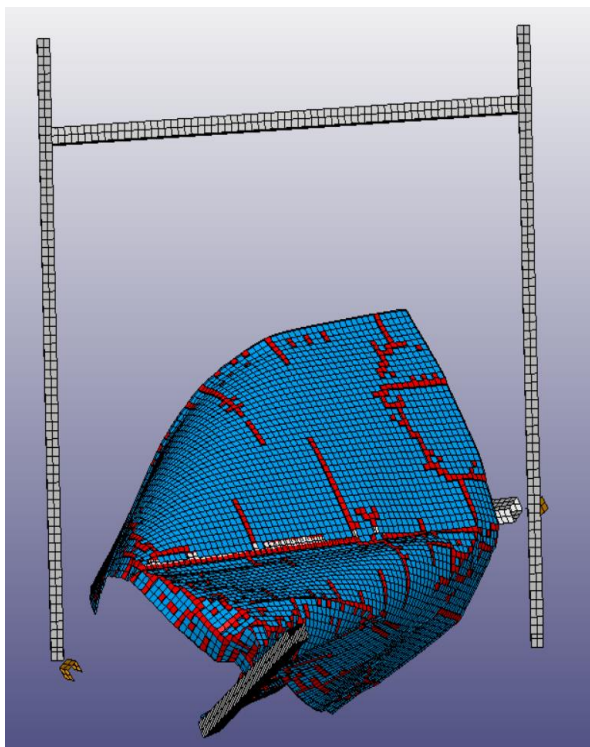
Figure 4.40: The comparison of first curtain wall failure using steel sheet versus glass + film



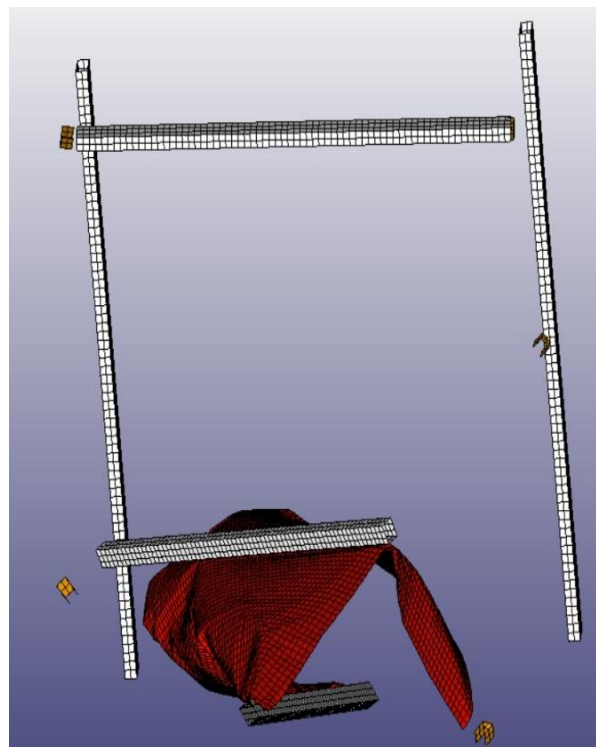
a) Front view (36 ms)



b) Back view (66 ms)

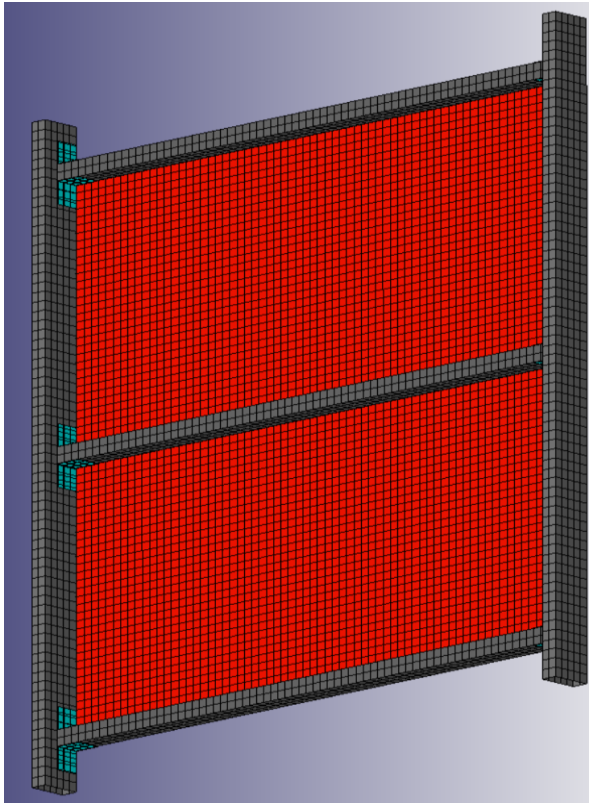


a) Front view (106 ms)

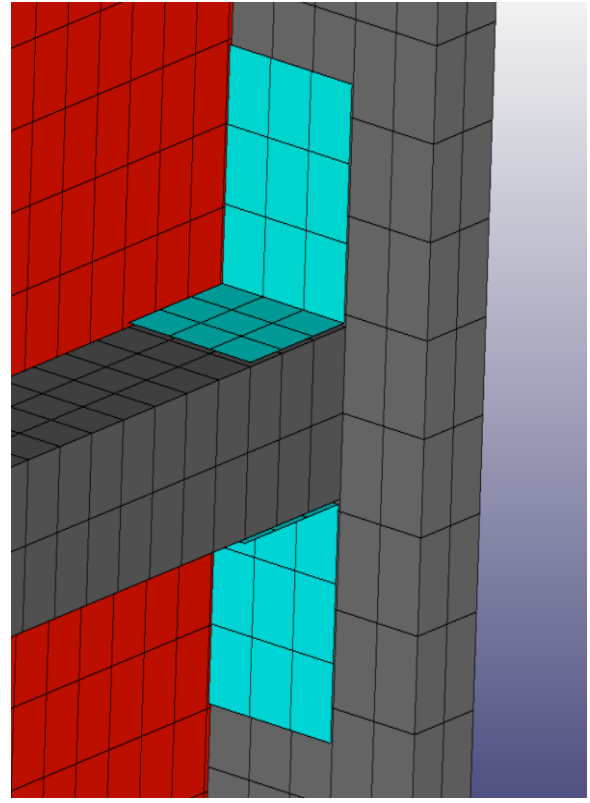


b) Back view (146 ms)

Figure 4.41: Failure mode of 1st curtain wall with weak U-angle connections (CW1+WC)



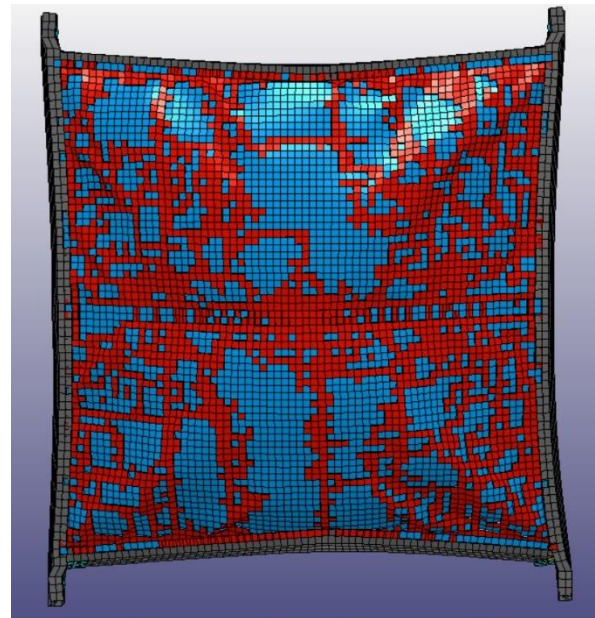
a) Overall view of hardened sections



b) Close-up view of top and bottom brackets

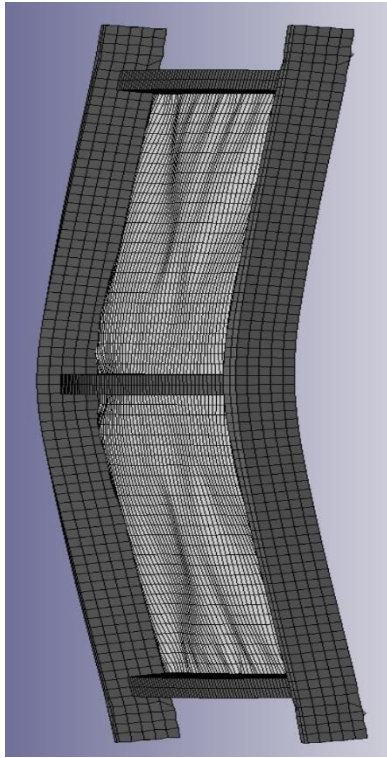


c) Transom deflection (back view)

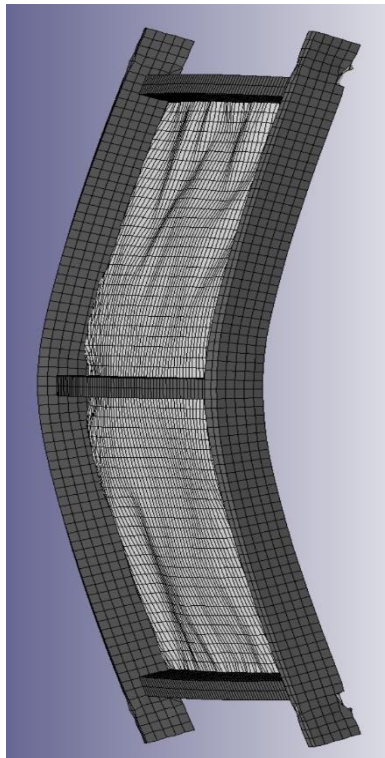


d) In-plane displacement (front view)

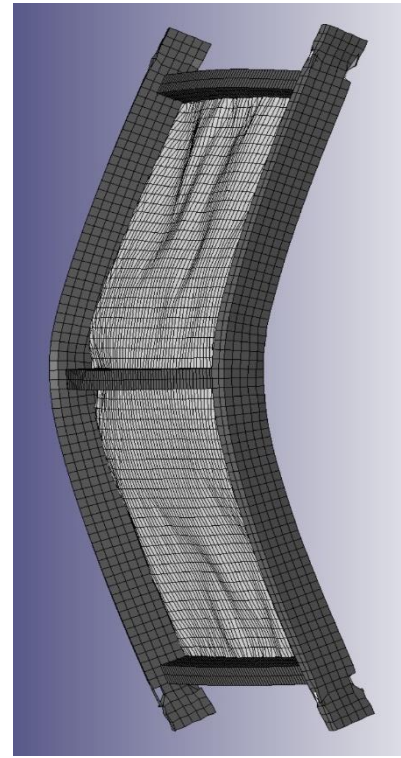
Figure 4.42: Curtain wall with weak U-angles and brackets on top and bottom (CW3+T&B)



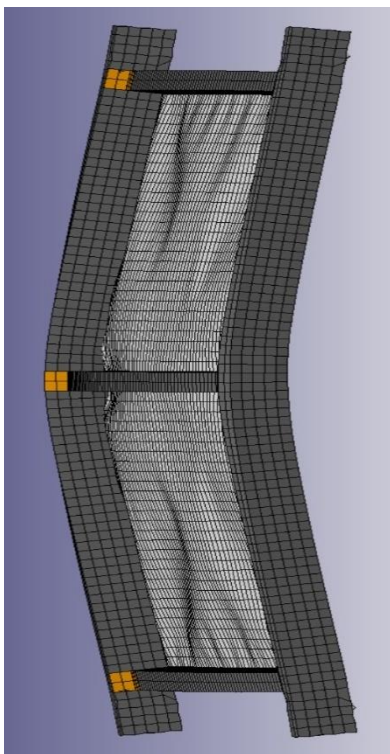
a) CW3_NB (1st shot)



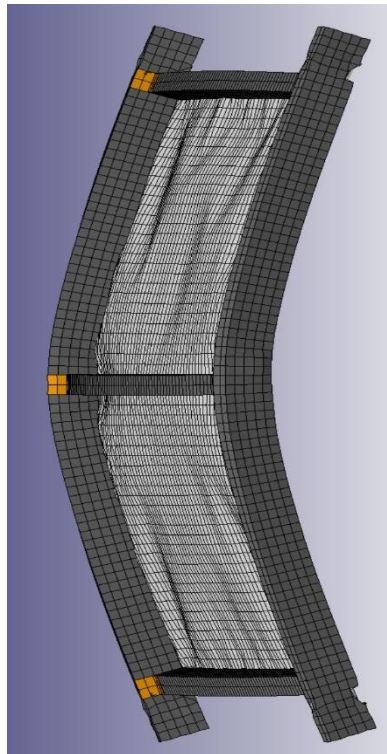
b) CW3_NB (2nd shot)



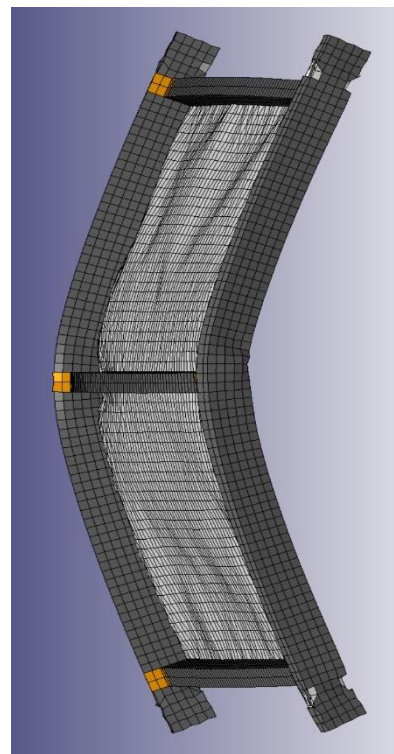
c) CW3_NB (3rd shot)



a) CW3 (1st shot)



b) CW3 (2nd shot)



c) CW3 (3rd shot)

Figure 4.43: The comparison of third curtain wall model with and without side brackets

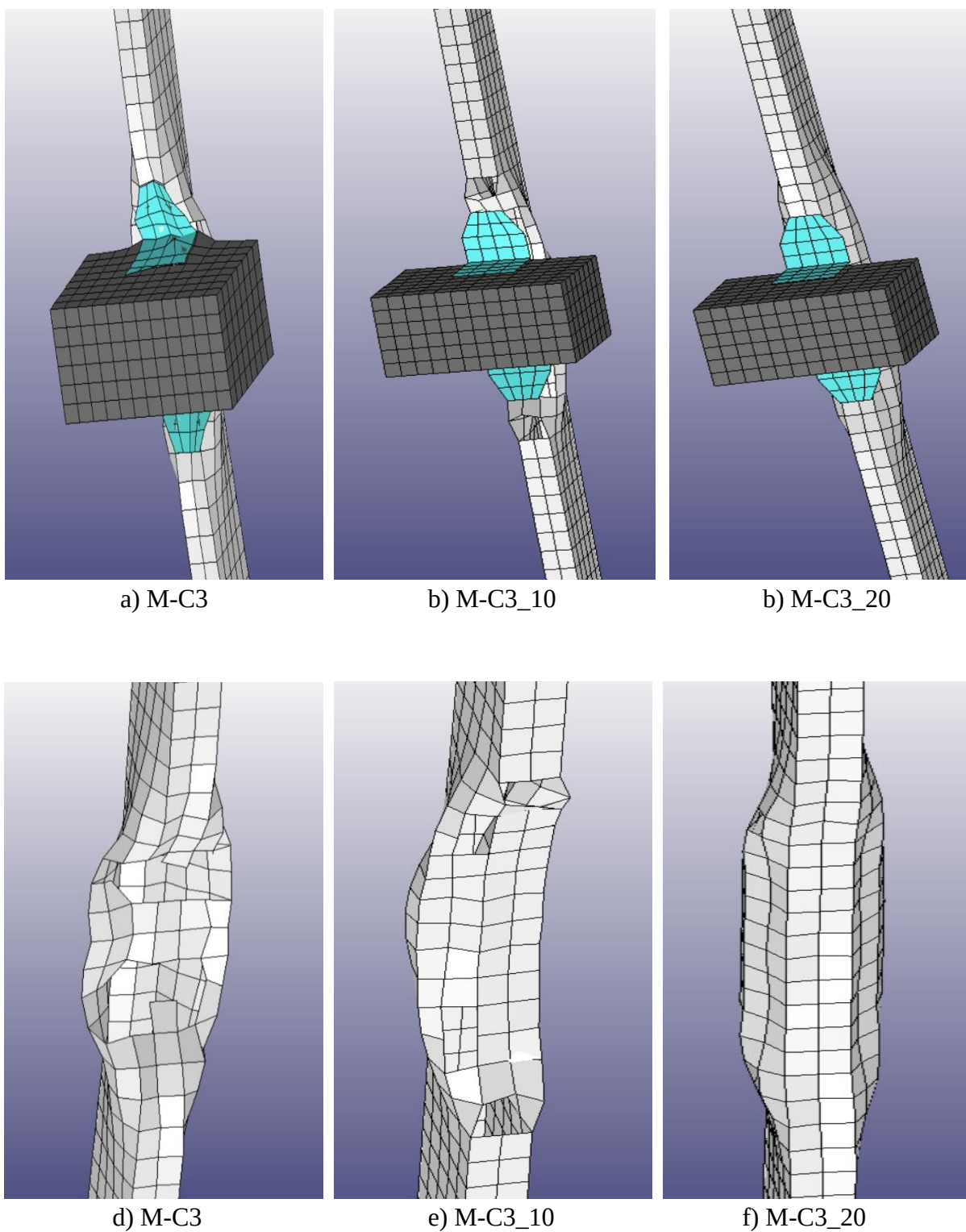
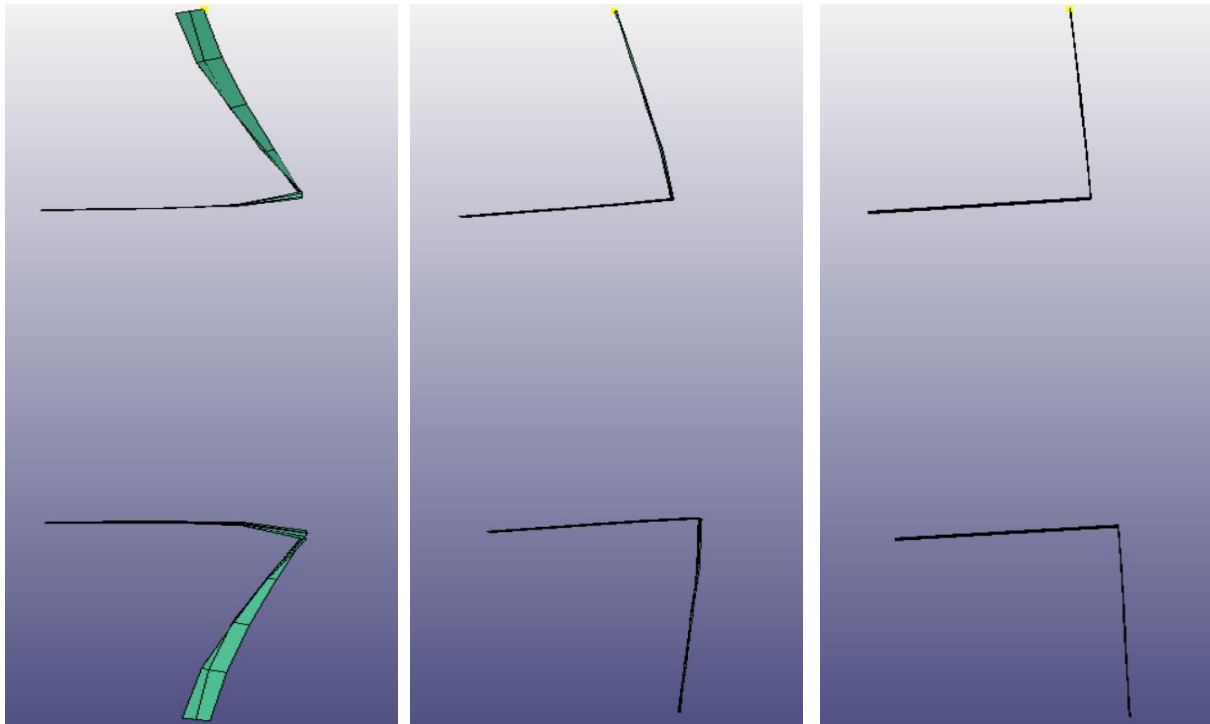


Figure 4.44: Observations at mid-height of mullions, and the effect of bracket thickness on the failure mode

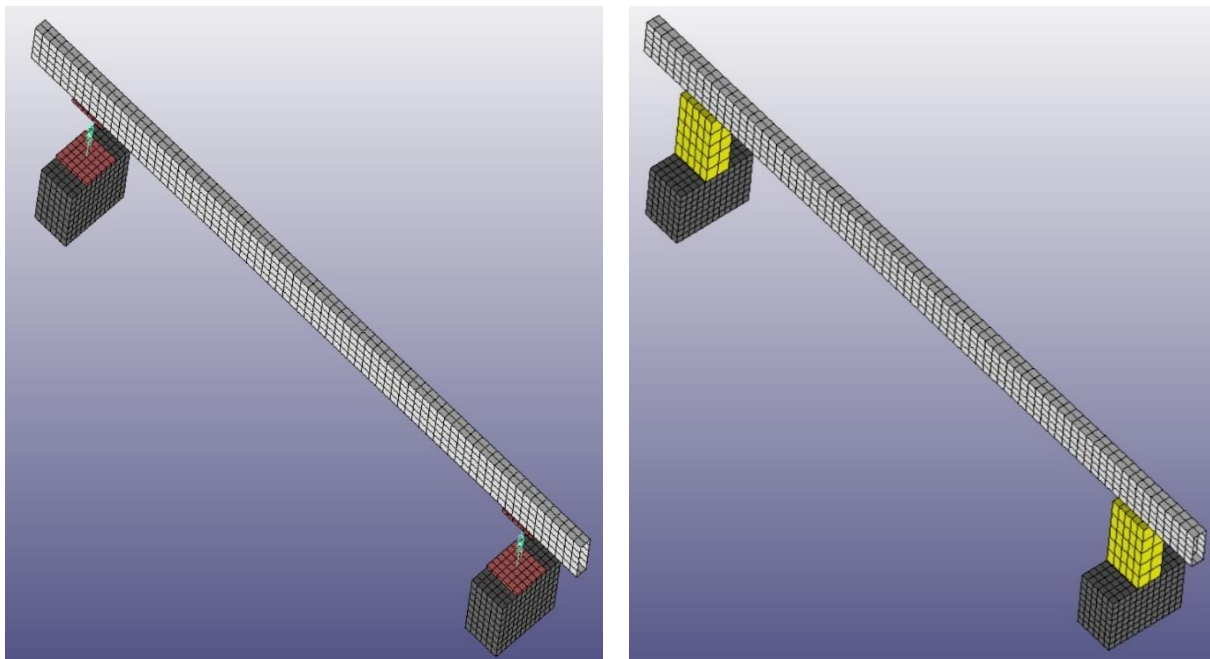


a) M-C3
Bracket thickness: 3mm
Max deflection: 40 mm

b) M-C3_10
Bracket thickness: 10mm
Max deflection: 23 mm

c) M-C3_20
Bracket thickness: 20mm
Max deflection: 3 mm

Figure 4.45: Bracket deflection at the top node of brackets along the applied load



a) Spring net length: 200 mm

b) XRD net length: 200 mm

Figure 4.46: SP-100 and XRD-100 models before applying the load

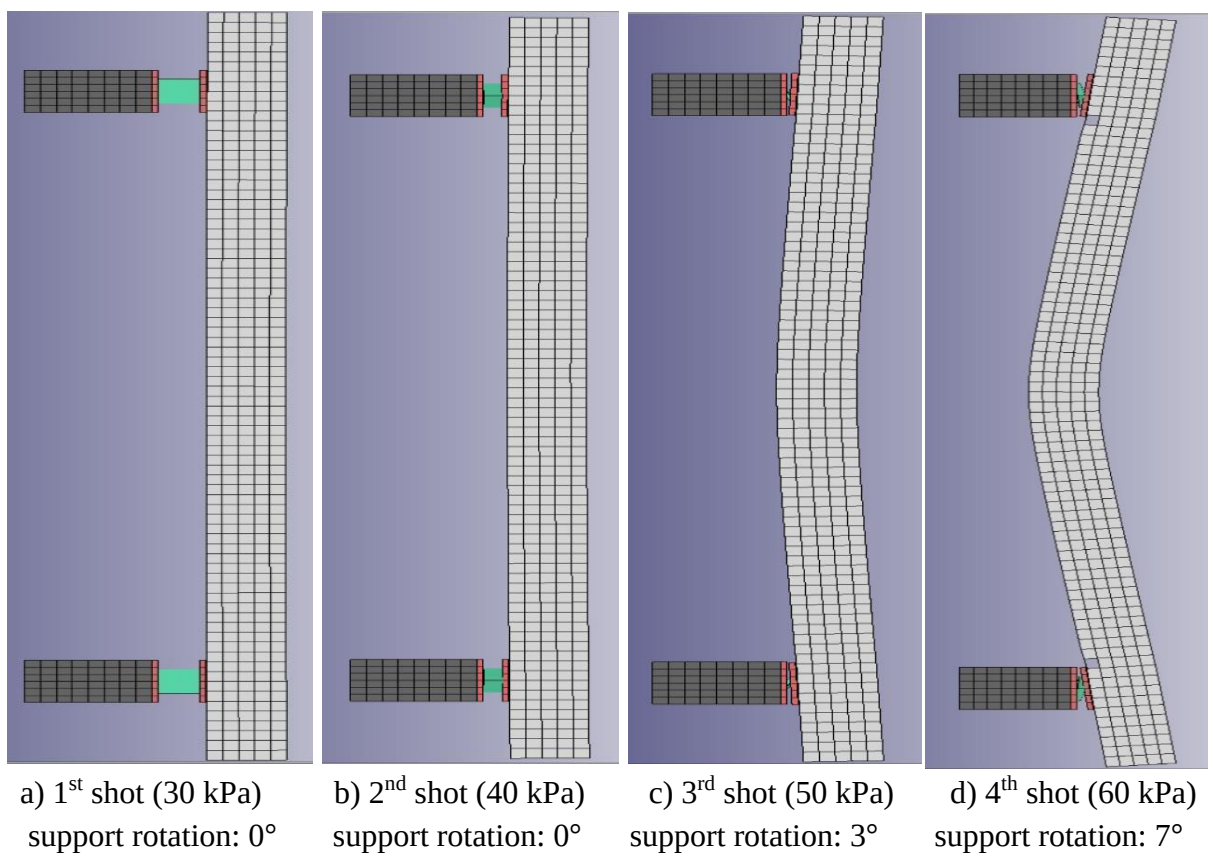


Figure 4.47: Comparison of SP-100 with inserted spring after each shot

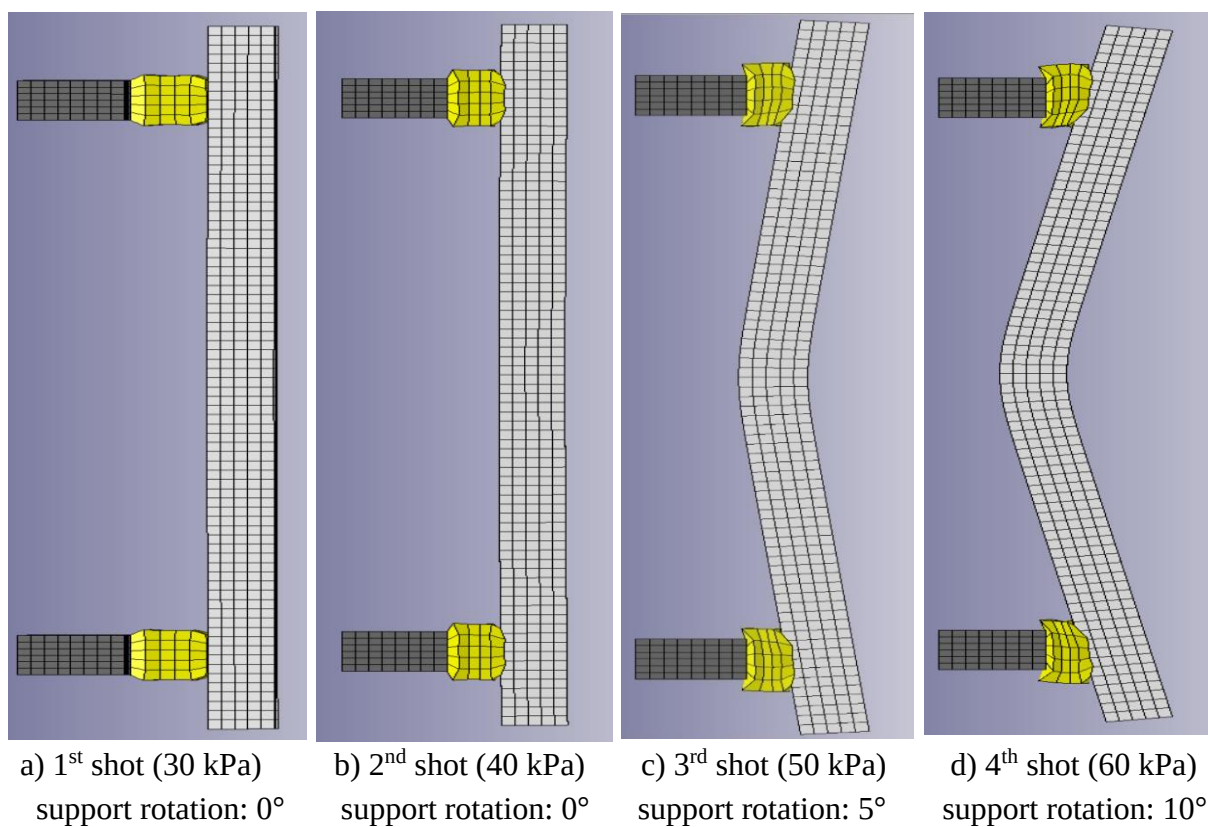


Figure 4. 48: Comparison of XRD-100 with inserted XRD rubber after each shot

CHAPTER 5. Analysis and Design of Curtain Wall Connections

Abstract

Curtain wall and glass window systems are widely used in building structures as building façade elements, typically consisting of aluminum frame elements supporting glass panes. Their primary function is to protect buildings against the effects of weather, acting as a buffer and an insulator while offering architectural functionalities. They are designed to provide resistance to gravity and wind loads. However, curtain walls and windows are the most lethal elements of the building façade when subjected to blast shock waves generated by accidental explosions or maliciously intended bomb attacks. Blast-resistant glass panes are glazed by security films or interlaminates to resist blast loads. Frame elements, such as vertical mullions and horizontal transoms can be strengthened by steel plates. This leaves connections between mullions and transoms, as well as those between mullions and supporting structural elements to be vulnerable unless they are designed to withstand blast loads. Research in curtain wall connection analysis and design under blast loads is scarce in the literature. Experimental and numerical research on curtain wall connections has been conducted as part of the current investigation, results of which are used to develop analysis and design guidelines. The applicability of single-degree-of-freedom (SDOF) dynamic analysis to generate connection design forces has been validated against numerical simulations. This paper provides SDOF analysis procedures suitable for use in connection design. It also provides design recommendations based on experimental and numerical observations.

Keywords: Blast-resistant curtain walls, connection design, dynamic analysis, mullion to transom connection, mullion to substrate connection, SDOF analysis, steel brackets.

5.1 Introduction:

Failure of building facades, and in particular curtain wall systems, under blast loads can lead to injuries and loss of lives. Many incidents have been documented in the literature, such as the 1995 Oklahoma City Bombing, the 2015 Mexico City hospital gas explosion (Norville et al. 1999), and Oslo bombing of 2011 (Fryberg 2002) to name a few. As building facades continue to incorporate large areas of glazing elements, especially in the form of curtain walls, the probability of human injuries or fatalities resulting from glass and aluminum framing element debris increases. Previous research on blast-resistant curtain walls, albeit scarce in the literature, has concentrated on the glazing techniques and the strengthening of mullions and transoms (Kirk et al. 2016, Benjamin et al. 2013, Anderson and Dover 2003, Adam et al. 2015, Timothy et al. 2015, Chiara and Claudio 2012). An extensive experimental program was undertaken at the University of Ottawa to investigate the performance of curtain walls under blast loads. Tests of curtain wall assemblies were conducted using a shock tube (Saatcioglu 2016, Saatcioglu et al. 2017a, 2017b, Saatcioglu et al. 2020 and Saatcioglu 2020). The first series of tests consisted of curtain walls protected by externally placed Guardian Coil as a protective material (Saatcioglu et al. 2017a). The second series consisted of curtain walls hardened by mechanically glazed protective security film with strengthened mullion to transom connections by externally placed brackets (Saatcioglu et al. 2017b). The tests demonstrated that the use of Guardian Coil can be partially successful and had the disadvantage of reduced esthetic appearance and functionality. Glass pane glazing improved performance with partial success obtained through connection strengthening. In general, the tests exhibited weaknesses in the connections as they were primarily designed for wind loads. Another series of tests were conducted on individual mullions with the aim of developing hardening techniques with steel plates, demonstrating the resulting enhancements in blast load capacity.

The research program at the University of Ottawa led to the development of a computer software, suitable for dynamic analysis of curtain wall components. Software RC-Blast performs non-linear SDOF dynamic analysis under blast loads (Jacques et al. 2013, Jacques 2014). Software CW-Blast performs SDOF analysis of curtain wall elements, including membrane analysis of glazed glass panes (Belec and Saatcioglu 2022). These computer

software were used to conduct analytical research, involving SDOF analysis of mullions and transoms, as well as the membrane analysis of glazed glass panes.

The last phase of curtain wall research at the University of Ottawa focused specifically on the connections. Curtain walls and single mullions were tested under simulated blast shock waves using the shock tube to investigate the behavior of mullion-transom and mullion-substrate connections (Nasseralshariati and Saatcioglu, 2023a). The latter series of tests consisted of mullions either rigidly connected to supporting reinforced concrete beams or connected through a damper material (rubber or springs). The experimental research was followed by numerical investigation using software LS-Dyna (Nasseralshariati and Saatcioglu, 2023b). It was concluded that the mullion-transom connections designed against wind loads, consisting of internally placed aluminum U-brackets, riveted on the mullions with inserted transoms, do not provide sufficient protection against strong blast loads. The use of internal steel U-brackets bolted on both the mullions and transoms perform well under blast loads. Strengthening existing weaker connections with externally placed L-brackets offers stronger connections as a blast retrofit strategy. The results also showed that steel L-brackets can be used to connect mullions to the supporting concrete substrate. When the brackets were secured to the top and bottom of the supporting concrete members, also connected to the flange of the mullions, they suffered excessive bending, indicating the need for stiffer brackets. This was more pronounced when the mullion was continuous over the supporting concrete element. In fact, the concrete element caused high bearing stresses on the aluminum mullion, having bent the brackets, crushing the mullions and resulting in bearing failure at the connection. The performance of the brackets and the connection showed a superior performance when the brackets were supported along the web of the mullion with attachments to the sides of the concrete element.

An important step in the connection design is the computation of design forces. While the numerical simulations provided excellent correlation with the test results and hence can be used to establish connection design forces, it may not be suitable for everyday design process. Instead, SDOF analysis may be a practical simplification for this purpose. The support

reactions for the mullions and transoms, established numerically by LS-Dyna FEM analysis, were validated against the results of SDOF analysis (Nasseralshariati and Saatcioglu, 2023b). A design procedure is outlined in the proceeding sections based on experimental observations and SDOF analysis.

5.2 SDOF Representation and Analysis Tools

SDOF dynamic analysis is a commonly used analysis technique in blast engineering. Blast-induced shock waves with short durations (in the order of millisecond) excite single structural and non-structural elements with short fundamental periods. Therefore, most analyses are conducted on single elements with distributed mass and a specific stiffness, subjected to uniformly distributed blast pressures. These elements can be transformed into a single spring-lumped-mass model and a concentrated time varying force. The resulting model can be analyzed as a SDOF system to compute mullion and transom support reactions. These reactions serve as connection design forces either at mullion-transom connections or at mullion substrate connections. The SDOF analysis is often conducted by available computer software. SBEDS (USACE 2009) and RC-Blast (Jacques 2014, Jacques et al. 2013) are examples of commonly used SDOF blast analysis software. A manual computation of support reactions can also be performed using design charts developed by the US Department of Defense (Uniform Facilities Code – UFC 2008). SDOF analysis for curtain wall elements is presented in the following sections.

5.2.1 SDOF Model and Transformation Factors

Mullions and transoms of a curtain wall system can be modelled as SDOF elements for dynamic analysis. Figure 5.1 shows a simply supported aluminum curtain wall element with stiffness “ k ” and distributed mass per unit length “ m ”. It also shows uniformly distributed pressure which varies with time “ $p(t)$ ”. This model can be transformed to a spring – single mass system with a concentrated time varying force, as also illustrated in Figure 5.1 by following the principles of conservation of energy and virtual work. This requires estimation of the deflected shape associated with the first mode response of the element. The shape function, “ $\phi(x)$ ” can be taken as the deflected shape under static uniformly distributed load. The kinetic

energy associated with the distributed mass can be set equal to the kinetic energy of the equivalent lumped mass system, from which the equivalent mass M_e can be derived. Similarly, the elastic flexural strain energy of the element can be set equal to the strain energy of the spring, from which the equivalent stiffness k_e can be derived. The equivalent concentrated force F_e can be derived using the principle of virtual work, by setting the virtual work done by the uniformly distributed pressure to that due to concentrated force. The resulting equivalent quantities can be expressed as shown below:

$$M_e = \int_0^L m(x)\phi^2(x)dx \quad (5-1)$$

$$k_e = \int_0^L EI(x)[\ddot{\phi}(x)]^2 dx \quad (5-2)$$

$$F_e(t) = \int_0^L p(t)\phi(x)dx \quad (5-3)$$

The governing SDOF equation of motion then becomes:

$$M_e \ddot{u}(t) + k_e u(t) = F_e(t) \quad (5-4)$$

The SDOF system can be analyzed to compute deformations and support reactions. The equivalent quantities for mass, stiffness and force can be obtained by applying transformation factors to the curtain wall element with distributed mass and distributed pressure to transform the element to a single mass concentrated force system with associated spring stiffness. This is shown below.

$$M_e = K_M mL \quad k_e = K_R k \quad F_e = K_L p(t)L \quad (5-5)$$

$$K_M mL \ddot{u}(t) + K_R k u(t) = K_L p(t)L \quad (5-6)$$

K_M , K_R , and K_L are transformation factors that can be computed as shown below.

$$K_M = \frac{M_e}{M_t} = \frac{\int_0^L m\phi(x)^2 dx}{mL} \quad (5-7)$$

$$K_R = \frac{k_e}{k} = \frac{\int_0^L EI\ddot{\phi}(x)^2 dx}{k} \quad (5-8)$$

$$K_L = \frac{F_e}{p(t)L} = \frac{\int_0^L p(t)\phi(x)^2 dx}{p(t)L} \quad (5-9)$$

The above quantities result in $K_M = 0.5$; $K_R = 0.64$; and $K_L = 0.64$. The K_R and K_L values are equal, as expected, since the spring stiffness is the force required to cause unit deflection and it is directly proportional to the load. Substituting $K_R = K_L$ in Equation 5-6 and dividing all terms by K_L ;

$$\frac{K_M}{K_L} mL\ddot{u}(t) + ku(t) = p(t)L \quad (5-10)$$

K_M/K_L is defined as the load-mass factor K_{LM} , which in this case is equal to $0.5/0.64 = 0.78$. The transformation factors change with the support conditions, as expected. Table 5.1 provides transformation factors for single elements with distributed mass, subjected to uniformly distributed pressure with different support conditions. These values can be used for mullion and transom analysis and include rotational restraint conditions at the supports.

The preceding paragraphs describe transformation of curtain wall elements to constant mass, single spring, and concentrated force model for SDOF formulation. When the elements experience non-linearity upon yielding, the transformation factors change as the shape function varies beyond the initial elastic phase and assume different shapes for different stages of loading. The same energy and work principles can be used for the inelastic range of deformations to develop transformation factors. Table 5.1 includes transformation factors for elements with different support conditions corresponding to elastic deformations, plastic (post-yield) deformations and elastic-plastic deformations (after the first or first set of yielding, but before the final yielding leading to the collapse mechanism).

Transom analysis requires an additional consideration because of its support conditions. They are attached to mullions with some flexibility. In this case, the SDOF analysis can still be employed by introducing springs at the supports. The resulting SDOF system has two springs that are connected in series in the direction of applied load. This is illustrated in Figure 5.2. The spring stiffness is expressed below in terms of the summation of flexibilities of two springs.

$$\frac{1}{k} = \frac{1}{k_1} + \frac{1}{k_2} \quad (5-11)$$

Where, k_1 is the bending stiffness of the transom and k_2 is the translational stiffness of the springs at the supports. The support translational spring stiffness can be established from the mullion stiffness. The same procedure may also be used in analyzing the performance of mullions supported with dampers or damping materials. In this case k_2 represents the stiffness of the damper material. It should be noted that the above procedure is straightforward as long as both k_1 and k_2 are in the elastic range. If either one experiences inelasticity, then the change in stiffness during response must be considered, necessitating step by step analysis, covering all stages of inelastic behaviour.

5.2.2. Resistance Functions and Determination of Stiffness “k”

Computation of the resistance function is a fundamental step for nonlinear SDOF analysis. The resistance function is defined as the force-displacement relationship for the SDOF spring under concentrated force. While the elastic bending stiffness can simply be obtained by using the sectional rigidity (EI), inelastic force-displacement relationship requires analysis of the element with specific length and support conditions. Often, the idealization of force-displacement relationship consisting of linear line segments provides sufficiently accurate stiffness values. For aluminum and steel curtain wall elements with simple supports a bilinear idealization can be used. In this case, the plastic capacity of the member is established at the formation of the mid-span hinge. Assuming constant resistance upon yielding, one can conveniently obtain a bi-linear relationship with the yield point at the intersection of the elastic branch and the post-yield region as illustrated in Figure 5.3(a). If the element has support fixity, multiple plastic hinges form prior to the formation of the collapse mechanism and associated plastic capacity of the element. In this case multi-linear segments of the force-displacement relationship need to be established. Figure 5.3(b) illustrates a tri-linear idealization of the resistance function for a curtain wall element with fixed-fixed supports at the ends. In this case, the first line segment extends up to a point where plastic hinges form near the fixed supports. Beyond this stage of loading, the element behaves as a simply supported beam with reduced stiffness, represented by the second line segment, until the mid-span starts hinging with

constant force resistance, represented by the third line segment. The third line segment ends when the mid-span hinge is fully formed and the collapse mechanism is formed. The resulting idealized resistance function defines the element stiffness during the many phases of inelastic response. Inelastic dynamic analysis of the SDOF system can then be conducted numerically while introducing the change in stiffness at each time step of the analysis. Typically, constant or linear acceleration step-by-step integration technique is used for such analysis.

5.2.3 Linear SDOF Dynamic Analysis

Once the mathematical model for a lumped mass-spring model is established, the maximum deflection of the system can be computed under static load and then multiplied by “Maximum Dynamic Load Factor - $(DLF)_{\max}$ ” to obtain a dynamic response ($u_{\max}$). Charts have been developed for $(DLF)_{\max}$ under different impulsive forcing functions. Figure 5.4 shows the variation of $(DLF)_{\max}$ as a function of t_d/T ratio for triangular and rectangular impulsive loads with t_d duration. The fundamental period of the SDOF system “ T ” can be computed from the natural frequency “ ω ” using equivalent mass (M_e) and equivalent stiffness (k_e) as shown below.

$$\omega = \sqrt{\frac{k_e}{M_e}} = \sqrt{\frac{K_L k}{K_M M_t}} = \sqrt{\frac{k}{K_{LM} M_t}} \quad (5-12)$$

$$T = \frac{2\pi}{\omega} \quad (5-13)$$

The maximum dynamic resistance “ R ” can be computed as the second term in the equation of motion.

$$R = k u_{\max} \quad (5-14)$$

The SDOF system is usually modelled such that the deflection “ u ” represents the maximum deflection of the element, from which stresses, moments and shears at the same location can be computed. However, support reactions cannot be computed directly from the element resistance, which is a function of deflection. Unlike members under static loading, the applied load is not the only action on the element that results in reactions. Because of the dynamic nature of the response, inertia forces react to the applied load, interfering with the magnitude of the support reactions. This is shown in Figure 5.5. While the applied pressure on the element may be distributed uniformly, the inertia force is a function of the acceleration,

which is a function of the deflected shape. Therefore, a free-body diagram with the applied pressure and the inertia force may have to be considered to solve the support reactions. The resulting support reactions, which will be used as connection design forces, become a function of the resistance “R” and the applied force “F” resulting from the applied pressure. The support reaction forces, computed as prescribed, are shown in Table 5.1.

5.2.4 Non-linear SDOF Dynamic Analysis

Inelastic dynamic analysis of a SDOF system can be conducted numerically while introducing the change in stiffness at each time step of the analysis. This results in the solution of the equation motion given in Eq. 5.4. Typically, constant, or linear acceleration step-by-step integration technique is used for such analysis through computer software implementing one of these numerical techniques. The US Department of Defense (Uniform Facilities Code – UFC 2008) developed charts that enable manual computation of dynamic response for elasto-plastic systems. If a bi-linear idealization of resistance function is available, as described in Section 5.2.2, then these charts can be used to determine maximum inelastic deflection of the element under dynamic forcing functions. Figure 5.6 depicts the UFC chart for members with bi-linear resistance function subjected to a triangular impulsive blast load. The horizontal axis for the chart is expressed in terms of t_d/T ratio as in the case of $(DLF)_{max}$ chart for elastic response given in Figure 5.4. The vertical axis gives displacement ductility ratio $\mu = u_m/u_y$ as the ratio of maximum displacement (u_m) to yield displacement (u_y). The chart consists of curves for different levels of R_m/F_1 where R_m is maximum resistance and F_1 is the maximum applied blast force. It can be used in conjunction with the transformation factors given in Table 5.1 for elasto-plastic response. The chart can be entered with a given t_d/T and R_m/F_1 ratios to determine the required ductility capacity (ductility demand) for the curtain wall element under investigation. Alternatively, they can be used for the design displacement capacity associated with the level of protection (LOP) required to establish maximum resistance R_m , which can then be used to compute support reactions as connection design forces using the equations in Table 5.1.

5.2.5 Available SDOF Dynamic Analysis Computer Software

Inelastic analysis of curtain wall elements (mullions and transoms) can be carried out using computer software that implements step-by-step integration procedure with appropriate member stiffness and applied load for each time increment. This technique is well known and it is used developing existing SDOF software. A review of two such software, i.e., RC-Blast (Jacques 2011, Jacques et al. 2013) and CW-Blast (Belec and Saatcioglu 2022), specifically developed for blast analysis at the University of Ottawa, are presented here as computerized analysis tools for curtain wall connection design.

RC-Blast: The program RC-Blast is written in Microsoft Visual Basic. It is intended for both practicing engineers and researchers with minimal hands-on training. The program is composed of SDOF analysis and structural analysis modules. The SDOF analysis module may be used to perform dynamic time-history analysis or generate pressure–impulse diagrams for any user defined SDOF system. The structural analysis module may be used to generate the load–deformation characteristics of one-way reinforced concrete elements for incorporation into the SDOF analysis module. While the generation of reinforced concrete resistance functions falls outside the scope of curtain wall analysis, load-deformation relationships for aluminum or steel mullions and transoms can be computed separately and input into the program. Dynamic time-history analysis can be performed by direct numerical integration of the dynamic equation of motion given in Eq. (5-4). There are no limitations on the user-defined applied pressure – time history, $P_r(t)$. It may be in the form of experimentally recorded free-field pressure-time relationship or blast parameters from other specialized software, such as CONWEP (Hyde 1992). Figure 5.7 shows screenshots of RC-Blast in SDOF analysis mode, illustrating input data and analysis output for a typical aluminum mullion analysis.

The program may also be used to develop Pressure-Impulse (PI) diagrams. A PI diagram is an iso-damage or iso-response contour plot consisting of a series of pressure–impulse combinations that generate the same level of structural response. By generating a PI diagram for a given structural element, and plotting specific pressure–impulse combinations corresponding to various anticipated explosive threats, the performance of a structure may be evaluated graphically. RC-Blast has the capability to develop pressure–impulse diagrams based

on the load–deformation characteristics of a given structural element. Points along the PI curves are generated by assuming a positive phase duration, t_d , and iterating on applied pressure, P_r , until the maximum SDOF displacement was equal to a specified target value. Each PI curve is composed of 13 points: the impulsive asymptote is calculated with a positive phase duration of 4% of the natural period, T_N , of the structure and the quasi-static asymptote with a positive phase duration of 1000% of T_N . The use of the aforementioned positive phase durations was found to adequately capture the quasi-static and impulse asymptotes with an acceptable level of accuracy for a wide range of SDOF systems. An automatic time step control algorithm was developed to ensure an acceptable level of accuracy for each iteration. Figure 5.8 shows typical PI diagrams generated by the software for a curtain wall mullion. Appendix E indicated the maximum mullion deflection and the relevant PI diagrams generated by RC-Blast program.

CW-Blast: Computer software was developed for analysis and design of curtain wall systems for Windows operating system. The software provides dynamic analysis of glazed glass panes and the supporting mullions and transoms. It utilizes the procedure described in the preceding sections. The dynamic equation of motion is solved using a step-by-step linear integration technique. The software provides analysis of curtain wall glass panes and mullions both in vertical and horizontal (transom) directions separately. It provides support reactions for the mullions that can be used for connection design, both for mullion to substrate connections, as well as mullion to transom connections. Figure 5.9 shows the cover page of the software with data and analysis options indicated. Accordingly, loading information can be specified as blast pressure-time relationship. Panel properties are entered in terms of the geometry and properties of glass panes. This data includes glass thickness, air gap for insulation, and protective film or interlayer properties. Panel analysis is conducted using two-way flexural action followed by membrane action of the broken glazed glass based on the membrane theory (Timoshenko 1940). The output includes support reactions of the glass pane, which can be used as distributed load on the supporting mullions. Tributary area for vertical mullions and transoms (as horizontal mullions) are defined with an option to consider two-way action with

trapezoidal load distribution. The support reactions for mullions can be either simple or fixed support. An option is provided for relaxing supports in out-of-plane direction for transom analysis to allow for the flexibility of supports provided by the mullions. Mullion data is entered as mullion properties where the cross-sectional and material properties for mullions are specified. Mullion analysis provides displacement-time history. Finally, a large number of output items are reported at the end, including support reactions and time history of results that can be exported as Excel files for plotting. Figure 5.10 and 5.11 show the above-mentioned input and output parameters. The transom support reaction analysis are conducted by using CW-Blast program and they are in Appendix D.

5.2.6 Validation of SDOF Analysis against Test Data and Numerical Analysis

The test data obtained from three curtain walls (CW1 through CW3) and nine mullions (M-C1 through M-C9) with different connections were available for maximum and residual mid-height deflections of mullions (Nasseralshariati and Saatcioglu 2023a). During the experimental phase, it was found that the glazed window and the steel sheets used do not experience considerable in-plane membrane action. Furthermore, the main focus of the current research was to study the behavior of the interior mullions for which the in-plane forces on either side would balance each other. Therefore, the design procedure developed ignores the effects of in-plane membrane action on connection design. SDOF analysis provides manual and computer computed deflections for maximum deflections values. The manual UFC charted solution provides maximum deflection without damping. The computer analysis obtained from RC-Blast provides displacement time histories but only the computed maximum displacement can be compared with the test data because of the change in mass during testing associated with the load application device. The total mass during tests includes the mass of the load application device, which separates from the mullions beyond the peak displacement. The change in mass during response cannot be handled by RC-Blast and hence the computed response beyond the peak cannot be compared with experimental deflections.

The first step in conducting SDOF analysis involves the computation of resistance functions for the mullions. The test mullions were either simply supported, as in the case of

tests CW1 through CW3 and M-C7 through M-C9, or fixed against rotation at the supports, as in the case of M-C1 through M-C6. Figure 5.12 illustrates the support conditions used during testing. These support conditions were used to develop bi-linear resistance functions for simply supported mullions, as illustrated in Fig. 5.3(a), and tri-linear resistance functions for fixed support mullions as shown in Fig. 5.3(b). The tri-linear resistance functions were then used to compute equivalent bi-linear resistance functions as the UFC charts were developed for bi-linear elasto-plastic systems.

M-C5 through M-C9 had damping materials at the supports. Their effect on mullion stiffness was included by adding the flexibilities of the support damping materials and the mullions as per Eq. 5-12. Tables 5.2 and 5.3 provide a summary of resistance function parameters for all the mullions, including their stiffness. Table 5.4 compares maximum mullion deflections recorded during tests with those computed with RC-Blast (step-by-step analysis with 2% of critical damping) and UFC charted solutions without damping, as described in Sections 5.2.5 and 5.2.4, respectively. The deflection comparisons show good agreement. It is noteworthy that there are inherent inaccuracies in the modelling of support conditions. It was observed during the tests that the L-brackets developed some bending, relaxing some of the initially introduced full fixity condition. Furthermore, lack of damping in the UFC charted solution may account for some of the discrepancies.

Additional comparisons of deflections were made against LS-Dyna FEM analysis results. Numerical analyses of all test specimens were conducted and reported with details elsewhere (Nasseralshariati and Saatcioglu 2023b). The numerically computed maximum mullion deflections are also included in Table 5.4 for comparison. They indicate that the SDOF analysis can be used for analysis and design of mullions.

An important outcome of SDOF analysis for connection design is the support reactions at the ends of mullions and transoms. The support reactions were not measured during the shock tube tests. However, numerical LS-Dyna analyses provided support reactions, which are compared with those computed with SDOF analysis. The mullions were connected to the substrate either through simple or fixed supports with corresponding rotational restraints. The

support reactions computed using the SDOF analysis approach were obtained from the expressions given in Table 5.1. The results are compared with those computed numerically in Table 5.5 and show excellent correlations for the mullions of curtain walls CW1 through CW3 with simple supports. The comparisons for mullions M-C1 through M-C4 with connections having L-brackets showed an average of 15% higher support reactions computed by the SDOF analysis relative to the numerical values. This is attributed to partial fixity of the supports obtained due to the bending of the L-brackets in the numerical analysis as opposed to full fixity assumed in the SDOF analysis. Similarly, mullions M-C5 through M-C9 showed 36% higher support reactions computed by the SDOF analysis. These mullions were equipped with rubber materials or springs to dampen the effects of blast loads, which had not been considered in the SDOF analysis. Indeed, tests of these mullions indicated that a large portion of the deflections recorded at their mid-heights were due to the compression of the support springs and associated rigid body movement in the direction of loading. The resistance of mullions increased to rigidly supported mullion level only after the springs are fully compressed. Neglecting the presence of support springs would result in higher support reactions as was the case in the SDOF analysis employed. Figure 5.13 shows mullions with and without spring dampers. They were subjected to approximately the same pressure-impulse combinations. Mullion M-C1 with rigid translational supports developed failure, whereas mullions M-C7 with support springs remained virtually elastic. When the applied load was increased by 40%, another mullion with support springs (M-C8) also developed failure after the support springs are completely compressed during loading. This indicates that the effect of supplementary damping devices at the supports should be modelled in the SDOF analysis by updating the mullion stiffness during response. This can be done by employing a step-by-step integration numerical procedure in the dynamic analysis.

While the support reactions for mullions can be used to design the connections between the mullions and the supporting substrate, the connections between transoms and mullions require the computation of the support reactions for transoms. This can also be achieved through SDOF dynamic analysis of transoms in much the same manner as for the mullions. An

important aspect of transom analysis is the fact that it is attached to mullions which have their own flexibilities. This should be considered as support flexibilities as shown in Figure 5.3. Computer software CW-Blast has support flexibilities incorporated in the analysis where a spring stiffness, equal to the stiffness of the mullions can be specified. Software CW-Blast was employed to generate support reactions for the three curtain walls tested and numerically analyzed using software LS-Dyna (Nasseralshariati and Saatcioglu 2023a, 2023b). Table 5.6 shows the transom support reaction comparison. The results indicates that the SDOF analysis can be used to compute connection design forces reasonably accurately.

5.3 Connection Design Procedure

Connection design recommendations are made separately for mullion-to-concrete substrate and mullion-to transom connections. The recommended design procedure is based on the computation of connection design forces through the SDOF analysis described in the previous section, followed by design and detailing requirements based on the experimental and numerical observations reported earlier and the computation of connection resistance. While numerous connection techniques are available in practice, the emphasis here is placed on steel brackets connected by rivets and bolts as their performance was investigated experimentally and numerically. However, the general design principles apply to the application of other mechanical and adhesive connections.

5.3.1 Mullion to Concrete Substrate Connections

Figure 5.14 illustrates examples of steel brackets used in practice for connecting curtain wall mullions to supporting concrete slabs or spandrel beams. The brackets are connected either through the mullion web to the edge of the supporting concrete element or through the mullion flange to the top and bottom of the concrete element. Similar L-brackets were selected in the experimental phase of current research to simulate the practice. The details of these brackets are shown in Figures 5.15 through 5.17 (Nasseralshariati and Saatcioglu 2023a). The same brackets were also investigated through a companion numerical investigation (Nasseralshariati and Saatcioglu 2023b). Based on the experimental and analytical observations the following potential failure modes are identified for connection design:

- i. Mullion failure due to bearing against concrete.
- ii. Shear failure of rivets.
- iii. Shear failure of bolts in concrete.
- iv. Axial forces in bolts.
- v. Potentially interaction of axial force, bending moment, and shear.

i. Mullion failure due to bearing against concrete: Bearing failure of mullions may occur when the mullion is connected to the top and bottom of the slab by means of L-brackets, which may allow the concrete to come in contact with the flange of the mullion. If the support reactions of the mullion are higher than the reactions associated with flexural failure of the mullion, and the brackets are not sufficiently rigid, the brackets may bend, resulting in complete bearing of the concrete slab against the aluminum. This type of failure was observed during the tests when the mullion was continuous over a concrete slab with reduced spans and increased support reactions. This is shown in Figure 5.18, which shows the bearing failure of aluminum upon bending of the brackets. This test was conducted under 72 kPa reflected pressure and 905 kPa-ms of reflected impulse. In a companion test specimen, when the brackets were supported at the edge of the slab, connected to the web of the mullion, the bearing area was increased, and bending of the brackets were prevented as the leg of the bracket attached to the slab was supported by the slab itself. The performance of this test specimen is shown in Figure 5.19, displaying no failure of the connection under the same applied blast load. Therefore, it may be preferable to connect the mullions to the edge of the slab with the other leg of the bracket riveted to the web unless the bracket stiffness is increased, or the connection forces are limited. The use of the same type of connection at the ends of a storey-high mullion, connected to the slabs at the top and bottom, did not show similar bearing failure. This is attributed to the increased mullion length and reduced flexural bending of the mullion. Indeed, this mullion developed compression web buckling in flexure at 28 kPa reflected pressure and 285 kPa-ms reflected impulse prior to developing connection failure. In this case the bending of the bracket legs is observed, as depicted in Figure 5.20, relaxing the fixity condition at the support but did not reach the critical level of connection force to induce bearing failure of the aluminum.

While the arrangement of L-brackets and their connections play important roles on bearing failure of aluminum as explained in the preceding paragraph, the rigidity of the brackets, as governed by the thickness of bracket legs is another parameter that dictates the failure load. The thickness of the bracket legs was 3.0 mm in the tests. The effect of bracket steel thickness was investigated numerically by increasing the thickness to 10 mm, 20 mm and 25 mm. The analysis was conducted under the same pressure-impulse combination as that used for the mullion that suffered bearing failure in the tests (72 kPa and 905 kPa-ms). The results of the numerical analysis shown in Figure 5.21 demonstrate that as the steel thickness in the brackets increased, the failure of the number of aluminum elements in the FEM mesh decreased along with the steel strains in the bracket. For the specific case considered in this analysis steel thickness of 25 mm resulted in the elimination of bearing failure while the brackets remained elastic (max. strain of 0.0016 less than the yield strain of 0.002). The results further indicate that when the brackets are attached to the edge of the slab the bracket steel thickness has to be increased substantially, whereas this is not necessary if the brackets are supported by the top and bottom of the slab, riveted to the mullions on the web. It should be noted that the bolts were intentionally oversized as reflected in the strain gauges values; so the bolt failure under axial loads were ignored. In the former case, it is recommended that the designer should check for the bending stresses in the bracket and ensure elastic behaviour.

ii. Shear failure of rivets: When the L-brackets are connected to the mullions through the use of rivets on the web, they provide bending resistance at the connection, which is provided by shear forces generated in each rivet. The shear force demand in each rivet caused by the bending of the connection region can be computed using the support moment, the distance of each rivet from the centre of rotation and the polar moment of inertia. This shear force demand should be checked against the rivet shear capacity increased by the dynamic increase factor of 1.1 for steel. Design forces obtained from the SDOF analysis based on curtain wall capacity shall be increased by a factor of 1.5 as a safety margin, as per CSA S850. A sample calculation is presented below for one of the test mullions and its connection, as illustrated in Figure 5.22.

Maximum moment taking into account the experiment support fixity and dynamic/elastic resistance ratio:

$$M_{max} = 2.3 \text{ kN.m}$$

$$\text{Rivet Shear Resistance} \times \text{Dynamic Increase Factor: } 9.5 \text{ kN} \times 1.1 = 10.5 \text{ kN}$$

	X (mm)	Y (mm)
Center of Rotation	0	0
Rivet 1	-15	45
Rivet 2	15	45
Rivet 3	-15	15
Rivet 4	15	15
Rivet 5	-15	-15
Rivet 6	15	-15
Rivet 7	-15	-45
Rivet 8	15	-45

The distance for rivets 1, 2, 7, 8: $C = \sqrt{15^2 \times 45^2} = 47.4 \text{ mm}$

The distance for rivets 3, 4, 5, 6: $C = \sqrt{15^2 \times 15^2} = 21.2 \text{ mm}$

Polar moment of inertia: $(\sum c^2) = (4 \times 47.4^2) + (4 \times 21.2^2) = 10800 \text{ mm}^2$

Shear due to the moment for rivets 1, 2, 7, 8: $V_{rivet} = \frac{2.3 \text{ kN.m} \times 10^6 \times 47.4 \text{ mm}}{10800 \text{ mm}^2} = 10.1 \text{ kN}$

Overstress rivet utilization: $\frac{\text{Shear demand}}{\text{Shear capacity}} = \frac{10.1 \text{ kN}}{10.5 \text{ kN}} = 0.96 < 1 \text{ OK}$

Shear due to the moment for rivets 3, 4, 5, 6: $\frac{2.3 \text{ kN.m} \times 10^6 \times 21.1 \text{ mm}}{10800 \text{ mm}^2} = 4.5 \text{ kN}$

Overstress rivet ratio: $\frac{\text{Shear demand}}{\text{Shear capacity}} = \frac{4.5 \text{ kN}}{10.5 \text{ kN}} = 0.43 < 1 \text{ OK}$

The above procedure can be used to ensure sufficient shear capacity in the rivets for bending resistance at the mullion-concrete substrate connection.

iii. Shear failure of bolts in concrete: L-brackets that are connected to concrete slab at the top and bottom are secured by means of embedded bolts. Moment restraints developed at the ends of mullions are provided by shear forces that developed in the bolts. Figure 5.23

illustrates shear force components associated with the mullion support moment, M . In addition, the bolts provide resistance to direct shear caused by support reaction, R . Total shear force demand on each bolt can then be computed by adding these two components, ie., shear due to moment resistance and the direct shear associated with support reaction. An example of bolt shear calculation is shown below for a test mullion.

Moment at the end of the mullion due to blast loads: $M_{max} = 2.3 \text{ kN.m}$

Support reaction at the support due to the blast load: $R = 58 \text{ kN}$

The shear portion for each bracket (two brackets in total): $\frac{58 \text{ kN}}{2} = 29 \text{ kN}$

Dynamic resistance of each bolt: Bolt shear resistance $\times$ Dynamic Increase Factor:

$V_{cap} = 38.7 \text{ kN} \times 1.1 = 42.5 \text{ kN}$

Top bracket $V_{bolt} = \frac{\frac{\text{Max Moment}}{\text{Concrete Depth}} + \text{Support Reaction}}{\text{Number of Bolts}} = \frac{\frac{2.3 \text{ kN.m}}{0.2 \text{ m}}}{2} + \frac{29 \text{ kN}}{2} = 20.5 \text{ kN}$

Shear demand $20.5 \text{ kN} < \text{Shear Capacity } 42.5 \text{ kN}$ OK 48% utilization

Bottom bracket $V_{bolt} = \frac{\frac{\text{Max Moment}}{\text{Concrete Depth}} + \text{Support Reaction}}{\text{Number of Bolts}} = \frac{\frac{2.3 \text{ kN.m}}{0.2 \text{ m}}}{1} + \frac{29 \text{ kN}}{1} = 40.5 \text{ kN}$

Shear demand $40.5 \text{ kN} < \text{Shear Capacity } 42.5 \text{ kN}$ OK 95% utilization

5.3.2 Mullion to Transom Connections

Horizontal transoms are attached to vertical mullions through U-brackets that are riveted to the mullion at transom connection locations. They are connected by inserting hollow transom section to the internal U-bracket as illustrated in Figure 5.24. When the test specimens were assembled in the laboratory, additional 4 rivets were used to secure the transom to the internal U-brackets mechanically. This is shown in Figure 5.24(b). The rivets that connect the U-brackets to the mullions are often designed to resist wind loads but may have to be strengthened for blast-resistance. The transom support reaction analysis is presented in Appendix D. These rivets are subjected to direct shear stresses during blast loading and may have to be checked against their shear capacities. Sample calculations for 4 curtain wall samples considered in the current investigation, shown in Figure 5.24, are illustrated below.

CW1:

Total number of rivets: 4 Transom support reaction: 23.50 kN

Rivet Shear Resistance $\times$ Dynamic Increase Factor: $9.5 \text{ kN} \times 1.1 = \mathbf{10.5 \text{ kN}}$

Shear per rivet: $23.5 \text{ kN} / 4 \text{ rivets} = 6 \text{ kN} < 10.5 \text{ kN}$ OK utilization 57%

CW2:

Total number of rivets: $4 + 3 = 7$ Transom support reaction: 20.46 kN

Rivet Shear Resistance $\times$ Dynamic Increase Factor: $9.5 \text{ kN} \times 1.1 = 10.5 \text{ kN}$

Shear per rivet: $20.5 \text{ kN} / 7 \text{ rivets} = 3 \text{ kN} < 10.5 \text{ kN}$ OK utilization 29%

CW3:

Total number of rivets: $4 + 3 = 7$ Transom support reaction: 33.18 kN

Rivet Shear Resistance $\times$ Dynamic Increase Factor: $9.5 \text{ kN} \times 1.1 = 10.5 \text{ kN}$

Shear per rivet: $33.18 \text{ kN} / 7 \text{ rivets} = 4.7 \text{ kN} < 10.5 \text{ kN}$ OK utilization 45%

CW3-NB:

Total number of rivets: 4 Transom support reaction: 33.18 kN

Rivet Shear Resistance $\times$ Dynamic Increase Factor: $9.5 \text{ kN} \times 1.1 = 10.5 \text{ kN}$

Shear per rivet: $33.18 \text{ kN} / 4 \text{ rivets} = 8.3 \text{ kN} < 10.5 \text{ kN}$ OK utilization 79%

5.3.3 Strengthening/Retrofitting Existing Curtain Wall Connections

The majority of existing curtain walls are designed for wind loads and are deficient against blast loads. The connection design procedures outlined in this paper are equally applicable to existing curtain wall connections. As demonstrated in the experimental and numerical phases of the current investigation (Nasseralshariati and Saatcioglu 2023a and 2023b), strengthening of transom-mullion connections can be done either by replacing existing deficient U-brackets in the connections with steel U-brackets with sufficient number of rivets for enhanced shear resistance, or more conveniently by adding external L-brackets to the connections and securing them mechanically as illustrated in Figure 5.25. Similarly, the steel L-brackets used for connecting mullions to the supporting substrate as illustrated in Figures 5.14 to 5.16 can be replaced or strengthened by means of L-brackets with sufficient number of connecting bolts and rivets for increased connection shear resistance.

5.3.4 Use of Supplementary Damping Devices at Mullion Substrate Connections

One of the connection types investigated for the curtain walls considered in the current investigation was the use of supplementary damping devices at mullion-substrate (supporting element) connections. A number of mullion tests were conducted with materials having different stiffness values for damping, i.e, different spring combinations and rubber material as proof of concept. It was observed experimentally, and substantiated numerically that the use of damping devices at the connections reduce or prevent damage to curtain walls (Nasseralshariati and Saatcioglu 2023a and 2023b). Based on these observations, the following steps are proposed for the design of damping devices for mullion-substrate connections. Like most design procedures, the proposed procedure is an iterative procedure:

- Select a material with an axial rigidity that is comparable to flexural rigidity of mullions that can be installed at the connections. It is preferable to select a material that remains elastic during response to blast loads to ensure the recovery of curtain wall displacements at the connections after the event while protecting excessive damage to the mullions.
- Select a device length (gap between the substrate and the mullion for installation) that limits mullion bending to design level of protection when it is fully compressed. It should be noted that upon the full compression of the damping material the mullion performs as having rigid translational supports.
- Perform SDOF analysis of the mullions using the combined stiffness of the dampers and the mullions by using total flexibility computed by summing the flexibilities of the dampers and the mullion as indicated in Figure 5.2.
- If the SDOF analysis conducted under the given threat level results in a tolerable level of protection for the mullions, install the selected damping material with due considerations given to proper installation and serviceability of the system before and after the blast loading. Otherwise, change the stiffness and/or the length of the damping device and repeat the above steps.

5.4 Conclusions

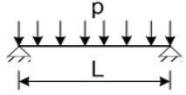
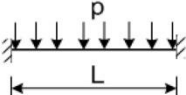
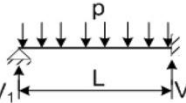
Design procedures were developed and recommended for connections of curtain walls against blast loads. The design procedures were developed based on experimental and numerical observations for mullion-to-substrate and mullion-to-transom connections. The following conclusions can be drawn based on the connection design procedures recommended for blast-resistant curtain walls.

- The design of connections between vertical mullions and the supporting reinforced concrete elements (slabs or spandrel beams), as well as the design of transom-mullion connections can be performed by computing support reactions under dynamic blast loads. This can be done through SDOF analysis. SDOF can be conducted manually by using the relevant UFC charts or through computer software utilizing analytical or numerical techniques.
- The SDOF analysis procedure presented for curtain wall connection design gives deflections and design forces that correlate well with those obtained during simulated blast tests and numerical investigations.
- Mullion-transom connections can be designed by providing steel U-brackets on the mullions secured by sufficient number of rivets, and by inserting transoms over the U-brackets, also secured by means of rivets. The total shear capacity of the rivets should be ensured to provide protection against shear demand under blast loads as computed by SDOF analysis.
- Strengthening of existing transom-mullion connections can be implemented conveniently by adding steel angle brackets, secured on the attached elements by means of rivets having sufficient shear capacity to overcome blast shear demands obtained from SDOF analysis. However, the use of rivets in flexure-critical mullion regions should be done with care to avoid damaging of the critical mullion positive moment region. The use of adhesives to connect the brackets at the connections may have limited capacity and should be implemented with care.
- Blast-resistant mullion-substrate connections can be designed using steel angle brackets either attached to the top and the bottom of the concrete element and the mullion flange

or to the edge of the concrete element and the mullion web. The former method requires sufficiently rigid brackets to remain elastic under the connection forces to avoid inelastic deformations and subsequent bearing of the mullions against concrete, which may potentially crush the aluminum. Hence, the latter procedure is recommended whenever practically possible. The same connection techniques can be implemented for strengthening and retrofitting existing connections with sufficiently strong rivet shear capacity.

- The use of supplementary damping materials between the mullions and the supporting substrates control damage to the mullions when subjected to blast loads. These materials can be designed using SDOF analysis with due considerations given to their stiffness relative to that of the mullions and the gap between the mullions and the supports to control the translational displacement at the supports.

Table 5.1: Transformation factors

k: Spring constant I: Moment of inertia E: Elastic modulus		F = pL p: Peak uniform blast pressure per length R: Resistance			M _s : Negative moment at support R _m : Maximum resistance For members with uniform mass	
Loading diagram	Strain range	Load factor, K _L	Mass factor, K _M	Load-mass factor, K _{LM}	Spring constant k	Dynamic Reaction V
	Elastic	0.64	0.50	0.78	$384EI/5L^3$	$0.39R+0.11F$
	Plastic	0.50	0.33	0.66	0	$0.38R_m+0.12F$
	Elastic	0.53	0.41	0.77	$384EI/L^3$	$0.36R+0.14F$
	Elastic-plastic	0.64	0.50	0.78	$384EI/5L^3$	$0.39R+0.11F$
	Plastic	0.50	0.33	0.66	0	$0.38R_m+0.12F$
	Elastic	0.58	0.45	0.78	$185EI/L^3$	$V_1=0.26R+0.12F$ $V_2=0.43R+0.19F$
	Elastic-plastic	0.64	0.50	0.78	$384EI/5L^3$	$V=0.39R+0.11F\pm M_s/L$
	Plastic	0.50	0.33	0.66	0	$V=0.39R_m+0.12F\pm M_s/L$

Adopted from Biggs (1964)

Table 5.2: Calculated values for mullion resistance functions with simple supports

Test Specimen	Elastic Yield Resistance (kN)	Yield Displacement (mm)	Stiffness (N/mm)	Elastic Support Reaction (kN)	Plastic Support Reaction (kN)
CW1	70	26	2,643	139.45	141.66
CW2	230	12.6	18,337	106.2	111.03
CW3	230	12.6	18,337	148.82	150.54
M-C7	31.3	26	1,242	45.97	54.04
M-C8	31.3	24	1,287	58.33	67.25
M-C9	31.3	25	1,273	45.87	51.35

Table 5.3: Calculated values for mullions resistance function with fixed supports

Test Specimen	Elastic Yield Resistance (kN)	Equivalent Yield Displacement (mm)	Equivalent Stiffness (N/mm)	Elastic Support Reaction (kN)	Plastic Support Reaction (kN)
M-C1	68.72	20	3,420	66.56	68.13
M-C2	68.72	20	3,384	82.53	86.72
M-C3	136.2	5	27,240	64.29	66.40
M-C4	136.2	5	27,240	62.56	64.57
M-C5	136.2	5	27,240	64.00	65.47
M-C6	136.2	5	27,240	65.18	67.32

Table 5.4: Comparisons of experimental and analytical maximum mid-span deflections

Test Specimen	Experimental (mm)	UFC Charts (mm)	RC-Blast (mm)	Numerical (mm)
CW1	351	468	461	326
CW2	16	13	13	10
CW3-Shot 1	167	170	168	145
CW3-Shot 2	133	-	-	152
CW3-Shot 3	132	-	-	168
M-C1	85	58	56	72
M-C2	37	78	80	84
M-C3	33	6	5	38
M-C4	13	5	5	5
M-C5	21	5	5	9
M-C6	36	6	5	23
M-C7	134	143	141	127
M-C8	172	216	219	153
M-C9	192	125	124	178

Table 5.5: Comparison of mullion support reactions; UFC-chart versus LS- DYNA

Model	UFC chart (kN)		Numerical (kN)
	Elastic Range	Plastic Range	
<i>CW1</i>	139.45	141.66	137.10
<i>CW2</i>	106.20	110.03	109.24
<i>CW3</i>	148.82	150.54	149.77
<i>M-C1</i>	66.56	68.13	57.84
<i>M-C2</i>	85.53	86.72	75.66
<i>M-C3</i>	64.29	66.40	52.14
<i>M-C4</i>	62.56	64.57	57.20
<i>M-C5</i>	64.00	65.47	51.82
<i>M-C6</i>	65.18	67.32	47.94
<i>M-C7</i>	45.97	54.00	31.72
<i>M-C8</i>	58.33	67.25	46.46
<i>M-C9</i>	45.86	51.35	30.50

Table 5.6: Comparisons of experimental, numerical and CW-Blast results for transoms

	CW-Blast (kN)		Numerical (kN)
	<i>Elastic Range</i>	<i>Plastic Range</i>	
<i>CW1</i>	17.49	19.08	23.50
<i>CW2</i>	14.41	15.47	20.46
<i>CW3</i>	47.13	-	33.18

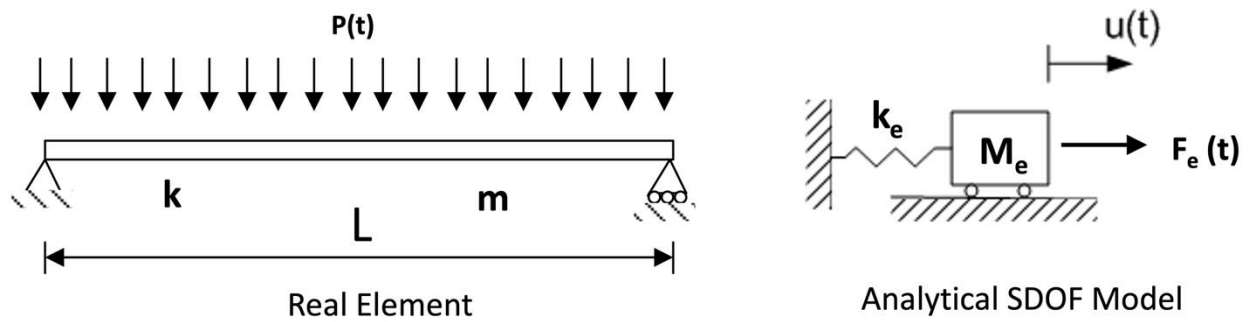


Figure 5.1: Modeling of a single degree of freedom element

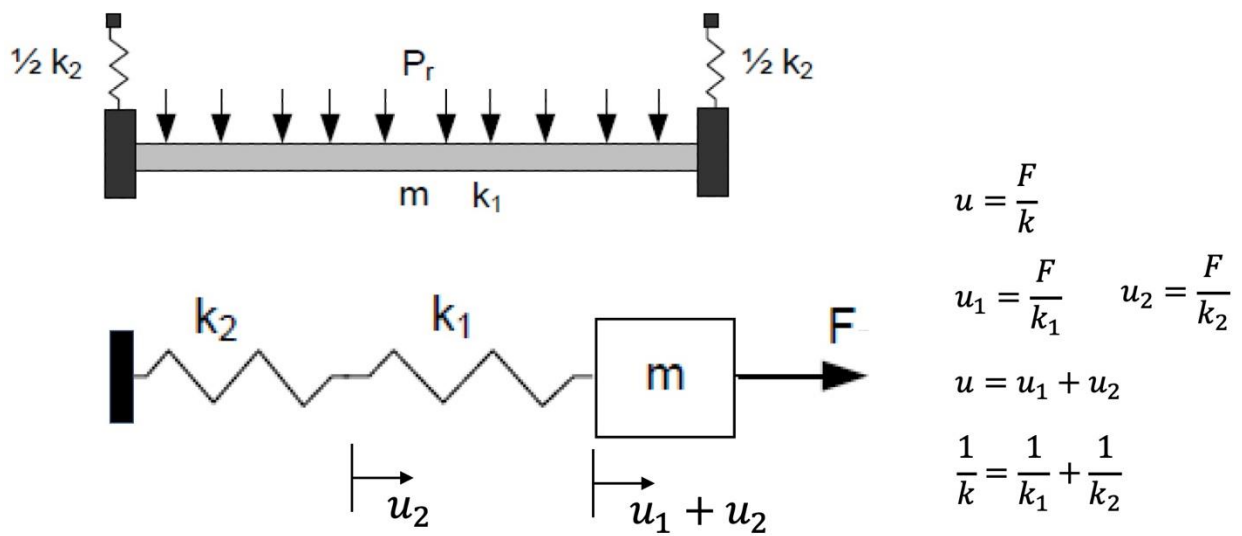
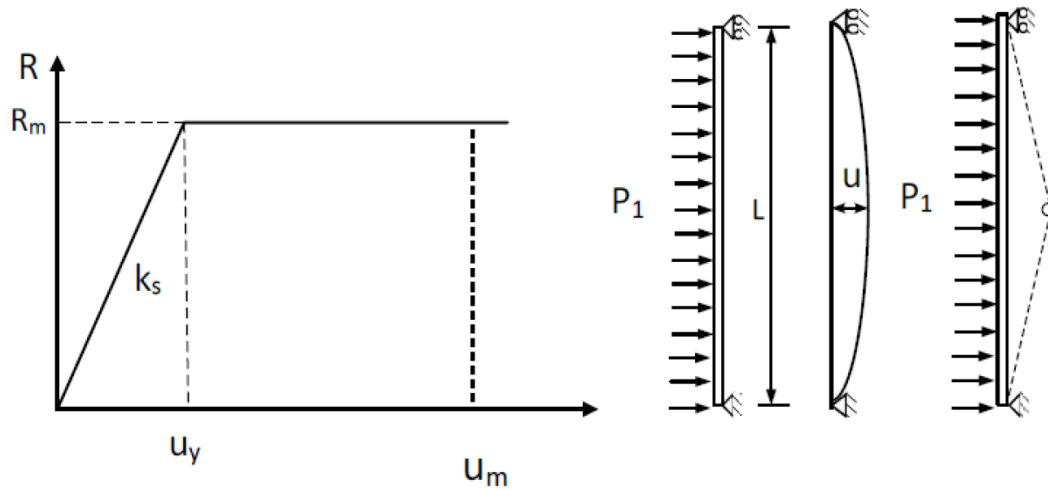
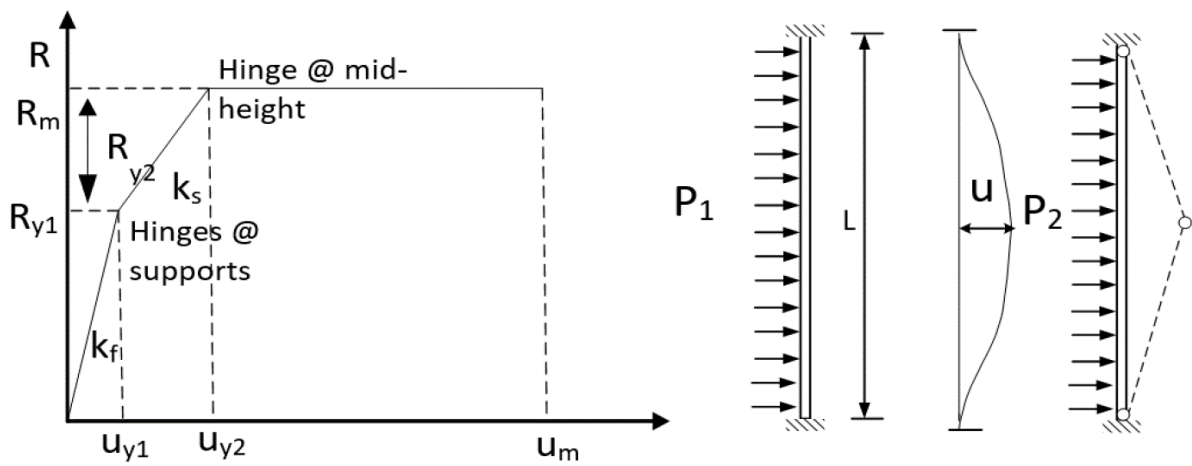


Figure 5.2: Springs in series



a) Bilinear resistance function for a simply supported element



b) Trilinear resistance function for an element fully fixed at the ends.

Figure 5.3: Resistance functions for mullions and transoms

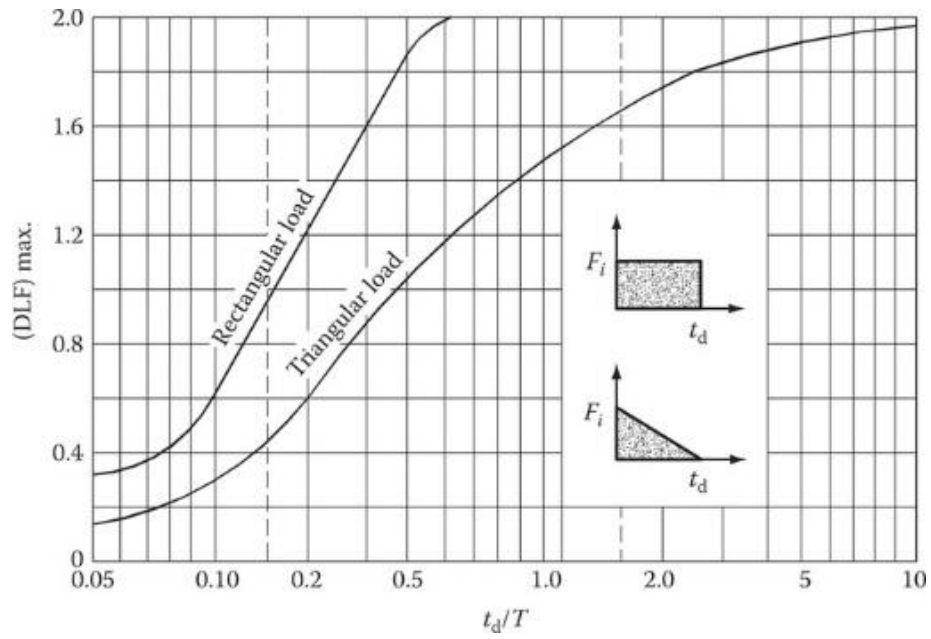


Figure 5.4: Variation of maximum Dynamic Load Factor with the ratio of duration of load to the period of the element

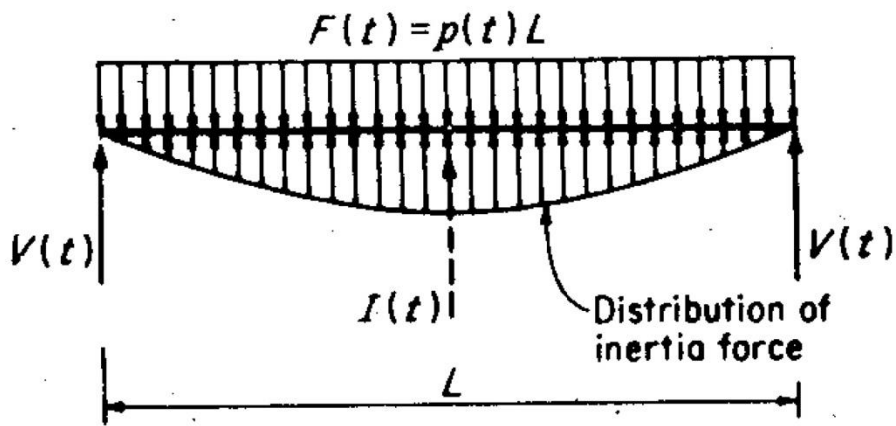


Figure 5.5: Distribution of blast pressure and inertia resistance

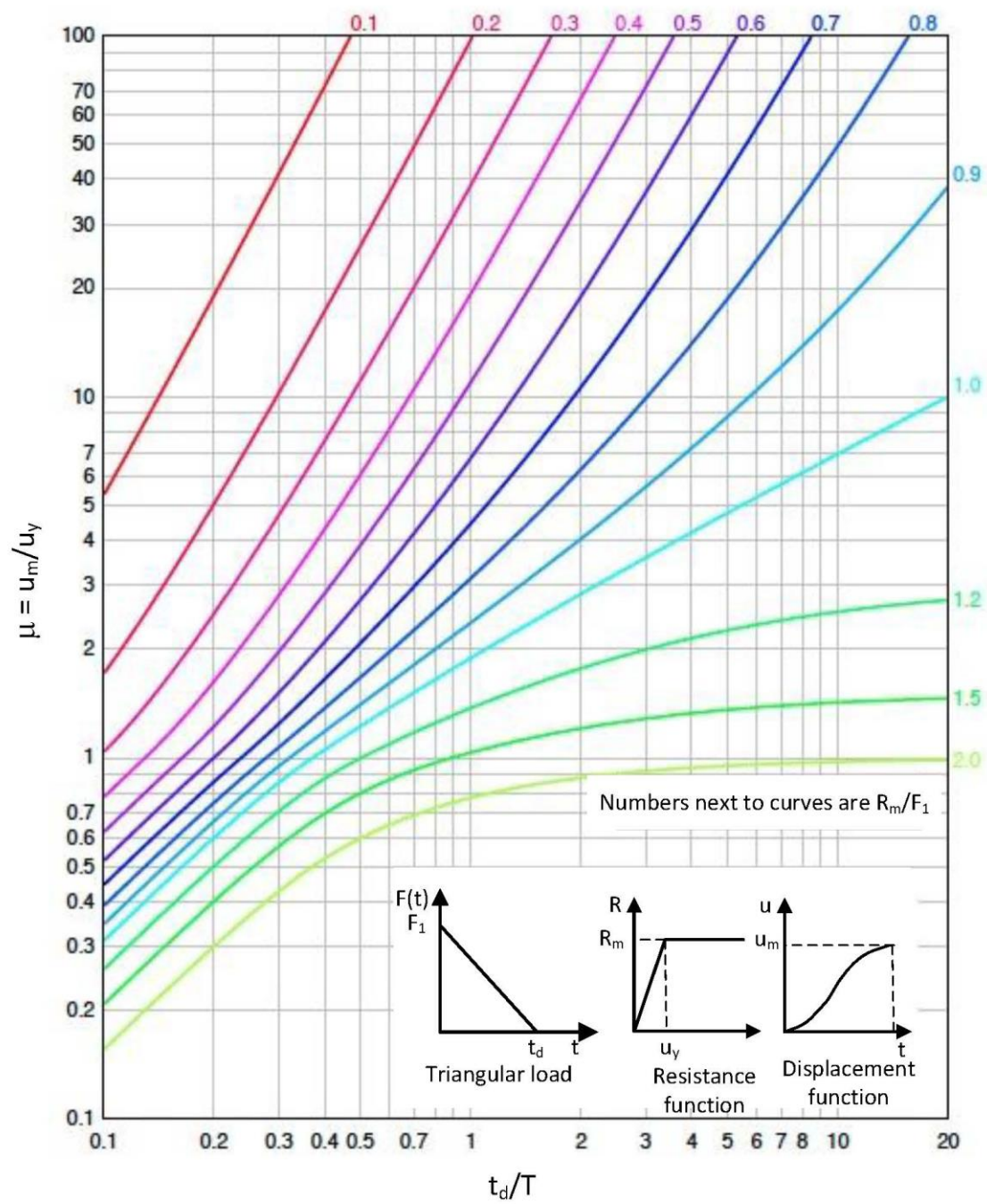


Figure 5.6: UFC chart for elasto-plastic response

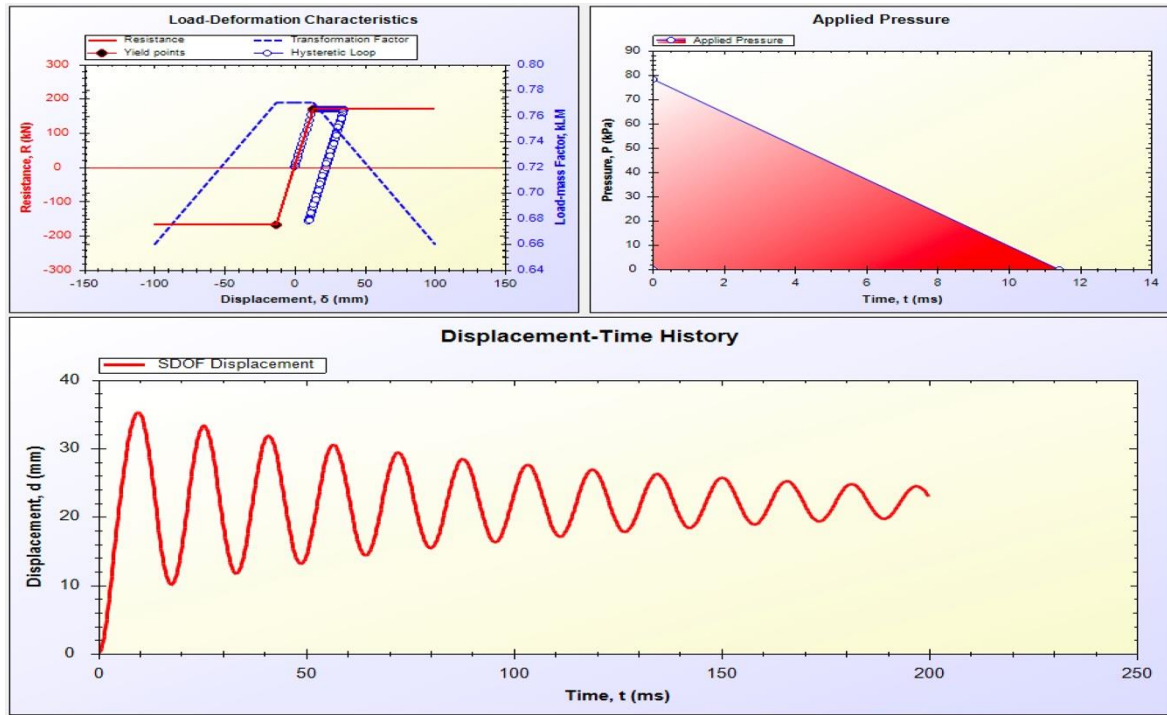


Figure 5.7: RC-Blast input and output screens

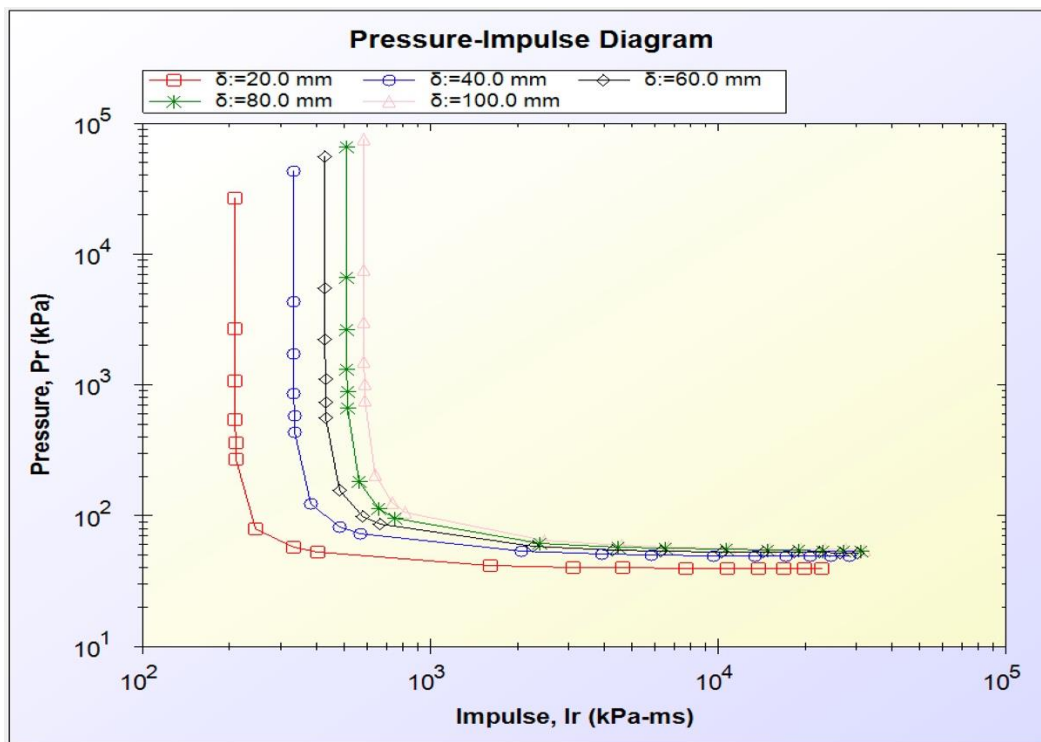


Figure 5.8: Sample P-I diagrams for different levels of displacement, δ .



Figure 5.9: Software CW-BLAST

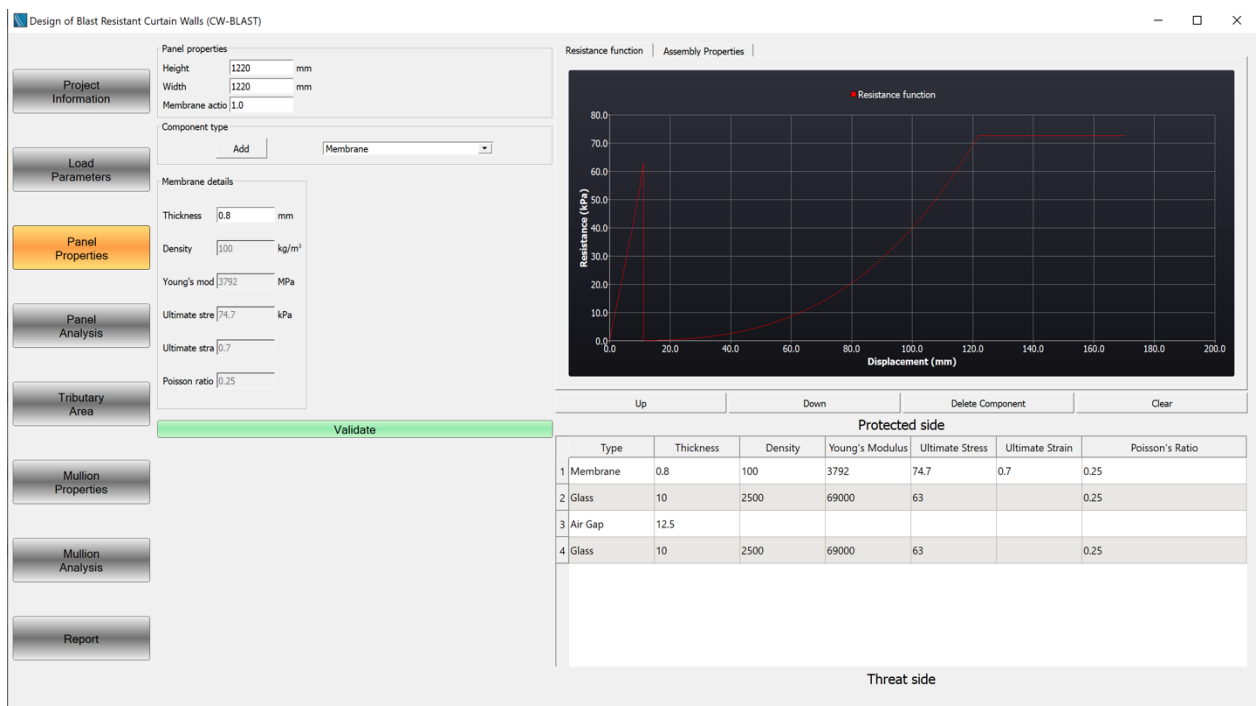


Figure 5.10: Input parameters for computation of panel resistance function

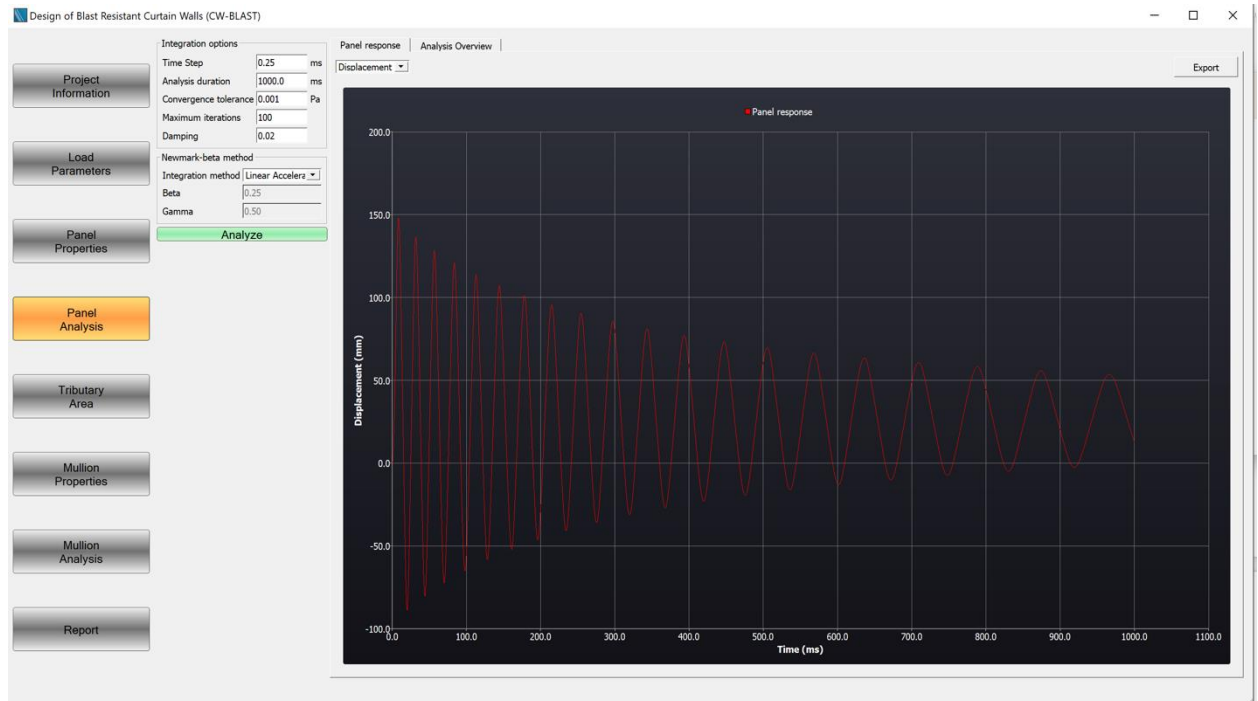


Figure 5.11: Analysis results – Software CW-BLAST



a) Simple support



b) Fixed support

Figure 5.12: Supports provided for test specimens



(a) Support are restrained against translation in the direction of loading (MC-1)



(b) Supports with spring dampers (MC-7)



(c) Specimen with damper springs (MC-8) loaded with higher load



(d) Supports in (a)

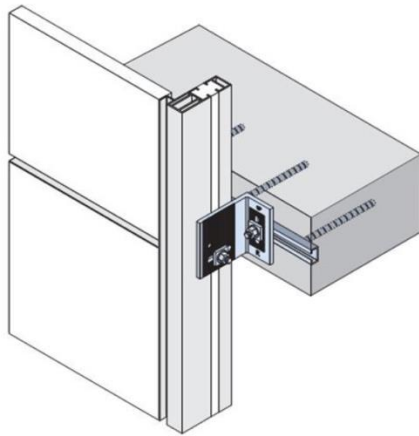


(e) Supports in (b)

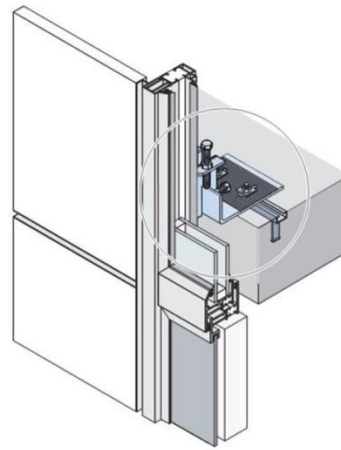


(f) Support at the end of testing

Figure 5.13: Mullion supports with and without spring dampers



(a) Connection to the slab edge



(b) Connection to top of slab

Figure 5.14: Mullion to reinforced concrete slab connections.



Mullion connected to top and bottom slabs



Brackets at the top slab edge connected to either side of the mullion web



Brackets at the bottom slab edge connected to either side of the mullion web

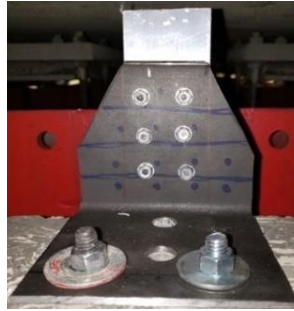
Figure 5.15: Mullion connected to top and bottom slabs through L-brackets attached to the web.



Mullion connected to top and bottom slabs



Brackets at top and bottom of slab
at the top end of mullion



Brackets at top and bottom of slab
at the bottom end of mullion

Figure 5.16: Mullion connected to top and bottom slabs through L-brackets attached to the flange.



Mullion continuous over



Brackets above and below slab
connected to mullion flange



Brackets at slab edge connected to
either side of the mullion web

Figure 5.17: Mullion continuous over a slab, extending to stories above and below, connected to the slab either at the slab edge or at the top and bottom of the slab.

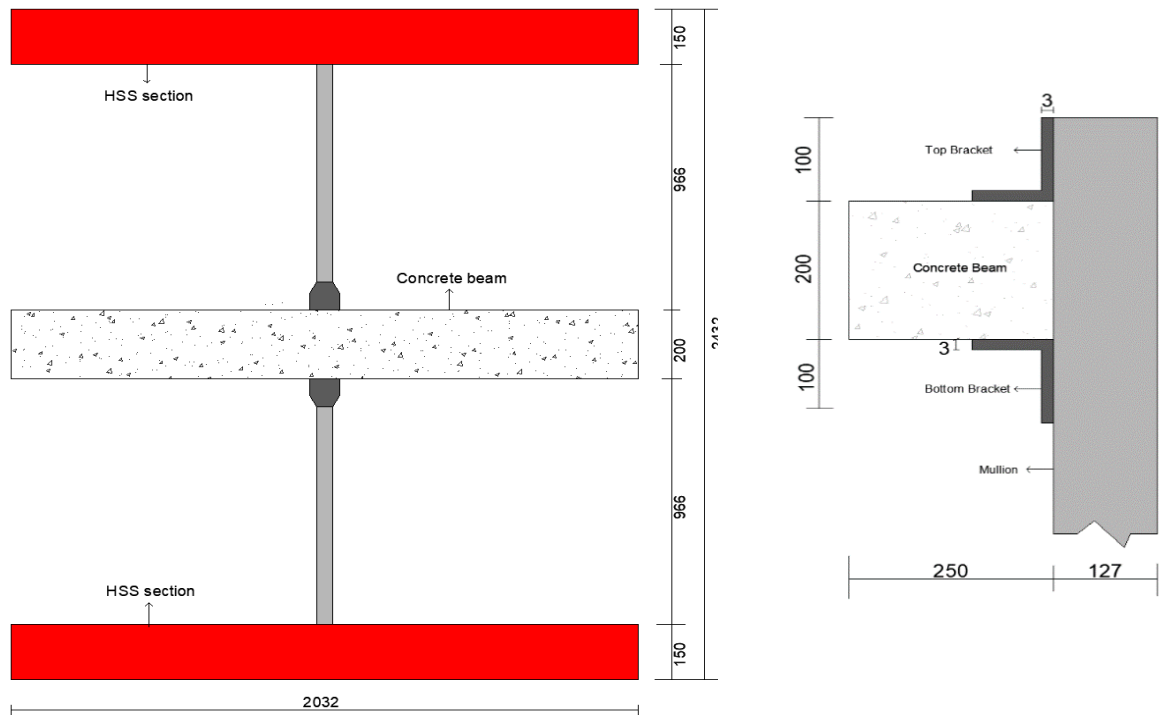


Figure 5.18: Bearing failure of a mullion when bearing against concrete – connection to top and bottom of slab.



Mullion prior to testing



Top span



Bottom span



Mullion after the test

Figure 5.19: Mullion connected to the edge of the slab – no bearing failure.



Bending of brackets at the ends

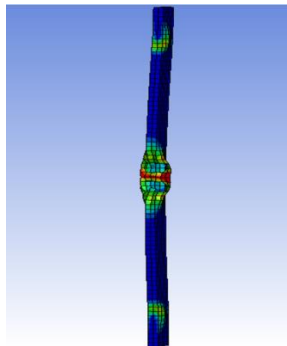


Web buckling

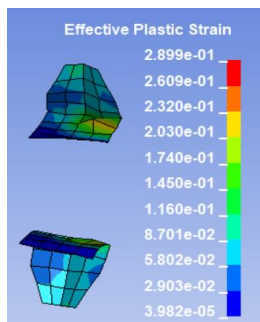


Flange buckling and failure

Figure 5.20: A story-long mullion connected to concrete slabs at the top and bottom of the slabs.

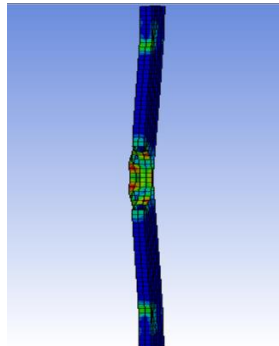


Element failed: 33

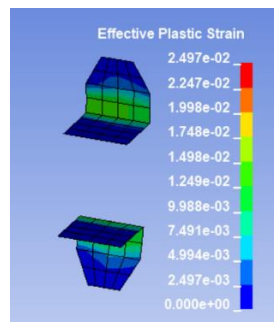


Max. strain: 0.2890

(a) Thickness: 3 mm

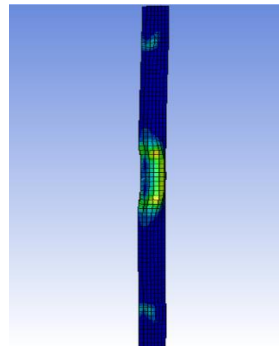


Elements failed: 14

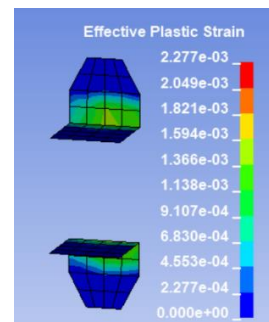


Max. strain: 0.0249

(b) Thickness: 10 mm

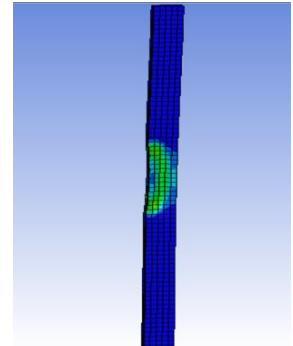


Elements failed: 2

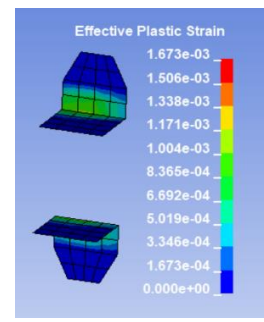


Max. strain: 0.0022

(c) Thickness: 20 mm



Elements failed: 0



Max. strain: 0.0016

(d) Thickness: 25 mm

Figure 5.21: Effect of bracket rigidity as governed by steel leg thickness.

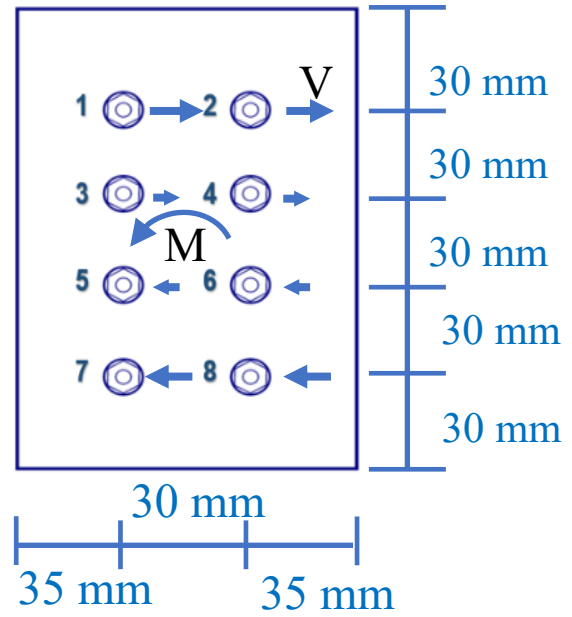
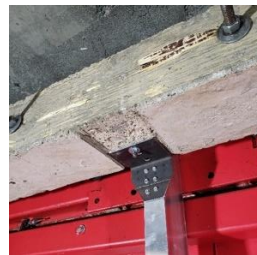


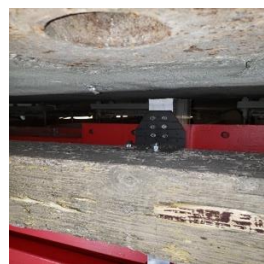
Figure 5.22: Shear in rivets resisting support moment.



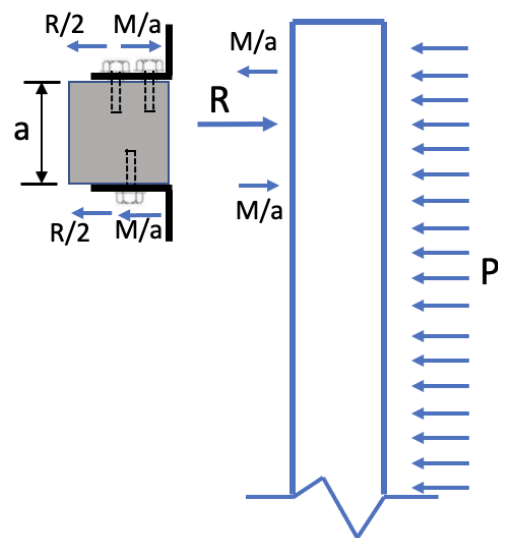
Mullion connected to top and bottom of slab



Bottom of slab



Top of slab



Forces on the bolts

Figure 5.23: Bolt shear



(a) U-bracket riveted on mullion



(b) Transom secured to U-bracket by 4 rivets

Figure 5.24: Assembly of transom-to-mullion connection.



CW1: Connection with internal U-brackets having 4 rivets, strengthened externally by epoxy glued aluminum brackets, which got detached during blast, without contributing to strength.



CW2: Steel hardened mullions and connection with internal U-brackets having 4 rivets, strengthened externally by riveted aluminum brackets.



CW3: Steel hardened mullions and transoms connected with internal U-brackets having 4 rivets, strengthened externally by riveted steel brackets.



CW3-NB: Connection with internal U-brackets attached with 4 rivets. This curtain wall did not have any external strengthening.

Figure 5.25: Mullion-to-transom connections considered.

CHAPTER 6. Summary and Conclusions

6.1 Summary

Combined experimental and analytical research was carried out to investigate improvements that can be attained at the connections of blast-resistant curtain walls. The experimental tests included full-size curtain walls and single mullions with retrofitted connections. A total of 12 tests were conducted on blast-resistant curtain wall components using the University of Ottawa Shock Tube facilities. The first phase focused on three full-size curtain walls built with hollow aluminum (6061-T6) sections. The mullion section size was 125 mm $\times$ 51 mm (5" $\times$ 2") with 2.44 m (96") height, and the transom section size was 76 mm $\times$ 51 mm (3" $\times$ 2") with 1.93 m (76") height. The second phase involved tests of four individual vertical mullions connected to reinforced concrete beams/slabs. The third phase included tests of five individual mullions connected to concrete beam/slab through damping materials consisting of springs and XRD rubber. Different retrofit techniques were implemented to strengthen the connections between mullions and transoms, as well as the mullions and the supporting concrete members. Different levels of reflected pressure-reflected impulse combinations were selected based on the estimated capacities of the retrofit methodologies implemented.

Numerical investigation, involving LS-DYNA analysis was conducted in parallel. Twelve numerical models were generated to validate the models against the test data. An additional 9 more models were also generated to conduct parametric investigation. The parameters considered included: i) the use of glass and film layer instead of the representative steel sheets used in the experimental program; ii) retrofit of the internal assembly of connectors between the mullions and transoms; iii) retrofitting weak internal connections by placing external steel bracket on top and bottom of transoms; iv) increasing the thickness of brackets used to connect mullions to concrete slabs; and v) improving the level of protection by using damper materials between the mullion and concrete slabs.

The analytical investigation was expanded to demonstrate the applicability of SDOF analysis to curtain wall connection design. The SDOF analysis was employed either by using

UFC-charts or the two software (RC-Blast and CW-Blast) developed at the University of Ottawa. This required the construction of resistance functions, assessment of elastic and inelastic support reactions for mullions and transoms, and the validation of the procedure against experimental and numerical results obtained in the earlier two phases of research. Design procedures were developed for mullion-to-transom and mullion to substrate connections.

6.2 Conclusions

The conclusions derived from experimental, numerical, and analytical phases of research are presented in different sub-sections, as follows below:

6.2.1 Conclusions from Experimental Phase of Research:

- Commercially produced curtain walls designed for wind load effects may not have sufficient mullion-to-transom connections when subjected to blast loads. Blast-resistant connections can be produced with the use internal steel U-brackets bolted on to the adjoining elements (mullions and transoms).
- The connections of curtain walls without considerations given to blast loads can be strengthened with the use of externally placed L-brackets at the connections to improve their blast resistance. These brackets, preferably made from steel, should not be bolted on the mullions at flexure critical locations to avoid premature rupturing of the mullions. The use of epoxy adhesion provides strengthening of the connections for low levels of blast loads. Under higher blast loads, mechanical connections should be used along with hardened mullions and transoms. Curtain walls hardened with steel plates, with strengthened connections using bolted L-brackets provide superior blast protection.
- Steel plates secured on the aluminum sections can develop fully composite behavior until after significant inelastic deformations are developed, followed by the buckling of the plates between the self-tapping screws used for securing the retrofit plates. The self-drilling metal screws used for retrofitting, including the attachment of the corner angles

performed well during the tests. No yielding and failure were observed until after the curtain wall was excessively damaged due to other causes, and even then, the damage to the screws were limited to a few locations.

- Vertical mullions of blast-resistant curtain walls can be connected to the reinforced concrete floor slabs (or spandrel beams) by means of steel brackets. The brackets may be oriented to be bolted on the front edge of the concrete member where the bolts primarily provide tensile resistance, or they can be bolted on top and/or bottom of the concrete element, in which case the bolts primarily provide shear resistance. The orientation of brackets depends on the location of vertical mullion (intermediate or edge) and the accessibility of concrete elements.
- The brackets must have sufficient rigidity so that they do not bend in the process of transferring support reactions to the concrete element. Where the support reactions are high and when the connection is made to the top and bottom of the slab edge, as was observed in Mullion M-C3 of the experimental program, the brackets may bend and place the burden on direct bearing of the aluminum on concrete. This may result in bearing failure of the mullion. In such cases, it is advisable to support the mullions from face of the slab with brackets bolted on to the sides (web) of the mullion.
- The use elastic dampers at mullion-substrate connections can reduce blast load effects and control damage to curtain walls. Selection of the stiffness and geometry of the dampers are paramount for proper functioning of the connections. The damper stiffness and the travel distance during response should be considered in design. The XRD rubber pads selected for connection protection offered limited protection because of insufficient stiffness and thickness.
- The use of elastic springs as dampers at the connections of curtain walls with the substrate offer blast protection. They should be designed for the design basis threat to possess sufficiently high stiffness and travel distance.

6.2.2 Conclusions from Numerical Phase of Research:

- The behaviour of curtain walls and single mullions retrofitted with different techniques can be properly predicated by LS-DYNA software. All the connection retrofitting techniques considered can be modeled accurately, and the anchorage can be idealized by using tied-contacts. Furthermore, the blast load can be applied either on the curtain wall exterior surface or through a load application device, both providing similar results.
- The numerical investigation proved that the use of steel sheets to simulate glass panes provides reliable results as representatives of curtain walls with glass panes. The simulations showed insignificant differences in deflections and support reactions between the two sets of analysis. However, less deflection was observed in the model for glass + film due to the high elasticity of the film layer.
- No failure was observed when steel inner U-brackets are used, which numerically indicated the effectiveness of proposed connection retrofit approach. Using steel U-brackets anchored to mullions and transoms can keep the curtain wall components together without any detachment, whereas the transoms with weak U-brackets (as designed for wind loads) show detachment of the members indicating the vulnerability of commonly used connections in real-life practices.
- Externally placed side brackets have insignificant effect on the strength of mullion-to-transoms connections so long as the inner U-brackets maintain their integrity. In this case the side brackets move along with the other components, without attracting much forces and stresses during response. Analysis without the side brackets resulted in similar response as that with the brackets. When both inner steel U-angles and externally placed side brackets are used, the transom support reactions are mostly resisted by the interior U-brackets.
- Steel plates riveted on aluminum sections can provide fully composite performance until after significant inelastic deformations are developed. However, the aluminum mullion that forms the core suffers failure at mid-height.

- Steel L-brackets placed externally on the top and bottom of transoms, anchored to the mullions, provide significant resistance to blast loads when inner U-angles are designed for wind. When such connection hardening technique is used, it is advisable to also harden the mullions and transoms by means of steel plates so that the attachment of external L-brackets do not damage the aluminum mullions in their critical regions.
- The use of steel L-brackets for connecting mullions to concrete substrate provides sufficient connection strength under blast loads. In such applications it is recommended to secure the brackets on the slab edge (rather than top and bottom surfaces of the slab), with brackets bolted on the web of the mullion. The bracket leg attached to the edge of the slab increases the bearing surface area for the mullion, preventing the crushing of aluminum. Moreover, the bracket leg attached to the mullion web provides additional strengthening to the mullion web, which also contributes towards the prevention of mullion crushing.
- Insufficient thickness and rigidity of L-brackets lead to the bending of the bracket legs, softening the rigid connections. The use of thicker steel brackets resulted less element elimination in the analysis on the tension side of the mullion, which indicated further strengthening of this region. However, significant local buckling of the compression side of mullion was observed.
- Using springs and XRD rubber materials at mullion-substrate connections resulted in reduced support rotations and increased level of protection against blast loads. These damper materials prevented failure of the mullion when compared with companion mullions without the dampers. The XRD rubber was not able to restore support displacements and acted like a memory foam. However, they provided better stress distribution in the mullion when compared with the use of springs.

6.2.3 Conclusions from SDOF Analysis Phase:

- The design of connections between vertical mullions and the supporting reinforced concrete elements (slabs or spandrel beams), as well as the design of transom-mullion connections can be performed by computing support reactions under dynamic blast

loads. This can be done through SDOF analysis. SDOF can be conducted manually by using the relevant UFC charts or through computer software utilizing analytical or numerical techniques.

- The SDOF analysis procedure presented for curtain wall connection design gives deflections and design forces that correlate well with those obtained during simulated blast tests and numerical investigations.
- Mullion-transom connections can be designed by providing steel U-brackets on the mullions secured by sufficient number of rivets, and by inserting transoms over the U-brackets, also secured by means of rivets. The total shear capacity of the rivets should be ensured to provide protection against shear demand under blast loads as computed by SDOF analysis.
- Strengthening of existing transom-mullion connections can be implemented conveniently by adding steel angle brackets, secured on the attached elements by means of rivets having sufficient shear capacity to overcome blast shear demands obtained from SDOF analysis. However, the use of rivets in flexure-critical mullion regions should be done with care to avoid damaging of the critical mullion positive moment region. The use of adhesives to connect the brackets at the connections may have limited capacity and should be implemented with care.
- Blast-resistant mullion-substrate connections can be designed using steel angle brackets either attached to the top and the bottom of the concrete element and the mullion flange or to the edge of the concrete element and the mullion web. The former method requires sufficiently rigid brackets to remain elastic under the connection forces to avoid inelastic deformations and subsequent bearing of the mullions against concrete, which may potentially crush the aluminum. Hence, the latter procedure is recommended whenever practically possible. The same connection techniques can be implemented for strengthening and retrofitting existing connections with sufficiently strong rivet shear capacity.

- The use of supplementary damping materials between the mullions and the supporting substrates control damage to the mullions when subjected to blast loads. These materials can be designed using SDOF analysis with due considerations given to their stiffness relative to that of the mullions and the gap between the mullions and the supports to control the translational displacement at the supports.

6.3 Recommendations for Future Research

The research project presented in this thesis focused on generating information and developing design procedure for selected types of steel anchorages for use in curtain wall design. The curtain walls that were simulated and investigated in the research project were representative of the curtain walls provided by one local manufacturer with internal U-Brackets used to connect the mullions to the transoms. Furthermore, the generic L-brackets selected for mullion to substrate connections, though representative of the ones used in practice, were of specific type. It is recommended that other curtain walls with different types of connections be tested and numerically investigated to expand the recommendations made. Furthermore, the complete curtain wall samples tested were few in numbers (three) and they were built with steel sheets as representative of glazed glass panes of equivalent mass and properties. It is recommended that more tests be conducted, preferably using glass panes. The scope of the research can also be expanded to blast-resistant curtain walls with steel mullions and transoms.

One of the unique aspects of the connections to concrete substrates investigated was the use of damping materials at the connections. Though the concept was proven with the few select specimens, it is recommended that additional experimental and numerical work be carried out using different damping materials with different stiffness and connection gaps for the installation of such devices. This will provide much needed additional data to expand the concept of connection dampers. This recommendation can be expanded to include inelastic damper materials, with sufficient elastic range to be effective for most threat scenarios with post elastic range for protection against increased levels of threat.

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Shock Tube Test Data for Full-size Curtain Walls and Single Mullions

Appendix A contains experimental results observed during tests of 12 full-size curtain walls and mullions tested with their own specific configuration. A brief description of each test is provided here, followed by the supplementary figures including specific failure observations, strain gauges, displacement-time history, and mullion displacements. A brief presentation of these results and detailed discussions are presented in Chapter 3. It also presents the collected data during the different shots of the experimental testing. This includes: (i) mullion and transom failure mode, (ii) section deformation, (iii) spring elastic behavior, and (iii) XRD rubber performance and stress-strain curve. In this appendix, full-size curtain walls and single mullion are mentioned with CW and M-C. Also, the numbers of tests are the same as proceeding chapters.

The blast testing of curtain walls was conducted at the Shock Tube Facility of the University of Ottawa's Blast Research Laboratory. A shock wave inside a shock tube may be generated by the buildup of high pressures which cause diaphragm(s) to burst and a shock wave to propagate through the shock tube.

The instrumentation used during testing consisted of a Data Acquisition system (DAS), pressure sensors, Linear Variable Displacement Transducers (LVDTs), two high-speed cameras, and the camera tracking software. The pressure and displacement transducers are connected to a computer located away from the shock tube for remote operation. The reflected pressure captured by the pressure sensors and the displacements measured by the LVDTs are transferred to DAS for processing. In addition, the bolts cast with concrete beams were instrumented by installing strain gauges 120 ohm to provide data after applying the blast load.

A.1: Test CW1

The CW1 was built with hollow aluminum (6061-T6) sections, which consisted of two vertical mullions, and three horizontal transoms. All sections were unhardened and it was subjected to 50 kPa reflected pressure. The mullions and transoms were connected to each other by means of U-angle steel connectors. In fact, using U-angle steel connector was one of the retrofitting techniques considered for full-size curtain walls. Although both vertical mullions suffered from rupturing on the tension side and buckling on the compression side, the steel connectors were able to hold the sections together. Gluing the side brackets could not develop sufficient composite action.



a) Support rotation



b) Failure at simple support

Figure A.1: Observation at simple support in CW1 test



Figure A.2: Steel sheet deformation with no rivet failure after CW1 test



Figure A.3: Close-up failure observation at mid-height of CW1 after testing

A.2: Test CW2

The CW2 was built similarly to the previous curtain wall, however, only vertical mullions were hardened in the second curtain wall by anchoring two L-shaped steel plates on the sections. The horizontal mullions were not hardened, but the mullion connections were strengthened using the L-shaped aluminum angles, and they were bolted to the mullions and transoms. The CW2 was subjected to a pressure-impulse combination of 36 kPa – 270 kPa-ms. This blast load resulted in the failure of the horizontal mullion when it broke into two pieces. The connections, strengthened by bolted aluminum angles, performed well and did not suffer any damage. The failure of the horizontal mullion induced increased membrane action in the steel sheet, which pulled the vertical mullions as evidenced by their deformed shape after the test.



a) Raptured middle transom



b) Rapture effect on transom connection

Figure A.4: Failure observations of CW2 after testing

A.3: Test CW3

CW3 was benefited from all retrofit approaches. Likewise, all its sections were hardened utilizing two L-shape steel plates anchored on the tension side of mullions and transoms. The connections were also strengthened by steel angles. CW3 was subjected to three blast shots as follows: 46 kPa, 50 kPa, and 61 kPa reflected pressure. Due to the first shot, the damage was limited to local buckling of the aluminum mullion and the steel plates at mid-height. Otherwise, the hardening technique performed well, without any distress in the connecting angles. After the second shot, Cracks were observed in the vertical mullions at mid-height and the L-shape retrofitting plates were slightly fractured. The connection angles remained in tack. The third shot was meant to assess the failure mode under increased threat levels. It caused the failure of the vertical mullions by rupturing on the tension side and buckling on the compression side.



a) Mullion residual deflection



b) Support rotation

Figure A.5: Test observation of CW3 after first shot



a) In tack anchored steel side brackets



b) Local tension Buckling

Figure A.6: Test observation of CW3 after second shot



a) Middle transom deflection



b) The core-aluminum failure

Figure A.7: Test observation of CW3 after third shot

A.4: Test M-C1

Specimen MC-1 was a single vertical mullion connected at the ends to reinforced concrete slabs at the top and bottom, representing a curtain wall mullion spanning over a storey height. It was connected to the slabs by means of steel brackets. The mullion was subjected to 22 kPa reflected pressure. The specimen failed by rupturing of the mullion at its critical mid-height location. The mullion to concrete connections remained in tack during the test, without any sign of distress. The strain gauges on the bolts embedded in concrete recorded maximum strain of 0.1%, which is significantly below the yield strain of 0.47%.

Schematic strain gauges locations

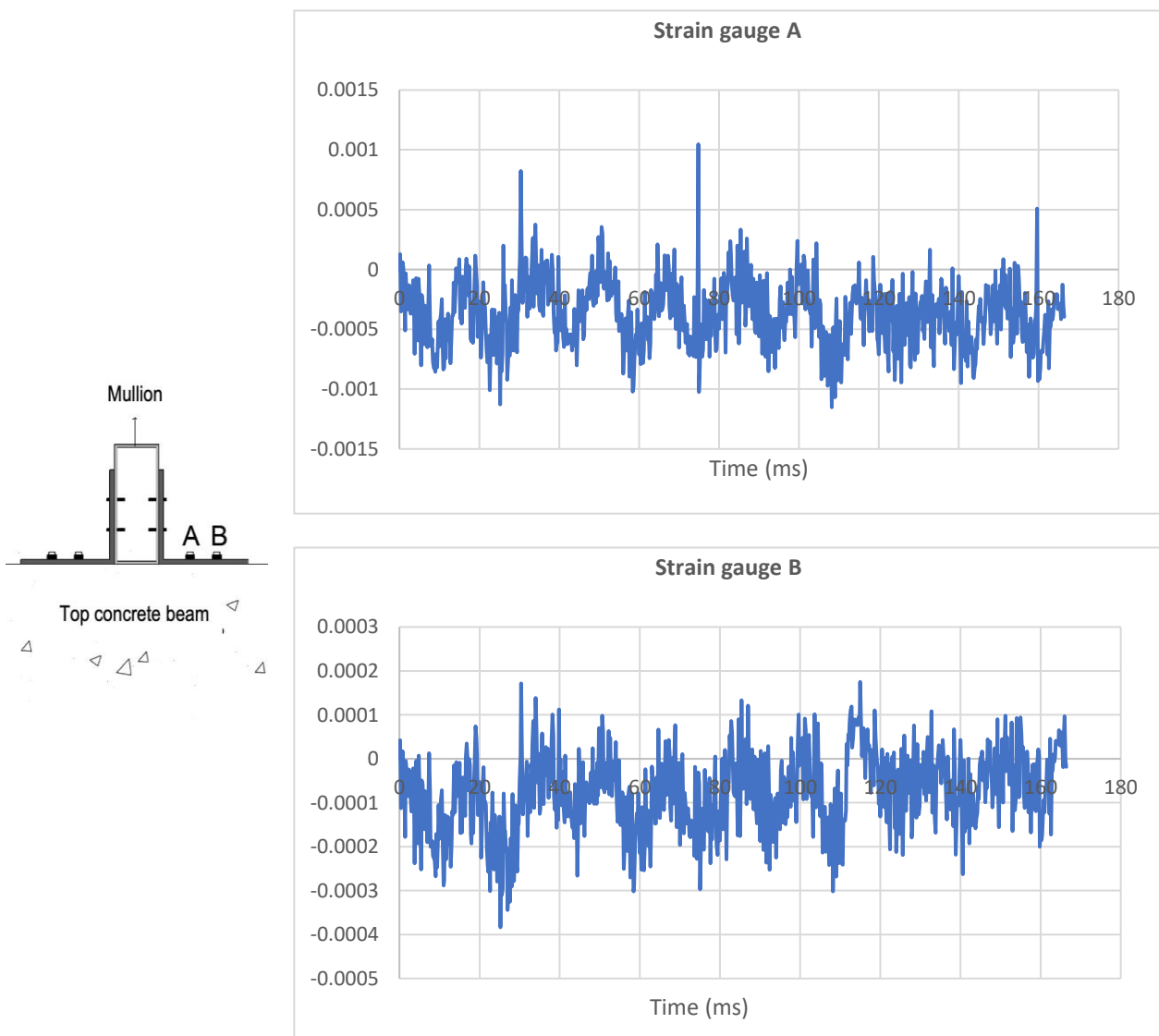


Figure A.8: The strain gauge data for M-C1 test

A.5: Test M-C2

Specimen MC-2 was a single vertical mullion connected at the ends to reinforced concrete slabs, representing a curtain wall mullion spanning over a storey height. It was connected to the slab by means of steel brackets. The mullion was subjected to 28 kPa reflected pressure, and suffered from local buckling of the mullion in the critical flexural region at mid-height. The connections remained in tack, although slight bending of the brackets was observed at the bottom of the top slab and at the top of the bottom slab.

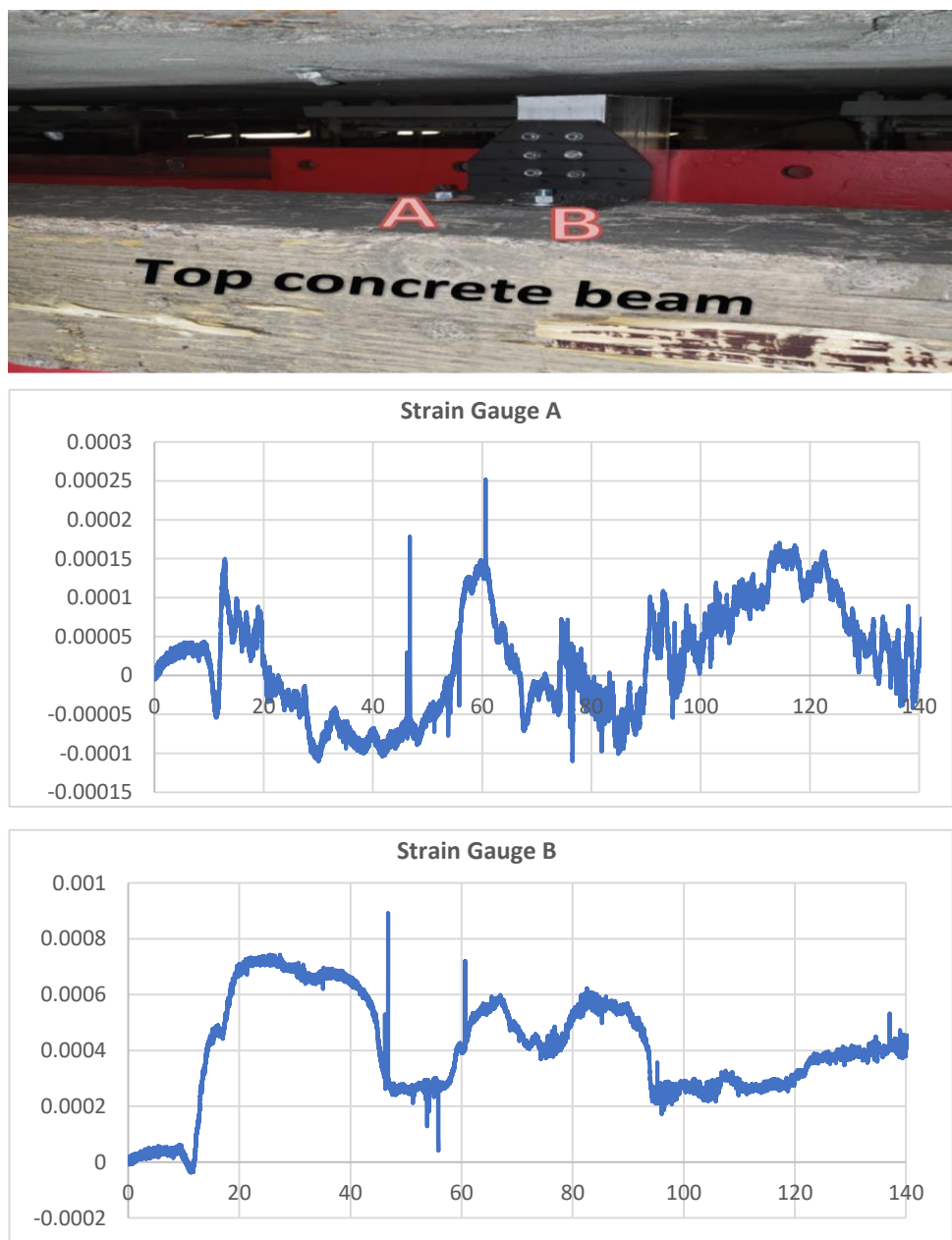


Figure A.9: The strain gauge data for M-C2 test

A.6: Test M-C3

Specimen MC-3 was a single vertical mullion simulating continuous mullion over a reinforced concrete slab. It was connected to the slab by means of steel brackets. The mullion was subjected to 72 kPa reflected pressure, and suffered from connection failure when the brackets were bent excessively, reducing the original 90-degree angle to approximately 45 degrees. The concrete slab bared against the mullion flange, crushing the aluminum mullion. Bending of the brackets strained the bolts that were used to connect the mullion excessively.



Figure A.10: The strain gauge data for M-C3 test



a) In tack fixed support



c) Close-up of failure

Figure A.11: Test observation of M-C3 after third shot

A.7: Test M-C4

MC-4 was a single vertical mullion simulating a continuous mullion over a reinforced concrete slab. It was connected to the slab by means of steel brackets that were bolted on the front face of the slab by means of 10-mm bolts that had been cast in concrete. It was subjected to the two-blast shots, which the second shot generated 70 kPa reflected pressure. M-C4 suffered local compression buckling at mid-height of the upper and lower segments. It indicated great performance, without any damage to the mullion and the bracket.

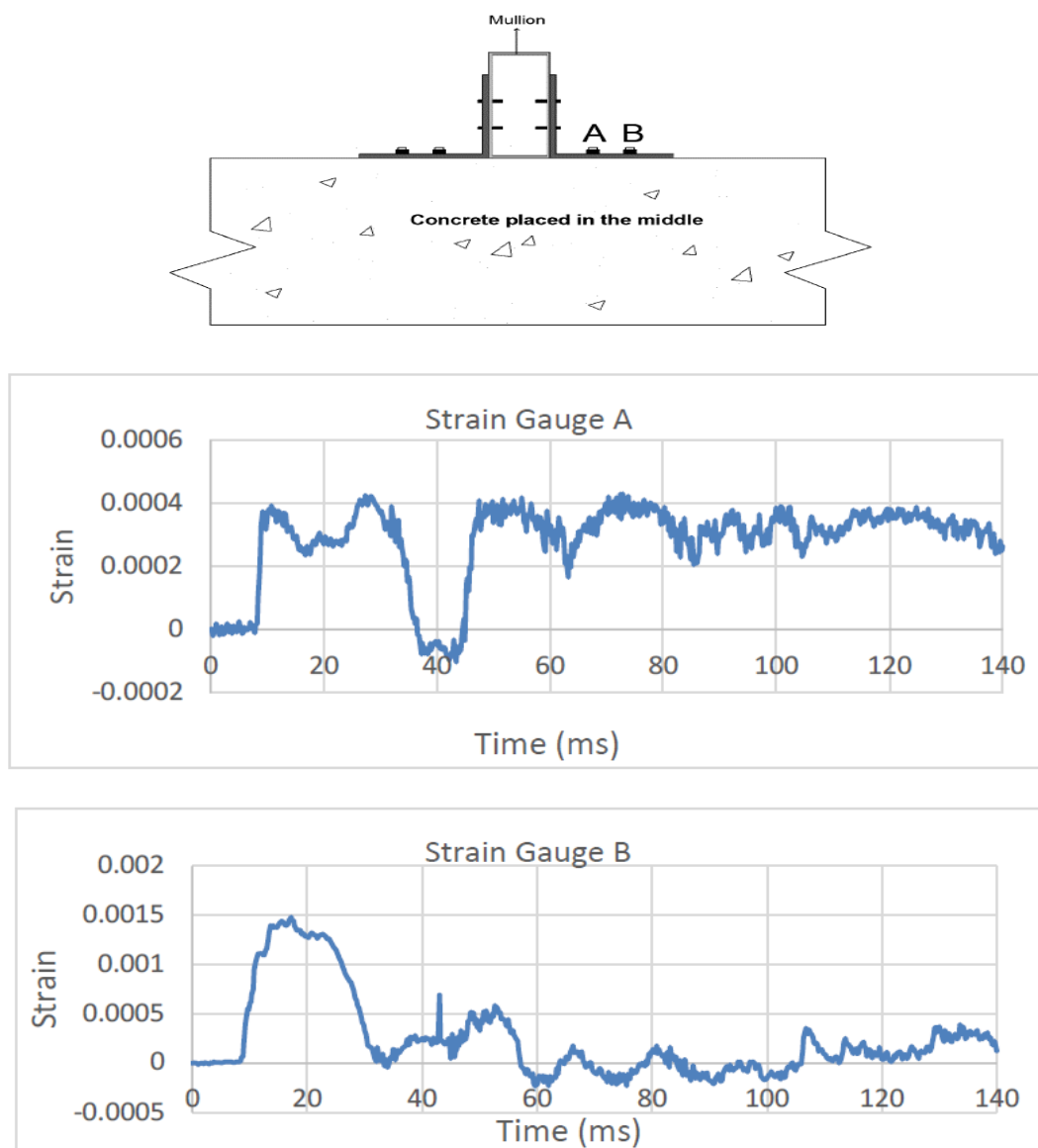


Figure A.12: The strain gauge data for M-C4 test



Figure A.13: The mullion deflection and buckling after M-C4 test



a) Local compression buckling



b) Loosened side bracket

Figure A.14: The pictures of MC-4 after test

A.8: Test M-C5

M-C5 was an individual mullion connected to concrete beam/slab through energy absorption components involving XRD rubber inserted between the mullions and the concrete slabs. A thin layer of rubber was utilized in this test due to the bolt length limitation. It was subjected to a single shot with a pressure-impulse combination of 71 kPa – 830 kPa-ms, and the LVDT reading on the lower segment below the slab indicated a maximum deflection of 21 mm. A better performance was observed in M-C5 by inserting damper, however, the thickness of rubber was not enough to significantly reduce the damage on the specimen. There was no damage induced on the connections, and less compression buckling was observed at the mid-height of top or bottom

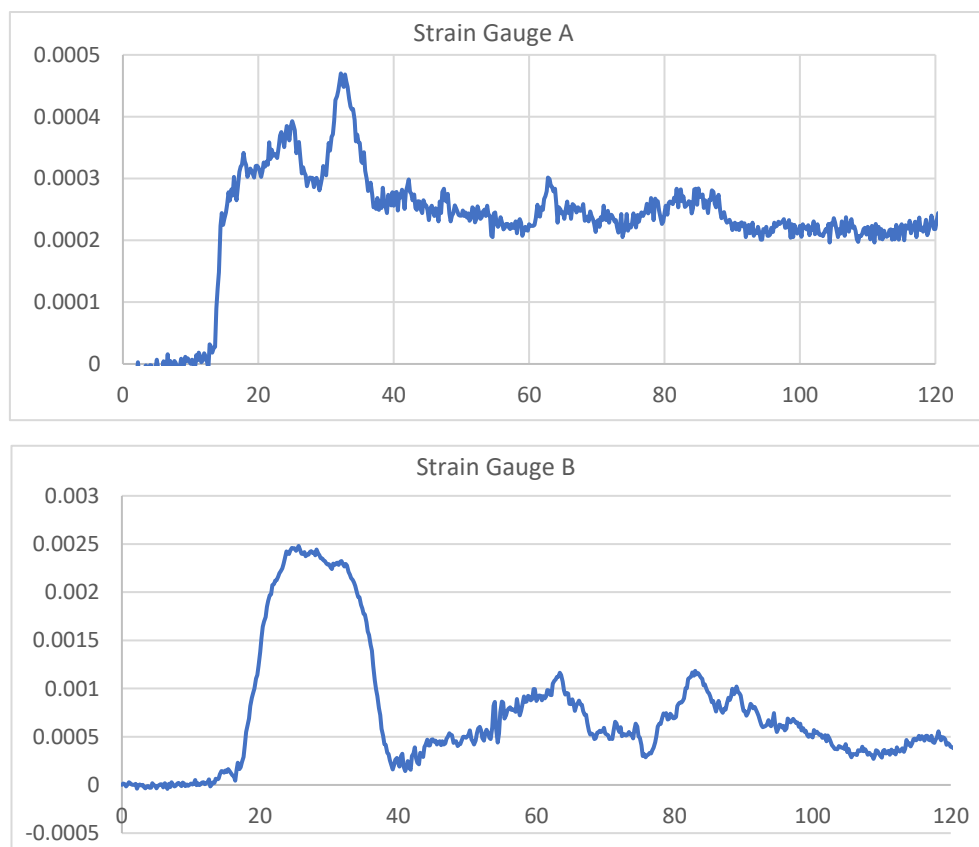
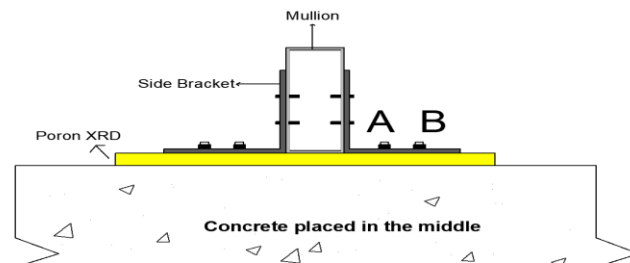
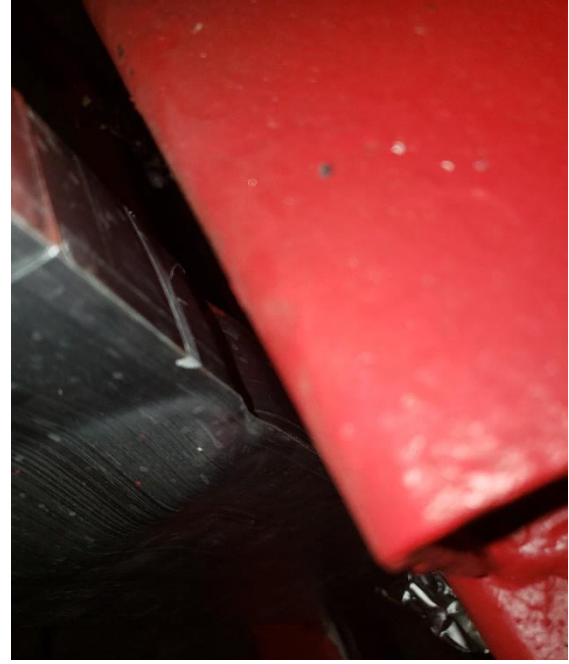


Figure A.15: The strain gauge data for M-C5 test



a) Compression buckling upper section



b) Compression buckling lower section



c) In tack right bracket with no loosening



d) Compressed XRD rubber

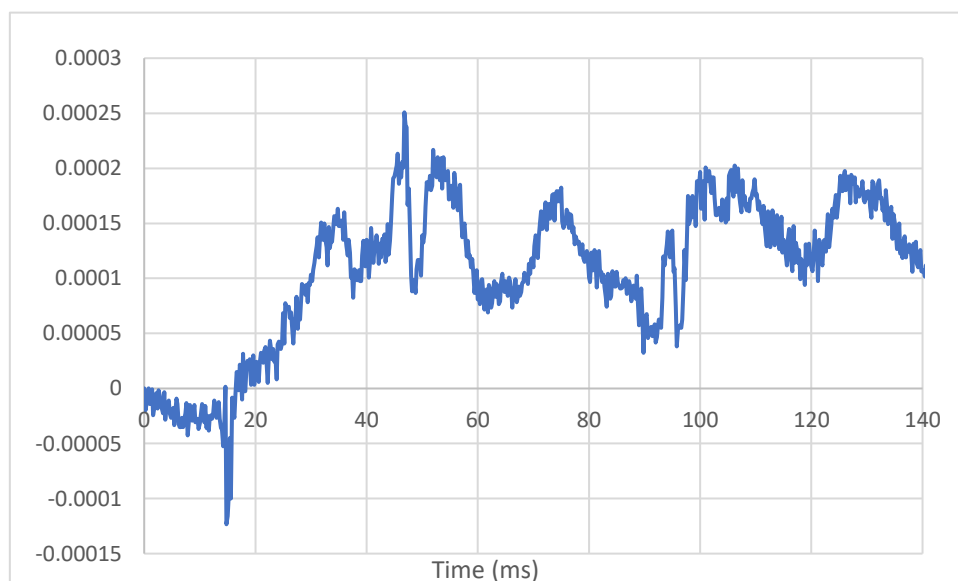
Figure A.16: M-C5 after applying the load

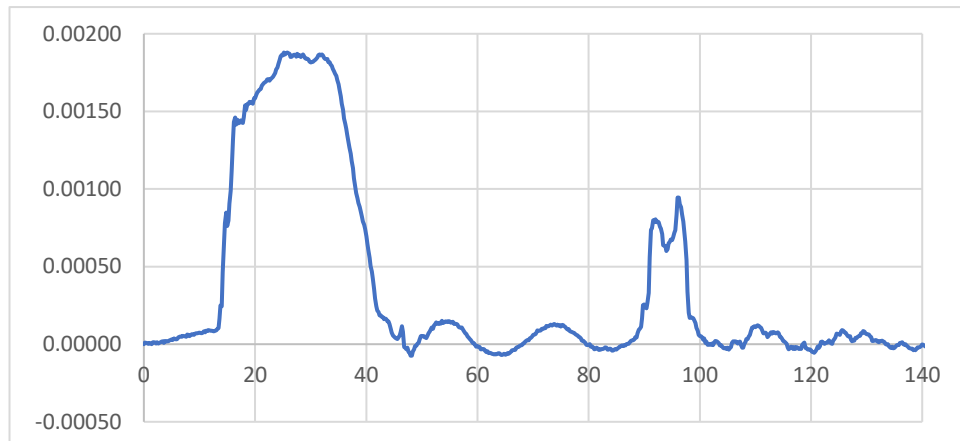
A.9: Test M-C6

M-C6 was an individual mullion using brackets attached to the top and bottom of concrete beam (slab) and the narrow side (flange) of the mullion. High performance XRD rubber was inserted between mullion, concrete beam and brackets. These components were connected together by means of bolts and rivets. It was subjected to one shot with a pressure-impulse combination of 73 kPa – 912 kPa-ms, and the LVDT reading on the lower segment below the slab indicated a maximum deflection of 36 mm. Bending of the brackets strained the bolts that were used to connect the mullion.



a) Strain gauge locations

b) Strain gauge **A**

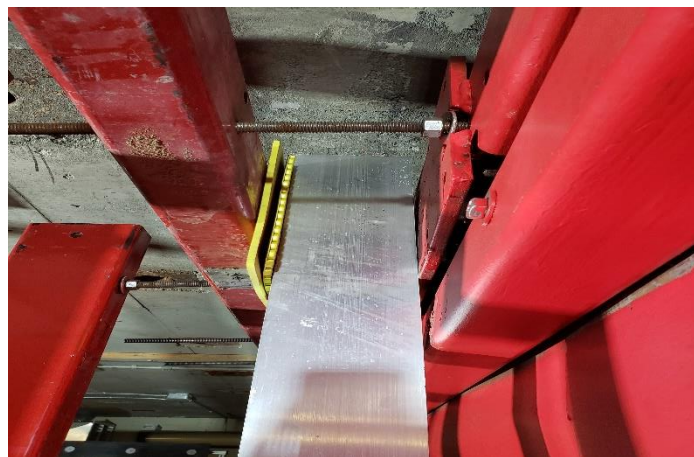


c) Strain gauge **B**

Figure A.17: The strain gauge data for M-C6 test



a) Compression buckling at upper and lower sections



b) In tack mullion fixed support

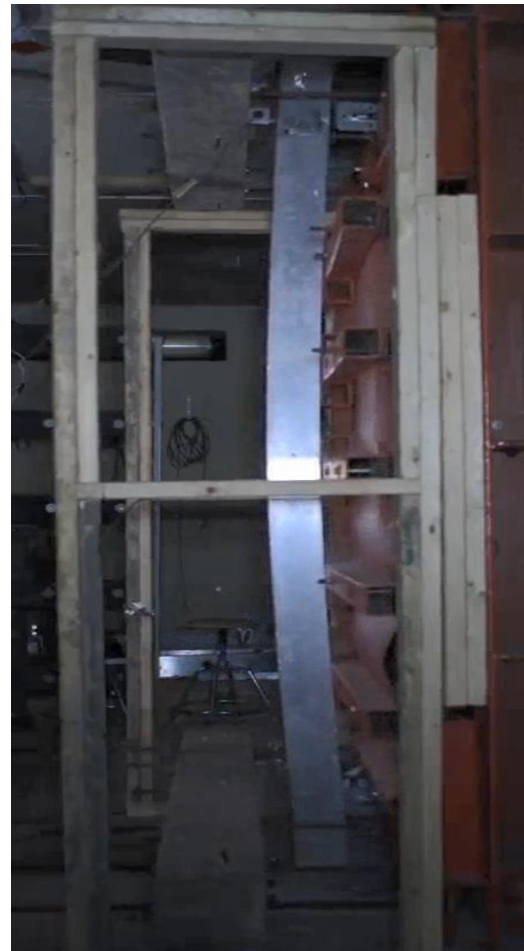
Figure A.18: M-C6 after applying the load

A.10: Test M-C7

The seventh mullion (M-C7) has inserted spiral springs with 8 N/mm stiffness. The net length of spring was 160 mm and single mullion prepared was considered for M-C7. The springs were inserted between the beam and mullion on the top and bottom concrete beams. This load was applied with a pressure-impulse combination of 24 kPa – 245 kPa-ms. By implementing this innovation in curtain wall system, a significant improvement was noted in M-C7 performance. No failure or deformation was observed in the specimen, proving the practicality of using energy damping elements in the curtain wall systems. However, mullion entered the inelastic region. Furthermore, mullion moved backward to the initial location after the test which was an ideal behavior.



a) Before test



b) Inelastic deformation

Figure A.19: M-C7 after applying the load

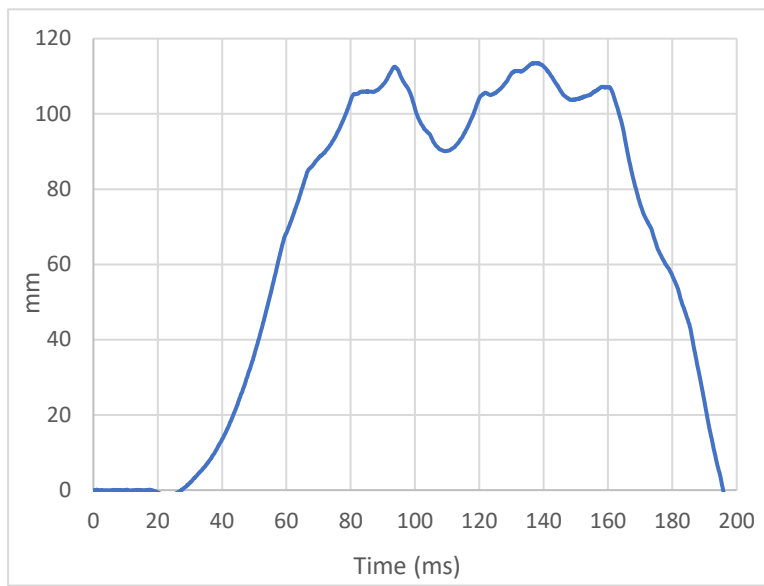


Figure A.20: LVDT data located at top spring

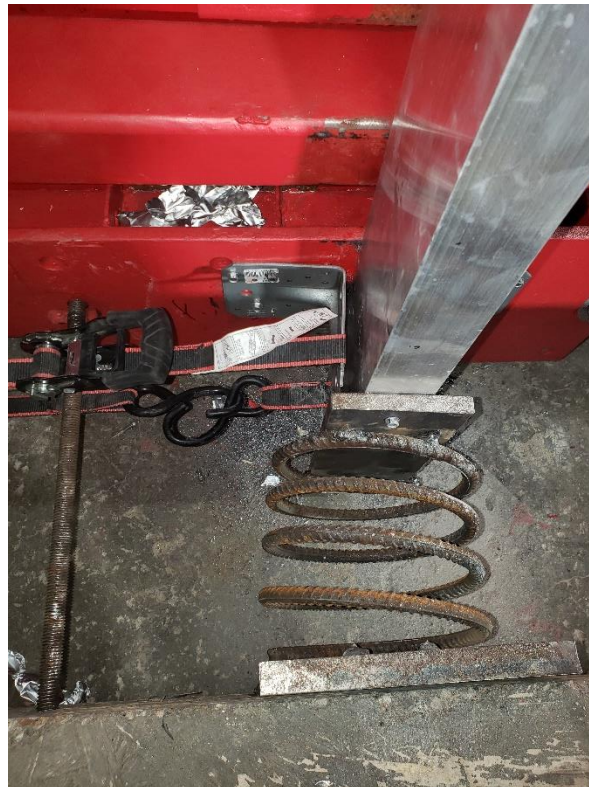
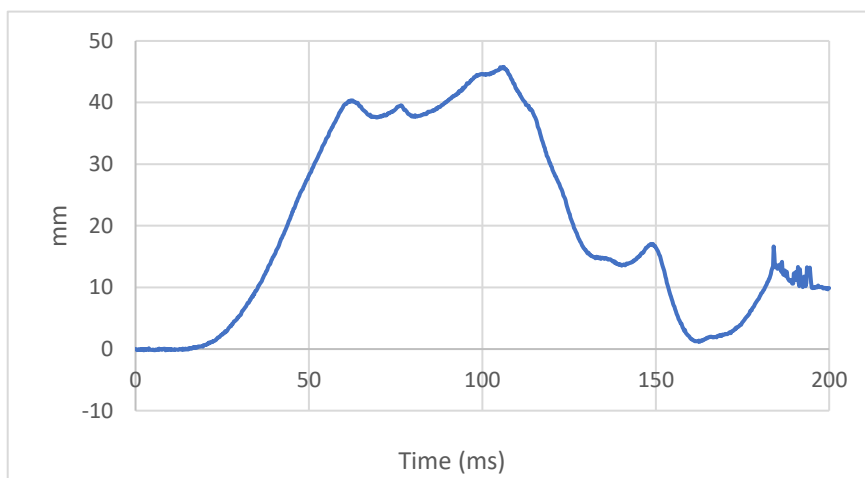


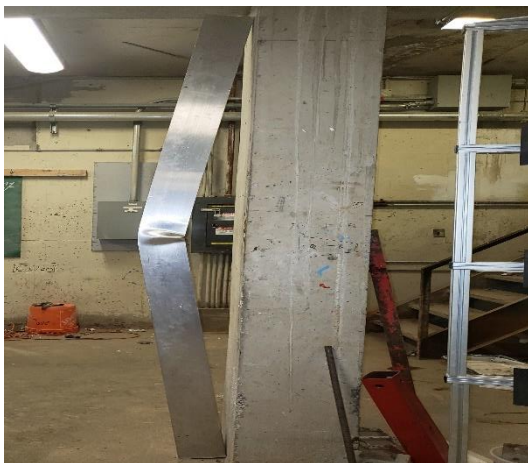
Figure A.21: M-C7 after test

A.11: Test M-C8

The eighth mullion (M-C8) was tested with single mullion configuration while a combination of two spiral springs with 30 N/mm total stiffness (8 N/mm + 22 N/mm) were inserted between the beams and mullion. Regarding the stiffer springs used in M-C8, a higher load with a pressure-impulse combination of 31 kPa – 312 kPa-ms was applied to M-C8. The applied shock-tube load caused 172 mm maximum deflection at the mid-height of mullion. M-C8 suffered failure at mid-height and compression buckling was observed in M-C8.



a) LDVT located at the top spring



b) Mullion deformation



c) support rotation at springs

Figure A.22: M-C8 observations after test

A.12: Test M-C9

The ninth mullion (M-C9) was tested with single mullion configuration while again a combination of two spiral springs provided 23 N/mm total stiffness (8 N/mm + 15 N/mm). The springs were inserted between the mullion and concrete beams located on top and bottom. However, the spring stiffness was less than M-C8, but the springs had longer net length to compare the level of protection based on CSA S-850. By applying blast load, a maximum deflection of 192 mm and 172 mm was noted in the mullion mid-height and the below spring respectively. The springs used in M-C9 test were able to damp the load properly and no failure was observed at either mullion or support. Nonetheless, the welded springs were broken, and M-C9 was not subjected to the progressive load as it was planned.

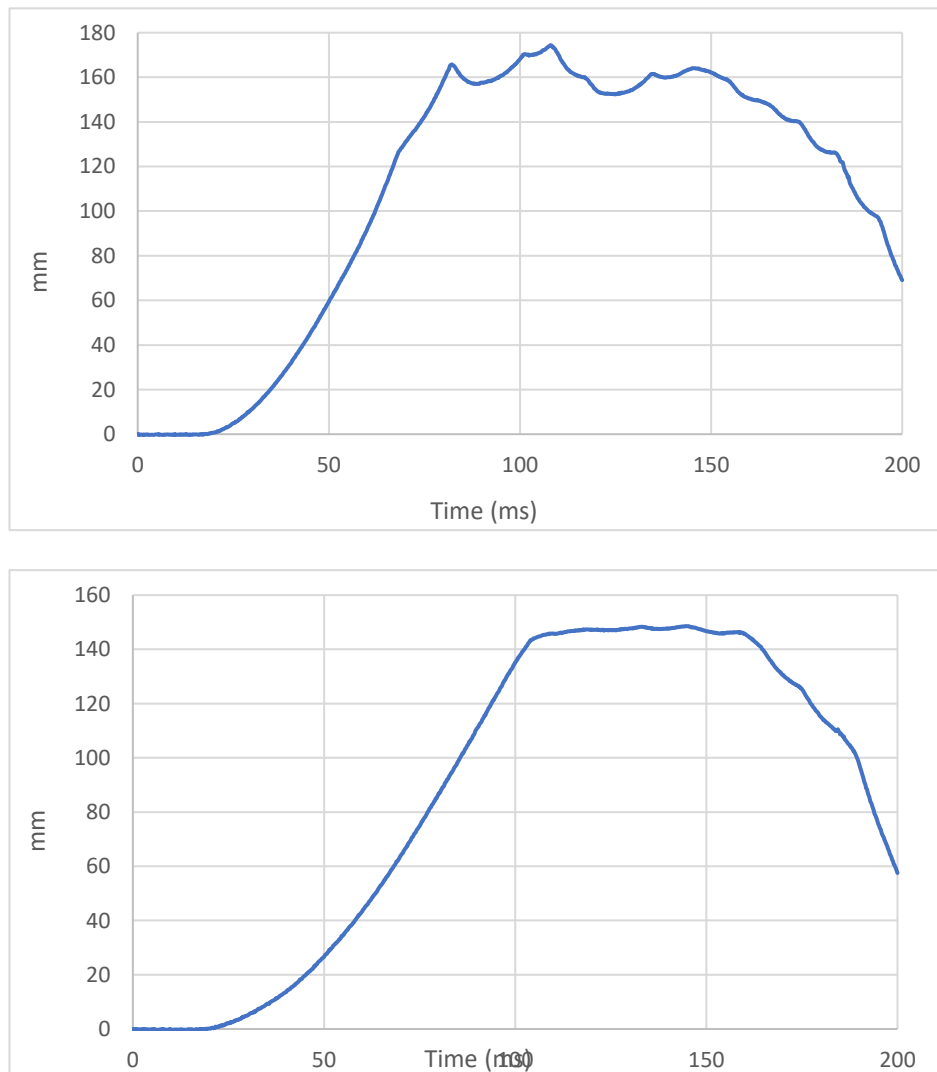


Figure A.23: LVDT results installed at the bottom spring



a) Spring on top



b) Spring on bottom

Figure A.24: Failure in the welding

Appendix B Finite Element Models

Details of Finite Element Method (FEM) and Supplementary Information for Retrofitted Connections

B.1: First Phase: Full-size Curtain wall

B.1.1 First Curtain Wall Using Steel Sheet (CW1)

The first curtain wall model (CW1) represented the specimen tested experimentally using #22 steel sheet to cover the exterior surface. The inner U-angles were retrofitted by utilizing steel U-angles. Besides, aluminum side brackets were glued to the mullion and transoms. The aluminum angles on the connections were detached shortly after the application of the blast shot showing lack of composite action, but the horizontal and vertical mullions remained connected.

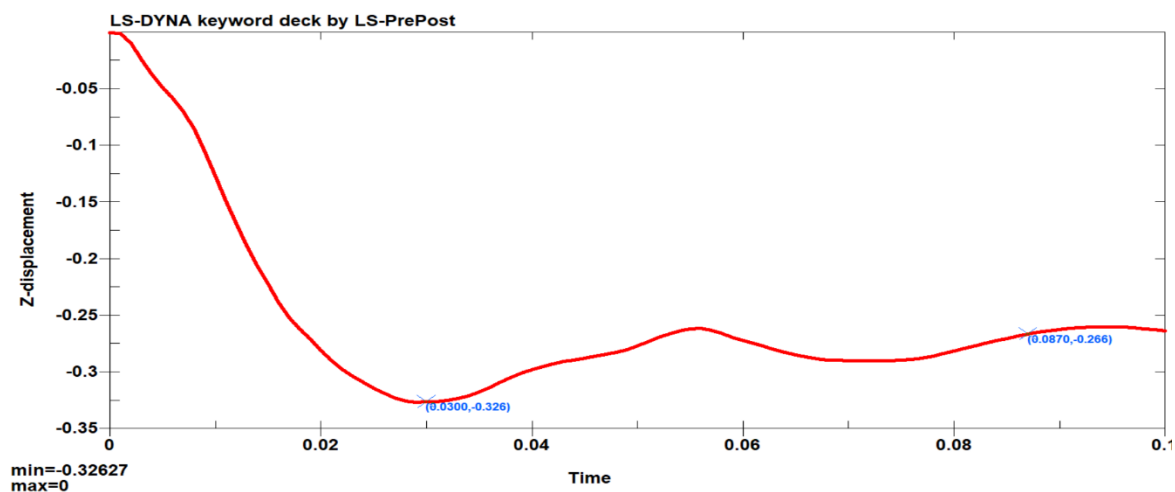


Figure B.1: Left mullion deflection

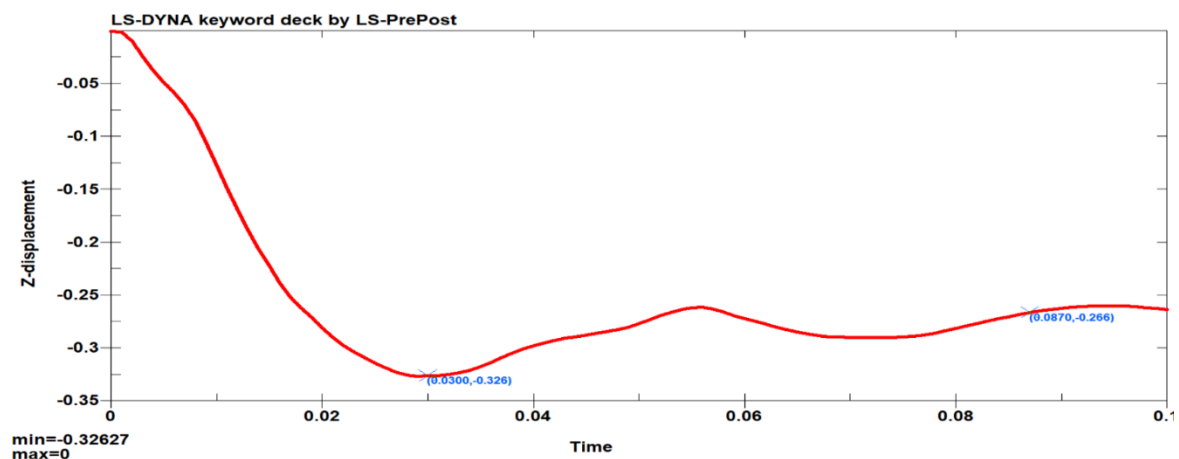


Figure B.2: Middle transom deflection

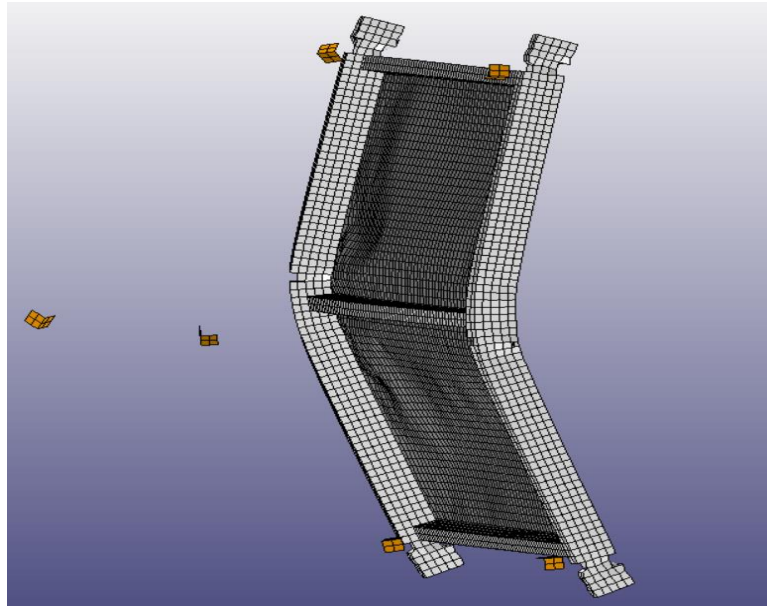


Figure B.3: Glued bracket instantly detached from the sections

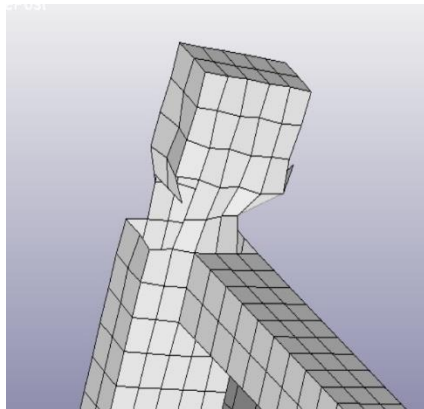


Figure B.4: Failure observation at simple supports

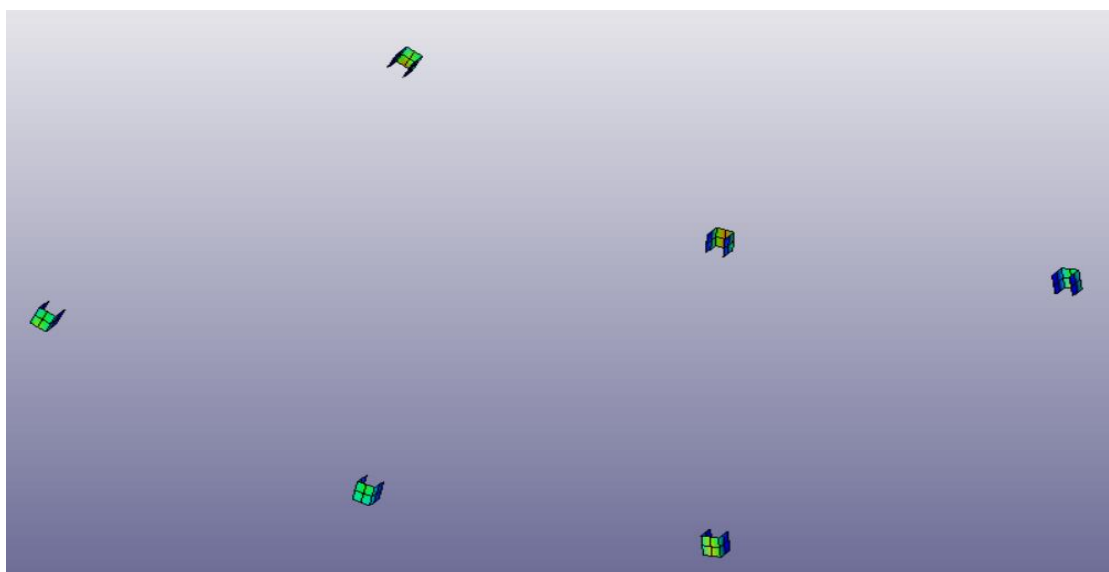


Figure B.5: Von mises stress in the internal U-angle connections

B.1.2 First Curtain Wall Using Glass + Film Layer (CW1+G&F)

The first curtain wall with glass and film was assessed as a parametric study to investigate the authenticity of results obtained during the curtain wall tests. It had exactly the same test configuration like the experiment, nonetheless, glass and film were modeled (instead of steel sheet) to cover the exterior layer of curtain wall. It illustrated that the practice used in the experiment can provide reliable results in term of maximum and residual deflections. Likewise, roughly similar support reaction was achieved for CW1+G&F compared to the reference numerical model (CW1).

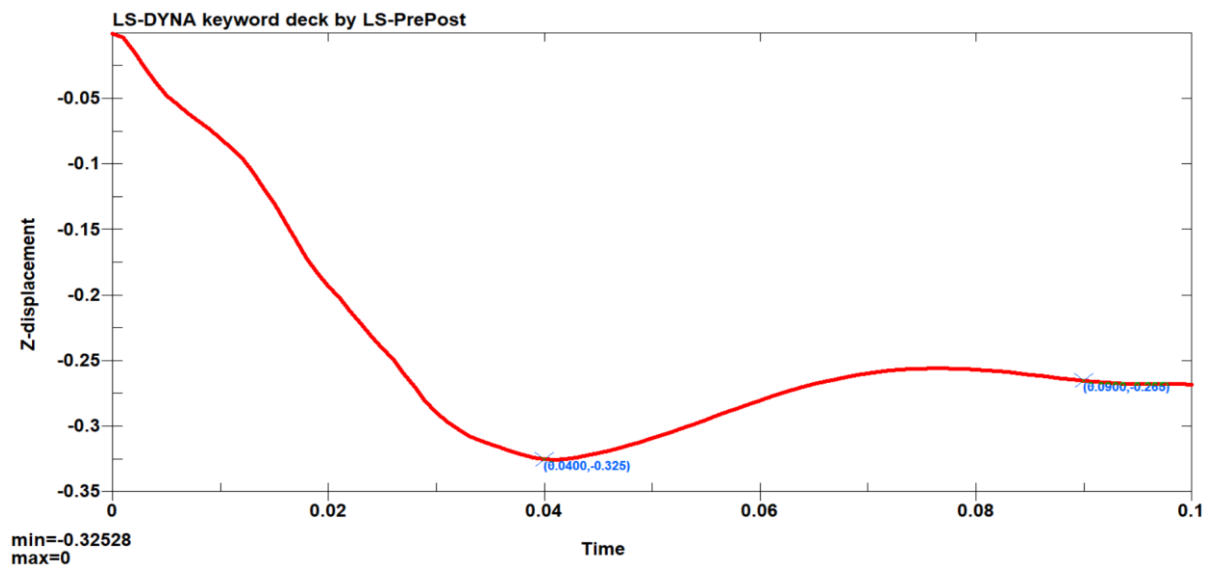


Figure B.6: Left mullion deflection

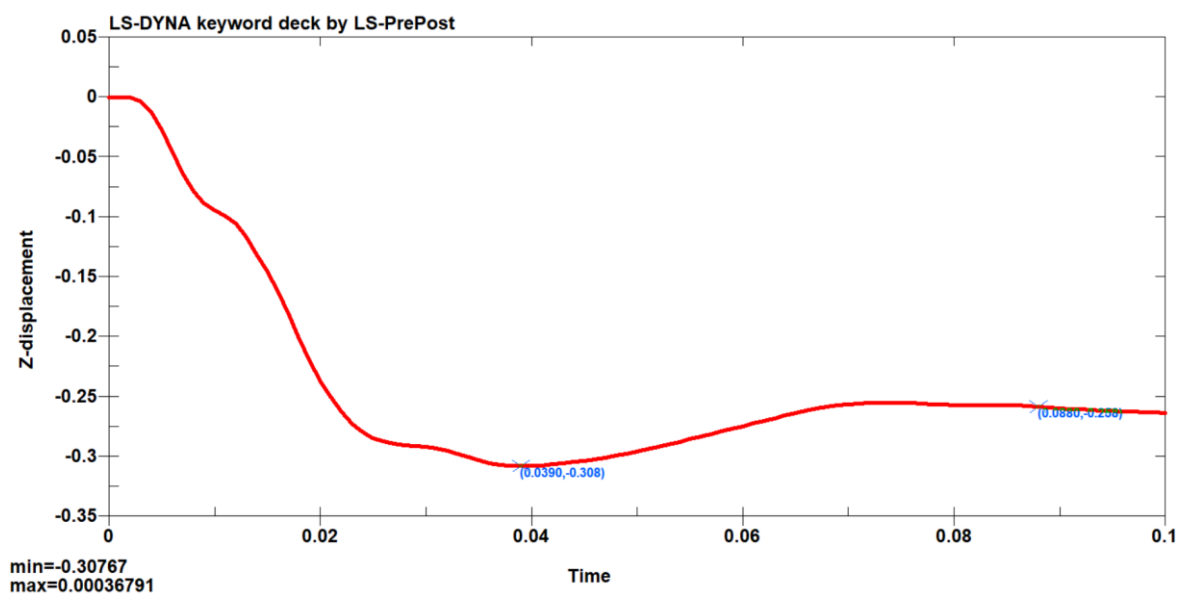
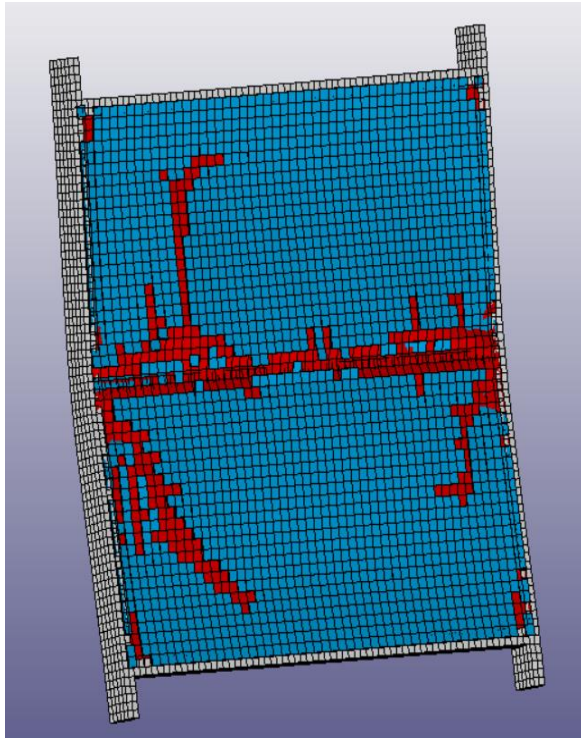
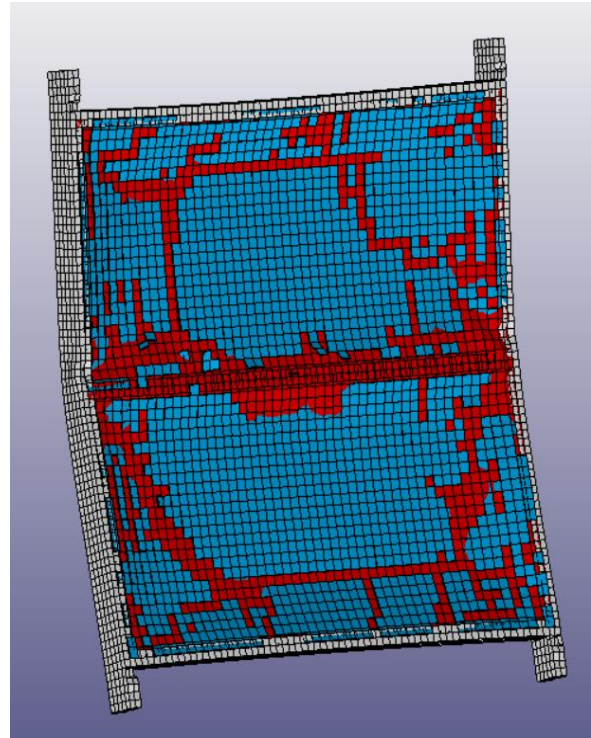


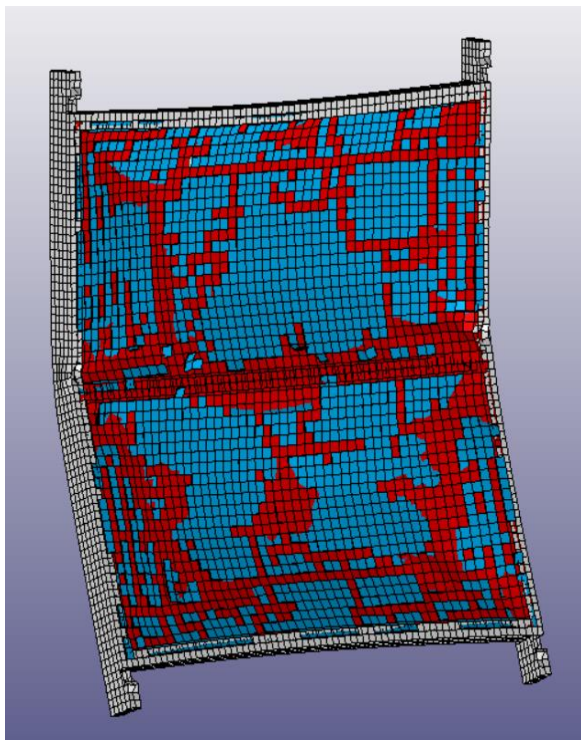
Figure B.7: Middle transom deflection



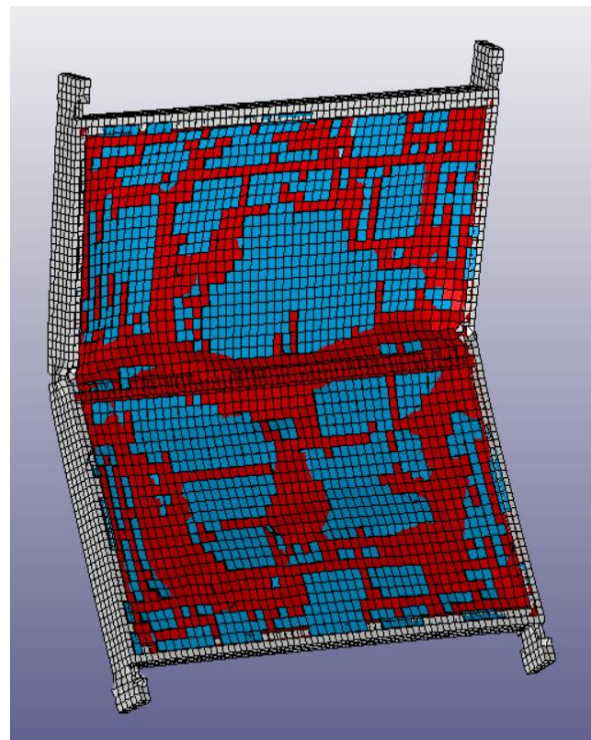
a) 8 ms



b) 14 ms



c) 20 ms



d) 28 ms

Figure B.8: Glass and film failure mode in CW+G&F model

B.1.3 First Curtain Wall with Weak Internal U-angle Connectors (CW1+WC)

This model simulated an ordinary curtain wall used in the industry which are designed against the wind load. These type of curtain walls are assembled by using thin aluminum U-angles, which are only anchored to the mullions by means of small pins. Therefore, by modeling CW1+WC, the vulnerability of these kind of connections against the blast load was assessed. The transoms experience instant disconnection from the mullions since they the ordinary connectors were not able to resist against a normal blast impulse (50 kPa reflected pressure). The mullions stayed on their location due to their boundaries. This model shed light on the importance of retrofitting the internal U-angles which was done in the first phase.

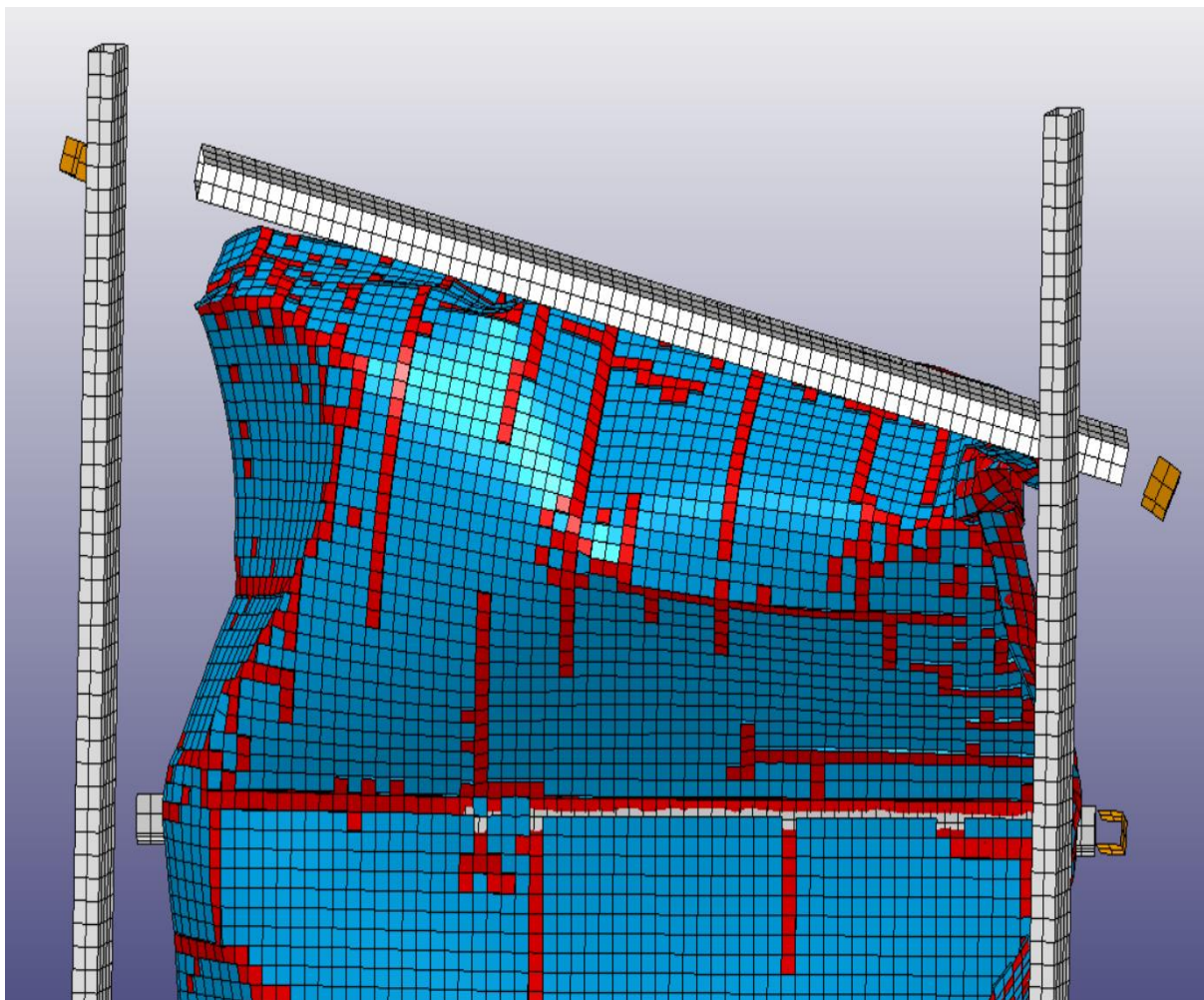


Figure B.9: Unattached transoms and U-angles of CW1+WC

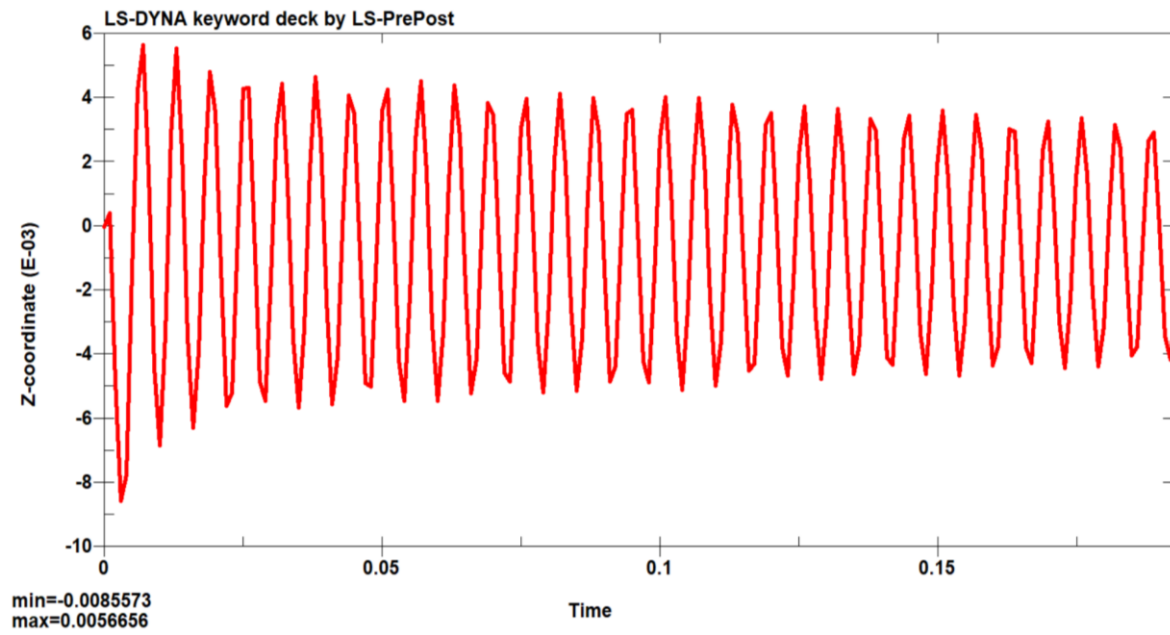


Figure B.10: The vibration in the mullion after transom detachment in CW1 +WC

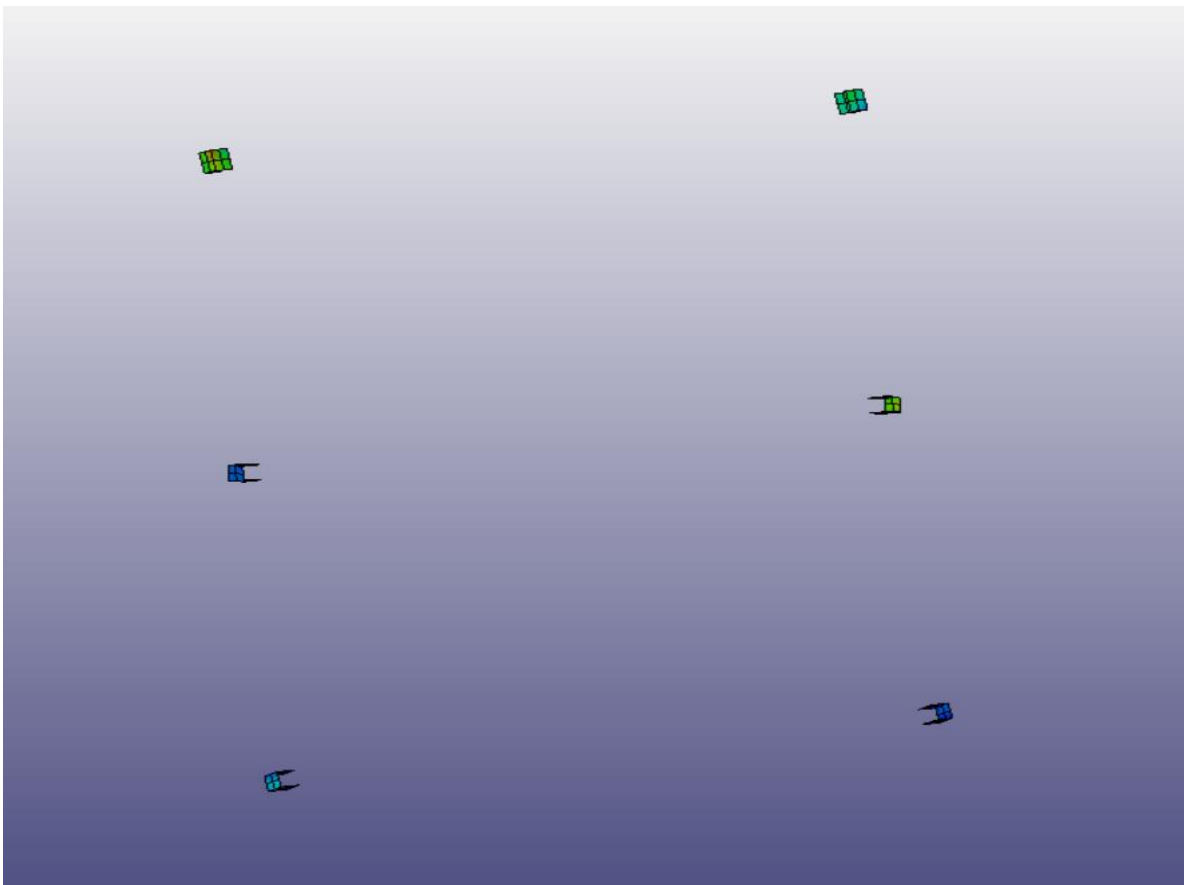


Figure B.11: The stress distribution in the U-angles after applying the load

B.1.4 Second Curtain Wall (CW2)

The second curtain wall had hardened vertical mullions and unretrofitted transoms. The inner U-angles were retrofitted and aluminum side brackets were anchored to the sections. The stiffened vertical mullions and the associated restraint in the translation of the horizontal mullion supports at its ends placed heavy burden on the horizontal mullion. The connections, strengthened by bolted aluminum angles, performed well and did not suffer any damage.

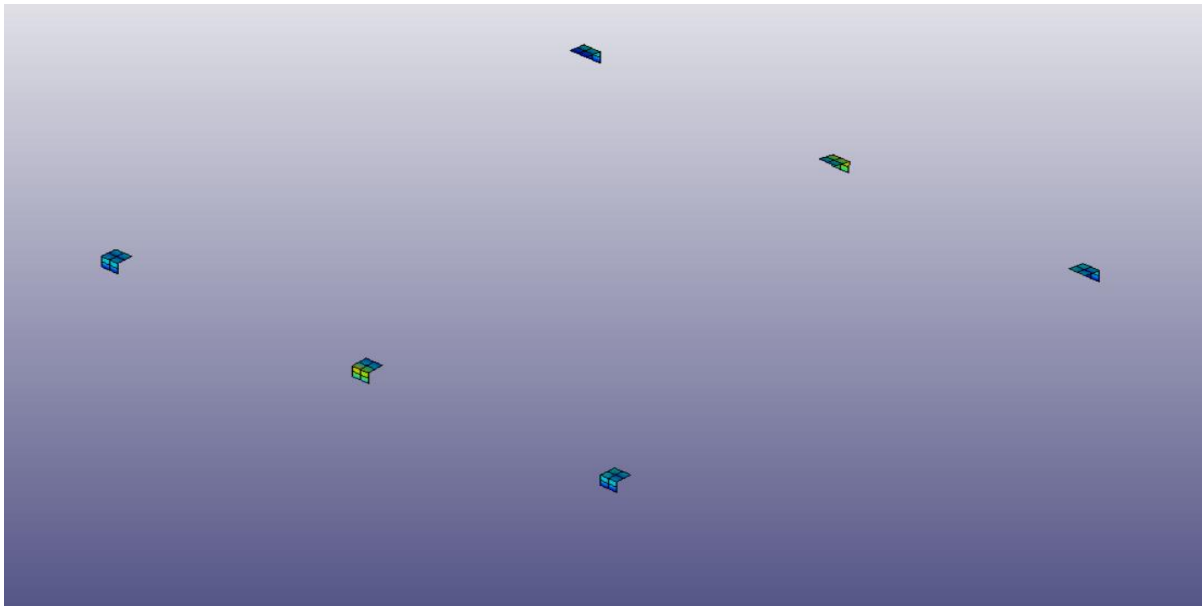


Figure B.12: The stress distribution in the side brackets of CW2

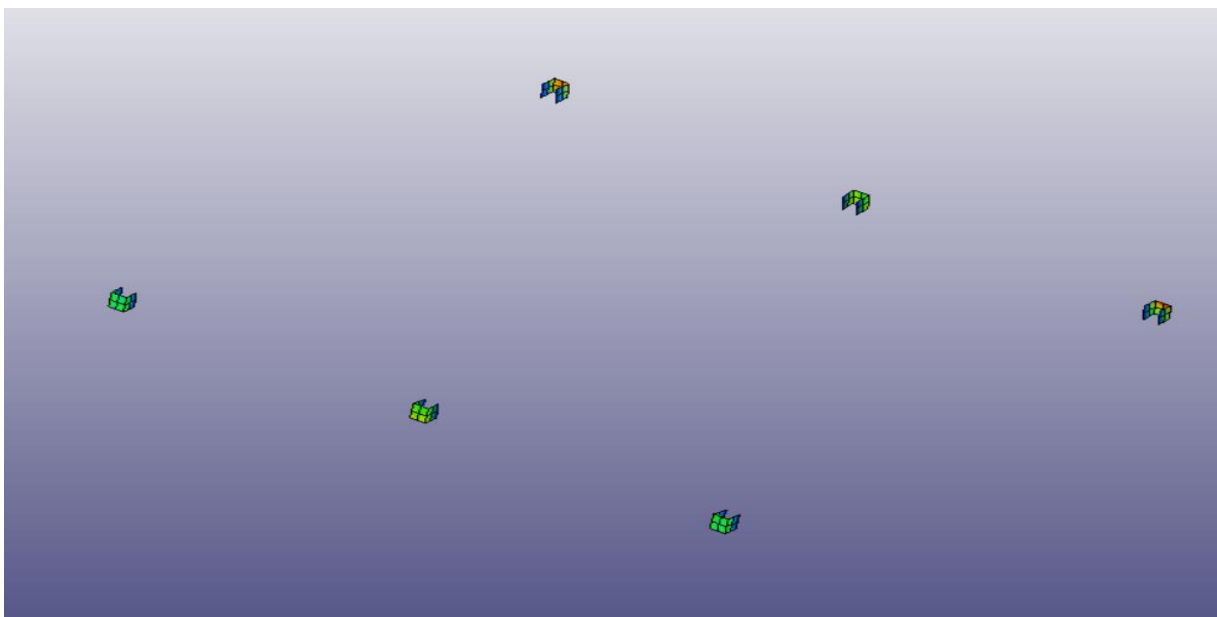
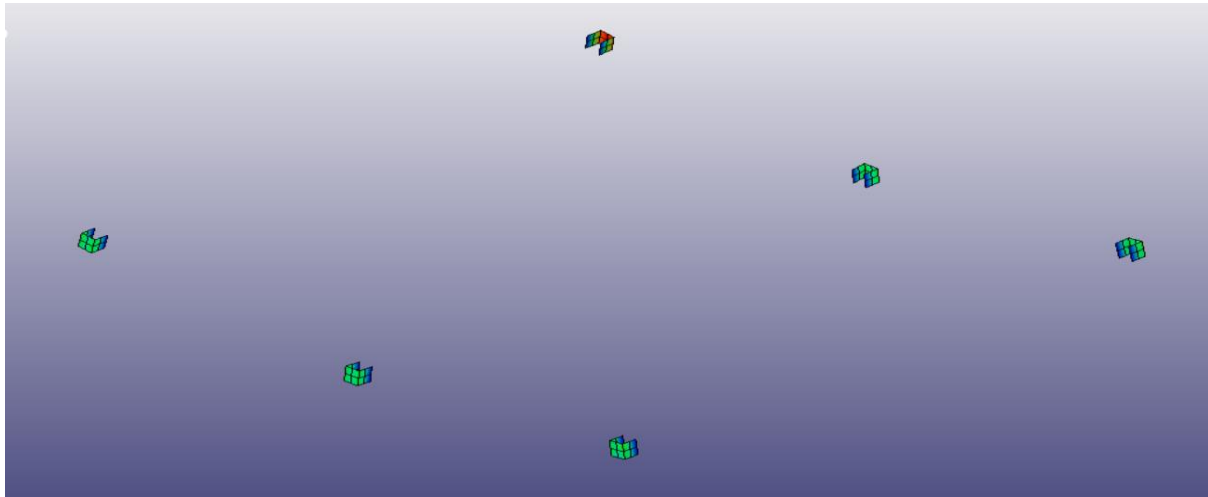


Figure B.13: The stress distribution in the U-angles of CW2

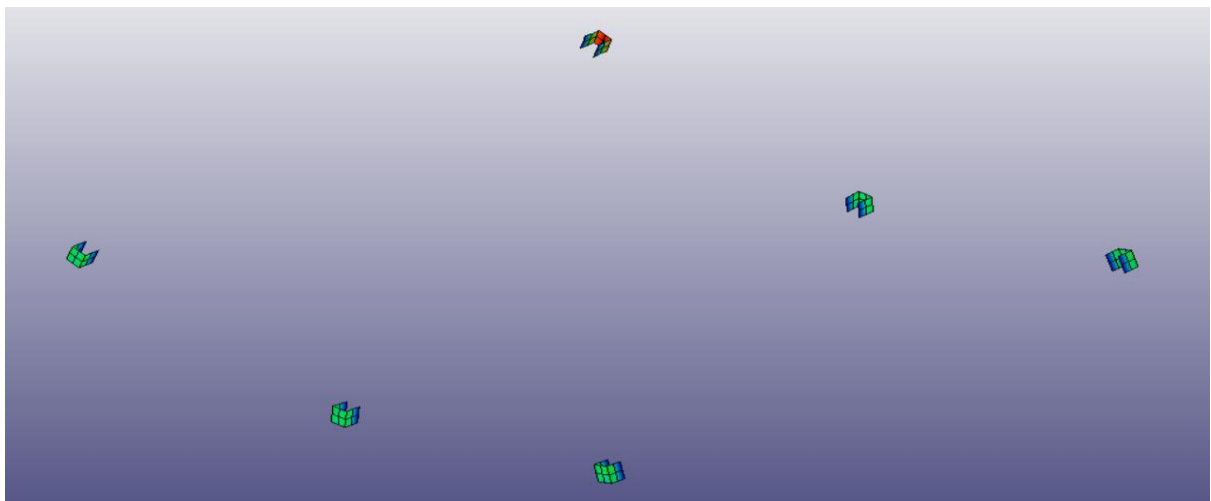
B.1.5 Third Curtain Wall (CW3)

The third curtain wall had all the mullions hardened by L-shaped steel plates, vertical and horizontal mullions alike. The internal connectors were retrofitted and the connections were also strengthened by steel angles. The third curtain wall was subjected to three blast shots. This model performed very well and showed that retrofitting the curtain wall section with steel sections would result in the high level of blast resistant technique.

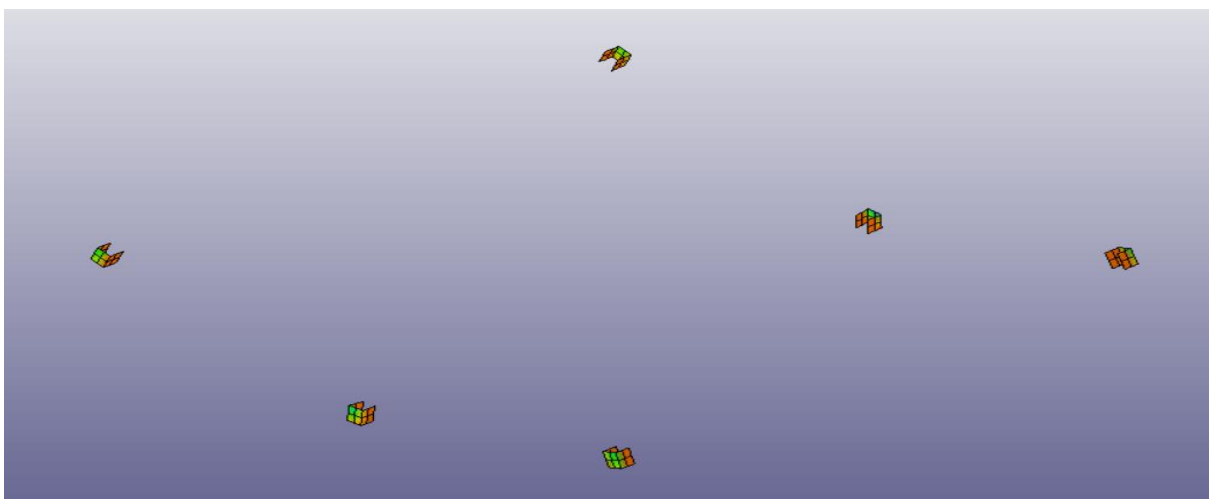
The first shot resulted in permanent deflections, which opened up gaps along the sides of the shock-tube. These gaps were closed with wood to prevent venting during the second shot. The second blast shot was more intense with a pressure-impulse combination of 50 kPa – 575 kPa-ms, resulting in a mid-height deflection of 164 mm. Cracks were observed in the vertical mullions at mid-height and the L-shape retrofitting plates were slightly fractured. The connection angles remained in tack. The third shot was meant to assess the failure mode under increased threat levels. A slightly increased pressure and impulse combination of 54 kPa and 595 kPa-ms was applied to already damaged curtain wall as the third shot. This was sufficient to cause the failure of the vertical mullions by rupturing on the tension side and buckling on the compression side. The third shot resulted in an additional deflection of 120 mm over and above the existing deformed shape at mid height. Figure 27 illustrates the curtain wall at the end of the third shot. The steel brackets remained intact until after the vertical mullion has failed.



a) 1st shot



b) 2nd shot

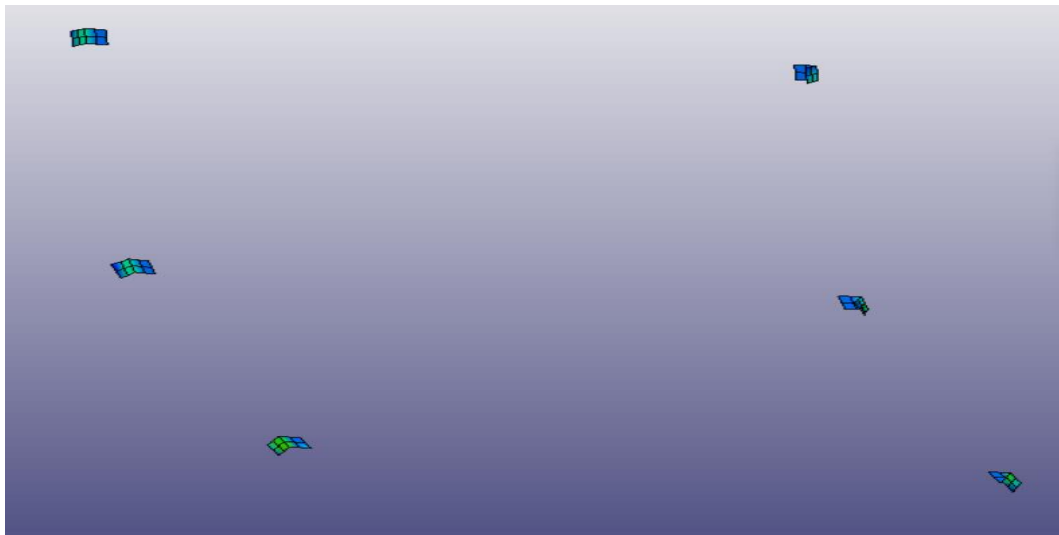


b) 3rd shot

Figure B.14: Stress distribution in U-angle connectors of CW3 model



a) 1st shot



b) 2nd shot



c) 3rd shot

Figure B.15: Stress distribution in L-shape side brackets of CW3

B.1.6 Third Curtain Wall without Side Brackets (CW3_NB)

This model was generated to investigate the effectiveness of using side brackets on the curtain wall, specifically when there is no disconnection expected between mullion and transoms. In fact, the main stress would be received by the U-angles and the portion of side brackets in the load distribution was a controversial topic. CW3_NB is exactly the third curtain wall model, nonetheless, the side brackets were removed, and then the outcomes were compared with CW3. The results indicated that using side bracket has a negligible effect on the connection strength. Therefore, placing them might just lead to a weak hole on the sections since the using brackets require anchoring operation. However, a less element elimination was observed in boundaries that showed the positive effect of using bracket near the supports.

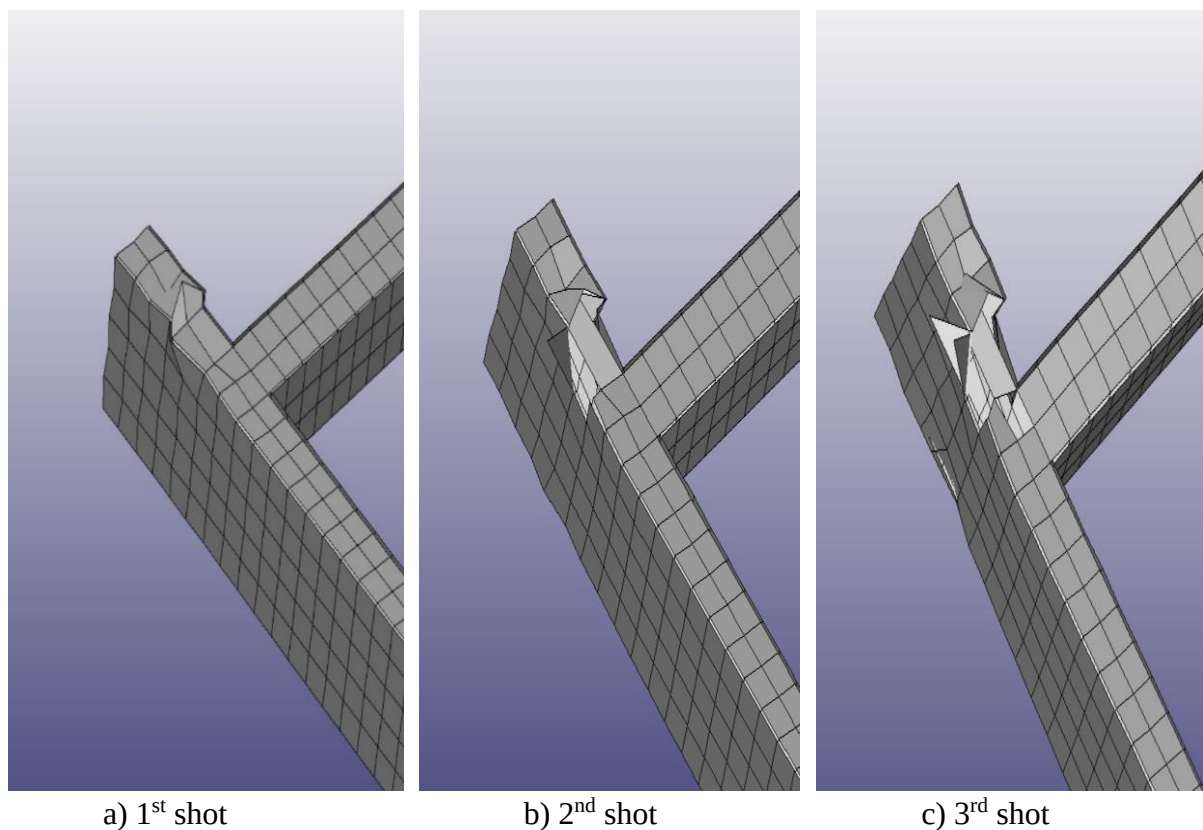
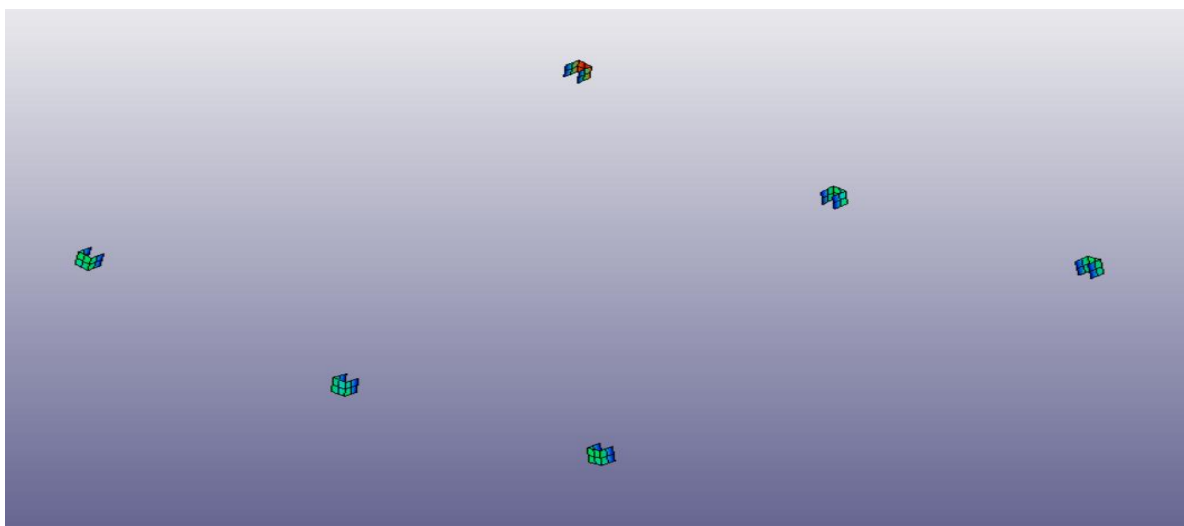
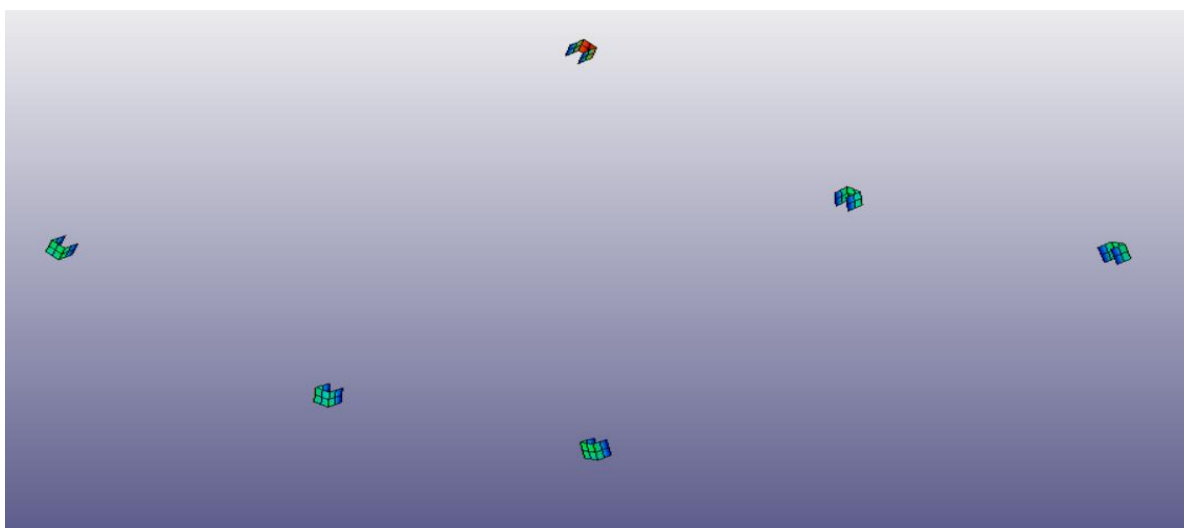


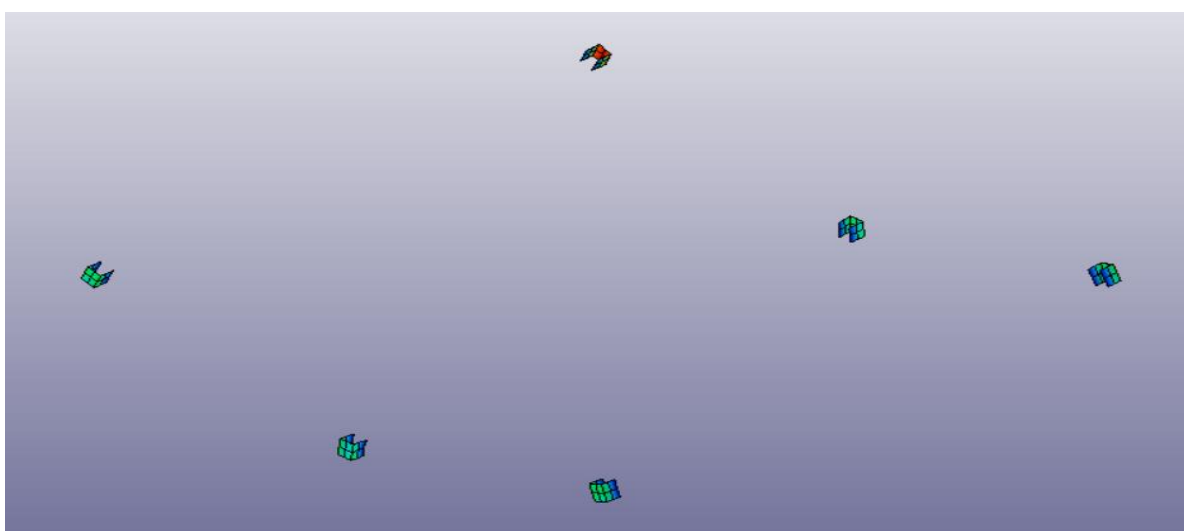
Figure B.16: Failure observation at boundaries of CW3_NB model



a) 1st shot



b) 2nd shot



c) 3rd shot

Figure B.17: Stress distribution in U-angle connectors of CW3_NB model

B.1.7 Third Curtain Wall with Brackets on Top and Bottom (CW3+T&B)

To propose a functional retrofit solution for existing curtain walls with weak internal U-angles, CW3+T&B model was generated. It had weak connectors for assembling the curtain wall similar to what is utilized in the industry, but transoms were retrofitted by steel bracket on top and bottom tied to the mullion perpendicular side. The steel bracket section size was 76 mm $\times$ 76 mm and 3 mm thickness, and glass + film was considered for this simulation. Furthermore, all curtain wall sections were hardened to prevent any failure on the sections since a high stress was expected on the mullions and transoms where brackets were anchored. CW3+T&B illustrated that using steel brackets connected on top and bottom on the curtain wall connections can be a proper retrofit approach because it kept the sections together without any disconnection. Compared to other models, less deflection in mullions, but higher deflection in transoms was observed in CW3+T&B. In fact, the internal weak connections were not contributing to provide stiffer connection, and the entire load/stress was received by the brackets. Also, more element elimination was observed in CW3+T&B which acknowledged the higher deflection in the horizontal sections.

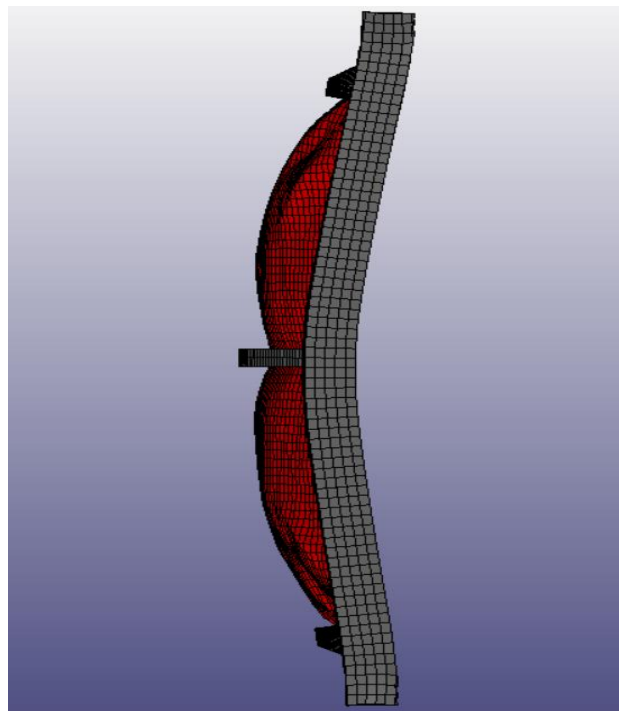


Figure B.18: Overall deflection of CW3+T&B model

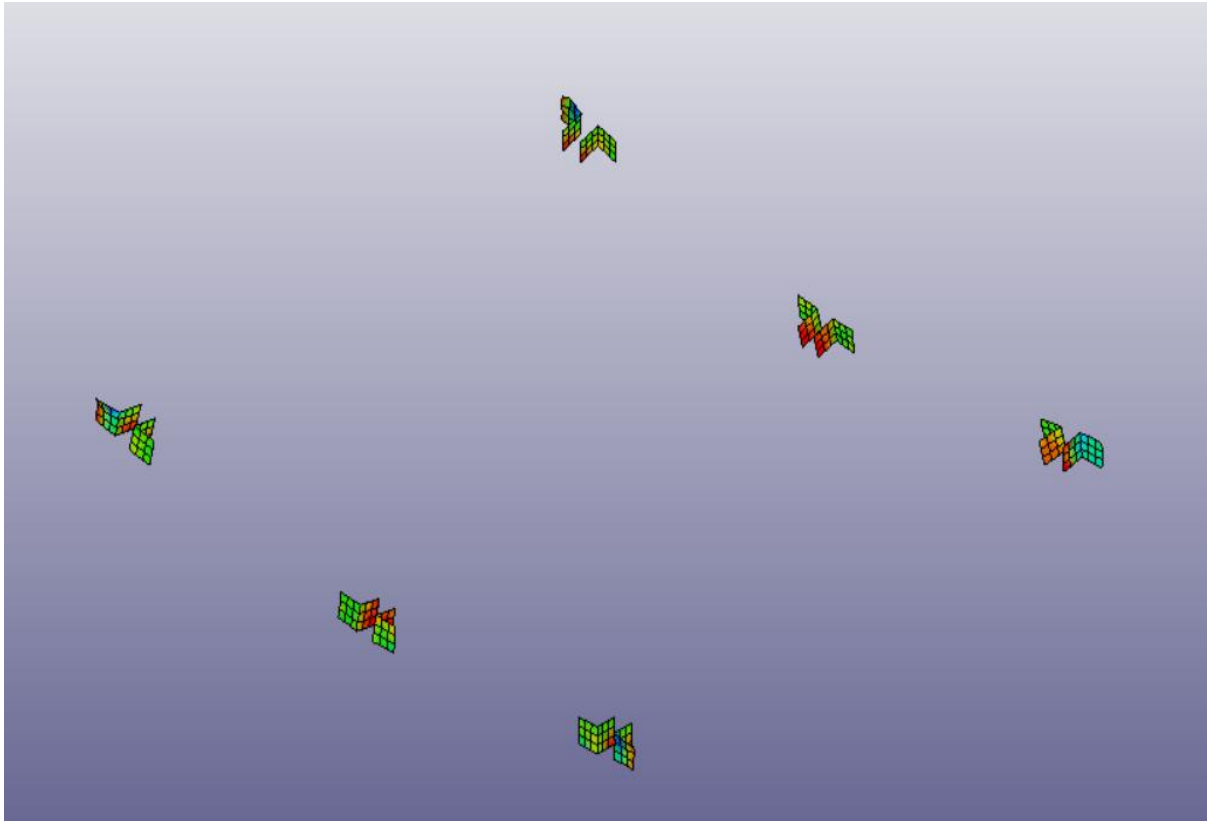


Figure B.19: Von mises stress in the top and bottom brackets of CW3+T&B model

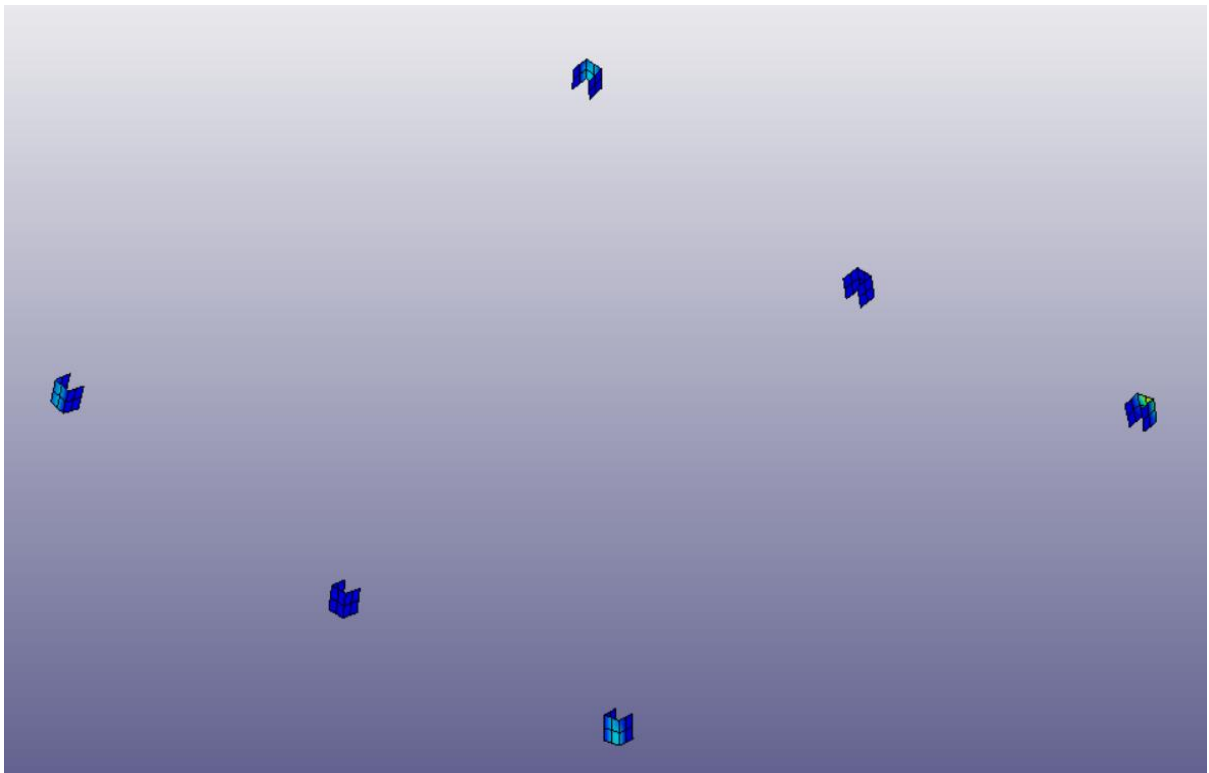
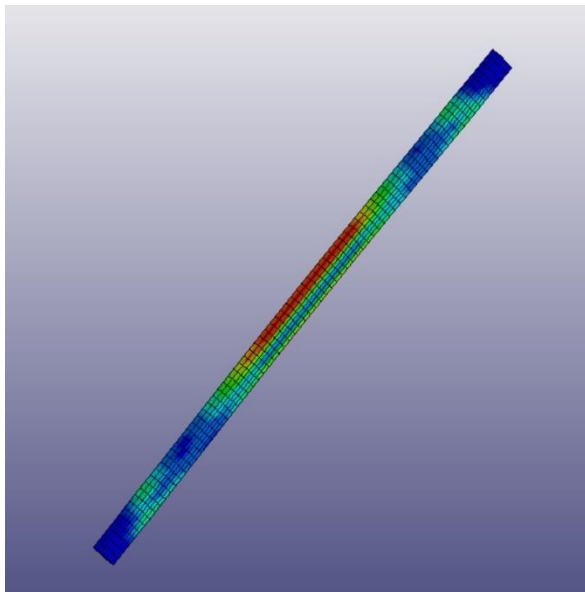


Figure B.20: Von mises stress in the internal weak U-angle connections of CW3+T&B model

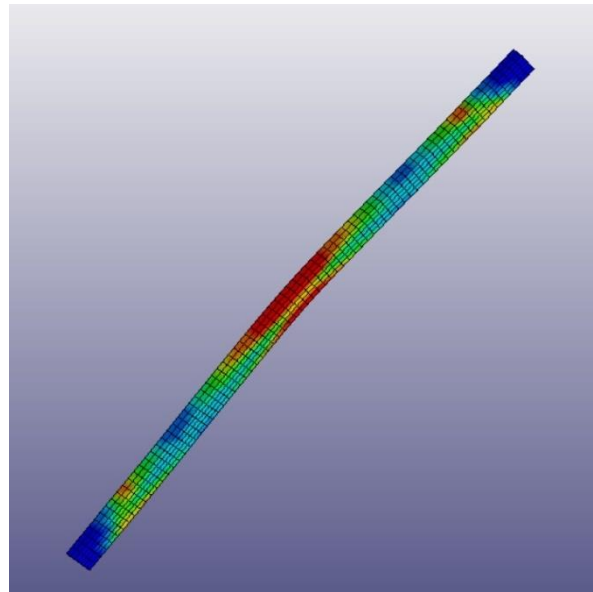
B.2: Second Phase: Full-size Single mullion

B.2.1 First Single Mullion (M-C1)

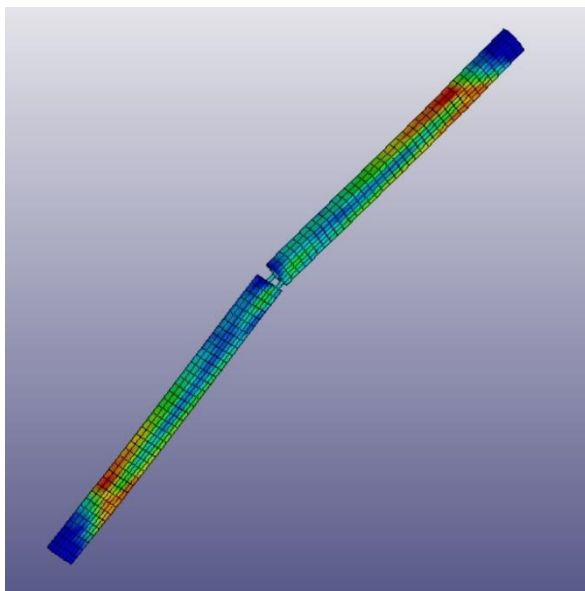
This model simulated a mullion connected at the ends using brackets attached to the front end of concrete beam (slab) and the two opposite sides (web) of the mullion. The model failed by rupturing of the mullion at its critical mid-height location from the tension and compression side. The same failure was observed during the experiment.



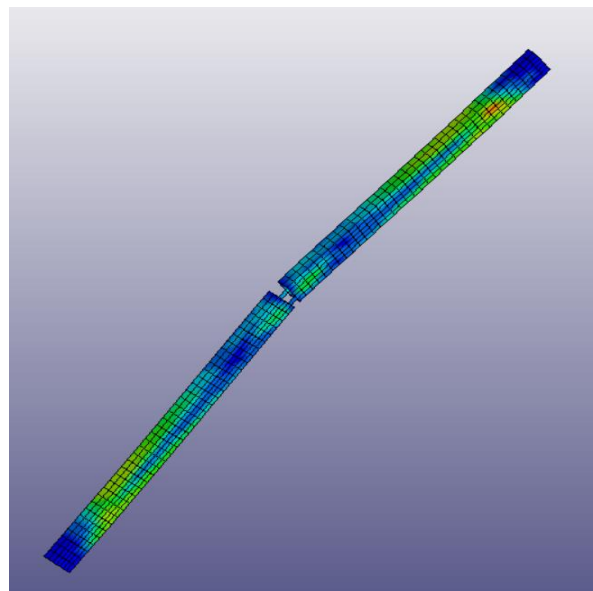
a) 3 ms



b) 5 ms



c) 8 ms

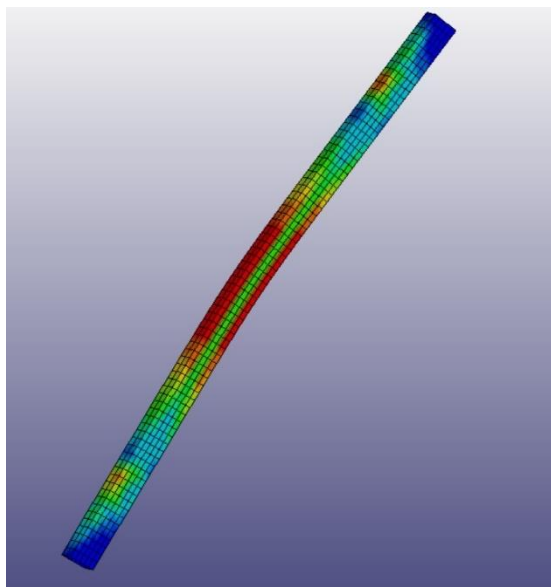


d) 14 ms

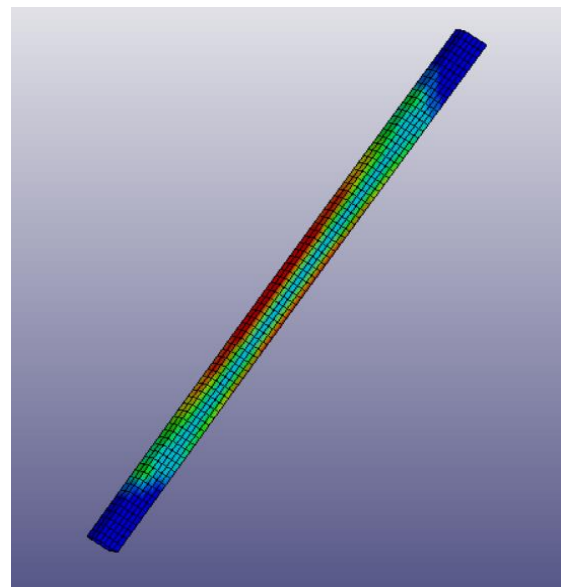
Figure B.21: Von mises stress in M-C1 mullion

B.2.2 Second Single Mullion (M-C2)

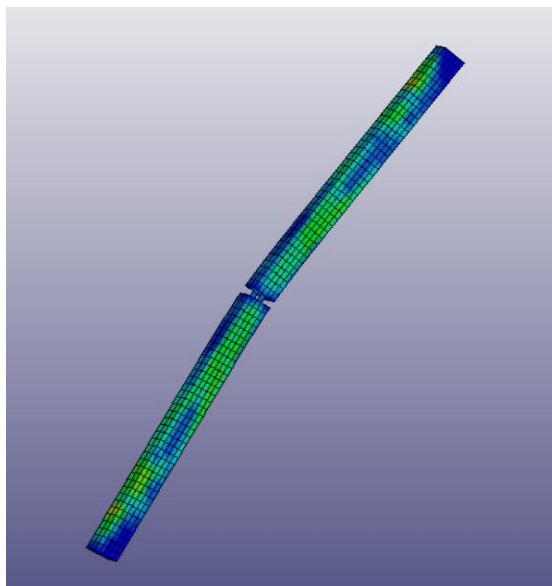
This model represented a mullion connected at the ends using brackets attached to the top and bottom of concrete beam (slab) and the narrow side (flange) of the mullion. This type of connection retrofit worked very well, and no disconnection was observed in the connections or brackets, nonetheless, the mullion ruptured in the critical flexural region at mid-height from both sides. The same failure mode was observed in the actual specimen, however, the experiment demonstrated local buckling at mid-height.



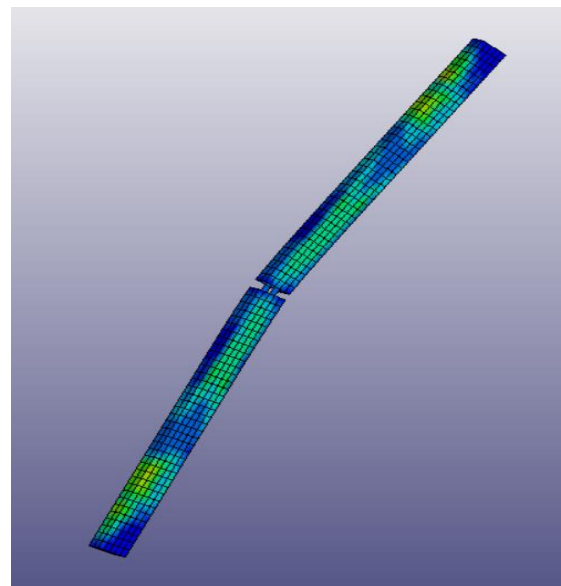
a) 3 ms



b) 5 ms



c) 8 ms

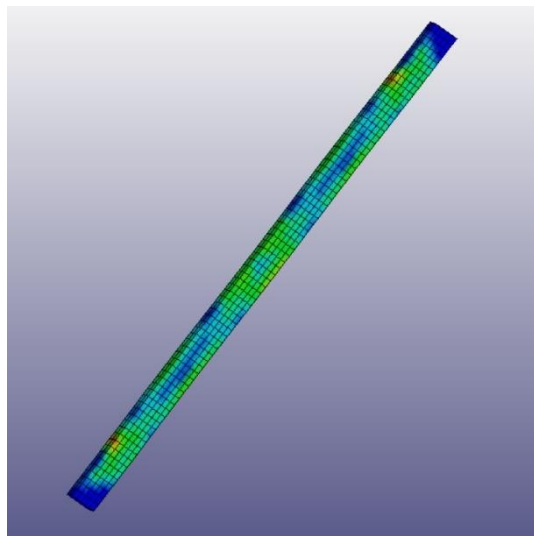


d) 12 ms

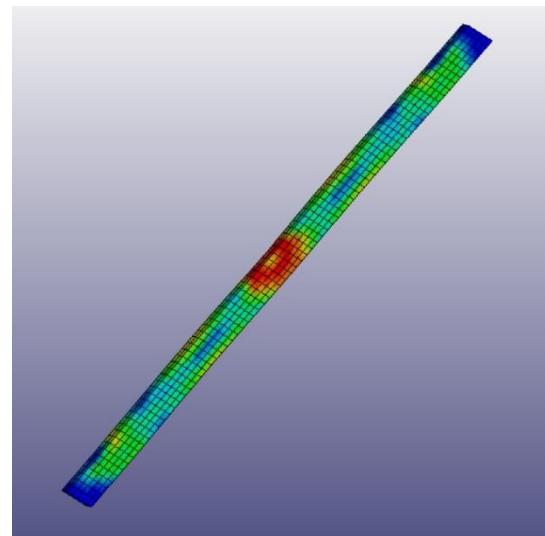
Figure B.22: Von mises stress in M-C2 mullion

B.2.3 Third Single Mullion (M-C3)

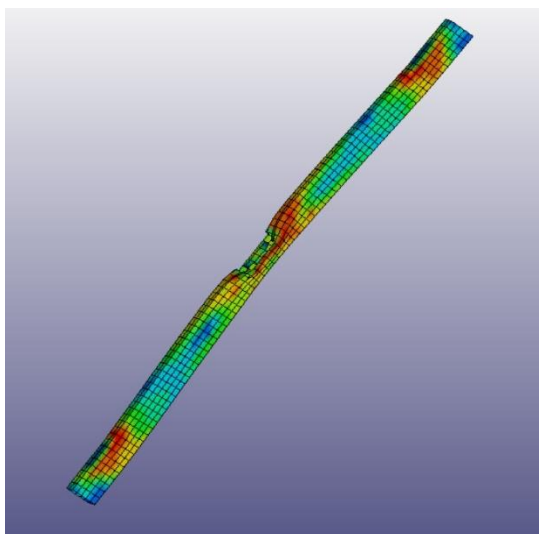
This model examined a mullion connected at mid-height over a concrete beam (slab) and connected using brackets attached to the top and bottom of concrete beam (slab) and the narrow side (flange) of the mullion. The numerical investigation showed that placing brackets on top and bottom of beam and along the applied load cannot help the connection between mullion and structural members. The mullion suffered from connection failure when the brackets were bent excessively, reducing the original 90-degree angle to approximately 45 degrees. The concrete slab bared against the mullion flange, crushing the aluminum mullion.



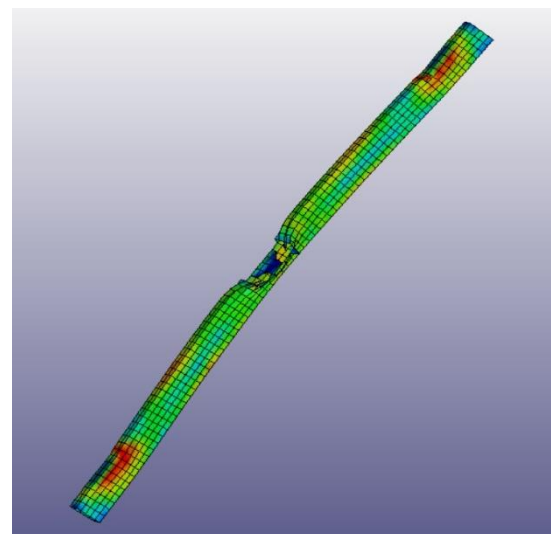
a) 3 ms



b) 5 ms



c) 8 ms



d) 12 ms

Figure B.23: Von mises stress in M-C3 mullion

B.2.4 Increasing the Bracket Thickness (M-C3_10 and M-C3_20)

M-C3 with continuous mullion configuration did not perform well, and its bracket did not have sufficient strength and thickness. Therefore, the same model was used, but the bracket's thickness was increased to 10 mm (M-C3_10) and 20 mm (M-C3_20) to assess the effect of bracket thickness on the retrofitting the mullion connection. The analysis showed that by thickening the brackets less element failure can be achieved, however, a significant local buckling would be expected in the perpendicular side of mullion. In fact, the thickened mullion strengthened the parallel side of mullion but the compression side would suffer buckling.

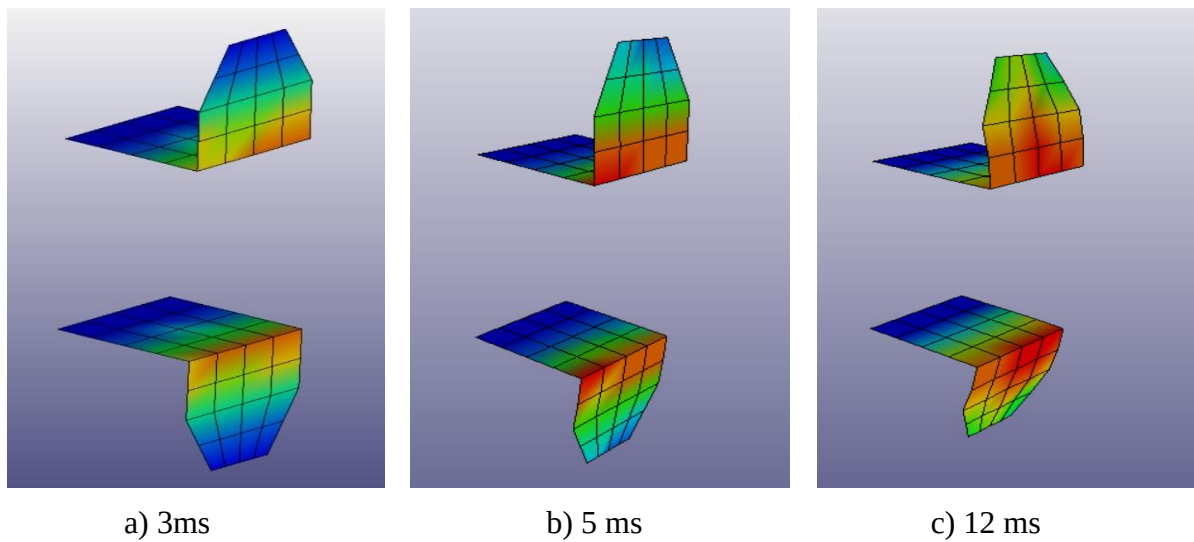


Figure B.24: Von mises stress in the top and bottom brackets of M-C3_10 model

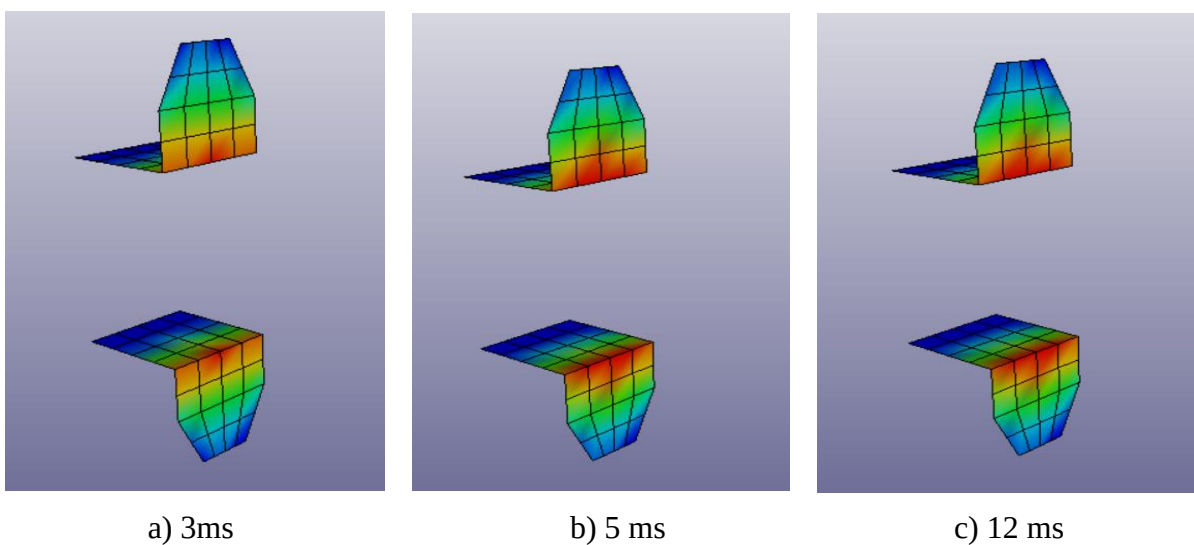


Figure B.25: Von mises stress in the top and bottom brackets of M-C3_20 model

B.2.5 Fourth Single Mullion (M-C4)

This model represented a mullion connected at mid-height over a concrete beam (slab) and connected using brackets attached to the front end of concrete beam (slab) and the two opposite sides (web) of the mullion. M-C4 model performed perfectly without any failure in upper or lower sections. The bracket leg attached to the end of the slab increased the bearing surface area, preventing the crushing of aluminum. Furthermore, the bracket leg attached to the mullion sides strengthened the mullion in the web, which also helped prevent the crushing of aluminum under bearing stresses. The brackets maintained their rigidity, as the rigid concrete slab prevented the bending of the brackets.

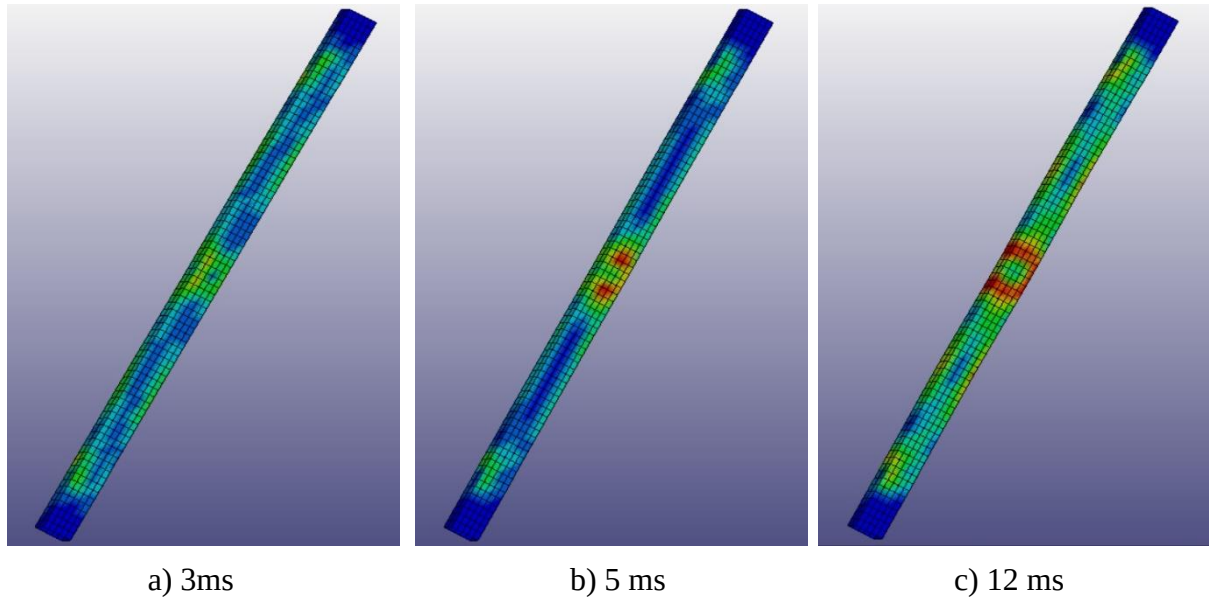


Figure B.26: Von mises stress in the mullion of M-C4 model

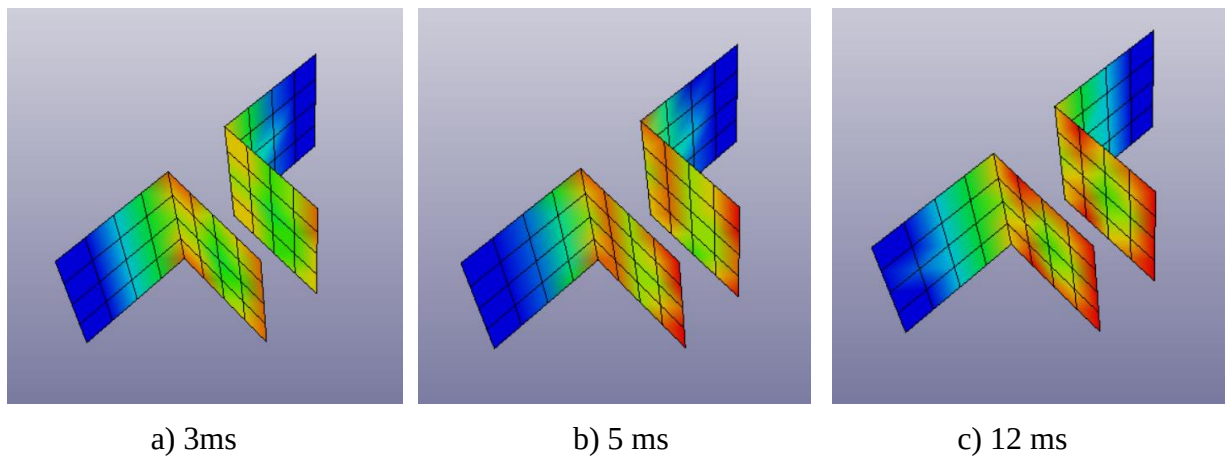


Figure B.27: Von mises stress in the side brackets of M-C4 model

B.3: Third Phase: Full-size Single Mullion with Inserted Energy Absorbing Components

B.3.1 Fifth Single Mullion (M-C5)

The fifth model was analyzed based on continuous mullion configuration. The Poron XRD rubber was inserted between the mullion and middle concrete slab. The inserted rubber was able to roughly damp the energy, but 10 mm thickness was not sufficient to notice a considerable behavior improvement. The mullion and brackets did not suffer failure, and less stress von mises stress was observed in the sections. A very good similarity was observed between the experiment and numerical investigation.

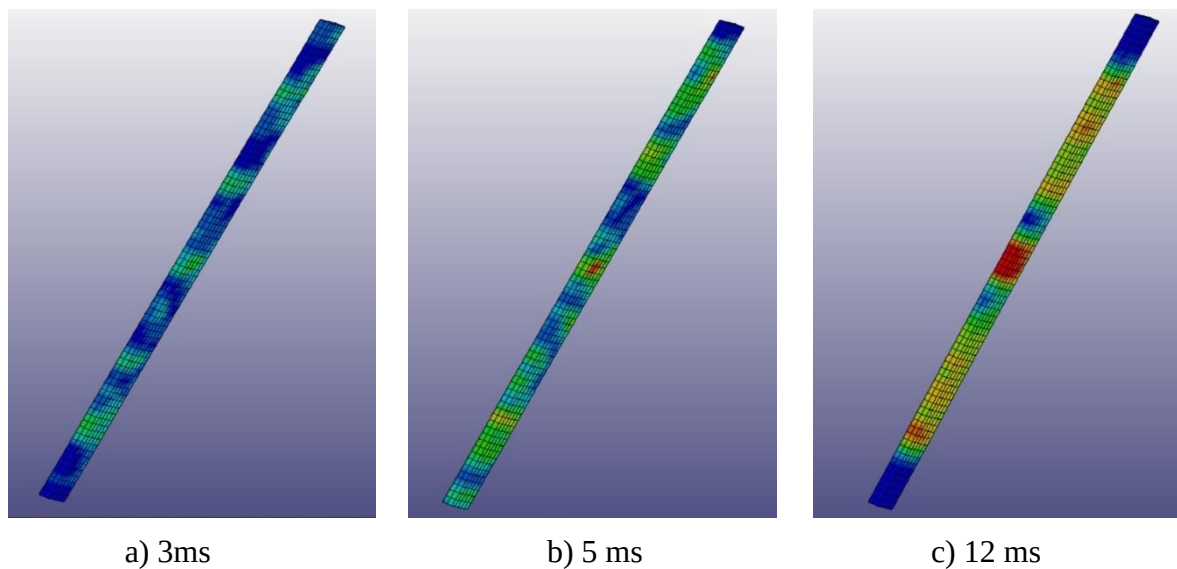


Figure B.28: Von mises stress in the mullion of M-C5 model

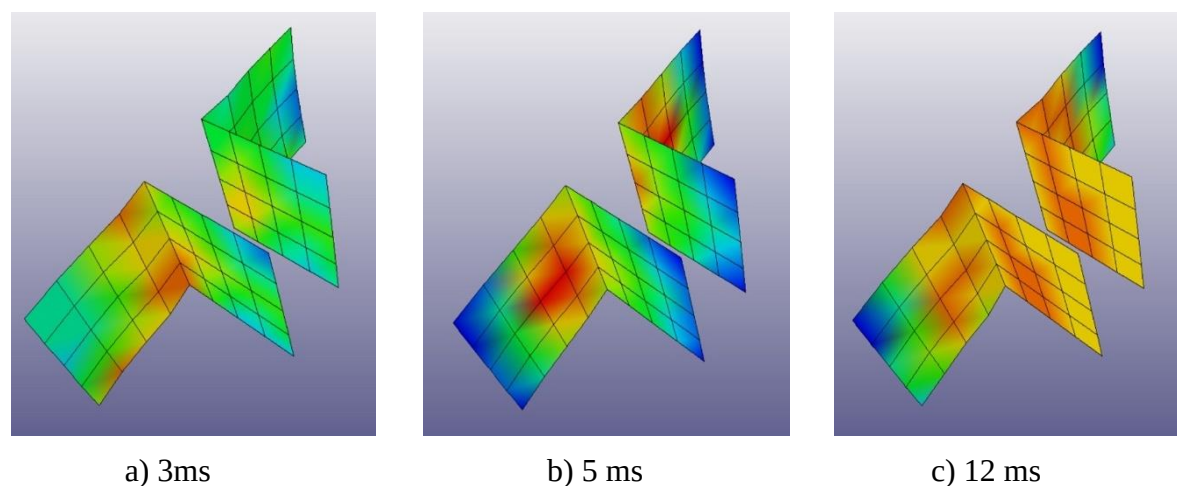


Figure B.29: Von mises stress in the side brackets of M-C5 model

B.3.2 Sixth Single Mullion (M-C6)

The sixth model was assessed based on continuous mullion configuration and steel brackets were placed on the top and bottom of concrete beam. Poron XRD rubber was inserted between the mullion, brackets, and concrete beams. In addition, rubbers were inserted between the mullion and at the ends of mullion where boundaries were defined. Similar to M-C3, the mullion suffered from connection failure when the brackets were bent excessively, reducing the original 90-degree angle to approximately 50 degrees. Clearly, this kind of mullion-to-concrete slab connection did not fulfill its function as expected.

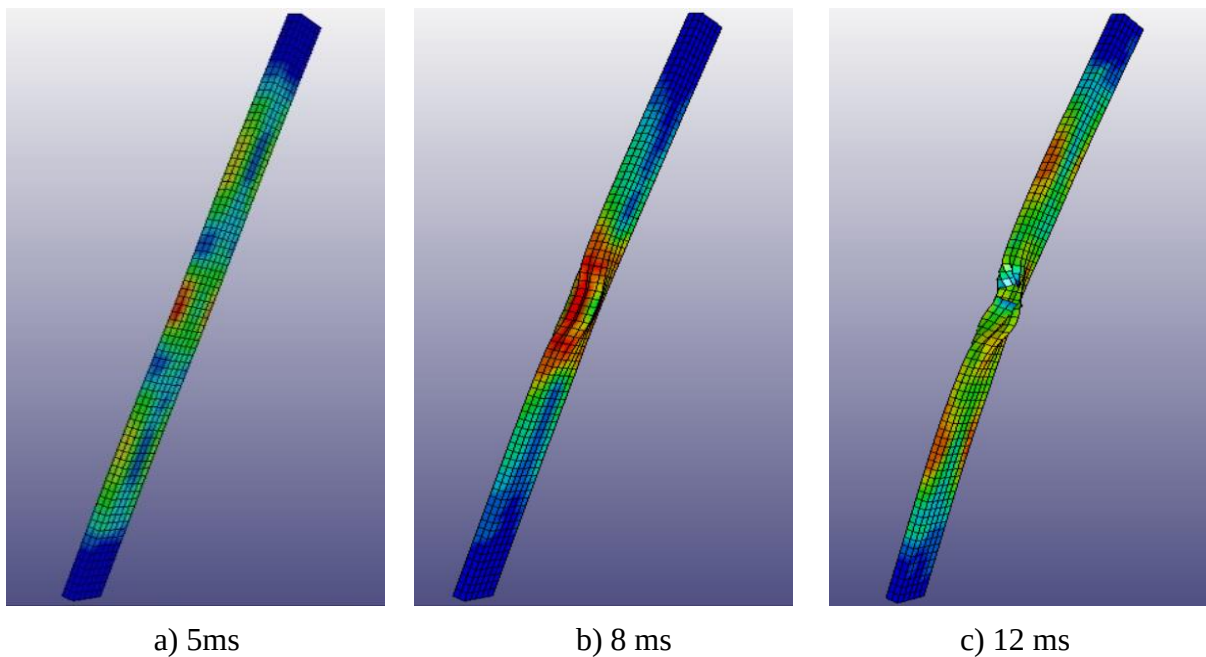


Figure B.30: Von mises stress in the mullion of M-C6 model

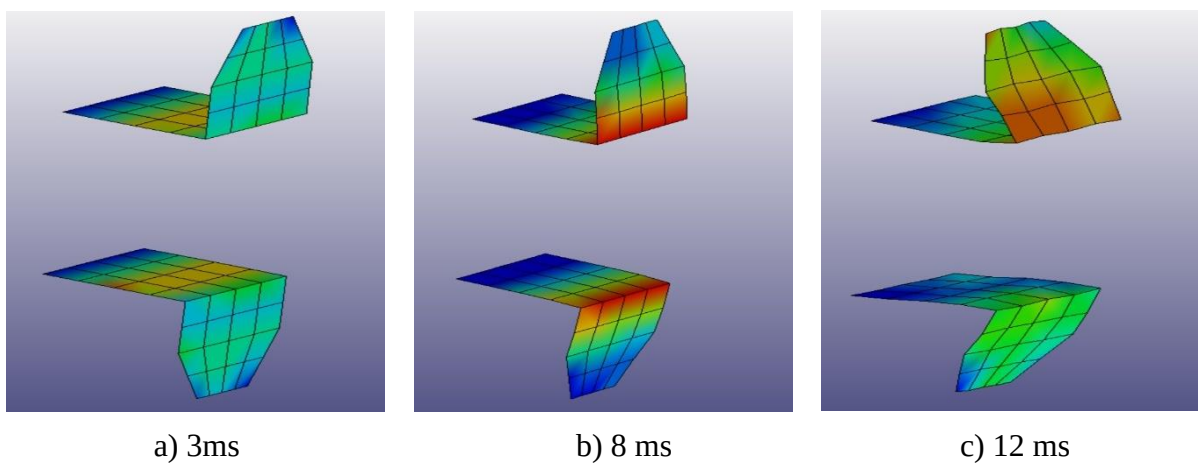
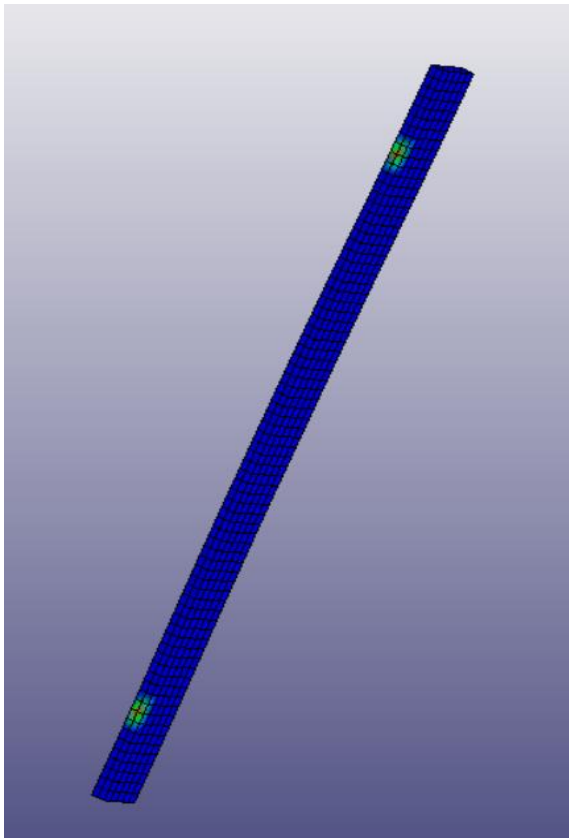


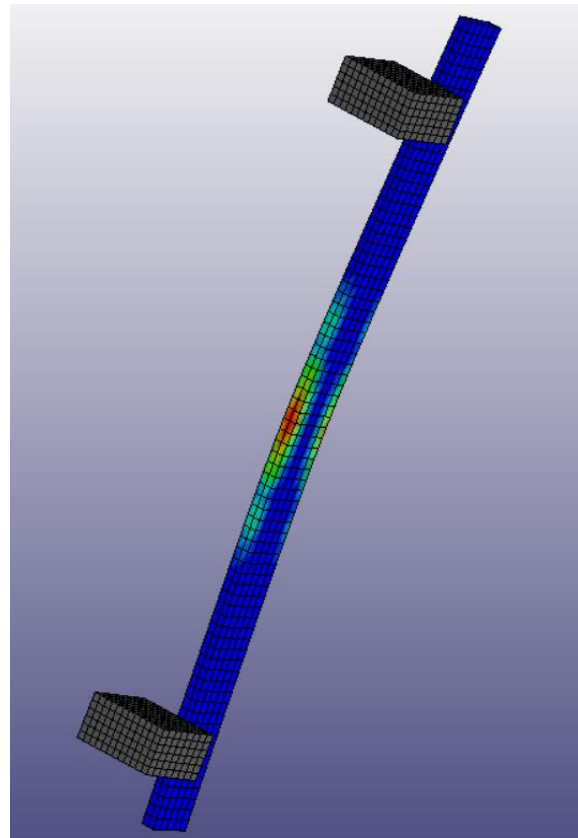
Figure B.31: Von mises stress in the side brackets of M-C6 model

B.3.3 Seventh Single Mullion (M-C7)

The seventh Mullion had spring at the ends of mullion and it was examined based on single mullion configuration. The spring stiffness was 8 N/mm with 160 mm net length for mullion to move along the applied load. The model showed an excellent performance since mullion was able to squeeze the springs but did not suffer failure either at mid-height or boundaries.



a) When mullion touched the beams



b) The maxim mullion deflection

Figure B.32: The developed effective plastic strain in M-C7 after applying the test

B.3.4 Eighth Single Mullion (M-C8)

The eight single mullion model had springs between the ends of mullion and concrete slabs. The spring had 30 N/mm stiffness, and 160 mm net length was considered for M-C8 similar to the experiment. By applying the load, the mullion squeezed the springs, however, the spring's stiffness and net length was not sufficient to save the mullion from failure. A failure occurred at the mid-height of mullion similar to what was observed in the actual test.

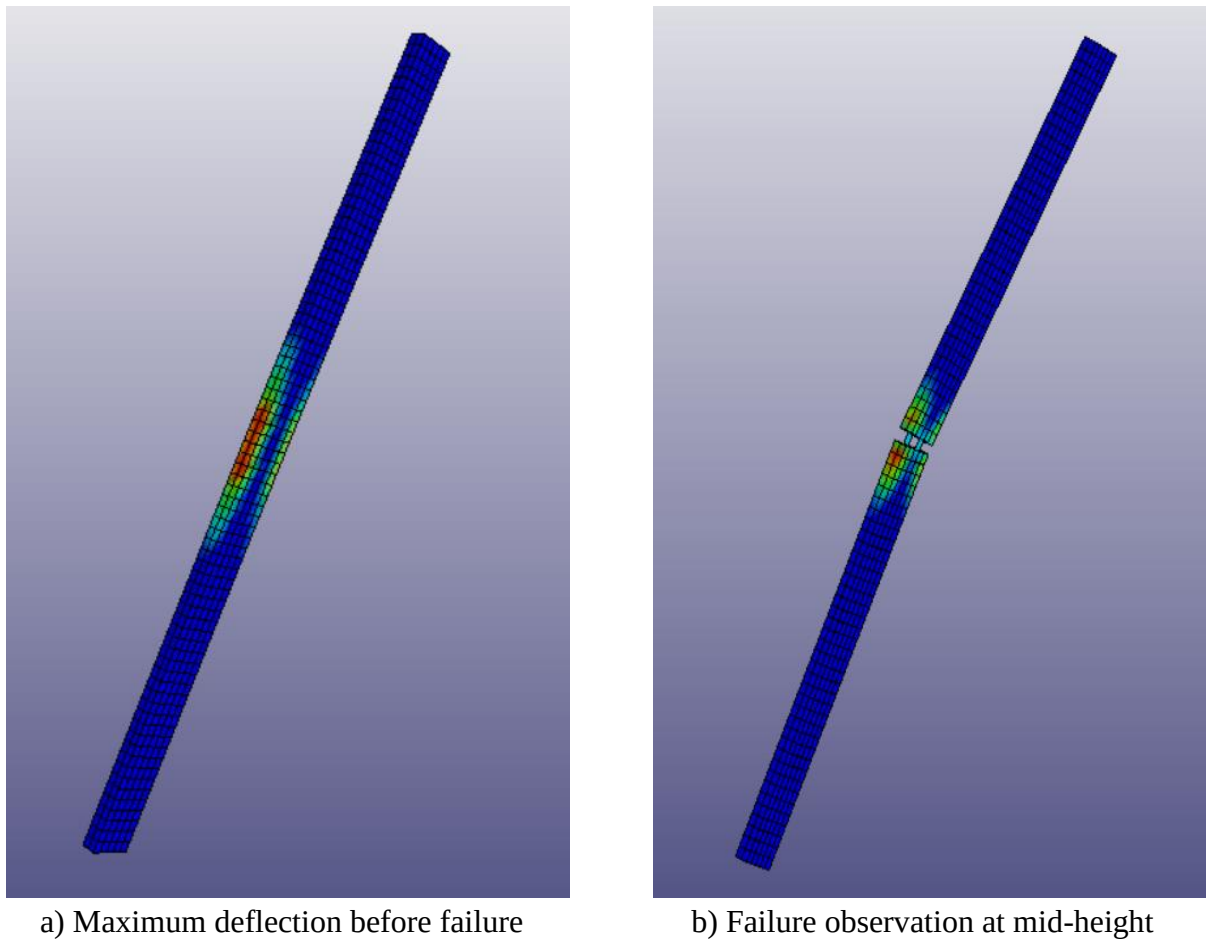


Figure B.33: The developed effective plastic strain in M-C8 after applying the test

B.3.5 Ninth Single Mullion (M-C9)

The ninth mullion had spring at the ends of mullion and it was examined based on single mullion configuration. The spring stiffness was 23 N/mm with 200 mm net length for mullion to move along the applied load. The model showed an excellent performance since mullion was able to squeeze the springs but did not suffer failure either at mid-height or boundaries. Nonetheless, a permanent bending was observed in the M-C9. This model to prove the concept of inserting energy absorbing components in the curtain wall system and validate the numerical results against the experimental outcomes.

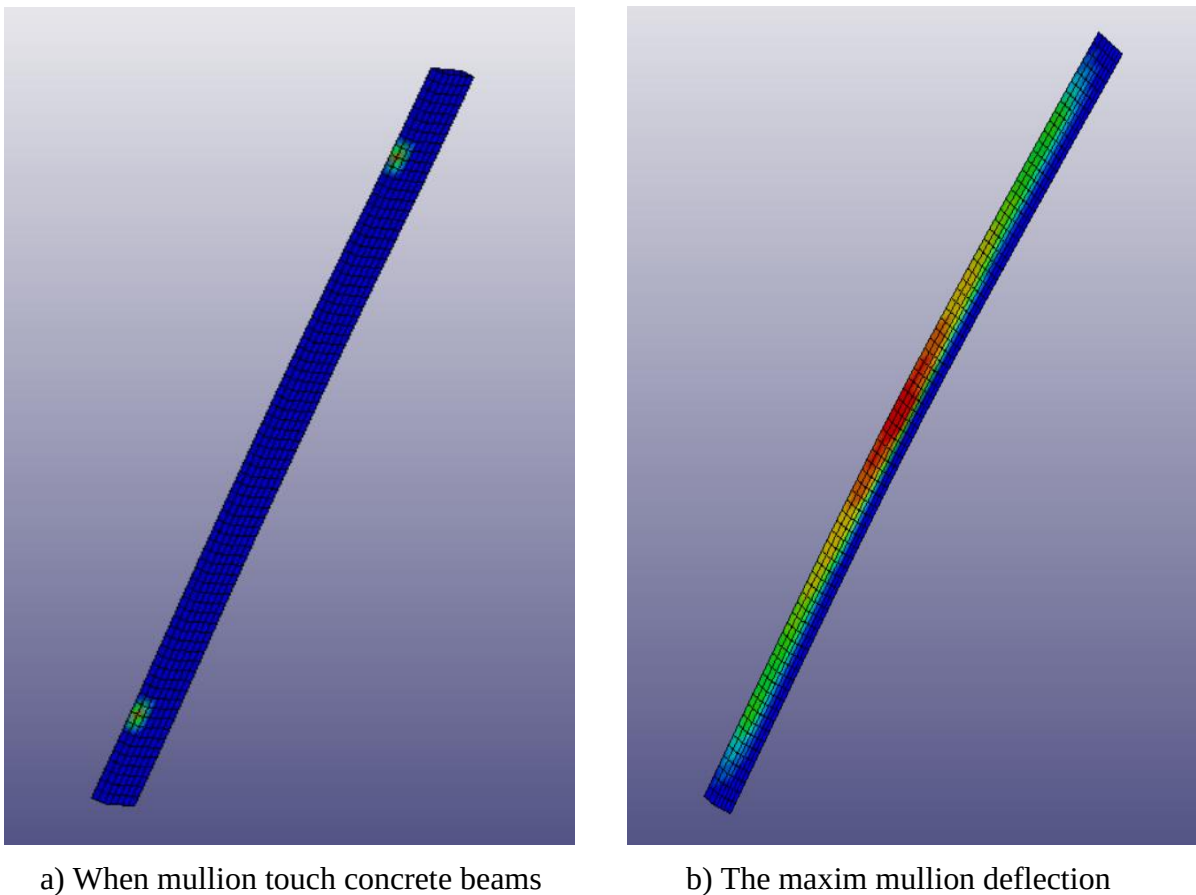


Figure B.34: The developed effective plastic strain in M-C9 after applying the test

B.3.6 Improving the Level of Protection by Using Damper (SP-100 and XRD-100)

Two parametric studies were conducted to compare the differences between response limit and support rotation (θ) by using rubber and spring based on CSA S-850-12 standard. Firstly, M-C9 model was reused, but spring stiffness was increased to 100 N/mm (SP-100). Secondly, again M-C9 was reused, but Poron XRD rubbers with 150 mm $\times$ 100 mm section size were replaced. The strain-stress curve for crushable foam was modified to provide 100 N/mm stiffness (XRD-100). The thickness of 260 mm was considered for rubber to have 200 mm potential net length considering the rubber volume and its secondary stiffness hardening.

After applying 4 shots on the models, it can be summarized that both spring and XRD rubber have proper potential to damp the applied load and provide significant level of protection improvement, however, a few behavior differences were noticed. The elastic behavior of spring was able to push the mullion back to the initial location after each shot that is an ideal scenario. Nonetheless, the rubber did not act elastically and could not push the mullion back. Consequently, the mullion stayed on rubbers since Poron XRD acts like a memory form. The ideal performance is what spring demonstrated because there would not be a gap between the mullion and existing façade after any blast incidence, whereas by using rubber, the façade structure would require adjustment, repair etc. However, it should be noted that no damage or failure was observed in the models using Poron XRD since rubber provided better stress distribution on the system, especially on the boundaries. Furthermore, a linear relationship exists between mullion translation and dampers' stiffness, however, by increasing the applied load and stiffness simultaneously, higher stress was observed on the mullion body.

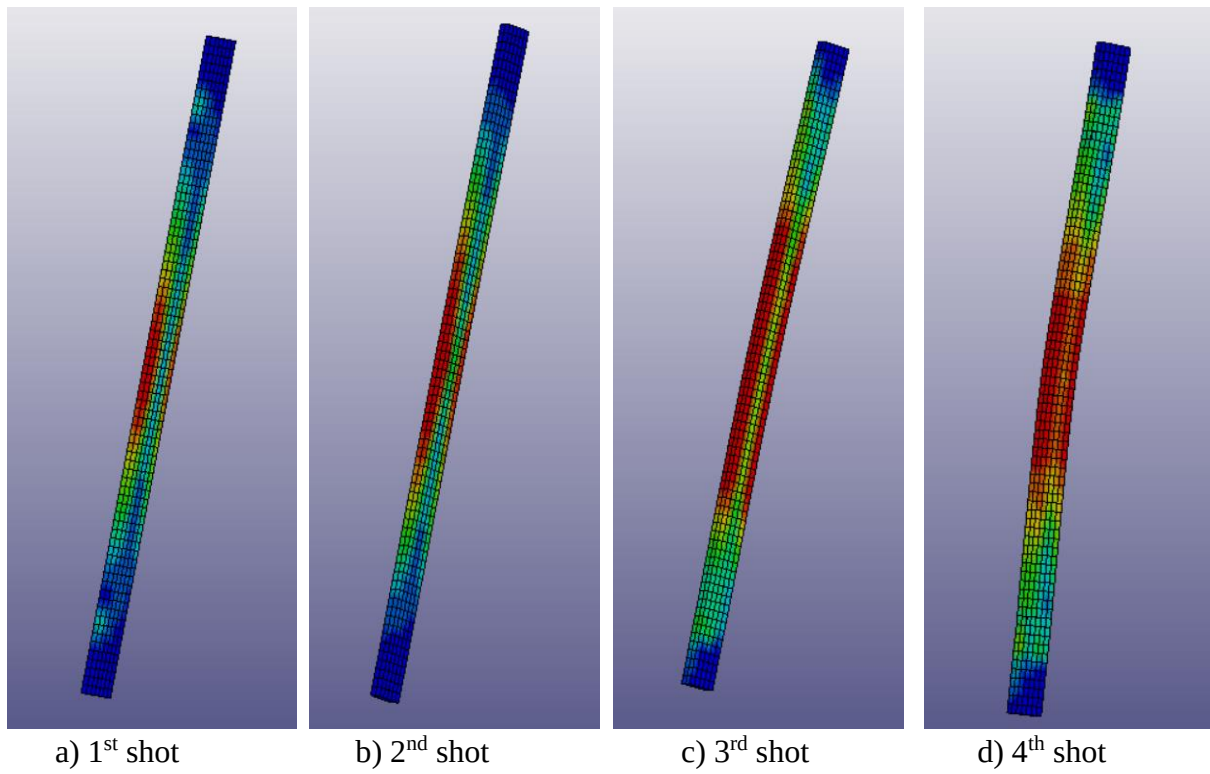


Figure B.35: Von mises stress in SP-100

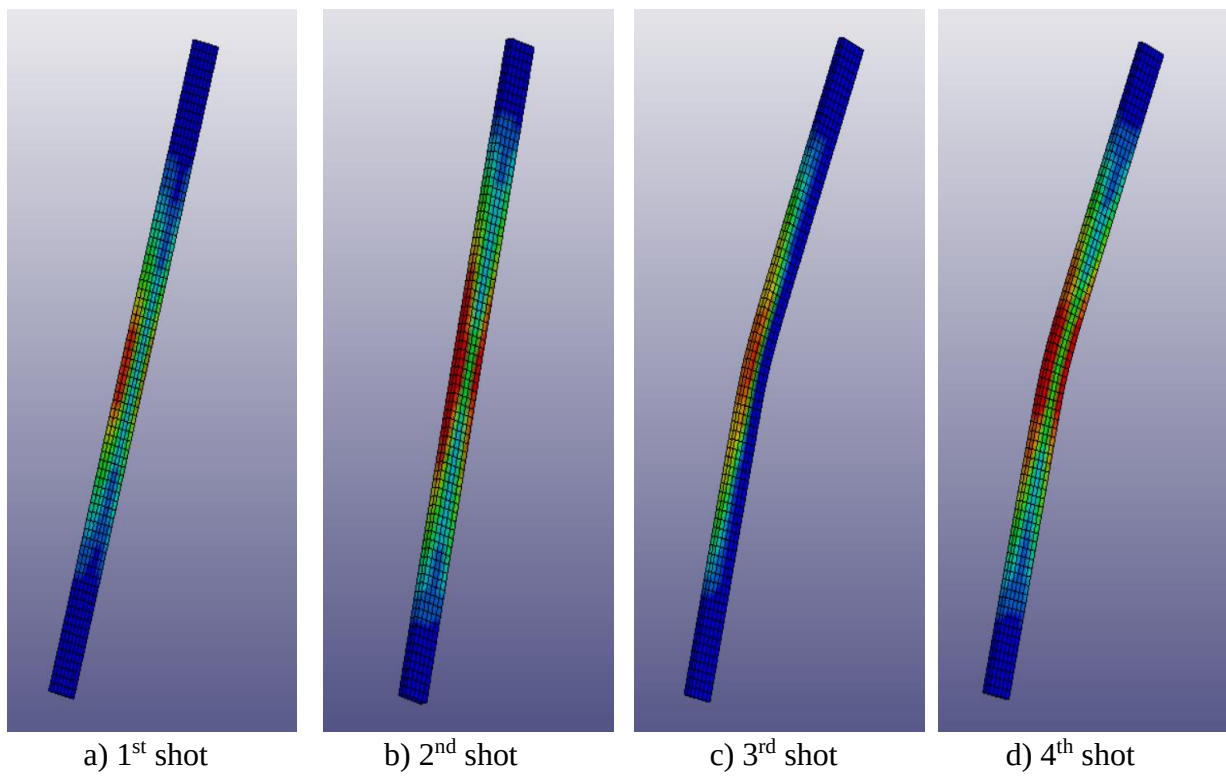


Figure B.36: Von mises stress in XRD-100

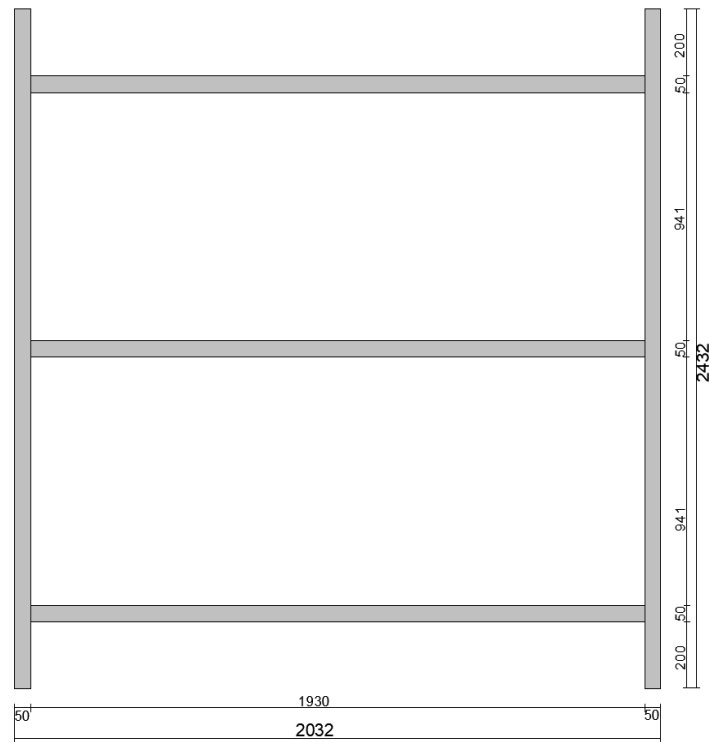
Appendix C UFC Charted Approach

Development of Resistance Functions UFC Charted Solution

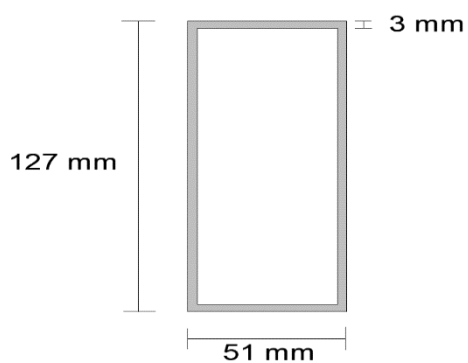
C.1 Resistance Function for First Curtain Wall

C.1.1 Resistance Function for Simple Supports

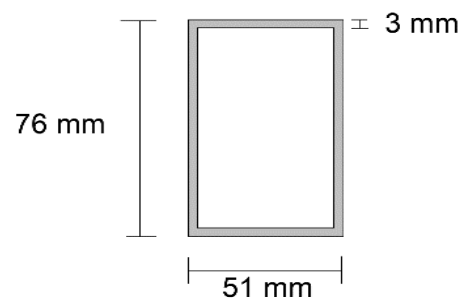
The cross-section details of first curtain wall are given as follows



a) Full size curtain wall sketch



b) Aluminum mullion



c) Aluminum transom

Figure C.1: First Curtain wall cross-section details (mm)

The numerical values of the properties for establishing resistance function are as follows:

Yield Strength of Aluminum (F_y) = 275 MPa

Modulus of Elasticity (E_A) = 70000 N/mm²

Mass Density of Aluminum = 2700 kg/m³

Length of Tributary Area (L) = 2032 mm

Width of tributary area (W) = 2032 mm

Total mass of system = 2 vertical mullion + 3 horizontal transom + steel sheet

= (2 × 6.8) + (3 × 3.8) + 32 = 57 kg

Moment of Inertia per mullion = 2,062,274 mm⁴

Stiffness of two mullions for simply supported condition (hence multiplied by 2.0):

$$K = \frac{2 \times 384 \times 70000 \times 2062274}{5 \times 2032^3} = 2643 \frac{N}{mm}$$

Moment Capacity of section:

$$M^- = M^+ = \frac{(fy)I}{y}$$

Here, M^- and M^+ are negative and positive moment capacities. In this equation, 'y' is the maximum distance to the tension fibers from the neutral axis which is half of the section depth (127/2 = 63.5 mm).

Hence, substituting the above calculated values in the moment equation,

$$M = \frac{2 \times 275 \times 2062247}{63.5} = 17,861,982 \text{ N.mm} = 17.86 \text{ kN.m}$$

Elastic resistance can be calculated as:

$$R_m = \frac{8 M}{L} = \frac{8 \times 17.86}{2.03} = 70.2 \text{ kN}$$

Displacement at yield can be calculated as:

$$U_y = \frac{R_m}{K_s} = \frac{70.2 \times 1000}{2643} = 26 \text{ mm}$$

Check performance under the pressure-impulse combination applied during the test:

P = 50 kPa (recorded); Ir = 500 kPa-ms (recorded); td = 20 ms (equivalent duration)

✚ Period of the mullions:

$$T_s = 2\pi \sqrt{\frac{0.78 \times 57}{2.64 \times 10^6}} = 0.026 \text{ s} = 26 \text{ ms}$$

$$T_d = \frac{2 I_r}{P_r} = \frac{2 \times 500}{50} = 20 \text{ ms} \quad \frac{T_d}{T_s} = \frac{20}{26} = 0.77 \quad \text{for } \mu = 1.0 \quad \text{read} \quad \frac{R_m}{F_1} = 1.35$$

$$F_1 = P_1 (\text{Tributary Area}) = 50 \times 2.03 \times 2.03 = 206 \text{ kN}$$

$$R_m (\text{demand}) = 1.35 \times 206 = 278 \text{ kN} > R_m (\text{capacity}) = 70.2 \text{ kN}$$

Therefore, the mullions as is (without any retrofit) will behave in the inelastic range.

Use $K_{LM} = 0.70$ since significant inelasticity is anticipated (a value closer to $K_{LM} = 0.66$). $T_s =$

$$2\pi \sqrt{\frac{0.7 \times 57}{2.64 \times 10^6}} = 0.025 \text{ s} = 25 \text{ ms} \quad \frac{T_d}{T_s} = \frac{20}{25} = 0.8 \quad ; \quad \frac{R_m}{F_1} = 0.34 \quad \text{read} \rightarrow \mu = 18$$

Considering $\mu = 18 \rightarrow$ UFC chart approach: $18 \times 26 = 468 \text{ mm}$

$$\text{Damping} = 0.02 \times 2\sqrt{K \cdot M} = 491 \text{ Ns/m}$$

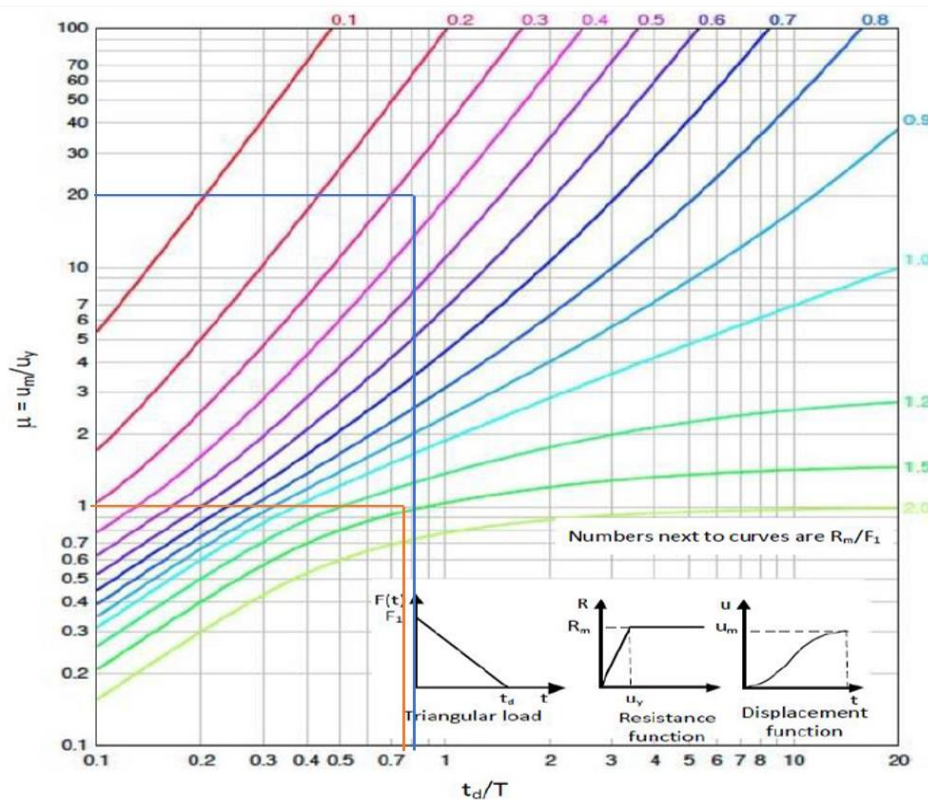


Figure C.2: UFC chart maximum response of elastoplastic SDOF systems (triangular pulses)

C.1.2 Elastic Resistance Reaction for First Curtain Wall with Simple Supports

Distributed Mass: $57 \times 0.78 = 45.3 \text{ kg}$

Moment of Inertia per mullion = $2,062,274 \text{ mm}^4$

$$K = \frac{2 \times 384 \times 70000 \times 2062274}{5 \times 2032^3} = 2643 \frac{N}{mm} = 2.64 \frac{N}{m}$$

$$T_s = 2\pi \sqrt{\frac{45.3}{2.64 \times 10^6}} = 0.026 \text{ s} = 26 \text{ ms}$$

$$T_d = \frac{2 I_r}{P_r} = \frac{2 \times 500}{50} = 20 \text{ ms}$$

$$\frac{T_d}{T} = \frac{20}{26} = 0.77$$

Dynamic Load Factor (DLF) = 1.45

Exposed specimen area to the blast load: $2032 \text{ mm} \times 2032 \text{ mm} = 4,129,024 \text{ mm}^2$

Elastic Force Resistance: $0.05 \frac{N}{mm^2} \times 4,129,024 \text{ mm}^2 = 206,451 \text{ N}$

Dynamic Resistance: $1.45 \times 206,451 \text{ N} = 299,354 \text{ N}$

Dynamic Reaction (V): $0.39 R + 0.11 F$

Dynamic Reaction = $(0.39 \times 299,354 \text{ N}) + (0.11 \times 206,451 \text{ N}) = 139,458 \text{ N}$

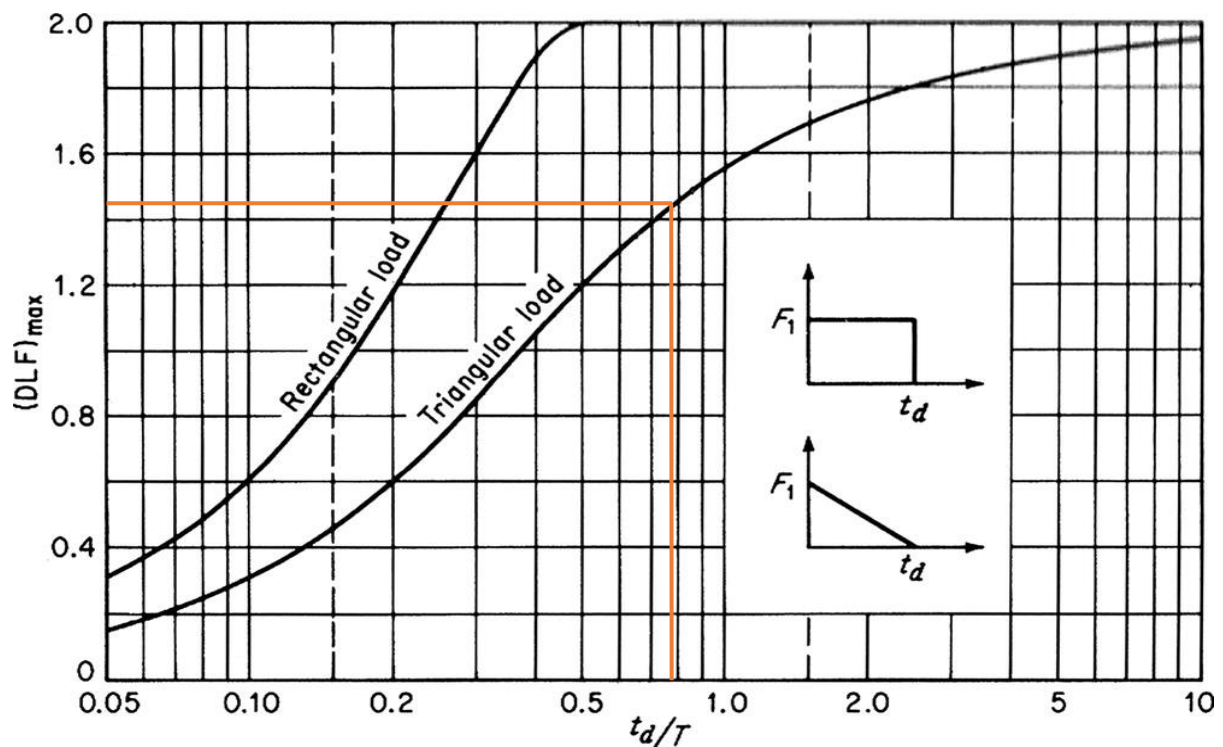


Figure C.3: Dynamic Load Factor graph for Triangular and Rectangular Loads

C.1.3 Plastic Strain Range Resistance Reaction for First Curtain Wall with Simple Supports

Distributed Mass: $57 \times 0.66 = 37.6 \text{ kg}$

Moment of Inertia per mullion = $2,062,274 \text{ mm}^4$

$$K = \frac{2 \times 384 \times 70000 \times 2062274}{5 \times 2032^3} = 2643 \frac{N}{mm} = 2.64 \frac{N}{m}$$

$$T_s = 2\pi \sqrt{\frac{37.6}{2.64 \times 10^6}} = 0.024 \text{ s} = 24 \text{ ms}$$

$$T_d = \frac{2 I_r}{P_r} = \frac{2 \times 500}{50} = 20 \text{ ms}$$

$$\frac{T_d}{T} = \frac{20}{24} = 0.83$$

Dynamic Load Factor (DLF) = 1.5

Exposed specimen area to the blast load: $2032 \text{ mm} \times 2032 \text{ mm} = 4,129,024 \text{ mm}^2$

Elastic Force Resistance: $0.05 \frac{N}{mm^2} \times 4,129,024 \text{ mm}^2 = 206,451 \text{ N}$

Dynamic Resistance: $1.5 \times 206,451 \text{ N} = 307,612 \text{ N}$

Dynamic Reaction (V): $0.38 R + 0.12 F$

⇒ Dynamic Reaction = $(0.38 \times 307,612 \text{ N}) + (0.12 \times 206,451 \text{ N}) = 141,667 \text{ N}$

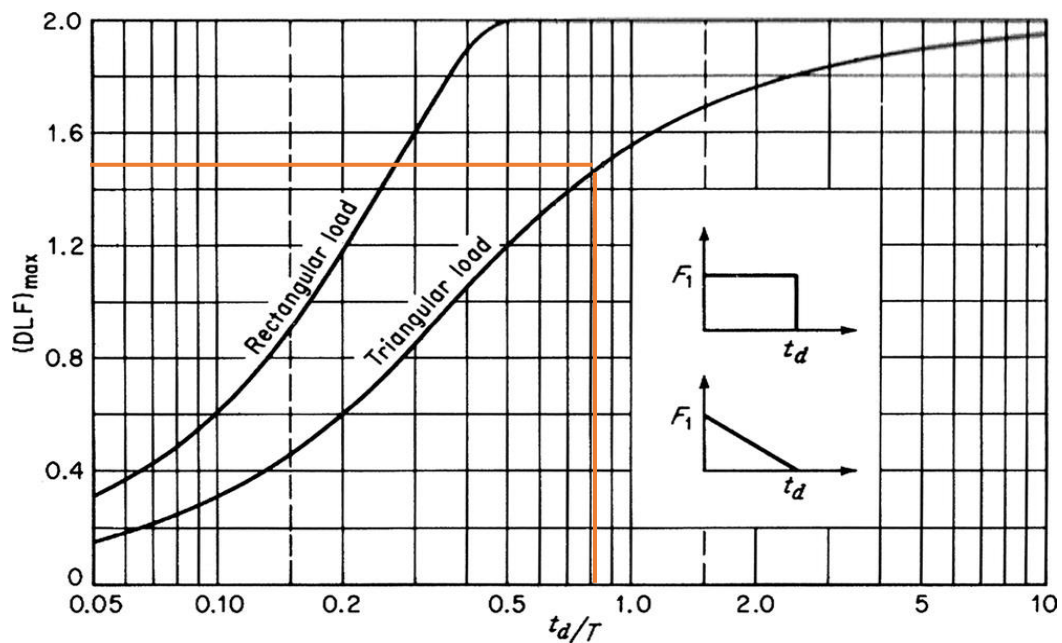
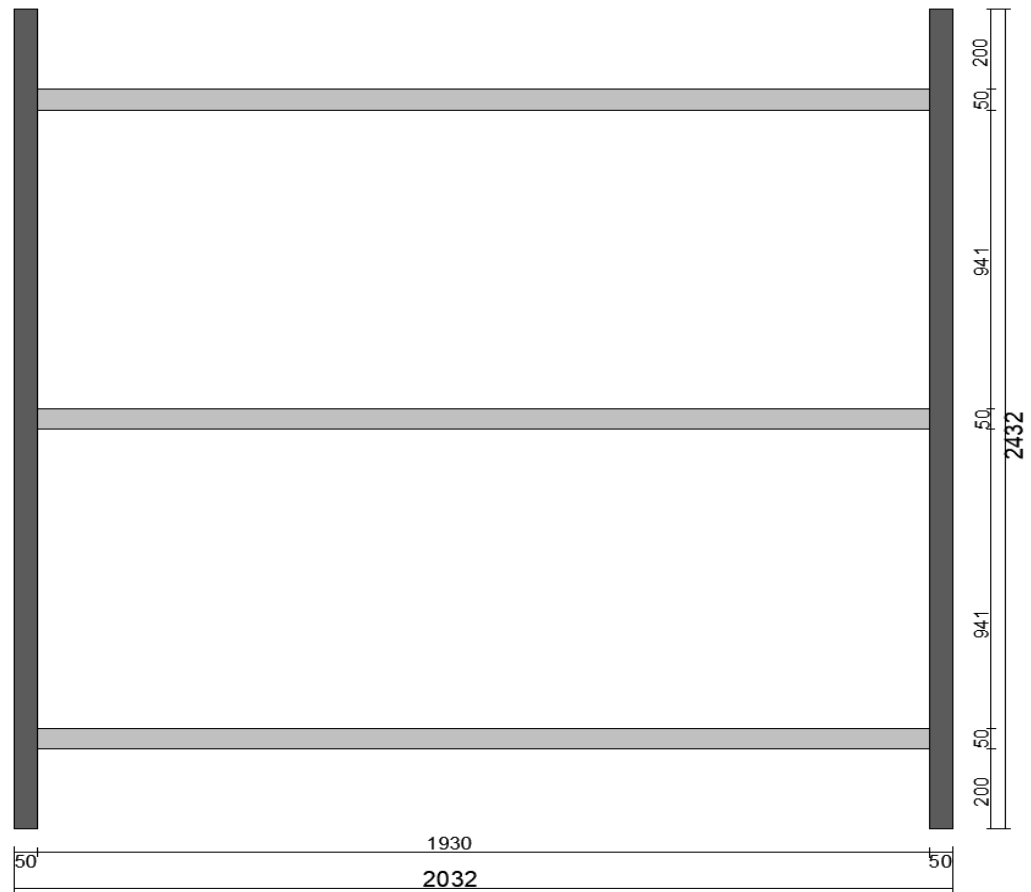


Figure C.4: Dynamic Load Factor graph for Triangular and Rectangular

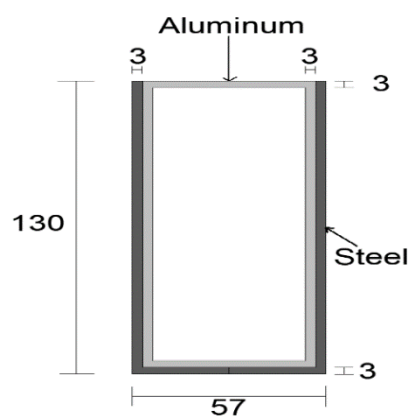
C.2 Resistance Function for Second Curtain Wall

C.2.1 Resistance Function for Simple Supports

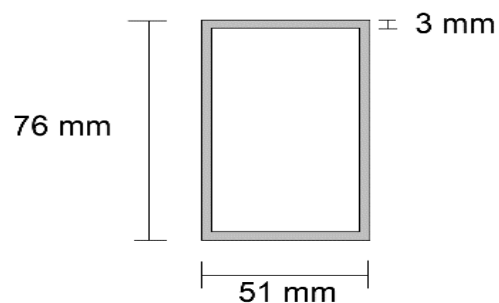
The cross-section details of second curtain wall (retrofitted mullions) are given as follows,



a) Full size curtain wall sketch



b) Hardened Mullion



c) Aluminum transom

Figure C.5: Second Curtain wall cross-section details (mm)

The numerical values of the properties needed for establishing resistance function are given as follows:

$$\text{Modulus of Elasticity of aluminum } (E_A) = 70,000 \frac{N}{mm^2}$$

$$\text{Modulus of Elasticity of steel } (E_s) = 200000 \frac{N}{mm^2}$$

$$\text{Density of steel} = 7850 \frac{kg}{m^3}$$

$$\text{Yield Strength of steel } (f_y) = 350 \text{ MPa} \quad ; \quad \text{Yield strength of aluminum} = 275 \text{ MPa}$$

$$\text{Length of tributary area (L)} = 2032 \text{ mm}$$

$$\text{Width of tributary area (W)} = 2032 \text{ mm}$$

$$\begin{aligned} \text{Total mass of second system} &= 2 \text{ mullion} + 3 \text{ transoms} + \text{steel sheet} + 4 \text{ retrofitting plates} = \\ &(2 \times 6.8) + (3 \times 3.8) + 32 + (4 \times 7.5) = 87 \text{ kg} \end{aligned}$$

Calculation of transformed M.I for the composite section:

$$\text{Transformation factor to convert aluminum into equivalent steel section: } n = \frac{E_A}{E_s} = 0.35$$

$$\text{Area of aluminum mullion} = 1032 \text{ mm}^2$$

$$\text{Transformed area of aluminum to steel section} = 0.35 \times 1032 = 361.2 \text{ mm}^2$$

$$\text{Moment of Inertia per mullion} = 2,062,274 \text{ mm}^4$$

$$\text{Transformed M.I of aluminum to steel section} = 0.35 \times 2062274 = 721,796 \text{ mm}^4$$

$$\text{Area of each L-shape steel retrofit plate} = (130 \times 3) + (28 \times 3) = 474 \text{ mm}^2$$

$$\text{Moment of inertia for each L-shape steel retrofit plate} = 549,304 \text{ mm}^4$$

Distance of the neutral axis from the extreme bottom (tension) fibers,

$$Y = 60.1 \text{ mm}$$

Moment of inertia for the transformed section = mullion + 2 retrofitting plate

$$I_{TR} = [721,796 + (361.2 \times 6.4^2)] + 2 [549304 + (462 \times 58.6^2)] = 5,008,178 \text{ mm}^4$$

Stiffness of two mullions for simply supported condition (hence multiplied by 2.0):

$$K = \frac{2 \times 384 \times 200000 \times 5008178}{5 \times 2032^3} = 18337 \frac{N}{mm}$$

Moment capacity of two vertical mullions (hence multiplied by 2):

$$M^- = M^+ = \frac{(fy) I}{y}$$

Here, M^- and M^+ are negative and positive moment capacities. Therefore, 'y' is the maximum distance to the tension fibers from the neutral axis. Thus, substituting the above calculated values in the moment equation,

$$M = \frac{2 \times 350 \times 5008178}{60.1} = 58,331,524 \text{ N.mm} = 58.331 \text{ kN.m}$$

Elastic Resistance of mullion,

$$R_m = \frac{8 M}{L} = \frac{8 \times 58.331}{2.03} = 230 \text{ kN}$$

Displacement at yield can be calculated as:

$$U_y = \frac{R_m}{K_s} = \frac{230 \times 1000}{18337} = 12.6 \text{ mm}$$

Check performance under the pressure-impulse combination applied during the test:

P = 36 kPa (recorded); Ir = 270 kPa-ms (recorded); td = 15 ms (equivalent duration)

Where K_{LM} is the load mass factor; $K_{LM} = 0.78$ for elastic response and $K_{LM} = 0.7$ for plastic response if entered to the plastic region.

$$T_s = 2\pi \sqrt{\frac{0.78 \times 87}{18.3 \times 10^6}} = 0.012 \text{ s} = 12 \text{ ms}$$

From the UFC Chart shown on the below:

$$T_d = \frac{2 I_r}{P_r} = \frac{2 \times 270}{36} = 15 \text{ ms}$$

$$\frac{T_d}{T_s} = \frac{15}{12} = 1.2 \quad \text{for } \mu = 1.0 \quad \text{read} \quad \frac{R_m}{F_1} = 1.55$$

$$F_1 = P_1 (\text{Tributary Area}) = 36 \times 2.03 \times 2.03 = 148 \text{ kN}$$

$$R_m (\text{demand}) = 1.55 \times 148 = 229 \text{ kN} = R_m (\text{capacity}) = 230 \text{ kN}$$

$$\text{Damping} = 0.02 \times 2\sqrt{K \cdot M} = 0.02 \times 2 \times \sqrt{18337000 \times 87} = 1598 \text{ Ns/m}$$

Considering $\mu = 1.0 \rightarrow$ UFC chart approach: $1 \times 12.6 = 12.6 \text{ mm}$

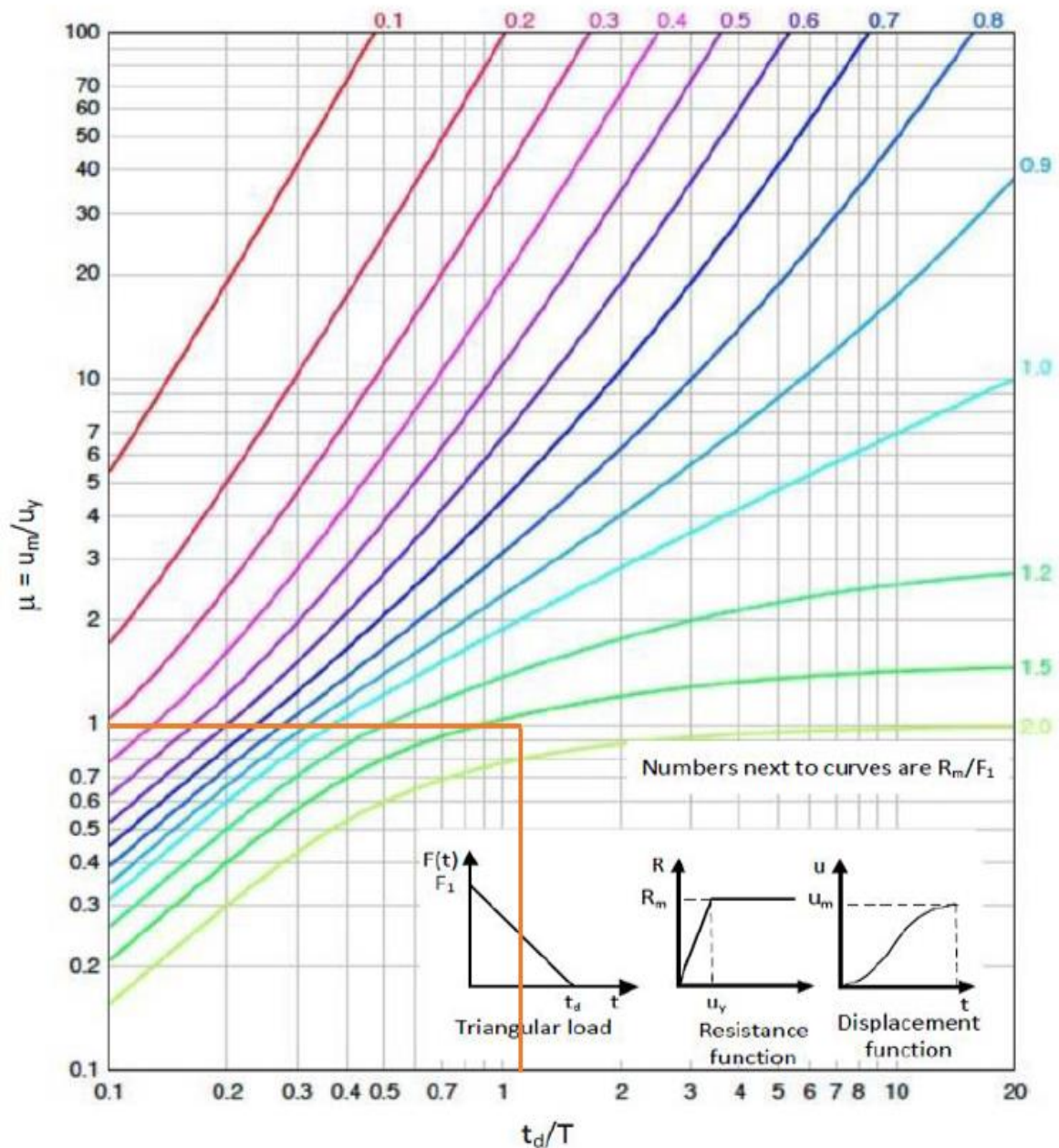


Figure C.6: UFC chart Maximum response of elastoplastic SDOF systems (triangular pulses)

C.2.2 Elastic Resistance Reaction for Second Curtain Wall with Simple Supports

Distributed Mass: $87 \times 0.78 = 67.9 \text{ kg}$

Moment of Inertia per mullion = $5,008,178 \text{ mm}^4$

$$K = \frac{2 \times 384 \times 200000 \times 5008178}{5 \times 2032^3} = 18337 \frac{N}{mm} = 18.337 \frac{N}{m}$$

$$T_s = 2\pi \sqrt{\frac{67.9}{18.33 \times 10^6}} = 0.012 \text{ s} = 12 \text{ ms}$$

$$T_d = \frac{2 I_r}{P_r} = \frac{2 \times 270}{36} = 15 \text{ ms}$$

$$\frac{T_d}{T} = \frac{15}{12} = 1.2$$

Dynamic Load Factor (DLF) = 1.55

Exposed specimen area to the blast load: $2032 \text{ mm} \times 2032 \text{ mm} = 4,129,024 \text{ mm}^2$

Elastic Force Resistance: $0.036 \frac{N}{mm^2} \times 4,129,024 \text{ mm}^2 = 148,645 \text{ N}$

Dynamic Resistance: $1.55 \times 148,645 \text{ N} = 230,400 \text{ N}$

Dynamic Reaction (V): $0.39 R + 0.11 F$

$$\Rightarrow \text{Dynamic Reaction} = (0.39 \times 230,400 \text{ N}) + (0.11 \times 148,645 \text{ N}) = 106,207 \text{ N}$$

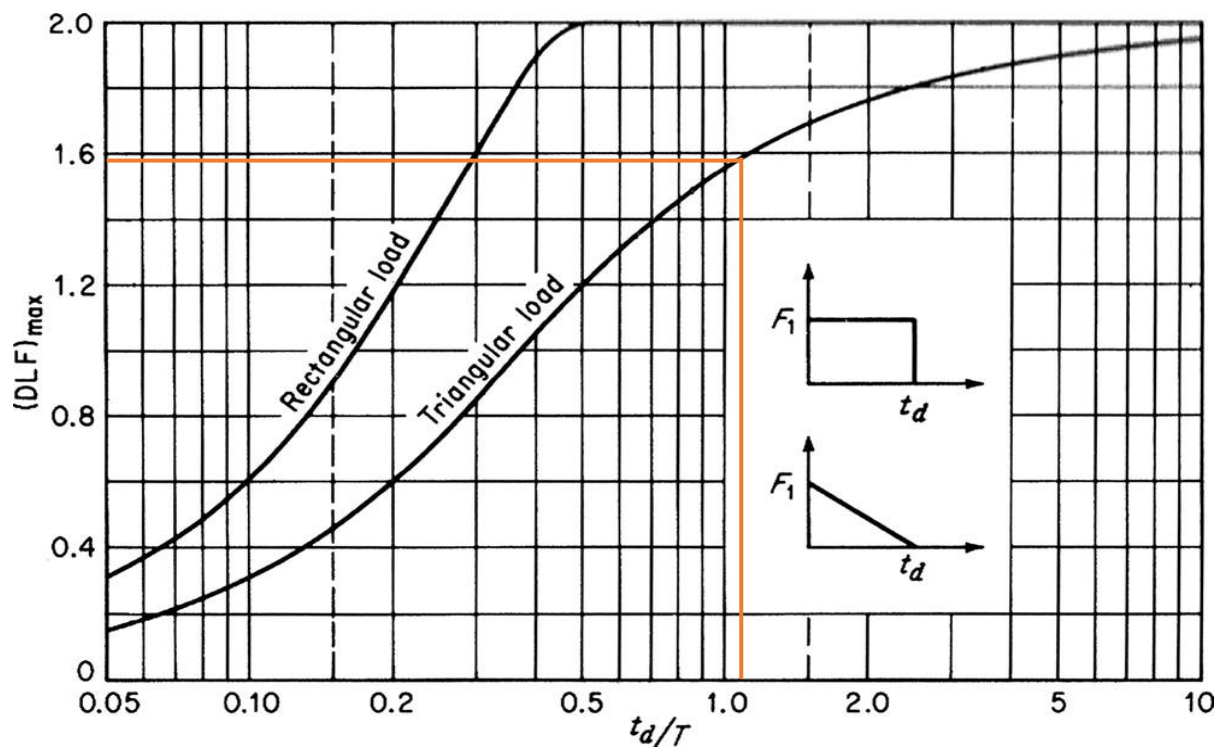


Figure C.7: Dynamic Load Factor graph for Triangular and Rectangular Loads

C.2.3 Plastic Resistance Reaction for Second Curtain Wall with Simple Supports

Distributed Mass: $87 \times 0.66 = 57.4 \text{ kg}$

Moment of Inertia per mullion = $5,008,178 \text{ mm}^4$

$$K = \frac{2 \times 384 \times 200000 \times 5008178}{5 \times 2032^3} = 18337 \frac{N}{mm} = 18.337 \frac{N}{m}$$

$$T_s = 2\pi \sqrt{\frac{57.4}{18.33 \times 10^6}} = 0.011 \text{ s} = 11 \text{ ms}$$

$$T_d = \frac{2 I_r}{P_r} = \frac{2 \times 270}{36} = 15 \text{ ms}$$

$$\frac{T_d}{T} = \frac{15}{11} = 1.36$$

Dynamic Load Factor (DLF) = 1.65

Exposed specimen area to the blast load: $2032 \text{ mm} \times 2032 \text{ mm} = 4,129,024 \text{ mm}^2$

Elastic Force Resistance: $0.036 \frac{N}{mm^2} \times 4,129,024 \text{ mm}^2 = 148,645 \text{ N}$

Dynamic Resistance: $1.65 \times 148,645 \text{ N} = 245,264 \text{ N}$

Dynamic Reaction (V): $0.38 R + 0.12 F$

$$\Rightarrow \text{Dynamic Reaction} = (0.38 \times 245,264 \text{ N}) + (0.12 \times 148,645 \text{ N}) = 111,038 \text{ N}$$

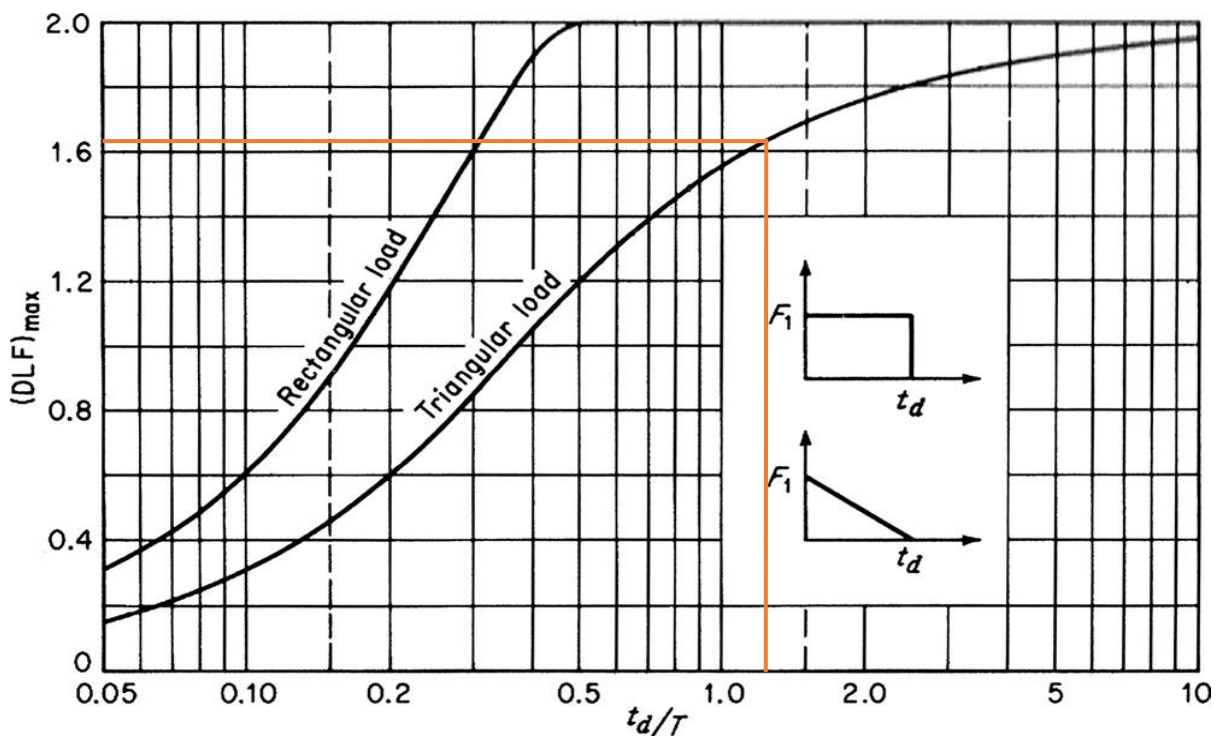
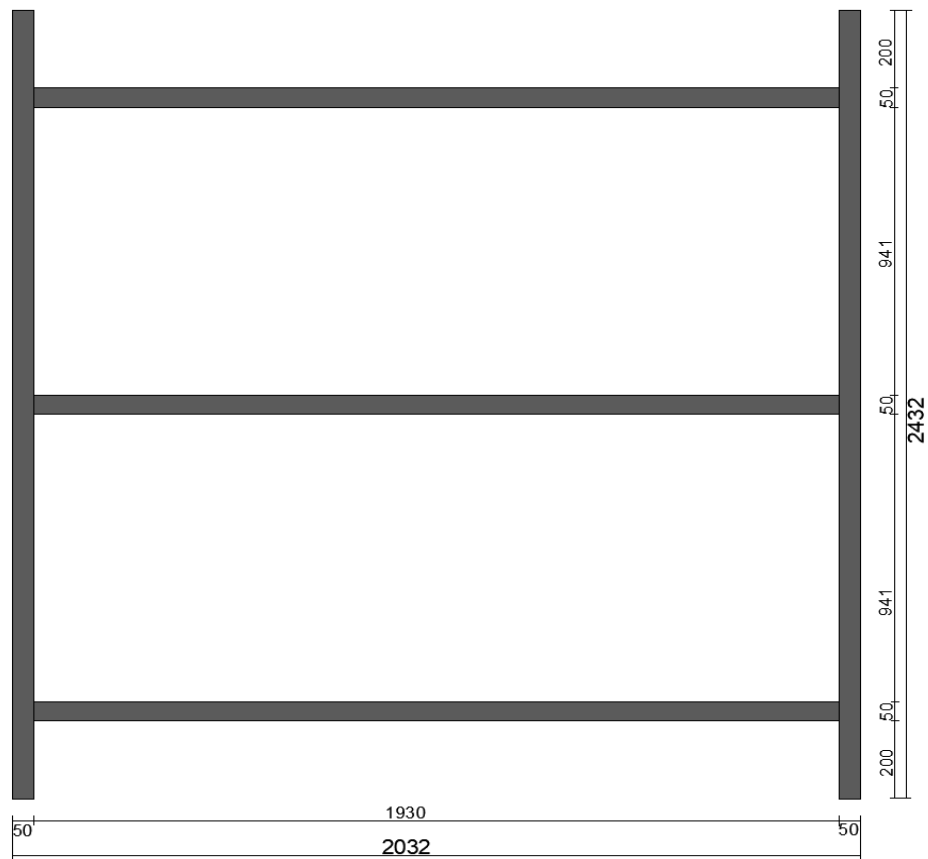


Figure C.8: Dynamic Load Factor graph for Triangular and Rectangular Loads

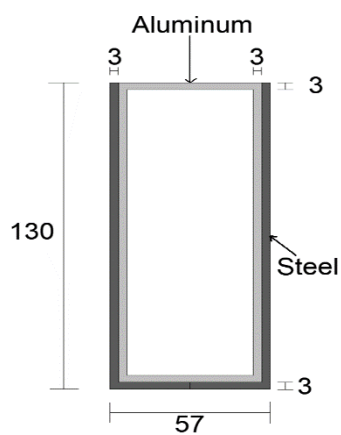
C.3 Resistance Function for Third Curtain Wall

C.3.1 Resistance Function for Simple Supports

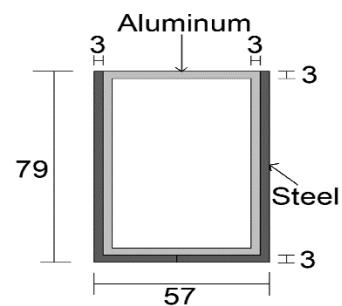
The cross-section details of second curtain wall (retrofitted mullions) are given as follows,



a) Full size curtain wall sketch



b) Hardened mullion (mm)



c) Hardened transom (mm)

Figure C.9: Third Curtain wall cross-section details

The numerical values of the properties needed for establishing resistance function are given as follows:

$$\text{Modulus of Elasticity of aluminum } (E_A) = 70,000 \frac{N}{mm^2} \text{ MPa}$$

$$\text{Modulus of Elasticity of steel } (E_s) = 200000 \frac{N}{mm^2}$$

$$\text{Density of steel} = 7850 \frac{kg}{m^3}$$

$$\text{Yield Strength of steel } (f_y) = 350 \text{ MPa} \quad ; \quad \text{Yield strength of aluminum} = 275 \text{ MPa}$$

$$\text{Length of tributary area (L)} = 2032 \text{ mm}$$

$$\text{Width of tributary area (W)} = 2032 \text{ mm}$$

Total mass of third system = 2 mullion + 3 transoms + steel sheet + 4 retrofit plates on the mullion + 6 retrofit plates on the transoms

$$= (2 \times 6.8) + (3 \times 3.8) + 32 + (4 \times 7.5) + (6 \times 4.7) = 115.2 \text{ kg}$$

Calculation of transformed M.I for the composite section:

$$\text{Transformation factor to convert aluminum into equivalent steel section: } n = \frac{E_A}{E_s} = 0.35$$

$$\text{Area of aluminum mullion} = 1032 \text{ mm}^2$$

$$\text{Transformed area of aluminum to steel section} = 0.35 \times 1032 = 361.2 \text{ mm}^2$$

$$\text{Moment of Inertia per mullion} = 2,062,274 \text{ mm}^4$$

$$\text{Transformed M.I of aluminum to steel section} = 0.35 \times 2062274 = 721,796 \text{ mm}^4$$

$$\text{Area of each L-shape steel retrofit plate on the mullion} = (130 \times 3) + (28 \times 3) = 462 \text{ mm}^2$$

$$\text{Moment of inertia for each L-shape steel retrofit plate on the mullion} = 549,304 \text{ mm}^4$$

Distance of the neutral axis from the extreme bottom (tension) fibers,

$$Y = 60.1 \text{ mm}$$

Moment of inertia for the transformed section = mullion + 2 retrofitting plate

$$I_{TR} = [721,796 + (361.2 \times 6.4^2)] + 2 [549304 + (462 \times 58.6^2)] = 5,008,178 \text{ mm}^4$$

Stiffness of two mullions for simply supported condition (hence multiplied by 2.0):

$$K = \frac{2 \times 384 \times 200000 \times 5008178}{5 \times 2032^3} = 18337 \frac{N}{mm}$$

Moment capacity of two vertical mullions (hence multiplied by 2):

$$M^- = M^+ = \frac{(fy) I}{y}$$

Here, M^- and M^+ are negative and positive moment capacities. Therefore, 'y' is the maximum distance to the tension fibers from the neutral axis. Thus, substituting the above calculated values in the moment equation. Since the specimen suffered failure during the experiment and entered in the plastic region, 300 MPa was selected as the yield strength for the aluminum and steel composite section.

$$M = \frac{2 \times 300 \times 5008178}{60.1} = 50,000,000 \text{ N.mm} = 50 \text{ kN.m}$$

Elastic Resistance of mullion,

$$R_m = \frac{8 M}{L} = \frac{8 \times 50}{2.03} = 197 \text{ kN}$$

Displacement at yield can be calculated as:

$$U_y = \frac{R_m}{K_s} = \frac{197 \times 1000}{18337} = 10.7 \text{ mm}$$

Check performance under the pressure-impulse combination applied during the test:

P = 46 kPa (recorded); Ir = 483 kPa-ms (recorded); td = 21 ms (equivalent duration)

Where K_{LM} is the load mass factor; $K_{LM} = 0.78$ for elastic response and $K_{LM} = 0.66$ for plastic response.

$$T_s = 2\pi \sqrt{\frac{0.78 \times 115.2}{18.33 \times 10^6}} = 0.014 \text{ s} = 14 \text{ ms}$$

From the UFC Chart shown on the below:

$$T_d = \frac{2 I_r}{P_r} = \frac{2 \times 483}{46} = 21 \text{ ms} \quad \frac{T_d}{T_s} = \frac{21}{14} = 1.5 \quad \text{for } \mu = 1.0 \quad \text{read} \quad \frac{R_m}{F_1} = 1.8$$

$$F_1 = P_1 (\text{Tributary Area}) = 46 \times 2.03 \times 2.03 = 189.6 \text{ kN}$$

$$R_m (\text{demand}) = 1.8 \times 189.6 = 341.2 \text{ kN} > R_m (\text{capacity}) = 197 \text{ kN}$$

Therefore, the mullions as are will behave in the inelastic range.

Use $K_{LM} = 0.70$ since significant inelasticity is anticipated (a value closer to $K_{LM} = 0.66$).

$$T_s = 2\pi \sqrt{\frac{0.7 \times 115.2}{18.3 \times 10^6}} = 0.013 \text{ s} = 13 \text{ ms}$$

$$\frac{T_d}{T_s} = \frac{21}{13} = 1.7 \quad \frac{R_m}{F_1} = \frac{197}{189.6} = 1.04 \quad \text{read} \quad \mu = 2.8$$

$$\text{Damping} = 0.02 \times 2\sqrt{K \cdot M} = 0.02 \times 2 \times \sqrt{18377000 \times 115.2} = 1840 \text{ Ns/m}$$

Considering $\mu = 13.5 \rightarrow \text{UFC chart approach: } 2.8 \times 10.7 = 30 \text{ mm}$

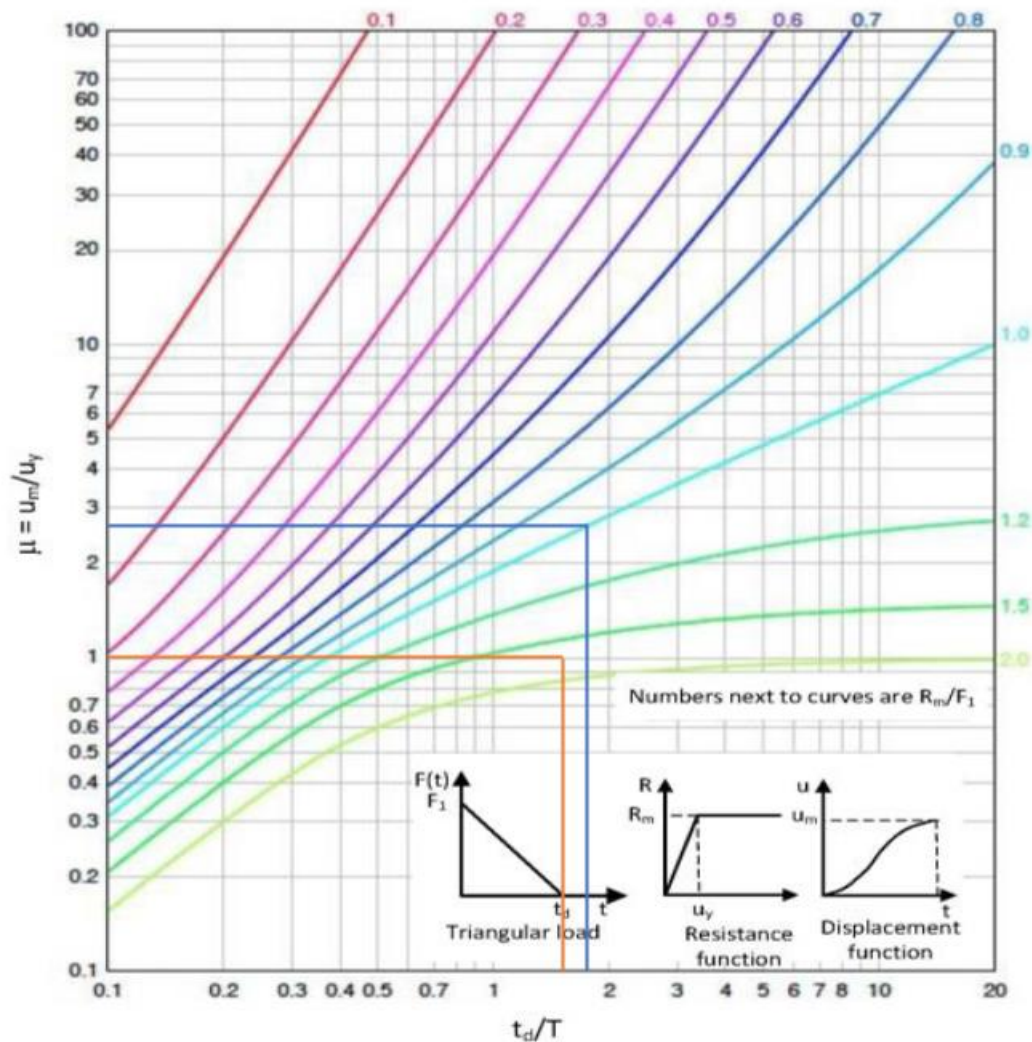


Figure C.10: UFC chart Maximum response of elastoplastic SDOF system (triangular pulses)

C.3.2 Elastic Resistance Reaction for Third Curtain Wall with Simple Supports

Distributed Mass: $115.2 \times 0.78 = 90 \text{ kg}$

Moment of Inertia per mullion = $5,008,178 \text{ mm}^4$

$$K = \frac{2 \times 384 \times 200000 \times 5008178}{5 \times 2032^3} = 18337 \frac{N}{mm} = 18.337 \frac{N}{m}$$

$$T_s = 2\pi \sqrt{\frac{90}{18.33 \times 10^6}} = 0.014 \text{ s} = 14 \text{ ms}$$

$$T_d = \frac{2 I_r}{P_r} = \frac{2 \times 483}{46} = 21 \text{ ms}$$

$$\frac{T_d}{T} = \frac{21}{14} = 1.5$$

Dynamic Load Factor (DLF) = 1.7

Exposed specimen area to the blast load: $2032 \text{ mm} \times 2032 \text{ mm} = 4,129,024 \text{ mm}^2$

Elastic Force Resistance: $0.046 \frac{N}{mm^2} \times 4,129,024 \text{ mm}^2 = 189,935 \text{ N}$

Dynamic Resistance: $1.7 \times 189,935 \text{ N} = 322,890 \text{ N}$

Dynamic Reaction (V): $0.39 R + 0.11 F$

$\Rightarrow$ Dynamic Reaction = $(0.39 \times 322,890 \text{ N}) + (0.11 \times 189,935 \text{ N}) = 148,820 \text{ N}$

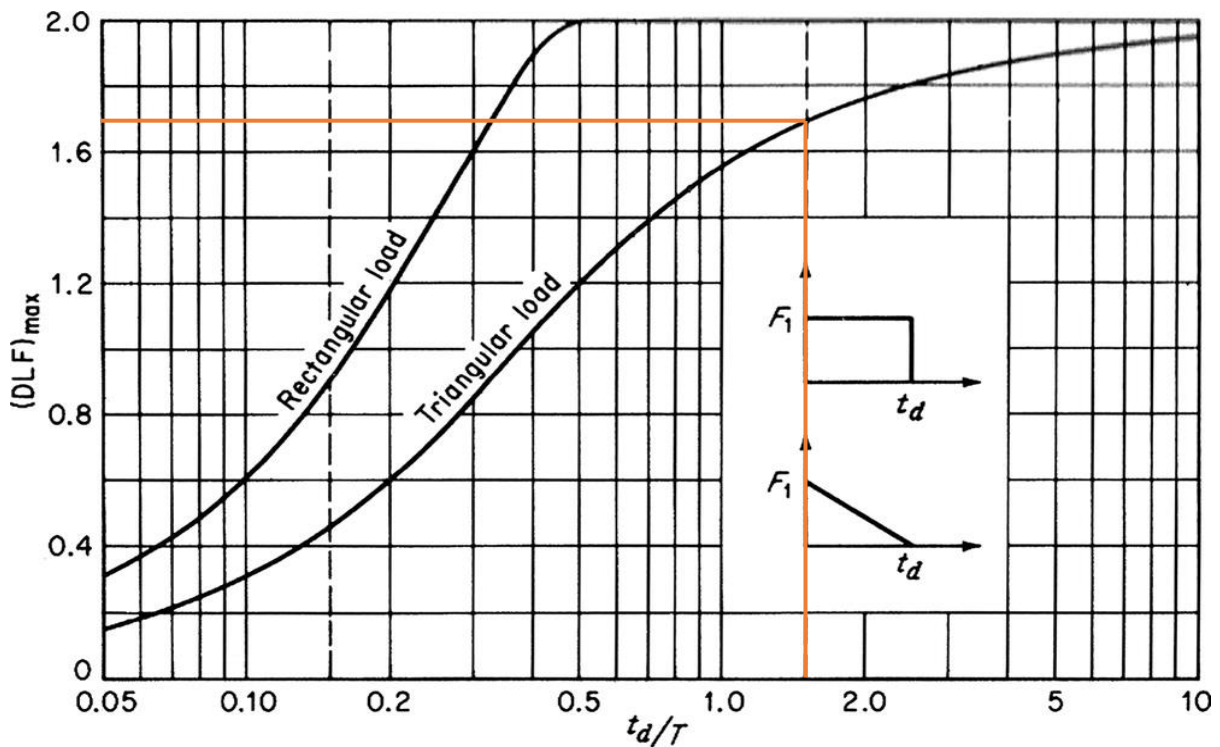


Figure C.11: Dynamic Load Factor graph for Triangular and Rectangular Loads

C.3.3 Plastic Resistance Reaction for Third Curtain Wall with Simple Supports

Distributed Mass: $115.2 \times 0.66 = 76 \text{ kg}$

Moment of Inertia per mullion = $5,008,178 \text{ mm}^4$

$$K = \frac{2 \times 384 \times 200000 \times 5008178}{5 \times 2032^3} = 18337 \frac{N}{mm} = 18.337 \frac{N}{m}$$

$$T_s = 2\pi \sqrt{\frac{76}{18.33 \times 10^6}} = 0.012 \text{ s} = 12 \text{ ms}$$

$$T_d = \frac{2 I_r}{P_r} = \frac{2 \times 483}{46} = 21 \text{ ms}$$

$$\frac{T_d}{T} = \frac{21}{12} = 1.75$$

Dynamic Load Factor (DLF) = 1.77

Exposed specimen area to the blast load: $2032 \text{ mm} \times 2032 \text{ mm} = 4,129,024 \text{ mm}^2$

Elastic Force Resistance: $0.046 \frac{N}{mm^2} \times 4,129,024 \text{ mm}^2 = 189,935 \text{ N}$

Dynamic Resistance: $1.77 \times 189,935 \text{ N} = 336,184 \text{ N}$

Dynamic Reaction (V): $0.38 R + 0.12 F$

$\Rightarrow$ Dynamic Reaction = $(0.38 \times 336,184 \text{ N}) + (0.12 \times 189,935 \text{ N}) = 150,542 \text{ N}$

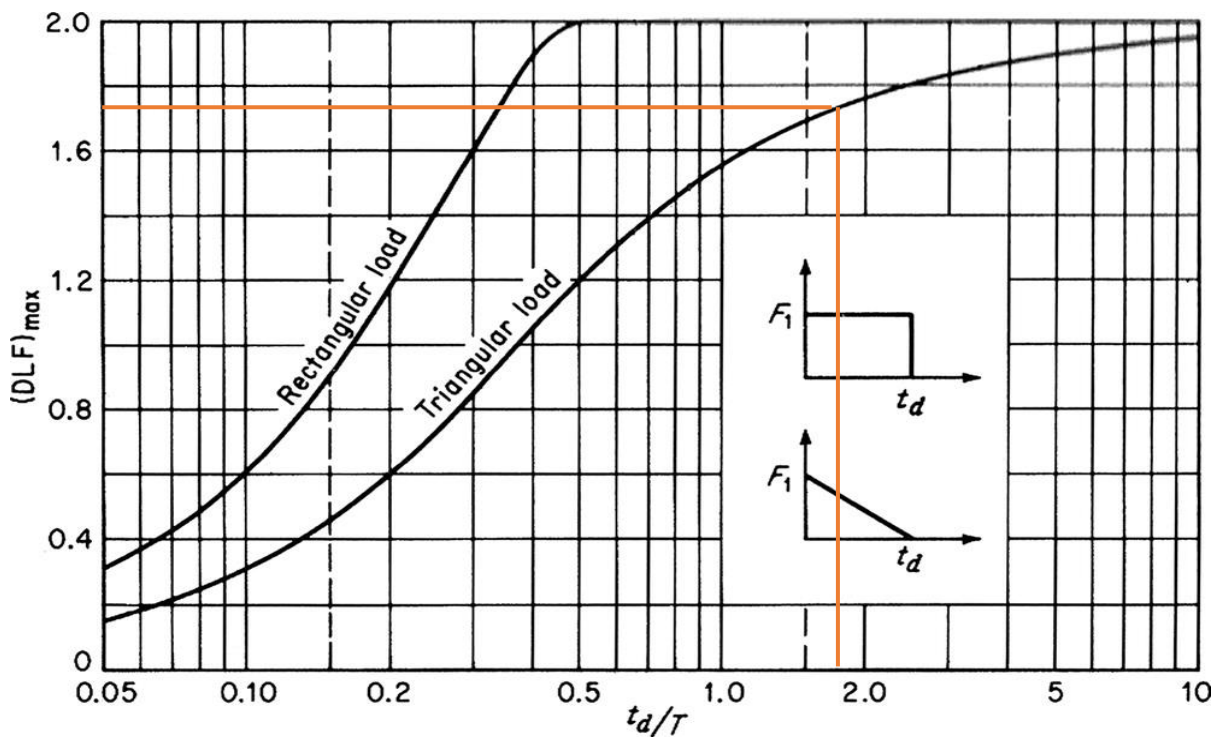
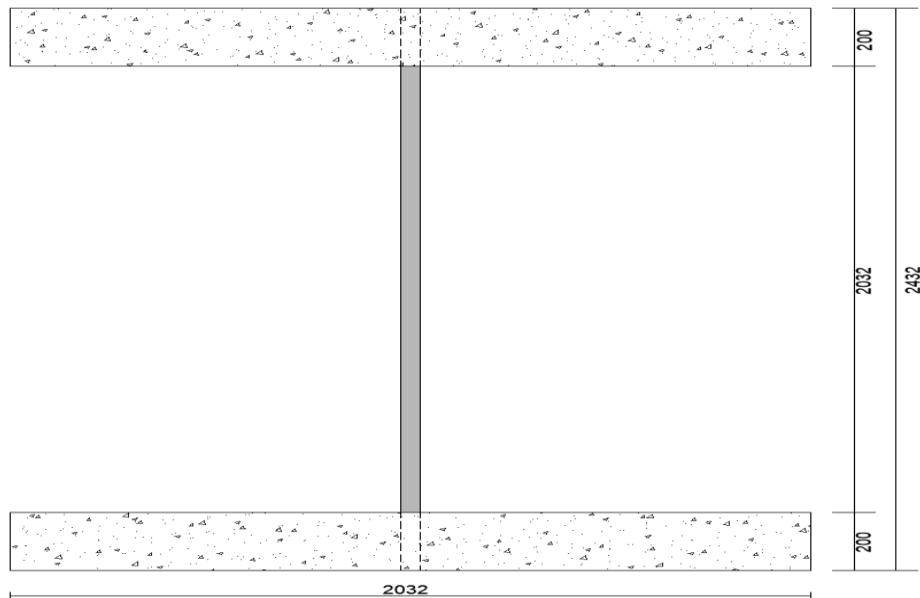


Figure C. 12: Dynamic Load Factor graph for Triangular and Rectangular Loads

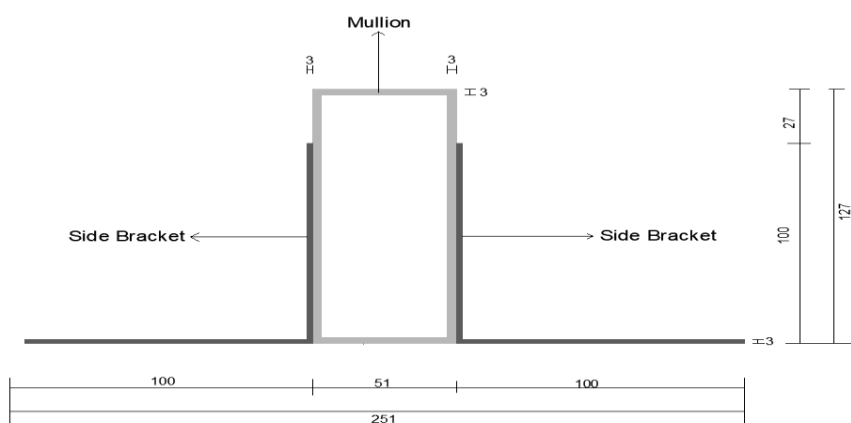
C.4 Resistance Function for First Single Mullion

C.4.1 Resistance Function for Fixed Supports

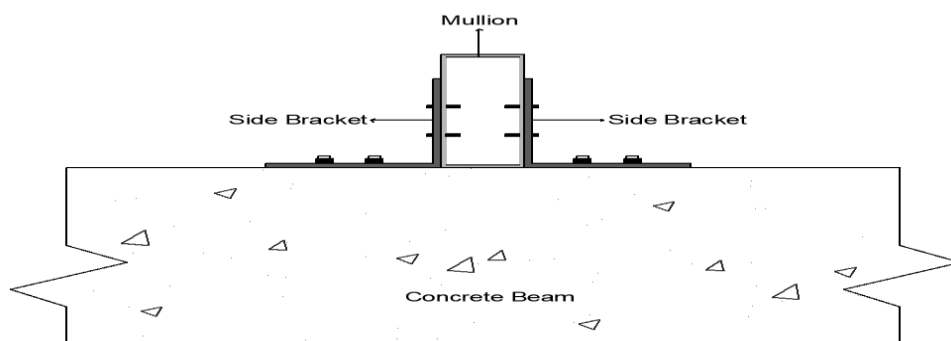
The cross-section details of first Mullion (M-C1) are given as follows:



a) Full-size sketch



b) Retrofitted connection with steel brackets on sides of mullion (top view)



c) Mullion connection to the concrete beam section

Figure C.13: First mullion configuration details (mm)

The numerical values of the properties needed for establishing resistance function are given as follows:

$$\text{Modulus of Elasticity of aluminum } (E_A) = 70,000 \frac{N}{mm^2}$$

$$\text{Modulus of Elasticity of steel } (E_s) = 200,000 \frac{N}{mm^2}$$

$$\text{Yield Strength of Aluminum } (f_y) = 275 \text{ MPa}$$

$$\text{Yield Strength of steel } (f_y) = 350 \text{ MPa} \quad ; \quad \text{Bracket section size: } 100 \text{ mm} \times 100 \text{ mm} \times 3 \text{ mm}$$

$$\text{Length of tributary area } (L) = 2083 \text{ mm}$$

$$\text{Width of tributary area } (W) = 2083 \text{ mm}$$

$$\text{Total mass of M1 system} = 1 \text{ mullion} + \text{load application device} = 7 \text{ kg} + 107 \text{ kg} = 114 \text{ kg}$$

$$\text{Moment of Inertia per mullion} = 2,062,274 \text{ mm}^4$$

$$\text{Area of aluminum mullion} = 1032 \text{ mm}^2$$

Stiffness for fixed supported beam assembly,

$$K = \frac{384 EI}{L^3} = \frac{384 \times 70000 \times 2062274}{2083^3} = 6133 \frac{N}{mm} = 6.133 \frac{N}{m}$$

Stiffness for simply supported beam assembly,

$$K = \frac{384 EI}{5L^3} = \frac{384 \times 70000 \times 2062274}{5 \times 2083^3} = 1226 \frac{N}{mm} = 1.226 \frac{N}{m}$$

Moment capacity of single mullion,

$$M^- = M^+ = \frac{(f_y) I}{y}$$

Here, M^- and M^+ are negative and positive moment capacities. Therefore, 'y' is the maximum distance to the tension fibers from the neutral axis. Thus, substituting the above calculated values in the moment equation,

$$M = \frac{275 \times 2062274}{63.5} = 8,931,108 \text{ N.mm} = 8.93 \text{ kN.m}$$

Considering the fixity at the supports, initial yield resistance is calculated as follows,

$$R_{y1} = \frac{12 M}{L} = \frac{12 \times 8.93}{2.08} = 51.52 \text{ kN}$$

Corresponding displacement at the supports is calculated as follows,

$$U_{y1} = \frac{R_{y1}}{K_f} = \frac{51.52 \times 1000}{6133} = 8.4 \text{ mm}$$

Now, hinges are formed at the supports, and the moment carrying capacity is reduced to half, which will be 4.47 kN.m. The beam will now behave as simply supported beam with the yield resistance calculated as follows,

$$R_{y2} = \frac{8 M}{L} = \frac{8 \times 4.47}{2.08} = 17.2 \text{ kN}$$

Now, the total resistance is the summation of R_{y1} and R_{y2} . Thus, $R_m = 68.72 \text{ kN}$.

The displacement at the formation of the hinge at support is given as follows,

$$U_{y2} = U_{y1} + \frac{R_{y2}}{K_s} = 8.4 + \frac{17.2 \times 1000}{1226} = 22.4 \text{ mm}$$

Now, the trilinear resistance function requires to be converted into bilinear resistance function. For doing this, the area under the trilinear and bilinear function is to be equated, which gives an equivalent yield displacement value for the corresponding yield resistance R_m . Following are the calculated values for the same.

$$U_y = 20 \text{ mm} \quad K_{eff} = \frac{R_m}{U_y} = \frac{68.72 \times 1000}{20} = 3436 \frac{N}{mm} = 3.436 \frac{N}{m}$$

Check performance under the pressure-impulse combination applied during the test:

P = 22 kPa (recorded); Ir = 248 kPa-ms (recorded); td = 22.5 ms (equivalent duration)

✚ Where K_{LM} is the load mass factor; $K_{LM} = 0.78$ for elastic response and $K_{LM} = 0.66$ for plastic response.

$$T_s = 2\pi \sqrt{\frac{0.78 \times 114}{3.436 \times 10^6}} = 0.032 \text{ s} = 32 \text{ ms}$$

From the UFC Chart shown on the below:

$$T_d = \frac{2 I_r}{P_r} = \frac{2 \times 248}{22} = 22.5 \text{ ms} \quad \frac{T_d}{T_s} = \frac{22.5}{32} = 0.7 \quad \text{for } \mu = 1.0 \quad \text{read} \quad \frac{R_m}{F_1} = 1.4$$

$$F_1 = P_1 (\text{Tributary Area}) = 22 \times 2.08 \times 2.08 = 95.2 \text{ kN}$$

$$R_m (\text{demand}) = 1.4 \times 95.2 = 133.3 \text{ kN} > R_m (\text{capacity}) = 68 \text{ kN}$$

Therefore, the mullions will behave in the inelastic range.

Use $K_{LM} = 0.7$ since significant inelasticity is anticipated (a value closer to $K_{LM} = 0.66$).

$$T_s = 2\pi \sqrt{\frac{0.7 \times 114}{3.436 \times 10^6}} = 0.030 \text{ s} = 30 \text{ ms} ; \quad \frac{T_d}{T_s} = \frac{22.5}{30} = 0.75 ; \quad \frac{R_m}{F_1} = 0.72 \rightarrow \mu = 2.9$$

$$\text{Damping} = 0.02 \times 2\sqrt{K \cdot M} = 0.02 \times 2 \times \sqrt{3436000 \times 114} = 792 \text{ Ns/m}$$

Considering $\mu = 1.0 \rightarrow$ UFC chart approach: $20 \times 2.9 = 58 \text{ mm}$

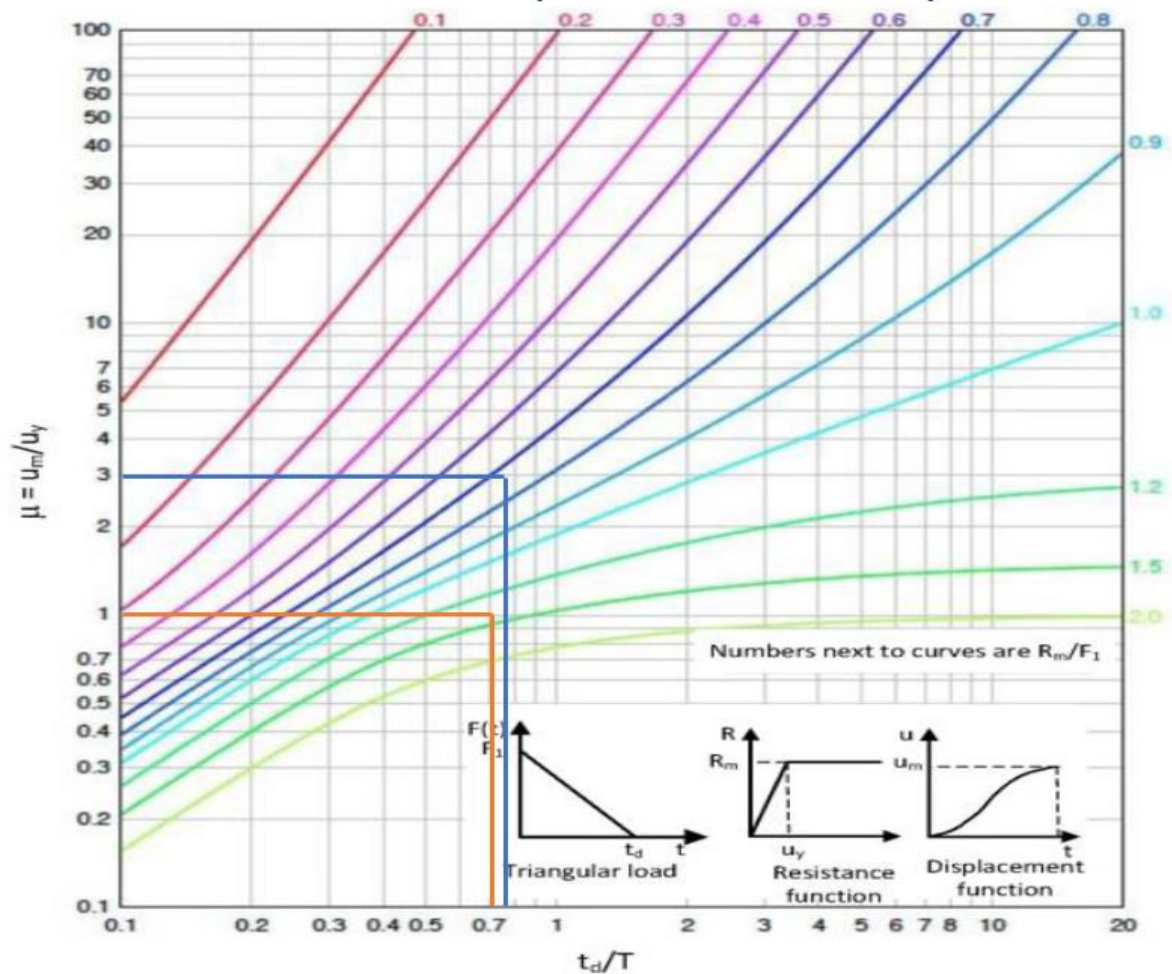


Figure C.14: UFC chart Maximum response of elastoplastic SDOF system (triangular pulses)

C.4.3 Elastic Strain Range Resistance Reaction for First Mullion with Fixed Supports

Distributed Mass: $114 \times 0.77 = 88 \text{ kg}$

Moment of Inertia per mullion = $2,062,274 \text{ mm}^4$

$$K = \frac{384 EI}{L^3} = \frac{384 \times 70000 \times 2062274}{2083^3} = 6133 \frac{N}{mm} = 6.133 \frac{N}{m}$$

$$T_s = 2\pi \sqrt{\frac{88}{6.133 \times 10^6}} = 0.024 \text{ s} = 24 \text{ ms} \quad T_d = \frac{2 I_r}{P_r} = \frac{2 \times 248}{22} = 22.5 \text{ ms}$$

$$\frac{T_d}{T} = \frac{22.5}{24} = 0.94 \quad \text{Dynamic Load Factor (DLF)} = 1.55$$

Exposed specimen area to the blast load: $2082 \text{ mm} \times 2082 \text{ mm} = 4,334,724 \text{ mm}^2$

Elastic Force Resistance: $0.022 \frac{N}{mm^2} \times 4,334,724 \text{ mm}^2 = 95,364 \text{ N}$

Dynamic Resistance: $1.55 \times 95,364 \text{ N} = 147,814 \text{ N}$

Dynamic Reaction (V): $0.36 R + 0.14 F$

$$\Rightarrow \text{Dynamic Reaction} = (0.36 \times 147,814 \text{ N}) + (0.14 \times 95,364 \text{ N}) = 66,564 \text{ N}$$

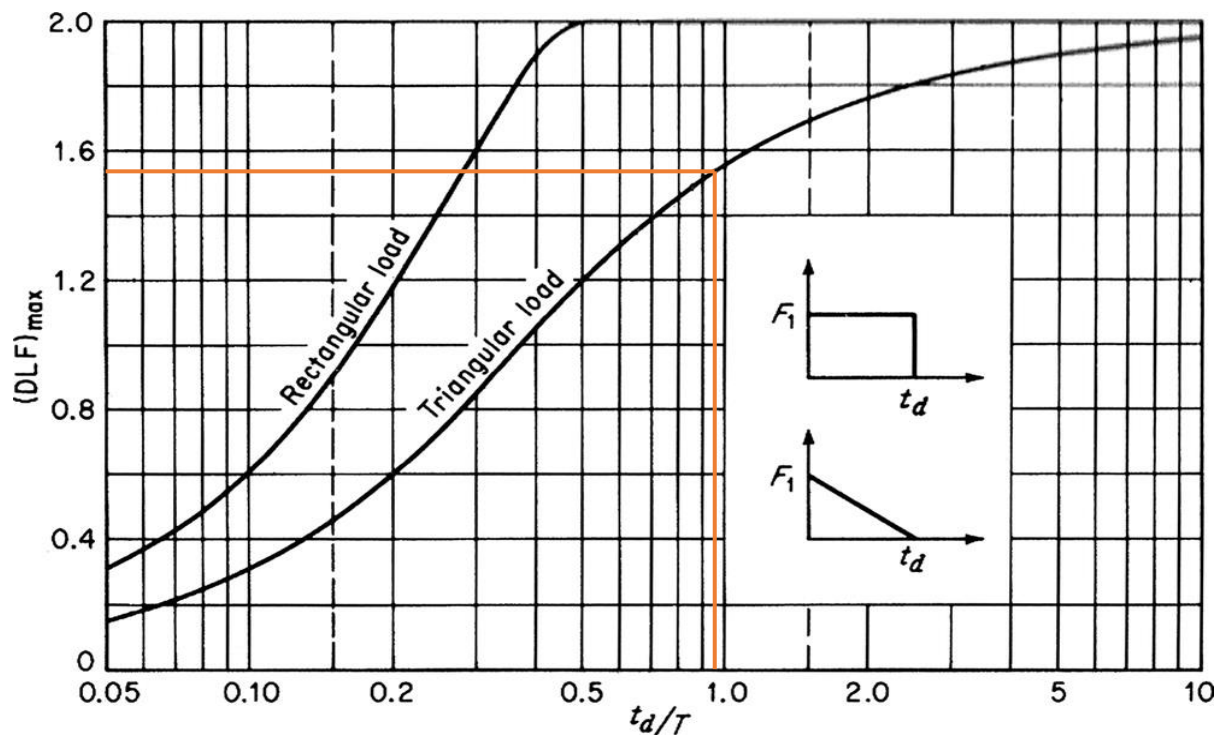


Figure C.15: Dynamic Load Factor graph for Triangular and Rectangular Loads

C.4.3 Plastic Strain Range Resistance Reaction for First Mullion with fixed Supports

Distributed Mass: $114 \times 0.78 = 89 \text{ kg}$

Moment of Inertia per mullion = $2,062,274 \text{ mm}^4$

$$K = \frac{384 EI}{L^3} = \frac{384 \times 70000 \times 2062274}{2083^3} = 6133 \frac{N}{mm} = 6.133 \frac{N}{m}$$

$$T_s = 2\pi \sqrt{\frac{89}{6.133 \times 10^6}} = 0.024 \text{ s} = 24 \text{ ms} \quad T_d = \frac{2 I_r}{P_r} = \frac{2 \times 248}{22} = 22.5 \text{ ms}$$

$$\frac{T_d}{T} = \frac{22.5}{24} = 0.94 \quad \text{Dynamic Load Factor (DLF)} = 1.55$$

Exposed specimen area to the blast load: $2082 \text{ mm} \times 2082 \text{ mm} = 4,334,724 \text{ mm}^2$

Elastic Force Resistance: $0.022 \frac{N}{mm^2} \times 4,334,724 \text{ mm}^2 = 95,364 \text{ N}$

Dynamic Resistance: $1.55 \times 95,364 \text{ N} = 147,814 \text{ N}$

Dynamic Reaction (V): $0.39 R + 0.11 F$

$$\Rightarrow \text{Dynamic Reaction} = (0.39 \times 147,814 \text{ N}) + (0.11 \times 95,364 \text{ N}) = 68,138 \text{ N}$$

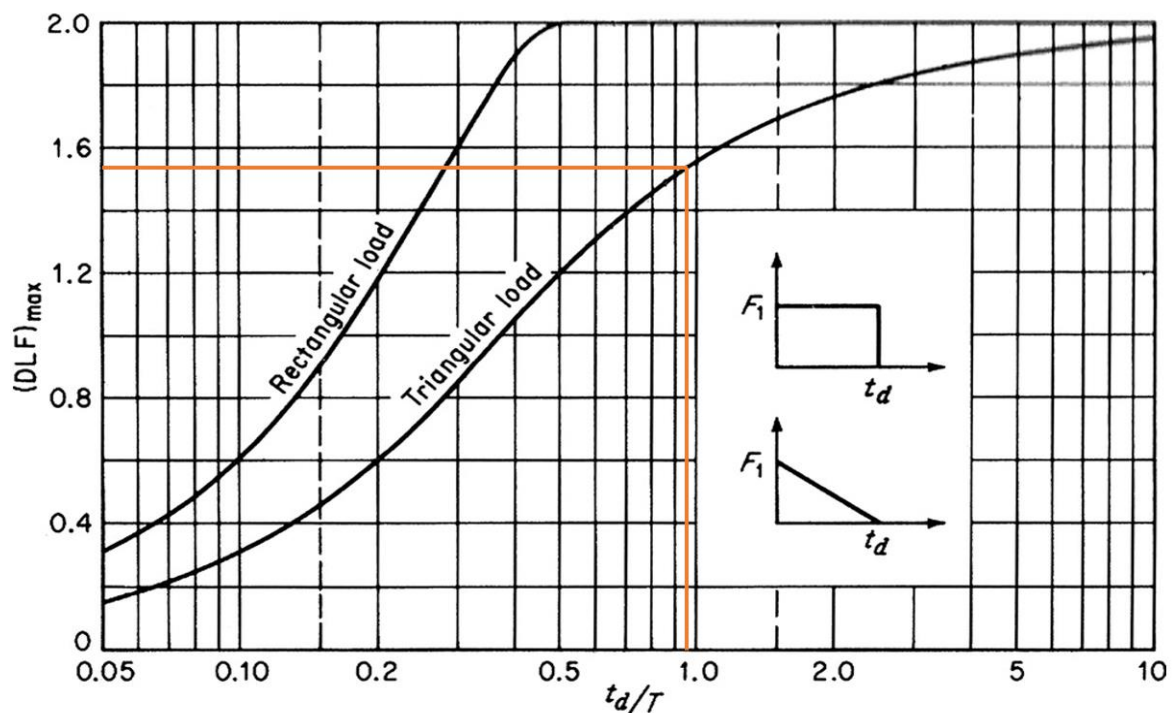
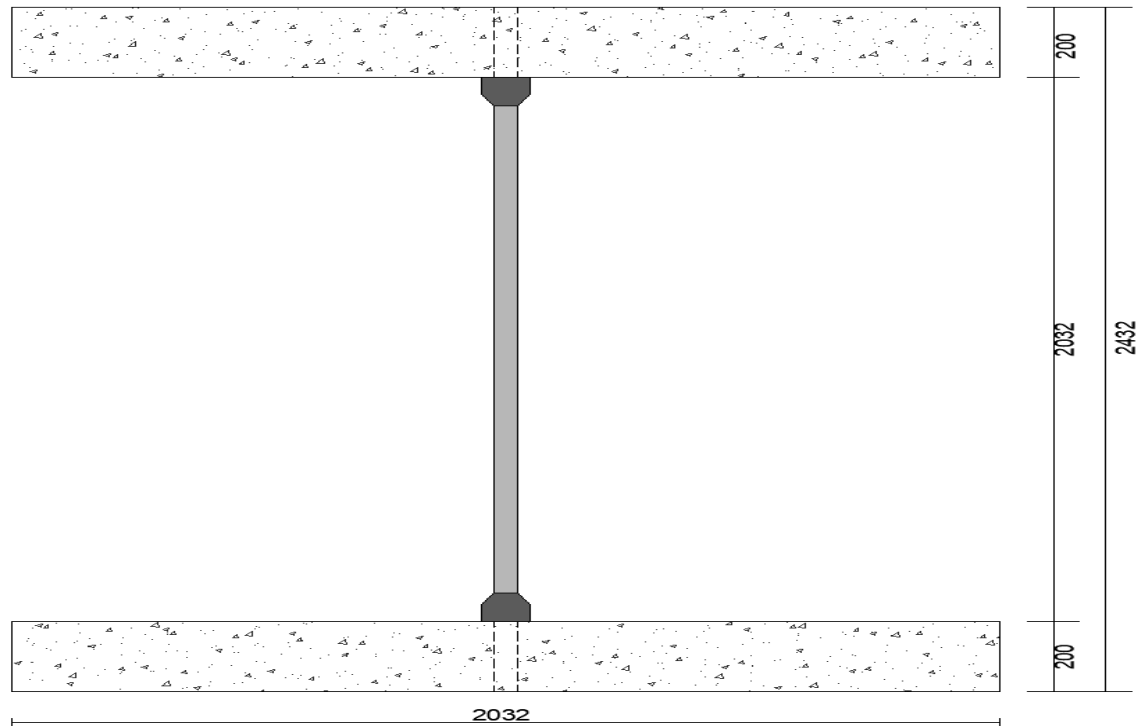


Figure C.16: Dynamic Load Factor graph for Triangular and Rectangular Load

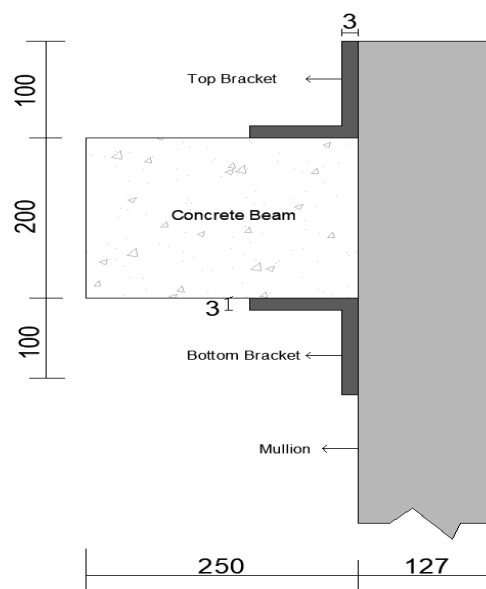
C.5 Resistance Function for Second Single Mullion

C.5.1 Resistance Function for Fixed Supports

The cross-section details of first Mullion (M-C2) are given as follows:



a) Full-size sketch



b) Retrofitted connection with top and bottom brackets (side view)

Figure C.17: Second mullion configuration details (mm)

The numerical values of the properties needed for establishing resistance function are given as follows:

$$\text{Modulus of Elasticity of aluminum } (E_A) = 70,000 \frac{N}{mm^2}$$

$$\text{Modulus of Elasticity of steel } (E_s) = 200,000 \frac{N}{mm^2}$$

$$\text{Density of aluminum} = 2710 \frac{kg}{m^3}$$

$$\text{Yield Strength of aluminum } (f_y) = 275 \text{ MPa}$$

$$\text{Yield Strength of steel } (f_y) = 350 \text{ MPa}$$

$$\text{Bracket section size: } 100 \text{ mm} \times 100 \text{ mm} \times 3 \text{ mm}$$

$$\text{Length of tributary area } (L) = 2083 \text{ mm}$$

$$\text{Width of tributary area } (W) = 2083 \text{ mm}$$

$$\text{Total mass of M2 system} = 1 \text{ mullion} + \text{load application device} = 7 \text{ kg} + 107 \text{ kg} = 114 \text{ kg}$$

$$\text{Moment of Inertia per mullion} = 2,062,274 \text{ mm}^4$$

$$\text{Area of aluminum mullion} = 1032 \text{ mm}^2$$

Stiffness for fixed supported beam assembly,

$$K = \frac{384 EI}{L^3} = \frac{384 \times 70000 \times 2062274}{2083^3} = 6133 \frac{N}{mm} = 6.133 \frac{N}{m}$$

Stiffness for simply supported beam assembly,

$$K = \frac{384 EI}{5L^3} = \frac{384 \times 70000 \times 2062274}{5 \times 2083^3} = 1226 \frac{N}{mm} = 1.226 \frac{N}{m}$$

Moment capacity of single mullion,

$$M^- = M^+ = \frac{(f_y) I}{y}$$

Here, M^- and M^+ are negative and positive moment capacities. Therefore, 'y' is the maximum distance to the tension fibers from the neutral axis. Thus, substituting the above calculated values in the moment equation,

$$M = \frac{275 \times 2062274}{63.5} = 8,931,108 \text{ N.mm} = 8.93 \text{ kN.m}$$

Considering the fixity at the supports, initial yield resistance is calculated as follows,

$$R_{y1} = \frac{12 M}{L} = \frac{12 \times 8.93}{2.08} = 51.5 \text{ kN}$$

Corresponding displacement at the supports is calculated as follows,

$$U_{y1} = \frac{R_{y1}}{K_f} = \frac{51.5 \times 1000}{6133} = 8.4 \text{ mm}$$

Now, hinges are formed at the supports, and the moment carrying capacity is reduced to half, which will be 4.47 kN.m. The beam will now behave as simply supported beam with the yield resistance calculated as follows,

$$R_{y2} = \frac{8 M}{L} = \frac{8 \times 4.47}{2.08} = 17.2 \text{ kN}$$

Now, the total resistance is the summation of R_{y1} and R_{y2} . Thus, $R_m = 68.72 \text{ kN}$

The displacement at the formation of the hinge at support is given as follows,

$$U_{y2} = U_{y1} + \frac{R_{y2}}{K_s} = 8.4 + \frac{17.2 \times 1000}{1226} = 22.4 \text{ mm}$$

Now, the trilinear resistance function requires to be converted into bilinear resistance function. For doing this, the area under the trilinear and bilinear function is to be equated, which gives an equivalent yield displacement value for the corresponding yield resistance R_m . Following are the calculated values for the same.

$$U_y = 20 \text{ mm} \quad K_{eff} = \frac{R_m}{U_y} = \frac{68.72 \times 1000}{20} = 3436 \frac{N}{mm} = 3.436 \frac{N}{m}$$

Check performance under the pressure-impulse combination applied during the test:

P = 28 kPa (recorded); Ir = 285 kPa-ms (recorded); td = 20 ms (equivalent duration)

Where K_{LM} is the load mass factor; $K_{LM} = 0.78$ for elastic response and $K_{LM} = 0.66$ for plastic response.

$$T_s = 2\pi \sqrt{\frac{0.78 \times 114}{3.436 \times 10^6}} = 0.032 \text{ s} = 32 \text{ ms}$$

From the UFC Chart shown on the below:

$$T_d = \frac{2 I_r}{P_r} = \frac{2 \times 285}{28} = 20 \text{ ms} \quad \frac{T_d}{T_s} = \frac{20}{32} = 0.62 \quad \text{for } \mu = 1.0 \quad \text{read} \quad \frac{R_m}{F_1} = 1.35$$

$$F_1 = P_1 (\text{Tributary Area}) = 28 \times 2.08 \times 2.08 = 121.1 \text{ kN}$$

$$R_m (\text{demand}) = 1.35 \times 121.1 = 163.5 \text{ kN} > R_m (\text{capacity}) = 68 \text{ kN}$$

Therefore, the mullions will behave in the inelastic range.

Use $K_{LM} = 0.7$ since significant inelasticity is anticipated (a value closer to $K_{LM} = 0.66$).

$$T_s = 2\pi \sqrt{\frac{0.7 \times 114}{3.436 \times 10^6}} = 0.03 \text{ s} = 30 \text{ ms}; \quad \frac{T_d}{T_s} = \frac{20}{30} = 0.6 \quad ; \quad \frac{R_m}{F_1} = 0.56 \rightarrow \mu = 4.3$$

$$\text{Damping} = 0.02 \times 2\sqrt{K \cdot M} = 0.02 \times 2 \times \sqrt{3436000 \times 114} = 792 \text{ Ns/m}$$

Considering $\mu = 1.0 \rightarrow$ UFC chart approach: $20 \times 4.3 = 86 \text{ mm}$

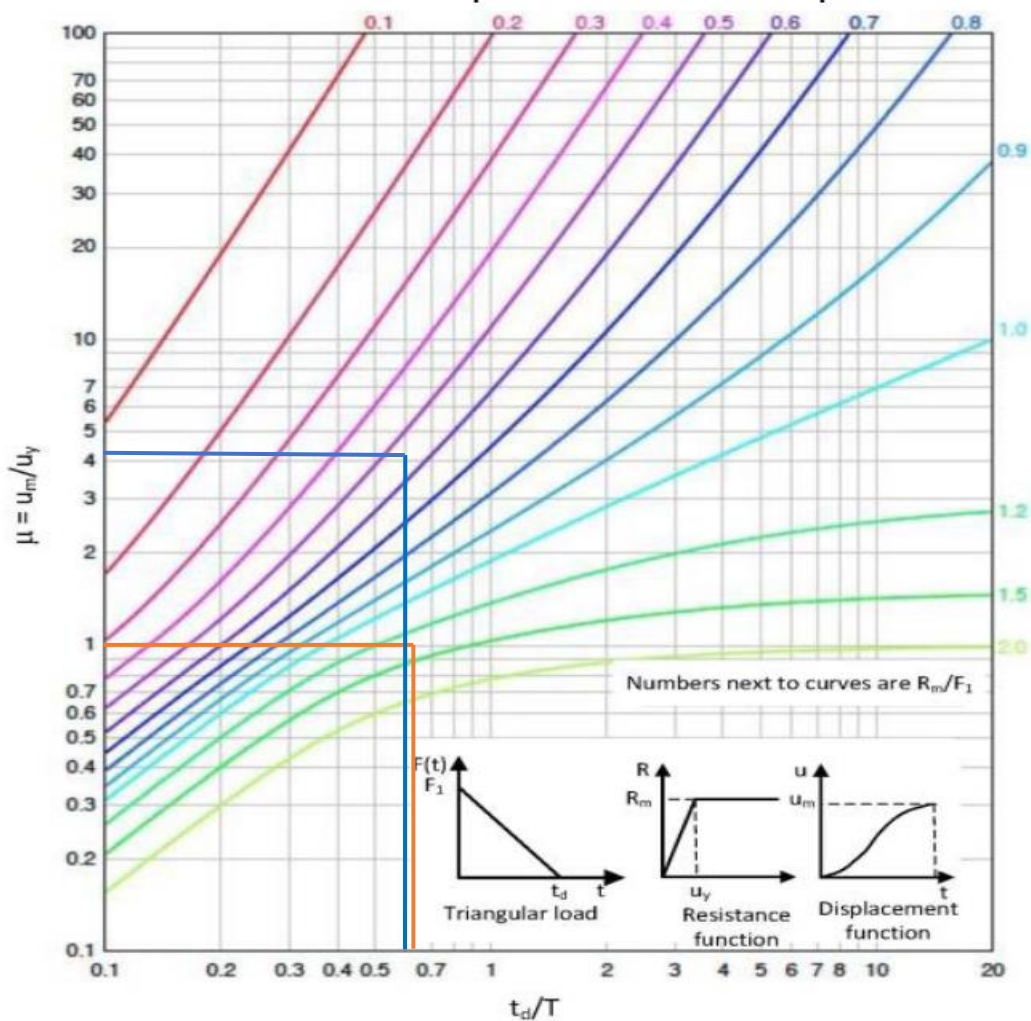


Figure C.18: UFC chart Maximum response of elastoplastic SDOF system (triangular pulses)

C.5.2 Elastic Resistance Reaction for Second Mullion with fixed Supports

Distributed Mass: $114 \times 0.77 = 88 \text{ kg}$

Moment of Inertia per mullion = $2,062,274 \text{ mm}^4$

$$K = \frac{384 EI}{L^3} = \frac{384 \times 70000 \times 2062274}{2083^3} = 6133 \frac{N}{mm} = 6.133 \frac{N}{m}$$

$$T_s = 2\pi \sqrt{\frac{88}{6.133 \times 10^6}} = 0.023 \text{ s} = 23 \text{ ms} \quad T_d = \frac{2 I_r}{P_r} = \frac{2 \times 285}{28} = 20 \text{ ms}$$

$$\frac{T_d}{T} = \frac{20}{23} = 0.87 \quad \text{Dynamic Load Factor (DLF)} = 1.5$$

Exposed specimen area to the blast load: $2082 \text{ mm} \times 2082 \text{ mm} = 4,334,724 \text{ mm}^2$

Elastic Force Resistance: $0.028 \frac{N}{mm^2} \times 4,334,724 \text{ mm}^2 = 121,372 \text{ N}$

Dynamic Resistance: $1.5 \times 121,372 \text{ N} = 182,058 \text{ N}$

Dynamic Reaction (V): $0.36 R + 0.14 F$

$$\Rightarrow \text{Dynamic Reaction} = (0.36 \times 182,058 \text{ N}) + (0.14 \times 121,372 \text{ N}) = 82,533 \text{ N}$$

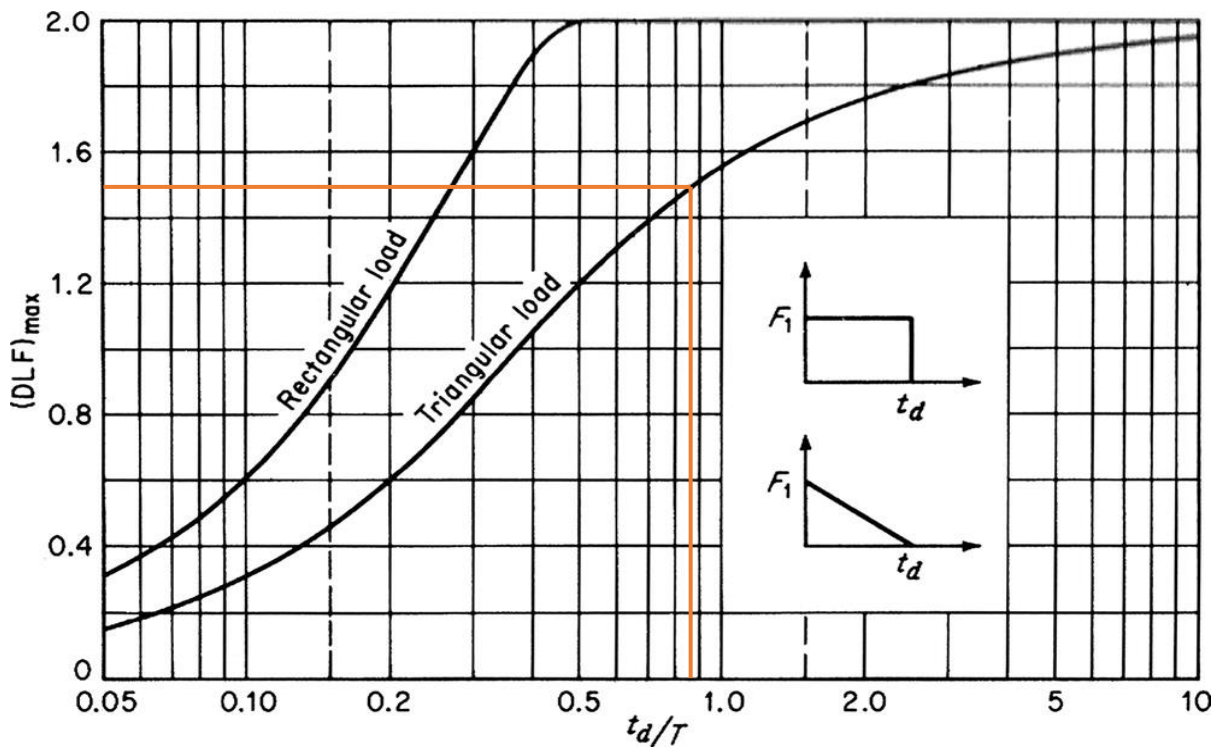


Figure C.19: Dynamic Load Factor graph for Triangular and Rectangular Loads

C.5.3 Plastic Resistance Reaction for Second Mullion with fixed supports

Distributed Mass: $114 \times 0.78 = 89 \text{ kg}$

Moment of Inertia per mullion = $2,062,274 \text{ mm}^4$

$$K = \frac{384 EI}{L^3} = \frac{384 \times 70000 \times 2062274}{2083^3} = 6133 \frac{N}{mm} = 6.133 \frac{N}{m}$$

$$T_s = 2\pi \sqrt{\frac{89}{6.133 \times 10^6}} = 0.024 \text{ s} = 53 \text{ ms}$$

$$T_d = \frac{2 I_r}{P_r} = \frac{2 \times 285}{28} = 20 \text{ ms}$$

$$\frac{T_d}{T} = \frac{20}{24} = 0.84$$

Dynamic Load Factor (DLF) = 1.55

Exposed specimen area to the blast load: $2082 \text{ mm} \times 2082 \text{ mm} = 4,334,724 \text{ mm}^2$

Elastic Force Resistance: $0.028 \frac{N}{mm^2} \times 4,334,724 \text{ mm}^2 = 121,372 \text{ N}$

Dynamic Resistance: $1.55 \times 121,372 \text{ N} = 188,126 \text{ N}$

Dynamic Reaction (V): $0.39 R + 0.11 F$

$$\Rightarrow \text{Dynamic Reaction} = (0.39 \times 188,126 \text{ N}) + (0.11 \times 121,372 \text{ N}) = 86,720 \text{ N}$$

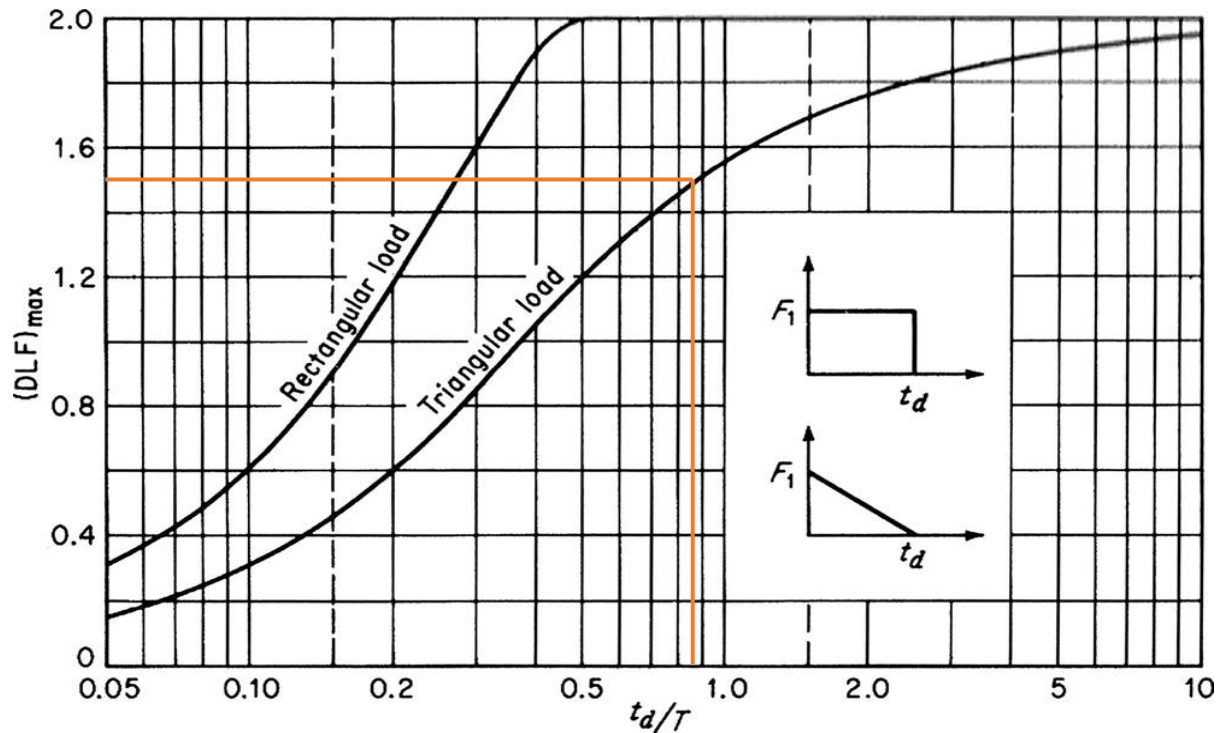
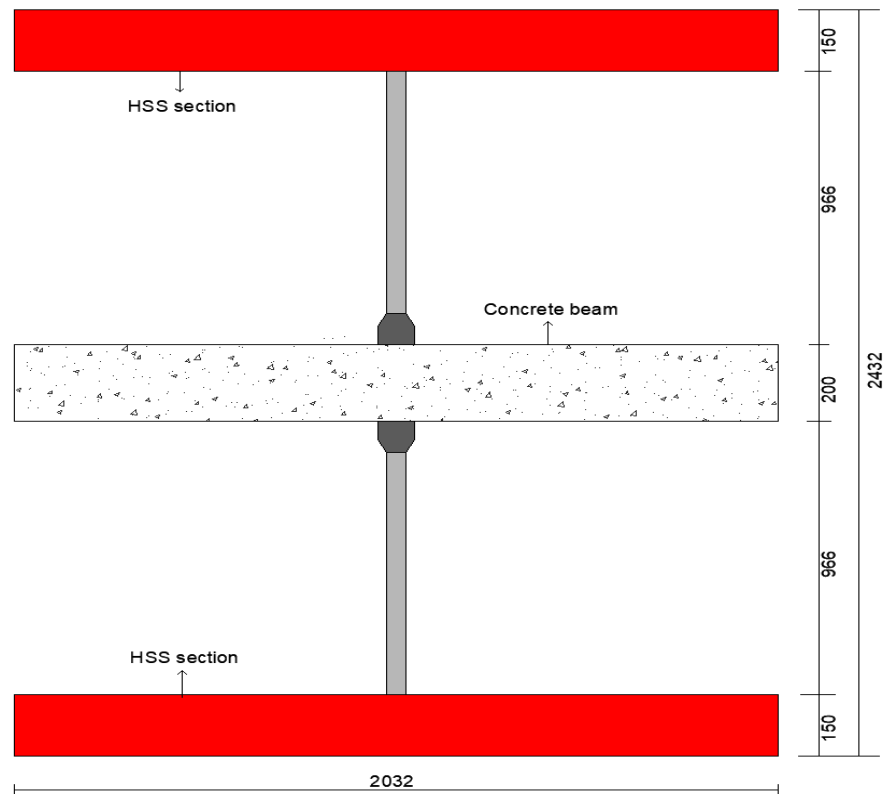


Figure C.20: Dynamic Load Factor graph for Triangular and Rectangular Loads

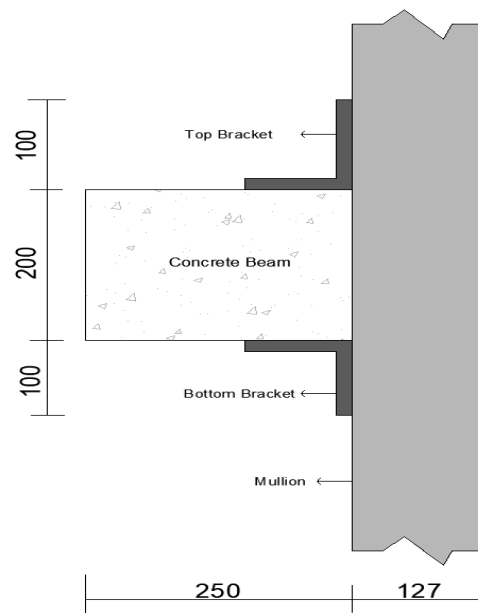
C.6 Resistance Function for Third Single Mullion

C.6.1 Resistance Function for Fixed Supports

The cross-section details of third Mullion (M-C3) are given as follows:



a) Full-size sketch



b) Retrofitted connection with top and bottom brackets (side view)

Figure C.21: Third mullion configuration details (mm)

Since the top and bottom panels are symmetric only half of mullion is considered. The numerical values of the properties needed for establishing resistance function are given as follows:

Modulus of Elasticity of aluminum (E_A) = $70,000 \frac{N}{mm^2}$ Density of aluminum = $2710 \frac{kg}{m^3}$

Modulus of Elasticity of steel (E_s) = $200,000 \frac{N}{mm^2}$

Bracket section size: 100 mm × 100 mm × 3 mm

Yield Strength of aluminum (f_y) = 275 MPa

Length of tributary area (L) = 1040 mm

Width of tributary area (W) = 2080 mm

Total mass of system = $\frac{1}{2}$ mullion + $\frac{1}{2}$ load application device =

$$\left[\frac{1}{2} \times 7 \text{ kg} \right] + \left[\frac{1}{2} \times 107 \text{ kg} \right] = 57 \text{ kg}$$

Moment of Inertia per mullion = 2,062,274 mm^4

Stiffness for fixed supported beam assembly,

$$K = \frac{384 EI}{L^3} = \frac{384 \times 70000 \times 2062274}{1040^3} = 49,280 \frac{N}{mm} = 49.28 \frac{N}{m}$$

Stiffness for simply supported beam assembly,

$$K = \frac{384 EI}{5L^3} = \frac{384 \times 70000 \times 2062274}{5 \times 1040^3} = 9,856 \frac{N}{mm} = 9.856 \frac{N}{m}$$

Moment capacity of single mullion,

$$M^- = M^+ = \frac{(f_y) I}{y}$$

Here, M^- and M^+ are negative and positive moment capacities. Therefore, ‘y’ is the maximum distance to the tension fibers from the neutral axis. Thus, substituting the above calculated values in the moment equation,

$$M = \frac{275 \times 2062274}{63.5} = 8,931,107 \text{ N.mm} = 8.9 \text{ kN.m}$$

Considering the fixity at the supports, initial yield resistance is calculated as follows,

$$R_{y1} = \frac{12 M}{L} = \frac{12 \times 8.9}{1.04} = 102 \text{ kN}$$

Corresponding displacement at the fix supports is calculated as follows,

$$U_{y1} = \frac{R_{y1}}{K_f} = \frac{102 \times 1000}{49,280} = 2 \text{ mm}$$

Now, hinges are formed at the supports, and the moment carrying capacity is reduced to half, which will be 4.45 kN.m. The beam will now behave as simply supported beam with the yield resistance calculated as follows,

$$R_{y2} = \frac{8 M}{L} = \frac{8 \times 4.45}{1.04} = 34.2 \text{ kN}$$

Now, the total resistance is the summation of R_{y1} and R_{y2} . Thus, $R_m = 136.2 \text{ kN}$.

The displacement at the formation of the hinge at support is given as follows,

$$U_{y2} = U_{y1} + \frac{R_{y2}}{K_s} = 2 + \frac{34.2 \times 1000}{9,856} = 5.47 \text{ mm}$$

Now, the trilinear resistance function requires to be converted into bilinear resistance function. For doing this, the area under the trilinear and bilinear function is to be equated, which gives an equivalent yield displacement value for the corresponding yield resistance R_m . Following are the calculated values for the same.

$$U_y = 5 \text{ mm} \quad K_{eff} = \frac{R_m}{U_y} = \frac{136.2 \times 1000}{5} = 27240 \frac{N}{mm} = 27.24 \frac{N}{m}$$

Check performance under the pressure-impulse combination applied during the test:

P = 72 kPa (recorded); Ir = 820 kPa-ms (recorded); td = 25 ms (equivalent duration)

Where K_{LM} is the load mass factor; $K_{LM} = 0.78$ for elastic 

response and $K_{LM} = 0.66$ for plastic response.

$$T_s = 2\pi \sqrt{\frac{0.78 \times 57}{27.24 \times 10^6}} = 0.008 \text{ s} = 8 \text{ ms}$$

From the UFC Chart shown on the below:

$$T_d = \frac{2 I_r}{P_r} = \frac{2 \times 905}{72} = 25 \text{ ms} \quad \frac{T_d}{T_s} = \frac{25}{8} = 3.1 \quad \text{for } \mu = 1.0 \quad \text{read} \quad \frac{R_m}{F_1} = 1.8$$

$$F_1 = P_1 (\text{Tributary Area}) = 72 \times 1.04 \times 2.08 = 155.75 \text{ kN}$$

$$R_m (\text{demand}) = 1.8 \times 155.75 = 280.4 \text{ kN} > R_m (\text{capacity}) = 136.2 \text{ kN}$$

Therefore, the mullions will behave in the inelastic range.

Use $K_{LM} = 0.7$ since significant inelasticity is anticipated (a value closer to $K_{LM} = 0.66$).

$$T_s = 2\pi \sqrt{\frac{0.7 \times 57}{27.24 \times 10^6}} = 0.007 \text{ s} = 7 \text{ ms} ; \quad \frac{T_d}{T_s} = \frac{25}{7} = 3.5 \quad ; \quad \frac{R_m}{F_1} = 0.87 \rightarrow \mu = 8$$

$$\text{Damping} = 0.02 \times 2\sqrt{K \cdot M} = 0.02 \times 2 \times \sqrt{27240000 \times 57} = 1576 \text{ Ns/m}$$

Considering $\mu = 1.0 \rightarrow$ UFC chart approach: $5 \times 8 = 40 \text{ mm}$

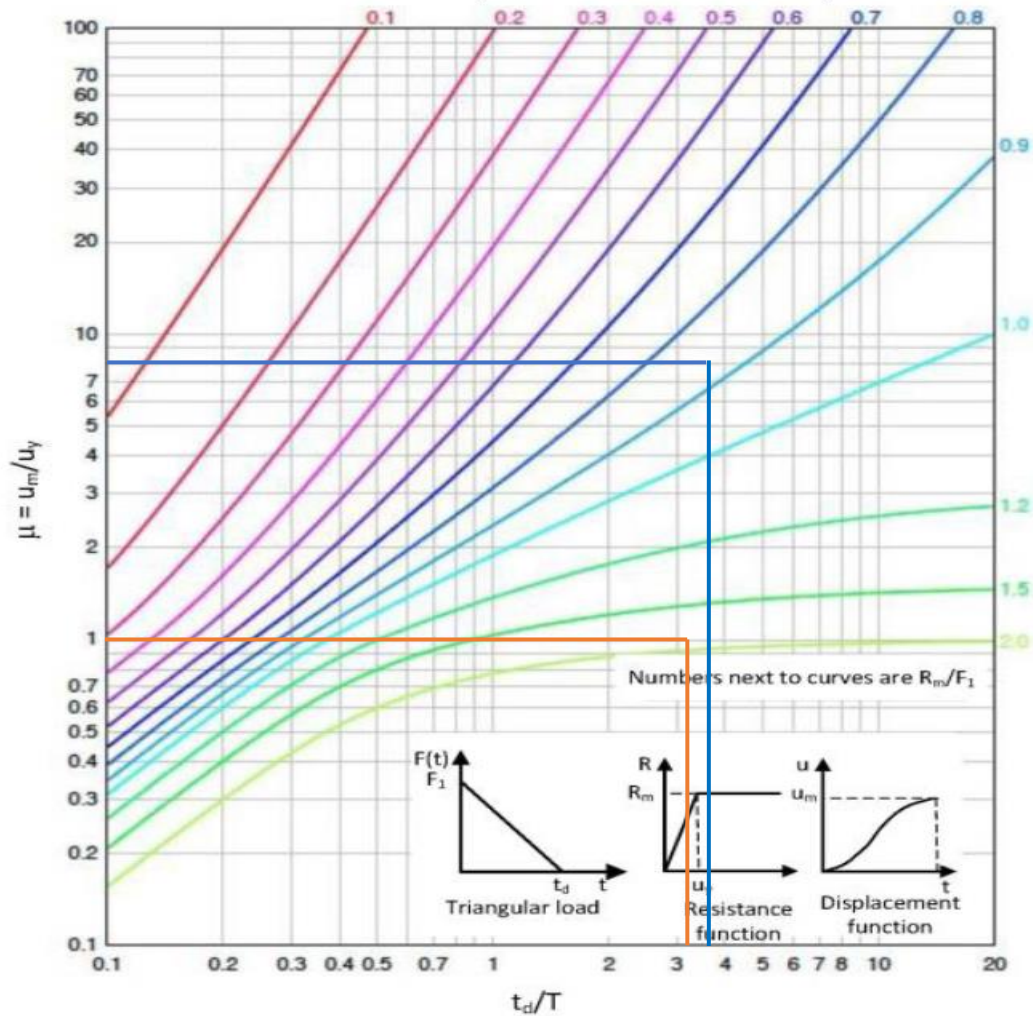


Figure C.22: UFC chart Maximum response of elastoplastic SDOF system (triangular pulses)

C.6.2 Elastic Strain Range Resistance Reaction for Third Mullion with Fixed Supports

Distributed Mass: $57 \times 0.77 = 44 \text{ kg}$

Moment of Inertia per mullion = $2,062,274 \text{ mm}^4$

$$K = \frac{384 EI}{L^3} = \frac{384 \times 70000 \times 2062274}{1040^3} = 49,280 \frac{N}{mm} = 49.28 \frac{N}{m}$$

$$T_s = 2\pi \sqrt{\frac{44}{49.28 \times 10^6}} = 0.006 \text{ s} = 6 \text{ ms} \quad T_d = \frac{2 I_r}{P_r} = \frac{2 \times 905}{72} = 25 \text{ ms}$$

$$\frac{T_d}{T} = \frac{25}{6} = 4.2$$

Dynamic Load Factor (DLF) = 1.9

Exposed specimen area to the blast load: $1040 \text{ mm} \times 2080 \text{ mm} = 2,163,200 \text{ mm}^2$

Elastic Force Resistance: $0.072 \frac{N}{mm^2} \times 2,163,200 \text{ mm}^2 = 155,750 \text{ N}$

Dynamic Resistance: $1.9 \times 155,750 \text{ N} = 295,925 \text{ N}$

Dynamic Reaction (V): $0.36 R + 0.14 F$

$\Rightarrow$ Dynamic Reaction = $(0.36 \times 295,925 \text{ N}) + (0.14 \times 155,750 \text{ N}) = 128,338 \text{ N}$

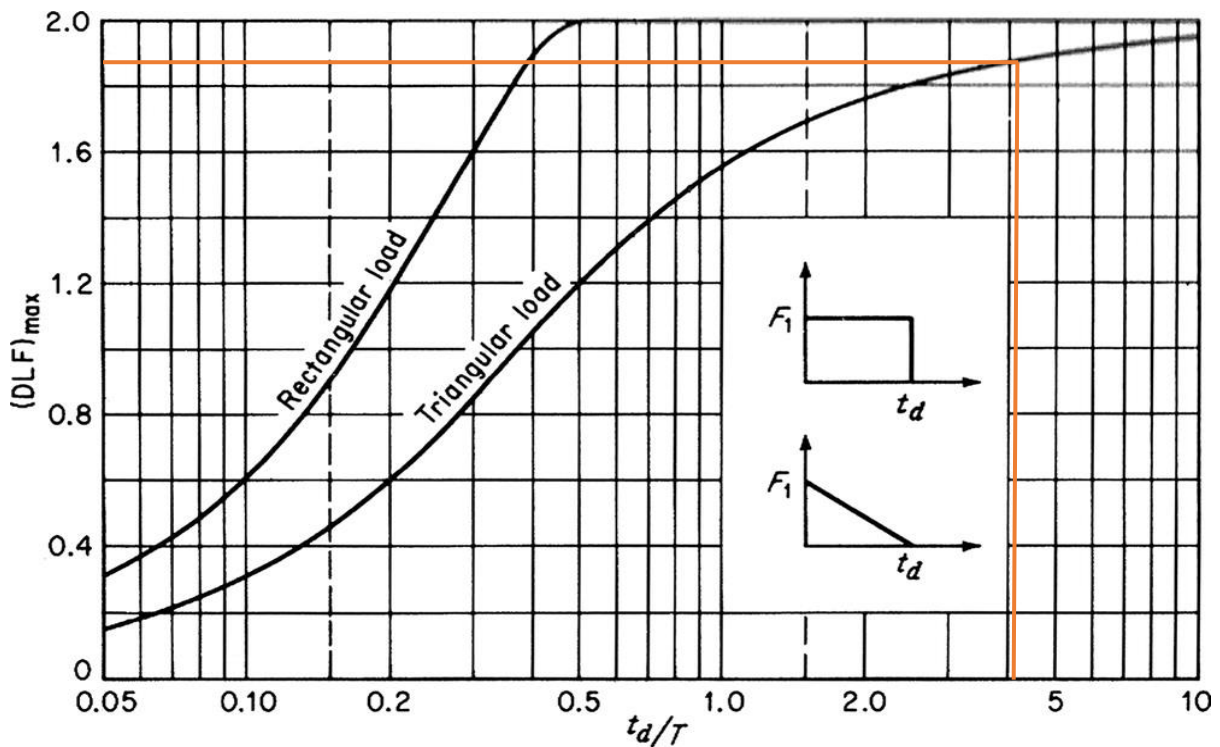


Figure C.23: Dynamic Load Factor graph for Triangular and Rectangular Load

C.6.3 Plastic Strain Range Resistance Reaction for Third Mullion with Fixed Supports

Distributed Mass: $57 \times 0.78 = 45 \text{ kg}$

Moment of Inertia per mullion = $2,062,274 \text{ mm}^4$

$$K = \frac{384 EI}{L^3} = \frac{384 \times 70000 \times 2062274}{1040^3} = 49,280 \frac{N}{mm} = 49.28 \frac{N}{m}$$

$$T_s = 2\pi \sqrt{\frac{45}{49.28 \times 10^6}} = 0.006 \text{ s} = 6 \text{ ms}$$

$$T_d = \frac{2 I_r}{P_r} = \frac{2 \times 905}{72} = 25 \text{ ms}$$

$$\frac{T_d}{T} = \frac{25}{6} = 4.2$$

Dynamic Load Factor (DLF) = 1.9

Exposed specimen area to the blast load: $1040 \text{ mm} \times 2080 \text{ mm} = 2,163,200 \text{ mm}^2$

Elastic Force Resistance: $0.072 \frac{N}{mm^2} \times 2,163,200 \text{ mm}^2 = 155,750 \text{ N}$

Dynamic Resistance: $1.9 \times 155,750 \text{ N} = 295,925 \text{ N}$

Dynamic Reaction (V): $0.39 R + 0.11 F$

$$\Rightarrow \text{Dynamic Reaction} = (0.39 \times 295,925 \text{ N}) + (0.11 \times 155,750 \text{ N}) = 132,543 \text{ N}$$

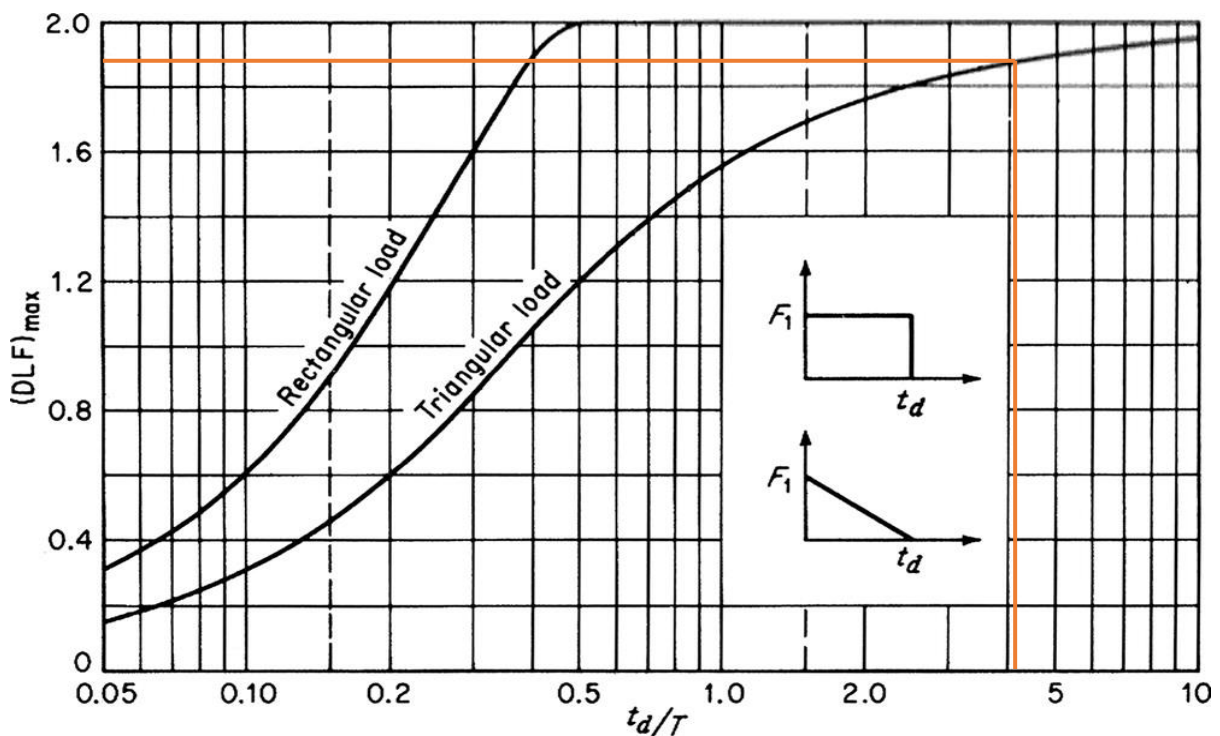
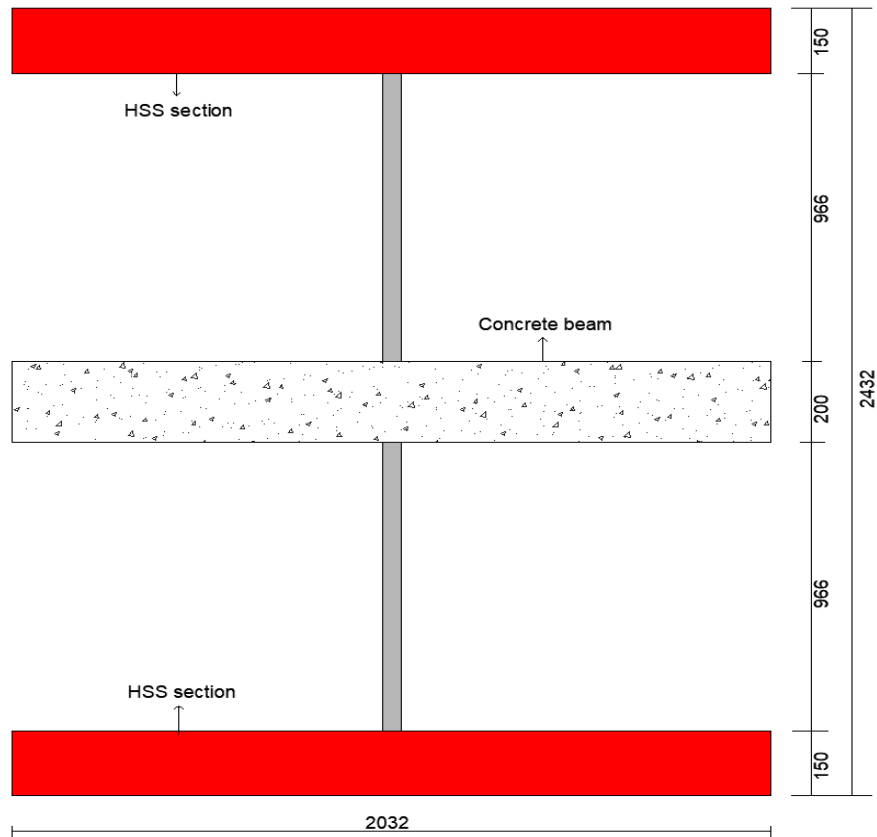


Figure C.24: Dynamic Load Factor graph for Triangular and Rectangular Loads

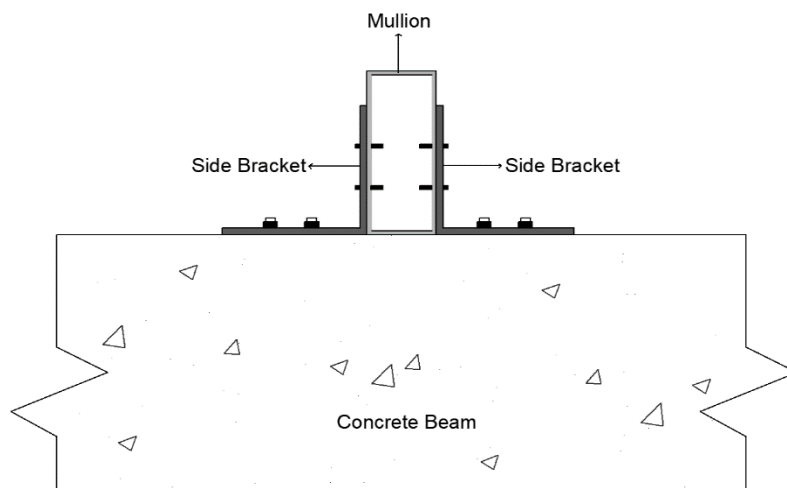
C.7 Resistance Function for Fourth Single Mullion

C.7.1 Resistance Function for Fixed Supports

The cross-section details of fourth Mullion (M-C4) are given as follows:



a) Full-size sketch



b) Retrofitted connection with side brackets (top view)

Figure C.25: Fourth mullion configuration details (mm)

Since the top and bottom panels are symmetric only half of mullion is considered with the half of load application device weight. The numerical values of the properties needed for establishing resistance function are given as follows:

$$\text{Modulus of Elasticity of aluminum } (E_A) = 70,000 \frac{N}{mm^2}$$

$$\text{Modulus of Elasticity of steel } (E_s) = 200,000 \frac{N}{mm^2}$$

$$\text{Density of aluminum} = 2710 \frac{kg}{m^3} \quad \text{Bracket section size: } 100 \text{ mm} \times 100 \text{ mm} \times 3 \text{ mm}$$

$$\text{Yield strength of aluminum} = 275 \text{ MPa} \quad \text{Yield strength of aluminum} = 350 \text{ MPa}$$

$$\text{Length of tributary area } (L) = 1040 \text{ mm}$$

$$\text{Width of tributary area } (W) = 2080 \text{ mm}$$

$$\text{Total mass of M3 system} = \frac{1}{2} \text{ mullion} + \frac{1}{2} \text{ load application device} =$$

$$\left[\frac{1}{2} \times 7 \text{ kg} \right] + \left[\frac{1}{2} \times 107 \text{ kg} \right] = 57 \text{ kg}$$

$$\text{Moment of Inertia per mullion} = 2,062,274 \text{ mm}^4$$

Stiffness for fixed supported beam assembly,

$$K = \frac{384 EI}{L^3} = \frac{384 \times 70000 \times 2062274}{1040^3} = 49,280 \frac{N}{mm} = 49.28 \frac{N}{m}$$

Stiffness for simply supported beam assembly,

$$K = \frac{384 EI}{5L^3} = \frac{384 \times 70000 \times 2062274}{5 \times 1040^3} = 9,856 \frac{N}{mm} = 9.856 \frac{N}{m}$$

Moment capacity of single mullion,

$$M^- = M^+ = \frac{(fy) I}{y}$$

Here, M^- and M^+ are negative and positive moment capacities. Therefore, 'y' is the maximum distance to the tension fibers from the neutral axis. Thus, substituting the above calculated values in the moment equation,

$$M = \frac{275 \times 2062274}{63.5} = 8,931,107 \text{ N.mm} = 8.9 \text{ kN.m}$$

Considering the fixity at the supports, initial yield resistance is calculated as follows,

$$R_{y1} = \frac{12 M}{L} = \frac{12 \times 8.9}{1.04} = 102 \text{ kN}$$

Corresponding displacement at the fix supports is calculated as follows,

$$U_{y1} = \frac{R_{y1}}{K_f} = \frac{102 \times 1000}{49,280} = 2 \text{ mm}$$

Now, hinges are formed at the supports, and the moment carrying capacity is reduced to half, which will be 4.45 kN.m. The beam will now behave as simply supported beam with the yield resistance calculated as follows,

$$R_{y2} = \frac{8 M}{L} = \frac{8 \times 4.45}{1.04} = 34.2 \text{ kN}$$

Now, the total resistance is the summation of R_{y1} and R_{y2} . Thus, $R_m = 136.2 \text{ kN}$.

The displacement at the formation of the hinge at support is given as follows,

$$U_{y2} = U_{y1} + \frac{R_{y2}}{K_s} = 2 + \frac{34.2 \times 1000}{9,856} = 5.5 \text{ mm}$$

Now, the trilinear resistance function requires to be converted into bilinear resistance function. For doing this, the area under the trilinear and bilinear function is to be equated, which gives an equivalent yield displacement value for the corresponding yield resistance R_m . Following are the calculated values for the same.

$$U_y = 5 \text{ mm} \quad K_{eff} = \frac{R_m}{U_y} = \frac{136.2 \times 1000}{5} = 27240 \frac{N}{mm} = 27.24 \frac{N}{m}$$

Check performance under the pressure-impulse combination applied during the test:

P = 71 kPa (recorded); Ir = 812 kPa-ms (recorded); td = 23 ms (equivalent duration)

Where K_{LM} is the load mass factor; $K_{LM} = 0.78$ for elastic response and $K_{LM} = 0.66$ for plastic response.

$$T_s = 2\pi \sqrt{\frac{0.78 \times 57}{27.24 \times 10^6}} = 0.008 \text{ s} = 8 \text{ ms}$$

From the UFC Chart shown on the below:

$$T_d = \frac{2 I_r}{P_r} = \frac{2 \times 812}{71} = 23 \text{ ms} \quad \frac{T_d}{T_s} = \frac{23}{8} = 2.9 \quad \text{for } \mu = 1.0 \quad \text{read} \quad \frac{R_m}{F_1} = 1.9$$

$$F_1 = P_1 (\text{Tributary Area}) = 71 \times 1.04 \times 2.08 = 153.6 \text{ kN}$$

$$R_m (\text{demand}) = 1.9 \times 153.6 = 291.84 \text{ kN} > R_m (\text{capacity}) = 136.2 \text{ kN}$$

Therefore, the mullions will behave in the inelastic range.

Use $K_{LM} = 0.7$ since significant inelasticity is anticipated (a value closer to $K_{LM} = 0.66$).

$$T_s = 2\pi \sqrt{\frac{0.7 \times 57}{27.24 \times 10^6}} = 0.007 \text{ s} = 7 \text{ ms} ; \quad \frac{T_d}{T_s} = \frac{23}{7} = 3.3 ; \quad \frac{R_m}{F_1} = 0.9 \rightarrow \mu = 6$$

$$\text{Damping} = 0.02 \times 2\sqrt{K \cdot M} = 0.02 \times 2 \times \sqrt{27240000 \times 57} = 1576 \text{ Ns/m}$$

Considering $\mu = 1.0 \rightarrow \text{UFC chart approach: } 6 \times 5 = 30 \text{ mm}$

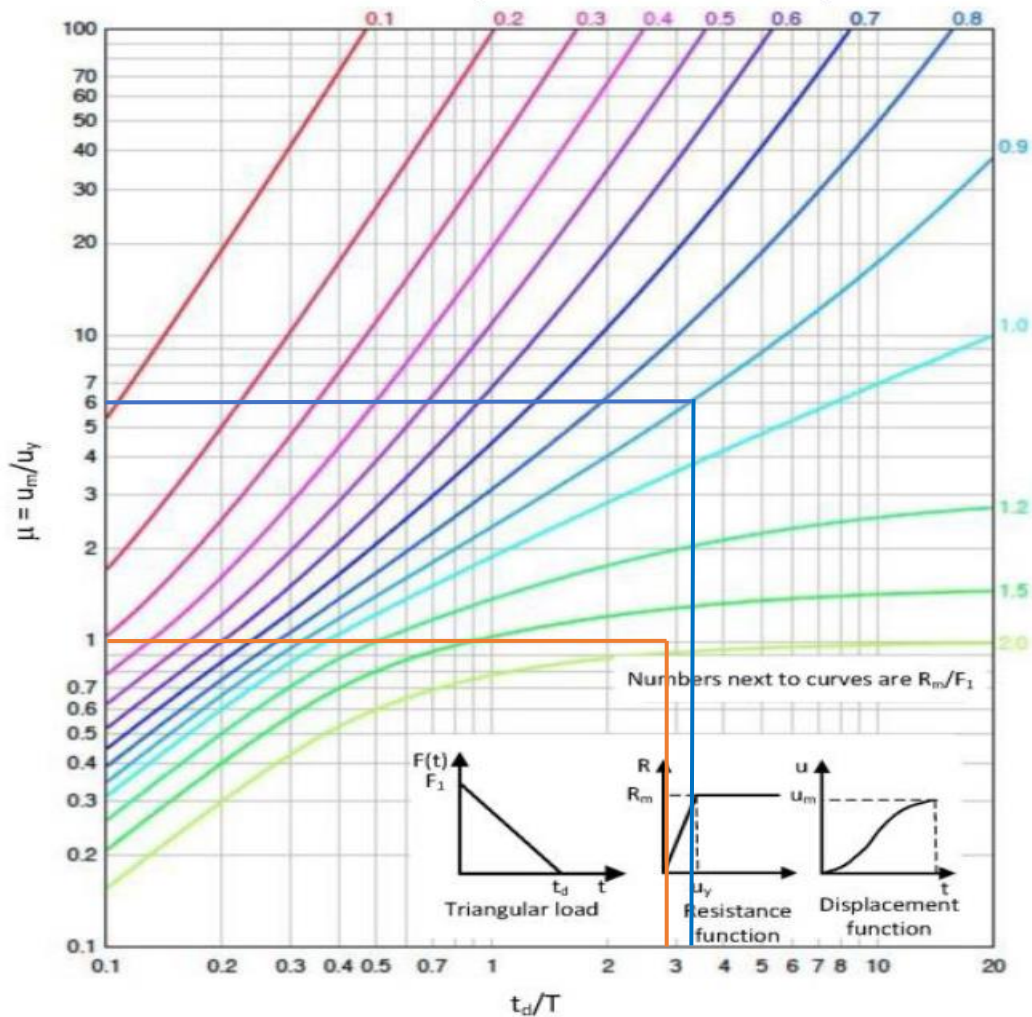


Figure C.26: UFC chart Maximum response of elastoplastic SDOF system (triangular pulses)

C.7.2 Elastic Resistance Reaction for Fourth Mullion with Fixed Supports

Distributed Mass: $57 \times 0.77 = 44 \text{ kg}$

Moment of Inertia per mullion = $2,062,274 \text{ mm}^4$

$$K = \frac{384 EI}{L^3} = \frac{384 \times 70000 \times 2062274}{1040^3} = 49,280 \frac{N}{mm} = 49.28 \frac{N}{m}$$

$$T_s = 2\pi \sqrt{\frac{44}{49.28 \times 10^6}} = 0.006 \text{ s} = 6 \text{ ms} \quad T_d = \frac{2 I_r}{P_r} = \frac{2 \times 812}{71} = 23 \text{ ms}$$

$$\frac{T_d}{T} = \frac{23}{6} = 3.8 \quad \text{Dynamic Load Factor (DLF)} = 1.87$$

Exposed specimen area to the blast load: $1040 \text{ mm} \times 2080 \text{ mm} = 2,163,200 \text{ mm}^2$

Elastic Force Resistance: $0.071 \frac{N}{mm^2} \times 2,163,200 \text{ mm}^2 = 153,587 \text{ N}$

Dynamic Resistance: $1.87 \times 153,587 \text{ N} = 287,208 \text{ N}$

Dynamic Reaction (V): $0.36 R + 0.14 F$

$$\Rightarrow \text{Dynamic Reaction} = (0.36 \times 287,208 \text{ N}) + (0.14 \times 153,587) = 124,897 \text{ N}$$

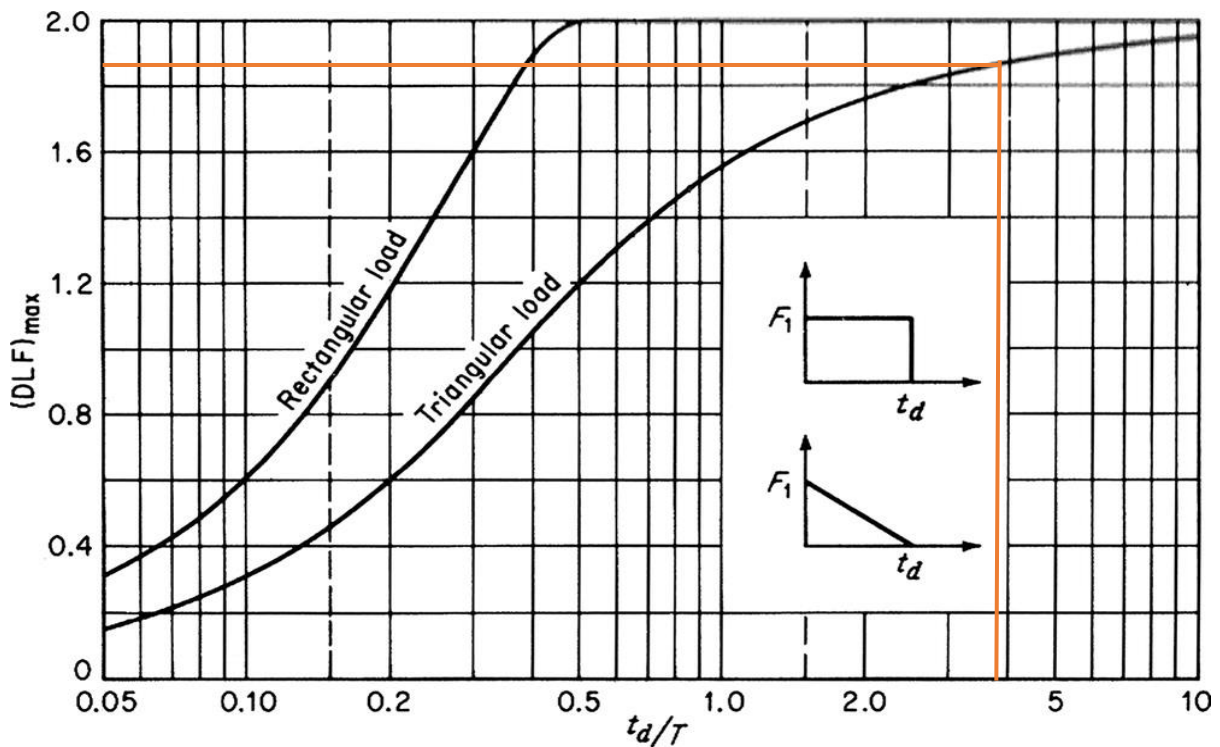


Figure C.27: Dynamic Load Factor graph for Triangular and Rectangular Loads

C.7.3 Plastic Resistance Reaction for Fourth Mullion with Fixed Supports

Distributed Mass: $57 \times 0.78 = 45 \text{ kg}$

Moment of Inertia per mullion = $2,062,274 \text{ mm}^4$

$$K = \frac{384 EI}{L^3} = \frac{384 \times 70000 \times 2062274}{1040^3} = 49,280 \frac{N}{mm} = 49.28 \frac{N}{m}$$

$$T_s = 2\pi \sqrt{\frac{45}{49.28 \times 10^6}} = 0.006 \text{ s} = 6 \text{ ms}$$

$$T_d = \frac{2 I_r}{P_r} = \frac{2 \times 812}{71} = 23 \text{ ms}$$

$$\frac{T_d}{T} = \frac{23}{6} = 3.8$$

Dynamic Load Factor (DLF) = 1.87

Exposed specimen area to the blast load: $1040 \text{ mm} \times 2080 \text{ mm} = 2,163,200 \text{ mm}^2$

Elastic Force Resistance: $0.071 \frac{N}{mm^2} \times 2,163,200 \text{ mm}^2 = 153,587 \text{ N}$

Dynamic Resistance: $1.87 \times 153,587 \text{ N} = 287,208 \text{ N}$

Dynamic Reaction (V): $0.39 R + 0.11 F$

$$\Rightarrow \text{Dynamic Reaction} = (0.39 \times 287,208 \text{ N}) + (0.11 \times 153,587 \text{ N}) = 128,906 \text{ N}$$

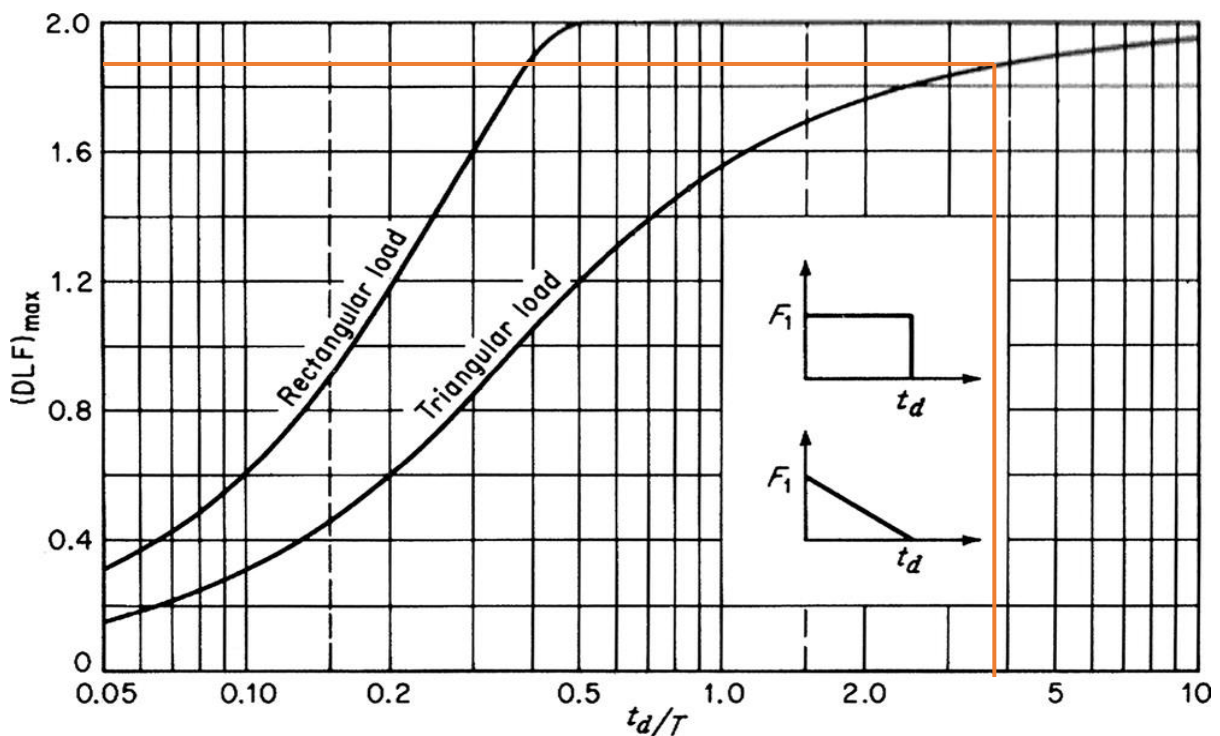
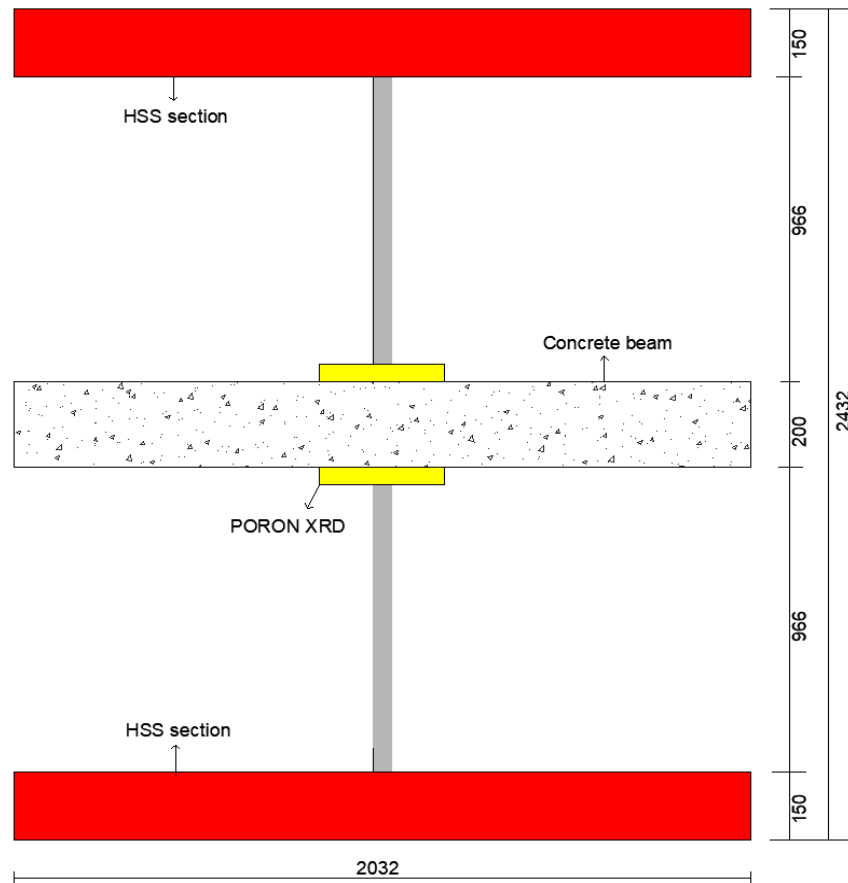


Figure C.28: Dynamic Load Factor graph for Triangular and Rectangular Loads

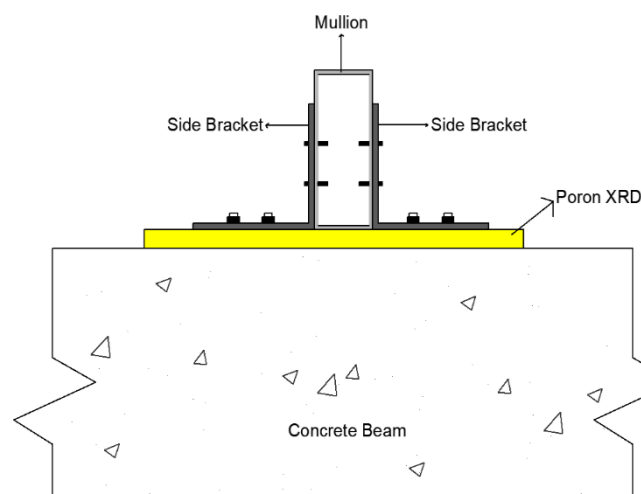
C.8 Resistance Function for Fifth Single Mullion, Inserted Rubber

C.8.1 Resistance Function for Fixed Supports

The cross-section details of fifth Mullion (M-C5) are given as follows:



a) Full-size sketch



b) Retrofitted connection by using side brackets and inserted XRD rubber (top view)

Figure C.29: Fifth mullion configuration details (mm)

Since the top and bottom panels are symmetric only half of mullion is considered with the half of load application device weight. The stiffness of Poron XRD is added to the whole stiffness of system and the calculation procedure is as follows:

$$\text{Modulus of Elasticity of aluminum } (E_A) = 70,000 \frac{N}{mm^2}$$

$$\text{Yield strength of aluminum} = 275 \text{ MPa}$$

$$\text{Length of tributary area (L)} = 1040 \text{ mm} \quad \text{Width of tributary area (W)} = 2080 \text{ mm}$$

$$\text{Poron XRD pad size: } 216 \text{ mm} \times 279 \text{ mm} \times 12.7 \text{ mm} \quad \text{XRD Density: } 240 \frac{kg}{m^3}$$

The potential length for XRD to get squeezed on top, middle, and bottom: 10mm

The weight of Poron XRD is negligible;

$$\text{Total mass of M3 system} = \frac{1}{2} \text{ mullion} + \frac{1}{2} \text{ load application device} =$$

$$\left[\frac{1}{2} \times 7 \text{ kg} \right] + \left[\frac{1}{2} 107 \text{ kg} \right] = 57 \text{ kg}$$

$$\text{Moment of Inertia per mullion} = 2,062,274 \text{ mm}^4$$

Stiffness for fixed supported beam assembly,

$$K = \frac{384 EI}{L^3} = \frac{384 \times 70000 \times 2062274}{1040^3} = 49,280 \frac{N}{mm} = 49.28 \frac{N}{m}$$

Stiffness for simply supported beam assembly,

$$K = \frac{384 EI}{5L^3} = \frac{384 \times 70000 \times 2062274}{5 \times 1040^3} = 9,856 \frac{N}{mm} = 9.856 \frac{N}{m}$$

Moment capacity of single mullion,

$$M^- = M^+ = \frac{(fy) I}{y}$$

Here, M^- and M^+ are negative and positive moment capacities. Therefore, 'y' is the maximum distance to the tension fibers from the neutral axis. Thus, substituting the above calculated values in the moment equation,

$$M = \frac{275 \times 2062274}{63.5} = 8,931,107 \text{ N.mm} = 8.9 \text{ kN.m}$$

Considering the fixity at the supports, initial yield resistance is calculated as follows,

$$R_{y1} = \frac{12 M}{L} = \frac{12 \times 8.9}{1.04} = 102 \text{ kN}$$

Corresponding displacement at the fix supports is calculated as follows,

$$U_{y1} = \frac{R_{y1}}{K_f} = \frac{102 \times 1000}{49,280} = 2 \text{ mm}$$

Now, hinges are formed at the supports, and the moment carrying capacity is reduced to half, which will be 4.45 kN.m. The beam will now behave as simply supported beam with the yield resistance calculated as follows,

$$R_{y2} = \frac{8 M}{L} = \frac{8 \times 4.45}{1.04} = 34.2 \text{ kN}$$

Now, the total resistance is the summation of R_{y1} and R_{y2} . Thus, $R_m = 136.2 \text{ kN}$.

The displacement at the formation of the hinge at support is given as follows,

$$U_{y2} = U_{y1} + \frac{R_{y2}}{K_s} = 2 + \frac{34.2 \times 1000}{9,856} = 5.5 \text{ mm}$$

Now, the trilinear resistance function requires to be converted into bilinear resistance function. For doing this, the area under the trilinear and bilinear function is to be equated, which gives an equivalent yield displacement value for the corresponding yield resistance R_m . Following are the calculated values for the same.

$$U_y = 5 \text{ mm} \quad K_{eff} = \frac{R_m}{U_y} = \frac{136.2 \times 1000}{5} = 27,240 \frac{N}{mm} = 27.2 \frac{N}{m}$$

Check performance under the pressure-impulse combination applied during the test:

P = 71 kPa (recorded); Ir = 830 kPa-ms (recorded); td = 23 ms (equivalent duration)

✚ Where K_{LM} is the load mass factor; $K_{LM} = 0.78$ for elastic response and $K_{LM} = 0.66$ for plastic response.

$$T_s = 2\pi \sqrt{\frac{0.78 \times 57}{27.24 \times 10^6}} = 0.008 \text{ s} = 8 \text{ ms}$$

From the UFC Chart shown on the below:

$$T_d = \frac{2 I_r}{P_r} = \frac{2 \times 830}{71} = 23 \text{ ms} \quad \frac{T_d}{T_s} = \frac{23}{8} = 2.9 \text{ for } \mu = 1.0 \text{ read } \frac{R_m}{F_1} = 1.9$$

$$F_1 = P_1 (\text{Tributary Area}) = 71 \times 1.04 \times 2.08 = 153.60 \text{ kN}$$

$$R_m (\text{demand}) = 1.9 \times 153.60 = 291.8 \text{ kN} > R_m (\text{capacity}) = 136.2 \text{ kN}$$

Therefore, the mullions will behave in the inelastic range.

Use $K_{LM} = 0.7$ since significant inelasticity is anticipated (a value closer to $K_{LM} = 0.66$).

$$T_s = 2\pi \sqrt{\frac{0.7 \times 57}{27.24 \times 10^6}} = 0.007 \text{ s} = 7 \text{ ms}; \quad \frac{T_d}{T_s} = \frac{23}{7} = 3.3 \quad ; \quad \frac{R_m}{F_1} = 0.88 \rightarrow \mu = 11$$

$$\text{Damping} = 0.02 \times 2\sqrt{K \cdot M} = 0.02 \times 2 \times \sqrt{27240000 \times 57} = 1576 \text{ Ns/m}$$

Considering $\mu = 1.0 \rightarrow \text{UFC chart approach: } 5 \times 11 = 55 \text{ mm}$

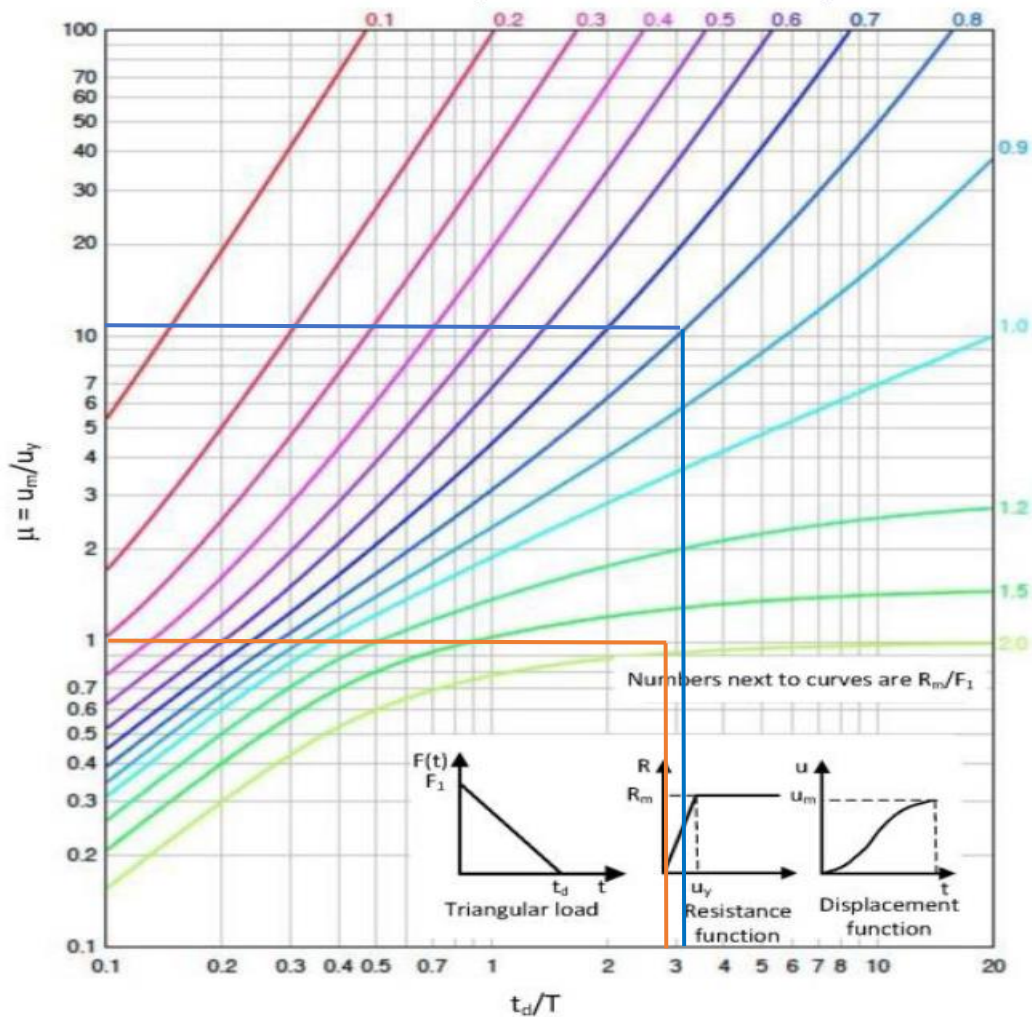


Figure C.30: UFC chart Maximum response of elastoplastic SDOF system (triangular pulses)

C.8.2 Elastic Strain Range Resistance Reaction for Fifth Mullion with Fixed Supports

Distributed Mass: $57 \times 0.77 = 44 \text{ kg}$

Moment of Inertia per mullion = $2,062,274 \text{ mm}^4$

$$K = \frac{384 EI}{L^3} = \frac{384 \times 70000 \times 2062274}{1040^3} = 49,280 \frac{N}{mm} = 49.28 \frac{N}{m}$$

$$T_s = 2\pi \sqrt{\frac{44}{49.28 \times 10^6}} = 0.006 \text{ s} = 6 \text{ ms} \quad T_d = \frac{2 I_r}{P_r} = \frac{2 \times 830}{71} = 23 \text{ ms}$$

$$\frac{T_d}{T} = \frac{23}{6} = 3.8 \quad \text{Dynamic Load Factor (DLF)} = 1.9$$

Exposed specimen area to the blast load: $1040 \text{ mm} \times 2080 \text{ mm} = 2,163,200 \text{ mm}^2$

Elastic Force Resistance: $0.071 \frac{N}{mm^2} \times 2,163,200 \text{ mm}^2 = 153,587 \text{ N}$

Dynamic Resistance: $1.9 \times 153,587 \text{ N} = 291,815 \text{ N}$

Dynamic Reaction (V): $0.36 R + 0.14 F$

$$\Rightarrow \text{Dynamic Reaction} = (0.36 \times 291,815 \text{ N}) + (0.14 \times 153,587 \text{ N}) = 126,555 \text{ N}$$

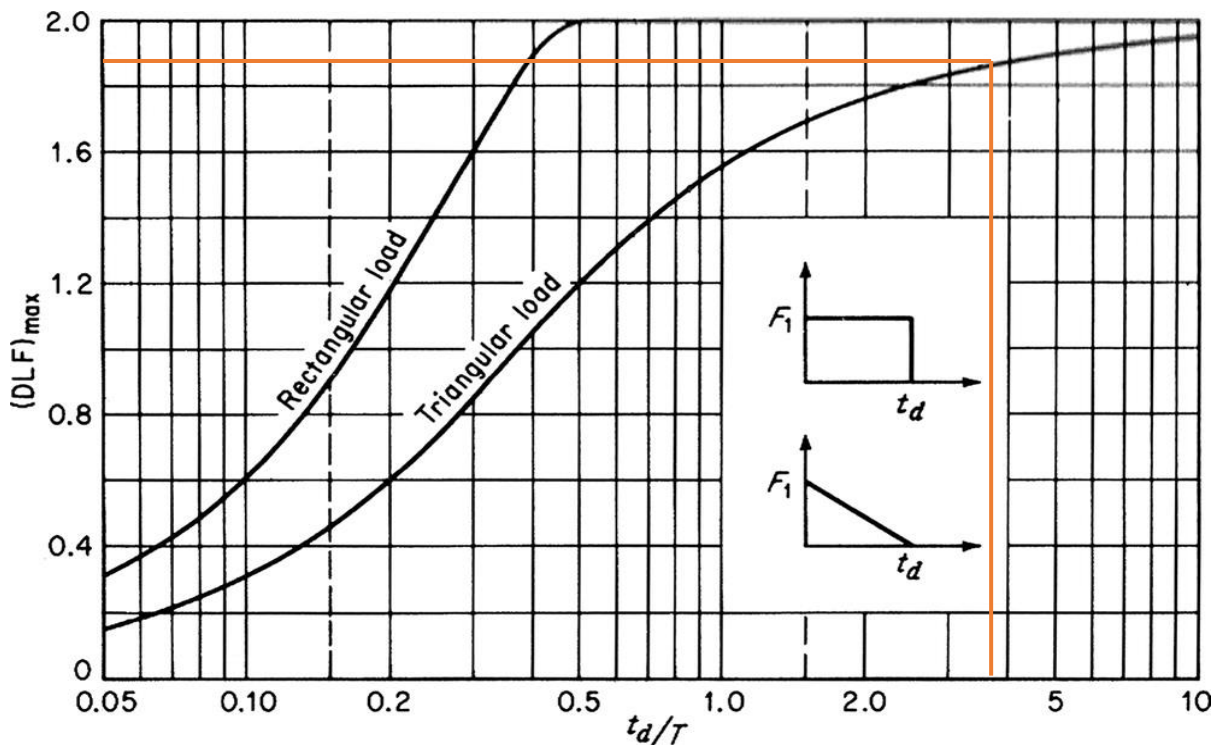


Figure C.31: Dynamic Load Factor graph for Triangular and Rectangular Loads

C.8.3 Plastic Strain Range Resistance Reaction for Fifth Mullion with Fixed Support

Distributed Mass: $57 \times 0.78 = 45 \text{ kg}$

Moment of Inertia per mullion = $2,062,274 \text{ mm}^4$

$$K = \frac{384 EI}{L^3} = \frac{384 \times 70000 \times 2062274}{1040^3} = 49,280 \frac{N}{mm} = 49.28 \frac{N}{m}$$

$$T_s = 2\pi \sqrt{\frac{44}{49.28 \times 10^6}} = 0.006 \text{ s} = 6 \text{ ms} \quad T_d = \frac{2 I_r}{P_r} = \frac{2 \times 830}{71} = 23 \text{ ms}$$

$$\frac{T_d}{T} = \frac{23}{6} = 3.8$$

Dynamic Load Factor (DLF) = 1.9

Exposed specimen area to the blast load: $1040 \text{ mm} \times 2080 \text{ mm} = 2,163,200 \text{ mm}^2$

Elastic Force Resistance: $0.071 \frac{N}{mm^2} \times 2,163,200 \text{ mm}^2 = 153,587 \text{ N}$

Dynamic Resistance: $1.9 \times 153,587 \text{ N} = 291,815 \text{ N}$

Dynamic Reaction (V): $0.39 R + 0.11 F$

$\Rightarrow$ Dynamic Reaction = $(0.39 \times 291,815 \text{ N}) + (0.11 \times 153,587 \text{ N}) = 130,702 \text{ N}$

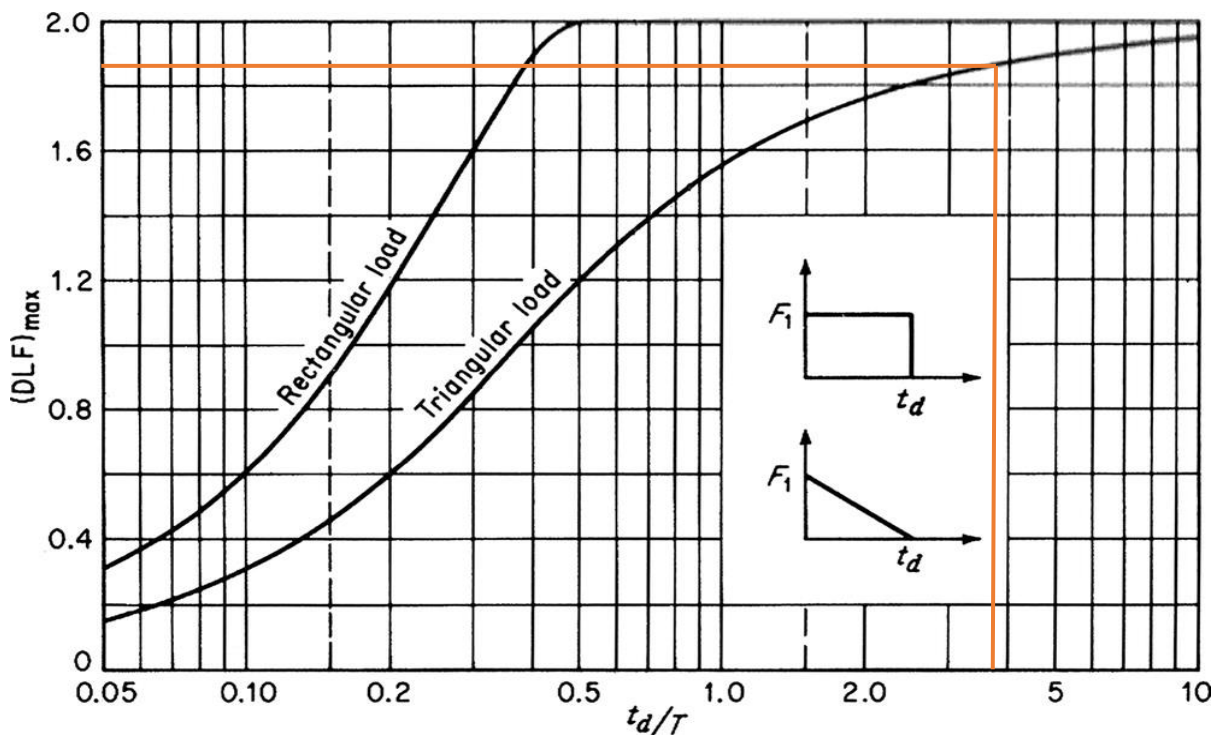
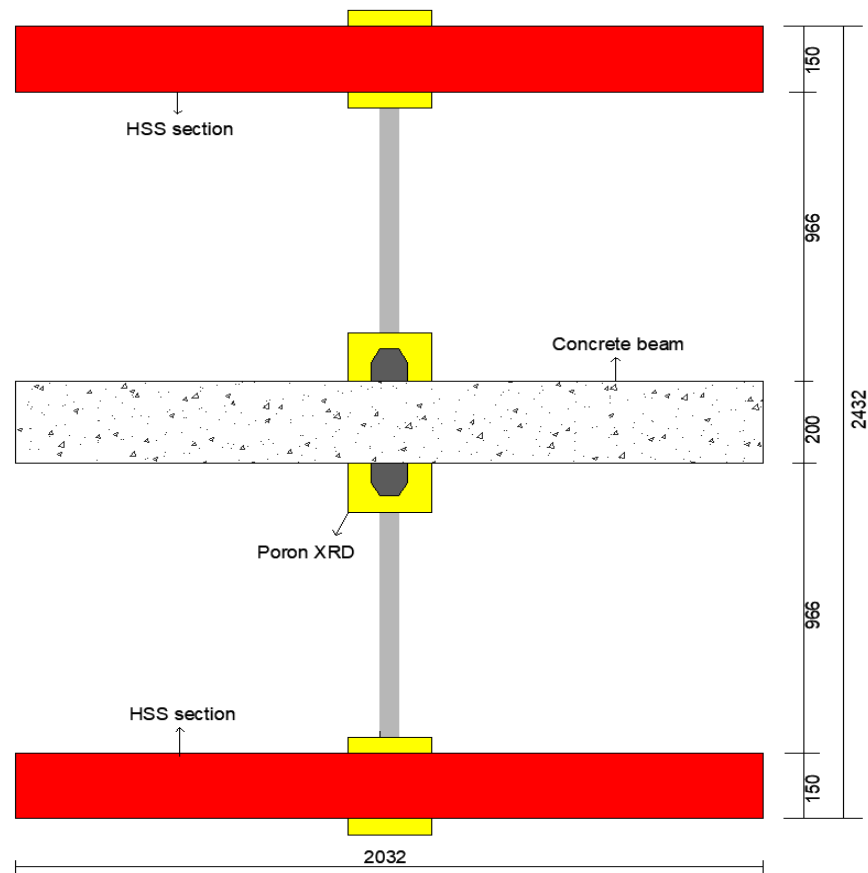


Figure C.32: Dynamic Load Factor graph for Triangular and Rectangular Loads

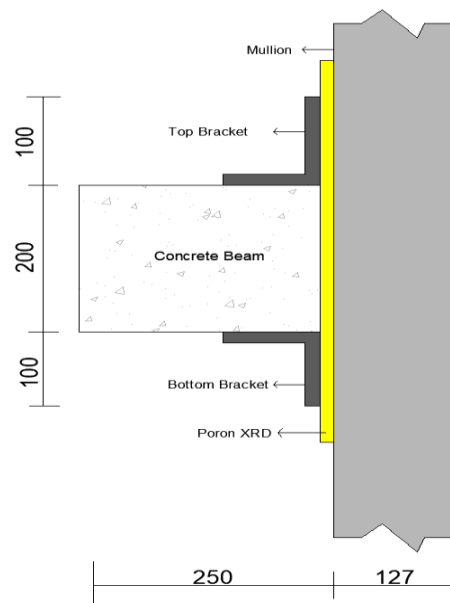
C.9 Resistance Function for Sixth Single Mullion, Inserted Rubber

C.9.1 Resistance Function for Fixed Supports

The cross-section details of sixth Mullion (M-C6) are given as follows:



a) Full-size sketch



b) Retrofitted connection by top and bottom brackets and inserted XRD rubber (side view)

Figure C.33: Sixth mullion configuration details (mm)

Since the top and bottom panels are symmetric only half of mullion is considered with the half of load application device weight. The stiffness of Poron XRD is added to the whole stiffness of system in the calculation procedure:

$$\text{Modulus of Elasticity of aluminum } (E_A) = 70,000 \frac{N}{mm^2}$$

$$\text{Poron XRD size: } 216 \text{ mm} \times 279 \text{ mm} \times 12.7 \text{ mm} \quad \text{XRD Density: } 240 \frac{kg}{m^3}$$

The potential length for XRD to get squeezed: 10mm

$$\text{Yield strength of aluminum} = 275 \text{ MPa} \quad \text{Bracket section size: } 100 \text{ mm} \times 100 \text{ mm} \times 3 \text{ mm}$$

$$\text{Length of tributary area (L)} = 1040 \text{ mm}$$

$$\text{Width of tributary area (W)} = 2080 \text{ mm}$$

$$\text{Total mass of M3 system} = \frac{1}{2} \text{ mullion} + \frac{1}{2} \text{ load application device} =$$

$$\left[\frac{1}{2} \times 7 \text{ kg} \right] + \left[\frac{1}{2} 107 \text{ kg} \right] = 57 \text{ kg}$$

$$\text{Moment of Inertia per mullion} = 2,062,274 \text{ mm}^4$$

Stiffness for fixed supported beam assembly,

$$K = \frac{384 EI}{L^3} = \frac{384 \times 70000 \times 2062274}{1040^3} = 49,280 \frac{N}{mm} = 49.28 \frac{N}{m}$$

Stiffness for simply supported beam assembly,

$$K = \frac{384 EI}{5L^3} = \frac{384 \times 70000 \times 2062274}{5 \times 1040^3} = 9,856 \frac{N}{mm} = 9.856 \frac{N}{m}$$

Moment capacity of single mullion,

$$M^- = M^+ = \frac{(fy) I}{y}$$

Here, M^- and M^+ are negative and positive moment capacities. Therefore, 'y' is the maximum distance to the tension fibers from the neutral axis. Thus, substituting the above calculated values in the moment equation,

$$M = \frac{275 \times 2062274}{63.5} = 8,931,107 \text{ N.mm} = 8.9 \text{ kN.m}$$

Considering the fixity at the supports, initial yield resistance is calculated as follows,

$$R_{y1} = \frac{12 M}{L} = \frac{12 \times 8.9}{1.04} = 102 \text{ kN}$$

Corresponding displacement at the fix supports is calculated as follows,

$$U_{y1} = \frac{R_{y1}}{K_f} = \frac{102 \times 1000}{49,280} = 2 \text{ mm}$$

Now, hinges are formed at the supports, and the moment carrying capacity is reduced to half, which will be 4.45 kN.m. The beam will now behave as simply supported beam with the yield resistance calculated as follows,

$$R_{y2} = \frac{8 M}{L} = \frac{8 \times 4.45}{1.04} = 34.2 \text{ kN}$$

Now, the total resistance is the summation of R_{y1} and R_{y2} . Thus, $R_m = 136.2 \text{ kN}$.

The displacement at the formation of the hinge at support is given as follows,

$$U_{y2} = U_{y1} + \frac{R_{y2}}{K_s} = 2 + \frac{34.2 \times 1000}{9,856} = 5.5 \text{ mm}$$

Now, the trilinear resistance function requires to be converted into bilinear resistance function. For doing this, the area under the trilinear and bilinear function is to be equated, which gives an equivalent yield displacement value for the corresponding yield resistance R_m . Following are the calculated values for the same.

$$U_y = 5 \text{ mm} \quad K_{eff} = \frac{R_m}{U_y} = \frac{136.2 \times 1000}{5} = 27,240 \frac{N}{mm} = 27.24 \frac{N}{m}$$

Check performance under the pressure-impulse combination applied during the test:

P = 73 kPa (recorded); Ir = 912 kPa-ms (recorded); td = 25 ms (equivalent duration)

Where K_{LM} is the load mass factor; $K_{LM} = 0.78$ for elastic response and $K_{LM} = 0.66$ for plastic response.

$$T_s = 2\pi \sqrt{\frac{0.78 \times 57}{27.24 \times 10^6}} = 0.008 \text{ s} = 8 \text{ ms}$$

From the UFC Chart shown on the below:

$$T_d = \frac{2 I_r}{P_r} = \frac{2 \times 912}{73} = 25 \text{ ms} \quad \frac{T_d}{T_s} = \frac{25}{8} = 3.1 \quad \text{for } \mu = 1.0 \quad \text{read} \quad \frac{R_m}{F_1} = 1.85$$

$$F_1 = P_1 (\text{Tributary Area}) = 73 \times 1.04 \times 2.08 = 158 \text{ kN}$$

$$R_m (\text{demand}) = 1.85 \times 158 = 292.3 \text{ kN} > R_m (\text{capacity}) = 142 \text{ kN}$$

Therefore, the mullions will behave in the inelastic range.

Use $K_{LM} = 0.7$ since significant inelasticity is anticipated (a value closer to $K_{LM} = 0.66$).

$$T_s = 2\pi \sqrt{\frac{0.7 \times 57}{27.24 \times 10^6}} = 0.007 \text{ s} = 7 \text{ ms} ; \quad \frac{T_d}{T_s} = \frac{25}{7} = 3.5 \quad ; \quad \frac{R_m}{F_1} = 0.88 \rightarrow \mu = 7$$

$$\text{Damping} = 0.02 \times 2\sqrt{K \cdot M} = 0.02 \times 2 \times \sqrt{27240000 \times 57} = 1576 \text{ Ns/m}$$

Considering $\mu = 1.0 \rightarrow$ UFC chart approach: $5 \times 7 = 35 \text{ mm}$

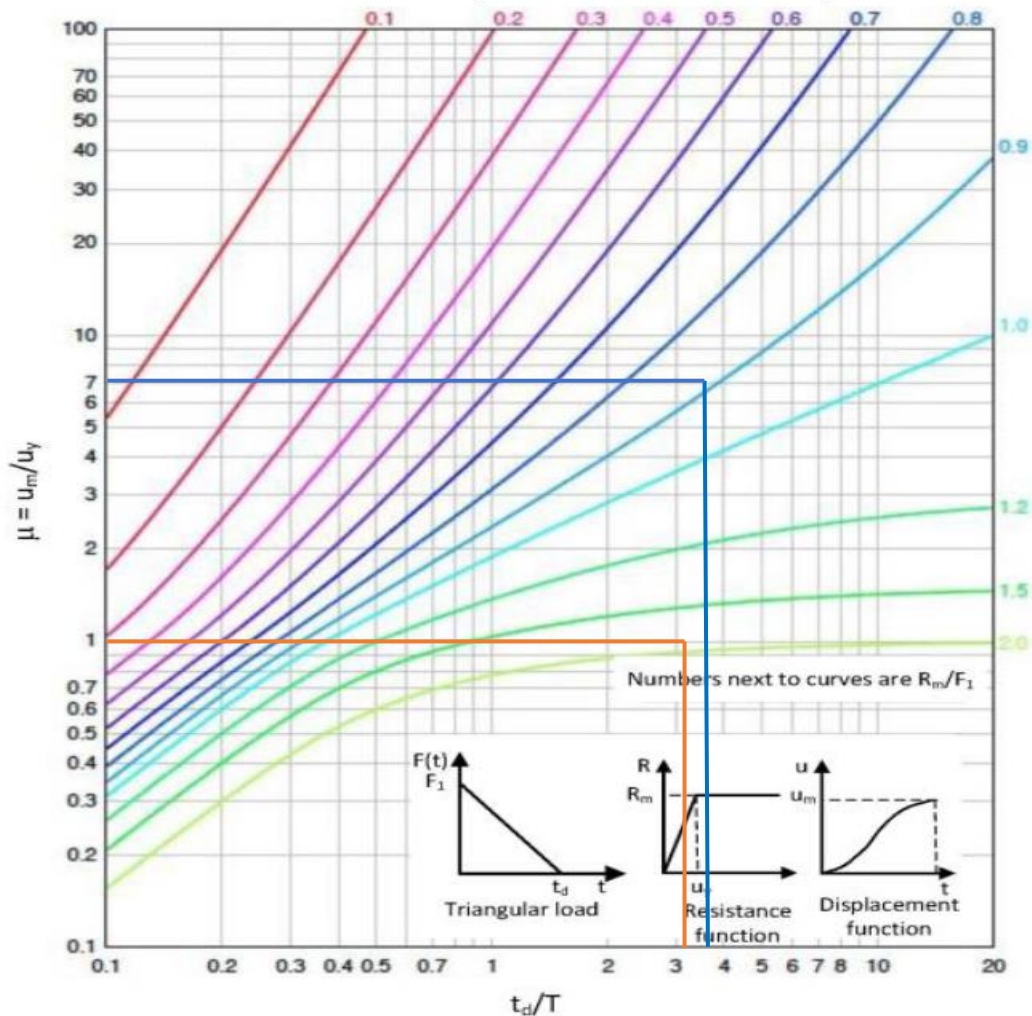


Figure C.34: UFC chart Maximum response of elastoplastic SDOF system (triangular pulses)

C.9.2 Elastic Strain Range Resistance Reaction for Sixth Mullion with Fixed Supports

Distributed Mass: $57 \times 0.77 = 44 \text{ kg}$

Moment of Inertia per mullion = $2,062,274 \text{ mm}^4$

$$K = \frac{384 EI}{L^3} = \frac{384 \times 70000 \times 2062274}{1040^3} = 49,280 \frac{N}{mm} = 49.28 \frac{N}{m}$$

$$T_s = 2\pi \sqrt{\frac{44}{49.28 \times 10^6}} = 0.006 \text{ s} = 6 \text{ ms} \quad T_d = \frac{2 I_r}{P_r} = \frac{2 \times 912}{73} = 25 \text{ ms}$$

$$\frac{T_d}{T} = \frac{25}{6} = 4.2 \quad \text{Dynamic Load Factor (DLF)} = 1.9$$

Exposed specimen area to the blast load: $1040 \text{ mm} \times 2080 \text{ mm} = 2,163,200 \text{ mm}^2$

Elastic Force Resistance: $0.073 \frac{N}{mm^2} \times 2,163,200 \text{ mm}^2 = 157,913 \text{ N}$

Dynamic Resistance: $1.9 \times 157,913 \text{ N} = 300,035 \text{ N}$

Dynamic Reaction (V): $0.36 R + 0.14 F$

$\Rightarrow$ Dynamic Reaction = $(0.36 \times 300,035 \text{ N}) + (0.14 \times 157,913) = 130,120 \text{ N}$

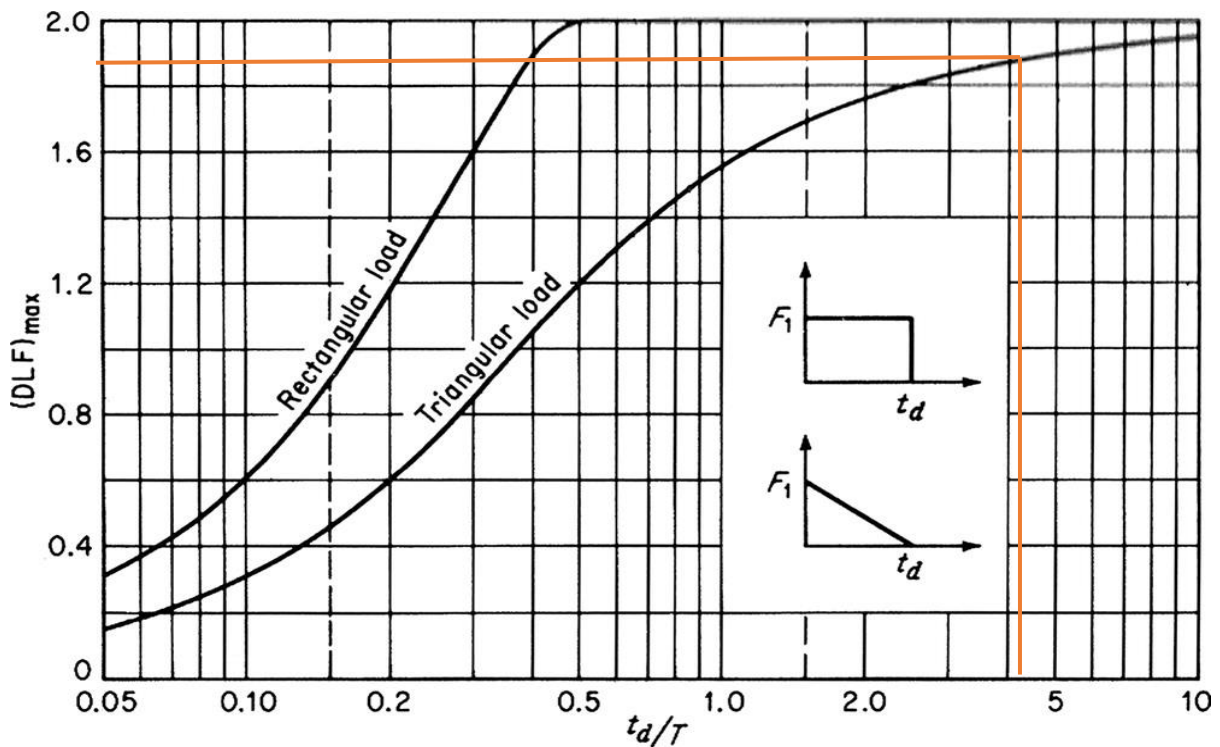


Figure C.35: Dynamic Load Factor graph for Triangular and Rectangular Loads

C.9.3 Plastic Strain Range Resistance Reaction for Sixth Mullion with Fixed Supports

Distributed Mass: $57 \times 0.78 = 45 \text{ kg}$

Moment of Inertia per mullion = $2,062,274 \text{ mm}^4$

$$K = \frac{384 EI}{L^3} = \frac{384 \times 70000 \times 2062274}{1040^3} = 49,280 \frac{N}{mm} = 49.28 \frac{N}{m}$$

$$T_s = 2\pi \sqrt{\frac{44}{49.28 \times 10^6}} = 0.006 \text{ s} = 6 \text{ ms} \quad T_d = \frac{2 I_r}{P_r} = \frac{2 \times 912}{73} = 25 \text{ ms}$$

$$\frac{T_d}{T} = \frac{25}{6} = 4.2 \quad \text{Dynamic Load Factor (DLF)} = 1.9$$

Exposed specimen area to the blast load: $1040 \text{ mm} \times 2080 \text{ mm} = 2,163,200 \text{ mm}^2$

Elastic Force Resistance: $0.073 \frac{N}{mm^2} \times 2,163,200 \text{ mm}^2 = 157,913 \text{ N}$

Dynamic Resistance: $1.9 \times 157,913 \text{ N} = 300,035 \text{ N}$

Dynamic Reaction (V): $0.39 R + 0.11 F$

$$\Rightarrow \text{Dynamic Reaction} = (0.39 \times 300,035 \text{ N}) + (0.11 \times 157,913 \text{ N}) = 134,384 \text{ N}$$

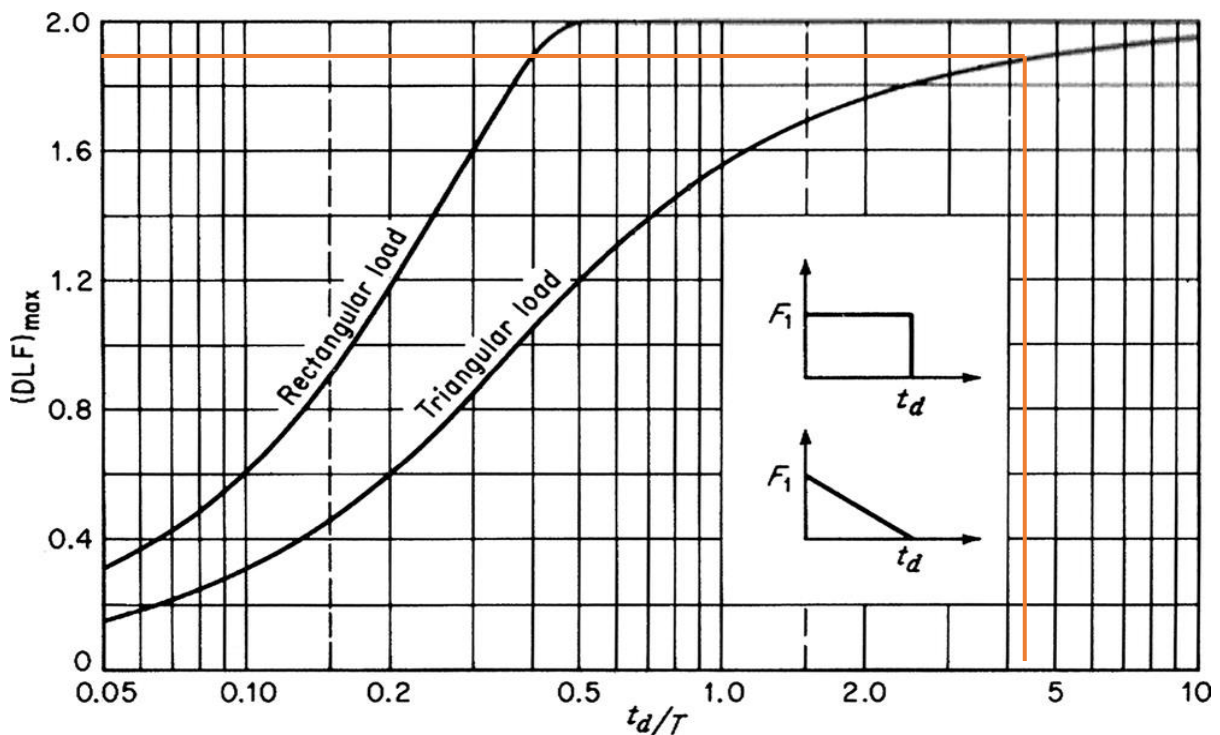
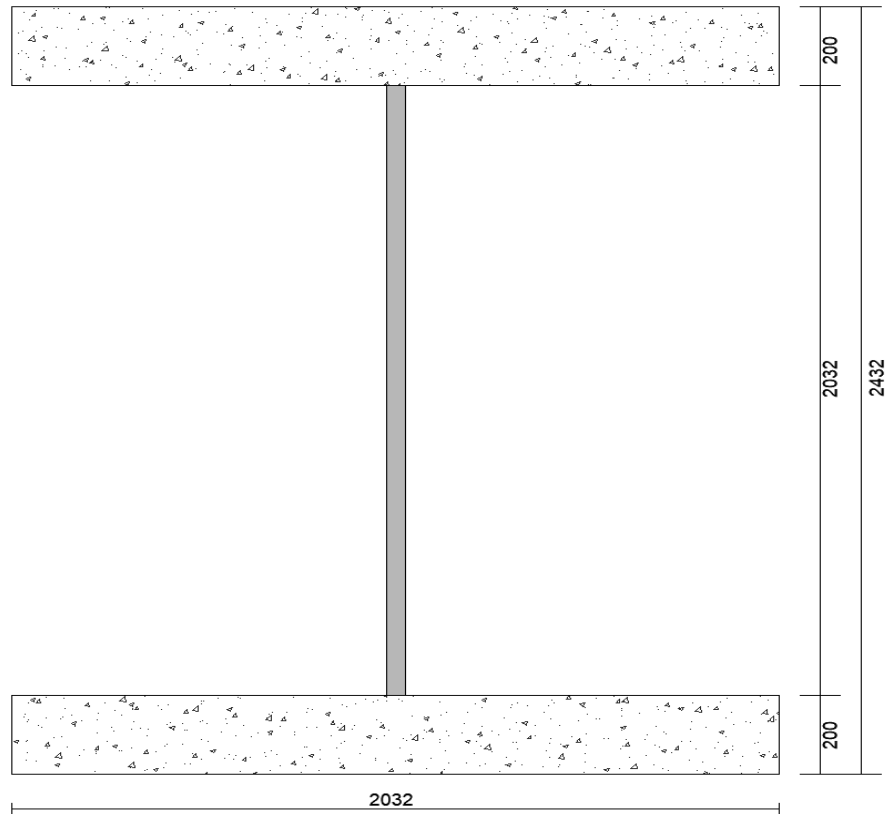


Figure C.36: Dynamic Load Factor graph for Triangular and Rectangular Loads

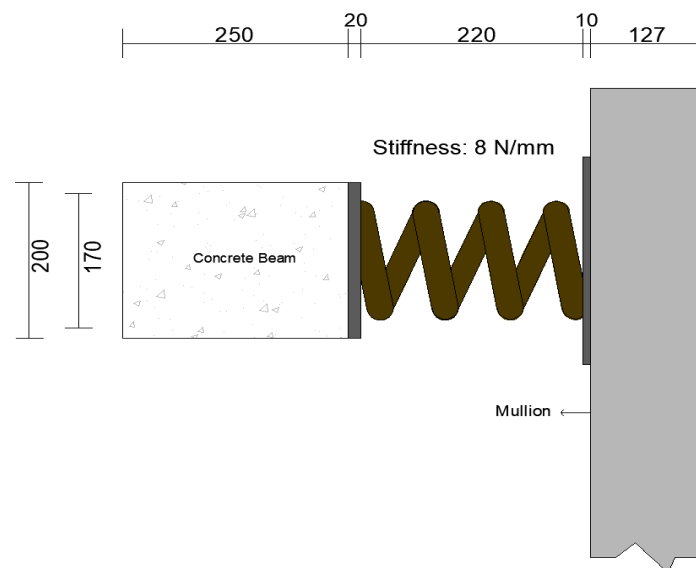
C.10 Resistance Function for Seventh Single Mullion, Inserted Spring

C.10.1 Resistance function for spring support

The cross-section details of seventh Mullion (M-C7) are given as follows:



a) Full-size sketch



b) Mullion connected at the ends to concrete beam through spring (side view)

Figure C.37: M-C7 cross-section details (mm)

The numerical values of the properties for establishing resistance function are as follows:

Yield Strength of Aluminum (f_y) = 275 MPa

Modulus of Elasticity (E_A) = 70,000 N/mm²

Length of Tributary Area (L) = 2083 mm

Width of tributary area (W) = 2083 mm

Spring stiffness: $8 \frac{N}{mm}$; Spring total length: 220 mm; Spring net length: 160 mm

Total mass of system = 1 mullion + load application device + 2 spring systems

= 7 + 107 + [2 × 14] = 142 kg

Moment of Inertia per mullion = 2,062,274 mm⁴

Stiffness of two mullions for simply supported condition + springs' stiffness:

$$K = \frac{384 \times 70000 \times 2062274}{5 \times 2083^3} + [2 \times 8] = 1242 \frac{N}{mm} = 1.242 \frac{N}{m}$$

Moment Capacity of section: $M^- = M^+ = \frac{(f_y)I}{y}$

Here, M^- and M^+ are negative and positive moment capacities. In this equation, 'y' is the maximum distance to the tension fibers from the neutral axis which is half of the section depth (127/2 = 63.5 mm).

Hence, substituting the above calculated values in the moment equation,

$$M = \frac{275 \times 2062274}{63.5} = 8,930,991 \text{ N.mm} = 8.93 \text{ kN.m}$$

Elastic resistance can be calculated as:

$$R_m = \frac{8 M}{L} = \frac{8 \times 8.93}{2.28} = 31.3 \text{ kN}$$

Displacement at yield can be calculated as: $U_y = \frac{R_m}{K_s} = \frac{31.3 \times 1000}{1242} = 26 \text{ mm}$

Check performance under the pressure-impulse combination applied during the test:

P = 24 kPa (recorded); Ir = 245 kPa-ms (recorded); td = 20 ms (equivalent duration)

✚ Period of the mullions: $T_s = 2\pi \sqrt{\frac{0.78 \times 142}{1.242 \times 10^6}} = 0.06 \text{ s} = 60 \text{ ms}$

$$T_d = \frac{2 I_r}{P_r} = \frac{2 \times 245}{24} = 20 \text{ ms} \quad \frac{T_d}{T_s} = \frac{20}{60} = 0.3 \quad \text{for } \mu = 1.0 \quad \text{read} \quad \frac{R_m}{F_1} = 0.85$$

$$F_1 = P_1 (\text{Tributary Area}) = 24 \times 2.08 \times 2.08 = 104 \text{ kN}$$

$$R_m (\text{demand}) = 0.85 \times 104 = 89 \text{ kN} > R_m (\text{capacity}) = 31.3 \text{ kN}$$

Although R_m capacity is less than R_m demand, the extra 160mm length for movement provided by the springs saved the mullion from yielding during the experiment. The springs (8 N/mm stiffness) damped the load and saved the mullion. The rest of UFC chart solution:

$$T_s = 2\pi \sqrt{\frac{0.7 \times 142}{1.226 \times 10^6}} = 0.056 \text{ s} = 56 \text{ ms}; \quad \frac{T_d}{T_s} = \frac{20}{56} = 0.35 \quad ; \quad \frac{R_m}{F_1} = 0.3 \rightarrow \mu = 5.5$$

Considering $\mu = 5.5 \rightarrow$ UFC chart approach: $26 \times 6 = 143 \text{ mm}$

$$\text{Damping} = 0.02 \times 2 \sqrt{K \cdot M} = 0.02 \times 2 \times \sqrt{1226000 \times 142} = 528 \text{ Ns/m}$$

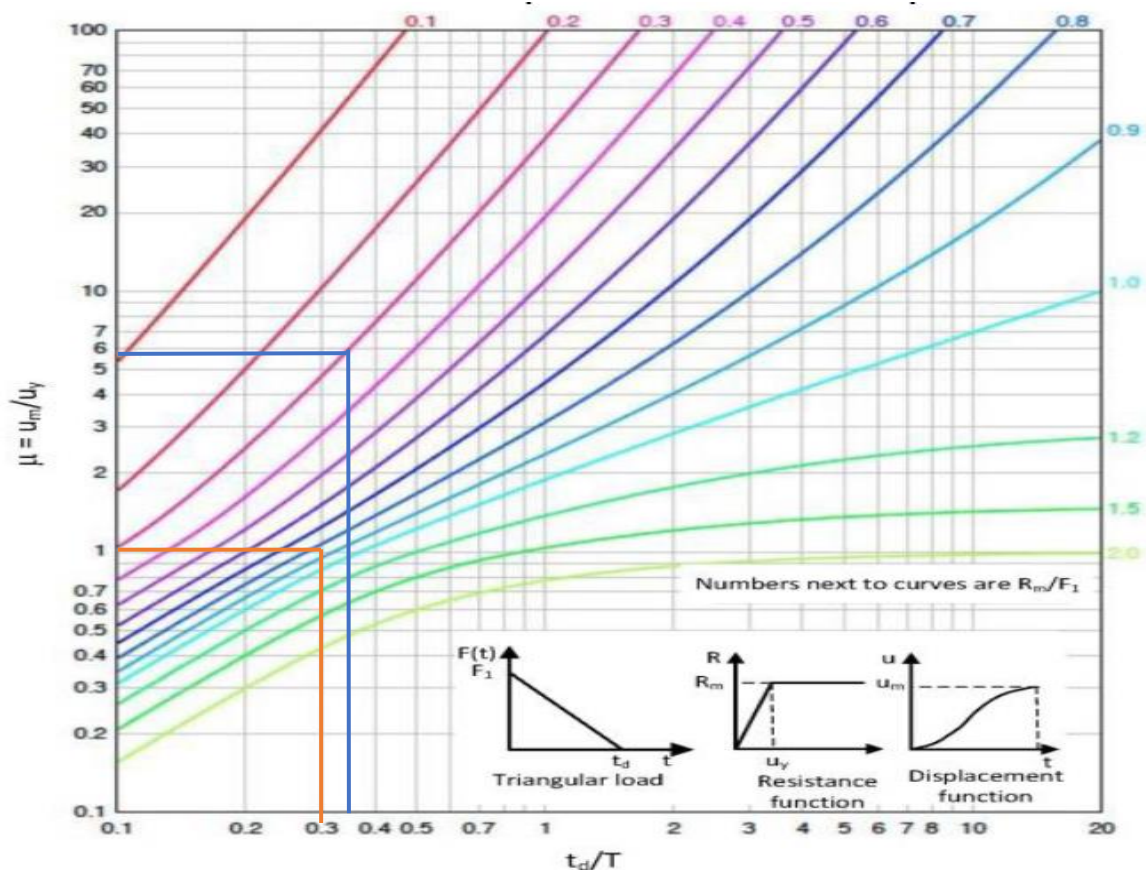


Figure C.38: UFC-chart maximum response of elastoplastic SDOF (triangular pulses)

C.10.2 Elastic Resistance Reaction for Seventh Mullion with Spring Supports

Distributed Mass: $148 \times 0.78 = 115 \text{ kg}$

Moment of Inertia per mullion = $2,062,274 \text{ mm}^4$

$$K = \frac{384 \times 70000 \times 2062274}{5 \times 2083^3} + [2 \times 8] = 1242 \frac{N}{mm} = 1.242 \frac{N}{m}$$

$$T_s = 2\pi \sqrt{\frac{148}{1.242 \times 10^6}} = 0.06 \text{ s} = 60 \text{ ms}$$

$$T_d = \frac{2 I_r}{P_r} = \frac{2 \times 245}{24} = 20 \text{ ms}$$

$$\frac{T_d}{T} = \frac{20}{60} = 0.3$$

Dynamic Load Factor (DLF) = 0.85

Exposed specimen area to the blast load: $2083 \text{ mm} \times 2083 \text{ mm} = 4,338,889 \text{ mm}^2$

Elastic Force Resistance: $0.024 \frac{N}{mm^2} \times 4,338,889 \text{ mm}^2 = 104,133 \text{ N}$

Dynamic Resistance: $0.85 \times 104,133 \text{ N} = 88,513 \text{ N}$

Dynamic Reaction (V): $0.39 R + 0.11 F$

$$\Rightarrow \text{Dynamic Reaction} = (0.39 \times 88,513 \text{ N}) + (0.11 \times 104,133 \text{ N}) = 45,975 \text{ N}$$

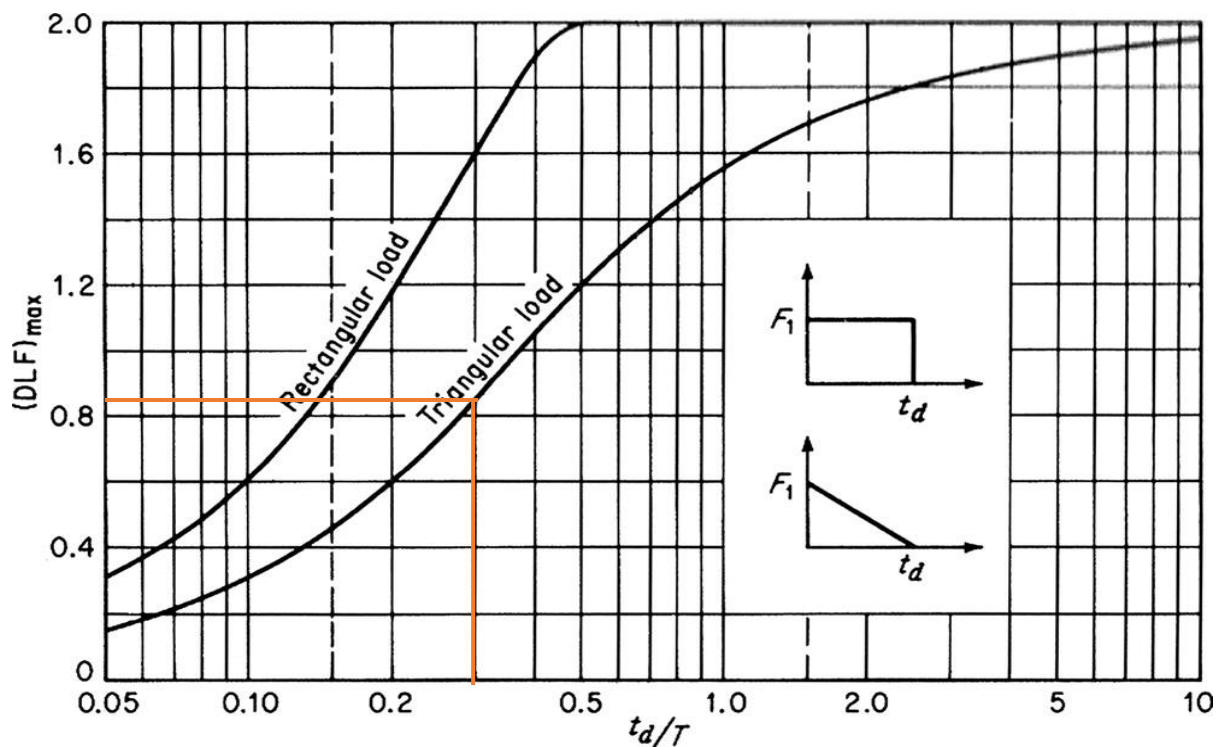


Figure C.39: Dynamic Load Factor graph for Triangular and Rectangular Loads

C.10.3 Plastic Strain Range Resistance Reaction for Seventh Mullion; Spring Supports

Distributed Mass: $142 \times 0.66 = 94 \text{ kg}$

Moment of Inertia per mullion = $2,062,274 \text{ mm}^4$

$$K = \frac{384 \times 70000 \times 2062274}{5 \times 2083^3} + [2 \times 8] = 1242 \frac{N}{mm} = 1.242 \frac{N}{m}$$

$$T_s = 2\pi \sqrt{\frac{94}{1.242 \times 10^6}} = 0.054 \text{ s} = 54 \text{ ms}$$

$$T_d = \frac{2 I_r}{P_r} = \frac{2 \times 245}{24} = 20 \text{ ms}$$

$$\frac{T_d}{T} = \frac{20}{54} = 0.37$$

Dynamic Load Factor (DLF) = 1.05

Exposed specimen area to the blast load: $2083 \text{ mm} \times 2083 \text{ mm} = 4,338,889 \text{ mm}^2$

Elastic Force Resistance: $0.024 \frac{N}{mm^2} \times 4,338,889 \text{ mm}^2 = 104,133 \text{ N}$

Dynamic Resistance: $1.05 \times 104,133 \text{ N} = 109,340 \text{ N}$

Dynamic Reaction (V): $0.38 R + 0.12 F$

Dynamic Reaction = $(0.38 \times 109,340 \text{ N}) + (0.12 \times 104,133 \text{ N}) = 54,045 \text{ N} \Rightarrow$

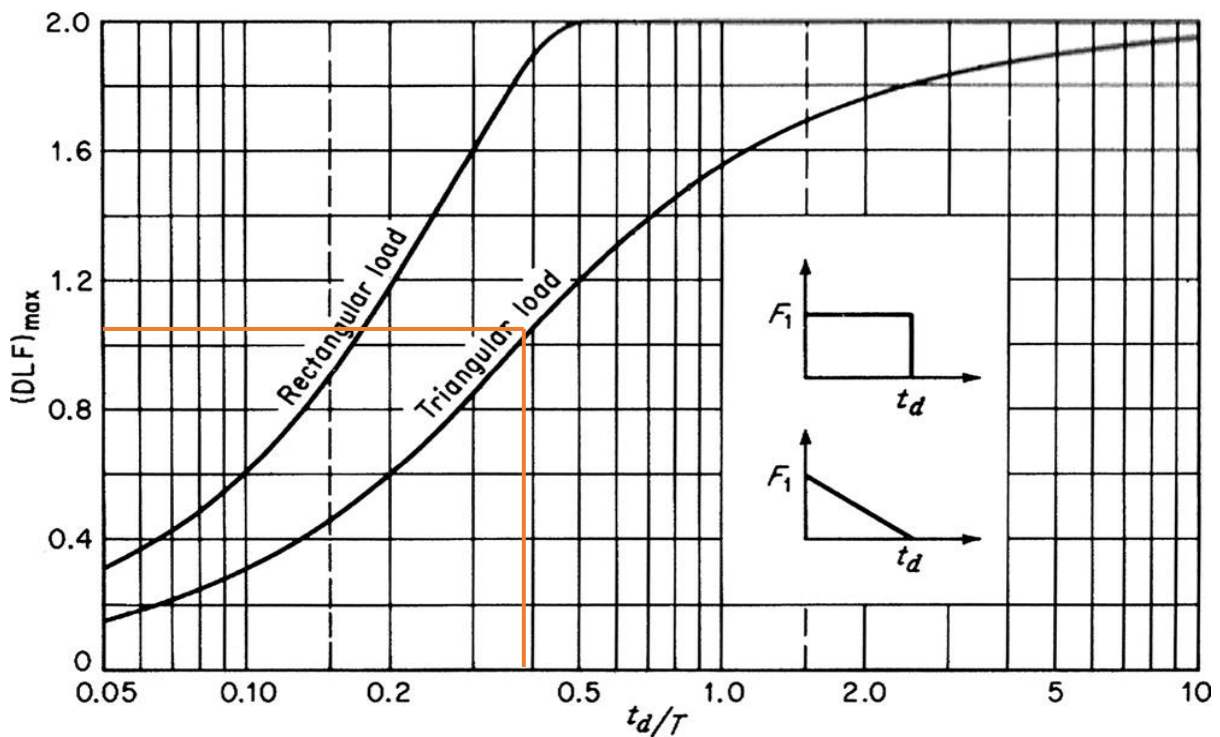
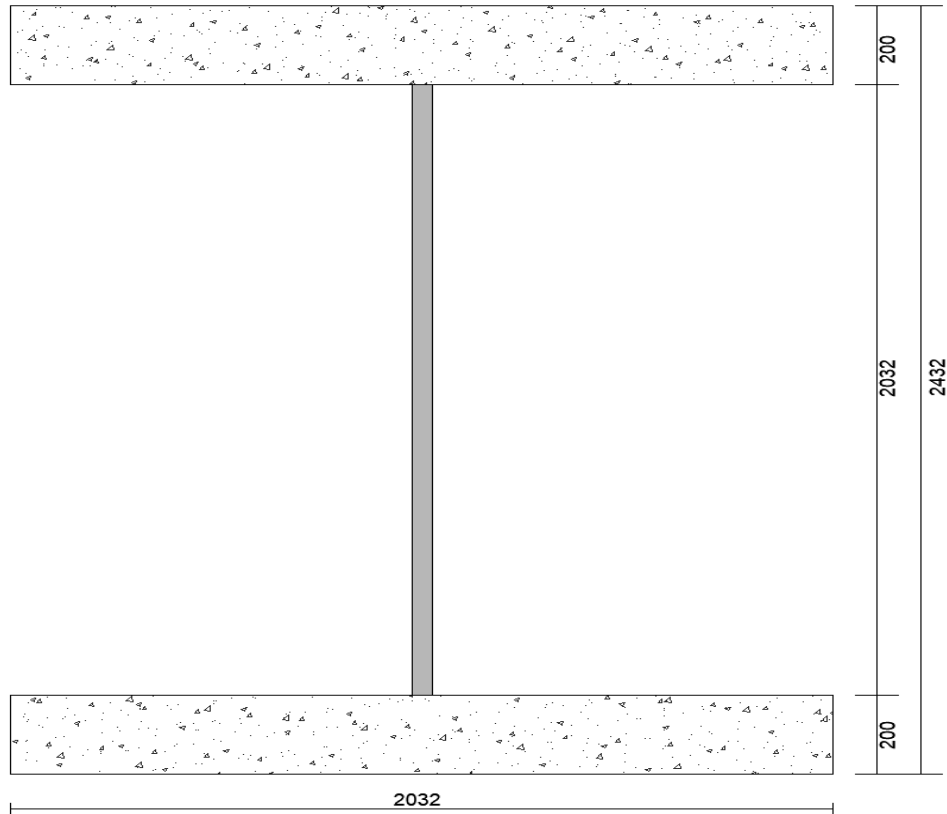


Figure C.40: Dynamic Load Factor graph for Triangular and Rectangular Loads

C.11 Resistance Function for Eighth Single Mullion, Inserted Spring

C.11.1 Resistance Function for Spring Support

The cross-section details of eighth Mullion (M-C8) are given as follows:



a) Full-size sketch

b) Mullion connected at the ends to concrete beam through spring (side view)

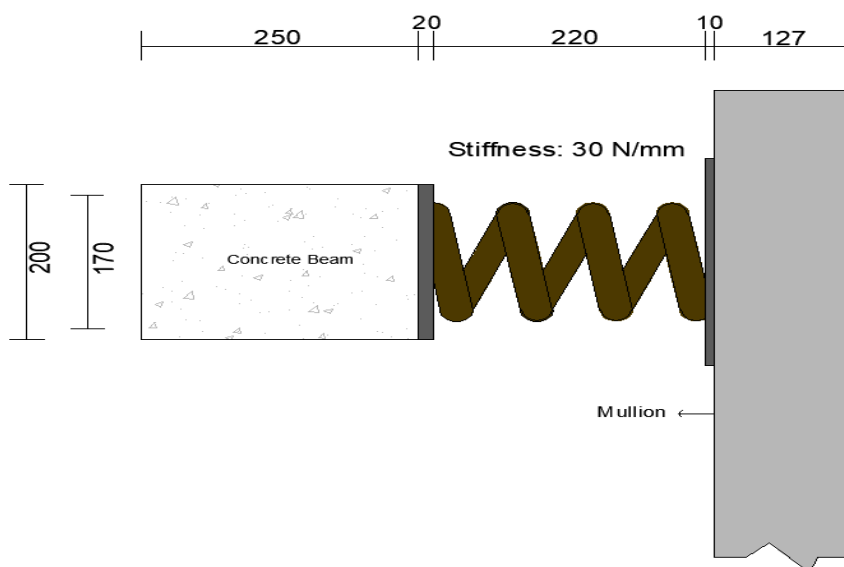


Figure C.41: Eighth mullion configuration details (mm)

The numerical values of the properties for establishing resistance function are as follows:

Yield Strength of Aluminum (f_y) = 275 MPa

Modulus of Elasticity (E_A) = 70,000 N/mm²

Length of Tributary Area (L) = 2083 mm Width of tributary area (W) = 2083 mm

Summation of springs' stiffness: $8 + 22 = 30 \frac{N}{mm}$

Springs total length: 220 mm; Spring net length: 160 mm

Total mass of system = single mullion + load application device + 2 spring systems

= 7 + 107 + [2 × 17] = 148 kg

Moment of Inertia per mullion = 2,062,274 mm⁴

Stiffness of two mullions for simply supported condition + springs' stiffness:

$$K = \frac{384 \times 70000 \times 2062274}{5 \times 2083^3} + [2 \times 30] = 1287 \frac{N}{mm} = 1.287 \frac{N}{m}$$

Moment Capacity of section:

$$M^- = M^+ = \frac{(f_y)I}{y}$$

Here, M^- and M^+ are negative and positive moment capacities. In this equation, 'y' is the maximum distance to the tension fibers from the neutral axis which is half of the section depth (127/2 = 63.5 mm).

Hence, substituting the above calculated values in the moment equation,

$$M = \frac{275 \times 2062274}{63.5} = 8,930,991 \text{ N.mm} = 8.93 \text{ kN.m}$$

Elastic resistance can be calculated as:

$$R_m = \frac{8 M}{L} = \frac{8 \times 8.93}{2.28} = 31.3 \text{ kN}$$

Displacement at yield can be calculated as:

$$U_y = \frac{R_m}{K_s} = \frac{31.3 \times 1000}{1287} = 24 \text{ mm}$$

Check performance under the pressure-impulse combination applied during the test:

P = 31 kPa (recorded); Ir = 312 kPa-ms (recorded); td = 20 ms (equivalent duration)

✚ Period of the mullion:

$$T_s = 2\pi \sqrt{\frac{0.78 \times 148}{1.287 \times 10^6}} = 0.06 \text{ s} = 60 \text{ ms}$$

$$T_d = \frac{2 I_r}{P_r} = \frac{2 \times 312}{31} = 20 \text{ ms} \quad \frac{T_d}{T_s} = \frac{20}{60} = 0.3 \quad \text{for } \mu = 1.0 \quad \text{read} \quad \frac{R_m}{F_1} = 0.85$$

$$F_1 = P_1 (\text{Tributary Area}) = 31 \times 2.08 \times 2.08 = 134 \text{ kN}$$

$$R_m (\text{demand}) = 0.85 \times 134 = 114 \text{ kN} > R_m (\text{capacity}) = 31.3 \text{ kN}$$

Therefore, the mullions will behave in the inelastic range.

Use $K_{LM} = 0.7$ since significant inelasticity is anticipated (a value closer to $K_{LM} = 0.66$).

$$T_s = 2\pi \sqrt{\frac{0.7 \times 148}{1.287 \times 10^6}} = 0.056 \text{ s} = 56 \text{ ms} ; \quad \frac{T_d}{T_s} = \frac{20}{56} = 0.36 \quad ; \quad \frac{R_m}{F_1} = 0.25 \rightarrow \mu = 9$$

$$\text{Damping} = 0.02 \times 2\sqrt{K \cdot M} = 0.02 \times 2 \times \sqrt{1287000 \times 148} = 552 \text{ Ns/m}$$

Considering $\mu = 9 \rightarrow$ UFC chart approach: $24 \times 9 = 216 \text{ mm}$

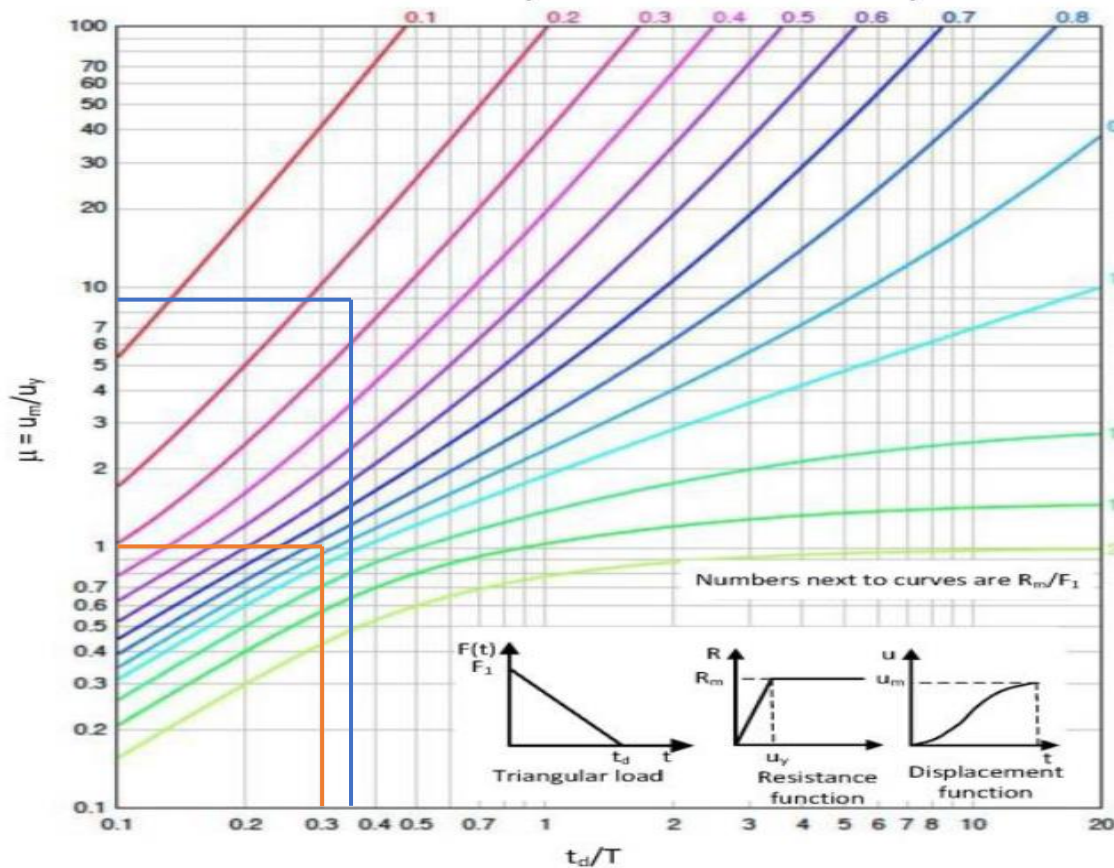


Figure C.42: UFC chart Maximum response of elastoplastic SDOF (triangular pulses)

C.11.2 Elastic Strain Resistance Reaction for Eighth Mullion with Spring Supports

Distributed Mass: $148 \times 0.78 = 116 \text{ kg}$

Moment of Inertia per mullion = $2,062,274 \text{ mm}^4$

$$K = \frac{384 \times 70000 \times 2062274}{5 \times 2083^3} + [2 \times 30] = 1287 \frac{N}{mm} = 1.287 \frac{N}{m}$$

$$T_s = 2\pi \sqrt{\frac{116}{1.287 \times 10^6}} = 0.06 \text{ s} = 60 \text{ ms}$$

$$T_d = \frac{2 I_r}{P_r} = \frac{2 \times 312}{31} = 20 \text{ ms}$$

$$\frac{T_d}{T} = \frac{20}{60} = 0.3$$

Dynamic Load Factor (DLF) = 0.83

Exposed specimen area to the blast load: $2083 \text{ mm} \times 2083 \text{ mm} = 4,338,889 \text{ mm}^2$

Elastic Force Resistance: $0.031 \frac{N}{mm^2} \times 4,338,889 \text{ mm}^2 = 134,505 \text{ N}$

Dynamic Resistance: $0.83 \times 134,505 \text{ N} = 111,640 \text{ N}$

Dynamic Reaction (V): $0.39 R + 0.11 F$

$\Rightarrow$ Dynamic Reaction = $(0.39 \times 111,640 \text{ N}) + (0.11 \times 134,505 \text{ N}) = 58,335 \text{ N}$

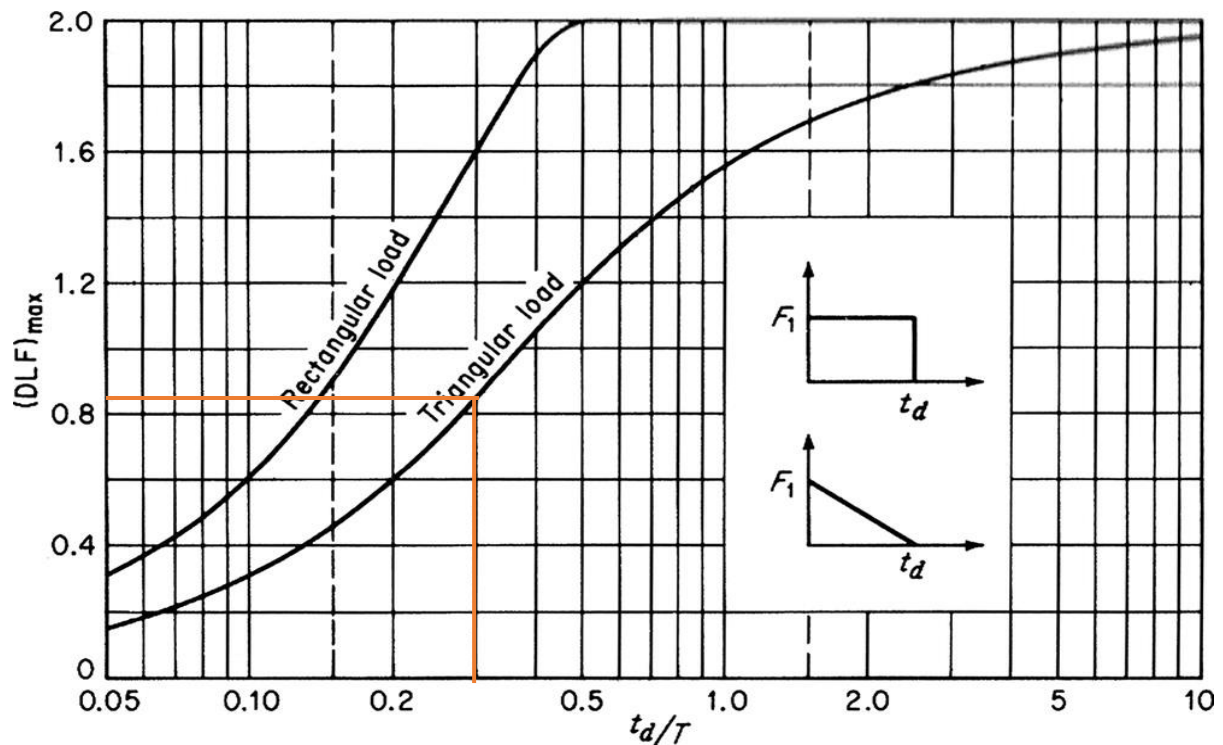


Figure C.43: Dynamic Load Factor graph for Triangular and Rectangular Loads

C.11.3 Plastic Strain Resistance Reaction for Eighth Mullion; Spring Supports

Distributed Mass: $148 \times 0.66 = 98 \text{ kg}$

Moment of Inertia per mullion = $2,062,274 \text{ mm}^4$

$$K = \frac{384 \times 70000 \times 2062274}{5 \times 2083^3} + [2 \times 30] = 1287 \frac{N}{mm} = 1.287 \frac{N}{m}$$

$$T_s = 2\pi \sqrt{\frac{98}{1.287 \times 10^6}} = 0.054 \text{ s} = 54 \text{ ms}$$

$$T_d = \frac{2 I_r}{P_r} = \frac{2 \times 312}{31} = 20 \text{ ms}$$

$$\frac{T_d}{T} = \frac{20}{54} = 0.37$$

Dynamic Load Factor (DLF) = 1

Exposed specimen area to the blast load: $2083 \text{ mm} \times 2083 \text{ mm} = 4,338,889 \text{ mm}^2$

Elastic Force Resistance: $0.031 \frac{N}{mm^2} \times 4,338,889 \text{ mm}^2 = 134,505 \text{ N}$

Dynamic Resistance: $1 \times 134,505 \text{ N} = 134,505 \text{ N}$

Dynamic Reaction (V): $0.38 R + 0.12 F$

$\Rightarrow$ Dynamic Reaction = $(0.38 \times 134,505 \text{ N}) + (0.12 \times 134,505 \text{ N}) = 67,252 \text{ N}$

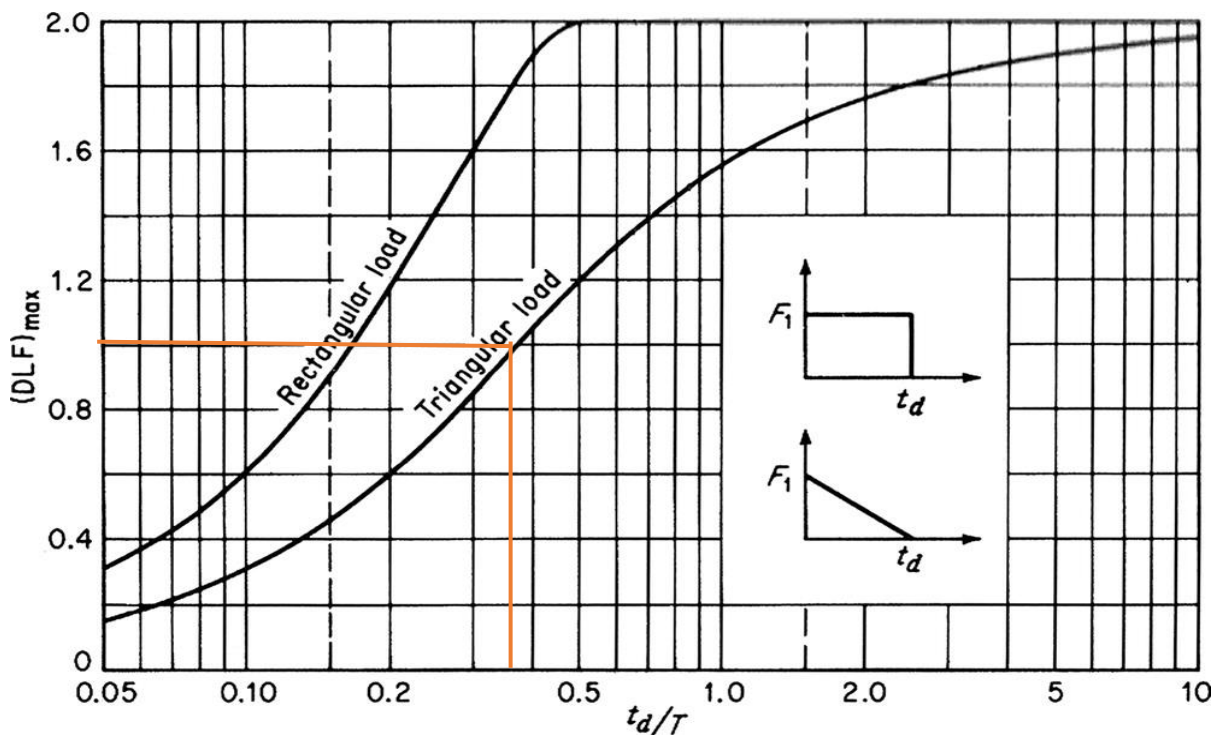
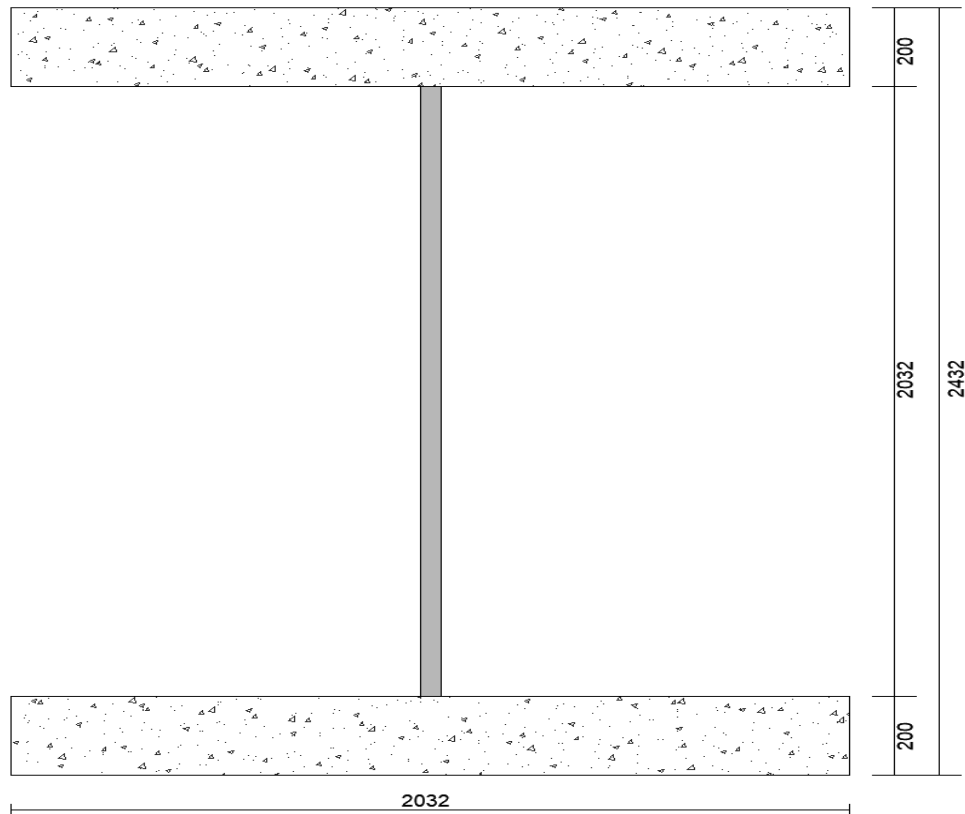


Figure C.44: Dynamic Load Factor graph for Triangular and Rectangular Loads

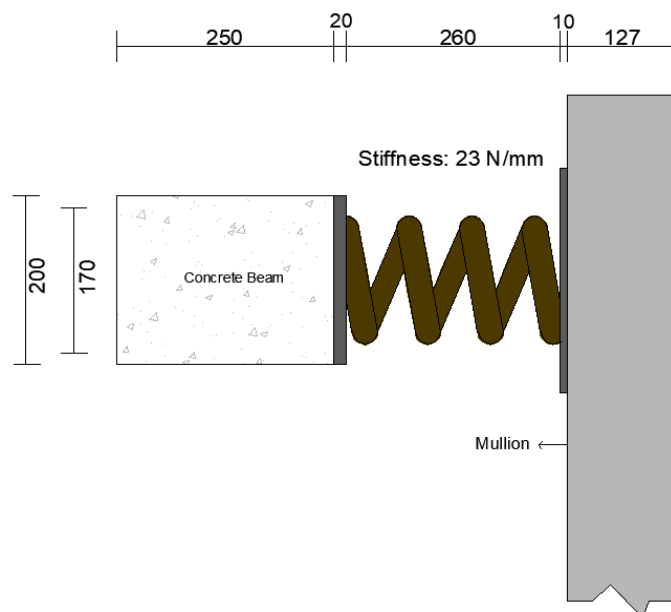
C.12 Resistance Function for Ninth Single Mullion, Inserted Spring

C.12.1 Resistance Function for Spring Supports

The cross-section details of ninth Mullion (M-C9) are given as follows:



a) Full-size sketch



b) Mullion connected at the ends to concrete beam through spring (side view)

Figure C.45: Ninth mullion configuration details (mm)

The numerical values of the properties for establishing resistance function are as follows:

Yield Strength of Aluminum (f_y) = 275 MPa

Modulus of Elasticity (E_A) = 70,000 N/mm²

Length of Tributary Area (L) = 2083 mm Width of tributary area (W) = 2083 mm

Summation of springs' stiffness: $8 + 15 = 23 \frac{N}{mm}$

Spring total length: 260 mm; Spring net length: 200 mm

Total mass of system = 1 mullion + load application device + 2 spring systems

= 7 + 107 + [2 × 17] = 148 kg

Moment of Inertia per mullion = 2,062,274 mm⁴

Stiffness of two mullions for simply supported condition + springs' stiffness:

$$K = \frac{384 \times 70000 \times 2062274}{5 \times 2083^3} + [2 \times 23] = 1273 \frac{N}{mm} = 1.273 \frac{N}{m}$$

Moment Capacity of section:

$$M^- = M^+ = \frac{(f_y)I}{y}$$

Here, M^- and M^+ are negative and positive moment capacities. In this equation, 'y' is the maximum distance to the tension fibers from the neutral axis which is half of the section depth ($127/2 = 63.5$ mm).

Hence, substituting the above calculated values in the moment equation,

$$M = \frac{275 \times 2062274}{63.5} = 8,930,991 \text{ N.mm} = 8.93 \text{ kN.m}$$

Elastic resistance can be calculated as:

$$R_m = \frac{8 M}{L} = \frac{8 \times 8.93}{2.28} = 31.3 \text{ kN}$$

Displacement at yield can be calculated as:

$$U_y = \frac{R_m}{K_s} = \frac{31.3 \times 1000}{1273} = 25 \text{ mm}$$

Check performance under the pressure-impulse combination applied during the test:

P = 22 kPa (recorded); Ir = 224 kPa-ms (recorded); td = 21 ms (equivalent duration)

Period of the mullions: $T_s = 2\pi \sqrt{\frac{0.78 \times 148}{1.273 \times 10^6}} = 0.06 \text{ s} = 60 \text{ ms}$ 🌈

$$T_d = \frac{2 I_r}{P_r} = \frac{2 \times 224}{22} = 21 \text{ ms} \quad \frac{T_d}{T_s} = \frac{21}{60} = 0.35 \quad \text{for } \mu = 1.0 \quad \text{read} \quad \frac{R_m}{F_1} = 0.95$$

$$F_1 = P_1 (\text{Tributary Area}) = 22 \times 2.08 \times 2.08 = 95 \text{ kN}$$

$$R_m (\text{demand}) = 0.95 \times 95 = 90.25 \text{ kN} > R_m (\text{capacity}) = 31.3 \text{ kN}$$

Although R_m capacity is less than R_m demand, the potential 200 mm extra space for mullion movement provided by springs saved the mullion from yielding. The springs (23 N/mm stiffness) damped the load and kept the mullion intact.

$$T_s = 2\pi \sqrt{\frac{0.7 \times 148}{1.273 \times 10^6}} = 0.056 \text{ s} = 56 \text{ ms}; \quad \frac{T_d}{T_s} = \frac{21}{56} = 0.38 \quad ; \quad \frac{R_m}{F_1} = 0.33 \rightarrow \mu = 5$$

Considering $\mu = 5.5 \rightarrow$ UFC chart approach: $25 \times 5 = 125 \text{ mm}$

$$\text{Damping} = 0.02 \times 2\sqrt{K \cdot M} = 0.02 \times 2 \times \sqrt{1273000 \times 148} = 549 \text{ Ns/m}$$

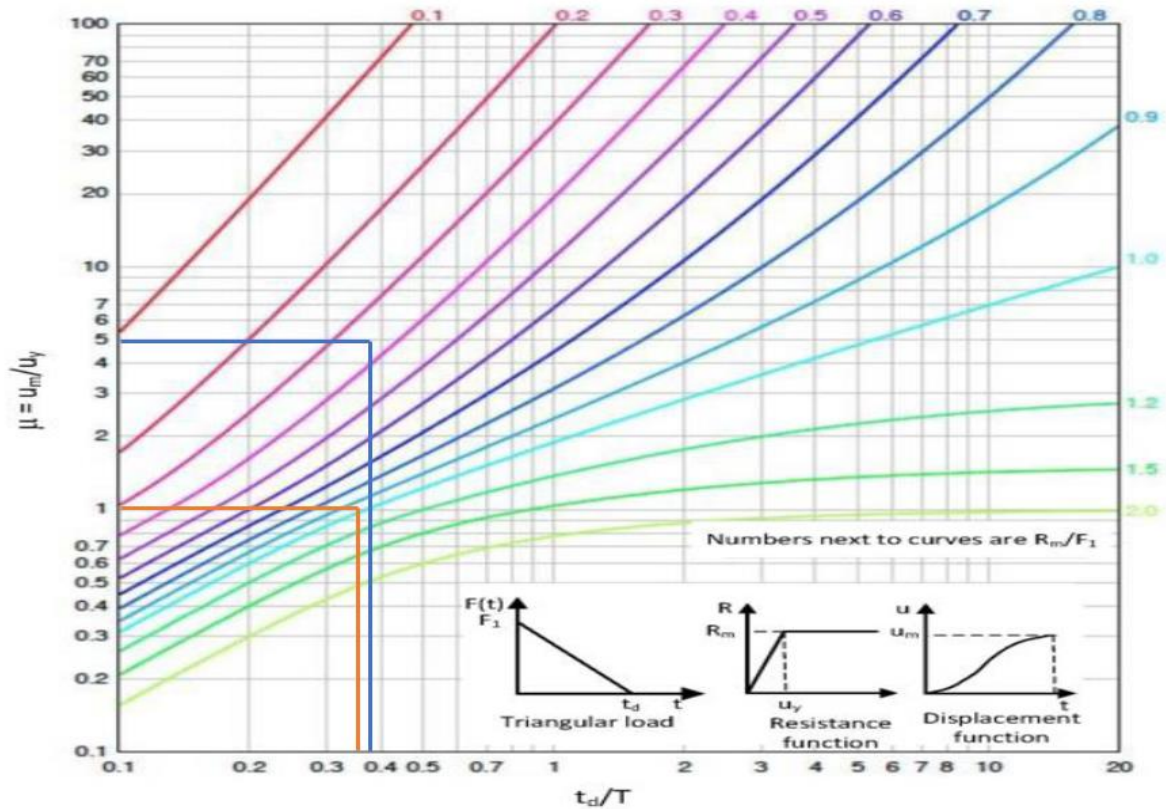


Figure C.46: UFC chart Maximum response of elastoplastic SDOF (triangular pulses)

C.12.2 Elastic Strain Range Resistance Reaction for Ninth mullion with spring Supports

Distributed Mass: $148 \times 0.78 = 116 \text{ kg}$

Moment of Inertia per mullion = $2,062,274 \text{ mm}^4$

$$K = \frac{384 \times 70000 \times 2062274}{5 \times 2083^3} + [2 \times 23] = 1273 \frac{N}{mm} = 1.273 \frac{N}{m}$$

$$T_s = 2\pi \sqrt{\frac{116}{1.273 \times 10^6}} = 0.06 \text{ s} = 60 \text{ ms}$$

$$T_d = \frac{2 I_r}{P_r} = \frac{2 \times 224}{22} = 21 \text{ ms}$$

$$\frac{T_d}{T} = \frac{21}{60} = 0.35$$

Dynamic Load Factor (DLF) = 0.95

Exposed specimen area to the blast load: $2083 \text{ mm} \times 2083 \text{ mm} = 4,338,889 \text{ mm}^2$

Elastic Force Resistance: $0.022 \frac{N}{mm^2} \times 4,338,889 \text{ mm}^2 = 95,455 \text{ N}$

Dynamic Resistance: $0.95 \times 95,455 \text{ N} = 90,682 \text{ N}$

Dynamic Reaction (V): $0.39 R + 0.11 F$

$$\Rightarrow \text{Dynamic Reaction} = (0.39 \times 90,682 \text{ N}) + (0.11 \times 95,455 \text{ N}) = 45,866 \text{ N}$$

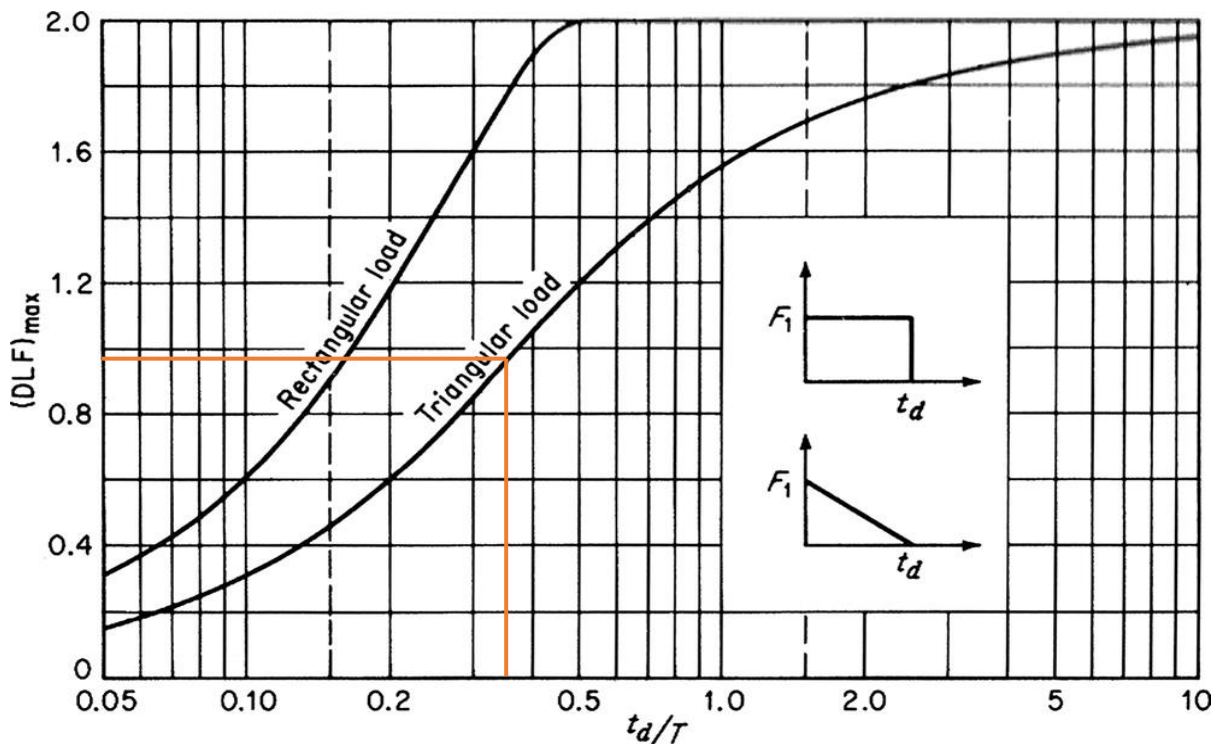


Figure C.47: Dynamic Load Factor graph for Triangular and Rectangular Loads

C.12.3 Plastic Strain Range Resistance Reaction for Ninth mullion; Spring Supports

Distributed Mass: $148 \times 0.66 = 98 \text{ kg}$

Moment of Inertia per mullion = $2,062,274 \text{ mm}^4$

$$K = \frac{384 \times 70000 \times 2062274}{5 \times 2083^3} + [2 \times 23] = 1273 \frac{N}{mm} = 1.273 \frac{N}{m}$$

$$T_s = 2\pi \sqrt{\frac{98}{1.273 \times 10^6}} = 0.05 \text{ s} = 50 \text{ ms}$$

$$T_d = \frac{2 I_r}{P_r} = \frac{2 \times 224}{22} = 21 \text{ ms}$$

$$\frac{T_d}{T} = \frac{21}{50} = 0.42$$

Dynamic Load Factor (DLF) = 1.1

Exposed specimen area to the blast load: $2083 \text{ mm} \times 2083 \text{ mm} = 4,338,889 \text{ mm}^2$

Elastic Force Resistance: $0.022 \frac{N}{mm^2} \times 4,338,889 \text{ mm}^2 = 95,455 \text{ N}$

Dynamic Resistance: $1.1 \times 95,455 \text{ N} = 105,001 \text{ N}$

Dynamic Reaction (V): $0.38 R + 0.12 F$

$\Rightarrow$ Dynamic Reaction = $(0.38 \times 105,001 \text{ N}) + (0.12 \times 95,455 \text{ N}) = 51,355 \text{ N}$

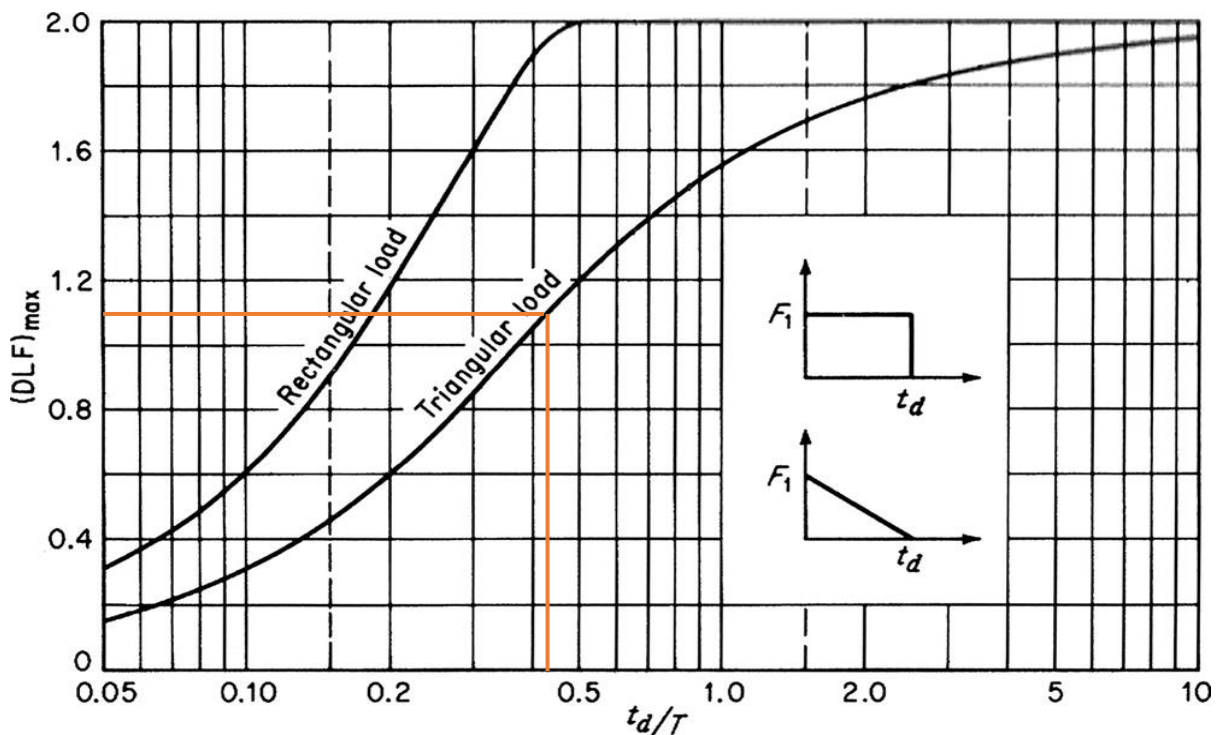


Figure C.48: Dynamic Load Factor graph for Triangular and Rectangular Loads

Appendix D. CW-Blast Software

Calculating the Middle Transoms Support Reaction Using CW-Blast

CW-Blast is an explicit SDOF program designed in particular for the analysis of various glazing systems subject to blast loading. The program was developed at the University of Ottawa (Gilbert Bélec et al. 2021) and, as of yet, is not publicly available. It has been validated with both static and dynamic experimental data with good results used for Global Affairs Canada (GAC). The user should define the panel properties as well as membrane details. In addition, the tributary area of curtain wall sections (mullion and transom) can be specified. Default material properties for mullion, transom, laminated glass, polyvinyl butyral, and air gap are provided, but the user may modify the values or create their own custom material. Support conditions may be pinned, fixed fixity or a combination of both. There are three options for inputting the blast load: charge weight and standoff, peak pressure and impulse, or time-pressure pairs. CW-Blast can provide the user with variety of outcomes including: resistance function, panel response, mullion and transom force-displacement, maximum displacement, maximum velocity, maximum acceleration, maximum resisting force, and support reaction for both mullion and transom.

The initial analysis should either assume the mullion/transom will behave elastically or inelastically so that the dynamic factors can be selected and from the result. The stiffness coefficient can be calculated by dividing the mullion support reaction to the maximum displacement obtained from CW-Blast software. Now, the analysis should be done again based on the section size and tributary area of the middle transom. The stiffness coefficient obtained from mullion analysis should be entered in the left/right release and the fixity has to be specified as pinned for the middle transom. Basically, left/right releases represent the equivalent spring constant of the vertical mullion that serves as a support point for the transom. These releases can invoke relaxation of support. Finally, after conducting the analysis, the middle transom support reaction is presented by CW-Blast in the analysis overview section.

D.1 Force Calculation of First Curtain Wall (CW1)

D.1.1 Resistant Function for Middle Transom (fixed supports)

The Tributary area details and CW-Blast software inputs are given as follows:

Yield Strength of Aluminum (F_y) = 275 MPa Mass Density of Aluminum = 2700 kg/m^3

Modulus of Elasticity of aluminum (E_A) = $70,000 \text{ N/mm}^2$

Mullion Section Size: $127 \text{ mm} \times 51 \text{ mm} \times 3 \text{ mm}$

Transom Section Size: $76 \text{ mm} \times 51 \text{ mm} \times 3 \text{ mm}$

Tributary Area for mullion (Width $\times$ Height) = $1016 \text{ mm} \times 2032 \text{ mm}$

Tributary Area for transom (Width $\times$ Height) = $2032 \text{ mm} \times 1016 \text{ mm}$

Yield strain: 0.004

Ultimate strain: 0.1

Load-mass factor: 0.7

✚ Triangular Pressure Impulse details:

Peak positive pressure: 50

Positive phase duration: 20

✚ CW-Blast Analysis Integration Options:

Convergence tolerance: 0.001 Pa

Maximum iterations: 100

Damping: 0.04

Analysis duration: 1000 ms

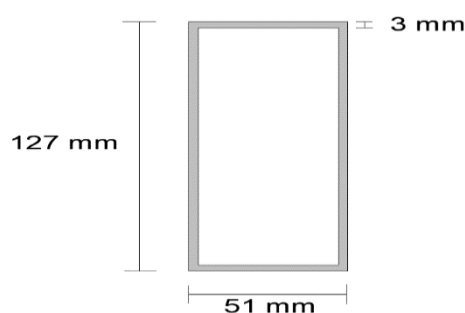
Time setup: 0.25

✚ Network-beta method are deemed as follows:

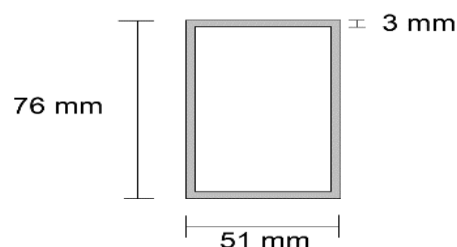
Integration method: Linear acceleration:

$Beta (\beta)$: 0.25

$Gamma (\gamma)$: 0.50



b) Aluminum mullion



c) Aluminum transom

Figure D.1: First curtain wall cross-section details

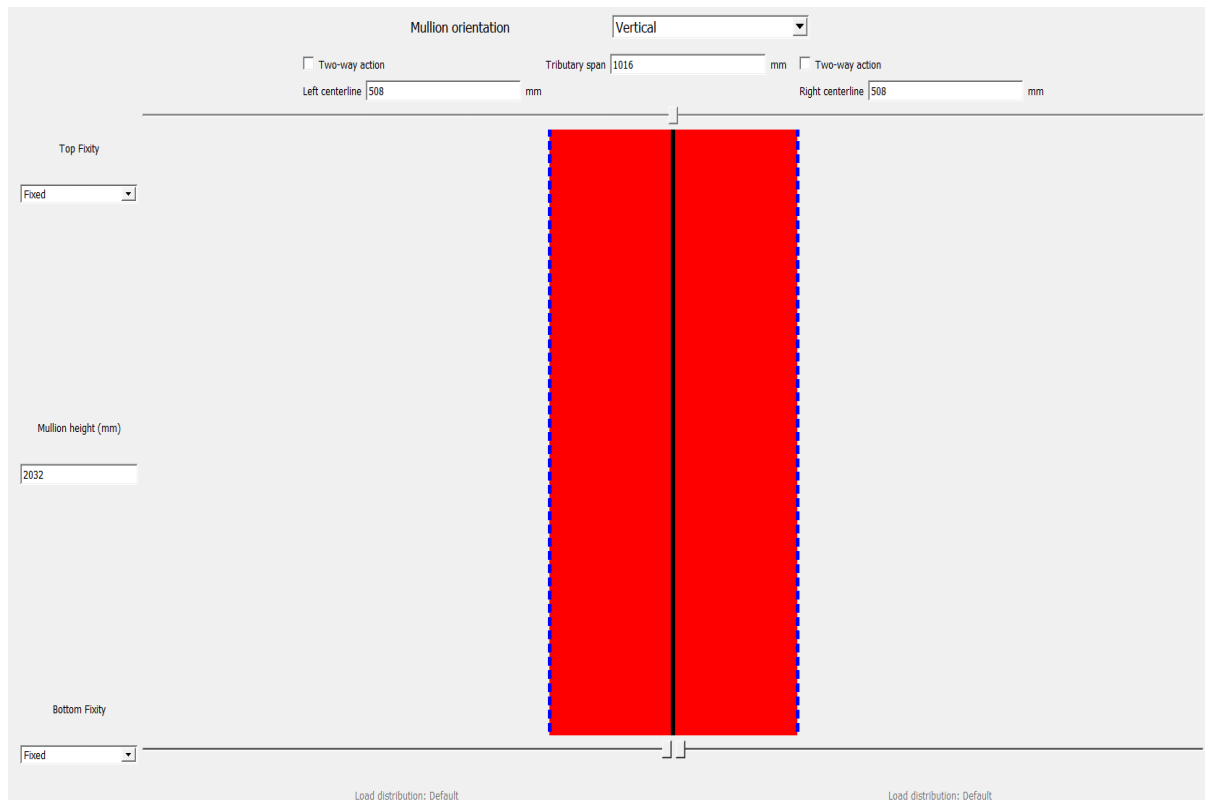


Figure D.2: CW1 mullion orientation and tributary area (fixed)

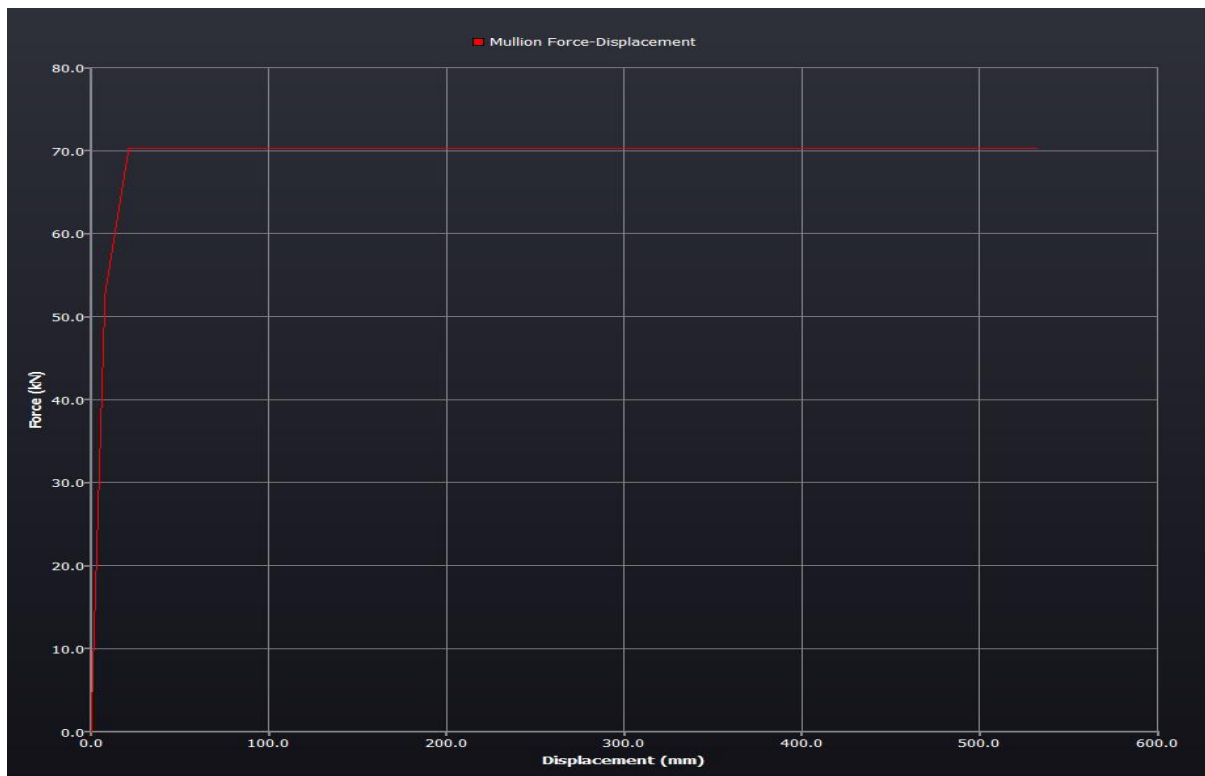


Figure D.3: Mullion Force-displacement graph

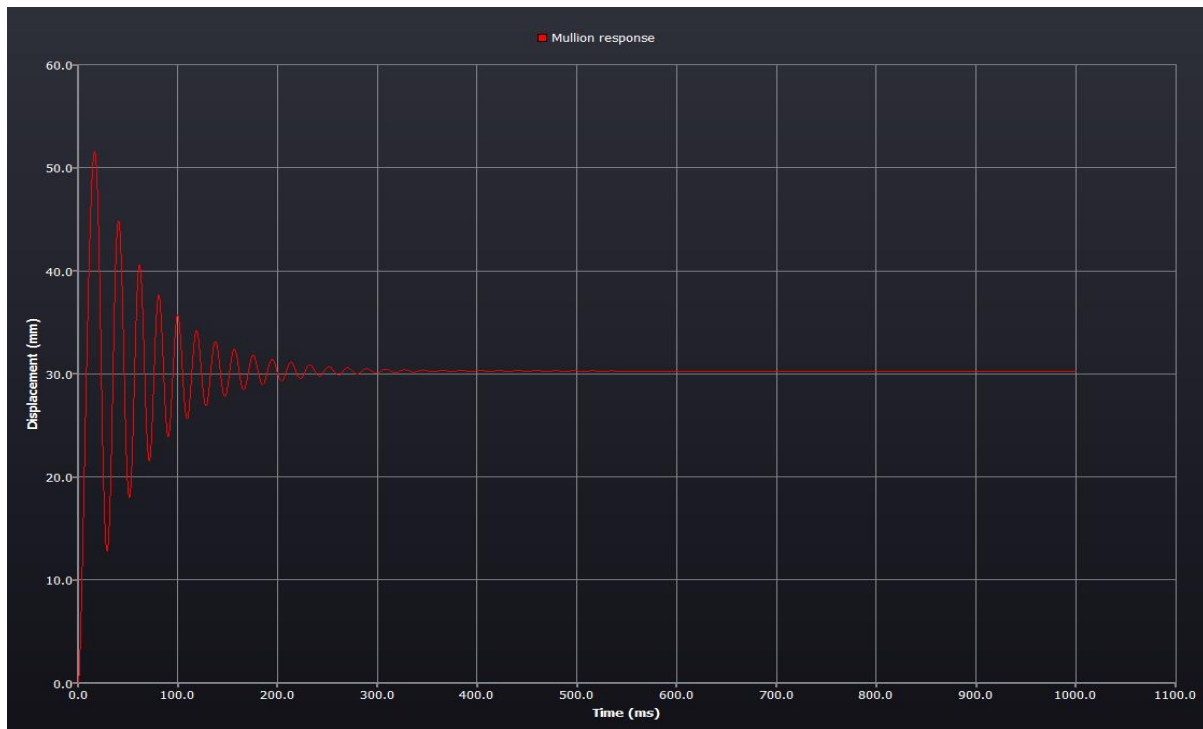


Figure D.4: Mullion response; Displacement-time history

According to the CW-Blast analysis for one single mullion in CW1 considering fixed support, the below outcomes are presented:

- a) Mullion has yielded
- b) Maximum displacement: 52.9842 mm at 16.75 ms
- c) Maximum velocity: 5516.03 m/s at 0.02325 ms
- d) Maximum acceleration: 1.69823×10^6 m/s² at 0.00025 ms
- e) Maximum resisting force: 70.3237 kN due to a displacement of 21.4442 mm
- f) Strain (0.0099) exceeded yield strain (0.004) but remained below rupture strain (0.1)

Table D.1: Dynamic support reaction of mullion (Fixed support)

	Elastic range	Plastic range
Dynamic support reaction (fixed)	39.64 _{kN}	39.27 _{kN}

D.1.2 Resistant Function for Middle Transom (pinned supports)

Stiffness coefficient for support releases at the end of the transom section:

$$K = \frac{F}{D} = \frac{39.64}{52} = 0.76 \frac{kN}{mm}$$

Mullion orientation: Horizontal

Left Release: 0.76 kN/mm, Mullion width (mm): 2032, Right Release: 0.76 kN/mm

Left Fixity: Pinned

Right Fixity: Pinned

Two-way action: ☐ Top centerline (mm): 508

Tributary height (mm): 1016

Two-way action: ☐ Bottom centerline (mm): 508

Load distribution: Default

Figure D.5: CW1 middle transom orientation and tributary area (pinned)

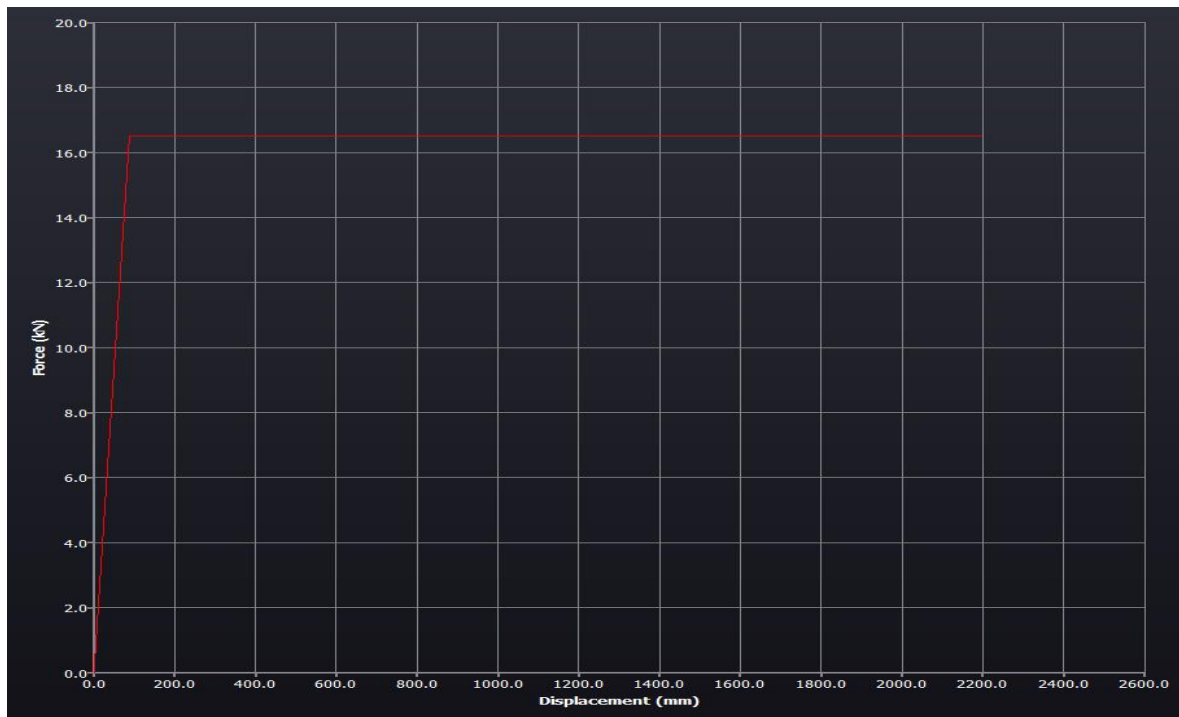


Figure D.6: Transom Force-displacement graph

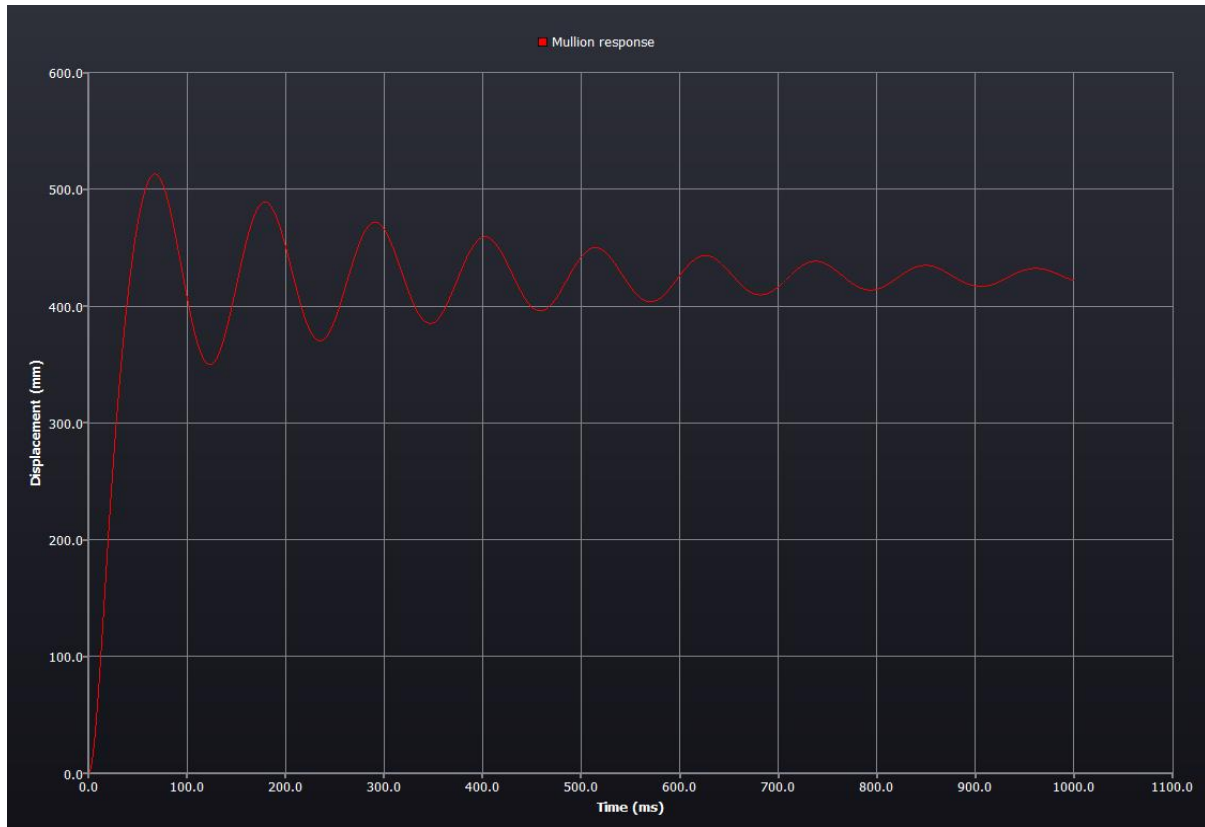


Figure D.7: Transom response; Displacement-time history

According to the CW-Blast analysis for the middle transom in CW1 considering pinned support, the below outcomes are presented:

- a) Transom has yielded
- b) Maximum displacement: 518.778 mm at 67.75 ms
- c) Maximum velocity: 13775.7 m/s at 0.01675 ms
- d) Maximum acceleration: 1.72148×10^6 m/s² at 0.00025 ms
- e) Maximum resisting force: 16.5079 kN due to a displacement of 88.9979 mm
- f) Strain (0.0236) exceeded yield strain (0.0233) but remained below rupture strain (0.1)

Table D.2: Dynamic support reaction of middle transom

	Elastic range	Plastic range
Dynamic support reaction (fixed)	17.49 _{kN}	19.08 _{kN}

D.2 Force Calculation of Second Curtain Wall (CW2)

D.2.1 Resistant Function for Mullion (fixed supports)

Due to the fact that mullions were hardened with two L-shape steel plates, the calculation of transformed M.I for the composite section should be also done. The Tributary area details and CW-Blast software inputs are given as follows:

Yield Strength of Aluminum (f_y) = 275 MPa

Modulus of Elasticity of aluminum (E_A) = 70,000 N/mm²

Modulus of Elasticity of steel (E_s) = 200000 N/mm²

Mass Density of Aluminum = 2700 kg/m³

Mullion Section Size: 127 mm × 51 mm × 3 mm

Transom Section Size: 76 mm × 51 mm × 3 mm

Tributary Area for mullion (Width × Height) = 1016 mm × 2032 mm

Tributary Area for transom (Width × Height) = 2032 mm × 1016 mm

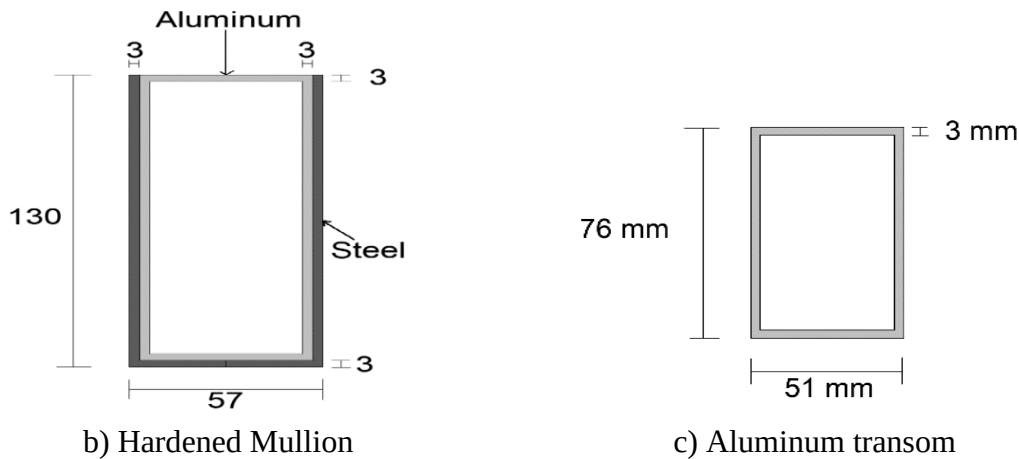


Figure D.8: Second curtain wall cross-section (mm)

✚ Calculation of transformed M.I for the composite mullion section (steel to aluminum):

Transformation factor to convert steel into equivalent aluminum section: $n = \frac{E_s}{E_A} = 2.85$

Area of aluminum mullion = 1,032 mm²

Area of each L-shape steel retrofit plate on the mullion = $(130 \times 3) + (28 \times 3) = 462 \text{ mm}^2$

Transformed area of steel retrofit plate to aluminum section = $2.85 \times 462 = 1,316 \text{ mm}^2$

Total area of mullion section and two retrofit plates = $1,032 + (2 \times 1,316) = 3,664 \text{ mm}^2$

Moment of Inertia per mullion = $2,062,274 \text{ mm}^4$

Moment of inertia for each L-shape steel retrofit plate on the mullion = $549,304 \text{ mm}^4$

Transformed M.I of steel plates to aluminum section = $2.85 \times 549,304 = 1,565,516 \text{ mm}^4$

Distance of the neutral axis from the extreme bottom (tension) fibers,

$$Y = 60.1 \text{ mm}$$

Moment of inertia for the transformed section = mullion + 2 retrofitting plate

$$I_{TR} = [2,062,274 + (1032 \times 6.4^2)] + 2 [1,565,516 + (1316 \times 58.6^2)] = 14,273,760 \text{ mm}^4$$

Yield strain: 0.004

Ultimate strain: 0.1

Load-mass factor: 0.7

 Triangular Pressure Impulse details:

Peak positive pressure: 36

Positive phase duration: 20

 CW-Blast Analysis Integration Options:

Convergence tolerance: 0.001 Pa

Analysis duration: 1000 ms

Time setup: 0.25

Maximum iterations: 100

Damping: 0.04

 Network-beta method are deemed as follows:

Integration method: Linear acceleration

Beta (β): 0.25

Gamma (γ): 0.50

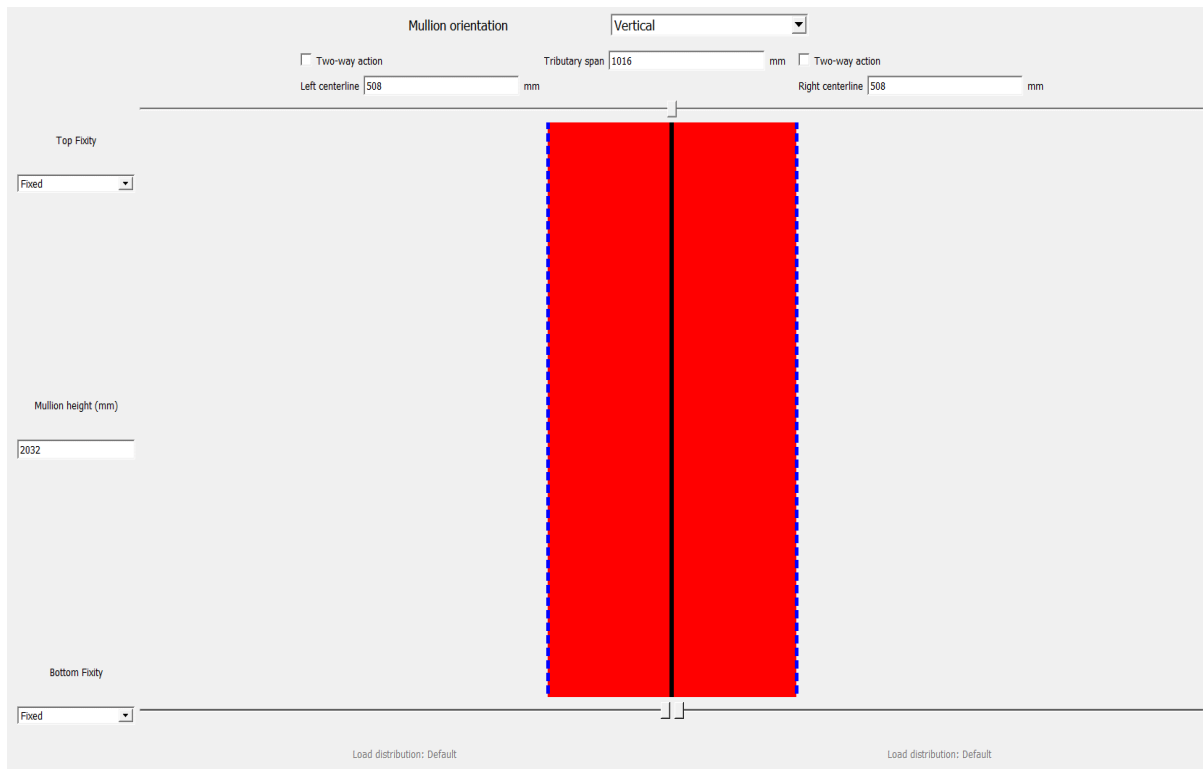


Figure D.9: CW2 mullion orientation and tributary area (fixed)

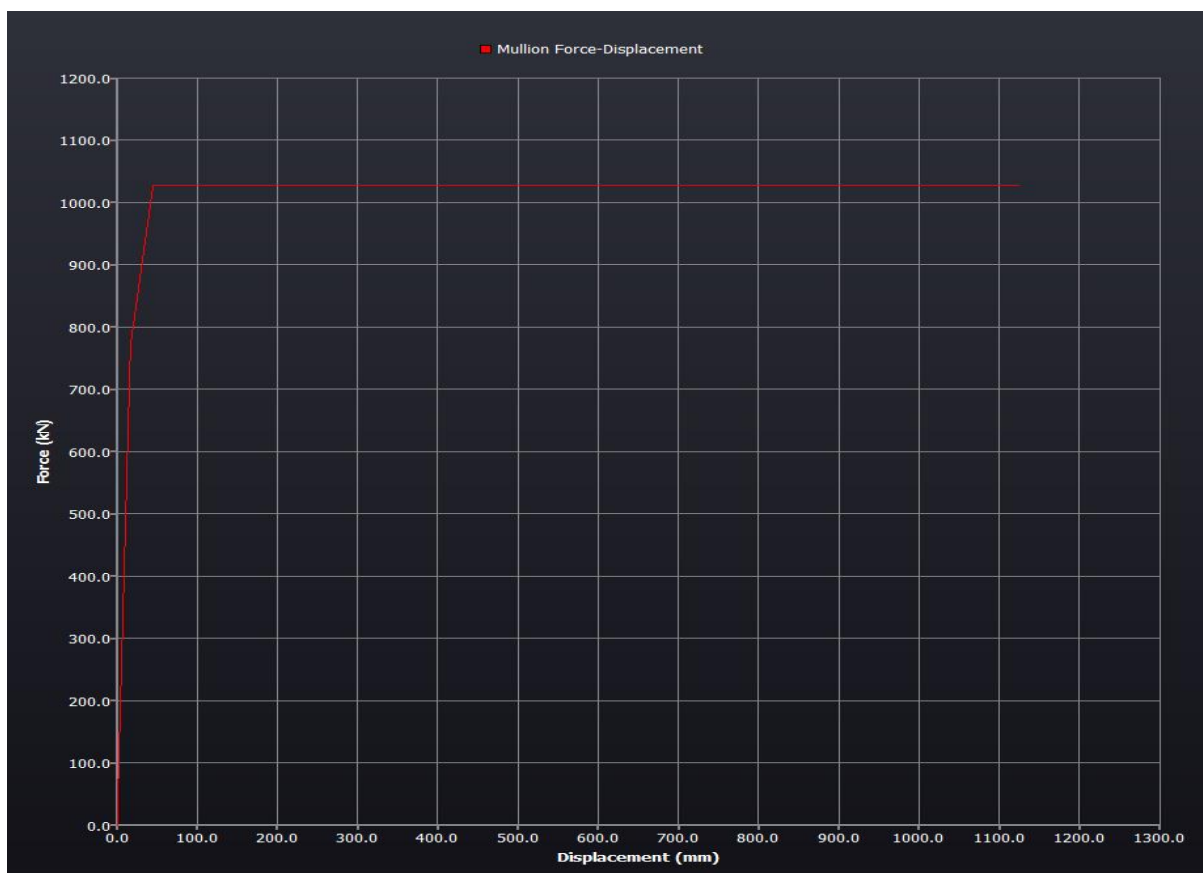


Figure D.10: Mullion Force-displacement graph

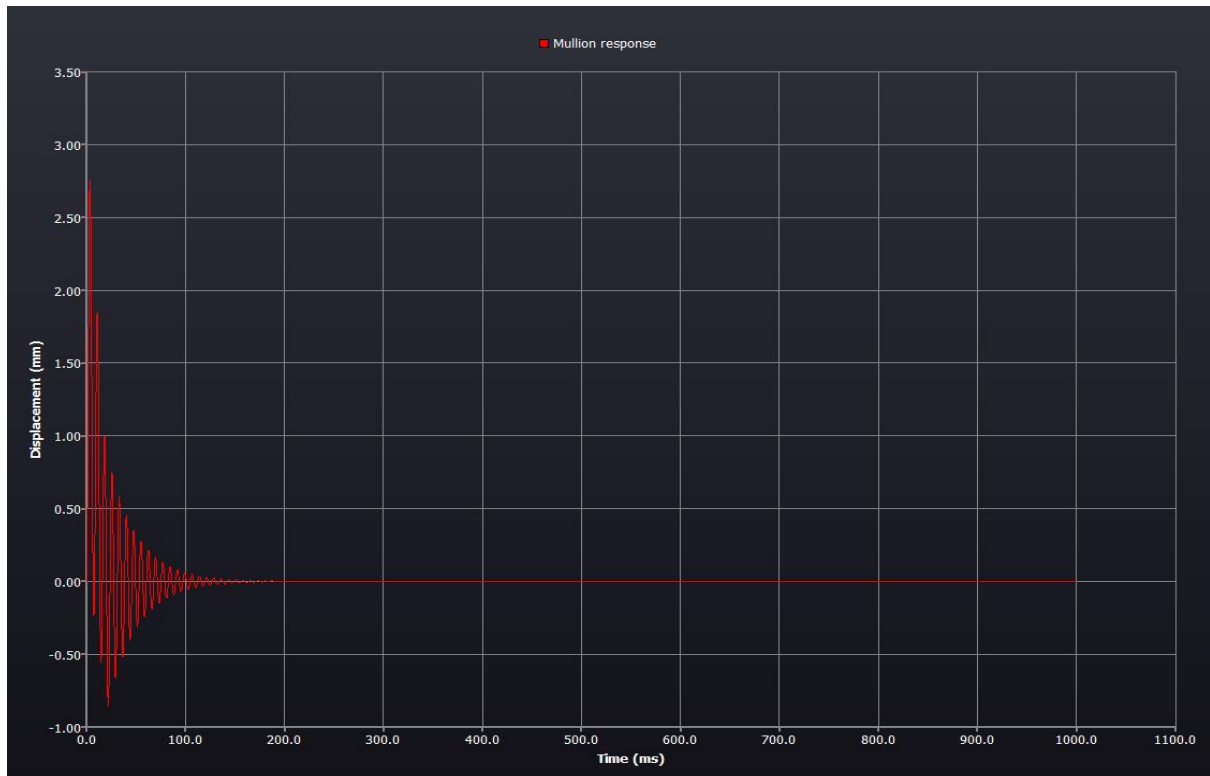


Figure D.11: Mullion response, Displacement-time history

According to the CW-Blast analysis for CW2 hardened mullion, the below outcomes are presented:

- a) Mullion remained elastic
- b) Maximum displacement: 2.75884 mm at 3.75 ms
- c) Maximum velocity: 1205.71 m/s at 0.00575 ms
- d) Maximum acceleration: 1.11622×10^6 m/s² at 0.00025 ms
- e) Maximum resisting force: 126.16 kN due to a displacement of 2.75884 mm
- f) Strain (0.000245318) less than yield strain (0.004)

Table D.3: Dynamic support reaction for mullion

	Elastic range	Plastic range
Dynamic support reaction (fixed)	56.02 <i>kN</i>	—

D.2.2 Resistant Function for Middle Transom (pinned supports)

Stiffness coefficient for support releases at the end of the transom section:

$$K = \frac{F}{D} = \frac{56.02}{2.76} = 20.3 \frac{kN}{mm}$$

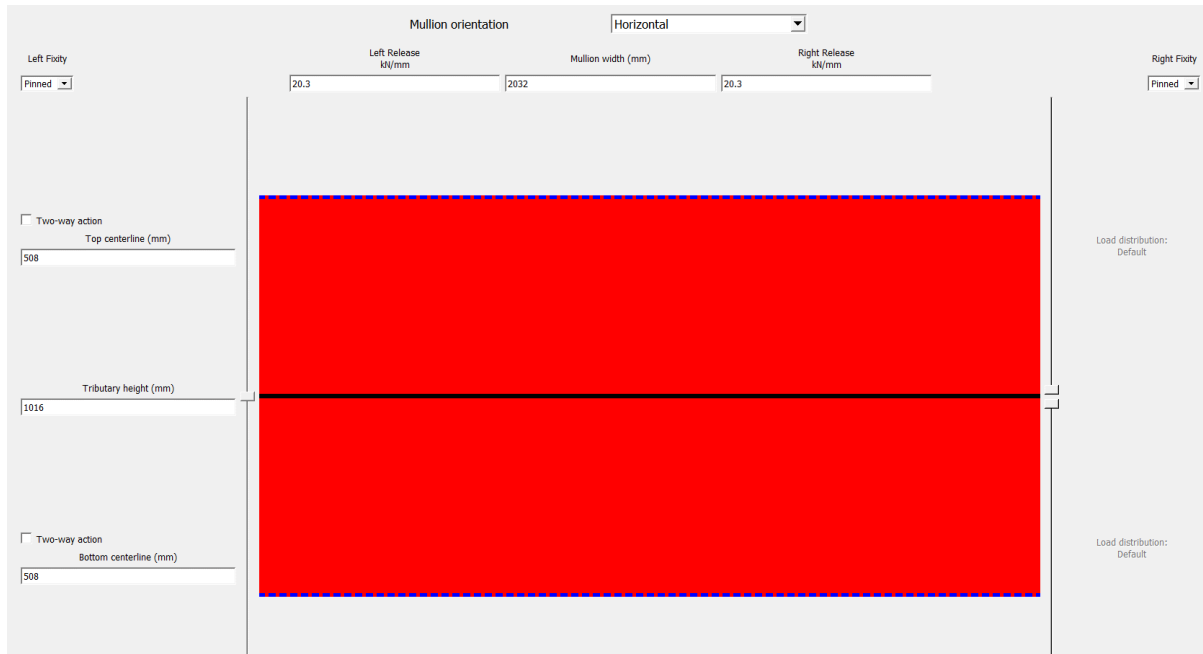


Figure D.12: CW2 middle transom orientation and tributary area (pinned)

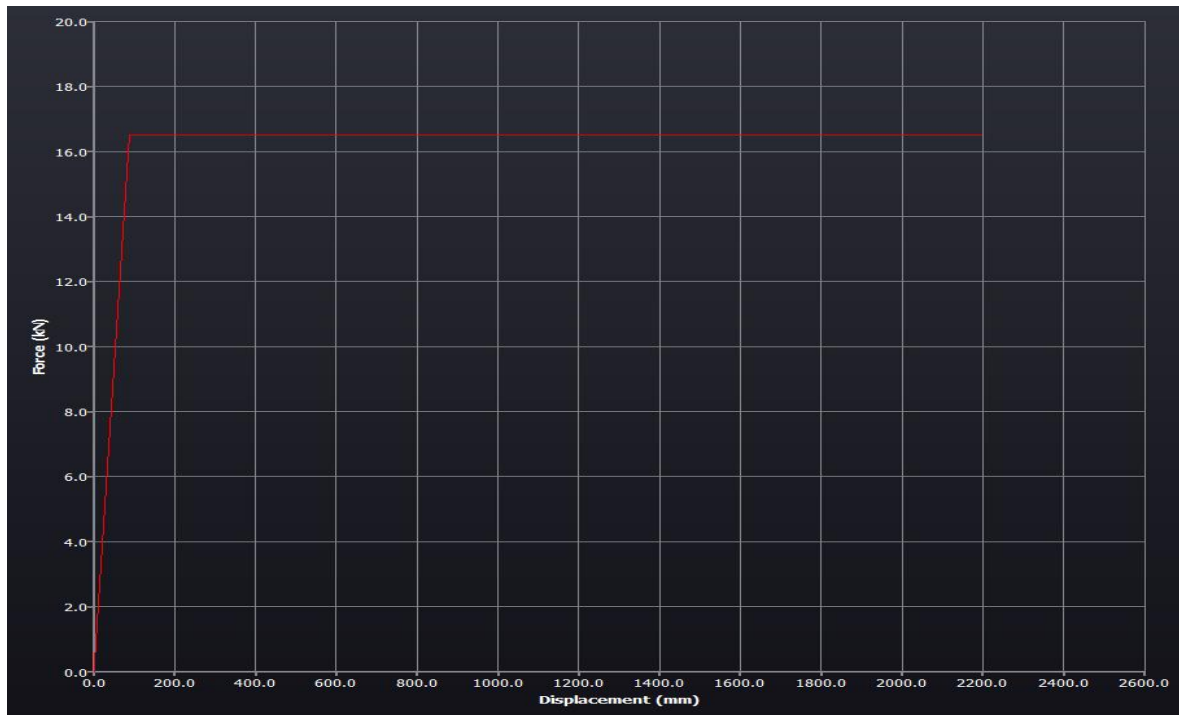


Figure D.13: Transom Force-displacement graph

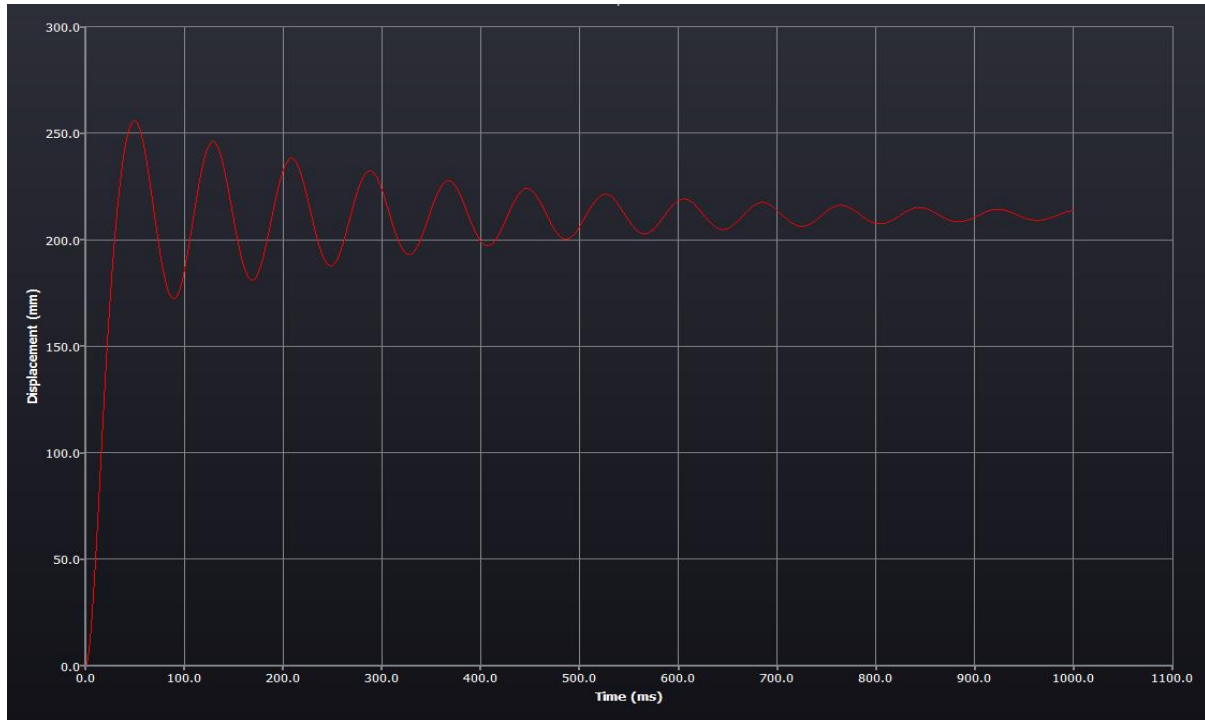


Figure D.14: Transom response; Displacement-time history

According to the CW-Blast analysis for the middle transom in CW2 considering pinned support, the below outcomes are presented:

- a) Transom has yielded
- b) Maximum displacement: 257.089 mm at 49.75 ms
- c) Maximum velocity: 8919.66 m/s at 0.0155 ms
- d) Maximum acceleration: 1.23916×10^6 m/s² at 0.00025 ms
- e) Maximum resisting force: 16.5079 kN due to a displacement of 47.1195 mm
- f) Strain (0.0223) exceeded yield strain (0.0233) but remained below rupture strain (0.1)

Table D.4: Dynamic support reaction for middle transom

	Elastic range	Plastic range
Dynamic support reaction (fixed)	14.41 <i>kN</i>	15.47 <i>kN</i>

D.3 Force Calculation of Third Curtain Wall (CW3)

D.3.1 Resistant Function for Mullion (fixed supports)

Due to the fact that mullions and transoms were hardened with two L-shape steel plates, the calculation of transformed M.I for the composite section should be also done. The Tributary area details and CW-Blast inputs are given as follows:

Yield Strength of Aluminum (f_y) = 275 MPa

Modulus of Elasticity of aluminum (E_A) = 70,000 N/mm²

Modulus of Elasticity of steel (E_s) = 200000 N/mm²

Mass Density of Aluminum = 2700 kg/m³

Mullion Section Size: 127 mm × 51 mm × 3 mm

Transom Section Size: 76 mm × 51 mm × 3 mm

Tributary Area for mullion (Width × Height) = 1016 mm × 2032 mm

Tributary Area for transom (Width × Height) = 2032 mm × 1016 mm

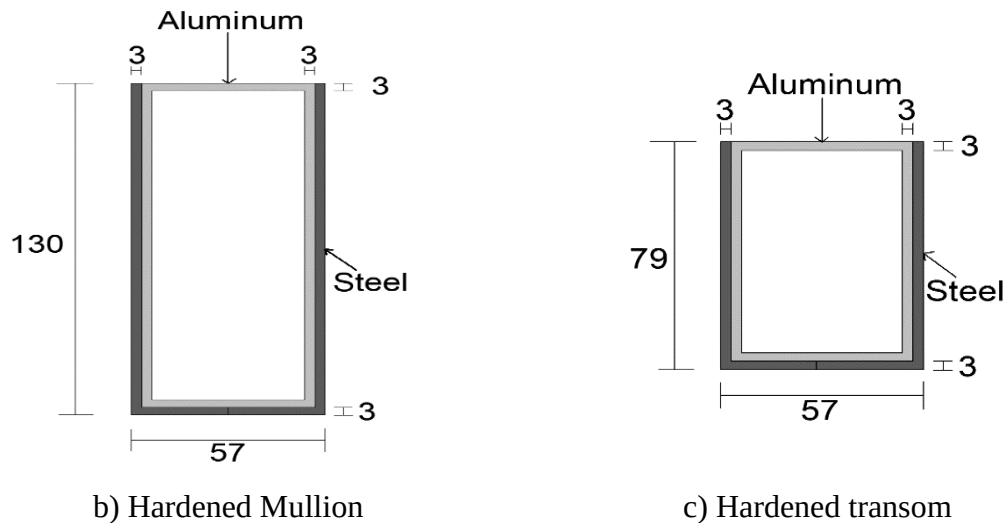


Figure D.15: Third curtain wall Cross-section (mm)

✚ Calculation of transformed M.I for the composite mullion section (steel to aluminum):

Transformation factor to convert steel into equivalent aluminum section: $n = \frac{E_s}{E_A} = 2.85$

Area of aluminum mullion = 1,032 mm²

Area of each L-shape steel retrofit plate on the mullion = $(130 \times 3) + (25.5 \times 3) = 467 \text{ mm}^2$

Transformed area of steel retrofit plate to aluminum section = $2.85 \times 467 = 1,331 \text{ mm}^2$

Total area of mullion section and two retrofit plates = $1,032 + (2 \times 1,331) = 3,694 \text{ mm}^2$

Moment of Inertia per mullion = $2,062,274 \text{ mm}^4$

Moment of inertia for each L-shape steel retrofit plate on the mullion = $549,304 \text{ mm}^4$

Transformed M.I of steel plates to aluminum section = $2.85 \times 549,304 = 1,565,516 \text{ mm}^4$

Distance of the neutral axis from the extreme bottom (tension) fibers:

$$Y = 60.1 \text{ mm}$$

Moment of inertia for the transformed section = mullion + 2 retrofitting plate

$$I_{TR} = [2,062,274 + (1032 \times 6.4^2)] + 2 [1,565,516 + (1316 \times 58.6^2)] = 14,273,760 \text{ mm}^4$$

✚ Calculation of transformed M.I for the composite transom section (steel to aluminum):

Transformation factor to convert steel into equivalent aluminum section: $n = \frac{E_S}{E_A} = 2.85$

Area of aluminum transom = 681 mm^2

Area of each L-shape steel retrofit plate on the transom = $(79 \times 3) + (25.5 \times 3) = 314 \text{ mm}^2$

Transformed area of steel retrofit plate to aluminum section = $2.85 \times 314 = 895 \text{ mm}^2$

Total area of transom section and two retrofit plates = $681 + (2 \times 895) = 2,471 \text{ mm}^2$

Moment of Inertia per transom: $579,398 \text{ mm}^4$

Moment of inertia of each L-shape steel retrofit plate on the transom = $203,056 \text{ mm}^4$

Transformed M.I of steel to aluminum section = $2.85 \times 203,056 = 578,710 \text{ mm}^4$

Distance of the neutral axis from the extreme bottom (tension) fibers:

$$Y = 35 \text{ mm}$$

Moment of inertia for the transformed section = Transom + 2 L-shape retrofit plates

$$I_{TR} = [579,398 + (681 \times 6^2)] + 2 [578,710 + (314 \times 4.5^2)] = 1,774,051 \text{ mm}^4$$

Yield strain: 0.004

Ultimate strain: 0.1

Load-mass factor: 0.7

 Triangular Pressure Impulse details:

Peak positive pressure: 46

Positive phase duration: 21

 CW-Blast Analysis Integration Options:

Convergence tolerance: 0.001 Pa

Analysis duration: 1000 ms

Time setup: 0.25

Maximum iterations: 100

Damping: 0.04

 Network-beta method are deemed as follows:

Integration method: Linear acceleration

Beta (β): 0.25

Gamma (γ): 0.50

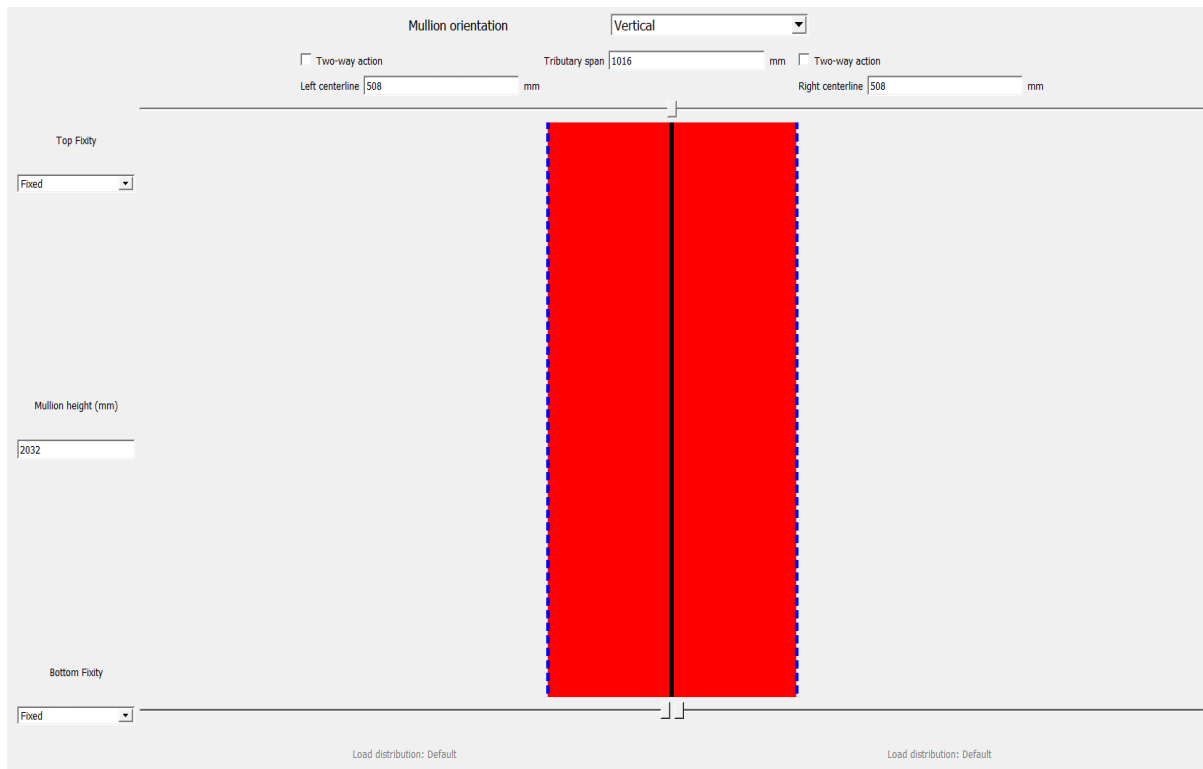


Figure D.16: CW3 mullion orientation and tributary area (fixed)

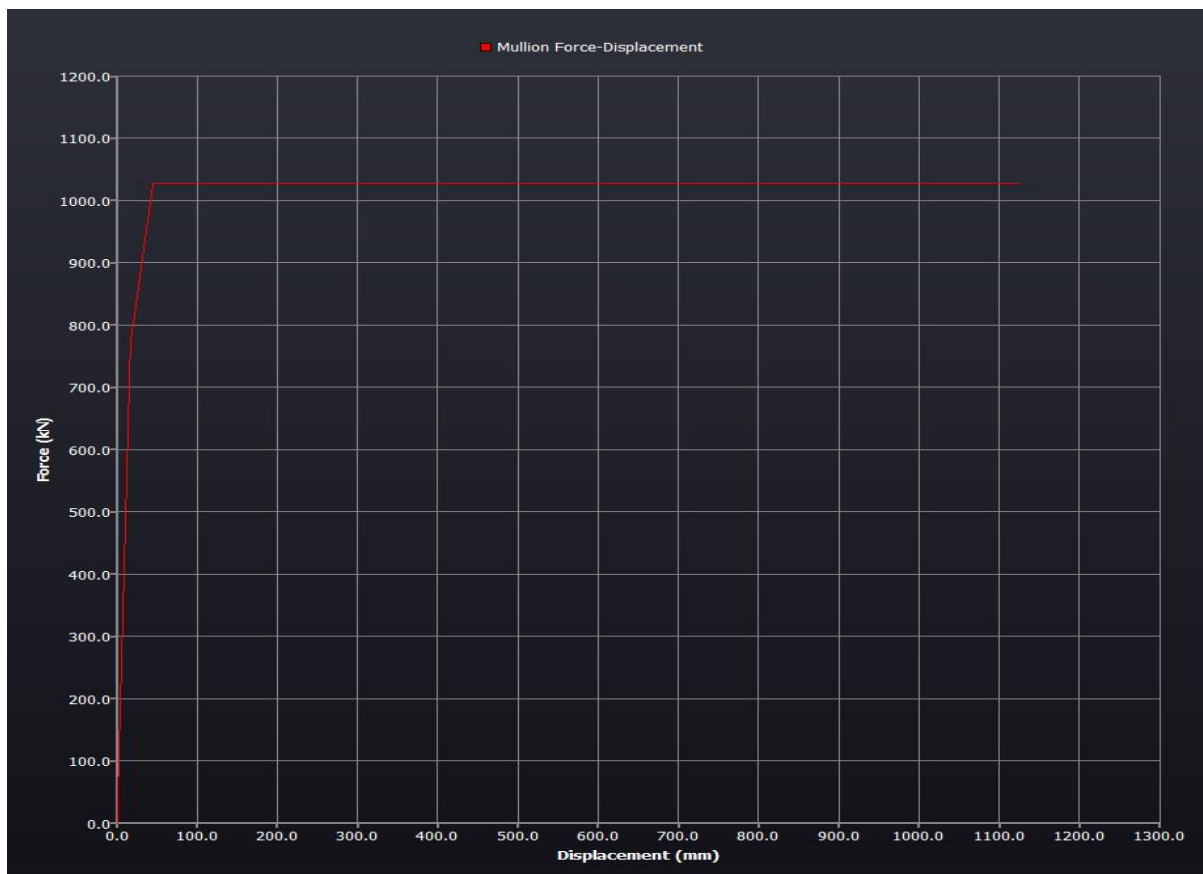


Figure D.17: Mullion Force-displacement graph

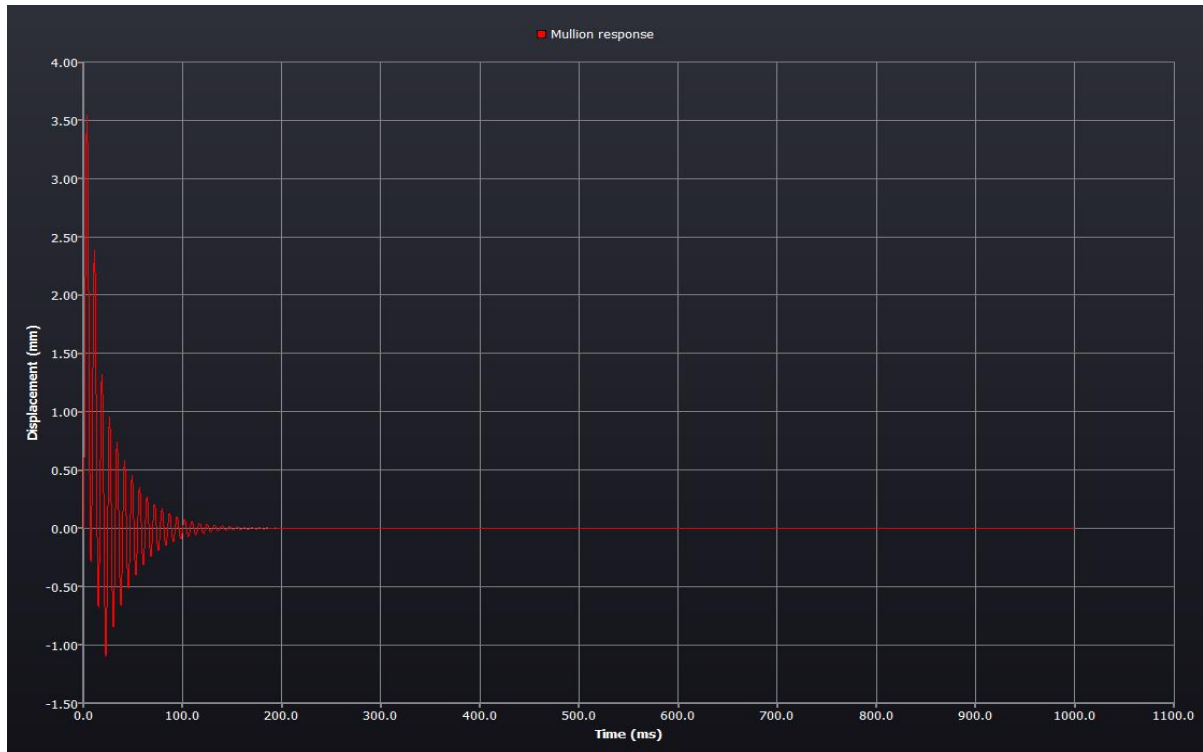


Figure D.18: Mullion response, Displacement-time history

According to the CW-Blast analysis for CW3 hardened mullion, the below outcomes are presented:

- g) Mullion remained elastic
- h) Maximum displacement: 3.54034 mm at 3.75 ms
- i) Maximum velocity: 1528.37 m/s at 0.00575 ms
- j) Maximum acceleration: 1.4032e+06 m/s² at 0.00025 ms
- k) Maximum resisting force: 161.898 kN due to a displacement of 3.54034 mm
- l) Strain (0.00031481) less than yield strain (0.004)

Table D.5: Dynamic support reaction for mullion

	Elastic range	Plastic range
Dynamic support reaction (fixed)	71.82 _{kN}	—

D.3.2 Resistant Function for Middle Transom (pinned supports)

Stiffness coefficient for support releases at the end of the transom section:

$$K = \frac{F}{D} = \frac{71.82}{3.54} = 20.3 \frac{kN}{mm}$$

The screenshot shows a software interface for defining a middle transom section. The main area is a large red rectangle representing the transom. The interface includes several input fields and controls:

- Mullion orientation:** A dropdown menu set to "Horizontal".
- Left Fixity:** A dropdown menu set to "Pinned".
- Right Fixity:** A dropdown menu set to "Pinned".
- Left Release kN/mm:** A text input field with the value "20.3".
- Mullion width (mm):** A text input field with the value "2032".
- Right Release kN/mm:** A text input field with the value "20.3".
- Two-way action:** A checkbox that is unchecked.
- Top centerline (mm):** A text input field with the value "508".
- Tributary height (mm):** A text input field with the value "1016".
- Bottom centerline (mm):** A text input field with the value "508".
- Load distribution:** Two labels, "Default", are positioned on the right side of the interface.

Figure D.19: CW3 middle transom orientation and tributary area (pinned)

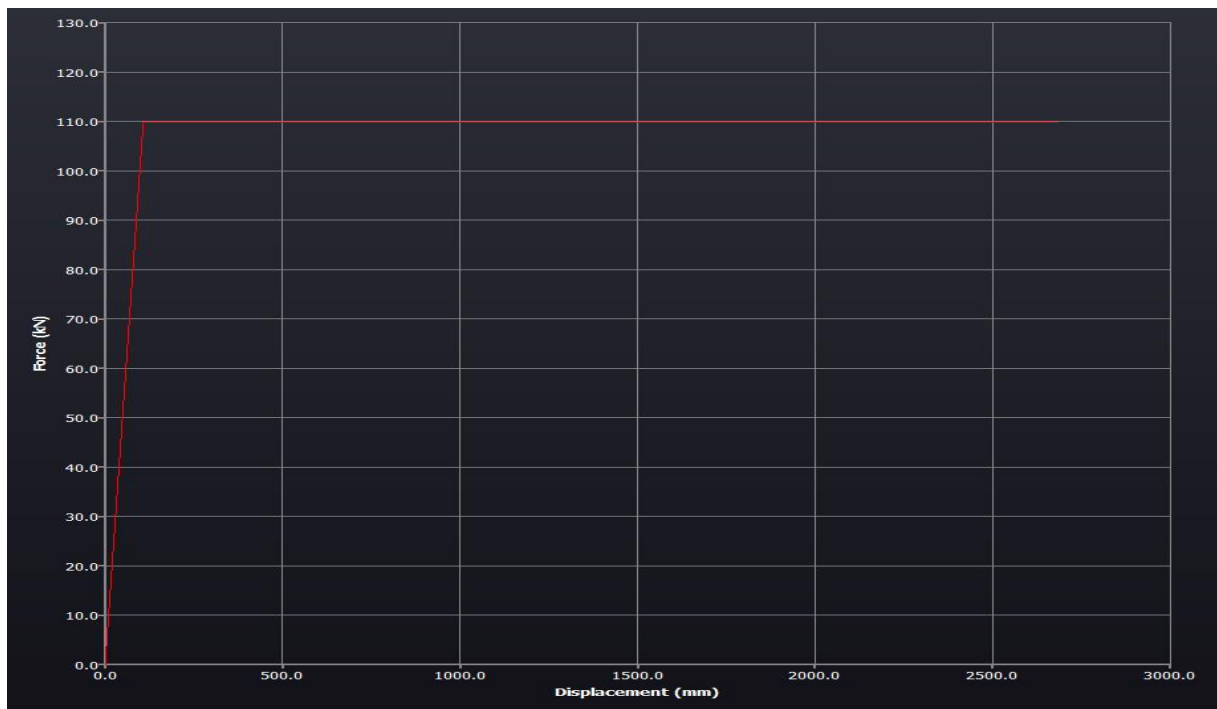


Figure D.20: Transom Force-displacement graph

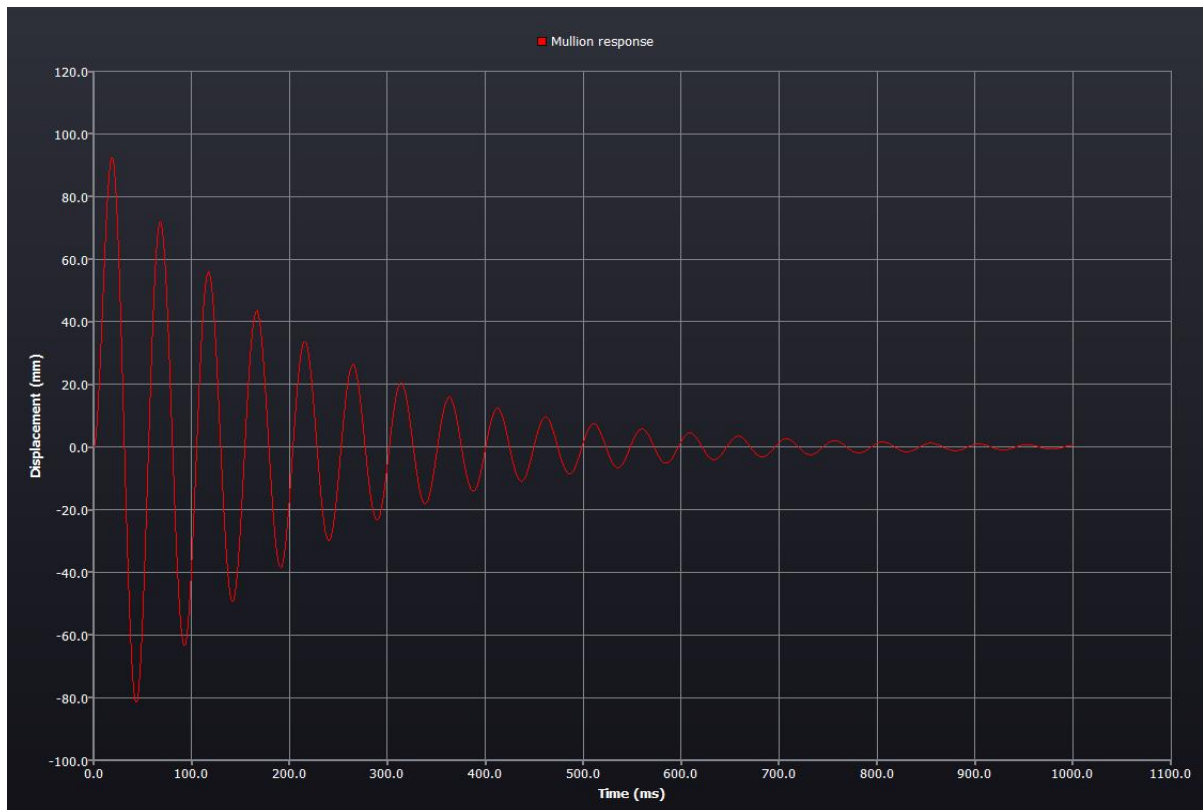


Figure D.21: Transom response; Displacement-time history

According to the CW-Blast analysis for the hardened middle transom in CW3 considering pinned support, the below outcomes are presented:

- g) Transom remained elastic.
- h) Maximum displacement: 92.5856 mm at 19 ms
- i) Maximum velocity: 11127.2 m/s at 0.031 ms
- j) Maximum acceleration: 1.46718×10^6 m/s² at 0.00025 ms
- k) Maximum resisting force: 94.6447 kN due to a displacement of 92.5856 mm
- l) Strain (0.00344928) less than yield strain (0.004)

Table D.6: Dynamic support reaction for the middle transom

	Elastic range	Plastic range
Dynamic support reaction (fixed)	47.13 kN	—

Appendix E: Mullion Deflection Comparison and PI Diagrams

E.1 First Mullion Maximum Deflection at Mid-height Obtained from RC-Blast, UFC-charted, and Experiment + Relevant PI Diagram

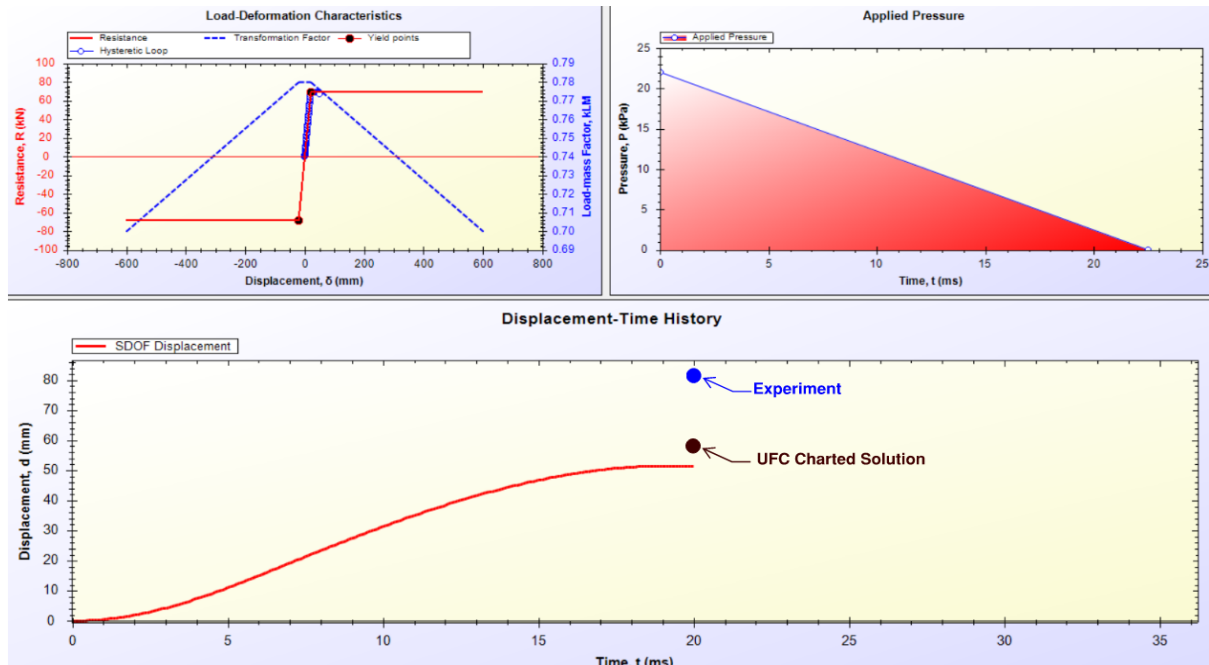


Figure E.1: M-C1 maximum displacement at mid-height

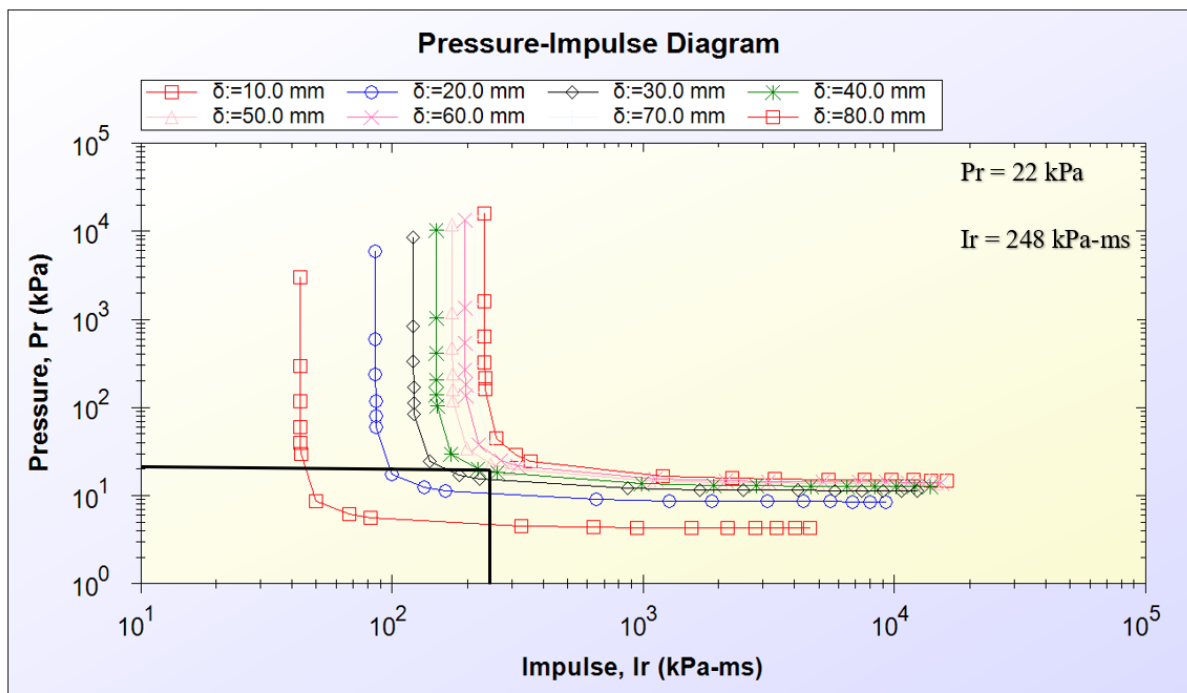


Figure E.2: P-I diagram for the first mullion with fixed supports

E.2 Second Mullion Maximum Deflection at Mid-height Obtained From RC-Blast, UFC-charted, and Experiment + Relevant PI Diagram

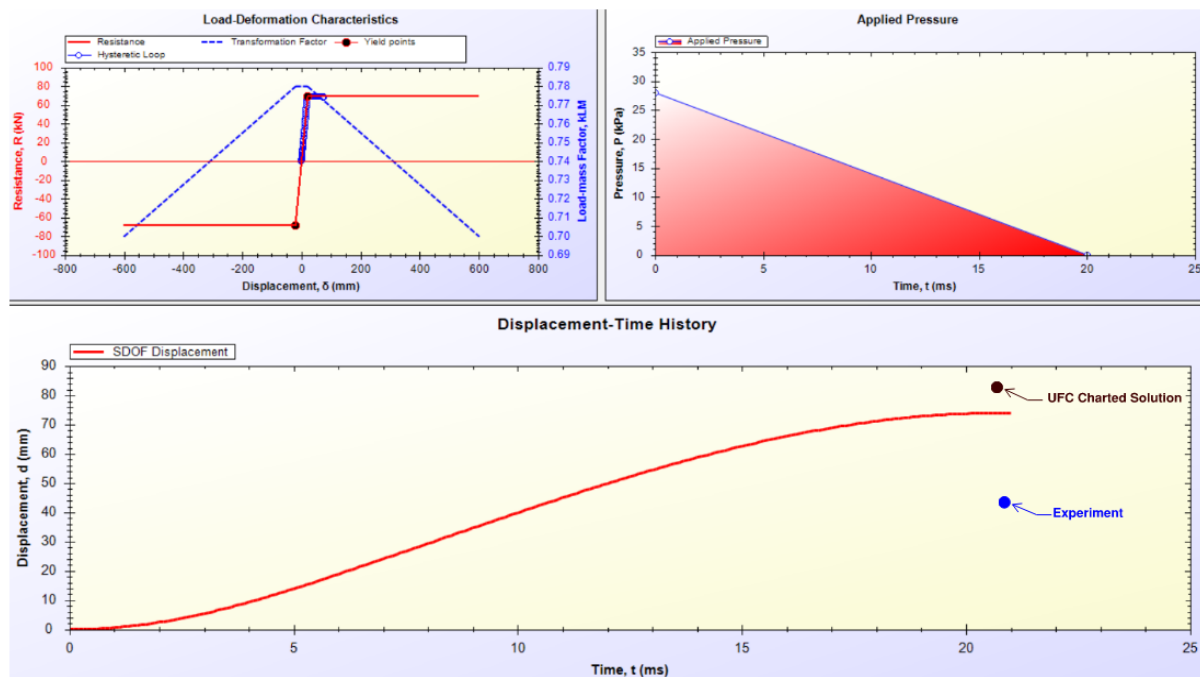


Figure E.3: M-C2 maximum displacement at mid-height

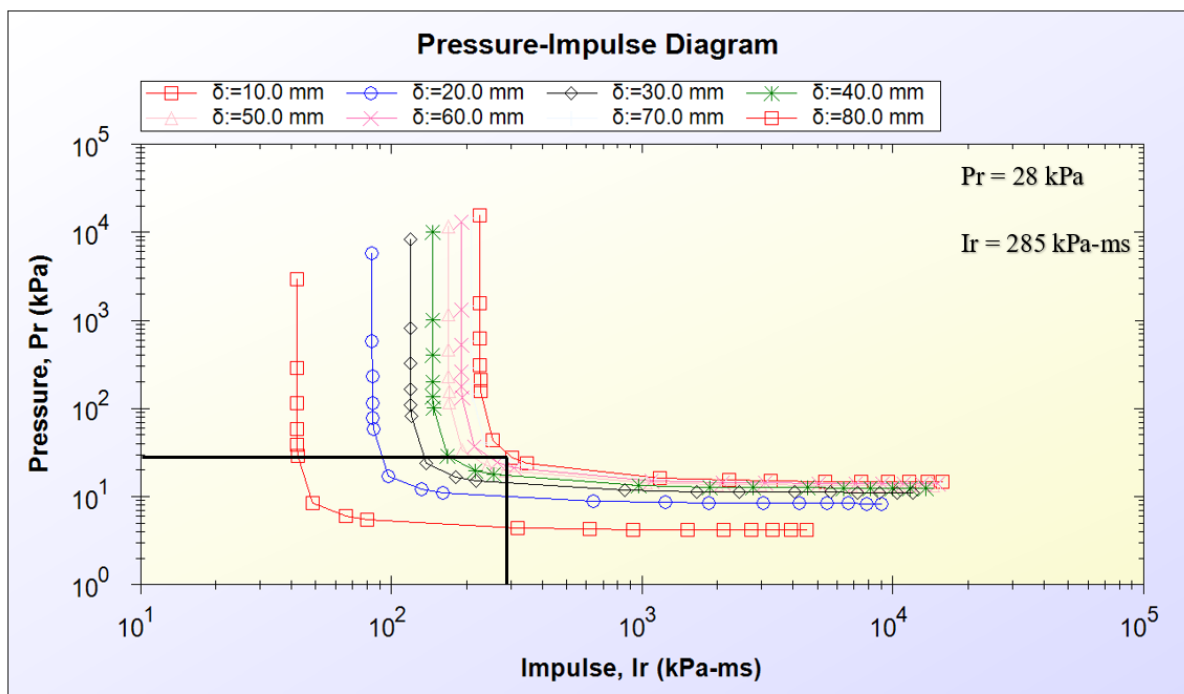


Figure E.4: P-I diagram for the second mullion with fixed supports

E.3 Third Mullion Maximum Deflection at Mid-height Obtained From RC-Blast, UFC-charted, and Experiment + Relevant PI Diagram

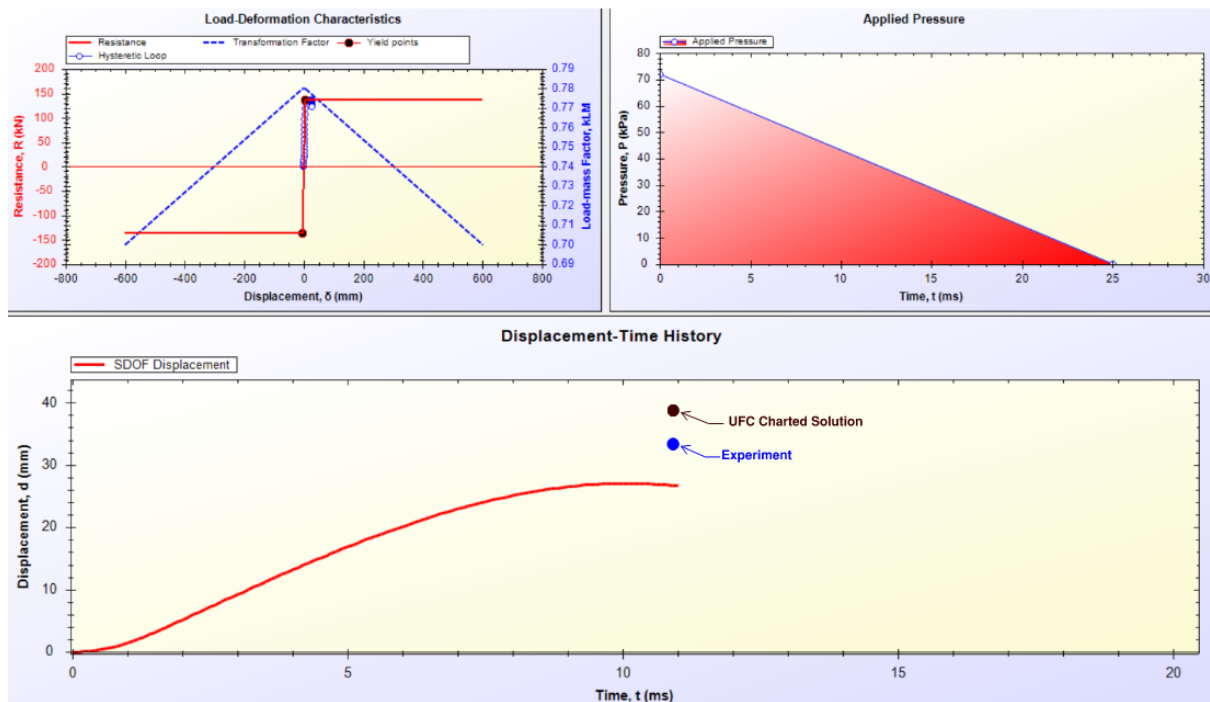


Figure E.5: M-C3 maximum displacement at mid-height

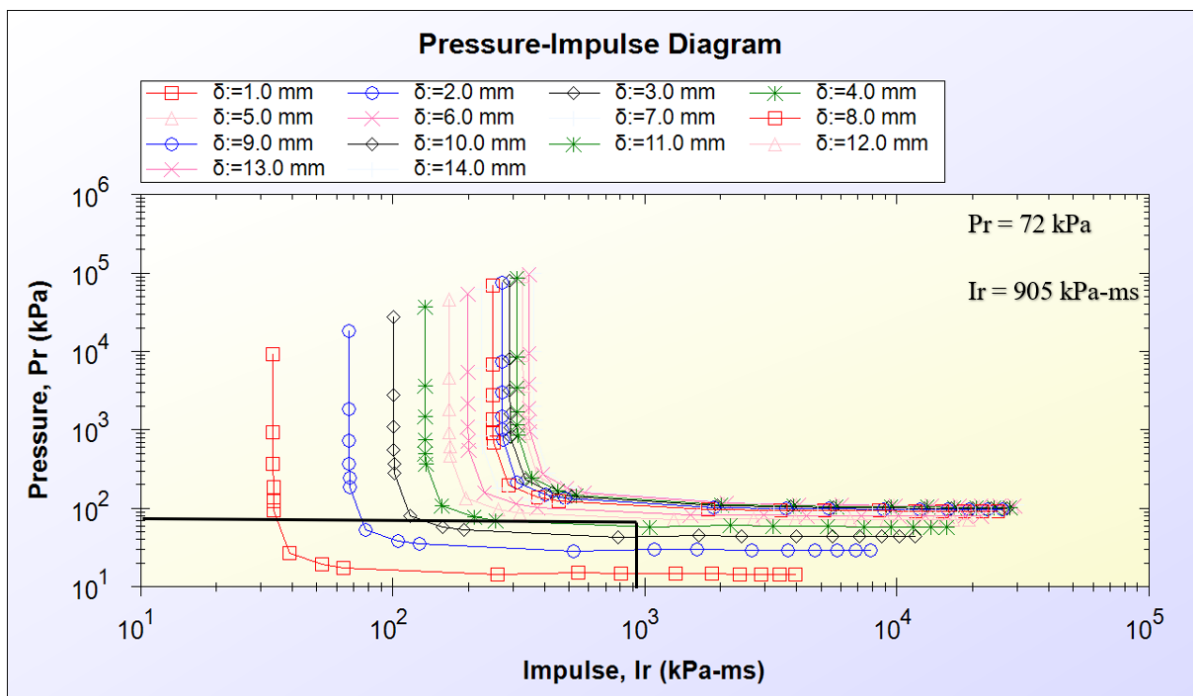


Figure E.6: P-I diagram for the third mullion with fixed supports

E.4 Fourth Mullion Maximum Deflection at Mid-height Obtained From RC-Blast, UFC-charted, and Experiment + Relevant PI Diagram

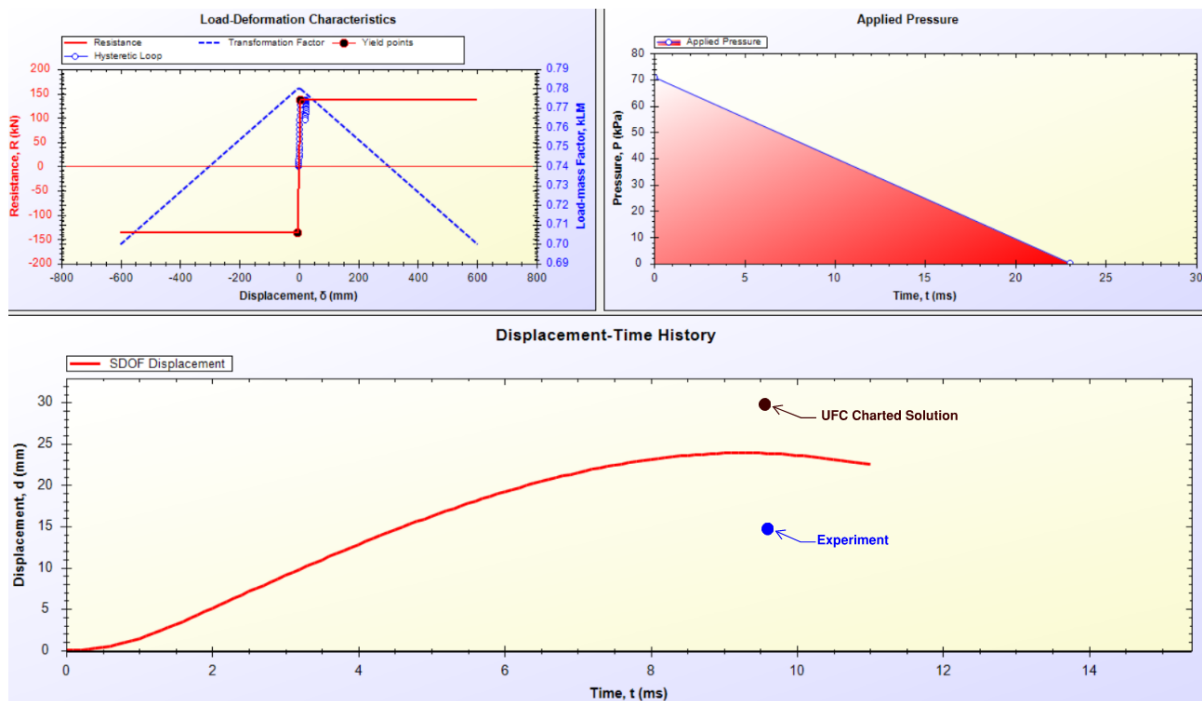


Figure E.7: M-C4 maximum displacement at mid-height

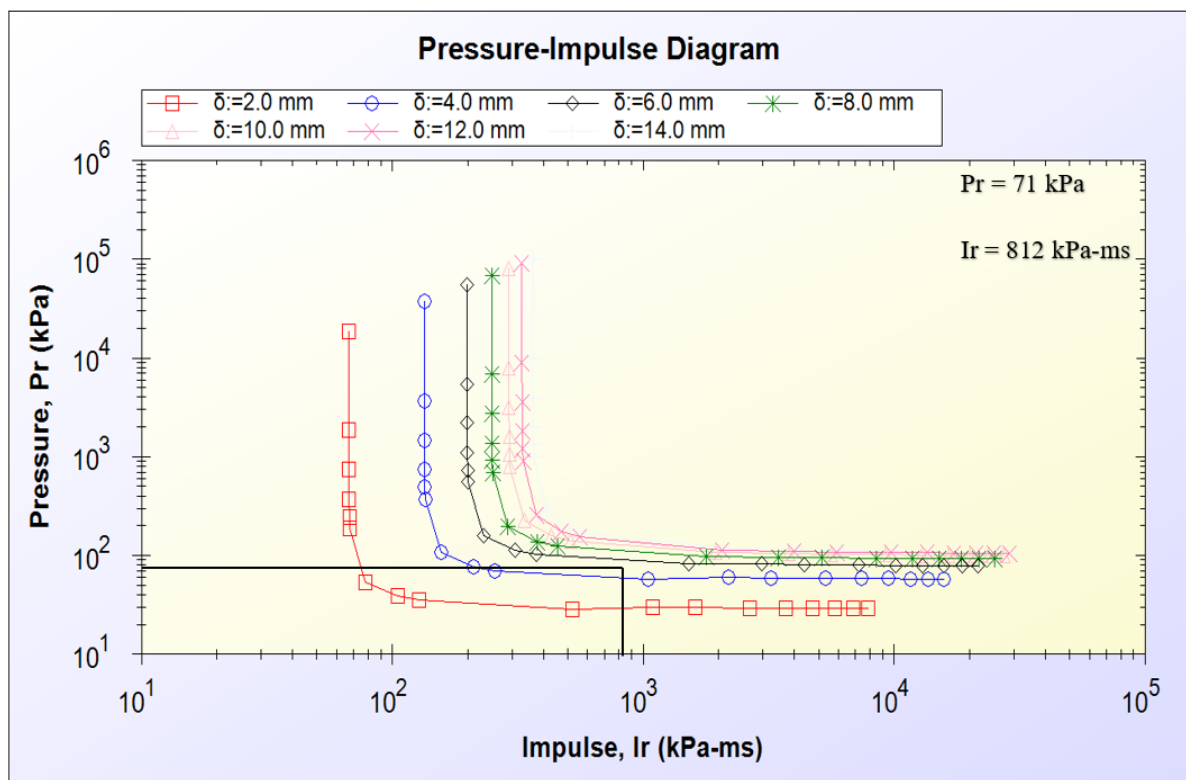


Figure E.8: P-I diagram for the fourth mullion with fixed supports

E.5 Fifth Mullion Maximum Deflection at Mid-height Obtained From RC-Blast, UFC-charted, and Experiment + Relevant PI Diagram

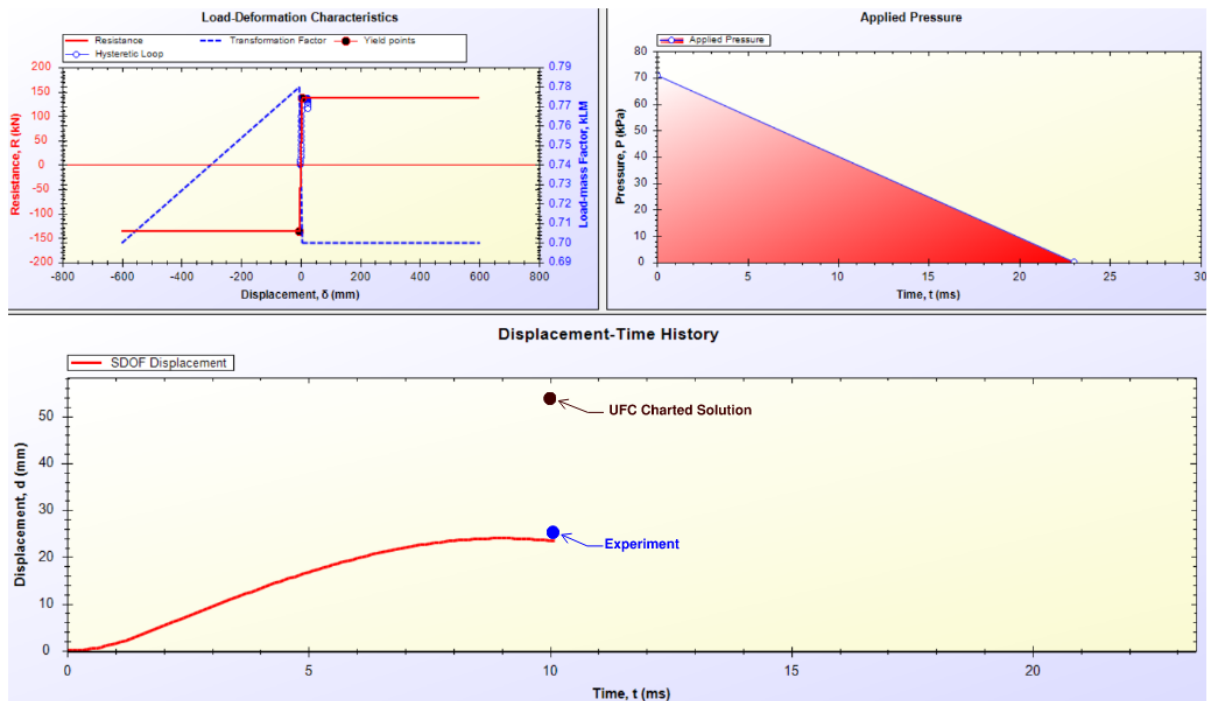


Figure E.9: M-C5 maximum displacement at mid-height

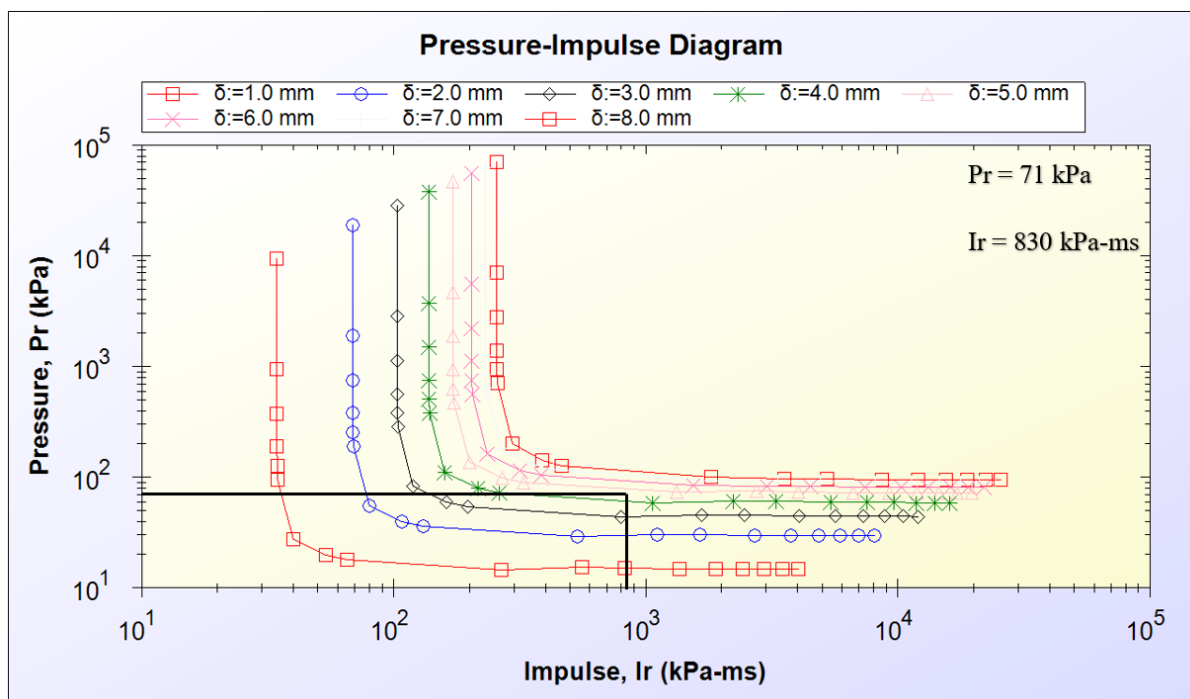


Figure E.10: P-I diagram for the fifth mullion with fixed supports

E.6 Sixth Mullion Maximum Deflection at Mid-height Obtained From RC-Blast, UFC-charted, and Experiment + Relevant PI Diagram

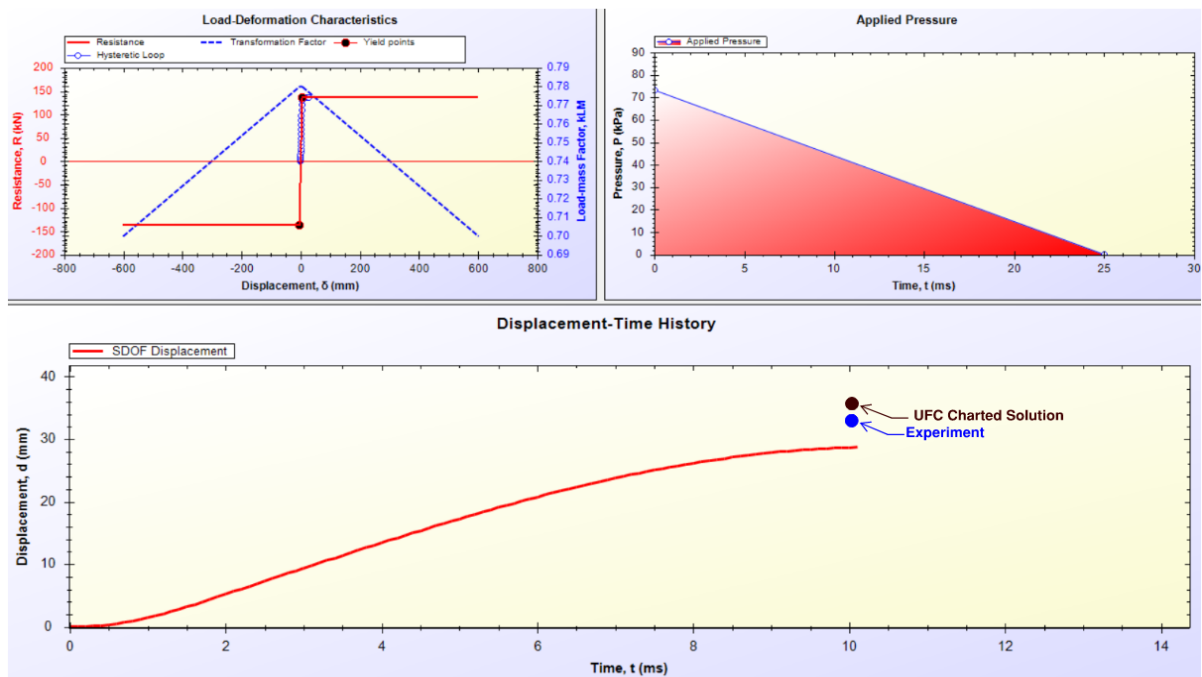


Figure E.11: M-C6 maximum displacement at mid-height

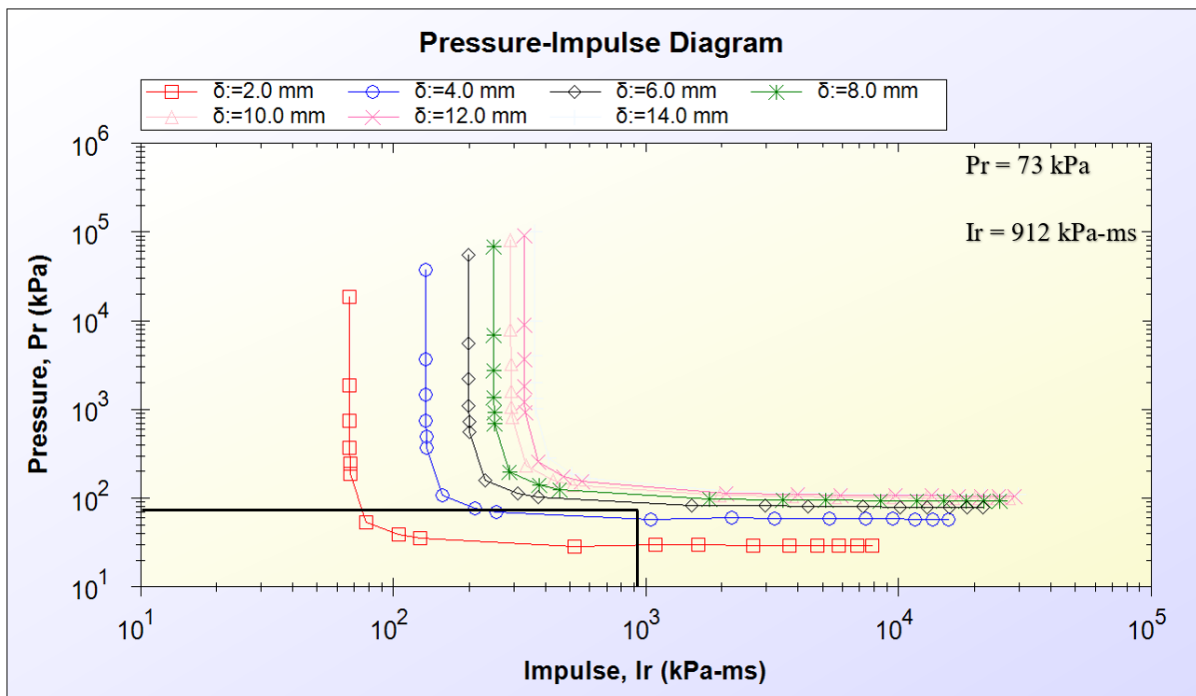


Figure E.12: P-I diagram for the sixth mullion with fixed supports

E.7 Seventh Mullion Maximum Deflection at Mid-height Obtained From RC-Blast, UFC-charted, and Experiment + Relevant PI Diagram

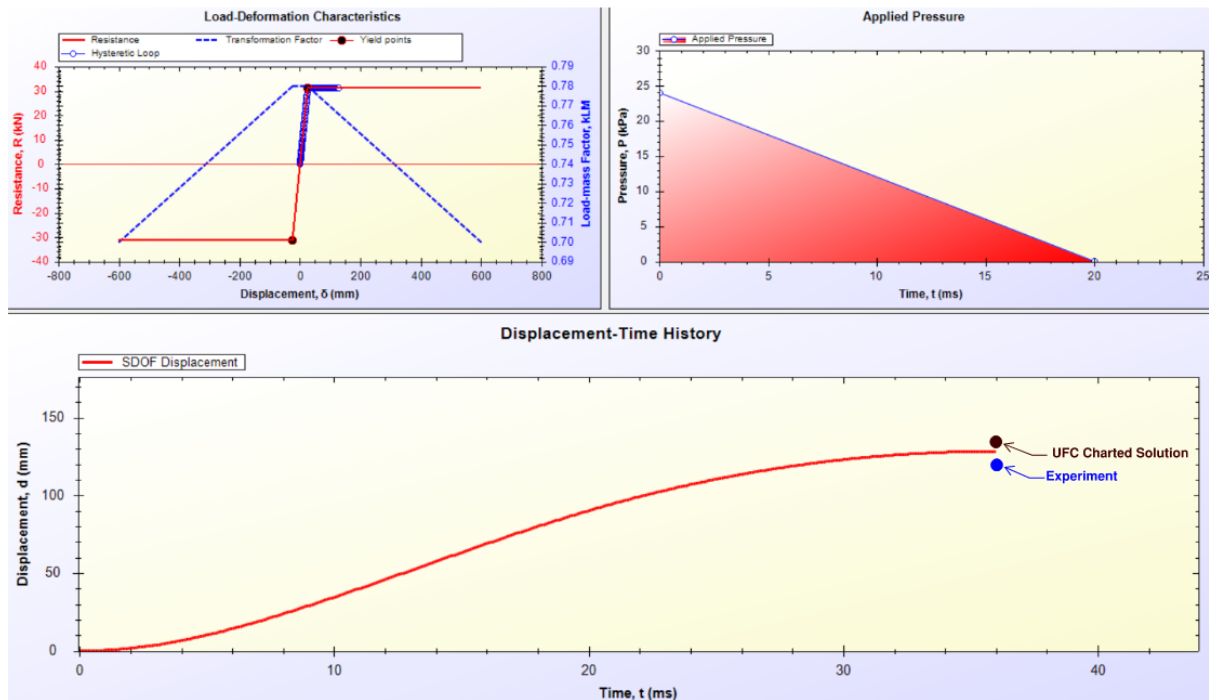


Figure E.13: M-C7 maximum displacement at mid-height

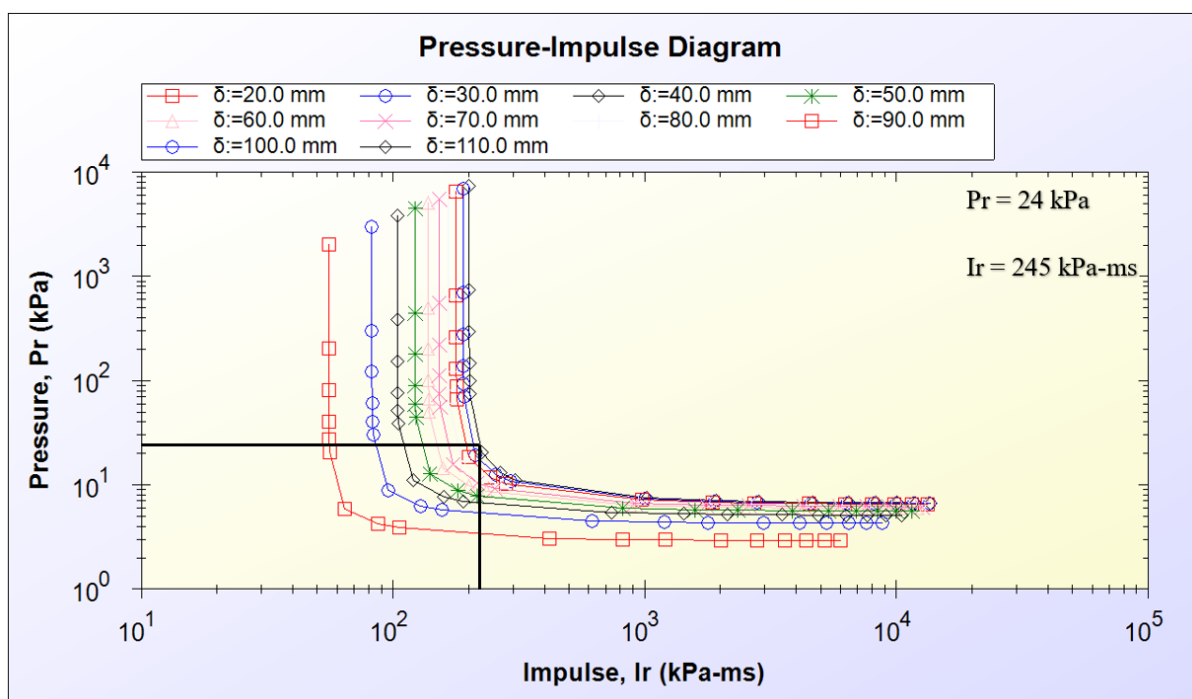


Figure E.14: P-I diagram for the seventh mullion with spring supports

E.8 Eighth Mullion Maximum Deflection at Mid-height Obtained From RC-Blast, UFC-charted, and Experiment + Relevant PI Diagram

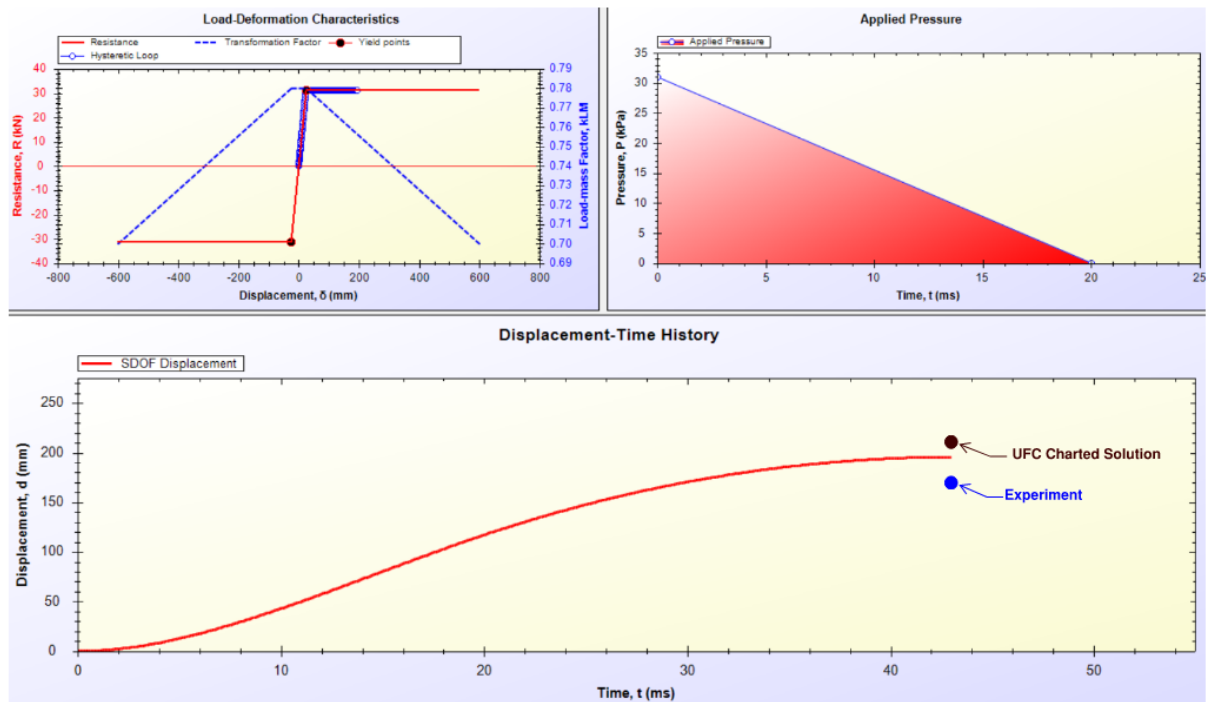


Figure E.15: M-C8 maximum displacement at mid-height

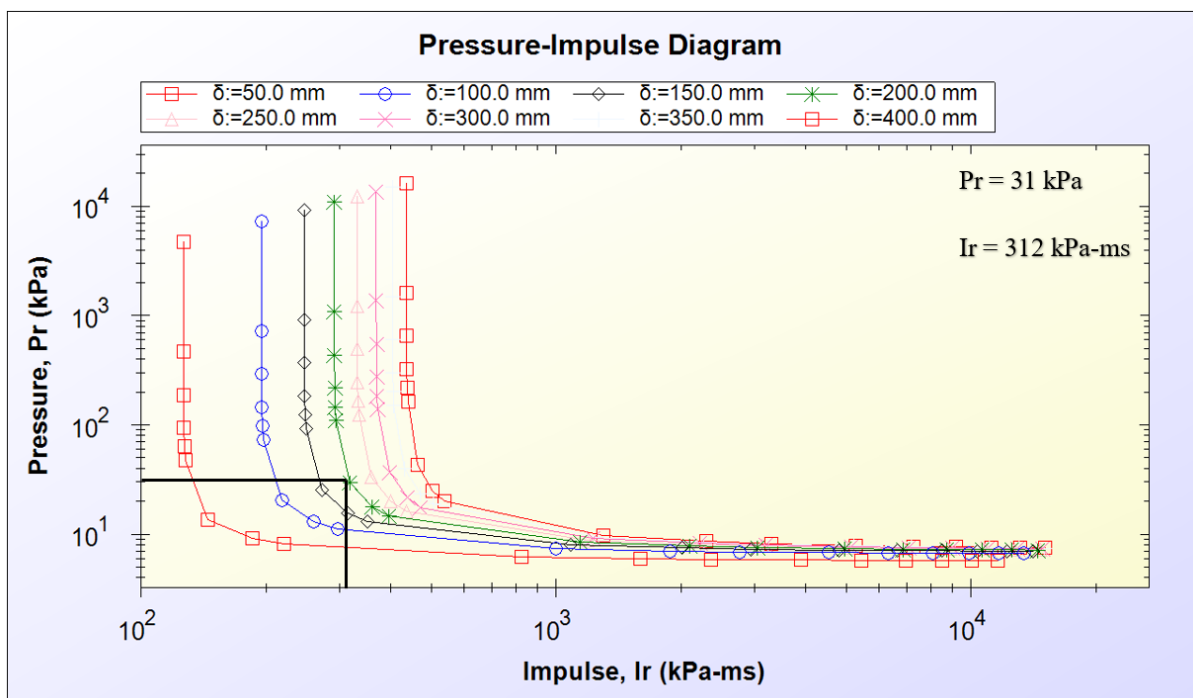


Figure E.16: P-I diagram for the eighth mullion with spring supports

E.9 Ninth Mullion Maximum Deflection at Mid-height Obtained From RC-Blast, UFC-charted, and Experiment + Relevant PI Diagram

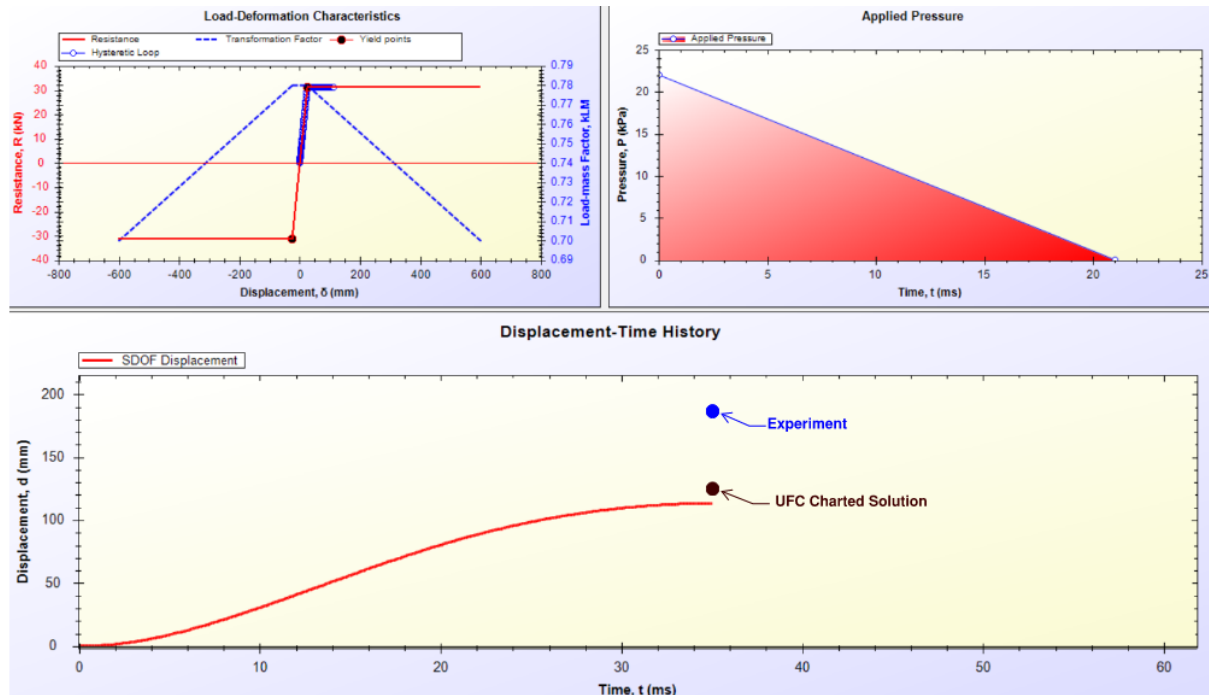


Figure E.17: M-C9 maximum displacement at mid-height

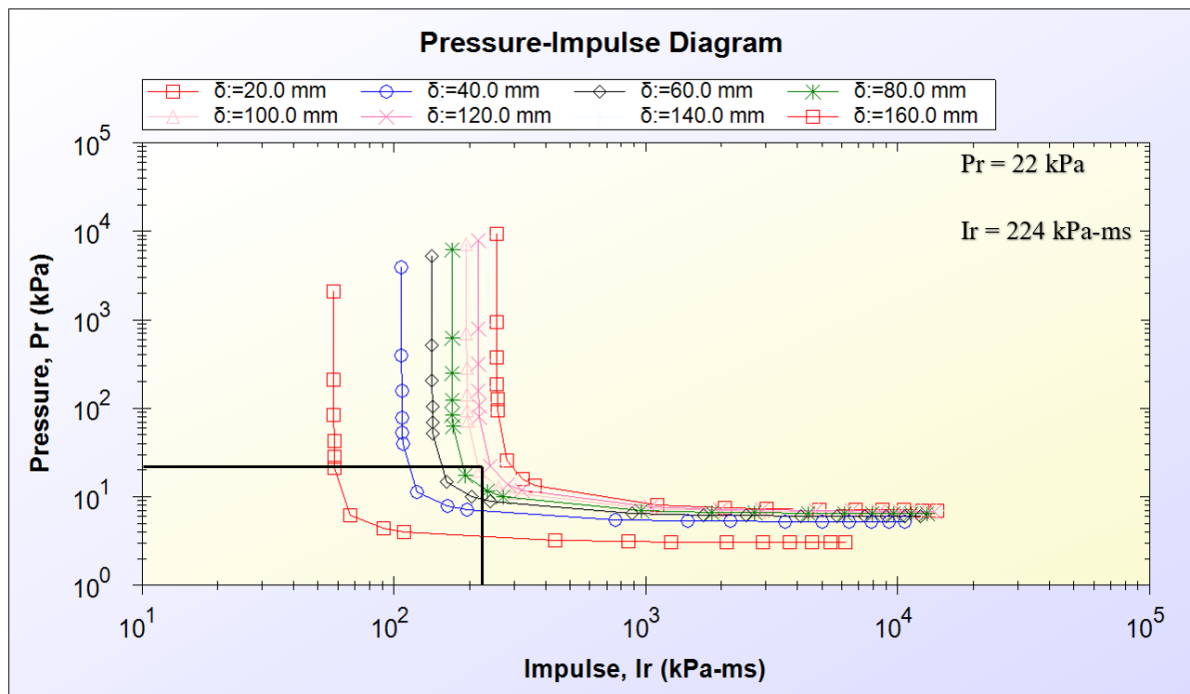


Figure E.18: P-I diagram for the ninth mullion with spring supports