

THE EFFECT OF STEEL STRAPPING TENSIONING TECHNIQUE AND FIBRE-REINFORCED POLYMER ON THE PERFORMANCE OF CROSS-LAMINATED TIMBER SLABS SUBJECTED TO BLAST LOADS

by

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ABSTRACT

Engineered wood products (EWP) are becoming extremely popular and a viable material option for the construction of residential, commercial, and hybrid buildings. Cross-laminated timber (CLT) is among one of the many EWP available in North America, which can be utilized for many different applications such as: walls, floors, and roofs. Despite the available requirements in the Canadian blast design standard (CSA, 2012) with regard to the design of wood structures, there are currently no provisions on how to retrofit timber structures to improve their performance when subjected to blast loads. The current study is aimed at investigating the effect of different retrofitting alternatives in order to improve the overall behaviour of CLT when exposed to out-of-plane bending.

The experimental program examined the behaviour of seventeen reinforced CLT slabs. Testing was conducted at the University of Ottawa by means of a shock tube capable of simulating high strain rates similar to those experienced during a blast event. The current study was divided into two phases. The first consisted of CLT slabs retrofitted with steel straps where strap spacing, location, and order of installation was investigated. The second phase focused on the development of dynamic properties of CLT panels when reinforced with GFRP. Lay-up configuration and fabric orientation were among the parameters explored.

The results from the experimental program show that reinforcing the panels with steel straps had minimal effect on the ultimate strength, but significant levels of post peak resistance and ductility was achieved. The horizontal straps were able to restrict the failure to small regions and to promote flexural failure by preventing rolling shear failure. It also eliminated flying debris and enhanced the ultimate strength, stiffness as well as ductility. Applying GFRP layers enhanced the overall behaviour of the slab resulting in a significant increase in peak resistance, ductility, and stiffness when compared to the dynamic results of an unretrofitted panel. The post peak resistance was also greatly improved. In particular, applying stacked quadraxial lay-up configuration significantly improved the ductility and resulted in the greatest post peak resistance. The effect of steel straps on damaged and retrofitted was relatively minimal, and only partial recovery of the resistance and the stiffness was achieved. GFRP with full confinement yielded better

performance compared to the unretrofitted and undamaged counterpart. More work is needed to quantify the benefits of using GFRP in these applications.

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NOTATIONS

Acronym		Definition
ANFO	=	Ammonium nitrate fuel oil
CFRP	=	Carbon fibre-reinforced polymer
CLT	=	Cross-laminated timber
DIF	=	Dynamic increase factor
DLF	=	Dynamic load factor
E.F.	=	Equivalent factor
EPI	=	Emulsion polymer isocyanate
EWP	=	Engineered wood products
FEA	=	Finite element analysis
FRP	=	Fibre-reinforced polymer
GFRP	=	Glass fibre-reinforced polymer
HFRP	=	Hybrid fibre-reinforced polymer
HM	=	High modulus
HS	=	High strength
HSC	=	High strength concrete
IM	=	Intermediate modulus
LSL	=	Laminated strand lumber
LVL	=	Laminated veneer lumber
MDOF	=	Multi-degree-of-freedom
MOE	=	Modulus of elasticity
MOR	=	Modulus of rupture
OSB	=	Oriented strand board
PAN	=	Polyacrylonitrile
P-I	=	Pressure-impulse
PRF	=	Phenol-resorcinol formaldehyde
PSL	=	Parallel strand lumber
PUR	=	Polyurethane
SEA	=	South-East Asia
SDOF	=	Single-degree-of-freedom

SH	=	Standard modulus
SPF	=	Spruce-pine-fir
SSTT	=	Steel strapping tensioning technique
TNT	=	Trinitrotoluene
WWM	=	Welded wire mesh

Symbol		Definition
α	=	Angle of incidence
γ	=	Specific heat ratio
$f(x, t)$	=	Distributed load per unit length
$\varphi(x)$	=	Shape function
$\bar{m}(x)$	=	Distributed mass
μ	=	Ductility ratio
a	=	Acceleration
A	=	Loaded area
a_0	=	Speed of sound in the air
b	=	Waveform parameter
c	=	Damping coefficient
ε_{cy}	=	Strain at corresponding wood compression yielding stress
ε_t	=	Strain at corresponding wood tension failure stress
E	=	Modulus of elasticity
f_c	=	Wood compression yielding stress
f_t	=	Wood tension failure stress
F	=	Total load
F_e	=	Equivalent load
F_o	=	Peak reflected pressure
$F(t)$	=	Total external force as a function of time
I^+	=	Positive impulse
I^-	=	Negative impulse
k	=	Stiffness
KE	=	Kinetic energy
K_L	=	Load transformation factor
K_{LM}	=	Load-mass transformation factor
K_m	=	Mass transformation factor
K_R	=	Resistance transformation factor
m	=	Mass
p_o	=	Ambient pressure

p_s	=	Peak overpressure
p_{so}	=	Peak incident pressure
p_{so}^-	=	Peak negative pressure
q_s	=	Dynamic peak pressure
Pr	=	Peak reflected pressure
R	=	Standoff distance
$R(t)$	=	Total internal force as a function of time
$R(y)$	=	Resistance
SE	=	Strain energy
t	=	Time
t_a	=	Time of arrival of shock wave
t_o	=	Positive phase duration of explosion
t_o^-	=	Negative phase duration of explosion
U_s	=	Blast wave front velocity
w	=	Natural frequency
W	=	Explosive weight
WD	=	Work done by the system
x_e	=	Elastic deformation
x_{max}	=	Maximum deformation
y_o	=	Initial displacement
y	=	Displacement
$\dot{y}_o$	=	Initial velocity
$\dot{y}$	=	Velocity
$\ddot{y}$	=	Acceleration
Z	=	Scaled distance

CHAPTER 1 – Introduction

1.1 Background

Over the past several decades, catastrophic environmental events, such as winds, earthquakes, and tsunamis have become identifiable amongst many engineers, researchers, and government officials. As a result, many documents with design guidelines on how to specifically determine and design for these loads have been developed and are continuously revised with growing increase in research in these particular fields. The study of the effect of blast loads on structural elements has become an area of interest within the research community due to deliberate (e.g. Oklahoma City, 2005) and accidental explosions (e.g. BP Texas City, 2005). Moreover, the need to further investigate the behaviour of materials under blast emerges from the devastating outcome from such event, where significant structural damage and loss of human life has been observed. Provisions have been enacted around the world to provide guidance on the analysis and design of structures against blast loading (CSA 2012, Federal Emergency Management Agency, 2011, ASCE/SEI, 2011, USADD 2008, Office of the Deputy Prime Minister, 2004). To date, only the Canadian blast design standard (CSA 2012) has provisions for the design of wood structures under the effect of blast loading, however the provisions are not fully developed. Furthermore, no provisions exist on how to retrofit timber elements to enhance their performance under blast effects.

The current study contributes to this field of knowledge by investigating the effect of different materials and wrapping techniques in order to enhance the properties of cross-laminated timber panels.

1.2 Research Needs

The available knowledge on the topic of blast effects on structures is primarily focused on reinforced concrete and steel structure elements, with limited studies that explicitly address the design or retrofitting of wood structures. This is in large part due to the insufficient blast resistant properties that wood possesses; wood is a light weight material in addition to exhibiting brittle behaviour. However, it is due to the advancements in timber technology and the emergence of engineered wood products that taller, larger, and

“important” buildings are beginning and continue to be constructed worldwide (e.g. Richmond Olympic Oval, SMEC Soccer Stadium, Bois-de-Boulogne Sports Center, Fort McMurray International Airport, etc.).

In order to encourage sustainable construction, Canadian provinces have issued provisions, Wood First Act (Government of BC, 2009), that require the use of wood as their primary choice of construction material for provincially funded buildings. As a result some of these public buildings would require considerations for blast design. Just recently the amended version of the Ontario Building Code allowed the construction of mid-rise buildings (up to six storeys) entirely out of wood (Government of Ontario, 2014). The majority of residential buildings in North America are made of wood and although they may not be the main target of an attack, they may be in the vicinity of a targeted building or an accidental blast event resulting in damage and loss of life.

The Canadian Standard Association (CSA) has developed and published a document (CSA S850-12) that deals explicitly with the analysis and design requirements for structures subjected to blast loading. The *Design and Assessment of Buildings Subjected to Blast Loads* standard provides design guidance for various construction materials including timber. The research that was used in the development of these specific standard provisions is based on very limited research that has focused on light frame wood stud walls (e.g. Jacques et al., 2013).

Recent developments in material and connections has allowed for the frequent use of engineered wood products, such as Cross-Laminated Timber (CLT) to be utilized in the construction of residential, commercial, and hybrid buildings. In 2005 FPInnovations launched a multi-disciplinary program under the Transformative Technologies Program of Natural Resources Canada. The research findings, in addition to knowledge acquired from European research, led to the preparation and development of a peer-reviewed document known, in North America, as the CLT Handbook (FPInnovations, 2011). The handbook presents technical design information on connection details, seismic, vibration, fire, and acoustic performance etc. The most recent edition of the CSA O86 standard (CSA, 2014) includes and outlines design requirements for generic cross-laminated timber produced according to the PRG320 manufacturing standard (ANSI/APA, 2012).

Despite the comprehensive knowledge available on the behaviour of CLT elements, their performance under blast load effects has not yet been established.

The ongoing research conducted at the University of Ottawa has vastly contributed to the understanding of the materialistic properties and overall behaviour under dynamic loading. Experimental investigation of CLT was undertaken to establish the dynamic properties of the material and to propose predictive models for the behaviour of CLT panels under the effect of blast loading (Poulin et al., 2017). Additionally, an investigation into the effect of realistic boundary conditions on the behaviour of cross-laminated timber slabs subjected to simulated blast loads (Côté, 2017) has been conducted. The current study builds on these efforts by investigating reinforcement and retrofitting applications for CLT panels in order to enhance the performance.

The blast wave that is generated from an explosion is known to cause the most structural damage; however, the main cause of occupant injury is directly related to flying debris and fragments (Bogosian and Avanesian, 2004). The CSA S850-12 standard states that any component, whether it is structural or non-structural, must be designed and detailed such that the risk of flying debris is reduced (CSA, 2012). The current research study also aims to investigate different options in order to reduce or mitigate the amount of generated debris.

1.3 Cross-Laminated Timber (CLT)

1.3.1 General

Cross-laminated timber (CLT) is an innovative engineered wood product that was originally developed in Europe and has since been gaining traction amongst the construction and wood industries in North America. CLT panels can be utilized for many applications (e.g. walls, floors, and roofs) which has resulted in the construction of numerous extraordinary buildings. This product is emerging as an advantageous alternative due to its many benefits such as ease of handling, quick and cost effective erection time, in addition to favourable thermal, acoustic, fire, and seismic properties.

A typical CLT panel consists of several layers of lumber boards that are stacked at 90 degrees (i.e. longitudinal and transverse laminations) and glued together along the wide

face, as shown in Figure 1.1. Although it is common to see alternating consecutive layers, ‘double layers’ can also be implemented in order achieve explicit material properties. Examples of these specific configurations are illustrated in Figure 1.2. The overall width of typical panels is generally between 0.6 – 3 m, and the length and thickness can reach maximum dimensions of 18 m, and 400 mm, respectively.

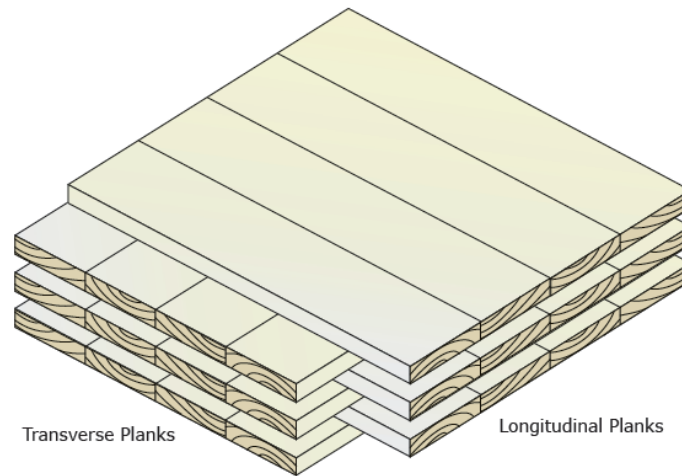
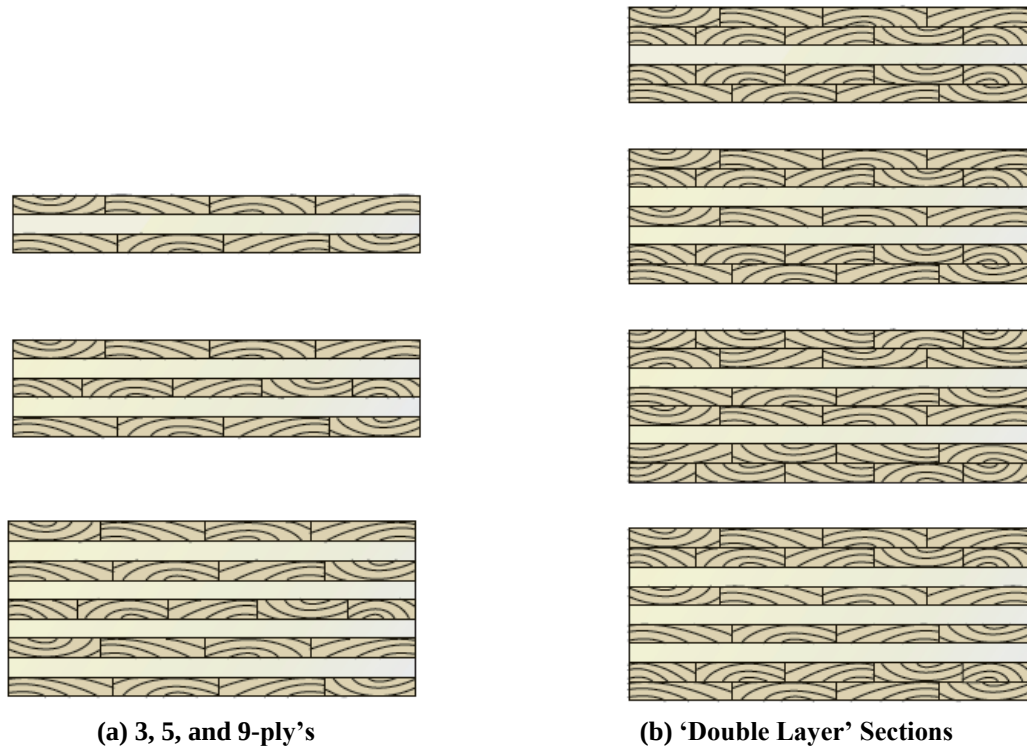


Figure 1.1: CLT Panel Configuration (FPInnovations, 2011)



(a) 3, 5, and 9-ply's

(b) 'Double Layer' Sections

Figure 1.2: Typical CLT Cross-Sections (FPInnovations, 2011)

In general, the longitudinal layers are normally oriented in the direction that is parallel to the load in order to maximize the resistance of the system, as shown in Figure 1.3. In the case of walls and floors these boards run parallel to the vertical loads, and the major span direction, respectively. The laminations are also capable of attaining significant stiffness and strength in both the longitudinal and transverse direction, resulting in two-way bending action.

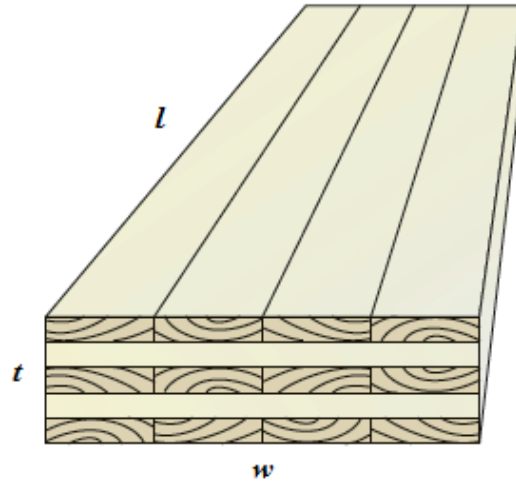


Figure 1.3: CLT Specimen (FPInnovations, 2011)

1.3.2 Raw Materials

Cross-laminated timber panels are fabricated with two fundamental components: lumber boards and structural adhesive. In North America, CLT is generally manufactured with dimensional lumber boards that conform to the CSA O141 (CSA, 2005) and PS 20 (NIST, 2015) standards. By strictly limiting the lumber to the aforementioned standards, designers and engineers will be able to use material properties and design values that are published for generic CLT elements (CSA, 2014 and NDS, 2015).

The lumber boards can be of varying width (60 – 240 mm) and thickness (10 – 50 mm), and are finger jointed together to create dimensions that are significantly greater than said individual boards. The lumber used is kiln dried and consists of visually and/or machine stress-rated boards. It is common practice to select machine rated lumber grades or Structural Light Framing No. 2 & Better grade lumber stock for layers that are in the major direction of loading; whereas, a No. 3 grade is typically used for laminations in the minor direction (FPInnovations, 2011). Figure 1.4 depicts a typical floor or roof assembly in which the major and minor direction of loading is along the longitudinal, and transverse layers, respectively.

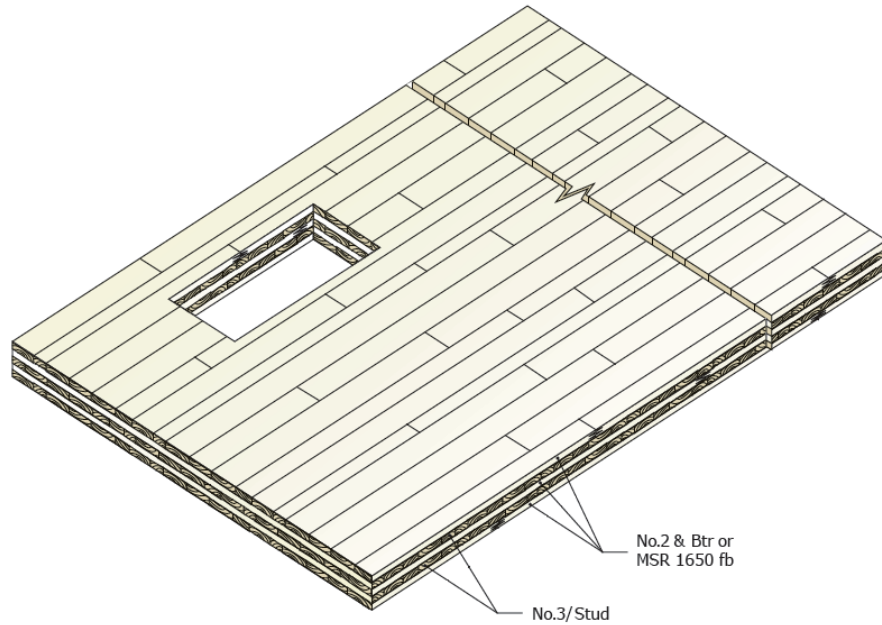


Figure 1.4: CLT Floor/Roof Assembly (FPInnovations, 2011)

The structural adhesive that is selected in the manufacturing process of CLT must meet the requirements of the *Evaluation of Adhesives for Structural Wood Products (Limited Moisture Exposure)* (CSA, 2017). Although these adhesives are known for having high moisture resistances, the standard is intended for products at a dry service condition. Likewise, such adhesives will have met standards (e.g. CSA O122.9 *Evaluation of Adhesives for Structural Wood Products (Exterior Exposure)*) for adhesives that are fitting for exterior exposure (FPInnovations, 2011). There are three commonly known structural adhesives that can be utilized in the manufacturing process of CLT, namely phenol-resorcinol formaldehyde (PRF), emulsion polymer isocyanate (EPI), and polyurethane (PUR).

1.3.3 Structural Applications

Cross-laminated timber is becoming the primary construction material of non-residential buildings, in addition to residential buildings. The figures shown below are a couple examples of contemporary and efficiently built structures that can be found in North America.

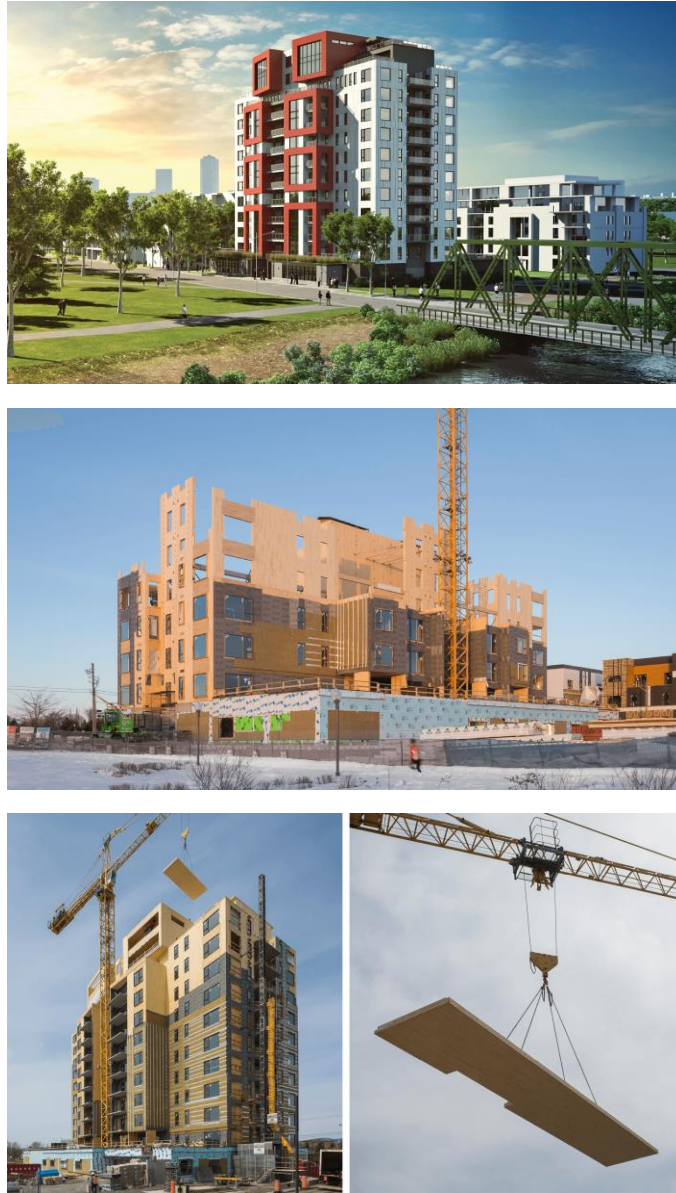


Figure 1.5: Origine 13-Storey Building, Québec, Québec (Nordic Structures, 2017)



Figure 1.6: University of Massachusetts Design Building, Amherst, Massachusetts (Nordic Structures, 2017)

1.4 Steel Strapping Tensioning Technique

1.4.1 General

Steel strapping can be used as a post-tensioning technique which can serve as external confinement to a structural element. This form of steel strapping is typically seen in the packaging industry (Figure 1.7); however some attempts have been made to utilize this material in the field of structural engineering (Lloyd, 2015; Sabri, 2013; Awang et al., 2013; Pin Lee et al., 2013). The underlying reason behind the implementation of this technique is their relatively low cost, and the ease of application; in addition to their wide versatile function.



Figure 1.7: Typical Industrial Applications (Signode, 2017)

1.4.2 Steel Straps

The use of a steel strapping system as a form of external confinement allows for additional reinforcement, bracing, and compression retention. The correct selection of a steel strap is solely based on the element to be fastened and stabilized. Regardless of the application, steel straps are readily available in two different forms of coil windings (e.g. mill wound and ribbon wound).

Steel strapping is ideal for heavy duty applications. One of the underlying advantageous is the variation of steel grades. The addition of carbon content generally leads to preferred mechanical properties (e.g. high tensile strength). Furthermore, steel straps can possess a series of finishes in order to improve their overall performance. Straps can be

painted, painted and waxed, or zinc painted and waxed. A painted finish offers a level of corrosion resistance. Painted and waxed, in addition to corrosion resistance, offers improved load transmission especially around load corners. Last, zinc and waxed finish, includes a zinc enriched coating that provides rust resistance and protects the strap at points of surface damage or scratches. Based on the particular application of interest a strap with the appropriate finish shall be chosen. The table below summarizes typical steel strapping properties.

Table 1.1: Magnus Strapping Properties (Signode, 2017)

Strap Width (mm)	Strap Thickness (mm)	Average Strength* (N)	Yield <i>M/kg</i>	Coil Winding	Strap Finish
12.7	0.51	6,780	19.7	Mill	Zinc
15.9	0.58	9,670	13.7	Mill	Painted & Waxed
19.0	0.64	12,450	10.5	Ribbon	Painted & Waxed
31.8	0.74	24,020	5.5	Ribbon	Zinc
50.8	1.12	55,600	2.2	Ribbon	Painted & Waxed

*Strap break strengths are listed as averages.

1.4.3 Joint Types

A steel strapping system constitutes of two fundamental components: a steel strap and a seal. There is an array of seals (e.g. snap-on, open-flange, thread-on, and push-type) that are available, depending on the use of such a system. They come in various shapes and sizes and can be installed prior to the strap tensioning or shortly after. Once the strap has been wrapped around the desired perimeter, it is then sealed by means of a joint. There are two commonly known joint types, namely, a seal-less joint and a basic seal joint. A seal-less joint does not require a seal as it seals the straps by creating an interlocking key pattern. The basic seal joint consists of either a crimp or a notch type joint. The latter locks the strap ends by cutting or notching the seal which in turn forms tabs at the edges of the seal. The strength, for a notch joint, is generated from the mechanical interlock that exists between the strapping and the seal. A crimp joint is formed by pressing or crimping undulations into the seal and the ends of the straps. In this case the strength is produced by the deformation of the seal which creates high frictional forces. Examples of these

joints are illustrated in Figure 1.8. In both cases, manual or pneumatic combination tools can be utilized to induce a tensile force in the strap and to seal the joint.

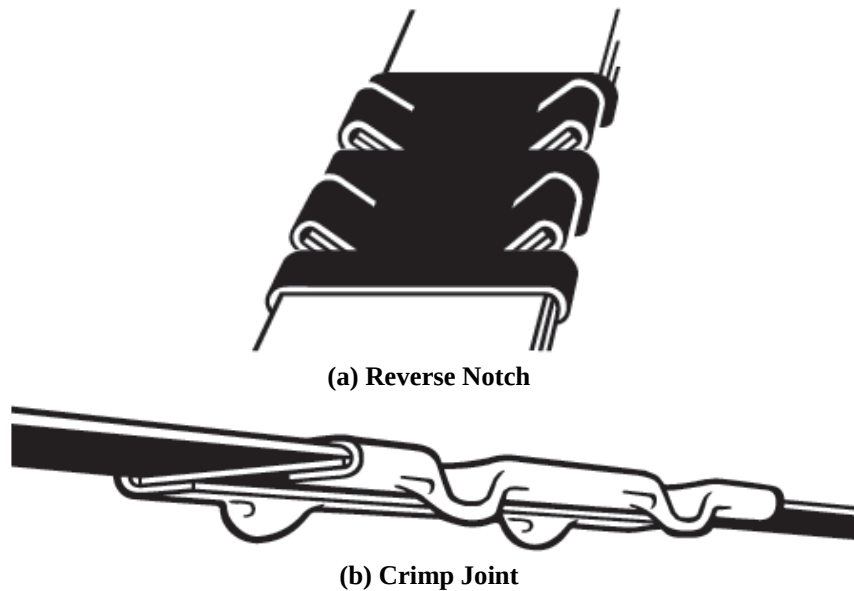


Figure 1.8: Basic Joint Types (Signode, 2017)

1.5 Fibre-Reinforced Polymer

1.5.1 General

Fibre-reinforced polymer (FRP) is a composite material that consists of a polymer matrix which is reinforced with fibres. Although fibreglass has typically been used on aircrafts, boats, and automobiles, FRP is becoming a promising construction material in many structural applications for both rehabilitation and strengthening purposes, as well as for the construction of newly built structures. Findings from numerous research programs have led to the development and integration of FRP in the field of civil engineering. Various structural components (e.g. beams, columns, slabs, and walls) remain of great interest within the research community and are continuously investigated in order to improve flexure, shear, and axial load capacity.

The fibres provide the stiffness and strength, while the polymer matrix (i.e. resin system) is responsible for binding and distributing the loads across all the fibres within the system (i.e. transfer of forces), in addition to protecting the fibres from environmental effects.

The mechanical properties of an FRP composite are heavily dependent on the materials properties of the constituents (e.g. fibres, resin, and additives), their volume ratios, orientation, etc. The orientation plays a fundamental role in the configuration of a composite material as it is directly linked to the properties of an FRP system. Generally by aligning the fibres in one direction (i.e. lamina, or layer) the maximum strength and stiffness can be obtained through a unidirectional ply, resulting in an anisotropic composite. Conversely, an irregular dispersion of the fibres will result in an isotropic composite.

One of FRP's leading advantages is its design flexibility and adaptability. Composite materials are fabricated in various shapes and forms. Sheets of fabric are generally used in order to repair or strengthen existing buildings or structural elements. Alternatively, bars or plates can be used as internal or external reinforcements in the case of new construction.

1.5.2 Fibres

There are three main types of fibres that are commonly used in the construction industry to date. They are: glass fibres, carbon or graphite fibres, and organic or natural fibres.

Glass fibre is the most commonly used plastic composite since the material exhibits appealing benefits (e.g. high strength and chemical resistance). Glass fibre-reinforced polymer (GFRP) is the result of mixing colemanite, limestone, kaolin, and sand. The proportion of these constituents leads to several types or grades of glass. E (electrical)-glass is used frequently as a result of being an economical choice while attaining high electrical properties, and degradation resistance. C (corrosion)-glass is very similar to E-glass; however the underlying difference is the removal of boron oxide from the chemical formulation. S (strength)-glass is known for providing a significant amount of strength, however at a higher cost. This form of GFPR is typically seen in structural applications mainly because of its higher tensile strength, and stiffness. Other types of GFRP include: A (alkaline)-glass, D (dielectric)-glass, R (reinforcement)-glass, and T (thermal insulator)-glass.

Carbon fibres (CFRP), also referred to as graphite fibres, are recognized for their high strength, stiffness, modulus of elasticity, and fatigue resistance. Additionally, they possess thermal and electrical conductivity, and low thermal expansion which allows this composite to be used in certain applications where GFRP or other natural fibres would be ineffective. Carbon fibres are generally produced by using the following precursors: polyacrylonitrile (PAN), pitch, and rayon (i.e. regenerated cellulose). PAN is the most widely utilized carbon fibre and can be classified based on their respective modulus of elasticity. Standard modulus (SH) or high strength (HS), intermediate modulus (IM), and high modulus (HM) are the known groups of a PAN carbon fibre. As the modulus of elasticity is increased, the tensile strength of the fibre experiences a decrease.

An organic or natural fibre includes anything from boron and silicon to carbide, ceramic, and nylon. Aramid is yet another example of fibre-reinforced polymer, with significant amount of fire, heat, and chemical resistance. Despite having good mechanical properties (e.g. high tensile strength, low density, moderately high modulus of elasticity, and low thermal expansion) these fibres are generally used in combination with others.

Figure 1.9 illustrates the stress strain relationship of each of the aforementioned fibres that have been discussed. Unlike steel, fibre reinforced polymer exhibits linear elastic behaviour up until rupture. Despite providing significantly greater tensile strength, when compared to steel, they fail in a brittle manner.

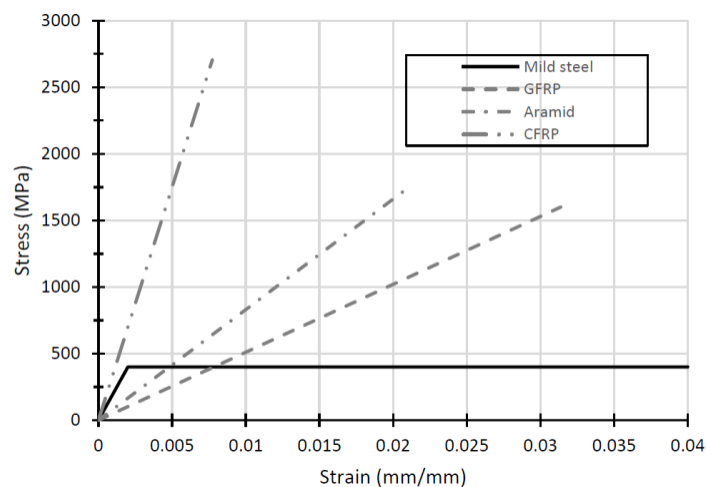


Figure 1.9: Typical Stress-Strain Curves for Steel and FRP (American Concrete Institute, 2008)

1.5.3 Fabric Orientation

The fibres within any FRP fabric are solemnly responsible for withstanding the applied forces which in turn delivers optimal mechanical properties. In order to improve the overall strength, the fibres are loaded in the primary fibre direction. There are three elementary arrangements of fibres, namely, uniaxial, biaxial, and multiaxial. Uniaxial fabrics consist of fibres aligned along the zero direction or in the direct axis of the applied load. Biaxial fabrics is an arrangement of fibres in two directions, typically interweaved at 0° and 90° , or at $\pm 45^\circ$. Multiaxial fabrics can refer to fabrics in which the fibres are oriented in three (i.e. triaxial) or in four (i.e. quadraxial) directions. The development of quadraxial fabrics (Figure 1.10), has allowed engineers to implement this type of reinforcement in various infrastructure applications.

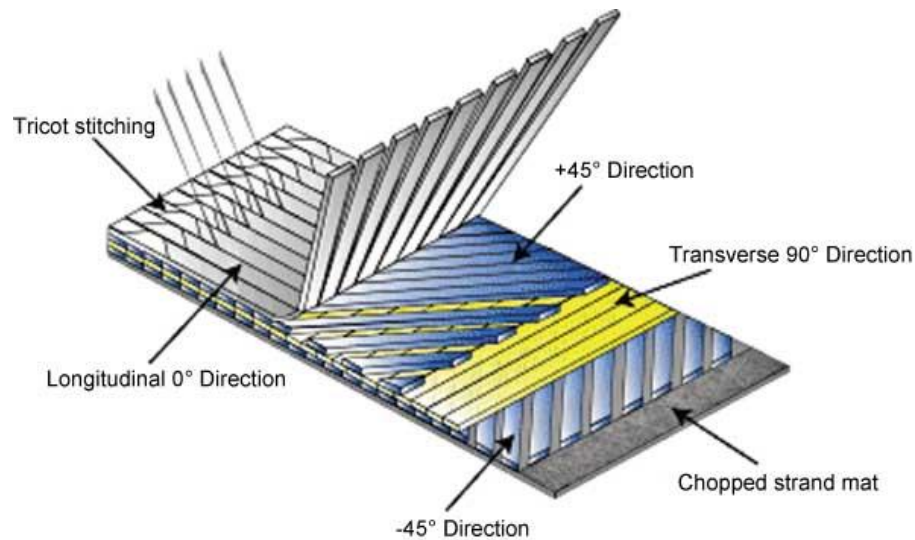


Figure 1.10: Typical Quadraxial Laminate (Vector Ply , 2017)

1.5.4 Resins

Resins or polymer matrices, usually accounts for 30 to 40 % of an FRP composite material. They are responsible for the distribution of loads, in addition to protecting the fibres from being damaged. They also provide lateral support against fibre buckling and transfer shear stresses within the composite system. Resins can be classified as either a thermoset, or thermoplastic.

Thermoset resins are typically manufactured in a liquid state and undergo an irreversible chemical reaction in which they harden and become solidified. Although there are many types of propriety products epoxy is frequently used in structural applications due to its excellent strength and stiffness properties at moderate to high temperatures. Epoxies can be used in a variety of applications due to their versatility and high corrosion, chemical, and fatigue resistance. They are known to settle at a slower rate; however once fully cured they become quite brittle.

Thermoplastic resins can soften and become a viscous liquid when exposed to heat prior to processing; after which they solidify. The process is reversible as there is no on-going chemical reaction throughout the curing stage. Despite certain advantages associated with thermoplastics (e.g. recyclability, unlimited storage life, ease of handling and repair, and short fabrication times), they prove to be difficult to reinforce with fibres because of their high viscosity (Mallick, 1993).

With a wide range of resins available, the selection of the appropriate matrix is crucial in the formation of an FRP composite. It is important to keep some key features in mind (e.g. adhesion, mechanical properties, and resistance to environmental deterioration).

1.6 Blast Loading

1.6.1 Types of Blast Loading

A blast load is typically characterized by a large and sudden release of energy from the source of an explosion. The origin of such an incident can be the product of a physical, nuclear, or chemical event which results in the formation of a shock wave (Cormie et al., 2009). A physical explosion is best described as the eruption of a volcano, the failure of compressed air tank cylinders, or when two liquids consisting of different temperatures are combined and create a volatile mixture. A nuclear explosion occurs when there is a re-arrangement of protons and neutrons amongst pre-existing nuclei in order to form new nuclei. Last, a chemical explosion is a result of an exothermic reaction and a rapid oxidation process between two recognized fuel elements: carbon and hydrogen atoms. The decomposition of these constituents can also be referred to as runaway reactions (Dusenberry, 2010).

1.6.2 Sources of Blast

Typical explosive materials exist in the form of a solid or liquid and are often referred to as 'condensed'. Upon an exothermic reaction, the fuel will decompose and produce a gas. As the newly formed gas expands it will create blast waves as it travels through the air and shock pressures as it travels through any solid material. The blast load on a structure is highly dependent on the state of the fuel, energy output, and standoff distance. In the case of a solid fuel source, the material will result in short load durations while producing high pressures. On the contrary, an explosive material in a liquid state leads to longer load durations and generates low pressures.

A chemical explosion (i.e. combustion) can be characterized by its reaction rate, which leads to two main types of energy release mechanisms. Detonation, which occurs when the source of the reaction is greater or equal to the speed of sound (i.e. supersonic), is known to form high intensity shock waves and creates an instantaneous rise in peak pressure, as shown in Figure 1.11. High explosive materials are classified based on their sensitivity to initiation or detonation. Primary (e.g. lead azide, mercury fulminate), secondary (e.g. TNT, cyclonite), and tertiary explosives (e.g. ammonium nitrate fuel oil, ANFO) can all be detonated by the ignition of a flame, spark, or even impact. However, the preceding categories are listed from the most to the least sensitive and thus a tertiary explosive would require a significantly powerful charge to initiate detonation.

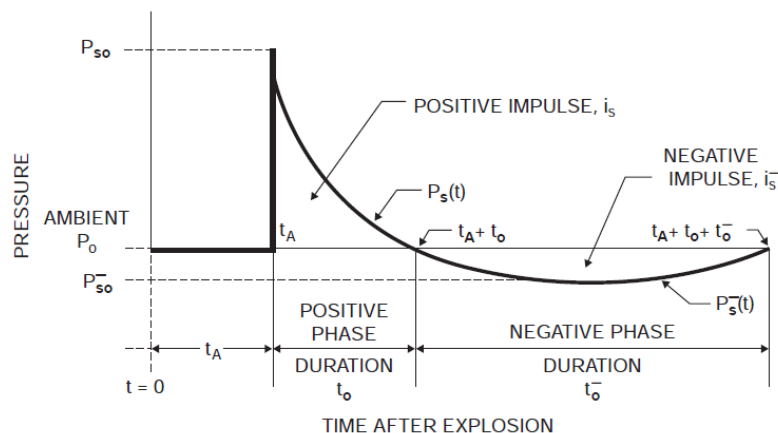


Figure 1.11: Shock Wave formed by a Detonation (Dusenberry, 2010)

Deflagration occurs when the source of the reaction is less than the speed of sound (i.e. subsonic). A pressure wave is developed and there is a steady increase prior to achieving peak pressure, as shown in Figure 1.12. A deflagration is normally linked to low explosives which are also known as propellants and pyrotechnics. Low explosives can be described as energetic materials that do not typically detonate and as a result generate pressure pulses that are smaller in amplitude and longer in duration when compared to high explosives. Deflagrate explosions can be just as hazardous as a detonation especially when there are high concentrations of flammable material present in the air. This ignitable fuel source is often referred to as ‘vapor cloud or ‘fuel-air’ explosion (Cormie et al., 2009). Vapor cloud explosions are most commonly seen in process, oil, and gas industries.

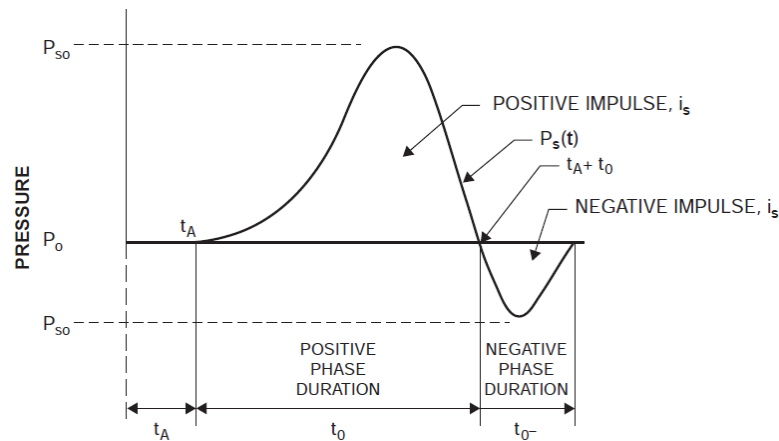


Figure 1.12: Pressure Wave formed by a Deflagration (Dusenberry, 2010)

1.6.3 Blast Wave Parameters

As mentioned in the previous subsection, a detonation from high explosives results in a blast wave, which is also known as a shock wave. The reaction of explosive material results in the production of a hot gas. As the gas creates a violent expansion, it begins to ‘push’ on the surrounding air particles and creates a thin layer of compressed air which is referred to as the blast wave. This shock front creates a peak incident pressure (p_{s0}) which can be defined as the instantaneous increase in atmospheric pressure at the time of arrival (t_a) of the shock front, as seen in Figure 1.11. In an ideal scenario, where the explosion is completely detonated in free air, the wave front will travel outward from the centre of the

explosion in a spherical profile and arrive at peak pressure at time t_a . As the shock wave propagates through the air, there is a noticeable increase in time and a decrease in the temperature, velocity, and most importantly pressure. As the overpressure begins to decay it returns to ambient pressure (p_o), which marks the end of the ‘positive phase’ that occurred within the duration time (t_o). It is usually the positive phase that has the most significant effect on surrounding structures. At time t_a+t_o , the pressure continues to dissipate and drops below ambient pressure to create a peak negative pressure (p_{so}). A vacuum or suction like effect is created by the overexpansion of the air particles, typically referred to as the ‘negative phase’. Eventually, at time $t_a+t_o+t_o^-$, equilibrium is re-established and the pressure returns to its ambient state. The peak positive pressure is generally greater than the peak negative pressure and therefore the peak incident pressure in the negative phase is often neglected in the analysis of primary structural elements (Krauthammer, 2008). However, negative pressure could govern the design of components such as window anchorages.

By developing a pressure-time history curve (as illustrated in Figures 1.11 and 1.12), peak pressures, phase durations, and impulses can be determined. The impulse is defined as the amount of energy that is brought on by a shock wave and can be found by simply calculating the area underneath the pressure-time history curve. The positive impulse (I^+) of the positive phase is calculated by integrating the pressure time history curve between times t_a and t_a+t_o . Similarly, the negative impulse (I^-) of the negative phase is calculated by integrating between times t_a+t_o and $t_a+t_o+t_o^-$. Both of these equations are expressed in Equations 1.1, and 1.2, respectively.

$$I^+ = \int_{t_a}^{t_a+t_o} P_s(t) dt \quad [\text{Eq. 1.1}]$$

$$I^- = \int_{t_a+t_o}^{t_a+t_o+t_o^-} P_s(t) dt \quad [\text{Eq. 1.2}]$$

The pressure-time history of any blast wave can be described by the exponential function presented in Equation 1.3 (Cormie et al., 2009).

$$P_s(t) = p_s \left[1 - \frac{t}{t_o} \right] \exp \left\{ -\frac{bt}{t_o} \right\} \quad [\text{Eq. 1.3}]$$

Where p_s is the peak static overpressure, t is the time, t_o is the positive phase duration, and b is the waveform parameter.

It is common practice to assume that the peak positive pressure decays linearly until time t_o . This simplified approach assumes that the pressure and the impulse of the idealized model and the exponential function remain the same. By introducing simple triangles, as shown in Figure 1.13 the positive impulse (i_s) and duration, in addition to the peak negative pressure (p_{so}^-) and impulse (i_s^-) can be determined with the following equations, respectively.

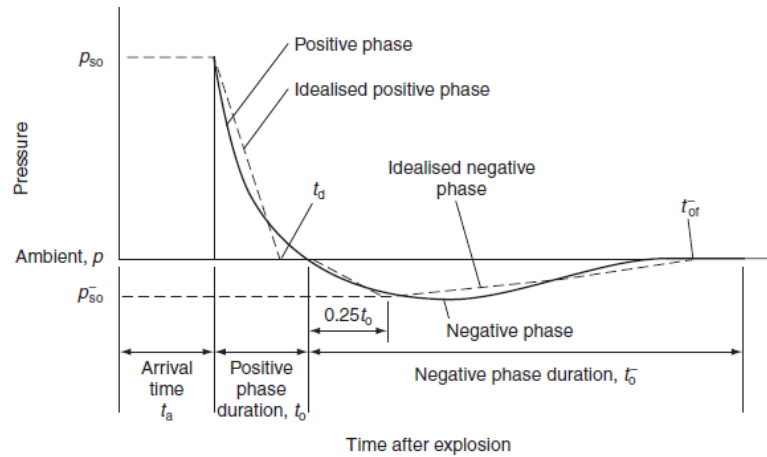


Figure 1.13: Idealization of the Positive Peak Phase (Cormie et al., 2009)

$$i_s = \frac{1}{2} t_o p_s \quad [\text{Eq. 1.4}]$$

$$t_o = \frac{2i_s}{p_s} \quad [\text{Eq. 1.5}]$$

$$p_{so}^- = -\frac{0.35}{Z} \quad (Z > 1.6) \quad [\text{Eq. 1.6}]$$

$$i_s^- \approx i_s \left[1 - \frac{1}{2Z} \right] \quad [\text{Eq. 1.7}]$$

Where Z is a scaled distance.

Analytical solutions have been developed in order to determine the blast wave front velocity and dynamic peak pressure, as seen in Equations 1.8, and 1.9 (Cormie et al., 2009).

$$U_s = \sqrt{\frac{6p_s + 7p_o}{7p_o}} a_0 \quad [\text{Eq. 1.8}]$$

$$q_s = \frac{5p_s^2}{2(p_s + 7p_o)} \quad [\text{Eq. 1.9}]$$

Where p_s is the peak static overpressure, p_o is the ambient air pressure, and a_0 is the speed of sound in the air at ambient pressure.

1.6.4 Scaling Laws

Scaling laws have been developed which allow designers to determine important blast parameters such as blast pressure, impulse, shock wave velocity, load duration, and arrival times (Hopkinson, 1915; Cranz, 1926). The Hopkinson-Cranz scaling law, which is commonly known as the ‘cube root scaling’, has thus become a widely used method in order to present blast wave relationships. Equation 1.12 can be used to calculate the dimensionless scaled distance which is then used in a series of curves, as illustrated in Figure 1.14, to determine blast data.

$$Z = \frac{R}{W^{1/3}} \quad [\text{Eq. 1.12}]$$

Where R is the distance from the centre of a spherical charge (i.e. standoff distance) expressed in meters, and W is the charge weight expressed in kilograms of TNT.

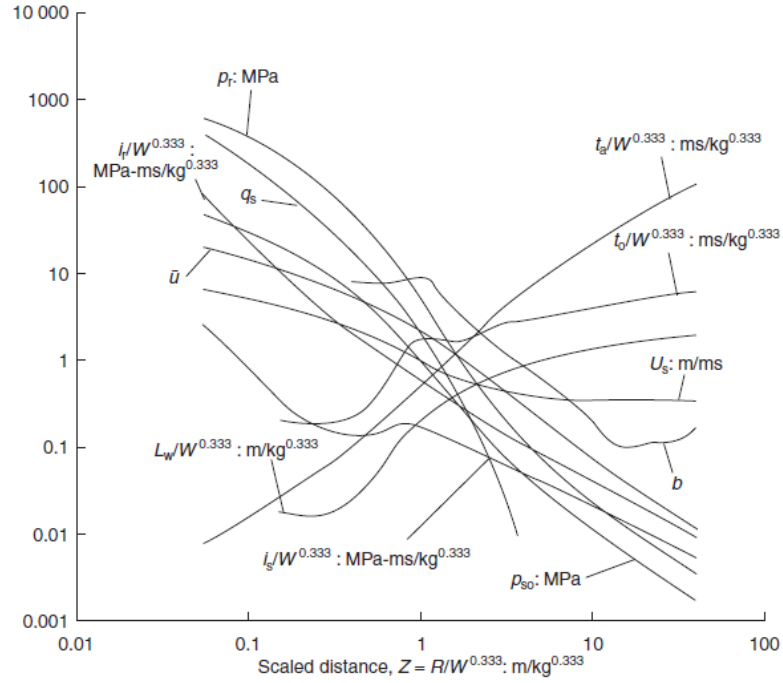


Figure 1.14: Blast Wave Parameters for a Spherical Charge of TNT (Cormie et al., 2009)

Due to its availability and purity, TNT has become the standard reference explosive, used to determine an equivalent weight that is capable of producing the same explosion energy, and blast impulse and pressure, of any explosion of interest (Dusenberry, 2010). Despite having a variety of techniques, a simplified method involving TNT equivalent factor (E.F.) is used. Once the TNT E.F. is established the mass of any high explosive in terms of TNT equivalent is found by simply multiplying the mass by its corresponding equivalency factor, as presented in Equation 1.14. Essentially a value that is greater than unity indicates that the corresponding explosive has a greater specific energy (i.e. explosive yield) (Cormie et al., 2009).

$$TNT \text{ E.F.} = \frac{Q_x}{Q_{TNT}} \quad [\text{Eq. 1.13}]$$

$$mass_{x \text{ of } TNT} = mass_x \times TNT \text{ E.F.} \quad [\text{Eq. 1.14}]$$

1.6.5 Wave Reflections

The detonation of an explosion causes the shock waves to propagate and interact with surrounding surfaces resulting in a loss of spherical shape, amplification in magnitude, and increase in velocity. The interaction between the wave and an object, or a medium denser than air, will cause the wave to reflect. However, depending on the size and geometry of the object the wave may in fact diffract around it. The incident pressure is defined as the pressure that is created by the detonation and is the initial wave that hits a structure. This incident wave is then reflected and due to the continuous arrival of air particles, a region of further compressed air is formed. This increase in pressure, which may be a combination of incident, stagnation, and dynamic pressure, is called the reflected pressure.

Normal reflections occur when the propagating blast waves interact with the surface of the structure at an angle of incidence, α , between zero and forty degrees. The direction of the wave is perpendicular to that of the surface, which is often called ‘face on loading’ (Cormie et al., 2009). Initially Rankine (1870) and Hugoniot (1884, 1887, 1889) developed a formula that would help in the determination of the reflected overpressure, p_r . Cormie et al. (2009) substituted the term for the peak static overpressure which led to Equation 1.15.

$$p_r = 2p_s \left[\frac{7p_o + 4p_s}{7p_o + p_s} \right] \quad [\text{Eq. 1.15}]$$

The above equation is a function of the peak incident overpressure, p_s , and the ambient pressure, p_o . Furthermore, it assumes that the air behaves as a real gas with a specific heat ratio of γ set to 1.4 ($\gamma = C_p/C_v$).

Contrary to regular reflections are Mach reflections. Mach reflections occur when the angle of incidence is greater than forty degrees up until ninety degrees. At a ninety degree angle there is no reflection; instead the surface is loaded by the incident overpressure which is also referred to as ‘side on pressure’ (Cormie et al., 2009). A Mach reflection tends to skim off the reflecting surfaces as opposed to bouncing off of them. A Mach

stem is formed when the reflected and incident shock fronts meet or combine at some point above the reflecting surface. Finally, the point at which the Mach stem is formed (i.e. the three shock waves intersect) is called the triple point (Cormie et al., 2009). Each of these concepts is illustrated in Figure 1.15.

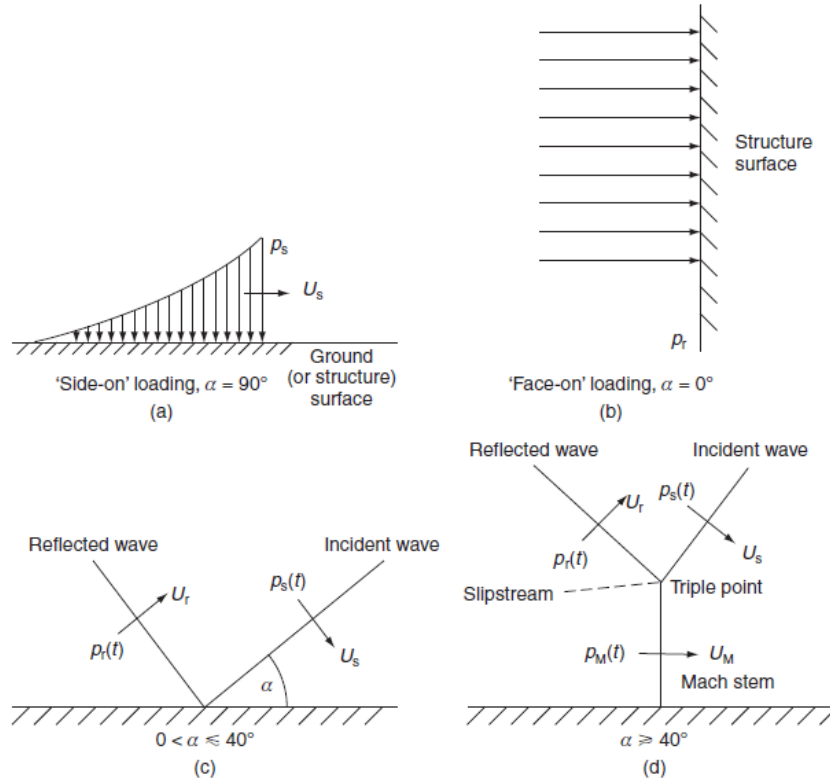


Figure 1.15: Types of Wave Reflections (Cormie et al., 2009)

1.7 Dynamic Analysis

1.7.1 Overview

The loading duration that is generated by a blast load is significantly smaller than what is experienced from an earthquake or a wind load (i.e. slow dynamic loads). For static (e.g. dead and live loads) and slower dynamic load cases (e.g. earthquake), it is common to use an equivalent static approach, however, in the case of a blast load, the inertial forces, and kinetic energy cannot be neglected in the analysis. Alternative methodologies, such as the use of dynamic equilibrium and the principle of conservation of energy, can be used. These equations are shown below as Equations 1.17, and 1.18, respectively.

$$F(t) - R(t) = ma \quad [\text{Eq. 1.17}]$$

$$WD = KE + SE \quad [\text{Eq. 1.18}]$$

Where $F(t)$ is the total external force as a function of time, $R(t)$ is the total internal force as a function of time, m is the mass, and a is the acceleration. WD is the work done by the system, and KE and SE is the kinetic, and strain energy, respectively.

The uncertainties associated with blast loads and the corresponding material properties justify the use of simplified modeling techniques such as single-degree-of-freedom (SDOF). SDOF has proven to accurately predict the response of various structural elements when subjected to blast loads (Viau, 2016; Lacroix, 2013; Ciornei 2012; Jacques, 2011). In cases where more than one type of failure occurs or when a localized failure mode occurs, the use of complex methods may be necessary.

1.7.2 Single Degree of Freedom (SDOF) Closed-Form Solution

The closed-form solution of a SDOF system is typically found by solving the differential equation consisting of a mass, damping, and stiffness component lumped at a single point. Equation 1.19, which is also known as the equation of motion, represents the equilibrium of such a system.

$$m\ddot{y} + c\dot{y} + ky = F(t) \quad [\text{Eq. 1.19}]$$

Where m is the mass, c is the damping coefficient, k is the stiffness, and $F(t)$ is the forcing function with respect to time. The remaining variables, $\ddot{y}$, $\dot{y}$, and y , represent the acceleration, velocity, and displacement of the system, respectively.

Figure 1.16 is a visual illustration of an idealized SDOF system which encompasses the aforementioned parameters.

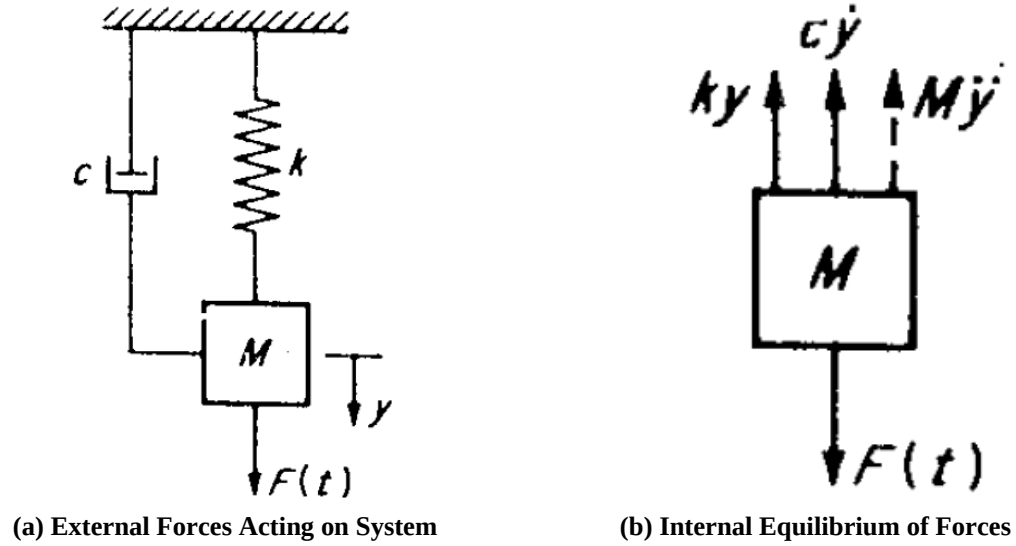


Figure 1.16: Typical Damped SDOF System

In the case of blast, the effects of damping can be neglected when the load duration is short, and when only the maximum dynamic response is of particular interest (Biggs, 1964). The equation of motion and displacement solution can thus be re-written in order to account for an un-damped free vibration system. These expressions set the damping coefficient and external forcing function equal to zero and are shown in Equations 1.20 and 1.21, respectively.

$$m\ddot{y} + ky = 0 \quad [\text{Eq. 1.20}]$$

$$y = \frac{\dot{y}_0}{w} \sin wt + y_0 \cos wt \quad [\text{Eq. 1.21}]$$

Where $\dot{y}_0$, and y_0 is the initial velocity, and displacement at time equals zero, respectively.

The natural frequency of vibration of a structure, denoted as w (rad/sec) can be found using Equation 1.22.

$$w = \sqrt{\frac{k}{m}} \quad [\text{Eq. 1.22}]$$

By assuming, that the forcing function can be idealized as a triangular loading function, the displacement solution for an external force with a time duration, t , leads to the following closed-form solution:

$$y = \frac{F_o}{k} (1 - \cos wt) \quad [\text{Eq. 1.23}]$$

Where F_o is the peak reflected pressure.

The dynamic displacement can be determined through the use of a dynamic load factor (DLF). The DLF is the ratio between the maximum dynamic displacement to the maximum static displacement, which results from the same statically applied force. This ratio is presented in Equation 1.24.

$$DLF = \frac{y}{y_{static}} \quad [\text{Eq. 1.24}]$$

1.7.3 Equivalent SDOF Analysis

Equation 1.23 assumes that the mass of the system is localized at a single point, however; in reality structural components have a distributed mass and stiffness. For this reason the structural component must be transformed into an equivalent SDOF system. A point of interest is generally selected (this can be a point where maximum deflection occurs), where mass, stiffness, and applied load is lumped (Biggs, 1964). This can be done by using transformation factors that are obtained by a pre-determined static deflected shape (Biggs, 1964). Equation 1.25 presents a modified equation of motion that accounts for the corresponding transformation factors.

$$K_M m \ddot{y}(t) + K_R R(y) = K_L A P_r \left(1 - \frac{t}{t_d}\right) \quad [\text{Eq. 1.25}]$$

Where $R(y)$ is the resistance, A is the loaded area, and P_r is the peak reflected pressure. The terms denoted as K_m , K_R , and K_L are the mass, resistance, and load transformation factors, respectively.

The transformation factors can be determined by equalling the potential energy, kinetic energy, and work done by the force. The mass transformation factor can be calculated using Equations 1.26 and 1.27 for a continuous, and a concentrated mass, respectively.

$$K_M = \frac{\int_0^L \bar{m}(x) \phi^2(x) dx}{\int_0^L \bar{m}(x) dx} \quad [\text{Eq. 1.26}]$$

$$K_M = \frac{\sum_{r=1}^i m_r \phi_r^2}{\sum_{r=1}^i m_r} \quad [\text{Eq. 1.27}]$$

Where $\bar{m}(x)$ is the distributed mass, $\phi(x)$ is the shape function that is based on the first mode deflected shape, m_r is the r^{th} mass, and i is the number of lumped masses.

The resistance factor, K_R , is equated to the load factor, K_L (USADD, 2008). Equations 1.28 and 1.29 present the formulas of the load factor for a distributed, and concentrated load, respectively.

$$K_L = \frac{F_e}{F} = \frac{\int_0^L f(x, t) \phi(x) dx}{\int_0^L f(x, t) dx} \quad [\text{Eq. 1.28}]$$

$$K_L = \frac{F_e}{F} = \frac{\sum_{r=1}^i F_r \phi_r}{\sum_{r=1}^i F_r} \quad [\text{Eq. 1.29}]$$

Where F_e is the equivalent load, F is the total load, $f(x, t)$ is the distributed load per unit length as a function of time, F_r is the r^{th} concentrated load, and i is the number of concentrated loads.

For simplicity, a load mass transformation factor denoted as K_{LM} , is introduced into the equivalent system which is defined as a ratio between the mass and load factor, as shown in Equation 1.30. The determination of a single transformation factor yields an equivalent SDOF system as presented in Equation 1.31.

$$K_{LM} = \frac{K_M}{K_L} \quad [\text{Eq. 1.30}]$$

$$K_{LM}m\ddot{y}(t) + R(y) = AP_R \left(1 - \frac{t}{t_d}\right) \quad [\text{Eq. 1.31}]$$

1.7.4 Resistance-Deflection Curve

The correlation that exists in a resistance-deflection curve is an essential part of the dynamic analysis of a structural element. In essence, this relationship depicts the resistance as a function of the displacement at the location of its corresponding ordinate. Resistance curves can be modeled as linear or non-linear; hence understanding the material behaviour becomes essential when choosing the appropriate resistance curve for the analysis.

1.7.5 Pressure-Impulse (P - I) Diagrams

One of the most effective tools in determining the performance level, and the overall response of any structural element, is a pressure-impulse diagram. This relationship is usually developed via a SDOF analysis method assuming certain failure modes and criteria; however it can also be generated from an actual blast event or experimental testing.

These diagrams allow the designer to select the appropriate level of damage using iso-damage curves. Figure 1.17 illustrates a typical pressure-impulse diagram which outlines the five response limits: superficial, moderate, and heavy damage, hazardous failure, and blowout. Figure 1.18 defines each of the above mentioned response limits in more detail.

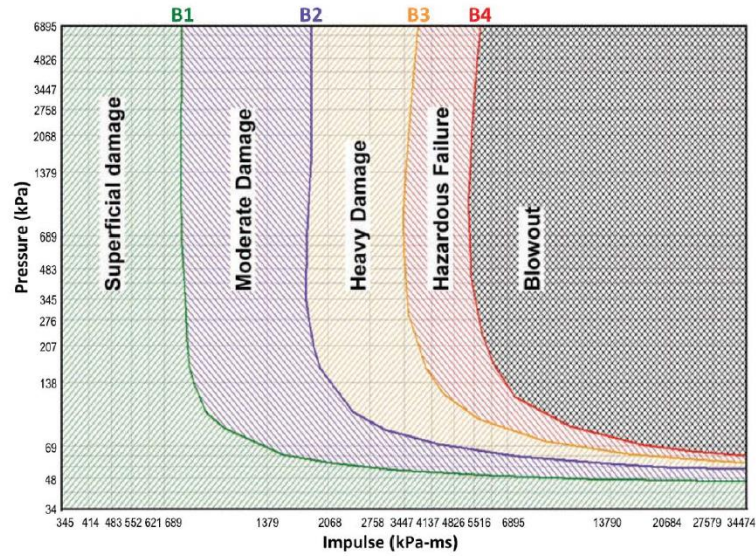


Figure 1.17: Typical P-I Diagram

Component damage level	Component damage level descriptions	Response limits
Blowout	Component is overwhelmed by blast load causing debris with significant velocities	Response greater than B4
Hazardous failure	Component has failed, and debris velocities range from insignificant	Response between B3 and B4
Heavy damage	Component has not failed, but it has significant permanent deflections causing it to be unrepairable	Response between B2 and B3
Moderate damage	Component has some permanent deflection. It is generally repairable, if necessary, although replacement may be more economical and aesthetic	Response between B1 and B2
Superficial damage	Component has no visible permanent damage	Response less than B1

Figure 1.18: Description of Response Limits

The CSA blast standard defines damage regions for the purpose of design by means of support conditions and ductility ratios. The ductility ratio, μ , is defined as the ratio between the maximum deflection and the maximum elastic deflection, as shown in Equation 1.32 (CSA, 2012).

$$\mu = \frac{x_{max}}{x_e} \quad [\text{Eq. 1.32}]$$

Where x_{max} is the maximum deformation, and x_e is the elastic deformation.

Typical response limits for wood structures are summarized in Table 1.2.

Table 1.2: Response Limits for Wood Structures

Element Type	B1		B2		B3		B4	
	μ_{max}	θ_{max}	μ_{max}	θ_{max}	μ_{max}	θ_{max}	μ_{max}	θ_{max}
Flexure	1	-	2	-	3	-	4	-
Combined flexure and compression	1	-	2	-	2	-	2	-
Axial compression	-	-	-	-	-	-	1	2.4°

*Where a dash (-) is shown, the corresponding parameter is not applicable as a response limit.

†Values shown are based on very limited test data; use specific test data if available.

‡Ductility ratio is based on axial deformation rather than lateral deformation.

There are three different types of loading regimes that can be found in a pressure-impulse diagram: impulsive, quasi-static, and dynamic. An impulsive regime is generally characterized by a load duration that is short relative to the response time of the system. A quasi-static regime has a load duration that is longer than the response time. In this case, due to the lengthened duration, the maximum deflection is achieved prior to the complete depletion of the load. This regime is highly dependent on the peak load and the structural stiffness. Finally, the dynamic regime is a transient regime that lies between the impulsive and quasi-static, as shown in Figure 1.19. It is more complex as it is greatly influenced by the loading history, response time, and load duration (Krauthammer, 2008).

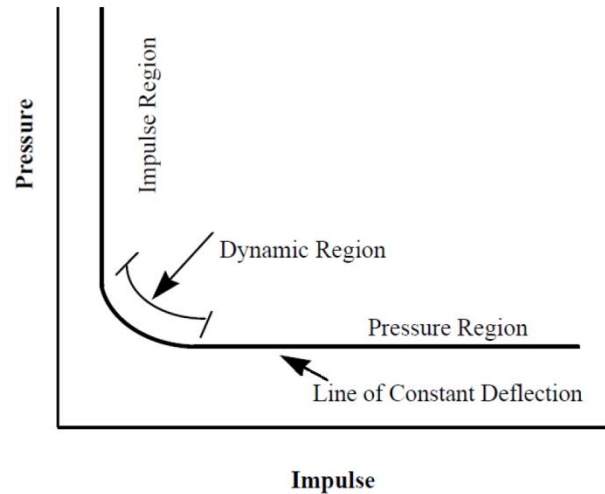


Figure 1.19: Pressure-Impulse Regimes

1.8 Research Objective

The overarching aim of this research project is to assess the effect of fibre-reinforced polymer and steel strapping on the performance of cross-laminated timber slabs subjected to blast loads. More specifically, the objectives are as follows:

1. Investigate the behaviour of externally reinforced CLT slabs when subjected to simulated blast loading;
2. Evaluate observed failure modes and document various levels of damage;
3. Compare research findings and suggest behavioural enhancements where necessary.

1.9 Scope

The aforementioned research objectives will be achieved by following the outlined steps:

- Conduct a detailed literature review on the behaviour of wood elements, particularly when subjected to blast loading (i.e. high strain rates), and investigate existing retrofitting techniques;
- Retrofit CLT slabs with industrial grade steel straps and with externally bonded fibre-reinforced polymer fabric sheets;
- Determine the initial stiffness and modulus of elasticity of reinforced CLT slab specimens prior to destructive testing for modeling purposes;

- Test a total of 17 reinforced CLT slabs dynamically, by using the University of Ottawa Shock Tube, and establish their level of damage;
- Perform component testing in order to establish the required material input properties;
- Discuss the research results and recommend potential design improvements.

1.10 Structure of Thesis

Chapter 1 provides a general introduction on the topic of blast loading, cross-laminated timber (CLT) slabs, steel strapping, fibre-reinforced polymer (FRP), and establishes the research needs, objectives, and scope of work.

Chapter 2 presents a detailed literature review with focus on studies that investigated the dynamic behaviour of wood and CLT structures, analytical modelling of such structures when exposed to blast loads, and various retrofitting techniques.

Chapter 3 describes the experimental methodology employed in the research project, a summary of the specimens and materials tested, and static and dynamic test setups.

Chapter 4 includes the experimental results, including the destructive dynamic tests, as well as material component coupon testing.

Chapter 5 discusses the experimental results and highlights key observations on the overall performance of reinforced CLT panels.

Chapter 6 provides a summary of the most significant research findings and proposes potential future research ideas and topics.

Appendix A presents the individual static test results for the cross-laminated timber coupons and corresponding failure modes.

Appendix B includes the stress-strain relationship alongside typical failure modes for the steel straps.

Appendix C includes the stress-strain relationship alongside typical failure modes for the FRP.

Appendix D provides the results for the dynamic destructive testing of the CLT slabs reinforced with steel straps.

Appendix E provides the results for the dynamic destructive testing of the CLT slabs reinforced with FRP.

CHAPTER 2 – Literature Review

2.1 General

An explosion or blast event, whether it is a deliberate (e.g. World Trade Center, 2001) or accidental (e.g. Lac-Mégantic rail disaster, 2014) attack, leads to destructive and catastrophic consequences. Many researchers and structural engineers are focusing on investigating the material behaviour and structural response of a system when subjected to a blast load or live explosion. This has led to the development of blast standards allowing engineers to design structures against blast loading and to mitigate potential damage and loss of human life by following set provisions. In addition, response limits have been developed which provide the necessary guidance needed to undertake blast analysis. The subsequent sections provide the fundamental background knowledge on the properties of wood as a material. In addition to a detailed literature review of previous studies of wood subjected to high strain rates, as well as EWP's reinforced with steel strapping and GFRP.

2.2 Wood as a Construction Material

2.2.1 Properties of Wood

Wood is a raw, renewable source that continues to play an essential part in the construction industry and when considering sustainable design due to its availability, ease of use, durability, and relatively low cost. Typical civil engineering applications of wood include, but are not limited to: floors, roofs, walls, trusses, piles, bridges, and buildings. Wood is also a natural material in which case the physical properties are highly dependent on the type of tree and the growth rate (Mamlouk et al., 2011).

Wood is an anisotropic material with different properties in each of its orthogonal directions. Wood consists of the following 3 axes: longitudinal (parallel to the grain), radial (cross the growth rings), and tangential (tangent to the growth rings), as illustrated in Figure 2.1.

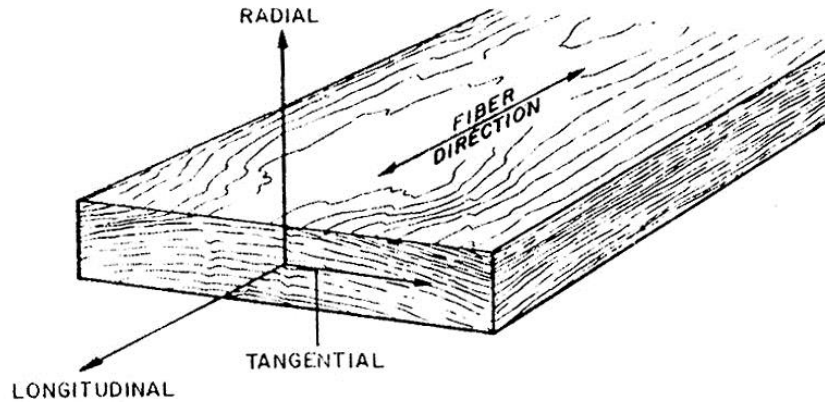


Figure 2.1: Fibre Direction (ARC261: Building Materials & Construction I)

Furthermore, wood can be categorized as a viscoelastic material such that the mechanical properties are time-dependent and are affected by load duration, rate of loading, fatigue, and creep. The *Standard Practice for Establishing Clear Wood Strength Values* (ASTM, 2011) standard discusses the relation between the bending stress as a function of the load duration, where it is noted that wood has the ability to sustain higher loads (i.e. increase in strength) for short durations. The current literature refers to ‘short durations’ in terms of seconds. However, it is becoming imperative to fully understand the behaviour of wood when subjected to a blast load, in which case the time duration is in the range of milliseconds. Wood is also a heterogeneous material, due to the defects (e.g. knots, waness, etc.) that naturally occur in the material which in turn directly affects its mechanical properties. As a result, clear wood behaves differently than the lumber and timber members that are used in the construction industry.

Flexural bending failures are identified based on the appearance of the fractured surface. These fractured surfaces can be classified as either fibrous or brash indicating a failure that contains splinters, and an abrupt failure, respectively. The failure mode for a piece of lumber with defects will be generally governed by the failure on the tension face. Figure 2.2 depicts the different potential forms of flexural failure.

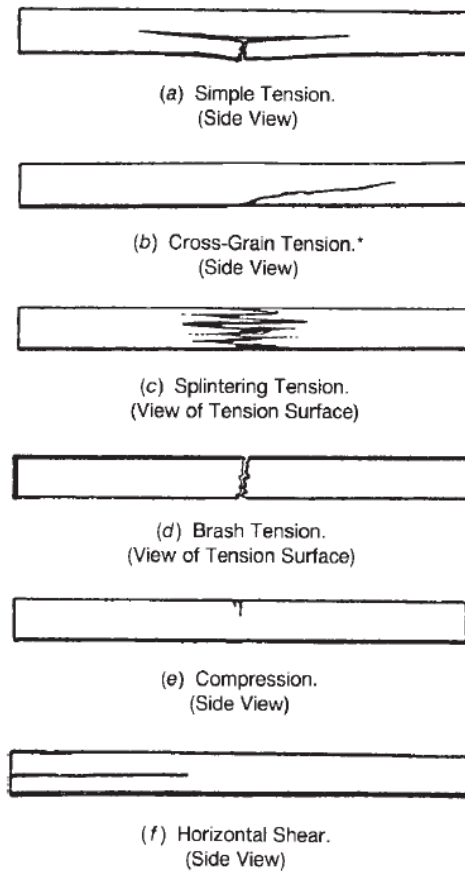


Figure 2.2: Typical Failure Modes in Static Bending (ASTM, 2014)

2.2.2 Flexural Resistance of Wood

A moment-curvature relationship of a wooden element can be derived by using the tensile and compressive strengths of the member (parallel to the grain). Bazan (1980) developed the stress-strain relationship for clear wood specimens. A decade later, Buchanan (1990) modified the stress-strain relationship for tension and compression in order to account for in-grade lumber, as shown in Figure 2.3.

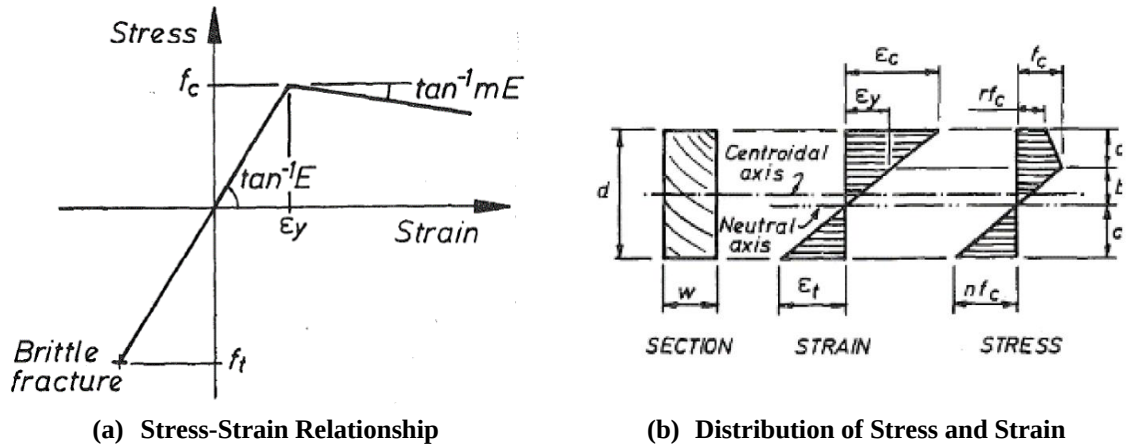


Figure 2.3: Model Proposed by Buchanan, 1990 (Buchanan, 1990)

The tensile behaviour of a wood member can be described as a linear elastic relation until failure, which occurs at a value of f_t with corresponding strain, ϵ_t . The compressive behaviour is best represented by a bi-linear relation where the first linear portion is defined by a compression yielding stress denoted f_c , with its corresponding strain, ϵ_{cy} after which there is a linear descending branch until failure. Buchanan (1990) defined the slope of this descending branch to be a constant ratio, denoted m , of the modulus of elasticity, E .

2.3 Behaviour of Wood under Impact Loading

Early research has focused primarily on impact loading tests conducted on small scale and clear wood specimens (i.e. free of defects). Liska (1950) conducted a study, where the effects of rapid loading on the compressive and flexural strength of wood members were investigated. A total of two softwoods and two hardwoods were tested. The behaviour of the wood samples were tested within a time duration in the range of 0.3 – 750 s. The author reported an increase in compressive and flexural strength between 10 – 40 % as the time to maximum load decreased. There was no significant effect found on the modulus of elasticity.

Nadeau and Bennett (1982) performed impact tests on defect-free Douglas-Fir wood specimens in order to investigate the effects of rate of loading (i.e. strain rate) and the duration of load on the strength wood. Approximately half of the specimens were

intentionally notched on the tension face as to promote flexural failure and to study the effects of initial cracks. The strain rate range for the notched and unnotched specimens were $3.2\text{E}^{-7} - 8.4\text{E}^{-3} \text{ s}^{-1}$, and $3.2\text{E}^{-7} - 6.5\text{E}^{-2} \text{ s}^{-1}$, respectively. The author reported that the unnotched specimens experienced a strength increase varying between 10 – 30 %. Although Nadeau and Bennett (1982) reported rate of loading effects in the stronger specimens, these effects were absent in the weaker elements due to the increase in initial micro-cracks. The study concluded that there are two specific regions in which the response to impact load differs. First, high stressing rate region, where the strength is independent of the strain rate. Second, low stressing rate region, where the strength is heavily influenced by the crack growth. Mindess and Madsen (1986) conducted a similar test and also reported a deficient increase in the notched members.

Sukontasukkul et al. (2000) performed impact loading tests on parallel strand lumber (PSL). Due to the nature of PSL, there were essentially two different types of failure modes: flexural failure on the tension face and shear (i.e. delamination) failure at the interface between the laminations. The study reported an average dynamic strength increase of 30 % for an average corresponding strain rate of approximately 3.1 s^{-1} . Moreover, there was a decrease in the apparent bending strength due to the impact of the hammer resulting in the crushing of wood fibres.

2.4 Behaviour of Wood under Simulated and Actual Blast Loads

The amount of research and literature related to the effects of wood subjected to blast loads are very limited and in some cases inaccessible. This section describes the key findings from these studies and identifies gaps that currently exist in research and that ultimately led to the need for the current study.

2.4.1 Coated Structural Lumber and Laminated Strand Lumber

During the development of the *Vehicle Bomb Mitigation Guide* (USAF, 2006), testing was conducted on a series of coated structural lumber (CSL), and laminated strand lumber (LSL) members, as these were the primary materials used to retrofit huts in South-East Asia. The primary objective of the study was to reduce debris, increase ductility, and ultimately to facilitate repairs following a blast event. Full-scale tests were conducted on the huts, where it was observed that the failure occurred in the wall and the

roof-to-wall interface. Uncoated LSL was selected as a potential alternative to reinforce and strengthen the CSL systems at primary locations (Syron, 2010). High strain rate compression tests were conducted on the LSL specimens with a strain rate range varying between $1\text{E}^{-5} - 1\text{E}^{-3} \text{ s}^{-1}$. The study reported an overall dynamic strength factor ranging between 1.31 – 1.37 and 1.16 – 1.41 for the longitudinal direction, and transverse direction, respectively.

2.4.2 Shock Tube Testing of Individual Wood Studs

Jacques et al. (2013) performed a study which focused on the behaviour of individual visually graded lumber studs subjected to simulated blast loads. The research program consisted of blast waves simulated by means of the shock tube. The primary focus of the study was to determine a dynamic increase factor on the material properties; in addition to validating a single degree-of-freedom (SDOF) model. For a strain rate range of $0.1 - 1.0 \text{ s}^{-1}$, an average dynamic increase factor for the modulus of rupture (MOR) and the modulus of elasticity (MOE) was found to be 1.41, and 1.14, respectively. The dynamic effects on the average of the MOE were found to be inconclusive due to the small sample size and the large variability in the results. The results produced by the SDOF model proved that this simple tool could be utilized to adequately predict the behaviour of wood studs. The authors were able to conclude that a linear-elastic resistance curve was able to capture the behaviour of individual wood studs subjected to high strain rates.

2.4.3 Shock Tube Testing of Light-Frame Stud Walls

Lacroix and Doudak (2014) conducted a series of full-scale static and dynamic tests on wall systems which ultimately led to the determination of a dynamic increase factor (DIF) for the MOR and MOE. Dynamic testing was performed at the University of Ottawa by means of a shock tube apparatus which has the ability to simulate blast waves. Strain rates ranging between $0.12 - 0.55 \text{ s}^{-1}$ yielded an average DIF for the modulus of rupture and modulus of elasticity of 1.40, and 1.18, respectively. The boundary conditions utilized in the research program were simply supported in order to avoid the influence of the connection detail; in addition to simplifying the modeling process. A material predictive model was developed which was able to accurately predict the dynamic wall response. The ductility, generated from the system, emerged from the mechanical

fasteners that were used to attach the sheathing panels to the framing elements. The authors presented an empirical formula to calculate the dynamic increase factor for the modulus of rupture of wood studs. This equation, valid for strain rates ranging between $1.67\text{E}^{-1} - 1.65\text{E}^3 \text{ s}^{-1}$, is a function of strain rate and was developed by comparing different studies in literature and data obtained from impact loading tests.

2.5 Retrofit Options for Light-Frame Wood Stud Walls

Wood is an inherently light material allowing it to become an advantageous construction material due its unique earthquake-resisting property. Nonetheless, wood is also a brittle material which may lead to poor performance under blast loads. This section includes research studies aimed at enhancing the capacity and/or ductility of wood elements.

2.5.1 Sheathing Deficiencies and Debris

Bogosian and Avanesian (2004) conducted a study on structural and non-structural elements subjected to blast loads, utilizing anthropomorphic test devices (ATDs), in order to numerically quantify the relationship between the blast pressure-impulse combinations, and blunt trauma injury levels. The study showed that the debris, generated from a blast load, was the main cause of injury and/or death. The authors observed that wood stud walls could potentially cause moderate to high injury levels based on the assumption that the wood stud walls would fail at the top and bottom connections resulting in debris striking the test dummies in the chest. On the contrary, the potential of injury caused by a window panel was found to be lower, where it was observed that the window panel tended to strike the test dummies at a head level. The findings of this study accentuated the need for further research on the behaviour of connection details and the role they play in affecting the overall behaviour of structural elements.

The South-East Asia (SEA) huts, as discussed in Section 2.4.1, were found to be blast deficient. The United States Air Force produced a report which described the test results along with potential retrofit options (USAF, 2006). The SEA huts included window and door openings and were built of dimensional lumber sheathed with plywood. The results showed that it was not the structural damage or component failure that would cause injuries to occupants, but rather structural collapse. Although the correlation between the structural failure and injuries was minuscule, flying debris still presented itself hazardous.

The authors proposed that debris hazard computational analysis is highly dependent on the precise location of the occupants. The study suggested eliminating windows, ensuring that doors open outwards, reinforcing with additional sheets of plywood on the interior and exterior of the walls, and providing additional dimensional lumber at the bottom of the walls and bottom truss chords in order to prevent connection failure.

Lacroix et al. (2013) performed dynamic tests on wood stud walls with varying pressure-impulse combinations. The research program consisted of a total of ten tests in which walls were built with either an OSB (11 mm thick) or plywood (18.5 mm) panel as the sheathing. The authors noticed that the walls built with plywood performed significantly better than those with OSB. It was observed, at higher pressures, that the OSB walls experienced ripping in the sheathing, flying debris, and often tore before the studs could fail which was detrimental to the overall system capacity and performance. Conversely, the walls fabricated with thicker plywood did not produce any debris and the damage was limited to minor cracks on the surface of the sheathing resulting in the wood studs to achieve their ultimate flexural strength.

Lacroix et al. (2013) also investigated potential retrofit options of the wood studs and sheathing. Two different type of retrofit options were explored, both of which consisted of 18.5 mm (3/4 in.) thick plywood. The first configuration was targeted to new construction where the plywood was installed directly onto the wood studs. The second configuration focused on reinforcing pre-existing light-frame wood stud walls built with OSB. In this case a sheet of plywood was fastened on top of the existing OSB panel. The reinforced walls were found to be stiffer than those with only one layer of OSB or plywood, and a reduction of nail pull-out was observed. The reinforced specimen's experienced, on average, a stud displacement of 18 % less than that observed in walls built only of plywood. Overall, an increase in capacity and stiffness, compared to typical construction detailing, was achieved.

Viau et al. (2016) investigated the behaviour of typical and retrofitted light-frame wood stud walls in the region corresponding to hazardous-blowout damage levels. The typical construction walls (built with OSB) showed that premature sheathing failure can occur prior to attaining full flexural response of the wood studs resulting in sheathing debris. In

order to mitigate this undesirable failure, walls were reinforced with thicker sheathing and screws were used instead of nails to fasten the panel to the studs resulting in a decrease of debris. A welded wire mesh (WWM) was also investigated. This system was used to strengthen the sheathing and to act as a catcher system. Both alternatives achieved the desired outcome. Not only was the catcher system able to retain debris but it allowed the sheathing to fail while minimal wood stud damage was observed. Moreover, an analytical model was developed which successfully validated the experimental results based on the inputs generated from static material component testing.

2.5.2 Boundary Connection Deficiencies

Sorensen and McGill (2011) performed a study that investigated the failure in the connections, when wood stud walls were exposed to blast loading. It was found that connection failure creates a significant amount of flying debris where the whole wall can become airborne and possibly strike occupants resulting in injuries and fatalities. The study found that maintaining the connection between the wall and floors is critical in limiting the risk of injury due to flying debris as well as reducing the probability of progressive collapse (U.S.A.C.E, 2008a).

Lloyd and Jacques (2011) developed a prototype of a stud retrofit system that would mitigate deficiencies in the connections for light-frame wood stud walls. The retrofit configuration consisted of a hanger fastener and tensioning rods which provided connectivity between the end of the wall studs and blocking or rim-joist members. The bracing system allowed for an effective load path and assisted in the distribution of the load into the diaphragm. The authors tested typical and retrofitted walls as part of their research program. The typical walls failed at the connections, as expected and experienced fastener pull-out. The retrofitted walls outperformed the latter and no connection failure was observed.

2.6 Retrofit Options for Engineered Wood Products

Lacroix (2017) conducted an experimental program that focused on establishing the dynamic characteristics and properties of glue-laminated timber (glulam) beams with and without the effect of FRP reinforcement. A total of twenty-six beams were tested with varying retrofit configurations (e.g. simple tension reinforcement, U-shaped tension

reinforcement, and partial and full length confinement), types of fabric (e.g. CFRP, and GFRP), and fabric orientation (e.g. unidirectional, and bi-directional). Static and dynamic tests of the retrofitted beams were conducted in order to compare the overall behaviour. The study also investigated the effect of multi-directional fabrics on the post-peak behaviour of the beams. Lacroix (2017) observed, in general, that the addition of FRP to the glulam beams resulted in a significant increase in the resistance and the post-peak behaviour. Configurations with a simple tension lamina experienced premature de-bonding at the wood-FRP interface. The addition of a confinement layer had the ability to prevent the de-bonding failure and improve the overall behaviour. Furthermore, a U-shaped tension lamina in combination with a partial or full length confinement layer was capable of delaying the de-bonding well beyond the peak load. The addition of FRP was also found to contribute to limiting the crack developed which in turn led to an increase in the tensile failure strain by a factor of 1.21. Bi-directional confinement, fabric in the $0/90^\circ$ and $\pm 45^\circ$ direction, with or without unidirectional reinforcement proved to enhance the post-peak behaviour achieving ductility ratios between 1.57 – 3.61. When compared to the unretrofitted specimens, retrofitted beams reached maximum increases in deflection, stiffness, and resistance by a factor of 1.24, 1.56, and 1.93, respectively.

2.7 Behaviour and Modeling of CLT Panels under Simulated Blast Loads

Cross-laminated timber (CLT) is an engineered wood product (EWP) that has become a viable construction material in North America. To date, several studies have investigated the material properties of CLT specifically relating to rolling shear (Flores et al., 2016; Li and Lam, 2016; Zhou et al., 2014). Although the behaviour of CLT when subjected to out-of-plane loads (Sikora et al., 2016; Jacquier, 2015; Steiger and Gülzow, 2009), in-plane loads (Yasumura et al., 2016; Popovski and Gavric, 2016; Ashtari et al., 2014; Ceccotti et al., 2006), and fire (Fragiacomo et al., 2013; Frangi et al., 2009; Frangi et al., 2004) has been explored, there is a minimal amount of research that focuses on the behaviour of CLT when subjected to short-duration loads such as blast loading. This section outlines recent studies that have been conducted at the University of Ottawa.

2.7.1 Cross-Laminated Timber Slabs with Simply Supported Conditions

Poulin et al. (2017) conducted a study that investigated the out-of-plane behaviour of CLT panels under static and dynamic loading in order to determine failure mechanisms and a representative dynamic increase factor (DIF) that can be used for the analysis and design of CLT panels. A total of eighteen CLT slabs were tested. The CLT panels were fabricated with spruce-pine-fir (SPF) and had a thickness of 105 mm and 175 mm, corresponding to a 3-ply, and a 5-ply, respectively. Each slab had a constant width and length of 445 mm, and 2500 mm, respectively. Both static and dynamic tests had a similar loading pattern (i.e. four point bending) and boundary conditions (i.e. simply supported). The authors determined an average dynamic increase factor on the resistance to be 1.28, with no increase in the dynamic stiffness. Flexural failure occurring at the mid-span of the slab was the most prominent failure mode observed; however rolling shear failure was detected prior to or at ultimate failure of the panel. Rolling shear developed within the transverse layers of the slab which occurred at about the same time as the flexural failure, which occurred at the finger joints on the tension face. The specimens investigated in the current study were taken from the same batch of CLT panels investigated in the study by Poulin et al. (2017). To facilitate direct comparison with the findings of the study by Poulin et al. (2017), Table 2.1 summarizes the dynamic results for both 3-ply and 5-ply elements.

An average resistance curve, which accounted for the effect of high strain rates and post peak residual behaviour, was developed and validated, using SDOF analysis, with the experimental dynamic test results, as illustrated in Figure 2.4. The average resistance curve for the 5-ply experiences an initial drop which represents the loss of the bottom longitudinal and transversal layers. As a result, the subsequent behaviour becomes very similar to that observed for a 3-ply. Once the deflection reached a value equal to the elastic limit, the ultimate resistance of a 5-ply specimen had decreased to 20 % of its peak strength. Similarly, the 3-ply post peak resistance curve was modelled as a sudden drop of 20 % of its ultimate resistance. The post peak residual displacements were determined based on the dynamic and static experimental findings, where it was observed that the ductility ratio did not exceed value of 2.5.

Table 2.1: Dynamic Test Results (Poulin et al., 2017)

Test Name	P_R	I_R	R_{max}	Δ_{Rmax}	Δ_{max}	μ	t_{f-max}	$\dot{\epsilon}$	K_{static}	Failure Mode
	<i>kPa</i>	<i>kPa – ms</i>	<i>kN</i>	<i>mm</i>	<i>mm</i>	-	<i>ms</i>	<i>s⁻¹</i>	<i>kN/mm</i>	
CLT3-4.1	11.6	90.0		16.1	16.1		26.8	5.00E ⁻⁰²	1.59	Elastic
CLT3-4.2	38.7	379.3	82.1	37.7	105.2	2.8	48.8	1.61E ⁻⁰¹	1.59	Rolling Shear
CLT3-5.1	48.2	442.3	80.7	40.5	88.6	2.2	25.6	1.44E ⁻⁰¹	2.10	Flexural
CLT3-6.1	52.6	613.7	88.3	45.8	143.7	3.1	35.0	1.52E ⁻⁰¹	1.88	Flexural
CLT3-7.1	6.5	79.1	-	-	11.8	-	30.6	2.86E ⁻⁰²	1.77	Elastic
CLT3-7.2	52.8	439.9	-	-	119.2	-	47.2	1.58E ⁻⁰¹	1.77	Rolling Shear
CLT3-8.1	5.8	59.4	-	-	10.1	-	28.0	2.16E ⁻⁰²	1.66	Elastic
CLT3-8.2	49.0	467.7	-	-	89.5	-	26.0	1.79E ⁻⁰¹	1.66	Rolling Shear, then Flexural
CLT3-9.1	12.2	131.2	-	-	18.7	-	31.2	5.22E ⁻⁰²	1.76	Elastic
CLT3-9.2	30.9	322.4	-	-	58.4	-	38.8	1.32E ⁻⁰¹	1.76	Rolling Shear
CLT3-10.1	9.1	92.6	-	-	12.3	-	26.2	3.61E ⁻⁰²	2.10	Elastic
CLT3-10.2	34.0	374.4	-	-	99.4	-	79.6	1.58E ⁻⁰¹	2.10	Flexural
CLT3-11.1	9.2	106.5	-	-	11.7	-	26.5	3.52E ⁻⁰²	2.09	Elastic
CLT3-11.2	32.9	345.4	-	-	72.5	-	35.2	1.19E ⁻⁰¹	2.09	Flexural
Average			83.7	41.3	112.5	2.7				
COV			0.04	0.08	0.21	0.14				
CLT5-4.1	53.7	610.4	176.7	32.4	80.7	2.5	34.2	2.05E ⁻⁰¹	5.64	Flexural
CLT5-5.1	58.6	690.7	171.7	29.6	58.2	2.0	26.2	1.80E ⁻⁰¹	5.14	Rolling Shear
CLT5-6.1	7.2	81.7	-	-	6.5	-	22.4	2.56E ⁻⁰²	5.53	Elastic
CLT5-6.2	55.8	602.4	-	-	77.9	-	31.4	2.13E ⁻⁰¹	5.53	Flexural
CLT5-7.1	7.3	81.8	-	-	6.4	-	22.8	3.28E ⁻⁰²	5.07	Elastic
CLT5-7.2	51.2	481.2	-	-	37.0	-	24.2	2.12E ⁻⁰¹	5.07	Flexural
Average			174.2	31.0	69.5	2.3				
COV			0.01	0.04	0.16	0.11				

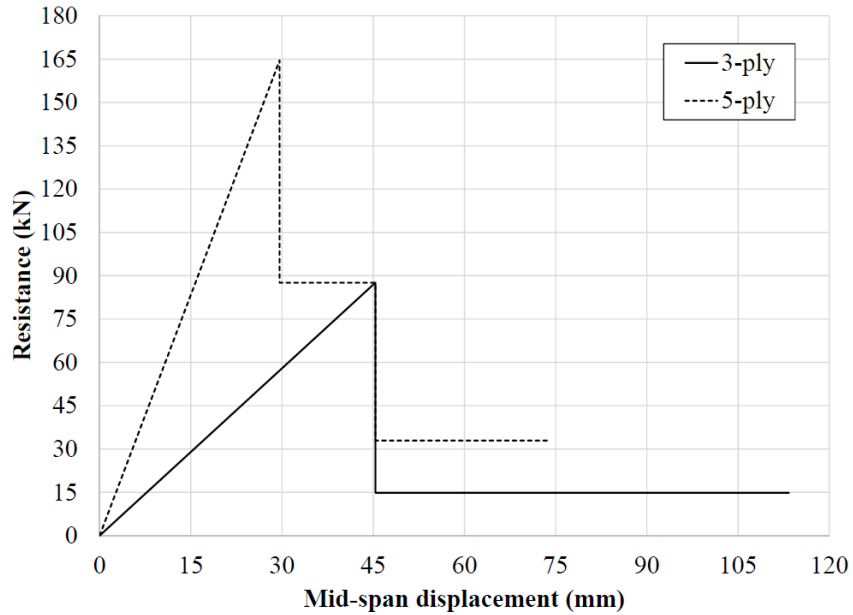


Figure 2.4: Dynamic Resistance Curves for Flexural Failure (Poulin et al., 2017)

2.7.2 Cross-Laminated Timber Slabs with Realistic Boundary Conditions

The preceding study was limited to idealized boundary conditions which resulted in fundamental knowledge relating to the behaviour of CLT slabs when subjected to high strain rates. Nevertheless, in order to fully understand the overall behaviour of CLT structures it is imperative to consider and include the connections that exist between the walls and the floor systems.

Côté (2017) conducted a study that focused on investigating of the effects of ‘realistic’ boundary conditions on the behaviour of cross-laminated timbers walls when subjected to simulated out-of-plane blast loads. A total of thirteen 5-ply CLT slabs were tested as part of this research project. Similar to the current study, the material that was used by Côté (2017) were a subset taken from the same lot as those investigated by Poulin et al. (2017). The length of the slabs was reduced to 2082 mm for the following two reasons. First, two panel pieces were cut from each end of a previously tested slab and were fastened to a shortened CLT slab by means of a series of connection configurations investigated in this study. In essence, this arrangement simulated a connection between a wall and the floors above and below. That being said, this allowed for the clear span of the test specimen to fit within the test setup (i.e. due to the limitations of the shock tube). Second, an area was

created which allowed for the test setup to clamp the top and bottom blocks. By doing so, the connection details would be the ones absorbing the impact. The author utilized a total of five different Heco-Topix screws which were used directly or in combination with two different types of steel angles. Configurations with seismic detailing were examined in order to assess the adequacy and performance of a CLT structure subjected to blast loads.

Côté (2017) observed that, in general, the connection details have a significant effect on the behaviour of CLT structures. A typical platform construction detail with screws connecting the block to the slab resulted in poor performance and experienced brittle failure. Conversely, a bearing type connection displayed significantly improved behaviour. Yielding was observed in the fasteners and/or the angle brackets due to the amount of energy the connection was able to dissipate. This led to a substantially reduced amount of energy left to cause damage or failure in the slab. The author concluded that the observed failure modes developed when connections were considered differed significantly from those observed when idealized boundary conditions were tested. A SDOF model, as presented by Poulin et al. (2017), proved to be an inadequate tool to accurately predict the behaviour of a CLT slab with varying connection details. Complex modelling techniques were deemed necessary in order to properly describe the behaviour.

2.8 Previous Research on Steel Straps as a Confinement Technique

The concept of using steel straps, pre-stressing strands, or Retrobelt as an alternative retrofitting technique in structural applications is a relatively recent concept. Research has primarily focused on strengthening bridge columns with deficiencies (e.g. Saatcioglu and Yalcin, 2003; Saatcioglu et al., 2002). This technique consists of external pre-stressing strands, along the transverse direction, which applies lateral confinement pressure which improves confinement, ductility, and shear capacity. This section outlines studies that have been conducted throughout the years. Given that this innovative and economical option is still at its infancy, the programs in the succeeding sections relate solemnly to concrete.

2.8.1 Seismic Testing on Concrete Elements

Yalcin (1997) experimentally investigated the effectiveness of external pre-stressing as a retrofit alternative on concrete elements simulating pre-1971 bridge columns. A total of

seven, seismically shear deficient, concrete columns with circular and square cross-sections were tested dynamically. The columns were reinforced with a total of 7-wire strands or steel straps depending on the desired configuration. In the case of a square section, steel spreader frames and raiser discs were utilized to transform the lateral pressure to an equivalent uniform distribution. The testing phase consisted of loading each column with a constant axial load which was equivalent to 15 % of the axial capacity under pure compression. The author observed improved behaviour from columns that were retrofitted with strands pre-stressed at 25 % of the ultimate strength of pre-stressing strands spaced at 150 mm. In conclusion, an increase in lateral drift capacity of 1 % in shear critical columns and up to 5 % in retrofitted columns was achieved.

Mes (1999) tested a total of six full-scale reinforced concrete columns. The research program was organized such that three pairs of columns were investigated. These test specimens were subjected to seismic loading with a constant axial load. Unretrofitted flexural dominant concrete columns developed up to 2 % of lateral drift. Conversely, retrofitted columns exhibited ductile behaviour and the overall lateral drift capacity was improved by 4 – 5 %. In conclusion, the proposed technique proved to be an effective alternative in increasing the concrete confinement and improving the inelastic behaviour.

Beauséjour (2000) conducted seismic testing on columns with splice deficiencies within the plastic hinge region. This research program consisted of a total of six columns, half of which served as control columns and thus creating three groups. The first group emphasized a long shear span within a circular column. The second group had a relatively long shear span within a square column. The columns within the third group were built with a short shear span in order to amplify the effects of the shear stress reversals. The simulated seismic transverse load and constant axial load was generated with the aid of three MTS hydraulic actuators. The control columns displayed an abrupt brittle failure mode and the drift capacity was limited to 1 %. Initial pre-stressing (i.e. 25 % of the ultimate strength of pre-stressing strands spaced at 150 mm) improved the lateral drift capacity from 1 – 2 %. By increasing the lateral pressure to 50 % of the ultimate strength of pre-stressing strands and by placing the strands at every 100 mm within the splice region, the lateral drift capacity increased to 4 %. Thus, the author concluded that

Retrobelt technology was an effective retrofit strategy that improves inelastic deformability and energy dissipation capacity.

Yarandi (2007) conducted a study that focused on investigating the effectiveness of external pre-stressing rectangular bridge columns. Once again, a total of six, seismically deficient, columns were designed to reflect pre-1970s design requirements. The author observed that the retrofit mechanism caused a shift from brittle to ductile behaviour in shear dominant columns. Likewise, the employed methodology demonstrated a significant improvement in seismic behaviour in flexural dominant columns. It was also found that adding external pre-stressing transverse confinement, shear failure was minimized, ductile behaviour became evident, and an increase in energy dissipation capacity was observed.

Sabri (2013) conducted a study that focused on the development of a seismic retrofit methodology that can be implemented and applied to buildings and bridge columns. High strength packaging steel straps served as external reinforcement and transverse pre-stressing. This research study investigated three full-scale seismically deficient concrete columns each with a different cross-sectional shape (i.e. circular, square, and rectangular). In order to prove the effectiveness of the Retrobelt system, the columns were designed such that the section was dominant in shear or flexure. The steel straps provided active and passive confinement pressures which in turn improved the ductility, flexural response, and shear capacity of the concrete columns. The author observed that reinforced circular columns can achieve a minimum of 6 % lateral drift with minimal strength decay in moment capacity. Similarly, square columns were capable of attaining a lateral drift of 6 %. The pre-stressing mitigated premature shear failure which resulted in an increase in shear resistance and altering the member behaviour (i.e. brittle to ductile failure). Rectangular columns, when loaded to 50 % of their concentric column capacity, resulted in a lateral drift of only 3.5 %. In this particular case, there was an insufficient amount of steel which avoided further improvements in ductility.

2.8.2 Blast simulation on Concrete Elements

Lloyd (2015) conducted a comprehensive research study that involved different retrofit systems in order to mitigate the damage to structural elements (i.e. column) when subject

to blast loads. The retrofits were intended to be effective in strengthening concrete columns while also increasing and improving the ductility. Of a total of thirteen columns, that were retrofitted and tested dynamically in the shock tube, only one was constructed with pre-stressed transverse confinement (i.e. steel straps). This technique resulted in lower maximum and residual displacements (i.e. post-ultimate displacement resistance) when compared to the control column. This retrofit option prevented the tension reinforcement in the concrete column from rupturing and the compression reinforcement from buckling. It also limited the amount of concrete crushing and spalling. The author concluded that pre-stressing steel jacketing proved to be an effective alternative that significantly increased the capacity of the column (i.e. mass and ductility).

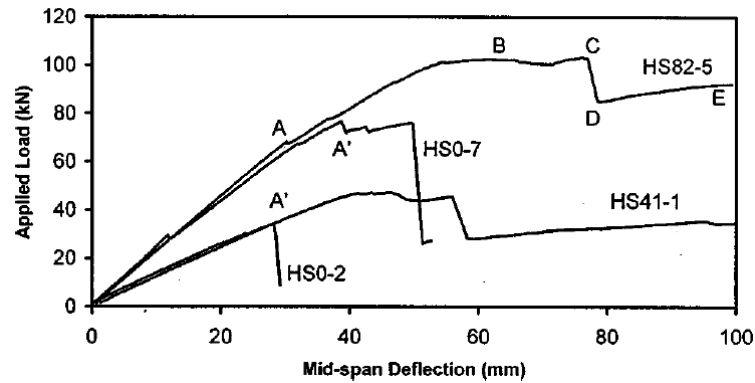
2.9 Previous Research on FRP as a Flexural Strengthening Technique

Over the years, it has become crucial to develop retrofitting techniques that address the performance of existing structures in order to meet recent design code requirements. The strengthening of wood, by means of fibre-reinforced polymers, dates back to the 1940s and has since attracted many researchers within the wood community. Initially wood based products (e.g. lumber, joists, plywood, and laminated wood beams) were reinforced with metal reinforcement such as: steel bars, steel plates, and aluminum plates. However, these retrofit techniques have not yet been successfully commercialized (Bulleit, 1984). Given an immense library of literature, this section will focus on the studies that are directed towards the techniques utilized for flexural reinforcement of wood members. The subsequent sections highlight the use of glass fibre-reinforced polymer (GFRP) bars and fabric, and carbon fibre-reinforced polymer (CFRP) fabric used as a reinforcing technique. A typical structural application, in which FRP and wood are used in combination, is in the rehabilitation of existing timber bridges (CSA, 2014; Hay et al., 2006; Buell and Saadatmanesh, 2005).

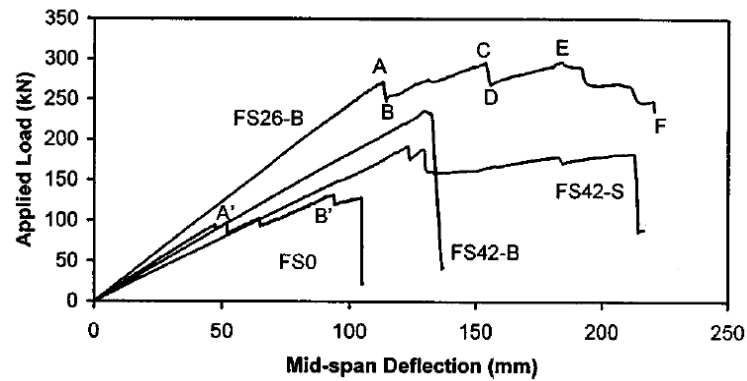
2.9.1 FRP Bars as Strengthening Alternatives

Gentile et al. (2002) conducted a research program to study, experimentally and analytically, the flexural behaviour of twenty-six creosote-treated sawn Douglas fir timber beams strengthened with GFRP bars. The experimental program consisted of twenty-two half-scale beams which were cut from large new timber bridge stingers, and

four full-scale timber bridge stringers which were obtained from a dismantled bridge that was in service for over 30 years. All specimens were tested to failure and subjected to monotonic load in a four-point bending configuration. The GFRP reinforcement utilized in the half-scale elements was made of 80 % (by weight) unidirectional E-glass and 20 % thermoset epoxy resin. The GFRP reinforcement selected for the full-scale members had 70 % unidirectional E-glass and 30 % recycled PET (polyester) resin. The author observed similar behaviour in terms of load deflection (as shown in Figure 2.7), strain distribution, ultimate strength, and failure mode between half-scale and full-scale timber beams. An average increase in ultimate tensile strain of 64 % was reported for the reinforced beams. The overall flexural behaviour shifted from a brittle tensile failure to a ductile compressive failure mode. No delamination or de-bonding of the GFRP bars was observed. In fact the reinforcing bars were able to control cracks and confine local ruptures. An analytical model was developed and successfully predicted the ultimate bending strength of GFRP reinforced timber beams.



(a) Typical Half-Scale Beams



(b) Typical Full-Scale Beams

Figure 2.5: Experimental Load Deflection Curves (Gentile et al., 2002)

2.9.2 FRP Fabrics as Strengthening Alternatives

Plevris and Triantafillou (1992) focused on developing an elementary understanding of the behaviour of FRP-reinforced wood systems. The study examined the effects of externally bonded composite materials on the ductility, stiffness, and failure modes when subjected to flexural bending. Furthermore, the authors developed a computer model with the ability to predict the behaviour of clear wood beams reinforced with FRP sheets bonded to the tension face. The CFRP sheets were made of unidirectional continuous fibres bonded with an epoxy resin which created a lightweight and high mechanical performance system. The authors observed a nonlinear response in which the compression of the wood governed the overall failure mode; followed by rupture of the reinforcement which led to the commencement of tensile failure. In conclusion, Plevris and Triantafillou (1992) reported a critical value of 3 % reinforcement, after which the addition of FRP on the members bending moment capacity levels off. Moreover, it was

found that the critical FRP area fraction was dependent on the following three parameters: the ratio of the tensile yield strain to compressive strain, the ratio of FRP ultimate strain to compressive yield strain, and the ratio of the elastic modulus of elasticity of FRP to that of wood.

Dagher et al. (1996) investigated the effect of FRP reinforcement on low grade eastern hemlock. The experimental program consisted of a total of twelve beams, nine of which were reinforced with FRP while the remaining were used as control beams. The specimens were divided into the following three quality categories, based on their modulus of elasticity (MOE): low, medium, and high. The low quality reinforced beams experienced an increase in resistance and stiffness of 43 %, and 31 %, respectively, when compared to the control group. The medium quality beams consisted of two different types of FRP which the author labelled FRP1 and FRP2. An increase of flexural capacity and stiffness of 51 % and 32 %, and 33 % and 37 % was reported for beams reinforced with FRP1, and FRP2, respectively. The high quality beams experienced an increase in strength and stiffness of values ranging between 24 – 27 %, and 25 – 29 %, respectively. In all three categories ductile behaviour, over the unreinforced counterparts, was observed. In addition to the experimental work, a nonlinear numerical model was developed. The model constituted of two programs; the first determined the moment-curvature relationship of the FRP-glulam section. The second utilized the generated moment-curvature relationship to determine the load-deflection curve for any given loaded beam. The model proved to be successful in determining the overall performance of glulam beams. The authors concluded that the addition of 1 – 3 % of FRP reinforcement resulted in substantial improvements in the mechanical properties of the members. This was due to, in large part, to the initiation of the compressive strength of the wood.

Johns and Lacroix (2000) presented a study that addressed the use of CFRP and GFRP fabric sheets to reinforce sawn timber sections measuring 38 m x 89 m. The testing samples consisted of twenty-five pairs of beams for a total of one hundred fifty specimens. Two groups were reinforced with two successive tension layers of CFRP sheets. The third group was reinforced with two successive layers of U-shaped GFRP

sheets. The test setup was designed such that three-point loading could be simulated over a clear span of 1500 mm. The experimental results reported an overall increase in strength and stiffness, when compared to the theoretical ones generated by simple models (i.e. transformed-section analysis and design code values). In most cases local deficiencies in the wood served as weak points and led to the initiation of cracks which ultimately led to failure. The authors observed that the addition of FRP as tensile reinforcement mitigated the openings of cracks, confined local ruptures, and bridged the defects. In conclusion, the weaker specimens (i.e. commercial wood) experienced an increase of strength ranging between 40 – 100 %.

Dempsey and Scott (2006) performed a study which investigated the effect of mechanically fastened FRP strips on fifteen pretreated No. 2 Southern Pine wood specimens. A total of fifteen glulam beams were subjected to four-point bending with a 1.6 m span. Three beams were tested without composite materials (i.e. control group) while the remaining twelve were reinforced with GFRP or a hybrid FRP (HFRP) with fastener spacing's of 50.8 mm, 76.2 mm, or 150.4 mm. The GFRP strips consisted of a multi-directional fabric with fibres in the 0°, 90°, and $\pm 45^\circ$ direction along the longitudinal axis. Contrarily, the HFRP strips consisted of two layers of continuous strand mats and a combination of E-glass rovings and carbon tow. The fasteners were mechanically installed which provided sufficient connection strength between the FRP and the wood specimen, without damaging the reinforcing material. The authors reported an increase ranging between 3 – 22 % and 8 – 19 %, for the ultimate moment, and stiffness, respectively for GFRP strips. Similarly, the HFRP strips resulted in an increase in moment and stiffness of 20 – 51 %, and 15 – 54 %, respectively, when compared to the control group. The addition of the strengthening strips allowed the specimens to withstand increasing loads and larger deflections which in turn prevented complete fracture in the beam. No delamination or fractured FRP was observed. The glulam beams experienced crushing of the wood on the compression face. It was concluded that mechanically fastened strips increased the flexural properties of glulam beams and that fastener spacing is inversely proportional to the moment capacity, stiffness, and ductility ratio.

Fiorelli and Dias (2013) conducted an experimental study of pinewood beams reinforced with unidirectional woven GFRP and CFRP. A reinforcement ratio (i.e. volume of FRP to volume of wood) of 1 % and 0.4 % for GFRP, and CFRP, respectively was implemented. An analytical model, to determine the ultimate bending moment, was derived by Triantafillou and Deskovic (1992), and Lindyberg (2000). The model assumed an elastic plastic wood behaviour and an elastic FRP and wood behaviour, when subjected to compressive, and tensile loads, respectively. The results obtained, experimentally and theoretically were compared and the error was reported to be acceptable. The authors reported a significant increase of 15 – 30 % in stiffness for both types of FRP reinforcement.

CHAPTER 3 – Experimental Program

3.1 General

This section consists of the construction, preparation, and testing of seventeen retrofitted CLT panel specimens subjected to four point bending. A detailed description of the methodology and procedures employed throughout the testing phases of this study are presented. In addition to the full scale testing program, material component tests were conducted in order to determine the properties of CLT, steel straps, and FRP. Since two different retrofit options were investigated, experimental testing was conducted in two distinct phases. The first phase consisted of CLT panels retrofitted with post-tensioned steel straps, while the second phase focused on the specimens reinforced with externally bonded FRP fabric.

3.2 Description of Unretrofitted Cross-Laminated Timber Panels

The CLT panels tested in this study constituted part of a larger research investigation involving tests on the material properties of CLT (Poulin et al., 2017) and the contribution of connections (Côté, 2017) under simulated blast loading. The material, for all three studies, was provided by Nordic Structures. The panels were delivered at the same time and all specimens were conditioned in a similar manner in the structures laboratory at the University of Ottawa. As a result direct comparisons, where appropriate, can be made with the preceding studies. In total, 17 CLT specimens were investigated as part of the current project, 12 of which were undamaged specimens and the remaining 5 were tested (i.e. damaged) prior to this study (Poulin et al., 2017) and presented various levels of damage. These specimens were used to investigate the effectiveness of the proposed retrofit techniques on structural members that had already been subjected to some level of loading.

The specimens consisted of 3, 5, and 7-ply CLT panels with total panel thicknesses of 105 mm, 175 mm, and 245 mm, respectively. The lumber laminations of each panel were made of Spruce-Pine-Fir (S-P-F) and measured approximately 35 mm in thickness and 80 mm in width. Furthermore, the laminations utilized in the longitudinal direction (i.e. major direction) consisted of 1950Fb machine stress rated lumber, while No.3/Stud

lumber was used in the transversal direction (i.e. minor direction). Each specimen measured 2500 mm in length, with a respective width of 445 mm for the 3 and 5-ply, and 300 mm for the 7-ply panels. Table 3.1 summarizes the quantity and condition of all test specimens while Figure 3.1 illustrates a typical CLT panel cross section depicting the aforementioned dimensions in millimeters.

Table 3.1: Condition of CLT Panels

Condition	No. of Ply's	Panel Thickness (mm)	Panel Width (mm)	Quantity
Undamaged (UD)	3	105	445	6
	5	175	445	6
Damaged (D)	3	105	445	1
	5	175	445	2
	7	245	300	2

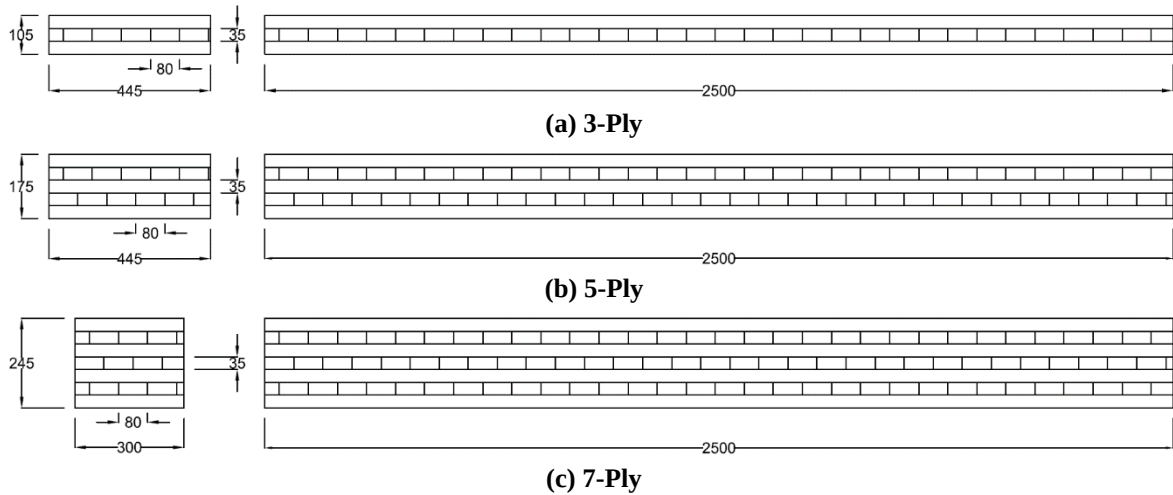


Figure 3.1: Cross-Laminated Timber Panels

3.3 Description of the Reinforced Specimens

CLT panels were retrofitted in order to improve their flexural behaviour and enhance the failure characteristics of the material. For the testing phase involving steel straps (Phase 1), strap orientation (i.e. transversal or longitudinal), spacing along the width and/or height, and the order in which the straps were installed were investigated. Furthermore, the effects of unidirectional and multidirectional FRP fabric (Phase 2) applied in various configurations (e.g. U-shape tension reinforcement vs. full confinement) were examined.

This section outlines the pertinent details of the reinforcing materials and includes a thorough description of each test matrix developed for both phases of testing.

3.3.1 Specimen Nomenclature

It is essential in understanding the correct panel labeling as these test names will be frequently referred to from here on in. Both phases of experimental testing included CLT panels of different sizes. Each panel has been assigned an ID based on the number of CLT layers followed by an identification number. For example: CLT5-1 refers to the first 5-ply panel tested in this study. As tabulated in Table 3.1, there are 6 undamaged 5-ply's and are thus labelled CLT5-1 to CLT5-6. This trend is also consistent with the 3-ply's. Previously tested and damaged panels have the letter "D" following the number of CLT layers. For example: CLT3-D1 indicates that the panel has been previously tested.

3.3.2 Phase 1: Steel Straps

The steel straps were supplied by Signode Packaging Systems. This type of system is comprised of two fundamental elements. A Magnus type steel strap, Figure 3.2 (a), was selected for this project due to its high strength, ductility, and shock resistance. Furthermore, this particular strap exhibits a hard and smooth surface (i.e. low friction) resulting in a uniform tensile distribution around corners. The strap measured 19 mm (3/4") and 0.74 mm (0.029") in width, and thickness, respectively, which consequently resulted in a cross sectional area of 14.06 mm². The strap had a painted and waxed finish to provide corrosion resistance.

A push type seal, as shown in Figure 3.2 (b), was selected due to its suitability to accompany the Magnus steel strap. The seal measured 38.1 mm (1.5") in length and was slightly wider than the steel strap. The fastening of the seal and steel strap was done using a pneumatic tool which creates a reverse notch joint as displayed in Figure 3.2 (c). The strength of the system is derived from the mechanical interlock that is generated between the seal and the strapping.

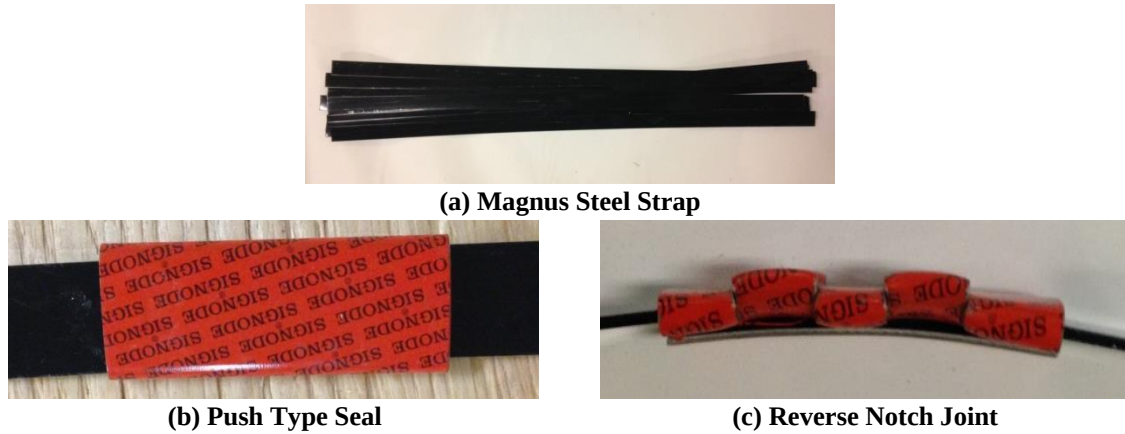


Figure 3.2: Steel Strapping System

The test specimens consisted of CLT panels with steel straps installed in either the horizontal (i.e. transversal) or vertical (i.e. longitudinal) direction. Table 3.2 summarizes the different variations of straps that were employed in this testing phase, where H and V denote horizontally, and vertically applied straps, respectively. The configuration column also provides the sequence in which the straps were installed. The number in the square parenthesis represents the number of steel straps in that particular direction. The column labelled strap location indicates where along the clear span of the panel the horizontal straps are located. For example: CLT5-1 consisted of 24 horizontal straps along the entire panel span, on top of which 5 vertical straps were installed in the longitudinal direction. The strap location reads ‘entire span’ which means that the horizontal straps are spaced out such that they cover the entire clear span of the panel. In cases where the horizontal straps do not cover the entire span of the CLT panel; the straps occupy only a portion of the panel length. Reference to the top and bottom one-thirds is provided in the table below.

The spacing of the horizontal and vertical straps was based on the number of transverse, and longitudinal layers, respectively. For example: a horizontal strap installed on every transverse layer along the clear span, resulted in a centre to centre spacing of approximately 90 mm (3-1/2”). Similarly, in the case of the vertical straps, each CLT panel consisted of 5 longitudinal layers which would have also resulted in the aforementioned spacing. CLT5-4 was slightly different in comparison to the others as it

consisted of horizontal straps installed at every other transverse layer along the clear span resulting in an approximate spacing of 180 mm (7") centre to centre. Figure 3.3 shows examples of the test configurations.

Table 3.2: Phase 1 Test Matrix

Specimen	Test Name	Condition	Configuration	Strap Location
1	CLT3-1	UD	H [24]	entire span
2	CLT3-2	UD	H [16]	top and bottom thirds
3	CLT5-1	UD	H [24] + V [5]	entire span
4	CLT5-2	UD	V [5] + H [25]	entire span
5	CLT5-3	UD	V [5] + H [17]	top and bottom thirds
6	CLT5-4	UD	V [5] + H [13]	entire span
7	CLT5-5	UD	V [9]* + H [8]	entire span
8	CLT5-D1	D	H [24]	entire span
9	CLT5-D2	D	V [5] + H [24]	entire span
10	CLT7-D1	D	H [24]	entire span

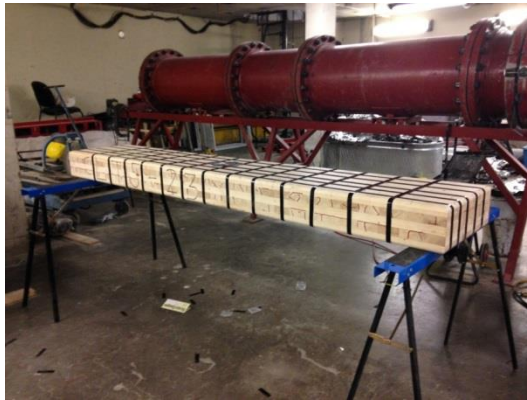
*Modified version of typical configuration



(a) Specimen No. 4



(b) Specimen No. 5



(c) Specimen No. 6



(d) Specimen No. 8

Figure 3.3: Typical Test Configurations

CLT5-5 was a modified version of a typical vertical strap configuration as it was the only specimen where the longitudinal straps were affixed on the tension face. This specific test consisted of a steel connection system located at the top and bottom of the specimen. The connection consisted of a steel circular hollow section, labelled SP1 (Figure 3.4), with an inner and outer diameter of 43.47 mm, and 50.13 mm, respectively, welded to a steel plate which measured 443 mm in length and 101.60 mm in width resulting in an approximate cross sectional area of $45,000 \text{ mm}^2$. The connection plate was fastened to the tension face of the panel with six type II screws and fourteen type III screws totalling to 20 Heco-Topix screws. Additional details regarding the type of screws, dimensions, and the total number utilized are summarized in Table 3.3 and illustrated in Figure 3.7.

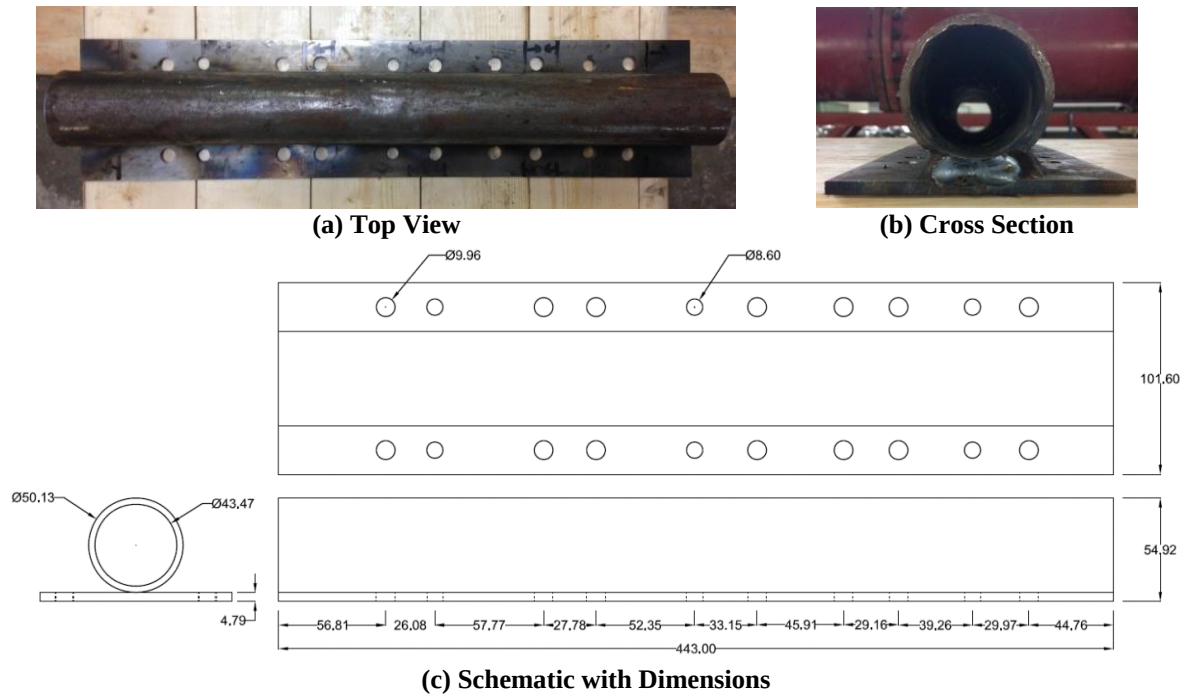


Figure 3.4: Steel Connection, SP1

A steel plate, labelled SP2, measuring 447 mm in length and 25.86 mm in width resulting in an approximate cross sectional area of $12,000 \text{ mm}^2$ was used to bring the steel straps closer to the specimens (Figure 3.5). This steel plate required 6 Heco-Topix screws in order to firmly secure the plate on the tension face of the panel. Figure 3.5 shows a schematic drawing including dimensions expressed in millimetres. Although the succeeding steel plates were shown separately, both of these pieces were installed adjacent to one another in order to create the steel connection system used to fasten the steel straps on the tension face of the specimen, as presented in Figure 3.6.

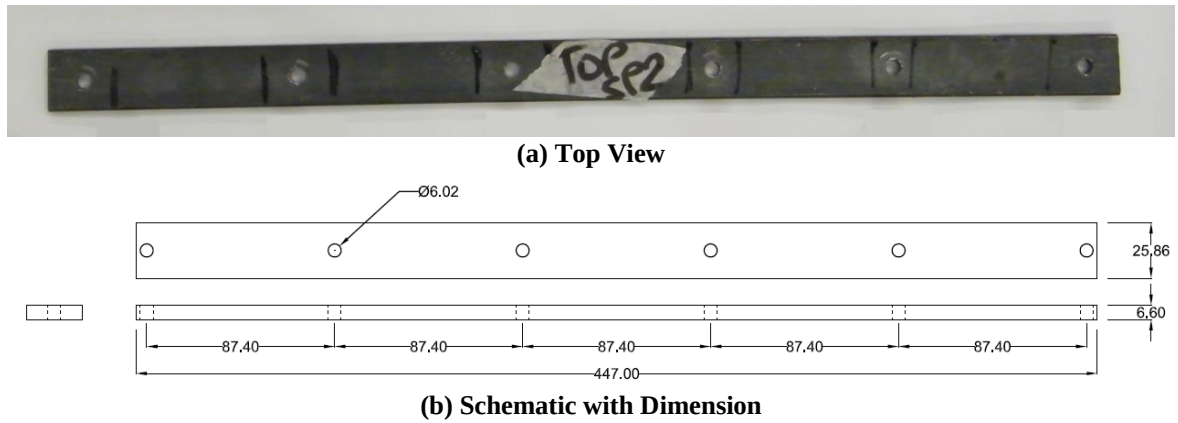


Figure 3.5: Steel Connection, SP2



Figure 3.6: Steel Connection System

Table 3.3: Heco-Topix Screw Details

Type	Screw Description	Length (mm)	Threaded Length (mm)	Thread Diameter (mm)	Quantity
I	6 x 100 Heco-Topix	100	70	6.3	12
II	8 x 120 Heco-Topix	120	70	8.3	12
III	10 x 120 Heco-Topix	120	120	10.3	28



(a) Type I



(b) Type II



(c) Type III

Figure 3.7: Heco-Topix Screws

As outlined in Table 3.2, CLT5-5 consisted of a total of nine vertical straps. Figure 3.8 illustrates this unique configuration.



(a) Isometric View



(b) Connection System



(c) Top View

Figure 3.8: Specimen No. 7 – CLT5-5

3.3.3 Phase 2: Fibre-Reinforced Polymer

The FRP fabric was supplied by Composites Canada. Four different types of glass fibre-reinforced polymer fabrics were used in this phase: uniaxial $[0^\circ]$, biaxial $[0/90^\circ$ and $\pm 45^\circ]$, and triaxial $[0^\circ, \pm 45^\circ]$. The fabric measured 1.27 m in width. In order to adhere the GFRP fabrics to the CLT panels, a two-part epoxy resin (MVS-410) and hardener (MVS-464) was utilized. This particular system penetrates wood surfaces quite well, in addition to being high heat and moisture resistance.

Three different reinforcement configurations were investigated, as summarized in Table 3.4. The first configuration consisted of a partial (U-shaped) fabric along the panel length which covered an approximate cross sectional perimeter of 540 mm. The second configuration (CM) was a confinement retrofit with emphasis on restraining the mid-span of the specimens. In this case, the layers of GFRP created an overlap at the mid-span doubling the fabric thickness. Moreover, as each sheet of fabric was installed, splicing occurred on the compression face. For instance: one layer of GFRP would consist of a total of two sheets overlapping in the middle. The top and bottom thirds would consist of one GFRP layer thick of fabric, the middle portion would result in two times a layer thickness, and finally the fabric thickness would quadruple at the precise location of the splice on the compression face. The third configuration (CL) was a full length confinement retrofit option that wrapped around the entire panel in one single piece of GFRP. Unlike CM, splicing occurred only on the compression face as each sheet was installed.

Table 3.4 lists the test configurations investigated in the current study. The layup of the GFRP fabric is expressed in terms of the number of layers, configuration type, followed by the fabric orientation in the square parenthesis. Similar to the steel strap test matrix, the order of the configuration layup follows the same order in which the fabric was installed. Figures 3.9 – 3.11 illustrate the various configuration layups. All units in the succeeding figures are expressed in millimetres.

Table 3.4: Phase 2 Test Matrix

Specimen	Test Name	Condition	Configuration	GFRP Location
11	CLT3-3	UD	2 U [0°]	clear span
12	CLT3-4	UD	2 CM [0/90°]	clear span
13	CLT3-5	UD	2 CM [$\pm 45^\circ$]	clear span
14	CLT3-6	UD	CM [0/90°] + CM [$\pm 45^\circ$]	clear span
15	CLT5-6	UD	2 CL [0, $\pm 45^\circ$]	clear span
16	CLT3-D1	D	2 CL [0, $\pm 45^\circ$]	clear span
17	CLT7-D2	D	2 CM [0/90°] + 2 CM [$\pm 45^\circ$]	clear span

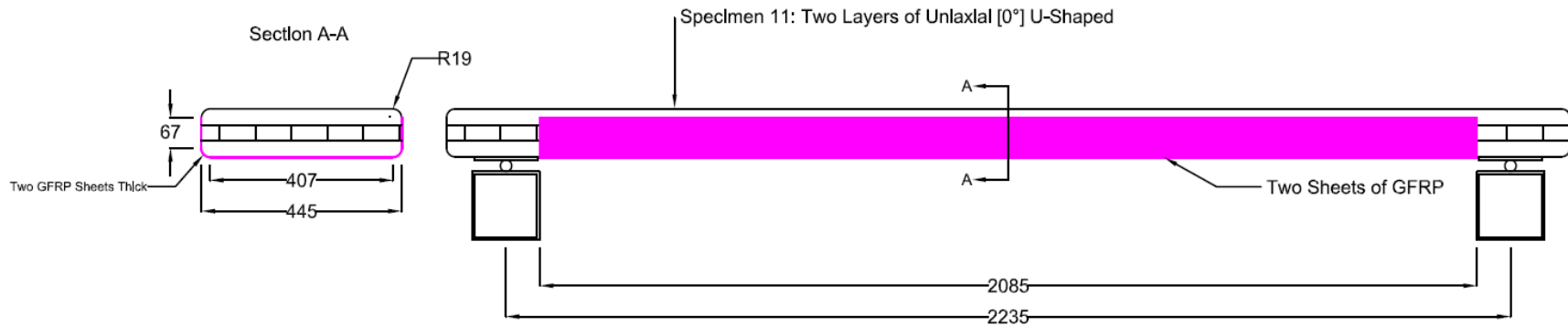


Figure 3.9: Specimen No. 11

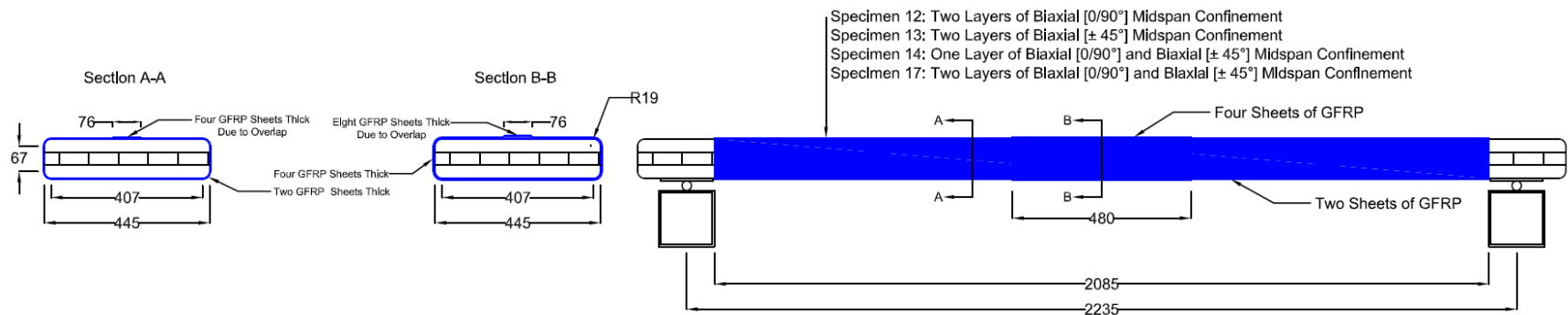


Figure 3.10: Specimen No. 12, 13, 14, and 17

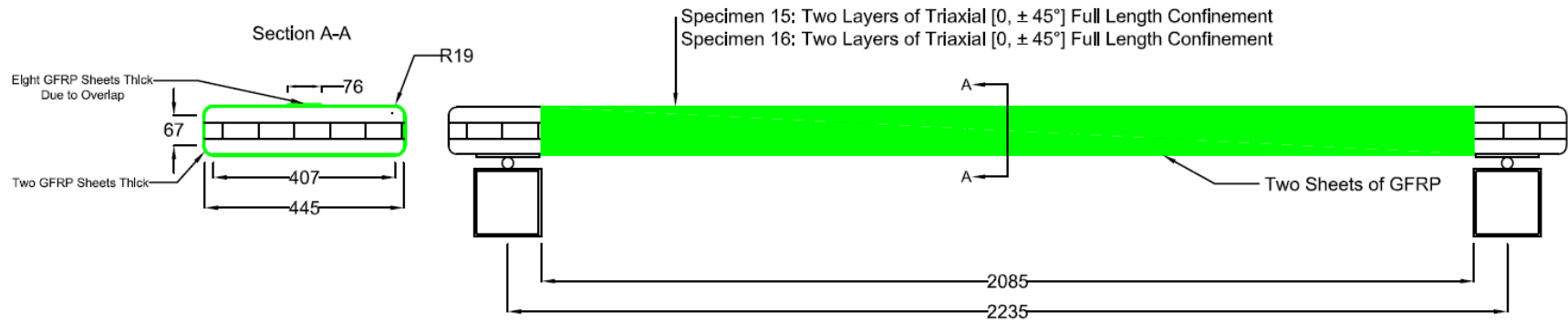


Figure 3.11: Specimen No. 15 and 16

3.4 Construction and Preparation of Specimens

The preparation of the CLT panels was executed in two stages. Prior to the installation of either retrofitting technique, the corners of each panel were routed to a radius of 19.05 mm (3/4") as shown in Figure 3.12. This was done to reduce localized stress concentrations in these particular areas. All previously damaged specimens were also routed, sanded, and smoothed in order to remove any loose splinters.



Figure 3.12: Routing Process

3.4.2 Installation of Steel Straps

The procedure undertaken to install the steel straps is outlined below:

1. Steel strapping was extracted from the dispenser (Figure 3.13) after which a seal was slotted through with the smooth side facing upwards.
2. The strap was then passed over top of the panel, wrapped around the panel, and fed through the seal. Approximately 50.8 mm (2") of the strap was measured and bent inwards.
3. The sag in the strapping was removed by manually tightening the strap to the panel and the strap and seal were aligned with the centre of either the transverse or longitudinal layer. A pneumatic tensioner (Figure 3.14 (a)) was used to pre-stress the strap. Once the strap was fully tensioned, a strapping tool (Figure 3.14 (b)) was used to fasten the seal to the steel strap, where two notches were formed along the length of the seal.

4. Steps 1 – 4 were repeated for the remaining straps in this phase of the experimentation.

The application of this retrofitting technique is shown in Figure 3.15.



Figure 3.13: Steel Strapping Dispenser



(a) Pneumatic Tensioner PN2-114



(b) RCNS2-34 Strapping Tool

Figure 3.14: Pneumatic Strapping Tools

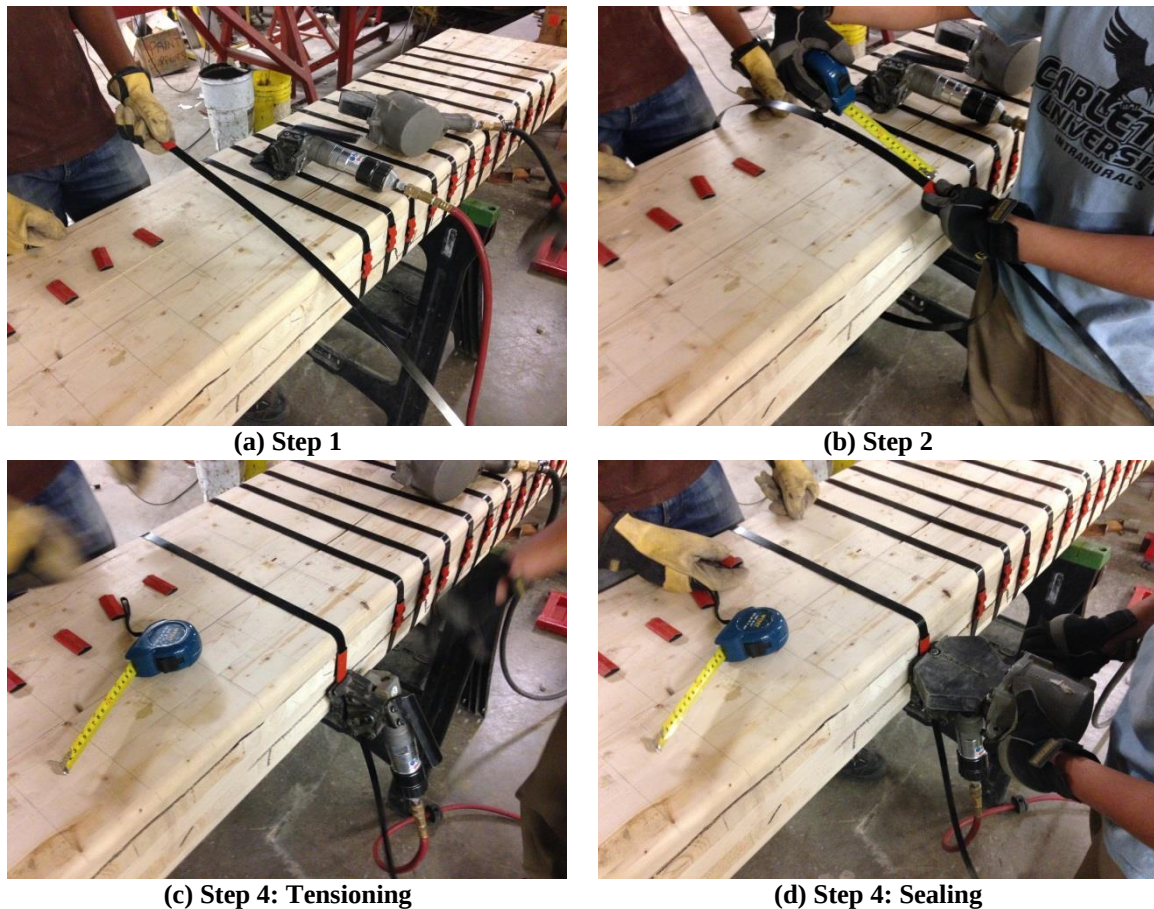


Figure 3.15: Steel Strapping Tensioning Technique Process

3.4.3 Installation of Externally Bonded GFRP Fabric

The application of externally bonded GFRP laminations was performed using the prescribed wet lay-up technique provided by the manufacturer. A detailed summary of said procedure is outlined below:

1. The GFRP fabric was cut to the correct dimensions, as per the developed test matrix and placed on a hard surface covered in plastic.
2. Prior to the application, the wooden surface was cleaned in order to remove and eliminate any dust or dirt.
3. The epoxy resin system was prepared according the manufacturers recommendation and was applied directly to both sides of a sheet with a paint roller (Figure 3.16 (a)). Once the laminate was saturated, a squidgy (Figure 3.16 (b)) was used to remove any excess epoxy.

4. A layer of epoxy was applied to the surface of the panel. After which the saturated laminate was placed directly on the wet surface.
5. Ribbed steel rollers (Figure 3.16 (c)) were used to remove air voids and excess epoxy, in addition to smoothing said surface.
6. Steps 3 – 8 were repeated for additional GFRP layers.
7. Finished specimens were left to cure for a total of 7 days.

The application of this retrofitting technique is shown in Figure 3.17.



(a) Paint Roller



(b) Squidgy



(c) Ribbed Steel Roller

Figure 3.16: GFRP Application Tools



(a) Step 3



(b) Step 4



(c) Step 5

Figure 3.17: GFRP Application Process

3.5 Material Component Testing

Material component testing was conducted in order to determine the stress-strain relationship of the three main constituents involved in this research project. A Universal Testing Machine (UTM) was used to perform the tests, where the applied force and displacement was recorded.

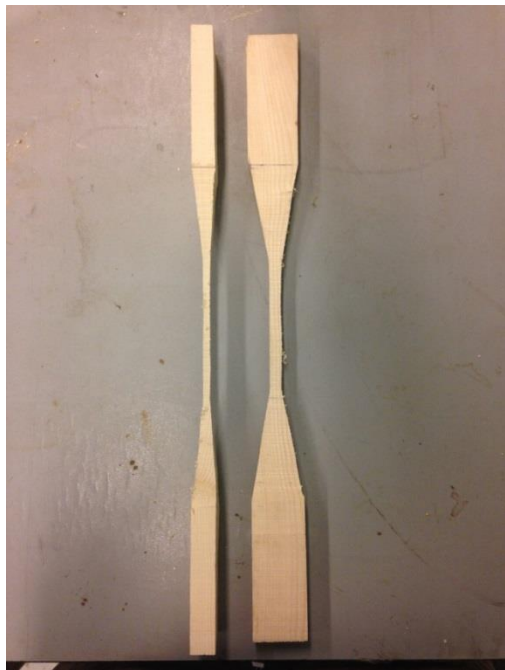
3.5.1 CLT Coupons

Longitudinal laminations from the cross-laminated timber panels were tested in compression and tension parallel to grain in accordance with the *Standard Test Methods for Small Clear Specimens of Timber* (ASTM D143). A total of five laminations, measuring 35 mm, 82 mm, and 200 mm in thickness, width, and in length, respectively were used for the compression coupons. Each coupon was subjected to a compressive force and an average loading rate of 1 mm/min. A typical compression coupon test is demonstrated in Figure 3.18.

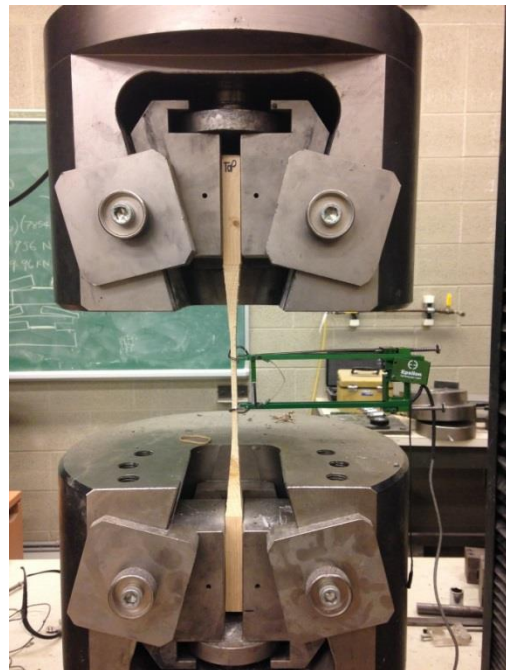
A total of five tension coupons, with length of 101.6 mm and cross sectional area of 45 mm², were subjected to an applied tensile force at a loading rate of 1.5 mm/min. The forming of the final coupon shape can be seen in Figure 19 (a) and a typical tension coupon test is illustrated in Figure 19 (b).



Figure 3.18: Compression Coupon Test



(a) Typical Shape



(b) Test Setup

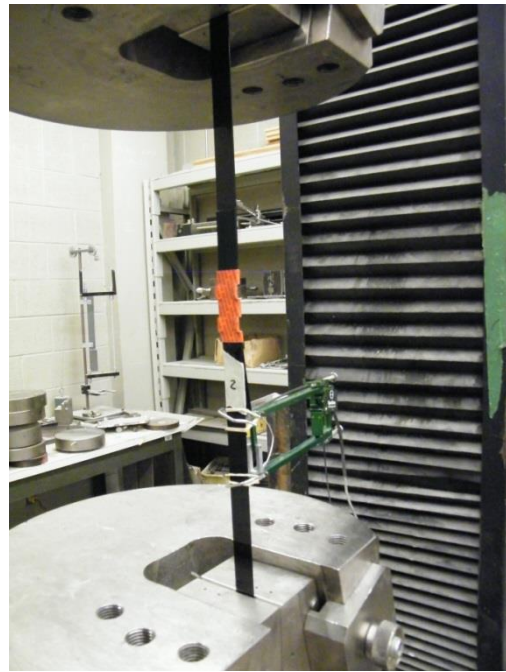
Figure 3.19: Tension Coupon

3.5.2 Steel Strap Coupons

Steel straps were extracted from the dispenser and cut to the adequate length. Two tests were conducted in order to investigate the properties of the strap, and the seal respectively. Group SS_{NS} contained no seal and comprised of five steel straps measuring 406.4 mm (16") in length. Group SS_S included the seal with a notch joint and comprised of five coupons measuring 558.8 mm (22") in total length. In both cases (Figure 3.20), the coupons were subjected to an applied tensile force and a loading rate of 4 mm/min until rupture.



(a) Group SS_{NS}



(b) Group SS_S

Figure 3.20: Steel Strapping Coupon Test

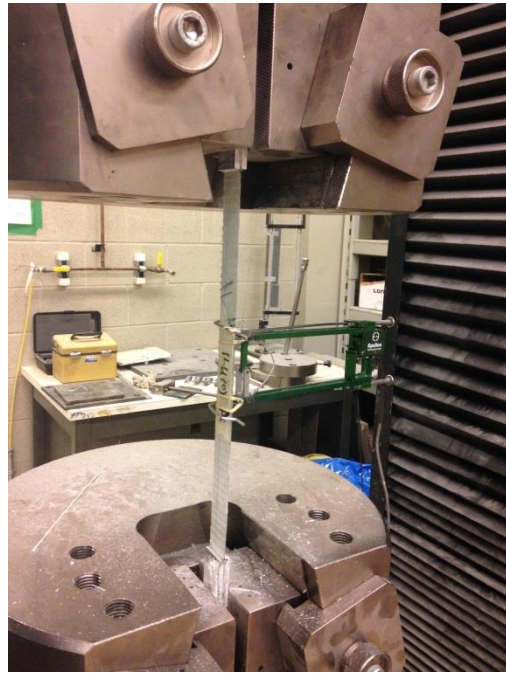
3.5.3 GFRP Coupons

The stress-strain relationship of a series of GFRP configurations was obtained by conducting coupon tests in accordance with *Standard Test Method for Tensile Properties of Polymer Matrix Composite Materials* (ASTM D3039). Glass fibre-reinforced polymer sheets were prepared by following the steps as outlined in Section 3.4.3. The cured laminations were then cut into strips measuring 25.4 mm (1") wide and varied between 406.4 – 558.8 mm (16 – 22") in length. A total of thirteen different lay-up configurations

were tested, each with five repeats. The coupons were held with four 76.2 mm (3”) long aluminum tabs in order to avoid stress concentration at the top and bottom grips. The coupons were cured for 7 days prior to testing, as depicted in Figure 3.21 (a). All coupons, as shown in Figure 3.21 (b), were subjected to a uniform applied tensile load with a loading rate of 6 mm/min.



(a) Coupon Preparation



(b) Test Setup

Figure 3.21: GFRP Coupon Test

3.5.4 Instrumentation

Four different component tests were conducted in order to determine the material properties for each element involved in the current research project. A 50 mm extensometer was used to measure the elongation of each specimen under a tensile force. Similarly, for the compression coupon tests, a linear variable displacement transducer (LVDT) was used in order to measure the change in length of each specimen.

3.6 Non-destructive Static CLT Panel Tests

The seventeen panels underwent static non-destructive tests, prior to dynamic destructive testing, in order to determine their initial stiffness. The static tests were conducted at the University of Ottawa's Structural Laboratory where CLT panels were subjected to a four point bending configuration with simply supported end conditions, as illustrated in Figures 3.22 and 3.23.

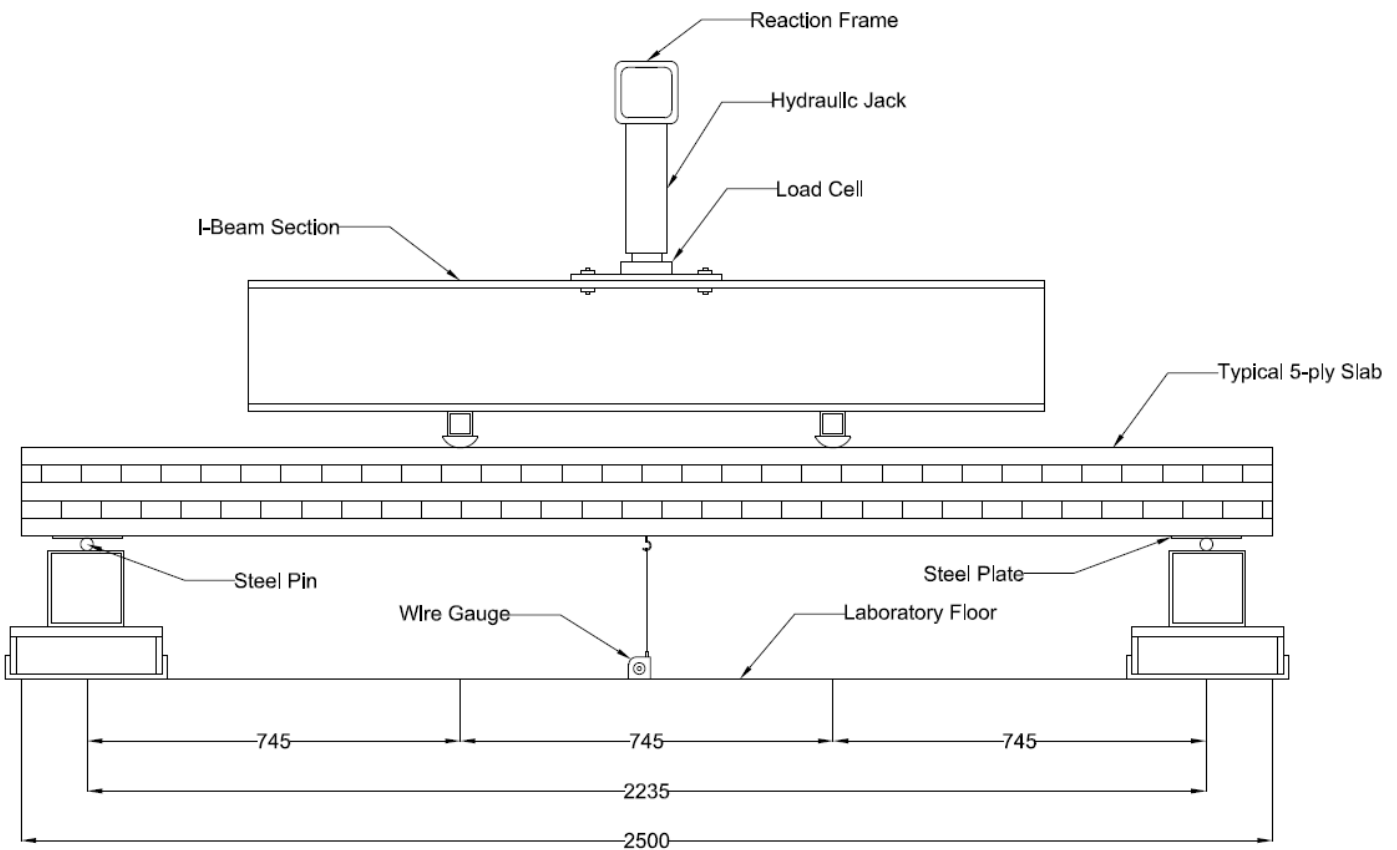


Figure 3.22: Schematic with Dimensions, mm



Figure 3.23: Typical Setup with a 5-Ply

3.6.2 Description of the Test Setup

The clear span for all panels was 2235 mm due to height constraints imposed by the shock tube. A steel plate, approximately 127 mm (5") wide and 445 mm (18") long, was fastened to the left and right side of the compression face of the panel to avoid any crushing of the material. The panel rested on two steel pins which were attached (welded) to an HSS section to provide idealized simply supported end conditions. A reaction frame, attached to a steel I-beam section, was utilized to transfer the applied force. The force was applied through the hydraulic jack and distributed to the third points of the clean span (745 mm apart).

3.6.3 Instrumentation

A load cell, mounted below the hydraulic jack, was used to measure the reactions at both supports. A wire gage, which was attached to a piece of 50.8 (2") mm by 101.6 mm (4") lumber below the panel, was used to measure the mid-span deflection, and strain gauges with a resistance and gauge length of 120 Ω , and 60 mm, respectively, were installed at the mid-span of the tension and compression face in order to develop strain-time history relationships. These parameters were recorded with a data acquisition system (DAS) at a sampling rate of 10 samples per second.

3.7 Destructive Dynamic CLT Panel Tests

The University of Ottawa's Shock Tube is a testing apparatus capable of simulating shock waves similar to those produced by far-field detonations of high explosives. Consequently, this testing apparatus was employed during the experimental program due to its ability to develop a wide range of pressures and impulses. This section provides an overview of the operation of the shock tube, in addition to the test setup and testing methodology employed in the current study.

3.7.1 Description of the University of Ottawa Shock Tube

The shock tube is pneumatically driven and consists of three main components: a variable length driver section, spool section, and expansion section followed by an end frame, as shown in Figure 3.24.

The driver section consists of a series of 12.5 mm thick steel pipes with an inner and outer diameter of 597 mm, and 812 mm, respectively. Combinations of these pipes may be selected such that the driver length varies from 305 – 5185 mm, which would yield different values of impulse. Each segment of pipe is bolted together with twenty 31.75 mm bolts and sealed using a synthetic gasket.

The spool section is approximately 90 mm long and is bolted to the driver and expansion section. A series of aluminum foils, of varying rupture strength, are installed between these two sections. The thickness and quantity of the aluminum foils is highly dependent on the pressure-impulse combination to which the specimen will be subjected. Compressed air is then introduced into the driver section such that the differential pressure between the driver and spool section, and the spool and expansion section does not exceed the capacity (i.e. rupture strength) of the foils. Once the desired pressure has been attained, the diaphragm pressure is drained causing the aluminum foils to rupture resulting in the complete release of the compressed air. This controlled mechanism can be referred to as double diaphragm firing system.

The compressed air generates a shock wave that travels through a 6096 mm long expansion chamber after which it interacts with the mounted specimen at the end frame. The expansion chamber transitions from the spool section consisting of a circular opening

of 597 mm in diameter to a 2032 x 2032 mm² opening at the end frame, which allows for testing of large or full scale structural or non-structural elements. The end frame, along the perimeter of the opening, consists of 203 mm wide and 12.5 mm thick stiffened steel plates. In addition, a total of twenty 19 mm diameter holes have been pre-fabricated to allow for a variation in load transfer devices (LTD).

A total of twelve pressure relief vents at the end frame allow for the development of a negative pressure phase, in addition to reducing the magnitude of tertiary peaks caused by wave reflection within the shock tube.



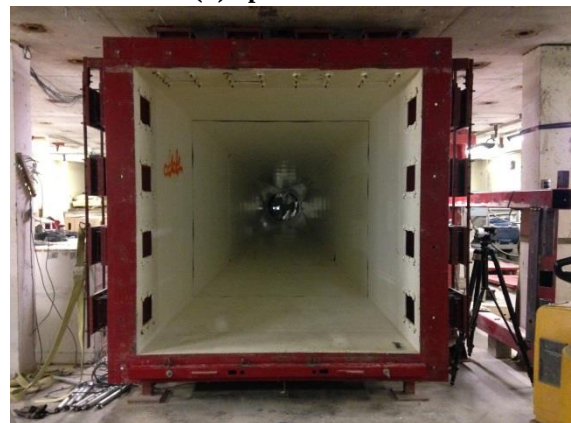
(a) Driver Section



(b) Spool Section



(c) Expansion Chamber



(d) End Frame

Figure 3.24: University of Ottawa Shock Tube

3.7.2 Description of the Test Setup

The dynamic test setup (Figure 3.25 and 3.26) reproduced identical loading pattern, clear span, and boundary conditions as the static test setup. To replicate the loading conditions, a load transfer device (LTD) was mounted onto the end frame of the shock tube. The LTD was composed of steel load transferring panels, top and bottom hinges to allow lateral movement (up to 200 mm), and two evenly spaced load transfer steel I-beam sections. This setup allowed the shock wave, generated by the sudden release of compressed air, to be transferred onto the specimen as two point loads acting at the third.

The top and bottom supports consisted of three HSS sections with the outer two pieces fastened together, as shown in Figure 3.27.

The panel was installed vertically and rested temporarily on spacing blocks to ensure a proper alignment and clear span of 2235 mm. The panel was firmly secured to the end frame by means of a strap, to avoid the panel from falling over. Load cells were placed at the support to measure the dynamic reaction forces. Two shoring jacks were situated between the top and bottom supports to ensure alignment and leveling of the setup. A bracket was bolted to the end frame to protect the measuring equipment from damage.

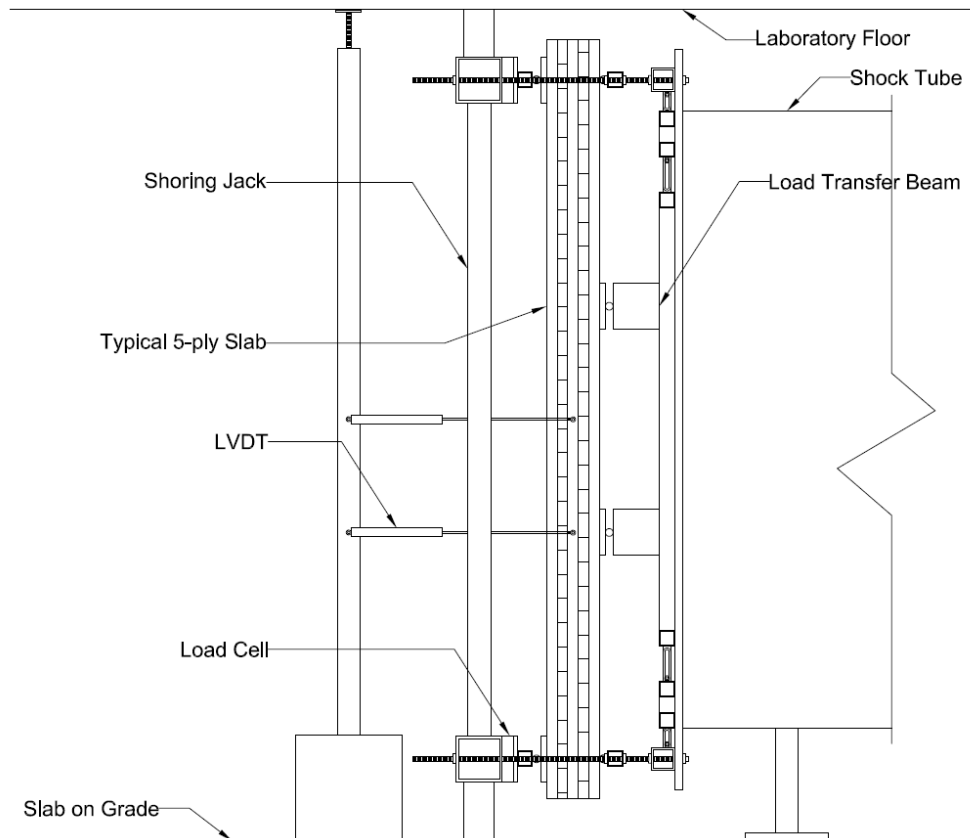


Figure 3.25: Dynamic Test Setup – Side View

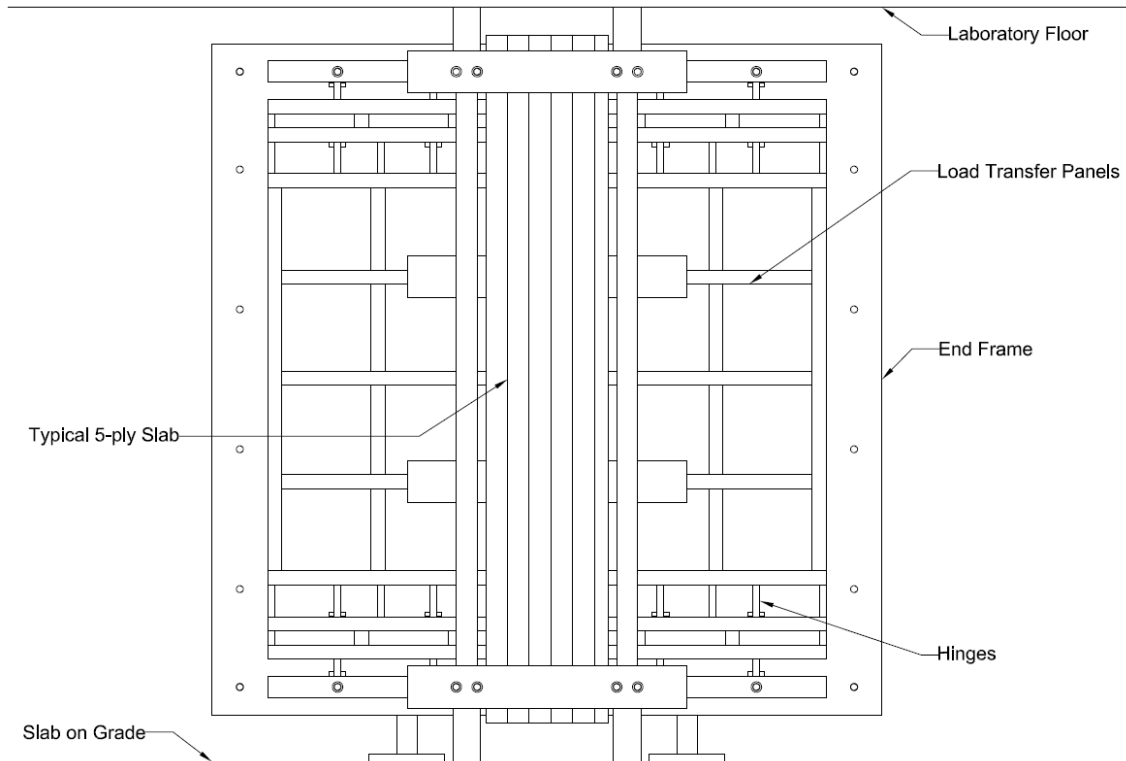


Figure 3.26: Dynamic Test Setup – Front View



Figure 3.27: Simply Supported End Condition Detail

3.7.3 Measuring Methods and Equipment

During the dynamic tests, as shown in Figures 3.28 (a) and (b), the reflected pressure acting on the test specimen was measured by two dynamic piezoelectric pressure sensors, located at the side and the bottom of the shock tube opening. The dynamic reactions of the system was measured using load cells clamped between two HSS sections, as shown in Figure 3.27. The panel displacement and shock tube movement was measured with a total of three 300 mm stroke LVDTs. The LVDTs were securely fastened to the mid-span and one-third span point, and they were encased in a steel bracket to protect them in case the specimen deflection was greater than the stroke of the LVDT.

The strain values of each material were measured by using two different types of general purpose strain gauges. The strain gauges were installed at the mid-span of the compression and tension face after sanding and cleaning any dust, debris, or splinters from the surface. A cyanoacrylate adhesive was used to glue the strain gauges onto the specimen. Electrical and duct tape was used to cover and protect the strain gauges installed on the steel strap retrofitted panels. On the contrary, the specimens retrofitted with GFRP required the use of a putty substance to ensure proper penetration of the epoxy resin system. The application of a strain gauge onto a steel strap was similar to that on a panel. The steel was sanded in order to remove the paint and wax coating, and thoroughly cleaned with an acid base mixture to remove impurities. In the case of GFRP, sanding the surface is sufficient.

The data acquisition system (DAS) has the ability to record data at a sampling rate of 100,000 samples per second. Moreover, a high speed camera capable of recording 1,000 frames per second with a resolution of 800 x 600 pixels was connected to the DAS to capture the specimen behaviour and failure. The recording process of data and video was simultaneously triggered by the shock wave travelling by the pressure sensors.



(a) Typical Phase 1 Setup



(b) Typical Phase 2 Setup

Figure 3.28: Actual Dynamic Test Setup

CHAPTER 4 – Experimental Results

4.1 General

The experimental results of this research project are summarized in the subsequent sections. The material properties of the wood (i.e. CLT), steel straps, and GFRP are summarized. Non-destructive static test results are presented followed by the destructive dynamic test results which have been divided into panels reinforced with steel straps, and GFRP, respectively. A dynamic resistance-displacement curve, specific to the panel depth, was obtained experimentally and can be found at the end of each of these sections.

4.2 Material Component Test Results

4.2.1 CLT Coupons

A total of five lumber laminations were subjected to a compressive force in order to obtain the corresponding experimental stress-strain curve. Each of the coupons experienced noticeable crushing and splitting of the fibres. The presence of defects (e.g. knots) was the primary location for initiating the failure. Figure 4.1 (a) illustrates a typical example of the observed failure mode. Wrinkling effect can be observed at the top and bottom of the specimen. The stress-strain curve, as shown in Figure 4.1 (b), was developed by using the average values tabulated in Table 4.1. It can be seen from the figure that wood in compression undergoes linear behaviour up until the compressive yield strength, followed by non-linearity once the wood fibres begin to crush.

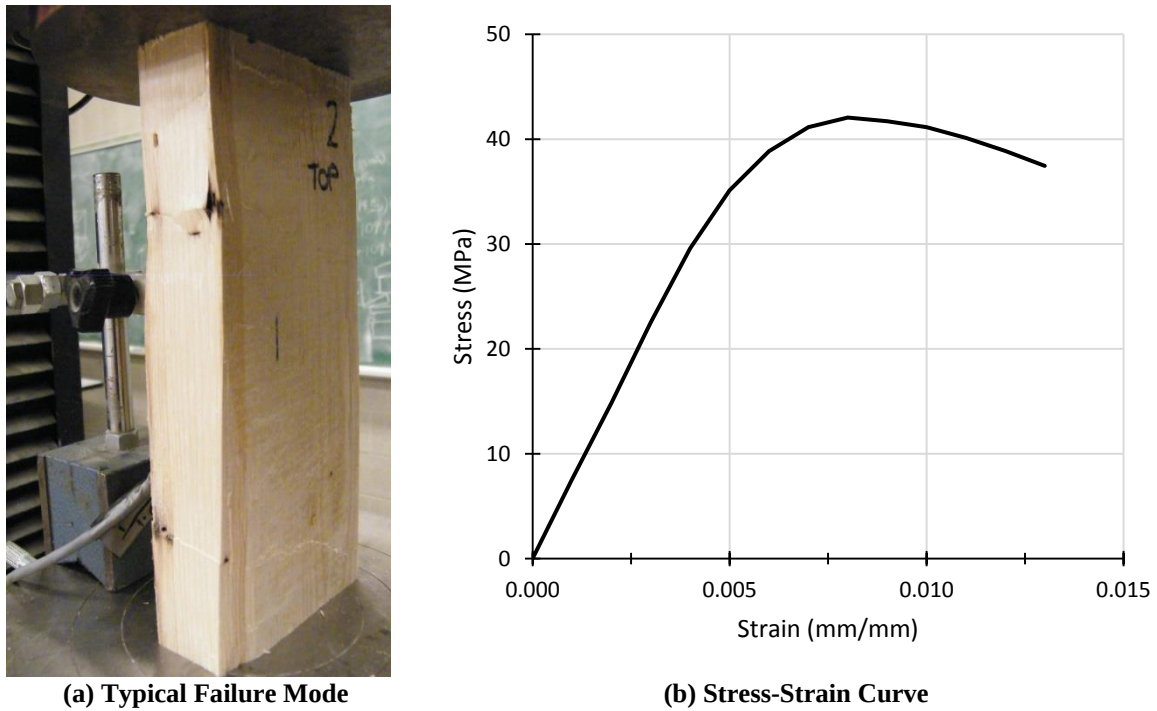


Figure 4.1: Compression Coupon Experimental Results

Table 4.1: Compression Coupon Test Results

No. of Ply's		E_{wc} ¹	f_c ²	f_{cu} ³	ϵ_{cu} ⁴ $\times 10^{-4}$
		MPa	MPa	MPa	mm/mm
CLT3	Average	7,759	42.5	38.1	145.1
CLT5	St. Dev.	1,291	6.9	5.3	24.5
CLT7	COV	0.17	0.16	0.14	0.17

¹Compression Modulus of Elasticity

²Compressive Yield Strength

³Ultimate Compressive Strength

⁴Ultimate Compressive Strain

A total of five lumber laminations were subjected to a tensile force in order to obtain the corresponding experimental stress-strain curve. The cross sections of the coupons were reduced in order to force the failure in the material rather than at the specimens end grips. All test specimens failed within the reduced sectional area. For the most part the failure was initiated near and surrounding knots in the lumber which resulted in simple tension failure. Figure 4.2 (a) illustrates a typical example of the observed failure mode. The stress-strain curve, as shown in Figure 4.2 (b), was developed by using the average values

tabulated in Table 4.2. Contrary to the relation developed by the static compression tests, the behaviour for the tension coupons was linear until ultimate failure.

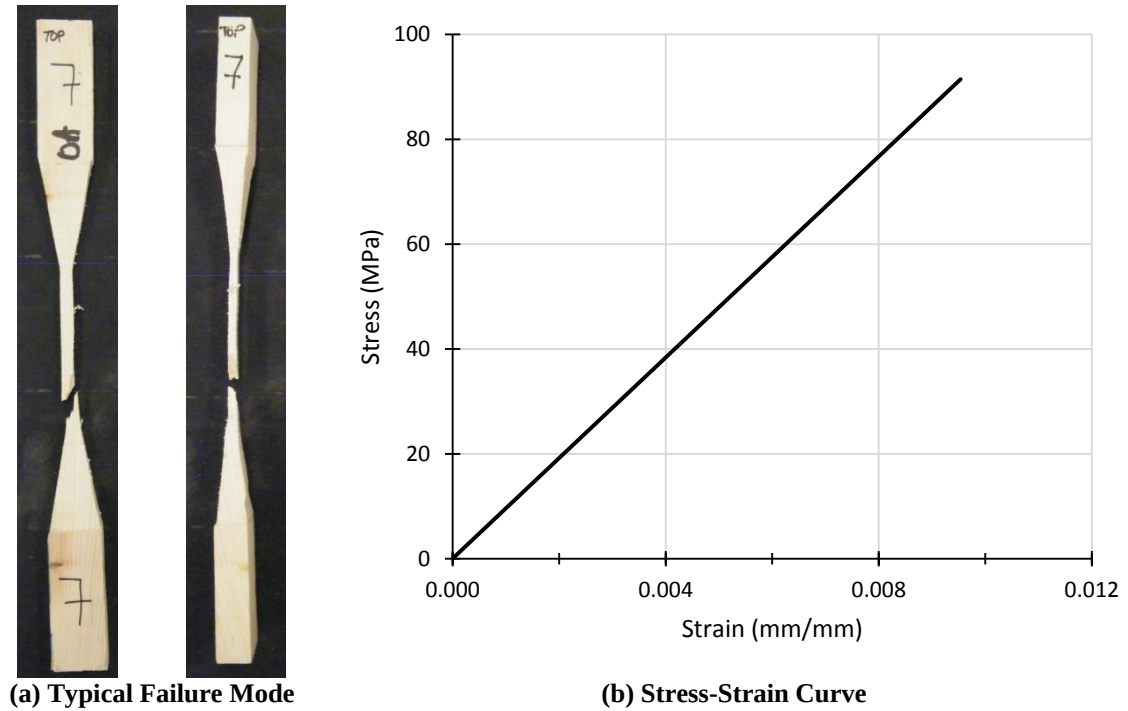


Figure 4.2: Tension Coupon Experimental Results

Table 4.2: Tension Coupon Test Results

No. of Ply's		E_{wt} ¹	f_t ²	ϵ_t ³ $\times 10^{-4}$
		MPa	MPa	mm/mm
CLT5	Average	9,574	91.0	95.4
	St. Dev.	1,128	25.3	25.0
	COV	0.12	0.28	0.26

¹Tensile Modulus of Elasticity

²Ultimate Tensile Strength

³Tensile Failure Strain

The properties described in Tables 4.1 and 4.2 represent the average values, standard deviations, and coefficient of variations for the compression, and tension wood coupons, respectively. The average modulus of elasticity extracted from the compression and tension stress strain curves is 7,800 MPa, and 9,500 MPa, respectively. The reported compressive yield strength and ultimate compressive strength were 42.5 MPa, and 38.1

MPa with a COV value of 0.16, and 0.14, respectively. Furthermore, the ultimate tensile strength was found to be 91.0 MPa with a COV value of 0.28. The coupons that were utilized to develop the aforementioned relationships and tables can be found in Appendix A – CLT Coupons Static Test Results.

4.2.2 Steel Strap Coupons

A total of ten steel straps were subjected to a tensile force in order to obtain the corresponding experimental stress-strain curves. In order to accurately represent the behaviour of the straps, tests were conducted on two types of coupons; one of which also included the seal used to securely fasten the straps. The test specimens with no seal, Group SS_{NS}, had a visible angular failure plane upon rupture. Figure 4.3 (a) highlights typical examples of the observed failure mode for this particular configuration. The stress-strain curve, as shown in Figure 4.3 (b), was developed by using the average values tabulated in Table 4.3. The behaviour of the straps was linear until the yield strength of the material after which a yield plateau is observed until the ultimate tensile rupture.

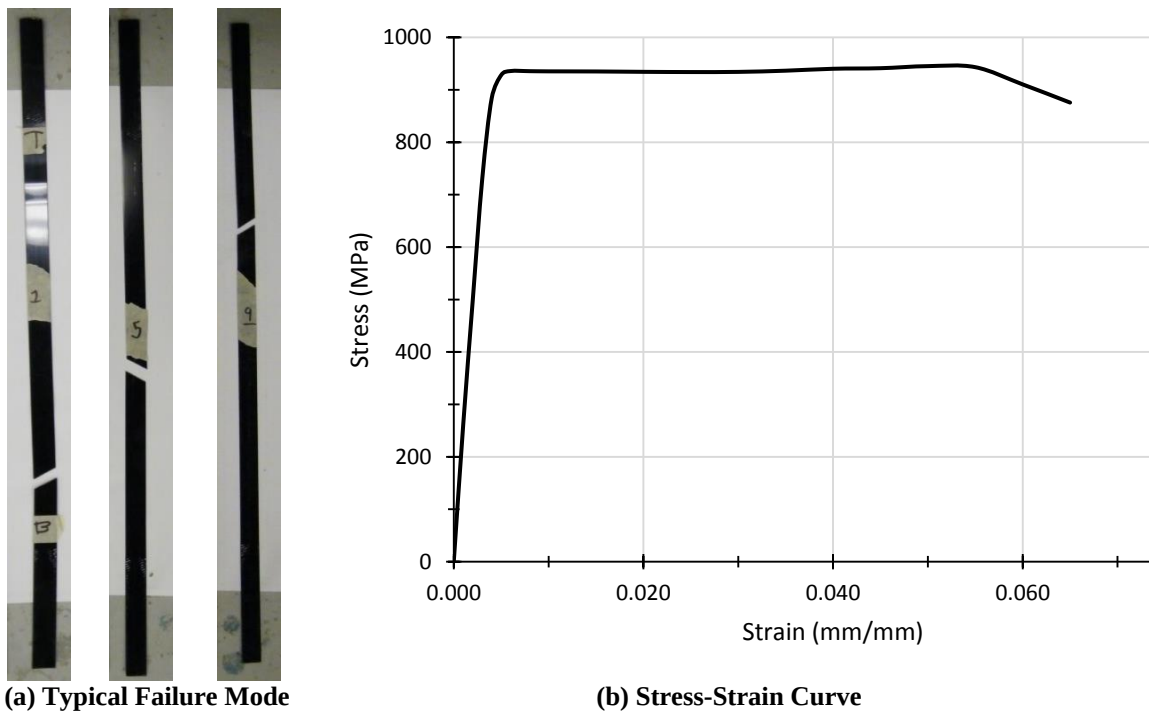


Figure 4.3: Magnus Steel Strap without Seal Coupon Experimental Results

Table 4.3: Magnus Steel Strap without Seal Coupon Test Results

Strap Type		E_{ss}^1	f_{ss}^2	f_{ssu}^3	$\epsilon_{ss}^4 \times 10^{-4}$
		MPa	MPa	MPa	mm/mm
SS _{NS}	Average	234,638	940.1	938.0	524.6
	St. Dev.	46,788	10.1	11.1	59.1
	COV	0.20	0.01	0.01	0.11

¹Magnus Steel Strap Modulus of Elasticity

²Yield Strength

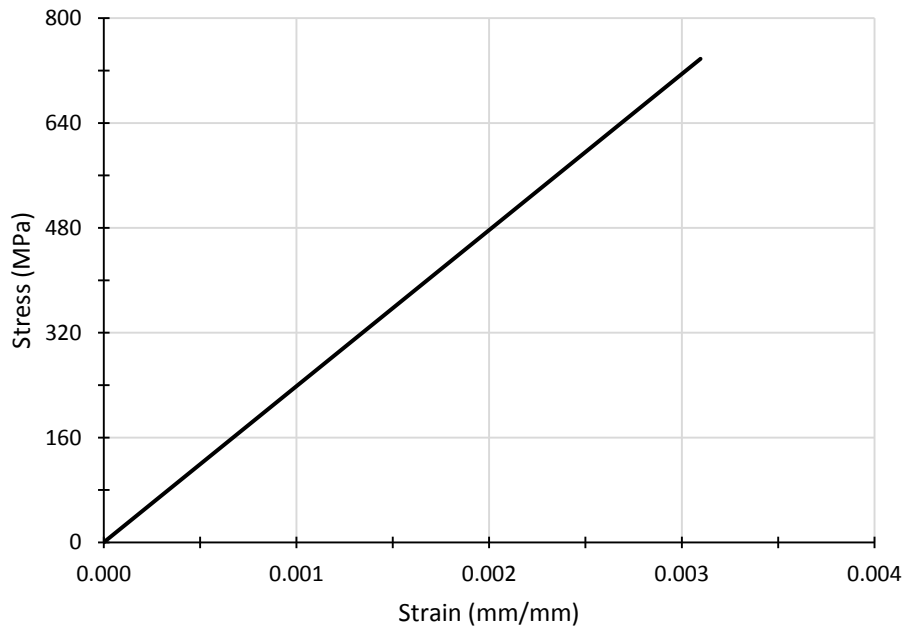
³Ultimate Tensile Strength

⁴Tensile Failure Strain

The test specimens that included the seal, Group SS_S, consistently failed at the location of the reverse notch joint (i.e. within the seal). Figure 4.4 (a) shows the failure mode observed at the fastened joint. Albeit this clear-cut failure mechanism, the strap steel strap snapped at the top or the underside of the seal, respectively. The stress-strain curve, as shown in Figure 4.4 (b), was developed by using the average values tabulated in Table 4.4. Unlike Group SS_{NS}, this relationship experienced linearity until the point of ultimate tensile strength.



(a) Typical Failure Mode



(b) Stress-Strain Curve

Figure 4.4: Magnus Steel Strap with Seal Coupon Experimental Results

Table 4.4: Magnus Steel Strap with Seal Coupon Test Results

Strap Type		E_{ss}^1	f_{ss}^2	$\epsilon_{ss}^3 \times 10^{-4}$
		MPa	MPa	mm/mm
SS _s	Average	239,388	738.0	31.0
	St. Dev.	14,550	17.2	1.6
	COV	0.06	0.02	0.05

¹Magnus Steel Strap Modulus of Elasticity

²Ultimate Tensile Strength

³Tensile Failure Strain

The properties described in Tables 4.3 and 4.4 represent the average values, standard deviations, and coefficient of variations for the coupons associated to Groups SS_{NS}, and SS_s, respectively. The average modulus of elasticity was found to be 234,638 MPa and 239,388 MPa, with a corresponding COV value of 0.20 and 0.06, for Groups SS_{NS}, and SS_s, respectively. The reported yield strength of Group SS_s was found to be 738.0 MPa which was approximately 21.5 % less than that reported by Group SS_{NS}. Furthermore, the ultimate yield strength recorded for the Magnus steel strap without a seal was found to be 938.0 MPa with a COV value of 0.01. Comparing both groups of coupons, there was very little variability observed in the experimental results. The coupons that were used to generate the previously mentioned relationships and data can be found in Appendix B – Steel Strap Coupons Static Test Results.

4.2.3 GFRP Coupons

A total of thirteen different lay-up configurations, each with five repeats, were subjected to tensile force in order to obtain the corresponding experimental stress-strain curve. Of the sixty-five coupons that were tested twenty-five consisted of 1 and 2 layers of homogenous fabric, including a built up quadraxial configuration while the remaining coupons consisted of four layers of fabric. Figure 4.5 (a) – (e) illustrates the typical failure modes observed for coupons with 1 layer of fabric with the following configurations, respectively: uniaxial [0°], biaxial [0/90°], biaxial [$\pm 45^\circ$], triaxial [0, $\pm 45^\circ$], and quadraxial [0/90°, $\pm 45^\circ$]. Coupons with uniaxial fabric experienced splitting along the 0° direction. The biaxial fabric with fibres in the 0° and 90° direction experienced a significant amount of splitting between the longitudinal fibres and lateral delamination between the transverse fibres. The biaxial fabric with fibres oriented in the

$\pm 45^\circ$ direction experienced rupture along the direction of the fibre. While this configuration consistently formed an angular failure plane along the cross-sectional area, white marks formed at several locations along the specimen indicating stretch in the fabric. The triaxial fabric encompassed the failure trends observed in the preceding configurations. A series of angular failure planes between the $0, \pm 45^\circ$ direction, delamination of the fibres along the $0, \pm 45^\circ$ interface, splitting and elongation of the fibres (i.e. formation of white marks), and rupture along the cross-sectional area. The quadraxial fabric experienced a failure mechanism that resembled the one observed in the triaxial fabric coupons. Some delamination between the two different fabrics was evident.

The stress-strain curve, as shown in Figure 4.6, was developed by using the average values tabulated in Table 4.5. In general, the GFRP specimens displayed linear behaviour up until the ultimate strength with varying corresponding failure strains. On the contrary, when the applied load was not aligned with the direction of the fibres, non-linear behaviour was observed. As expected, the uniaxial fabric resulted in a relatively greater modulus of elasticity and ultimate strength. The triaxial fabric resulted in the greatest failure strain when compared to the other fabrics.

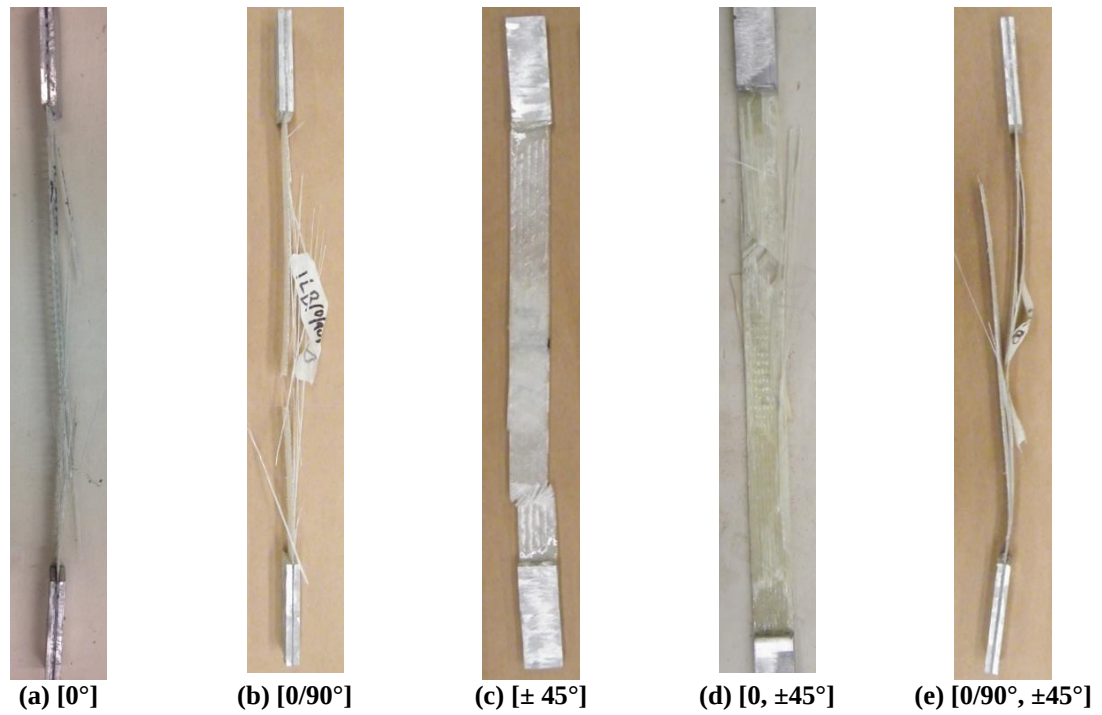


Figure 4.5: 1 Layer of GFRP Typical Failure Modes

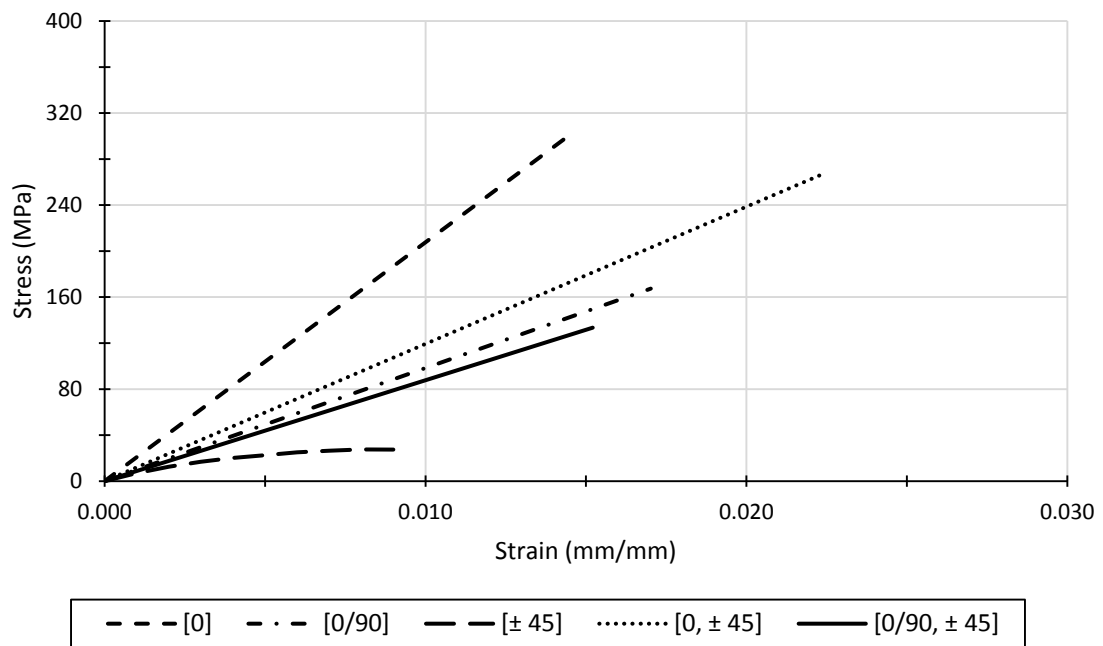


Figure 4.6: 1 Layer of GFRP Stress-Strain Curve

Table 4.5: 1 Layer of GFRP Coupon Test Results

Configuration		E_{GFRP}^1	f_{GFRP}^2	$\epsilon_{f GFRP}^3 \times 10^{-3}$
		MPa	MPa	mm/mm
[0°]	Average	21,218	303.1	14.6
	St. Dev.	959	103.1	4.8
	COV	0.05	0.34	0.33
[0/90°]	Average	10,384	167.3	17.0
	St. Dev.	593	25.0	2.3
	COV	0.06	0.15	0.13
[± 45°]	Average	6,020	25.9	7.3
	St. Dev.	1,074	4.6	2.1
	COV	0.18	0.18	0.29
[0, ± 45°]	Average	12,408	267.2	22.4
	St. Dev.	946	67.2	3.9
	COV	0.08	0.25	0.17
[0/90°, ± 45°]	Average	9,459	133.4	15.2
	St. Dev.	385	12.7	1.9
	COV	0.04	0.10	0.13

¹GFRP Modulus of Elasticity²Ultimate Strength³Failure Strain

The properties described in Table 4.5 represent the average values, standard deviations, and coefficient of variation for the coupons with 1 layer of GFRP. The modulus of elasticity of the uniaxial fabric was found to be 21,218 MPa with a corresponding COV value of 0.05. The reported modulus of elasticity of the triaxial, biaxial [0/90°], quadraxial and biaxial [± 45°] fabric was approximately 41.5 %, 51.1 %, 55.4 %, and 71.6 % less than the uniaxial fabric, respectively. The ultimate yield strength recorded for the uniaxial, triaxial, biaxial [0/90°], quadraxial and biaxial [± 45°] fabric was found to be 303.1 MPa, 267.2 MPa, 167.3 MPa, 133.4 MPa and 25.9 MPa with a corresponding COV value of 0.34, 0.25, 0.15, 0.10, and 0.18, respectively.

Figures 4.7 and 4.8 illustrate the stress-strain curves for coupons with 2 and 4 layers of GFRP which were generated with the average values tabulated in Tables 4.6, and 4.7, respectively.

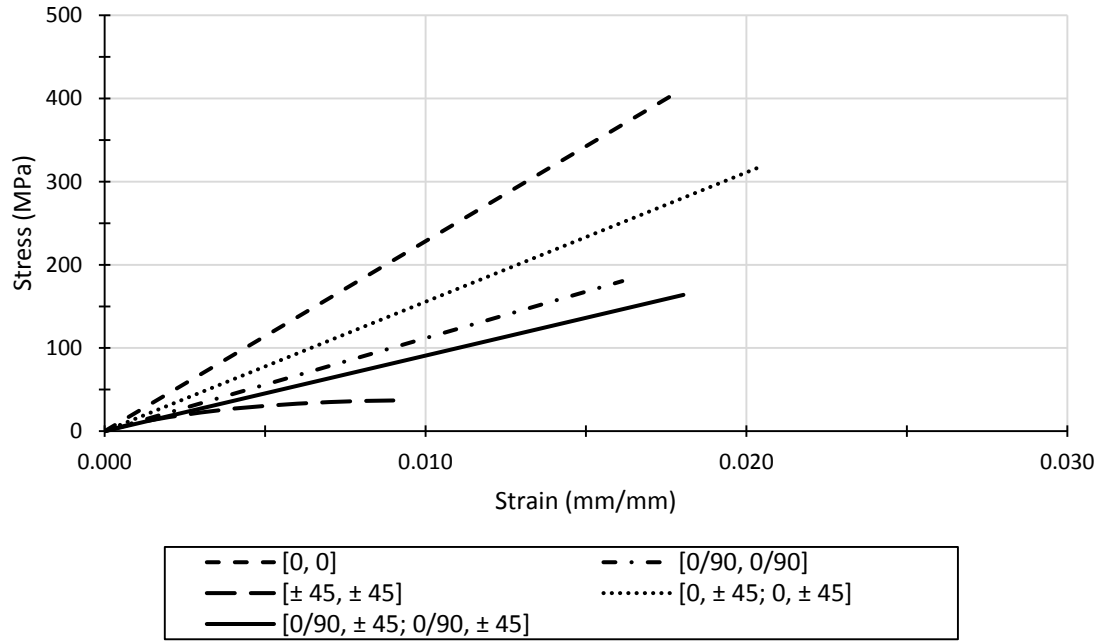


Figure 4.7: 2 Layers of GFRP Stress-Strain Curve

Table 4.6: 2 Layers of GFRP Coupon Test Results

Configuration		E_{GFRP}^1	f_{GFRP}^2	$\epsilon_{f GFRP}^3 \times 10^{-3}$
		MPa	MPa	mm/mm
$[0^\circ]_2$	Average	23,395	409.2	17.9
	St. Dev.	2,526	51.2	2.2
	COV	0.11	0.13	0.13
$[0/90^\circ]_2$	Average	11,937	180.3	16.1
	St. Dev.	2,364	10.9	3.1
	COV	0.20	0.06	0.19
$[\pm 45^\circ]_2$	Average	7,378	37.2	9.3
	St. Dev.	811	3.1	0.5
	COV	0.11	0.08	0.05
$[0, \pm 45^\circ]_2$	Average	16,339	319.0	20.5
	St. Dev.	1,785	41.9	4.8
	COV	0.11	0.13	0.24
$[0/90^\circ, \pm 45^\circ]_2$	Average	9,863	163.9	18.0
	St. Dev.	867	12.7	2.3
	COV	0.09	0.08	0.13

¹GFRP Modulus of Elasticity

²Ultimate Strength

³Failure Strain

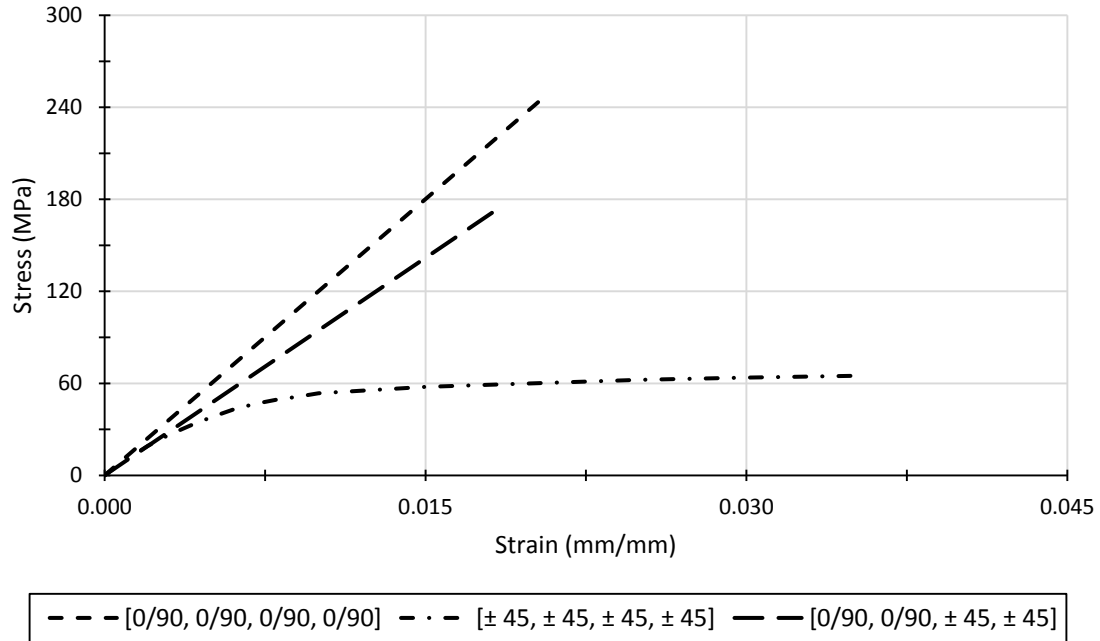


Figure 4.8: 4 Layers of GFRP Stress-Strain Curve

Table 4.7: 4 Layers of GFRP Coupon Test Results

Configuration		E_{GFRP}^1	f_{GFRP}^2	$\epsilon_{f GFRP}^3 \times 10^{-3}$
		MPa	MPa	mm/mm
[0/90°] ₄	Average	12,982	249.5	20.8
	St. Dev.	2,066	12.9	3.9
	COV	0.16	0.05	0.19
[± 45°] ₄	Average	7,692	59.0	25.3
	St. Dev.	1,099	7.2	12.8
	COV	0.14	0.12	0.50
[0/90°] ₂ [± 45°] ₂	Average	10,461	175.8	18.6
	St. Dev.	1,603	4.3	3.0
	COV	0.15	0.02	0.16

¹GFRP Modulus of Elasticity

²Ultimate Strength

³Failure Strain

The addition of an extra layer of GFRP resulted in an overall increase in the modulus of elasticity, ultimate strength and failure strain of 4 – 32 %, 8 – 44 %, and 18 – 27 %, respectively. In the case of biaxial [0/90°] and biaxial [± 45°], 4 layers of GFRP resulted in an increase of 25.0 % and 27.8 % in modulus of elasticity and 49.1 %, and 127.8 % in

ultimate strength, respectively. The coupons that were used to develop the aforementioned relationships and data can be found in Appendix C – GFRP Coupons Static Test Results.

4.3 Non-destructive Static Test Results

The unretrofitted panels were preloaded within their elastic limit in order to determine the stiffness prior to dynamic loading.

4.3.1 CLT3

A total of six 3-ply CLT panels were subjected to four point bending in order to obtain the corresponding force-displacement curve. Table 4.8 summarizes the key experimental data acquired from each non-destructive test, in addition to the average values, standard deviations, and coefficient of variation. The force-displacement curve, as shown in Figure 4.9, was developed by plotting the applied load values against the mid-span deflection which in turn was used to determine the stiffness. The average static stiffness of the 3-ply CLT panels was found to be 2.0 kN/mm with a COV value of 0.07. This particular relationship experienced linear behaviour in the test range (approximately 20 % of the anticipated ultimate static resistance).

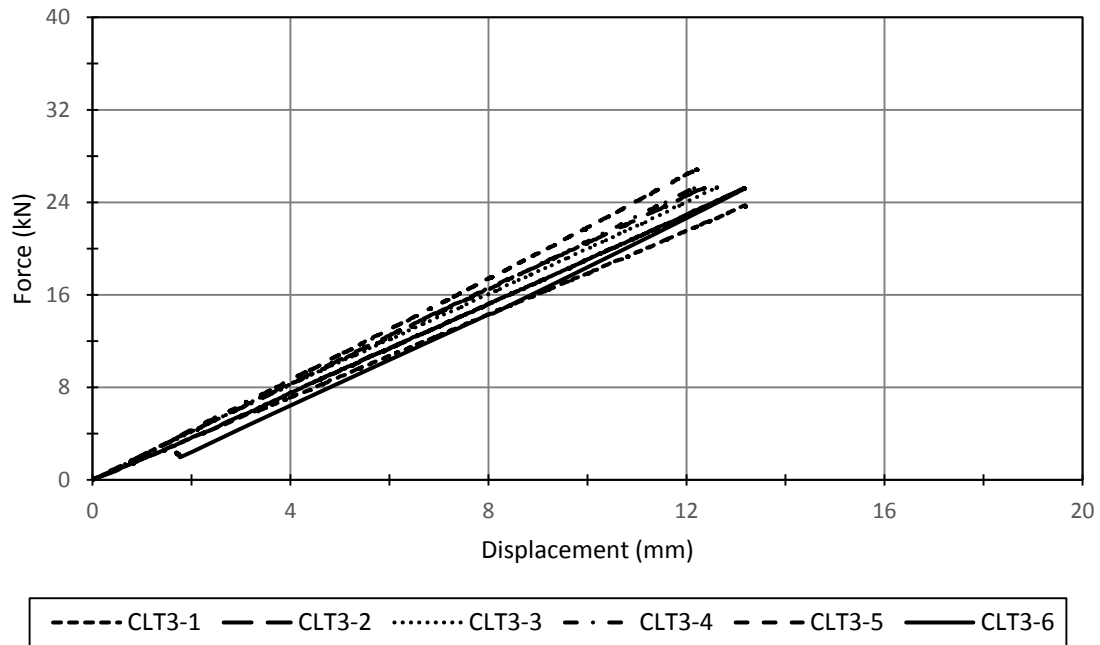


Figure 4.9: Force-Displacement Curve – 3-Ply

Table 4.8: Non-Destructive Static Test Results – 3-Ply

Test Name	K^1
	kN/mm
CLT3-1	1.8
CLT3-2	2.1
CLT3-3	2.0
CLT3-4	2.1
CLT3-5	2.2
CLT3-6	1.9
Average	2.0
St. Dev.	0.1
COV	0.07

¹Stiffness

4.3.2 CLT5

A total of seven 5-ply CLT panels were subjected to a four point bending configuration in order to obtain the corresponding force-displacement curve. Table 4.9 summarizes the key experimental data acquired from each non-destructive test, in addition to the average values, standard deviations, and coefficient of variation. The force-displacement curve for the 5-ply CLT specimens was developed by plotting the displacement values against

the applied load which in turn resulted in the development of the stiffness, as shown in Figure 4.10. The average static stiffness of a 5-ply CLT panel was found to be 5.4 kN/mm with a COV value of 0.06. This particular relationship experienced linear behaviour up until 20 % of the ultimate static resistance.

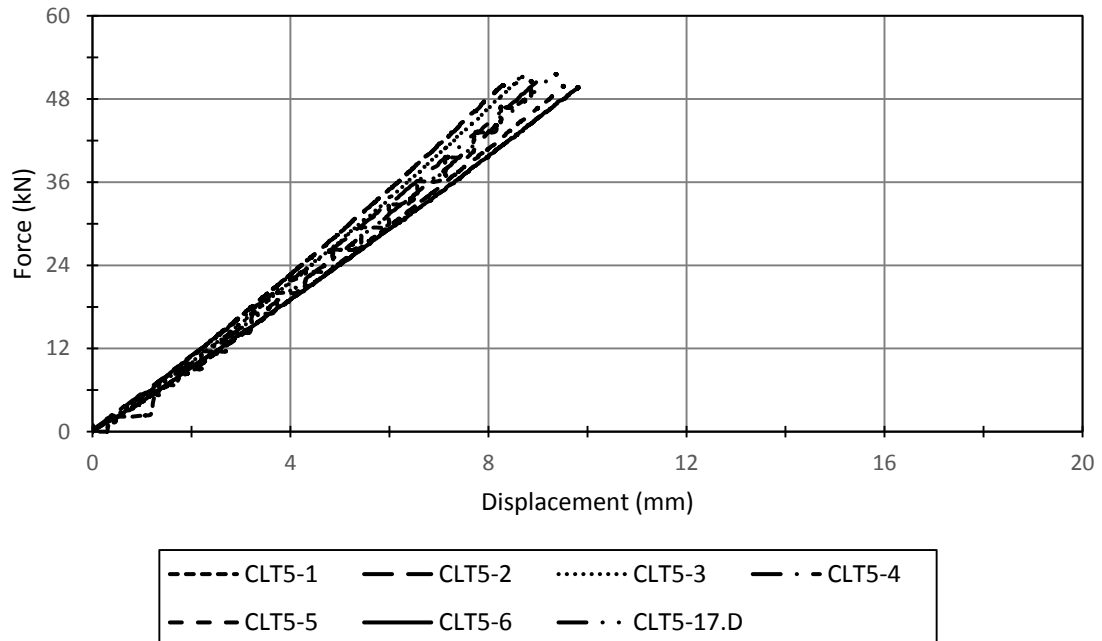


Figure 4.10: Force-Displacement Curve – 5-Ply

Table 4.9: Non-Destructive Static Test Results – 5-Ply

Test Name	K^1
	kN/mm
CLT5-1	5.6
CLT5-2	5.8
CLT5-3	5.7
CLT5-4	5.5
CLT5-5	5.0
CLT5-6	5.0
CLT5-D1	5.3
Average	5.4
St. Dev.	0.3
COV	0.06

¹Stiffness

4.3.3 CLT7

A total of two 7-ply CLT panels were subjected to a four point bending configuration in order to obtain the corresponding force-displacement curve. Table 4.10 summarizes the key experimental data acquired from each non-destructive test, in addition to the average values, standard deviations, and coefficient of variation. The force-displacement curve, as shown in Figure 4.11, was developed by plotting the displacement values against the applied load which in turn resulted in the development of the stiffness. The average static stiffness of a 7-ply CLT panel was found to be 7.1 kN/mm with a COV value of 0.01. This particular relationship experiences linear behaviour up until 20 % of the ultimate static resistance.

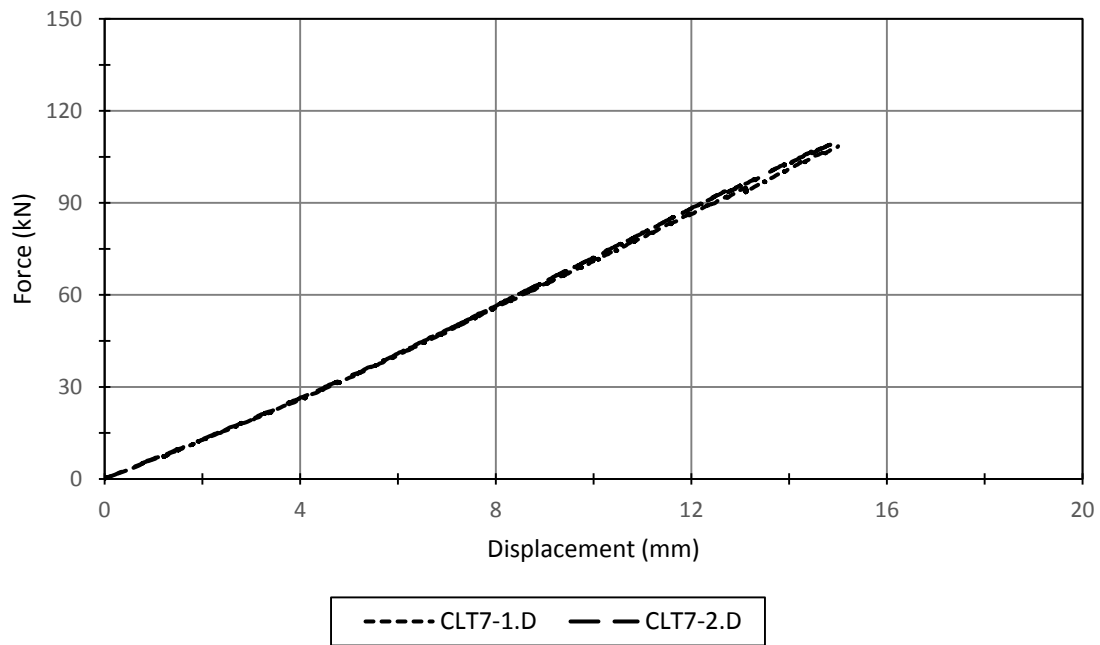


Figure 4.11: Force-Displacement Curve – 7-Ply

Table 4.10: Non-Destructive Static Test Results – 7-Ply

Test Name	K^1
	kN/mm
CLT7-D1	7.0
CLT7-D2	7.2
Average	7.1
St. Dev.	0.1
COV	0.01

¹Stiffness

4.4 Dynamic Terminology

The behaviour of wood changes when FRP fabric is used to reinforce the wood material. Wood no longer experiences a brittle behaviour dominated by tension failure. Some post peak resistance is present in the composite section. Lacroix (2017) described the behaviour of a composite wood-FRP member to behave linear elastically up to what the author termed R_{max} , with a corresponding deflection at peak resistance, Δ_{Rmax} , as seen in Figure 4.12. A sudden drop is observed immediately following the peak resistance followed by a post peak plateau. Lacroix (2017) only accounts post peak resistance magnitudes of 50% or more of peak resistance. The ultimate resistance of the member, denoted as R_f , relates to the point at which the resistance falls below 50 % of the maximum peak resistance, or where de-bonding of the FRP occurred. The ultimate failure occurs at a corresponding deflection, denoted as Δ_{Rf} . Lacroix (2017) emphasized the relevance of the residual resistance since it contributes to the overall dissipation of energy. In the case where the flexural resistance is no longer of significance, the maximum recorded displacement may be greater than that recorded at the ultimate deflection. This phenomenon is illustrated by the dashed line and is denoted as Δ_{max} . Conversely, when a member is subjected to a force within its elastic limit the maximum deflection will yield a smaller value compared to the ultimate deflection.

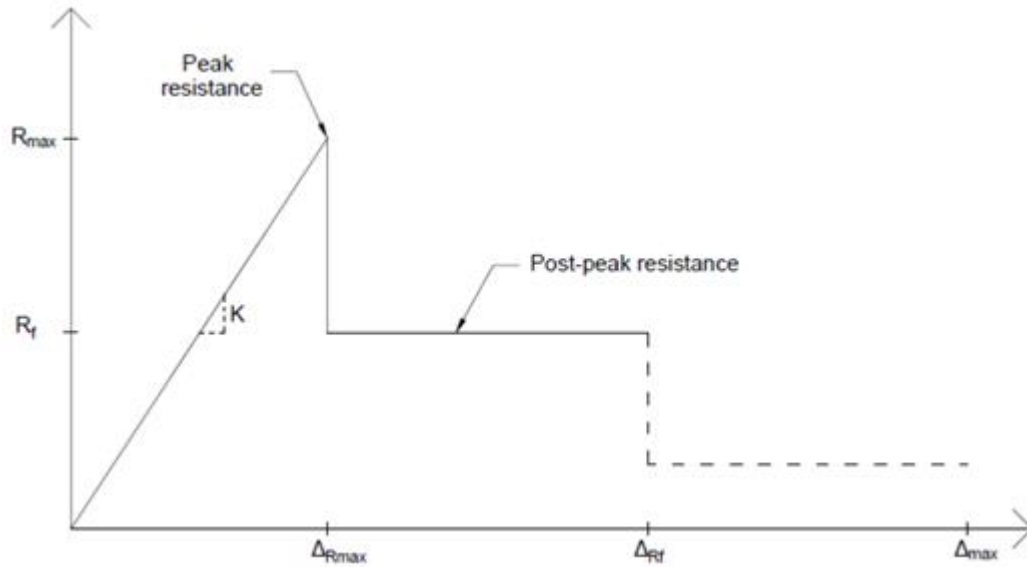


Figure 4.12: Resistance Curve Terminology (Lacroix, 2017)

4.5 Destructive Dynamic Test Results

The following section summarizes the dynamic test results corresponding to Phase 1 and 2 of the experimental program, as described in Chapter 3. Individual configurations are briefly outlined and failure modes are discussed. Details pertaining to the applied pressure-impulse combinations for each test are also presented in this chapter. A summary section, for each phase of testing, has been included to illustrate the dynamic resistance-displacement relationship for each cross-section of CLT and their respective initial condition. Experimental data relating, but not limited to, maximum displacement, maximum resistance, and stiffness has been reported in the subsequent table; followed by pertinent strain data.

4.5.1 Test Nomenclature

In an attempt to accurately describe the observed failure modes, each material (i.e. CLT, steel straps, and GFRP) had a designated labeling system. In this chapter, the tension face is also referred to as the “front face” of the panel. As a result, the left and right faces refer to panel sides as seen on the left, and right hand side of the panel, respectively. The longitudinal laminations were labelled 1 through 5, for panels of 3 and 5-ply elements,

and 1 through 3 for panels of 7-ply elements; where lamination 1 was the first on the left side of the specimen.

Steel straps were installed in the horizontal (i.e. transverse) and vertical (i.e. longitudinal) directions along the clear span. The horizontal straps followed a numeric labeling convention. Strap 1 was found at the top of the panel. The vertical straps were categorized alphabetically and were also associated to each longitudinal lamination of the specimen. As such, strap A was affiliated with the first lamination located on the left side of the panel

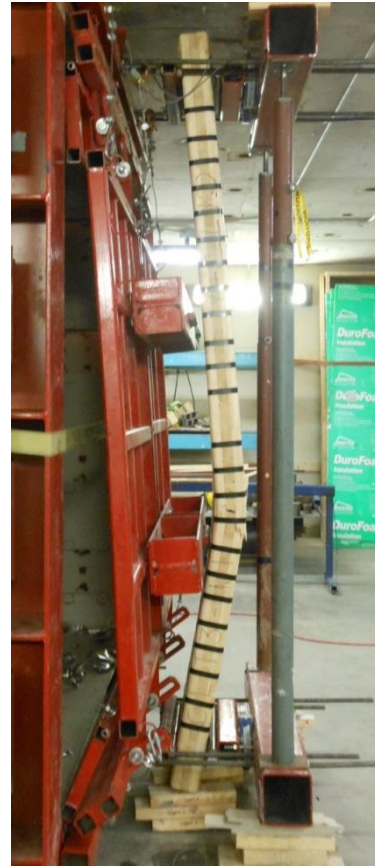
4.5.2 Phase 1: Steel Straps

4.5.2.1 CLT3-1

Specimen 1, CLT3-1, consisted of a total of twenty-four horizontal steel straps that were installed at every transverse layer along the clear span of the panel, as shown in Figure 4.13 (a). The member was subjected to a driver pressure of 424.0 kPa, which resulted in a reflected pressure and impulse of 53.7 kPa, and 577.7 kPa-ms, respectively. The specimen reached a maximum displacement of 187.5 mm in 66.0 ms. Figure 4.13 (b) illustrates the flexural failure that occurred at the bottom third of the panel.



(a) Before



(b) After

Figure 4.13: Specimen No. 1

It was observed that the failure occurred at locations of finger joints (laminate 1 in Figure 4.14 (a)) or knots (laminate 3 and 4 Figure 4.14 (b)). The failure then propagated along the width of the panel, fracturing the remaining longitudinal laminates on the tension face, as shown in Figure 4.14 (b). Rolling shear failure was also observed near the support but it was very limited by the presence of the horizontal straps. There were no other indications of failure elsewhere along the CLT panel. The straps helped contain the failure mechanism within a localized region. There was minimal to no failure in the steel straps. On the contrary, the steel straps contained the panel which in turn resulted in very minimal debris and further expansion of the member.



(a) Finger Joint on Laminate 1



(b) Tension Face

Figure 4.14: CLT3-1 Failure Mode

4.5.2.2 CLT3-2

Specimen 2, CLT3-2, consisted of a total of sixteen horizontal steel straps that were installed at every transverse layer along the top and bottom thirds of the panel, as shown in Figure 4.15 (a). The member was subjected to a driver pressure of 353.7 kPa, which resulted in a reflected pressure and impulse of 52.5 kPa, and 594.6 kPa-ms, respectively. The specimen reached a maximum displacement of 273.8 mm in 51.8 ms. Figure 4.15 (b) illustrates the failure that occurred at the mid-span of the panel.



(a) Before



(b) After

Figure 4.15: Specimen No. 2

The failure in this specimen was dominated by flexure, which initiated at a finger joint located on the right side of laminate 1. The failure then propagated along the width of the panel resulting in the rupture of all longitudinal layers on the tension face, as shown in Figure 4.16 (a). Rolling shear failure was very limited. Compression face failure was evident, as shown in Figure 4.17.



(a) Tension Face



(b) Right Face

Figure 4.16: CLT3-2 Flexural Failure



Figure 4.17: CLT3-2 Compression Failure

The horizontal straps confined the specimen in the transverse direction and ultimately promoted a flexural failure. No failure was observed in any of the straps in fact they all remained intact and tensioned.

4.5.2.3 CLT5-1

Specimen 3, CLT5-1, consisted of a total of twenty-four horizontal steel straps followed by five vertical steel straps that were installed at every transverse and longitudinal layer along the clear span of the panel. CLT5-1.1 was subjected to a driver pressure of 597.1 kPa, which resulted in a reflected pressure and impulse of 71.0 kPa, and 881.0 kPa-ms, respectively. A maximum displacement of 76.9 mm was reached in 26.8 ms. The panel returned to its original position with minor rolling shear damage and a few cracks at the finger joints located on the tension face. Initial delamination was noticeable on the video replay within the top third at a time and corresponding deflection of 18.0 ms. and 56.1 mm, respectively. The horizontal and vertical straps showed no sign of damage; however the vertical straps experienced some slack at the mid-span on the compression face of approximately 1 – 2 cm which in turn resulted in taught straps on the tension face. Given that the panel did not fail, as shown in Figure 4.18, it was deemed acceptable to re-test the specimen.

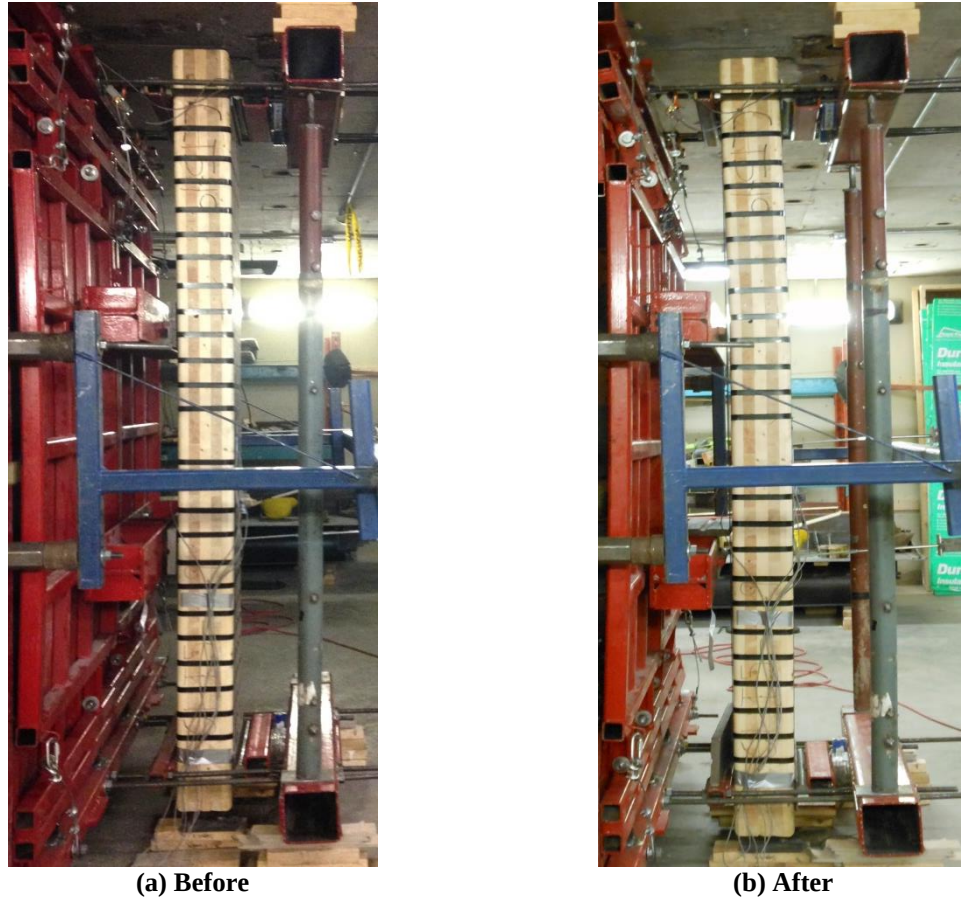


Figure 4.18: Specimen No. 3 – Shot 1

CLT5-1.2 was subjected to a driver pressure of 455.1 kPa, which resulted in a reflected pressure and impulse of 58.2 kPa, and 829.2 kPa-ms, respectively. The panel reached a maximum displacement of 212.1 mm in 56.4 ms. Figure 4.19 illustrates the shear and flexural failure that occurred at the top third of the panel.



(a) Before



(b) After

Figure 4.19: Specimen No. 3 – Shot 2

The specimen underwent significantly more rolling shear failure than in the first shot, where transverse laminate shear failure initiated at a time of 14.0 ms with a corresponding deflection of 32.7 mm. Longitudinal shear failure could be observed at a time and corresponding deflection of 18.0 ms, and 59.0 mm, respectively, as shown in Figure 4.20.



(a) Right Face



(b) Top Face

Figure 4.20: CLT5-1.2 Shear Failure

The flexural failure initiated at a finger joint located on laminate 1, as highlighted in Figure 4.21. The failure then propagated through the entire width of the panel, where compression failure could also be observed, as illustrated in Figure 4.22. Such failure is favorable due to its ductile characteristics.



(a) Finger Joint on Laminate 1



(b) Tension Face

Figure 4.21: CLT5-1.2 Flexural Failure



Figure 4.22: CLT5-1.2 Compression Failure

The horizontal straps remained intact, whereas the vertical straps experienced some damage: for example strap E, located on laminate 5, snapped at the seal on the top face. Whereas straps A through D experienced slack on the compression face and as a result created a gap of approximately 18 mm at the location of failure. Furthermore, the aforementioned straps bent slightly in the direction of the specimen deflected shape. Overall, the straps were able to confine the member and reduce the level debris.

4.5.2.4 CLT5-2

Specimen 4, CLT5-2, consisted of a total of five vertical steel straps followed by twenty-five horizontal steel straps that were installed at every longitudinal and transverse layer along the clear span of the span. CLT5-2.1 was subjected to a driver pressure of 537.1 kPa, which resulted in a reflected pressure and impulse of 68.6 kPa, and 983.1 kPa-ms, respectively. The specimen reached a maximum displacement of 123.9 mm in 50.8 ms. Unlike the preceding specimen, there was initial localized failure on the tension face in laminates 1 to 4. Wood crushing and finger joint failure was observed. The straps limited the amount of rolling shear expected; however, longitudinal shear failure occurred at the top end of the specimen. Through video review, it could be documented that the initial shear failure occurred at a time of 22.0 ms and a corresponding deflection of 64.6 mm followed by initial laminate separation at a time and deflection of 34.0 ms, and 106.9 mm, respectively. The vertical and horizontal steel straps remained undamaged. Due to the significantly small amount of residual displacement, as shown in Figure 4.23, the member was tested once more with the primary goal of promoting flexural failure.



(a) Before



(b) After

Figure 4.23: Specimen No. 4 – Shot 1

CLT5-2.2 represents the second shot of the specimen described above. The specimen was subjected to a driver pressure of 595.0 kPa, which resulted in a reflected pressure and impulse of 62.4 kPa, and 981.4 kPa-ms, respectively. The specimen reached a maximum displacement of 123.9 mm in 52.2 ms. Figure 4.24 (b) illustrates the shear and flexural failure that occurred at the top third of the panel.



(a) Before



(b) After

Figure 4.24: Specimen No. 4 – Shot 2

Shear failure was one of the predominant failure modes of this particular specimen occurring at a time and corresponding deflection of 14.0 ms and 34.1 mm, respectively, as seen in Figure 4.25.



(a) Front Face



(b) Top Face

Figure 4.25: CLT5-2.2 Shear Failure

Flexural failure, as shown in Figure 4.26, was limited to the top third of the panel. The failure developed at the location of a finger joint located on laminate 1, followed by a strap failure. The failure propagated through all five longitudinal laminates in addition to the compression face of the panel, as illustrated in Figure 4.27. Very little rolling shear cracks were observed.



(a) Left Face



(b) Right Face

Figure 4.26: CLT5-2.2 Flexural Failure



Figure 4.27: CLT5-2.2 Compression Failure

The horizontal straps were left undamaged except for strap 9. This particular strap snapped and broke off which caused the opening of the longitudinal laminates. The vertical straps experienced a high level of distress. Straps A through E failed at the top of the member and bent inwards in the direction of the compression failure. Furthermore, while strap A buckled at the bottom support, strap E had begun to buckle at the bottom support potentially due to pinching effects.

4.5.2.5 CLT5-3

Specimen 5, CLT5-3, consisted of a total of five vertical steel straps followed by sixteen horizontal steel straps that were installed at every longitudinal and transverse layer along the top and bottom thirds of the panel, as shown in Figure 4.28 (a). The member was subjected to a driver pressure of 623.3 kPa, which resulted in a reflected pressure and impulse of 69.9 kPa, and 1,295.7 kPa-ms, respectively. The specimen reached a maximum displacement of 208.9 mm in 57.8 ms. Figure 4.28 (b) illustrates the flexural failure that occurred at the mid-span of the panel.

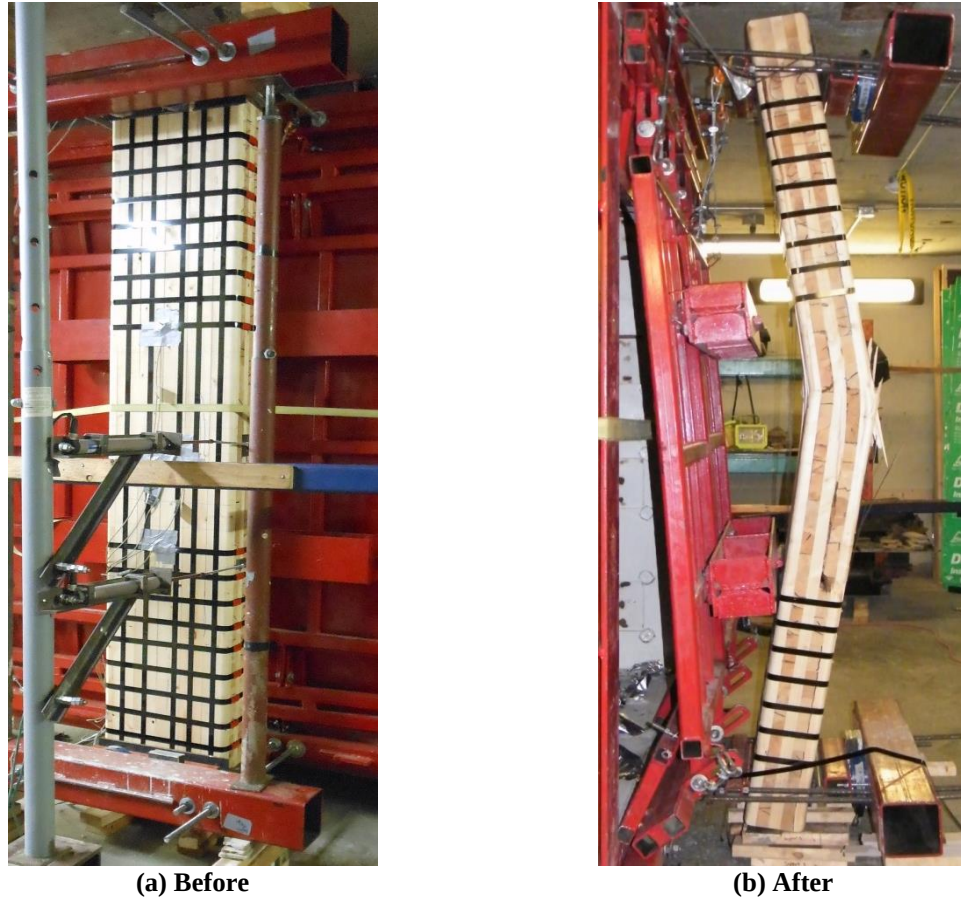


Figure 4.28: Specimen No. 5

The failure mode was very similar to that observed for CLT5-1. Flexural failure was evident at the mid-span of the panel, which initiated at a finger joint, at a time and corresponding deflection of 30.0 ms, and 133.3 mm, respectively. The absence of horizontal straps in the middle third of the specimen resulted in more deflection of the wood at that particular location. While rolling shear was suppressed at the end of the member (confined by horizontal straps) a significant amount of rolling shear was observed at the location of failure, as shown in Figure 4.29.



Figure 4.29: CLT5-3 Flexural Failure

Longitudinal shear failure was also observed, as demonstrated in Figure 4.30.



(a) Bottom Face



(b) Top Face

Figure 4.30: CLT5-3 Shear Failure

The horizontal straps generally experienced no damage; however straps 9 and 10 snapped off and were found on the ground after the test. The vertical straps underwent significant damage. On the tension face, straps A through C were found loose, strap D was very tight, and strap E had snapped at the location of finger joint on laminate 5. On the compression face, all straps experienced some slack which created a gap between the panel of approximately 4.5 cm at the location of failure. Furthermore, straps A through C snapped

at the bottom of the panel, while strap E experienced some slack; due to the rupture at the location of failure.

4.5.2.6 CLT5-4

Specimen 6, CLT5-4, consisted of a total five vertical steel straps followed by thirteen horizontal steel straps that were installed at every longitudinal and every other transverse layer along the clear span of the panel. The member was subjected to a driver pressure of 650.9 kPa, which resulted in a reflected pressure and impulse of 66.4 kPa, and 1,138.1 kPa-ms, respectively. The panel reached a maximum displacement of 223.8 mm in 42.8 ms. Figure 4.31 illustrates the flexural failure that occurred at the top third of the panel.

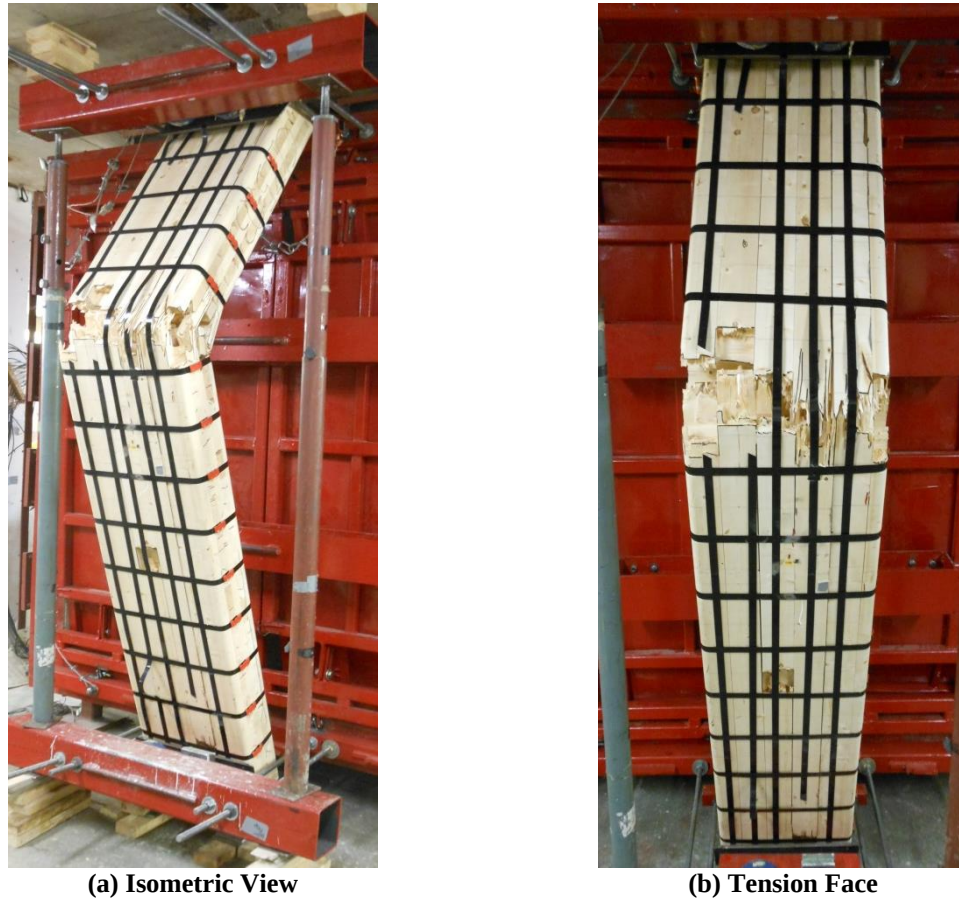


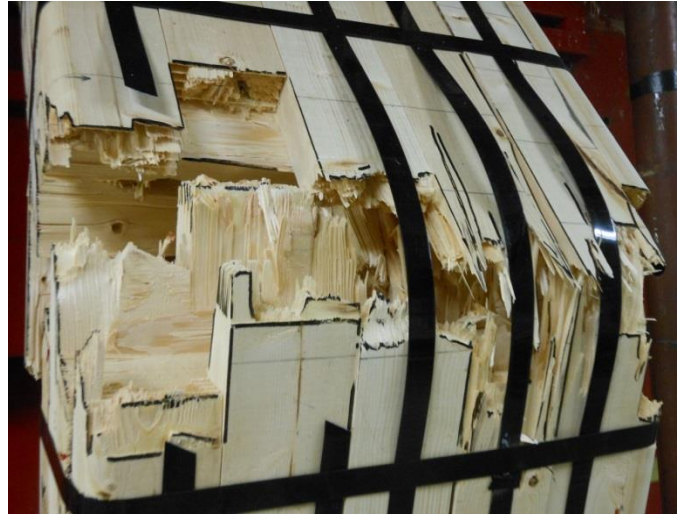
Figure 4.31: Specimen No. 6

The flexure failure developed at the finger joint, as shown in Figure 4.32 (a). The failure propagated transversely through the remaining longitudinal laminates hence fracturing

the outermost layer (Figure 4.32 (b)). Laminates 1 and 2 failed precisely at a finger joint; while laminates 3 and 4 ruptured at the bottom third, and the bottom face, respectively thus creating a gap of approximately 2 cm. Rolling shear was minimal throughout the span of the panel.



(a) Finger Joint on Laminate 1



(b) Tension Face

Figure 4.32: CLT5-4 Failure Mode



Figure 4.33: CLT5-4 Compression Failure

Although the horizontal straps were spaced at 180 mm (7") centre to centre, no damage in the straps was observed. The horizontal straps helped confine the damage as well as the vertical straps, thereby reducing the amount of debris. Contrarily, the vertical straps underwent various levels of damage. Straps A and B snapped at the location of the

flexural failure. All straps experienced some slack, as shown in Figure 4.32 (b). Strap D failed and broke towards the bottom portion of the member. Strap E, although it seems like it was in pristine condition, snapped at the seal on the compression face at the top of the specimen.

4.5.2.7 CLT5-5

Specimen 7, CLT5-5, consisted of vertical straps fastened directly to the tension face. This specimen consisted of a total of nine vertical straps followed by eight horizontal straps along the clear span of the panel. The aim was to install two vertical straps per longitudinal laminate but due to the fabrication process this was unattainable hence the nine straps labelled A through J. The horizontal straps were not meant to confine the panel, but rather to secure the top and bottom ends and contain the vertical straps at the mid-span and adjacent to the connections at both ends. The member was subjected to a driver pressure of 632.9 kPa, which resulted in a reflected pressure and impulse of 65.7 kPa, and 871.1 kPa-ms, respectively. The panel reached a maximum displacement of 256.2 mm in 50.0 ms. Figure 4.34 (b) illustrates the flexural failure that occurred between the top third and mid-span of the panel.

From video review, it was observed that the panel experienced initial failure at a time and corresponding deflection of 14.0 ms, and 12.9 mm, respectively. The failure propagated through the entire width of the panel, fracturing all the longitudinal laminates on the tension face. Laminates 1 through 3, as shown in Figure 4.35 (a), failed at finger joints. The failure was contained within a small region. This was also the case on the compression side where an almost horizontal failure plane could be observed, as shown in Figure 4.36. Furthermore, the development of shear failure, at the top of the panel, occurred at time of 18.0 ms and deflection of 33.0 mm. There was no sign of damage below the mid-span of the panel.



(a) Isometric View



(b) Right Face

Figure 4.34: Specimen No. 7



(a) Tension Face



(b) Compression Face

Figure 4.35: CLT5-5 Failure Mode



Figure 4.36: CLT5-5 Compression Failure

The horizontal straps located at the mid-span, at the connection detail at the bottom of the panel, and the bottom end of the panel remained intact as their purpose was to prevent rolling shear and shear failure. Vertical straps A through H failed at the seals located at the top of the panel. These straps were found hanging as shown in Figure 4.34 (a). There was no indication of damage to the seals; nonetheless a significant amount of wood crushing was observed which could be a result of the crimping action at the time of installation or a direct result of testing.

Furthermore, there was no significant damage in the connections, welds, or screws.

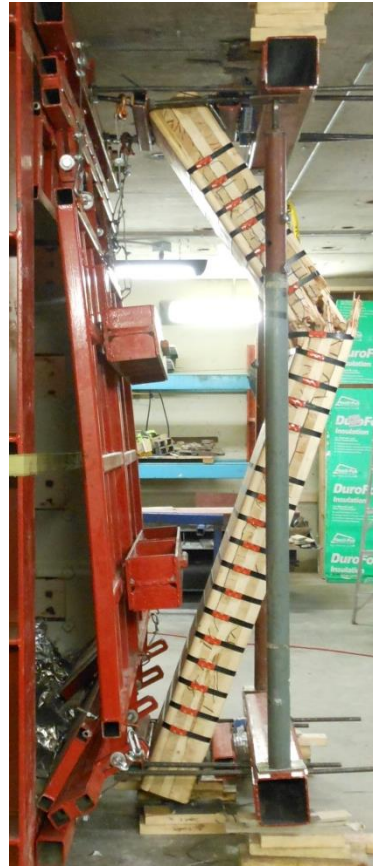
4.5.2.8 CLT5-D1

Specimen 8, CLT5-D1, consisted of twenty-four horizontal steel straps that were installed at every transverse layer along the clear span of the panel, as shown in Figure 4.37 (a). The member was subjected to a driver pressure of 439.9 kPa, which resulted in a reflected pressure and impulse of 53.2 kPa, and 811.8 kPa-ms, respectively. The panel reached a maximum displacement of 236.8 mm in 46.6 ms. Figure 4.37 (b) illustrates the flexural failure that occurred at the top third of the panel.

Flexural failure developed at an area of existing damage (a finger joint located on laminate 1) and propagated transversely along the longitudinal laminates of the outermost layer on the tension face, as illustrated in Figure 4.38. The damage made its way across the member depth to the compression side, as shown in Figure 4.39. Only a small amount of additional rolling shear cracks were observed as the horizontal straps confined the laminations.

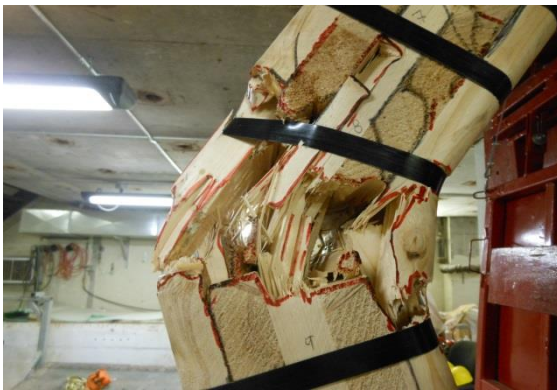


(a) Before



(b) After

Figure 4.37: Specimen No. 8



(a) Right Face



(b) Tension Face

Figure 4.38: CLT5-D1 Failure Mode



Figure 4.39: CLT5-D1 Compression Failure

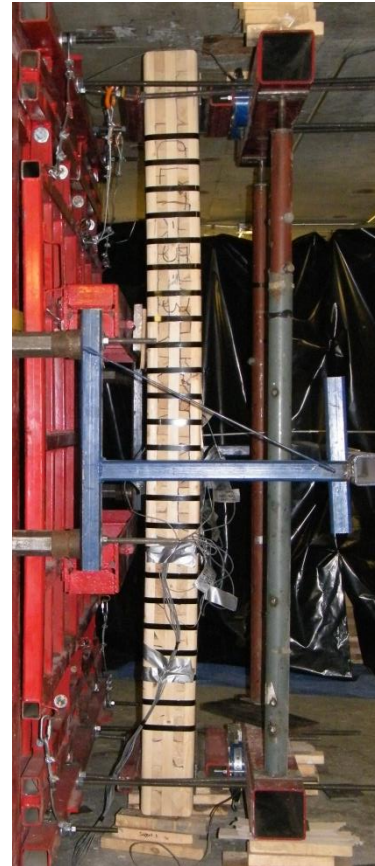
The horizontal straps remained intact and no damage was observed; however minor shifting in the location of some of the straps was observed. Straps 1 to 7 appeared to have shifted upwards. There was an obvious upward shift of strap 8 on the right side of the tension face; in addition to wood crushing on the compression face. Given the condition of the panel prior to testing, at the time of installation, the horizontal straps were able to bring the panel back to its original state.

4.5.2.9 CLT5-D2

Specimen 9, CLT5-D2, consisted of a total of five vertical steel straps followed by twenty-four horizontal steel straps that were installed at every longitudinal and transverse layer along the clear span of the panel. CLT5-D2.1 was subjected to a driver pressure of 217.9 kPa, which resulted in a reflected pressure and impulse of 33.5 kPa, and 437.6 kPa-ms, respectively. The panel reached a maximum displacement of 64.6 mm in 41.0 ms. Initial failure occurred at a time and corresponding deflection of 14.0 ms, and 8.0 mm, respectively. Approximately 20 ms later the transverse and longitudinal layers began to open up. No additional rolling shear cracks were observed. Little residual displacement was observed following the test. The horizontal and vertical straps did not display any signs of damage; in fact they all remained intact and in place. Since the panel returned to its original position, as shown in Figure 4.40, it was decided to re-test the specimen.



(a) Before



(b) After

Figure 4.40: Specimen No. 9 – Shot 1

CLT5-D2.2 was subjected to a driver pressure of 380.6 kPa, which resulted in a reflected pressure and impulse of 51.2 kPa, and 702.8 kPa-ms, respectively. The panel reached a maximum displacement of 163.2 mm in 56.6 ms. Figure 4.41 (b) illustrates the flexural failure that occurred at the top third of the panel. Initial failure was observed at a time and deflection of 26.0 ms, and 92.8 mm, respectively. The panel failed at the exact location where, in the previous shot, had experienced initial cracking (Figure 4.42). Little damage was found on the compression face and very little rolling shear cracks were observed.



(a) Before



(b) After

Figure 4.41: Specimen No. 9 – Shot 2



(a) Isometric View



(b) Right Face

Figure 4.42: CLT5-D2 Flexural Failure

With the exception of strap 10, the horizontal straps were undamaged. Strap 10 broke at a time of 38.0 ms and corresponding deflection of 140.0 mm. Conversely, the vertical straps underwent some damage. Although straps A, B, and E remained intact, straps C and D failed.

4.5.2.10 CLT7-D1

Specimen 10, CLT7-D1, consisted of a total of twenty-four horizontal steel straps that were installed at every transverse layer along the clear span of the panel. The member was subjected to a driver pressure of 413.0 kPa, which resulted in a reflected pressure and impulse of 58.3 kPa, and 693.6 kPa-ms, respectively. The panel reached a maximum displacement of 234.1 mm in 84.2 ms. Figure 4.43 illustrates the flexural failure that occurred at the top third of the panel.



(a) Isometric View



(b) Tension Face

Figure 4.43: Specimen No. 10

This member had a very similar failure mode to that observed for CLT5-D1. The failure in the panel occurred at the same location of the existing damage. The failure was contained in a small region, as shown in Figure 4.44. It was clear that the finger joints located on the left and right side of the panel were areas of weakness. Failure on the compression face was also limited to a very small area, as seen in Figure 4.45 (a). Although additional rolling shear cracks were limited, the failure shifted to longitudinal shear failure, (Figure 4.45 (b)).



(a) Left Face



(b) Right Face

Figure 4.44: CLT7-D1 Flexural Failure



(a) Compression Failure



(b) Shear Failure

Figure 4.45: CLT7-D1 Additional Failure Modes

The horizontal straps did not experience any damage. Strap 7 did slide downwards leaving black paint on the right side of the panel. Similarly, straps 8 and 9 shifted upwards and downwards, on the left and right side of the panel, respectively.

4.5.2.11 Summary

The dynamic resistance-displacement curves, as illustrated in Figures 4.46 – 4.49, for each of the aforementioned tests have been developed and plotted with members of identical cross-sectional area and initial condition. Table 4.11 summarizes key

experimental data acquired from each destructive test. Parameters such as driver pressure, reflected pressure, reflected impulse, maximum displacement, maximum dynamic resistance, ductility ratio, and stiffness have been tabulated. Table 4.12 summarizes the recorded strain data for the same tests as described in Table 4.11. Furthermore, Appendix D – Phase 1 Dynamic Test Results can be referred to for more information regarding each destructive dynamic test, such as: test details, panel nomenclature, physical properties, and a series of relationships.

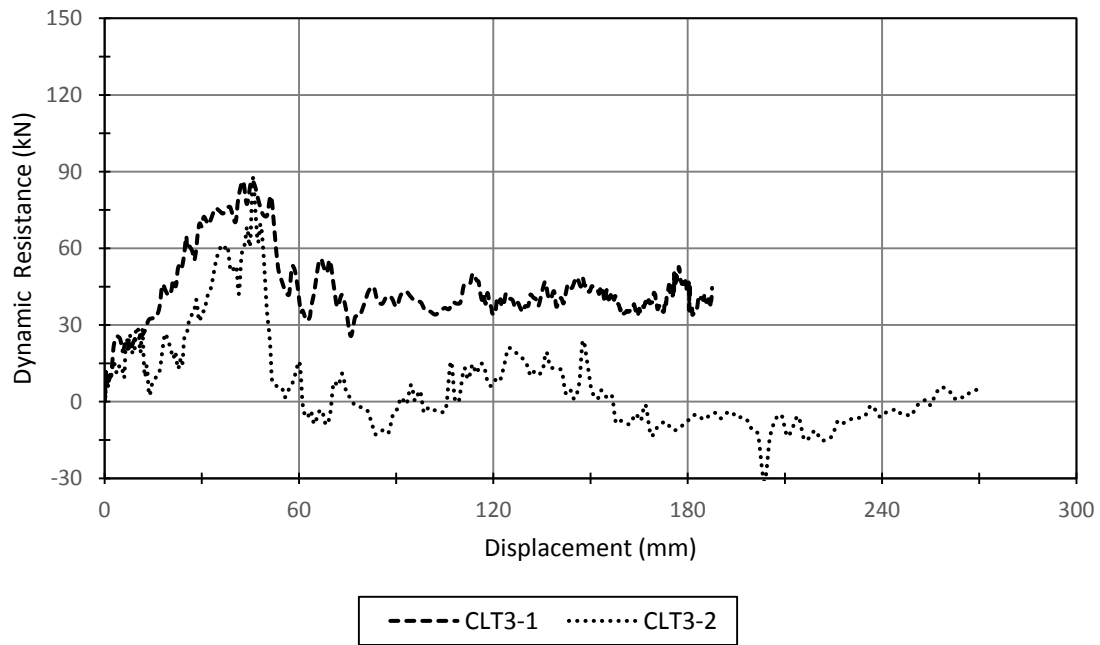


Figure 4.46: Dynamic Resistance-Displacement Curve – Undamaged 3-Ply's

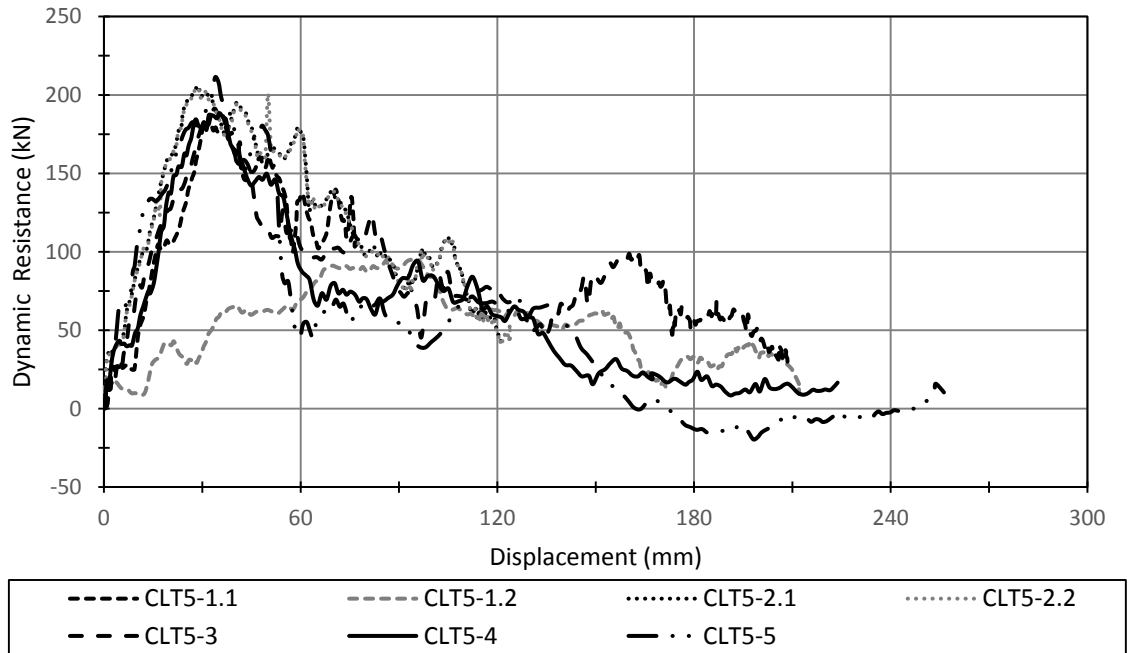


Figure 4.47: Dynamic Resistance-Displacement Curve – Undamaged 5-Ply's

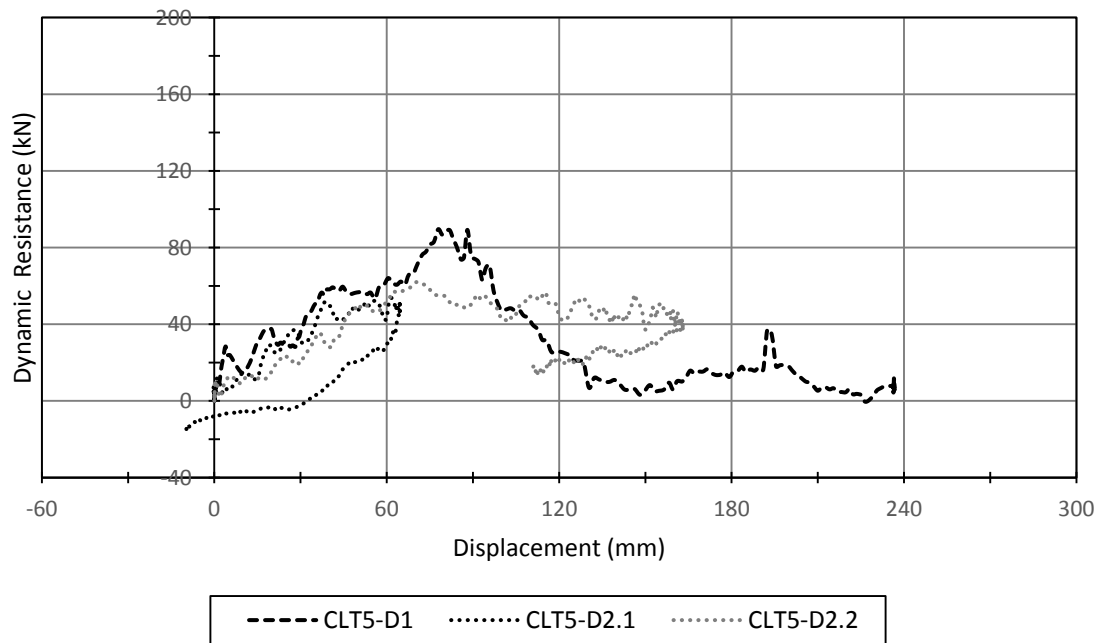


Figure 4.48: Dynamic Resistance-Displacement Curve – Damaged 5-Ply's

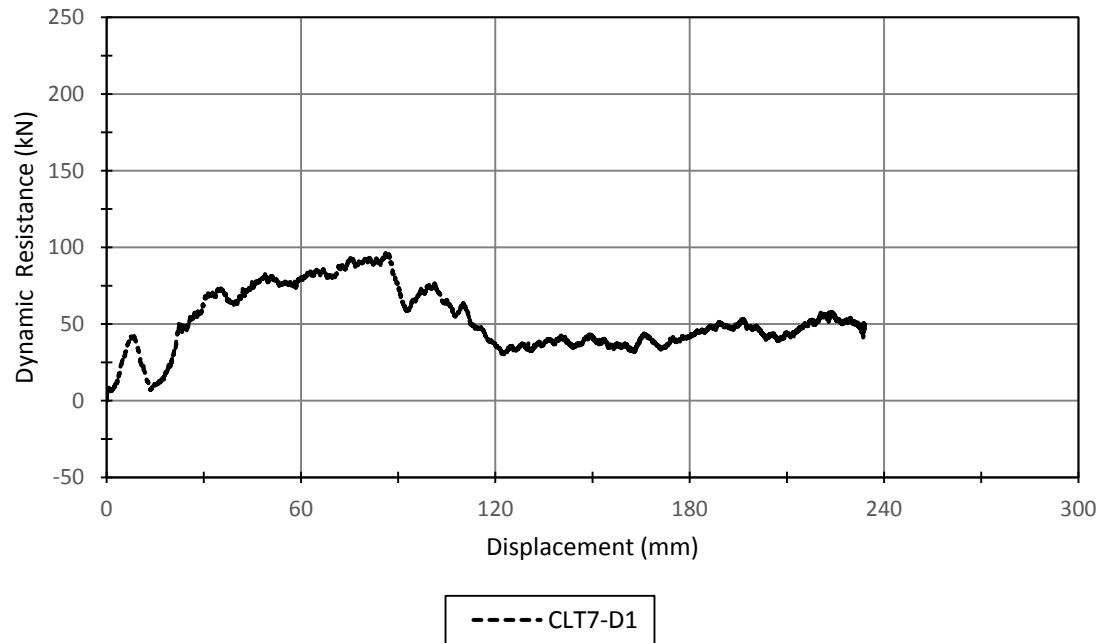


Figure 4.49: Dynamic Resistance-Displacement Curve – Damaged 7-Ply

Table 4.11: Destructive Dynamic Test Results for Phase 1 – Part 1

Test Name	Driver Pressure	P_R^1	I_R^2	Δ_{max}^3	$t_{\Delta max}^4$	R_{max}^5	Δ_{Rmax}^6	t_{Rmax}^7	R_f^8	Δ_{Rf}^9	μ^{10}	K^{11}
	<i>kPa</i>	<i>kPa</i>	<i>kPa – ms</i>	<i>mm</i>	<i>ms</i>	<i>kN</i>	<i>mm</i>	<i>ms</i>	<i>kN</i>	<i>mm</i>	-	<i>kN/mm</i>
CLT3-1	424.0	53.7	577.7	187.5	66.0	86.4	45.3	15.4	44.5	115.9	1.2	2.0
CLT3-2	353.7	52.5	594.6	273.8	51.8	87.3	45.8	18.8	87.3	156.5	1.1	1.3
CLT5-1.1	597.1	71.0	881.0	76.9	26.8	191.3	33.7	13.0	155.4	53.5	2.3	5.5
CLT5-1.2	455.1	58.2	829.2	212.1	56.4	96.3	87.0	23.0	57.8	151.2	1.8	1.1
CLT5-2.1	537.1	68.6	983.1	123.9	50.8	206.0	28.5	12.6	168.6	62.0	2.9	7.7
CLT5-2.2	595.0	62.4	981.4	123.9	52.2	204.5	28.9	12.6	163.0	61.8	3.6	7.4
CLT5-3	623.3	69.6	1,295.7	208.9	57.8	182.9	32.5	12.6	104.2	85.5	2.7	5.9
CLT5-4	650.9	66.4	1,138.1	223.8	42.8	188.4	35.2	16.0	73.6	125.2	1.7	6.4
CLT5-5	632.9	65.7	871.1	256.2	50.0	211.3	33.9	18.2	54.7	141.4	1.6	6.9
CLT5-D1	439.9	53.2	811.8	236.8	46.6	89.6	77.9	22.6	89.6	77.9	1.4	1.1
CLT5-D2.1	217.9	33.5	437.6	64.6	41.0	53.8	61.3	34.4	53.8	61.3	1.0	1.0
CLT5-D2.2	380.6	51.2	702.8	163.2	56.6	62.0	70.3	22.8	62.0	70.3	2.2	0.8
CLT7-D1	413.0	58.3	693.6	234.1	84.2	96.5	86.3	25.0	96.5	86.3	1.3	1.2

¹Reflected Pressure²Reflected Impulse³Maximum Recorded Displacement⁴Time to Maximum Recorded Displacement⁵Maximum Dynamic Resistance⁶Deflection at Maximum Dynamic Resistance⁷Time to Maximum Dynamic Resistance⁸Ultimate Failure Resistance⁹Ultimate Failure Deflection¹⁰Ductility Ratio¹¹Stiffness

Table 4.12: Destructive Dynamic Test Results for Phase 1 – Part 2

Test Name	$\varepsilon_{ft}^1 \times 10^4$	t_{eft}^2	Δ_{eft}^3	$\varepsilon_{fc}^4 \times 10^4$	t_{efc}^5	$\varepsilon_{fh}^6 \times 10^4$	t_{efh}^7	$\varepsilon_{fv}^8 \times 10^4$	t_{efv}^9
	mm/mm	ms	mm	mm/mm	ms	mm/mm	ms	mm/mm	ms
CLT3-1	45.52	33.6	133.9	-32.10	36.2	-	-	N/A	N/A
CLT3-2	31.73	18.4	44.0	-29.71	30.8	N/A	N/A	N/A	N/A
CLT5-1.1	46.23	15.8	47.44	-43.01	15.6	-	-	-	-
CLT5-1.2	19.35	30.0	129.24	-31.20	23.4	-	-	-	-
CLT5-2.1	31.42	19.0	54.42	-30.32	27.2	-	-	19.68	38.0
CLT5-2.2	31.12	19.4	56.41	-30.23	27.2	-	-	19.75	38.2
CLT5-3	18.63	8.6	18.50	-52.55	33.8	-	-	63.11	51.4
CLT5-4	19.63	20.2	60.48	-21.46	36.0	-	-	37.83	24.2
CLT5-5	29.46	18.0	33.02	-102.08	31.4	-	-	0.375	71.0
CLT5-D1	12.29	20.2	60.54	-20.55	22.0	-	-	N/A	N/A
CLT5-D2.1	3.52	25.2	39.15	-	-	10.65	35.2	-	-
CLT5-D2.2	-	-	-	-	-	-	-	-	-
CLT7-D1	20.13	19.65	55.06	-5.808	22.7	-	-	N/A	N/A

¹Strain at Wood Tensile Rupture

²Time to Wood Tensile Rupture

³Displacement at Wood Tensile Rupture

⁴Maximum Wood Compressive Strain

⁵Time to Maximum Wood Compressive Strain

⁶Horizontal Strap Tensile Failure Strain

⁷Time to Horizontal Strap Tensile Failure Strain

⁸Vertical Strap Tensile Failure Strain

⁹Time to Vertical Strap Tensile Failure Strain

4.5.3 Phase 2: Glass Fibre-Reinforced Polymer

4.5.3.1 CLT3-3

Specimen 11, CLT3-3, consisted of two uniaxial $[0^\circ]$ layers in a U configuration along the clear span of the panel. The member was subjected to a driver pressure of 424.0 kPa, which resulted in a reflected pressure and impulse of 65.8 kPa, and 747.2 kPa-ms, respectively. The specimen reached a maximum displacement of 217.1 mm in 46.6 ms. Figure 4.50 highlights the damage sustained following the blast wave. The deflected shape of the member indicated that failure occurred at the top third, at an approximate time of 24.0 ms, and deflection of 62.6 mm.

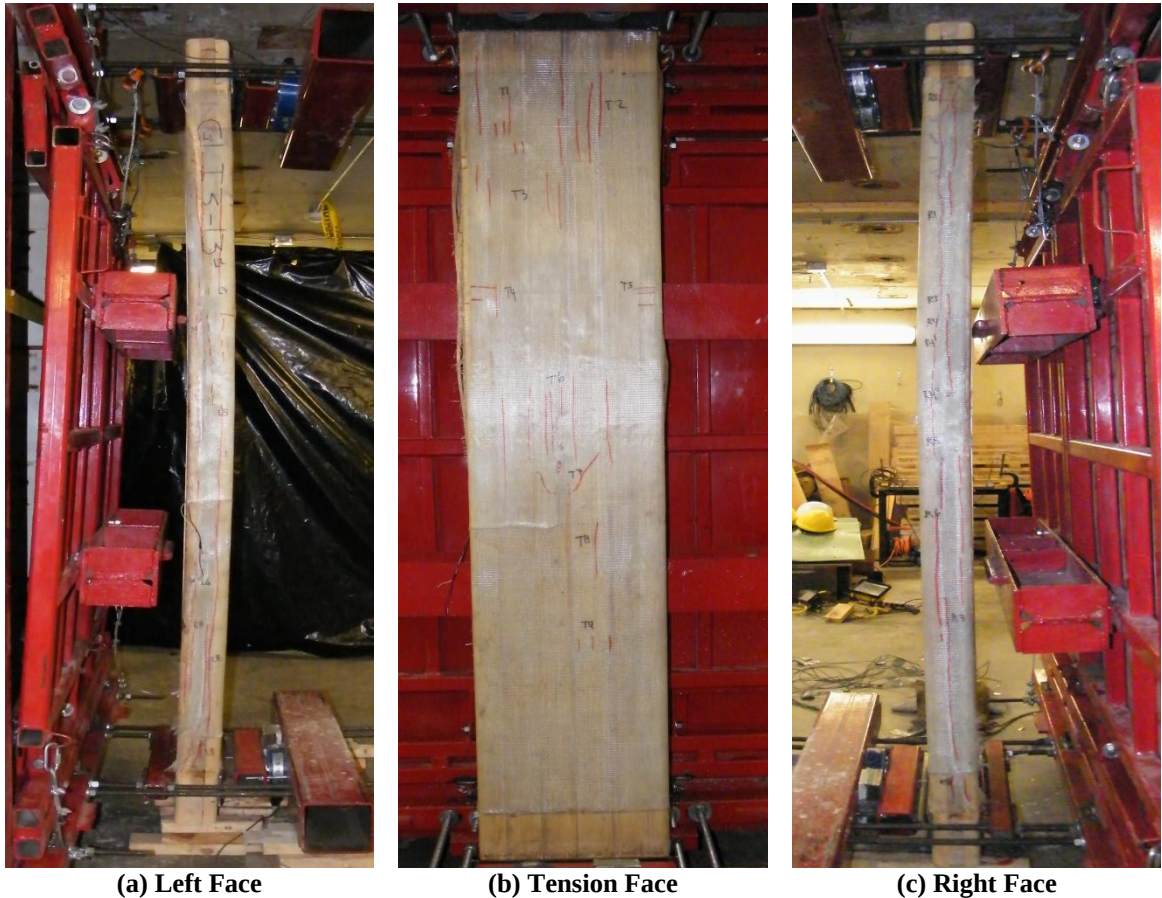


Figure 4.50: Specimen No. 11

The failure initiated on the left face at a time and corresponding deflection of 14.0 ms, and 19.1 mm, respectively. A noticeable amount of torn GFRP was observed in the

longitudinal direction, with limited amount of splitting in the CLT member. Stretching in the GFRP fabric was visible (Figure 4.50 (b)). Slight separation between the GFRP and wood surface was also observed. The right face of the panel experienced very similar failure modes as the left face (e.g. tears, delamination, splitting, and white marks indicating the fabric stretching).

Upon removal of the GFRP, there were no apparent signs of flexural failure; however, slight longitudinal and transverse cracks had formed, particularly on laminates 3 and 4, as shown in Figure 4.51 (a). There was also no evidence of compression failure. Slight separation between the longitudinal and transverse laminates was observed, as highlighted in Figure 4.52. This was primarily a result of some rolling shear cracks and minor longitudinal shear failure, as shown in Figure 4.53.



(a) Tension Face



(b) Compression Face

Figure 4.51: CLT3-3 Failure Mode



(a) Left Face



(b) Right Face

Figure 4.52: CLT3-3 Delamination of Laminates



Figure 4.53: CLT3-3 Shear Failure

4.5.3.2 CLT3-4

Specimen 12, CLT3-4, consisted of two biaxial $[0/90^\circ]$ layers in a CM configuration along the clear span of the panel. The member was subjected to a driver pressure of 430.9 kPa, which resulted in a reflected pressure and impulse of 56.7 kPa, and 841.6 kPa-ms, respectively. The specimen reached a maximum displacement of 215.0 mm in 50.6 ms. Figure 4.54 highlights the damage sustained following the shock wave. The specimen experienced significant amount of deflection, as shown in Figure 4.51 (c). The initial

panel failure was observed at a time and corresponding deflection of 62.0 ms, and 210.9 mm, respectively.

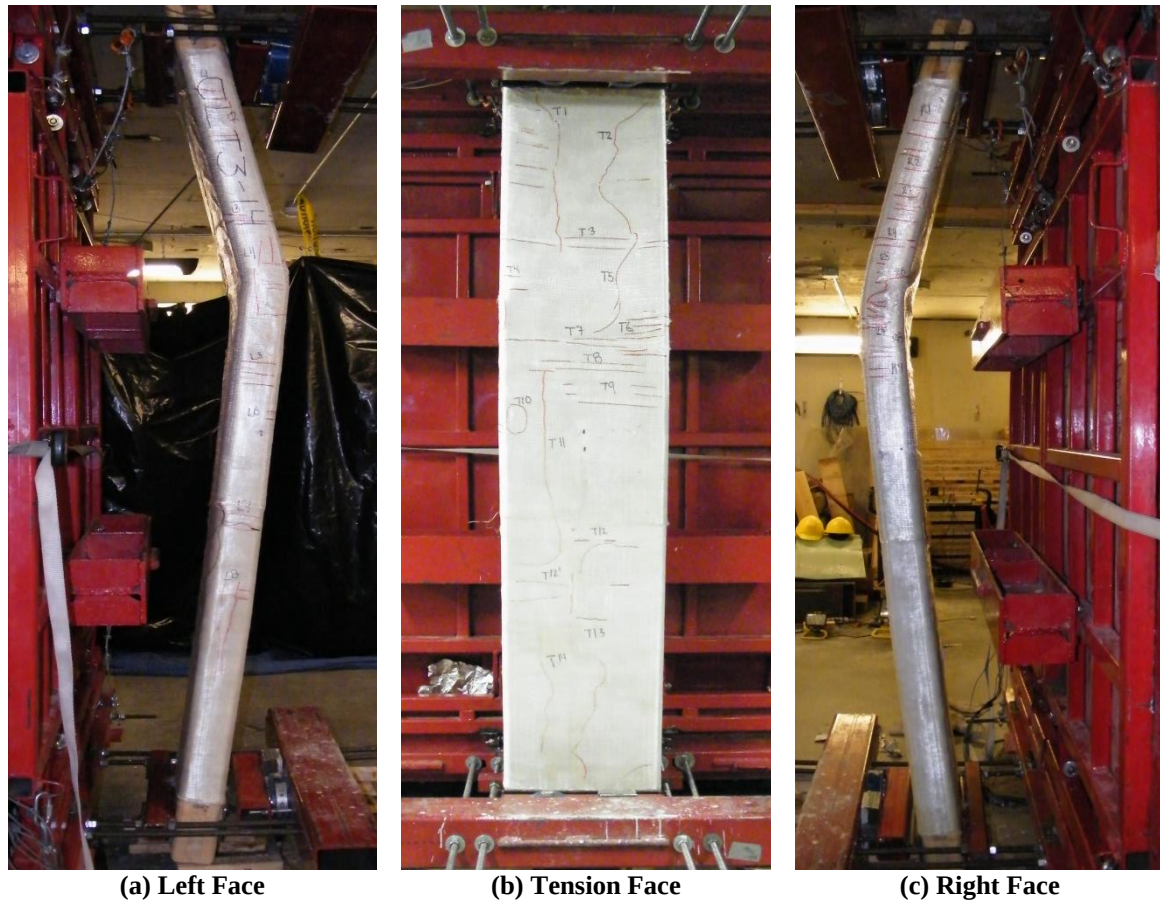


Figure 4.54: Specimen No. 12

The GFRP damage was concentrated at the top third of the member where flexural failure was evident. Wrinkling of the GFRP fabric was evident on the compression face and local separation around the failure region was observed on the tension face. Overall, the GFRP fabric experienced significant amount of stretching, especially along the 90° direction.

Upon removal of the GFPR, it was clear that flexural failure had been reached, as shown in Figure 4.55 (a), including rupture of the five longitudinal laminates. The compression face experienced some damage that propagated across the entire member width, as shown

in Figure 4.55 (b). Permanent deformations were observed and relatively minor longitudinal shear failure also developed (Figure 4.56).

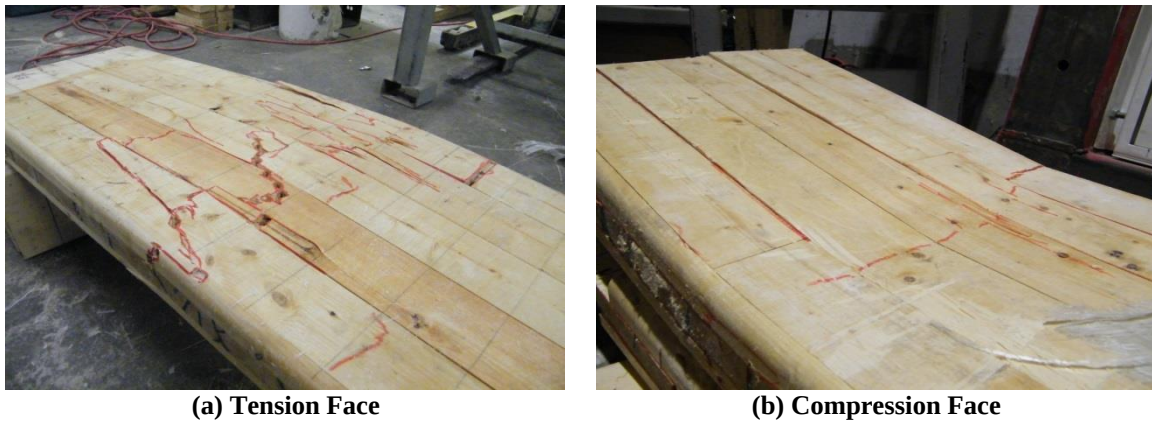


Figure 4.55: CLT3-4 Failure Mode



Figure 4.56: CLT3-4 Shear Failure

4.5.3.3 CLT3-5

Specimen 13, CLT3-5, consisted of two biaxial [$\pm 45^\circ$] layers in a CM configuration along the clear span of the panel. The member was subjected to a driver pressure of 481.9 kPa, which resulted in a reflected pressure and impulse of 60.3 kPa, and 805.7 kPa-ms, respectively. The specimen reached a maximum displacement of 180.6 mm in 59.4 ms. Figure 4.57 highlights the damage sustained by the panel following the blast wave. Failure occurred at the top third of the panel, at an approximate time of 24.0 ms, and deflection of 62.6 mm.

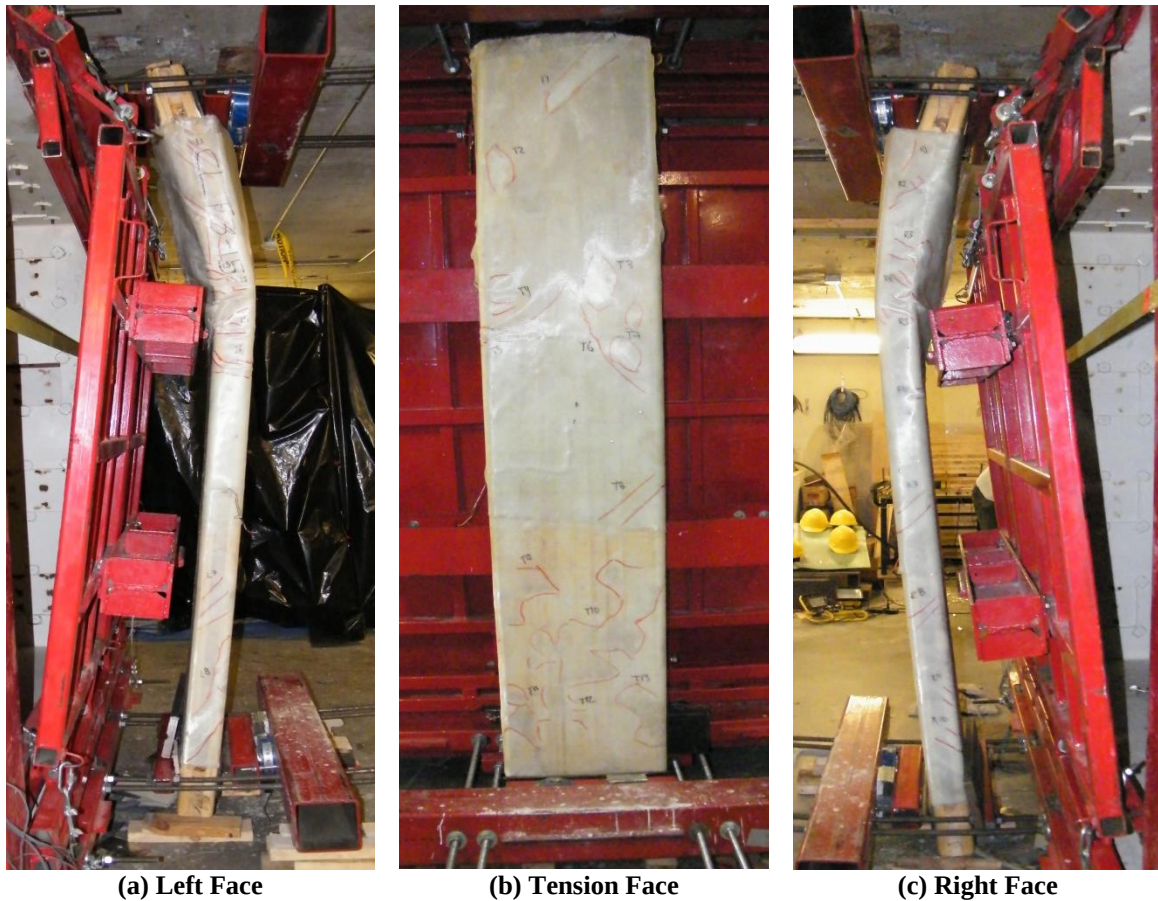
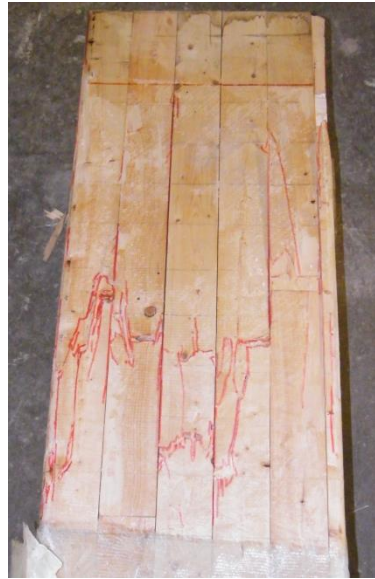


Figure 4.57: Specimen No. 13

It was quite evident that GFRP failure occurred at the top third of the panel. Significant separation between the GFRP and the CLT was observed at the failure region. Significant stretching and buckling of the GFRP was observed. There were various areas, primarily on the tension face, of white stretch marks along the $\pm 45^\circ$ direction, and splitting of the fibres. The fabric appeared distressed at the location of failure; including the adjacent faces of the specimen. Buckling of the fabric was noticed on the left and right face of the panel, which had also propagated onto the compression face.

When the GFRP fabric was removed, the flexure failure was clearly visible on the tension face, as shown in Figure 4.58 (a). Significant rolling shear and separation between the longitudinal and transverse laminates were observed, as shown in Figure 4.59. Very limited failure was observed on the compression side, as highlighted in Figure 4.58 (b).



(a) Tension Face



(b) Compression Face

Figure 4.58: CLT3-5 Failure Mode



(a) Left Face



(b) Right Face

Figure 4.59: CLT3-5 Delamination of Laminates

4.5.3.4 CLT3-6

Specimen 14, CLT3-6, consisted of one biaxial $[0/90^\circ]$ layer followed by one biaxial $[\pm 45^\circ]$ layer along the clear span of the panel. The member was subjected to a driver pressure of 418.5 kPa, which resulted in a reflected pressure and impulse of 54.6 kPa, and 737.2 kPa-ms, respectively. The specimen reached a maximum displacement of 132.1 mm in 45.2 ms. Relatively minor damage could be observed in the specimen provided

that slight permanent deformation is seen in the specimen after the test (Figure 4.60). Failure occurred at a time and deflection of 30 ms, and 108.8 mm, respectively.

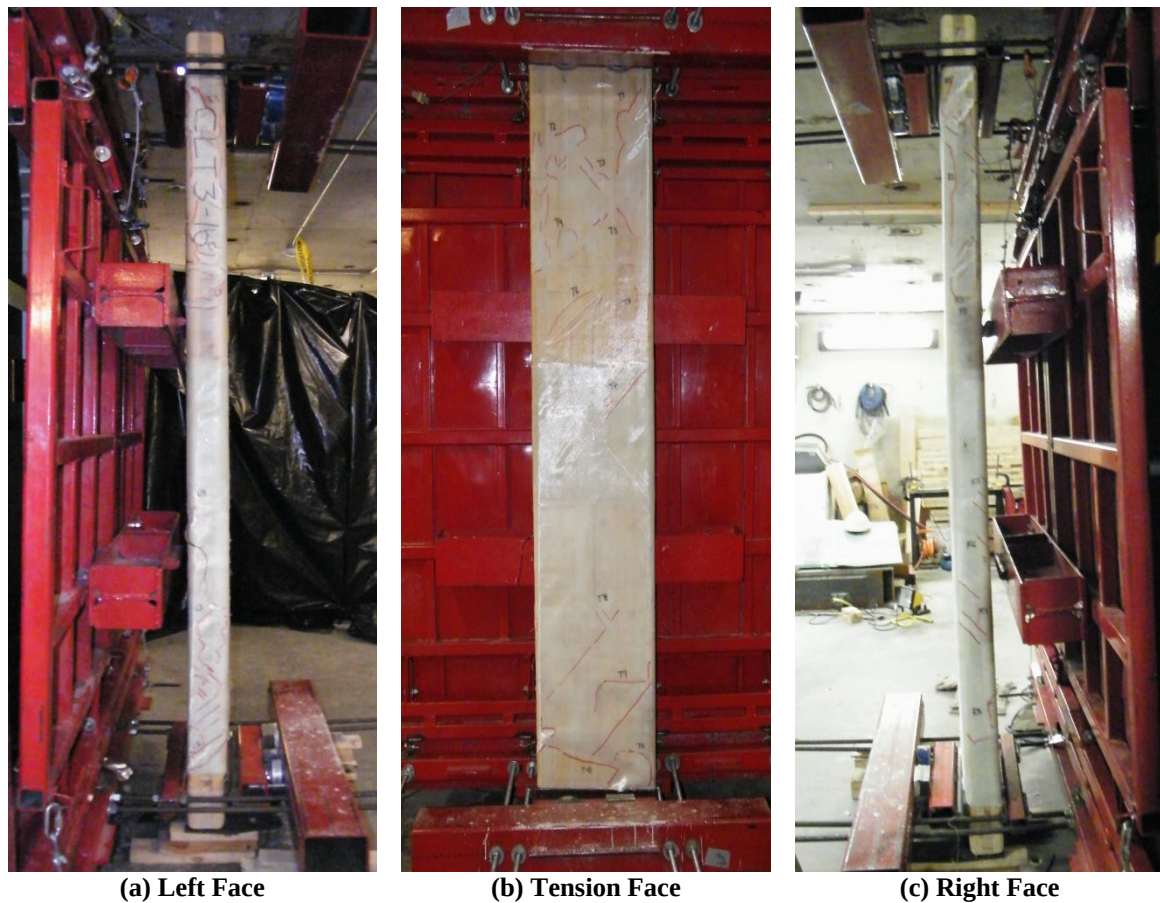


Figure 4.60: Specimen No. 14

In general, the GFRP damage was seen along all faces and localized at the top or bottom of the member. Significant amount of stretching of the fibers along the $\pm 45^\circ$ direction was observed. Localized buckling and wrinkling in the GFRP fabric was observed. Limited splitting and tearing of the fibres, and stretching of the fabric was also noted on the compression face.

Once the GFRP was removed, there was no obvious damage to the panel, as shown in Figure 4.61. Similarly, there was no detected failure on the compression face. Minor rolling shear cracks were observed on the right and left face of the panel. Any

deterioration on the surface of the wood was done upon the removal of the GFRP. Minimal amount of residual displacement was observed.

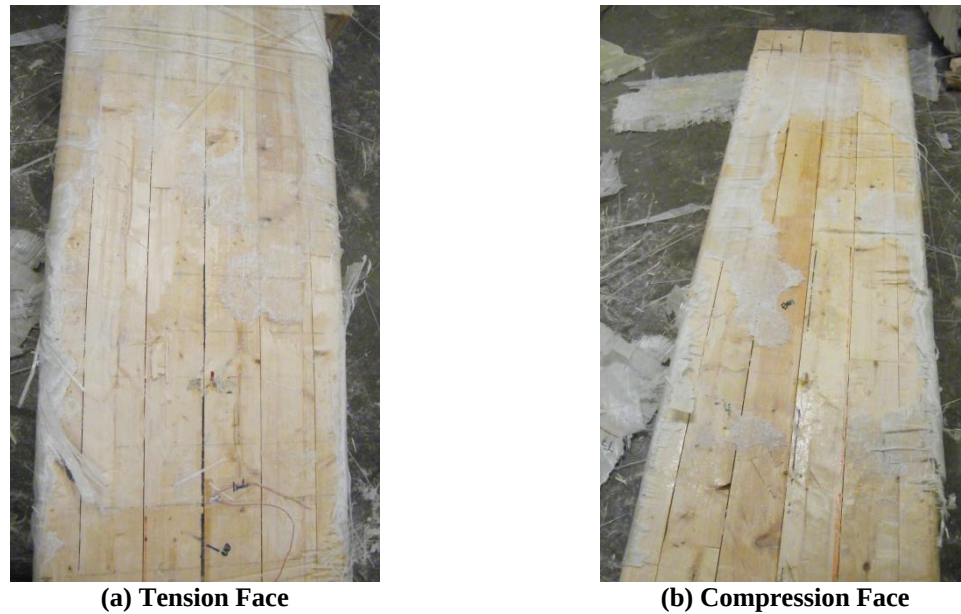


Figure 4.61: CLT3-6 Failure Mode

4.5.3.5 CLT5-6

Specimen 15, CLT5-6, consisted of two triaxial $[0^\circ, \pm 45^\circ]$ layers in a CL configuration along the clear span of the panel. The member was subjected to a driver pressure of 650.9 kPa, which resulted in a reflected pressure and impulse of 92.9 kPa, and 1,258.4 kPa-ms, respectively. The specimen reached a maximum displacement of 163.7 mm in 61.2 ms. Figure 4.62 highlights the damage sustained by the GFRP following the shock wave. The failure of this particular panel occurred at the bottom third at an approximate time and deflection of 34 ms, and 115.0 mm, respectively.

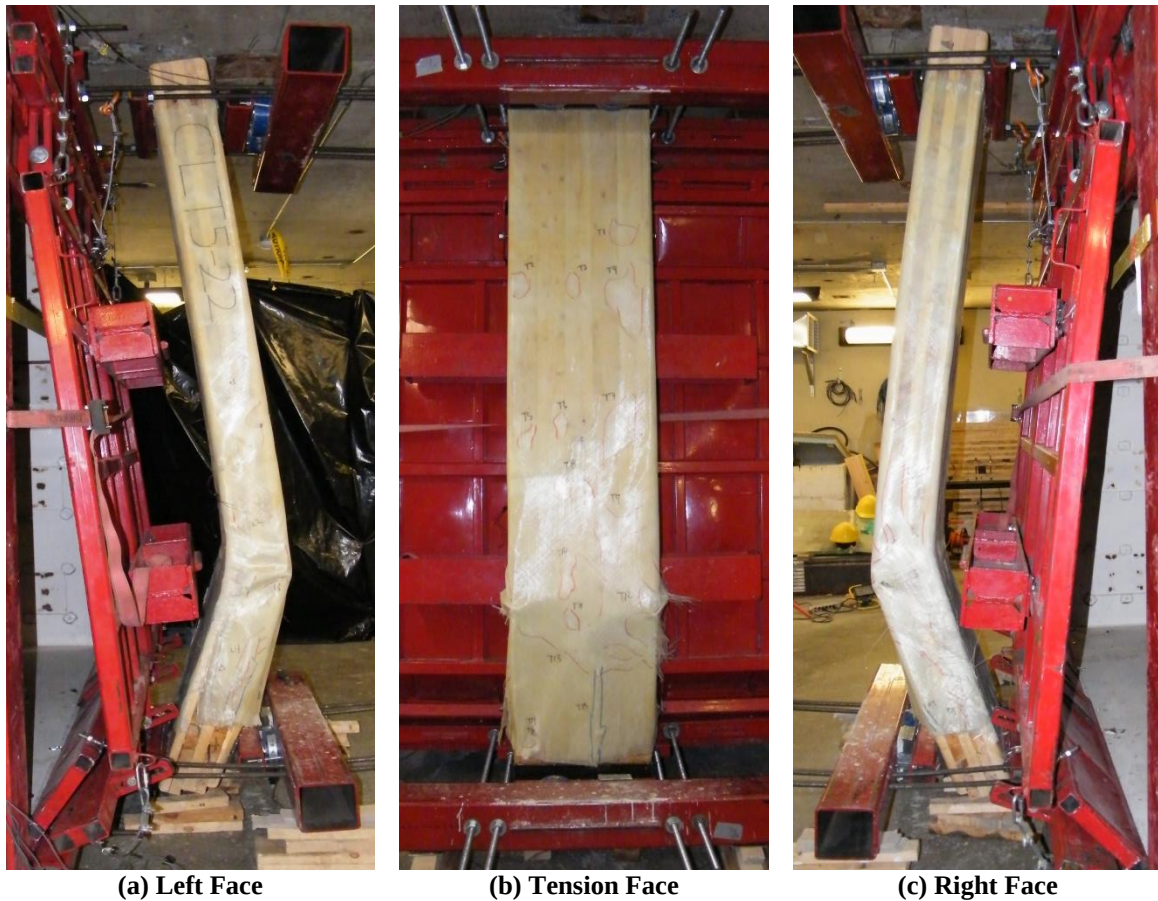


Figure 4.62: Specimen No. 15

Some separation was observed at the damaged region between the GFRP and the CLT here was a significant amount of delamination, on the tension face, from the bottom third to the bottom of the member due to the rupturing of the wood. Crimping and buckling of the fabric was obvious on the left side of the specimen. The fabric began to stretch and as a result a wavy texture developed. There was also a longitudinal rip which generated from the bottom of the panel. Slight compression failure was observed. Overall, there was an abundance of splintering and tearing of the fabric surrounding the area of primary failure.

Upon removal of the GFRP, flexural failure was noticed at the bottom third, with complete failure of the five longitudinal laminations on the tension face, as shown in Figure 4.63 (a). Little damage could be observed on the compression face, as illustrated

in Figure 4.63 (b). Separation between longitudinal and transverse laminates was observed primarily due to rolling shear failure, as seen in Figure 4.64.



(a) Tension Face



(b) Compression Face

Figure 4.63: CLT5-6 Failure Mode



(a) Left Face



(b) Right Face

Figure 4.64: CLT5-6 Delamination of Laminates

4.5.3.6 CLT3-D1

Specimen 16, CLT3-D1, consisted of two triaxial $[0^\circ, \pm 45^\circ]$ layers in a CL configuration applied along the clear span of a previously tested CLT panel, as shown in Figure 4.65. The member was subjected to a driver pressure of 321.3 kPa, which resulted in a reflected pressure and impulse of 38.8 kPa, and 544.3 kPa-ms, respectively. The

specimens reached a maximum displacement of 122.5 mm in 54.2 ms. Given the initial damaged condition of the specimen, there was a surprisingly limited amount of residual displacement.

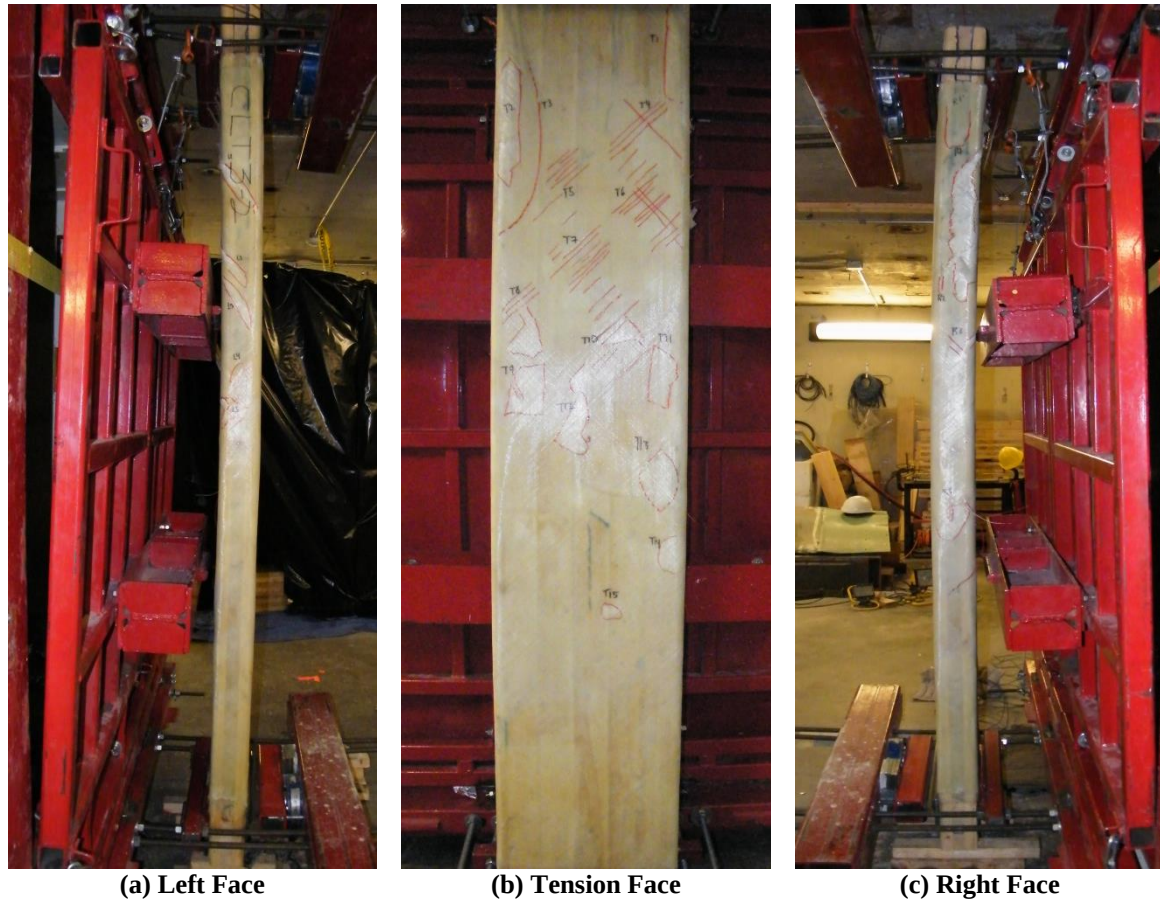


Figure 4.65: Specimen No. 16

The GFRP damage was localized at the top third of the member with little to no observable damage in the rest of the specimen. Some fibre stretching, along the $\pm 45^\circ$ direction was observed. There was significant amount of splitting of the fibres on the left and right face of the panel. The splitting resulted in tearing of the fabric which undoubtedly exposed the CLT. These tears propagated onto the compression face where additional fracturing of the fabric was seen.

Upon removal of the GFRP, there was no additional damage observed on the panel, as illustrated in Figure 4.66. The longitudinal laminations on the tension and compression

face remained intact with no observable flexure failure on the tension or compression face. No additional signs of rolling shear cracks were observed.

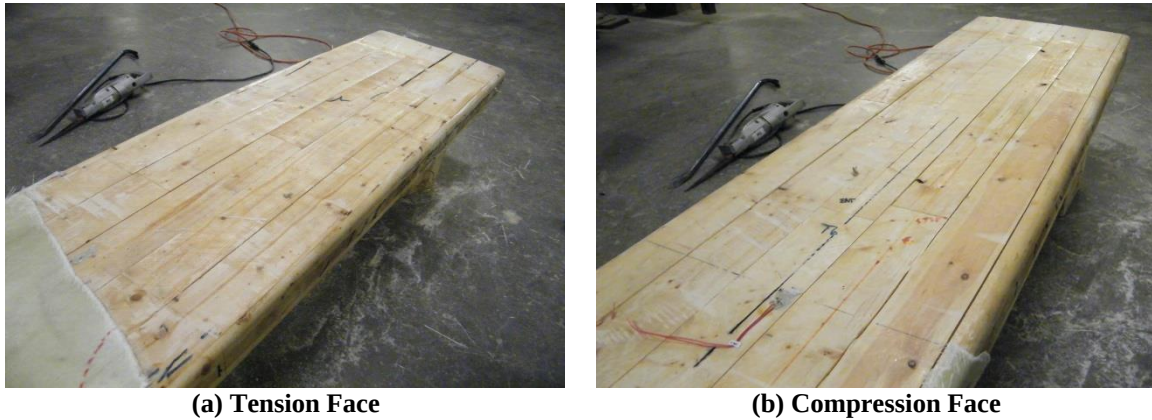


Figure 4.66: CLT3-D1 Failure Mode

4.5.3.7 CLT7-D2

Specimen 17, CLT7-D2, consisted of two biaxial $[0/90^\circ]$ layers in a CM configuration followed by two biaxial $[\pm 45^\circ]$ layers along the clear span of the panel. CLT7-D2.1 was subjected to a driver pressure of 483.3 kPa, which resulted in a reflected pressure and impulse of 59.2 kPa, and 950.7 kPa-ms, respectively. The specimen reached a maximum displacement of 44.1 mm in 26.0 ms. Minimal residual displacement was observed and the only apparent damage was slight stretching of the GFRP fabric. It was therefore deemed acceptable to re-test the specimen, as shown in Figure 4.67.



(a) Before



(b) After

Figure 4.67: Specimen No.17 – Shot 1

CLT7-D2.2 was subjected to a driver pressure of 550.2 kPa, which resulted in a reflected pressure and impulse of 55.9 kPa, and 1,030.5 kPa-ms, respectively. The specimen reached a maximum displacement of 61.7 mm in 26.2 ms. Figure 4.68 illustrates the failure that occurred at the top third of the panel at a time and deflection of 26.0 ms, and 61.7 mm, respectively. The GFRP experienced tearing and ripping of the fabric at the top of the member on the left and right face of the panel around the failure region.

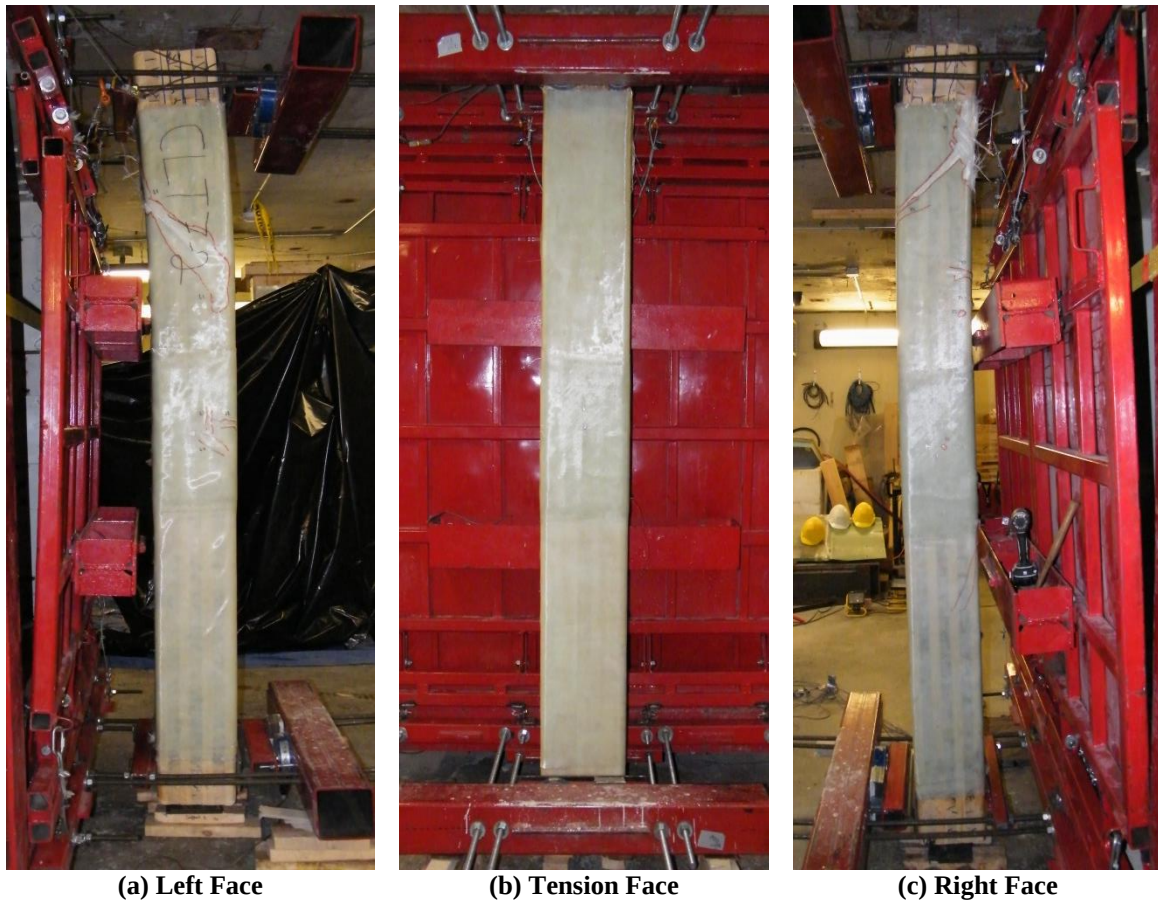


Figure 4.68: Specimen No. 17 – Shot 2

Upon removal of the GFRP, flexural failure was evident (Figure 4.69 (a)) through the rupture of laminations 1 through 3. There was very little to no damage on the compression face, as shown in Figure 4.69 (b). Aside from the expansion of previous cracks, there were no additional rolling shear cracks observed. Limited shear failure, as presented in Figure 4.70, could be observed at the top of the panel.

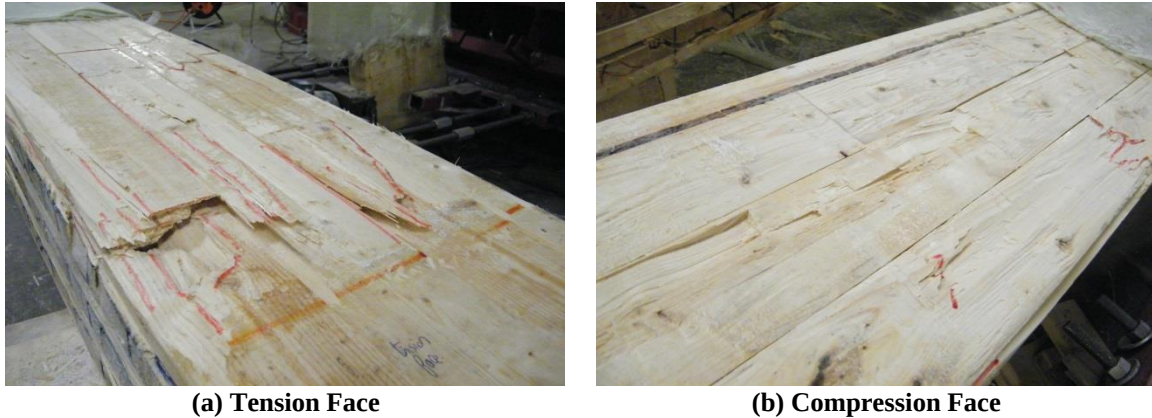


Figure 4.69: CLT7-D2 Failure Mode



Figure 4.70: CLT7-D2 Shear Failure

4.5.3.8 Summary

The dynamic resistance-displacement curves, as illustrated in Figure 4.71 – 4.74, for each of GFRP reinforced CLT specimens were developed and plotted with members of identical cross-sectional area and initial condition. Table 4.13 summarizes key experimental data acquired from each destructive test. Parameters such as driver pressure, reflected pressure, reflected impulse, maximum displacement, maximum dynamic resistance, ductility ratio, and stiffness have been tabulated. Table 4.14 summarizes the recorded strain data for the same tests as described in Table 4.13. Furthermore, Appendix E – Phase 2 Dynamic Test Results can be referred to for more information regarding each destructive dynamic test conducted for the current research study. Test details, panel nomenclature, physical properties, and a series of relationships have been reported and documented.

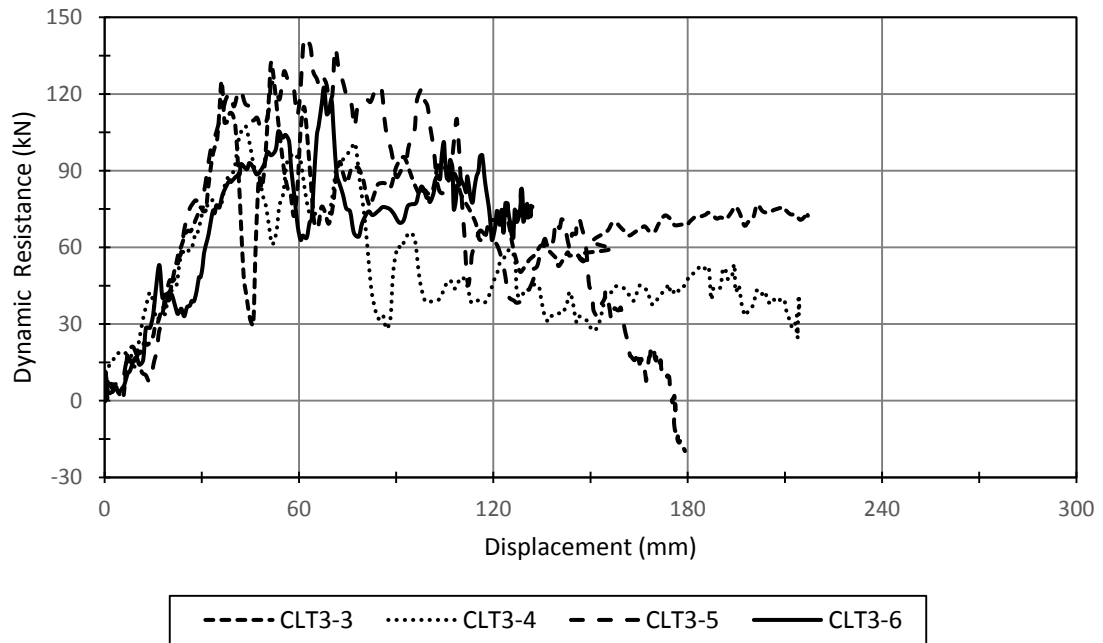


Figure 4.71: Dynamic-Resistance Curve – Undamaged 3-Ply's

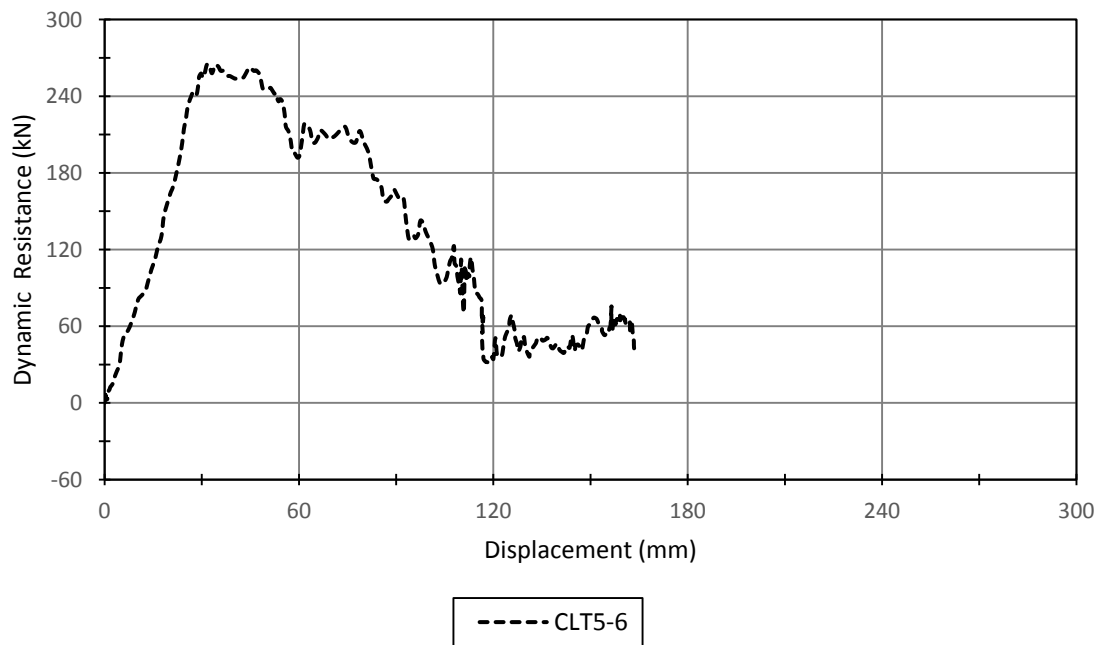


Figure 4.72: Dynamic-Resistance Curve – Undamaged 5-Ply

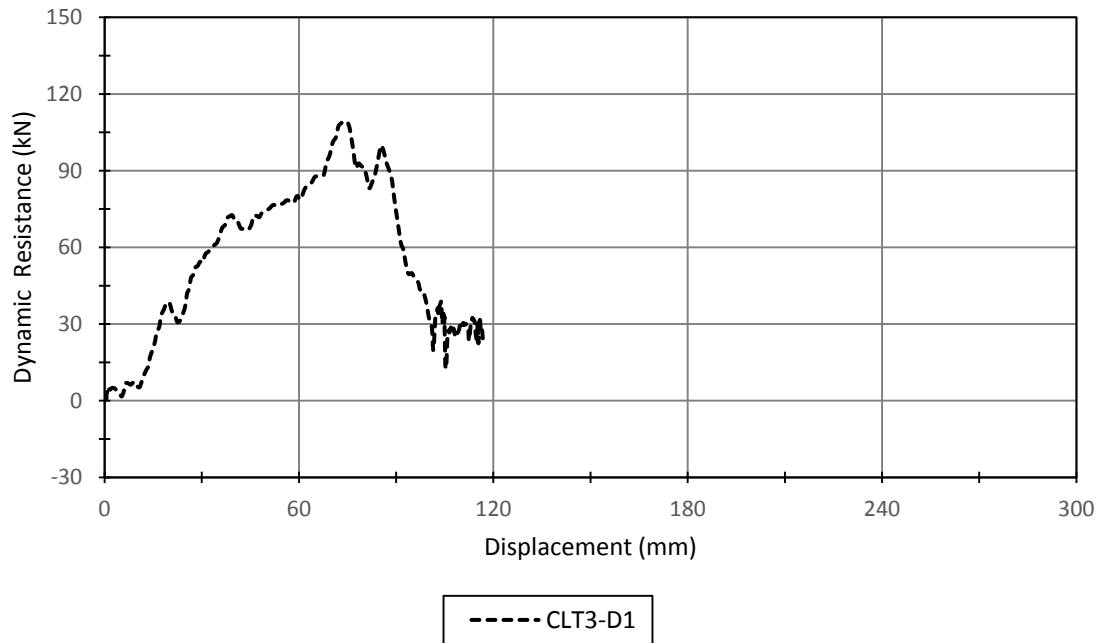


Figure 4.73: Dynamic-Resistance Curve – Damaged 3-Ply

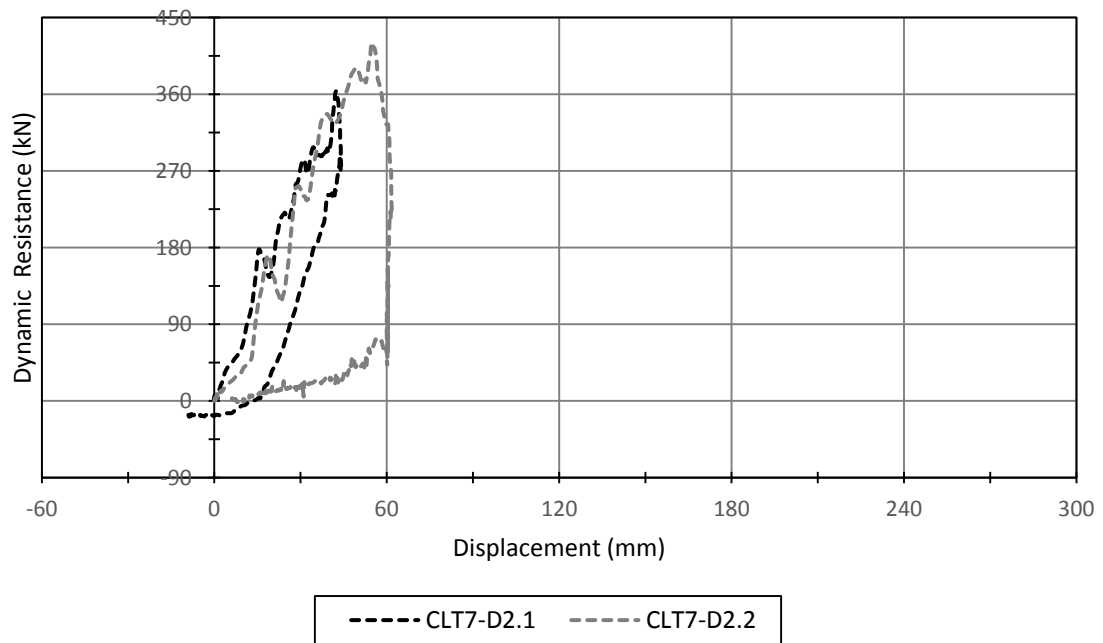


Figure 4.74: Dynamic-Resistance Curve – Damaged 7-Ply

Table 4.13: Destructive Dynamic Test Results for Phase 2 – Part 1

Test Name	Driver Pressure	P_R ¹	I_R ²	Δ_{max} ³	$t_{\Delta max}$ ⁴	R_{max} ⁵	Δ_{Rmax} ⁶	t_{Rmax} ⁷	R_f ⁹	Δ_{Rf} ¹⁰	μ ⁸	K ¹¹
	<i>kPa</i>	<i>kPa</i>	<i>kPa – ms</i>	<i>mm</i>	<i>ms</i>	<i>kN</i>	<i>mm</i>	<i>ms</i>	<i>kN</i>	<i>mm</i>	-	<i>kN/mm</i>
CLT3-3	424.0	65.8	747.2	217.1	46.6	132.2	45.9	20.6	74.2	114.6	3.5	3.1
CLT3-4	430.9	56.7	841.6	215.0	50.6	107.2	43.7	15.0	50.7	139.2	2.8	2.3
CLT3-5	481.9	60.3	805.7	180.6	59.4	142.0	61.5	18.2	66.6	148.6	2.4	2.7
CLT3-6	418.5	54.6	737.2	132.1	45.2	122.5	67.7	21.2	74.0	132.1	1.9	1.9
CLT5-6	650.9	92.9	1,258.4	163.7	61.2	264.5	31.6	13.0	204.8	80.0	3.1	8.4
CLT3-D1	321.3	38.8	544.3	122.5	54.2	109.3	74.0	25.0	28.4	122.4	1.3	1.5
CLT7-D2.1	483.3	59.2	950.7	44.1	26.0	364.0	42.5	23.0	364.0	42.5	0.8	8.2
CLT7-D2.2	550.2	55.9	1,030.5	61.7	26.2	420.5	55.1	21.6	420.5	55.1	1.1	7.7

¹Reflected Pressure²Reflected Impulse³Maximum Recorded Displacement⁴Time to Maximum Recorded Displacement⁵Maximum Dynamic Resistance⁶Deflection at Maximum Dynamic Resistance⁷Time to Maximum Dynamic Resistance⁸Ultimate Failure Resistance⁹Ultimate Failure Deflection¹⁰Ductility Ratio¹¹Stiffness

Table 4.14: Destructive Dynamic Test Results for Phase 2 – Part 2

Test Name	$\varepsilon_{ft}^1 \times 10^4$	t_{eft}^2	Δ_{eft}^3	$\varepsilon_{fc}^4 \times 10^4$	t_{efc}^5	$\varepsilon_{fh}^6 \times 10^4$	t_{efh}^7	$\varepsilon_{fv}^8 \times 10^4$	t_{efv}^9
	<i>mm/mm</i>	<i>ms</i>	<i>mm</i>	<i>mm/mm</i>	<i>ms</i>	<i>mm/mm</i>	<i>ms</i>	<i>mm/mm</i>	<i>ms</i>
CLT3-3	51.97	33.8	124.76	-31.72	16.6	101.12	50.6	-	-
CLT3-4	21.32	13.6	34.45	-23.20	13.6	21.55	13.4	-	-
CLT3-5	-	-	-	-	-	36.98	18.0	-29.71	17.2
CLT3-6	37.15	17.0	48.13	-	-	62.57	46.4	-42.38	27.8
CLT5-6	27.54	16.8	49.52	-30.57	15.8	33.52	17.8	-32.90	16.0
CLT3-D1	-	-	-	-39.66	28.6	55.13	44.0	-35.68	28.6
CLT7-D2.1	22.92	19.8	37.71	-21.83	22.6	22.78	20.0	-23.72	20.2
CLT7-D2.2	23.14	20.2	52.25	-23.99	18.8	24.09	21.0	-26.50	20.4

¹Strain at Wood Tensile Rupture²Time to Wood Tensile Rupture³Displacement at Wood Tensile Rupture⁴Maximum Wood Compressive Strain⁵Time to Maximum Wood Compressive Strain⁶GFRP Tensile Failure Strain⁷Time to GFRP Tensile Failure Strain⁸GFRP Compressive Failure Strain⁹Time to GFRP Compressive Failure Strain

CHAPTER 5 – Discussion

5.1 General

A total of seventeen retrofitted cross-laminated timber (CLT) panels were tested to failure under simulated blast loads. The experimental program was divided into two distinct phases: CLT panels reinforced with steel straps and GFRP. This chapter discusses the behaviour of the CLT specimens and presents key findings for different configurations and confinement details.

5.2 Effect of Retrofitting Techniques on Panel Stiffness

The current study utilizes published results obtained by Poulin et al. (2017) as a reference to investigate the behaviour of retrofitted CLT panels. Poulin et al. (2017) performed static and dynamic tests in order to investigate the failure modes of CLT panels and to establish an appropriate dynamic increase factor for the purpose of analysis and design. A summary of this research program was reported in Section 2.7.

In the current study, non-destructive static tests were conducted on unretrofitted CLT panels in order to evaluate their stiffness. The static and dynamic values of stiffness obtained in the current study are compared with those reported by Poulin et al. (2017) in Table 5.1. The results show that the stiffness of unretrofitted panels are consistent with those found by Poulin et al. (2017) (comparing column [1] to column [2]), emphasizing that the CLT panels, which originated from the same batch and same manufacturer are in fact comparable. The slight discrepancy found in the initial static stiffness can be attributed to variability in the material. Results from Poulin et al. (2017) seem to indicate that no increase in stiffness is found due to the dynamic effects (comparing column [3] to column [2]). When comparing the results from the dynamic retrofitted panels to the dynamic tests on unretrofitted panels conducted by Poulin et al. (2017) (column [4] to column [3]) and to static tests on unretrofitted panels (column [4] to column [1]) similar values are obtained. This finding further emphasizes that the increases in stiffness is only due to the retrofitting of the specimens and not to high strain rate effects. Based on the results it can be concluded that retrofitting CLT panels with the techniques and quantity investigated in this study provides an average increase in stiffness in the range of 10 % to

28 % compared to the unretrofitted panels. It can also be indirectly concluded that no increase in stiffness is observed due to high strain rate effects.

The CLT panels reinforced with GFRP resulted in an average increase in stiffness by a factor of 1.53 and 1.30, for a 3, and 5-ply, respectively, when compared with specimens retrofitted with steel straps.

Table 5.1: Comparison in Stiffness between Static and Dynamic Tests

Panel ID	<i>K</i>							
	<i>kN/mm</i>							
	[1] ¹	[2] ²	[3] ³	[4] ⁴	[2] / [1]	[3] / [2]	[4] / [1]	[4] / [3]
CLT3-1	2.0	2.1	1.9	2.0	1.05	0.90	1.00	1.05
CLT3-2				1.3			0.65	0.68
CLT3-3				3.1			1.55	1.63
CLT3-4				2.3			1.15	1.21
CLT3-5				2.7			1.35	1.42
CLT3-6				1.9			0.95	1.00
AVG	-	-	-	2.2	-	-	1.11	1.17
CLT5-1.1	5.4	5.9	5.3	5.5	1.09	0.90	1.02	1.04
CLT5-2.1				7.7			1.43	1.45
CLT5-3				5.9			1.09	1.11
CLT5-4				6.4			1.19	1.21
CLT5-5				6.9			1.28	1.30
CLT5-6				8.4			1.56	1.58
AVG	-	-	-	6.8	-	-	1.26	1.28

¹Unretrofitted Static Value Obtained in the Current Study

²Unretrofitted Static Value Obtained by Poulin et al. (2017)

³Unretrofitted Dynamic Value Obtained by Poulin et al. (2017)

⁴Retrofitted Dynamic Value Obtained in the Current Study

5.3 Behaviour of CLT Panels Reinforced with Steel Straps

A total of seven undamaged specimens were reinforced with steel straps and tested destructively under dynamic loading. The test matrix was designed to explore the following parameters: strap spacing (i.e. distance between transverse and longitudinal layers), location of straps (i.e. thirds, and mid-span), and order of strap installation. The aim of this retrofitting technique was to reduce or eliminate the effect of rolling shear, which had developed in unretrofitted CLT panels (Poulin et al., 2017). Particular

emphasis was also placed on the enhancement of material property, particularly the post peak strength and ductility.

5.3.1 Effect of Horizontal Strap Reinforcement Configurations

Table 5.2 summarizes the peak resistance, post peak resistance, and ductility ratio and compares the results with the corresponding average values reported by Poulin et al. (2017). The ductility ratios presented in the subsequent sections of this chapter, are based on a post peak resistance equal to 50 % of the maximum dynamic resistance of the CLT member as anything below this threshold was deemed “insignificant”.

Table 5.2: Comparison of Material Properties for Panels Reinforced with Horizontal Steel Straps

Panel ID	R_{max}			R_f			μ		
	kN			kN			-		
	$[1]^1$	$[2]^2$	$[2] / [1]$	$[1]^1$	$[2]^2$	$[2] / [1]$	$[1]^1$	$[2]^2$	$[2] / [1]$
CLT3-1	83.7	86.4	1.03	14.9	44.5	2.99	1.0	2.0	2.00
CLT3-2		87.3	1.04		87.3	5.86		1.3	1.30
AVG	-	86.9	1.04	-	65.9	4.43	-	1.7	1.65

¹Unretrofitted Dynamic Value Obtained by Poulin et al. (2017)

²Retrofitted Dynamic Value Obtained in the Current Study

The results in Table 5.2 show that the addition of horizontal straps on every transverse layer of the CLT panel (CLT3-1) slightly increased the ultimate strength when compared to the average dynamic values obtained by Poulin et al. (2017). An increase of approximately 67 % in maximum displacement was also obtained. As presented in Table 5.2 and illustrated in Figure 5.1 (a), significant levels of post peak resistance was achieved in the retrofitted specimens, where increases by factors ranging from 2.99 to 5.86 were obtained. The ductility ratio also increased by a factor of 2.0.

Installing the straps at only the top and bottom thirds of the panel, resulted in almost no increases in ultimate strength and deflection, as expected. Most of the unretrofitted 3-ply panels failed in flexure at the mid-span region (Poulin 2017) and as such confining the ends of the panel against rolling shear failure was not expected to be very effective. There was also no improvement in the post peak strength, as shown in Figure 5.1 (b). In

general, it can be concluded that the horizontal straps were able to restrict the failure to small regions and to promote flexural failure.

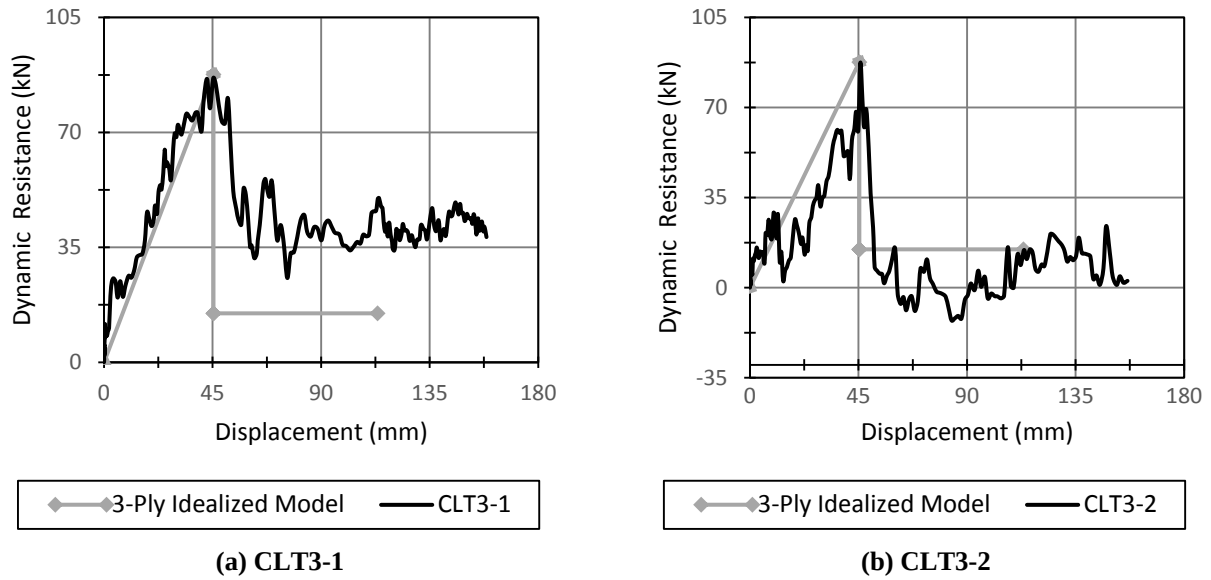


Figure 5.1: Horizontal Strap Location Variation Resistance Curves

5.3.2 Effect of Vertical Strap Reinforcement Configurations

Table 5.3 presents the behaviour of the panels through the addition of vertical straps and compares it to the unretrofitted panels. It can be seen that the addition of vertical straps on top of the horizontal straps improved the post peak resistance by a factor of 1.77. CLT5-1.1 did not fail and almost completely returned to its original position. When re-tested, specimen CLT5-1.2 experienced a decrease in strength and stiffness by a factor of 0.50, and 0.20, respectively, relative to the first shot (Figure. 5.2 (a)). The results indicate that although no apparent failure was observed, the specimen had suffered significant damage after the first shot.

Table 5.3: Comparison of Material Properties for Panels Reinforced with Vertical Steel Straps

Panel ID	R_{max}			R_f			μ		
	kN			kN			-		
	[1] ¹	[2] ²	[2] / [1]	[1] ¹	[2] ²	[2] / [1]	[1] ¹	[2] ²	[2] / [1]
CLT5-1.1	174.2	191.3	1.10	87.6	155.4	1.77	1.5	2.3	1.53
CLT5-1.2		96.3	0.55		57.8	0.66		1.8	1.20
CLT5-2.1		206.0	1.18		168.6	1.92		2.9	1.93
CLT5-2.2		204.5	1.17		163.0	1.86		3.6	2.40
CLT5-3		182.9	1.05		104.2	1.19		2.7	1.80
CLT5-4		188.4	1.08		73.6	0.84		1.7	1.13
CLT5-5		211.3	1.21		54.7	0.62		1.6	1.07
AVG	-	183.0	1.05	-	111.0	1.27	-	2.4	1.58

¹Unretrofitted Dynamic Value Obtained by Poulin et al. (2017)

²Retrofitted Dynamic Value Obtained in the Current Study

Changing the order of strap installation (CLT5-2) did not affect the failure mode of the slab; however having the horizontal straps on top of the vertical ones further reduced the rolling shear effect and completely eliminated flying debris. This installation technique not only confined the wood specimen, but also the vertical straps which reduced the sagging on the compression face. The ultimate strength, stiffness as well as ductility were improved by installing the vertical straps underneath horizontal ones, as illustrated in Figure 5.2 (b).

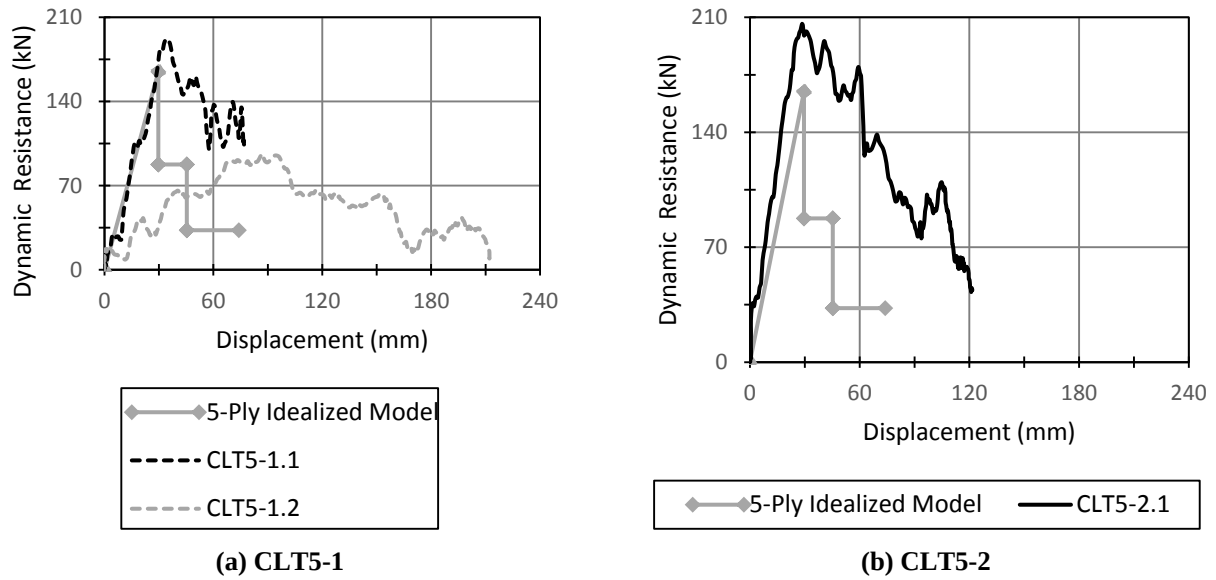


Figure 5.2: Order of Strap Installation Resistance Curves

Specimen CLT5-3 also consisted of vertical straps installed underneath horizontal straps, which were spaced at every transverse layer at the top and bottom third of the panel. This configuration resulted in an increase in ductility by a factor of 1.80, as shown in Figure 5.3. An increase in post peak strength was attained but found to be 36 % less than that reported for CLT5-2. This could possibly be attributed to the confinement of the horizontal straps not being continuous across the length of the panel but rather concentrated at the outer one thirds of the specimen.

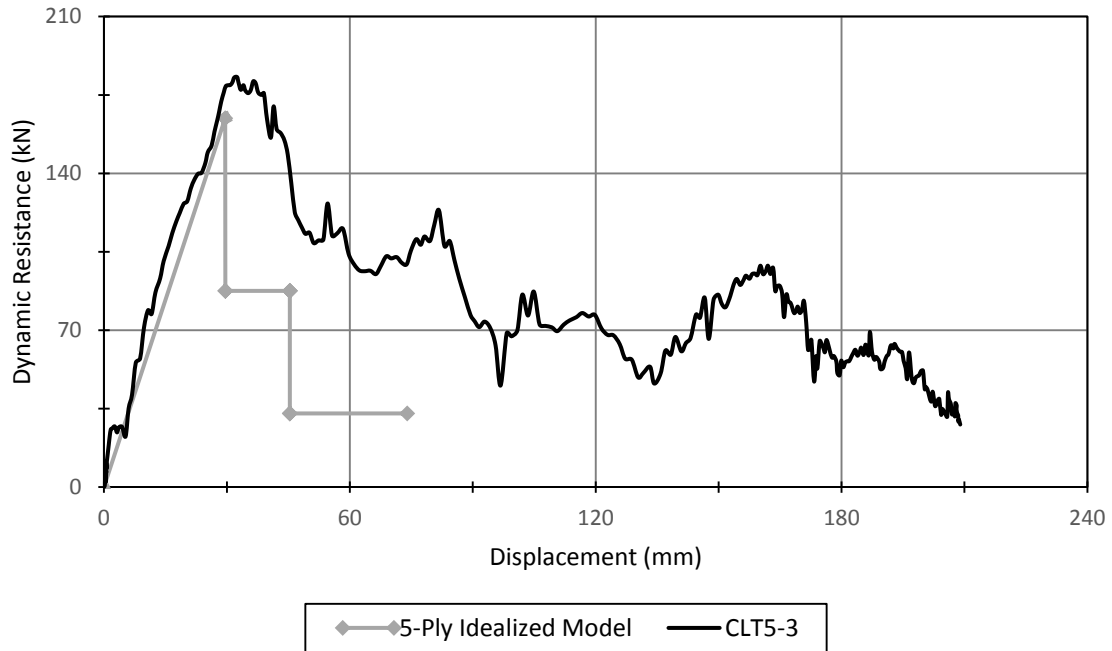


Figure 5.3: Addition of Vertical Straps Resistance Curve

Strap spacing was also explored and the results showed that a slight decrease in the post peak resistance can be observed when the straps were placed at every second transverse wood ply rather than at every ply, as shown in Figure 5.4. Vertical strap failure was observed while the horizontal ones experienced shifting rather than rupture. Typically the vertical straps failed at the seal, which was consistent with the coupon testing. Similar to the preceding configurations, despite doubling the horizontal strap spacing, the amount of rolling shear cracks was limited.

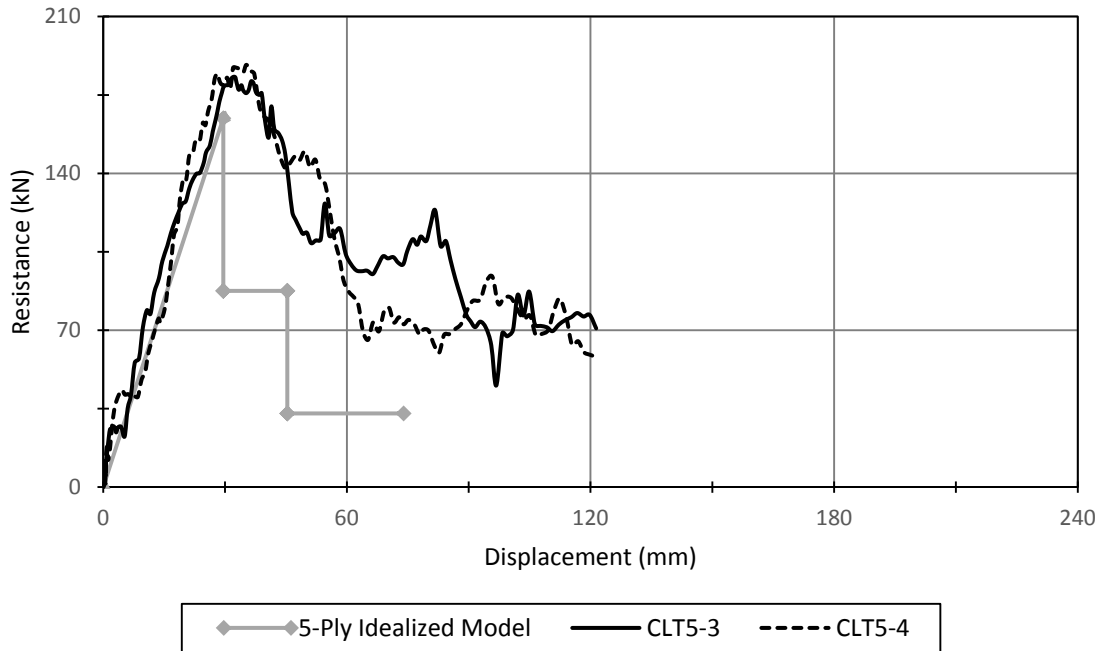


Figure 5.4: Strap Spacing Variation Resistance Curve

Fastening the vertical straps on the tension face, rather than wrapped around the height of the member (CLT5-5), resulted in significant increases in resistance, and stiffness by a factor of 1.21, and 1.30, respectively, when compared to the dynamic stiffness results obtained by Poulin et al. (2017). However, unlike any of the aforementioned panels, CLT5-5 experienced a significant amount of delamination between the longitudinal and transverse wood layers. While flexural failure was evident, the development of longitudinal shear had commenced soon after. Post peak resistance was significantly reduced, well below 50 % of the peak capacity.

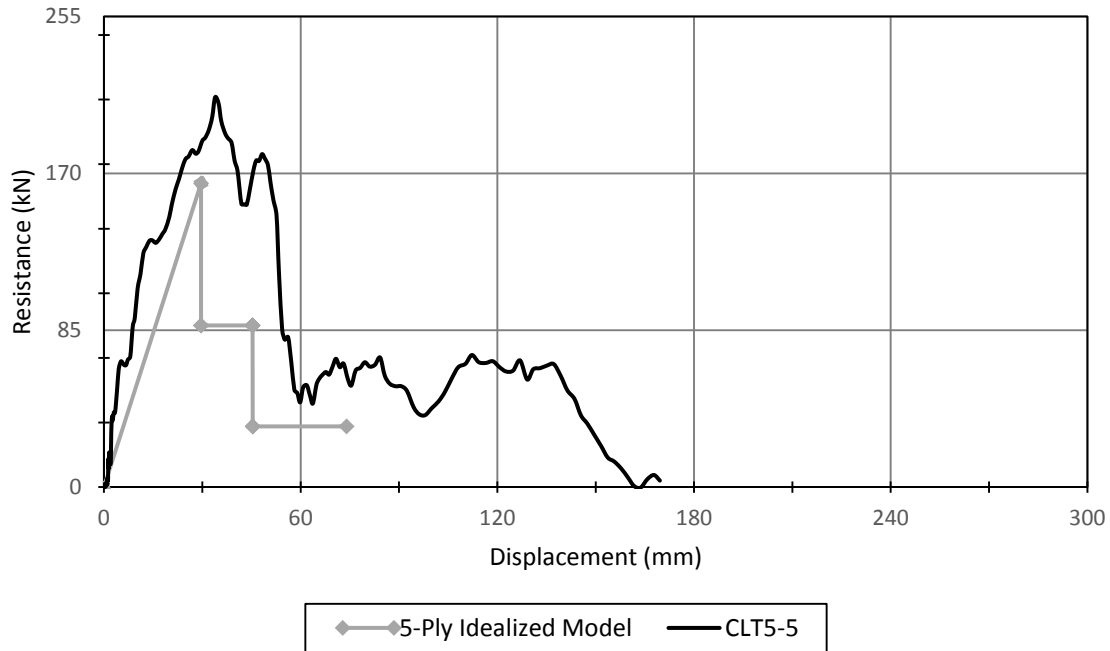


Figure 5.5: Vertical Straps Fastened to Tension Face Resistance Curve

5.4 Behaviour of GFRP Reinforced CLT Panels

A total of four undamaged specimens were reinforced with GFRP and tested destructively under dynamic loading. The main objective of this retrofitting technique was to investigate the effect of GFRP on the overall behaviour of a typical CLT panel. Table 5.4 summarizes the behaviour and compares them with the corresponding averages reported by Poulin et al. (2017).

Table 5.4: Comparison of Material Properties for Panels Reinforced with GFRP

Panel ID	R_{max}			R_f			μ		
	kN			kN			-		
	[1] ¹	[2] ²	[2] / [1]	[1] ¹	[2] ²	[2] / [1]	[1] ¹	[2] ²	[2] / [1]
CLT3-3	83.7	132.2	1.58	14.9	74.2	4.98	1.0	3.5	3.50
CLT3-4		107.2	1.28		50.7	3.40		2.8	2.80
CLT3-5		142.0	1.70		66.6	4.47		2.4	2.40
CLT3-6		122.5	1.46		74.0	4.97		1.9	1.90
AVG	-	126.0	1.51	-	66.4	4.45	-	2.7	2.65
CLT5-6	174.2	264.5	1.52	87.6	204.8	2.34	1.5	3.1	2.07

¹Unretrofitted Dynamic Value Obtained by Poulin et al. (2017)

²Retrofitted Dynamic Value Obtained in the Current Study

5.4.2 Effect of Uniaxial GFRP Reinforcement Configurations

Two U-shaped uniaxial GFRP layers, installed on the tension face of the specimen (CLT3-3), enhanced the overall behaviour of the slab, as illustrated in Figure 5.6. This configuration resulted in an increase in peak resistance, ductility, and stiffness by a factor of 1.58, 3.50, and 1.63, respectively, when compared to the dynamic results of an unretrofitted panel (Poulin et al., 2017). The post peak resistance was also greatly improved. A ductility ratio of 4.7 can be attributed to the minimal amount of de-bonding between the GFRP laminas and the panel. Despite a significant amount of damage to the GFRP, there was no evident flexural failure. CLT3-3 was the only specimen that was not fully confined therefore it cannot be used as a baseline for panels with full confinement.

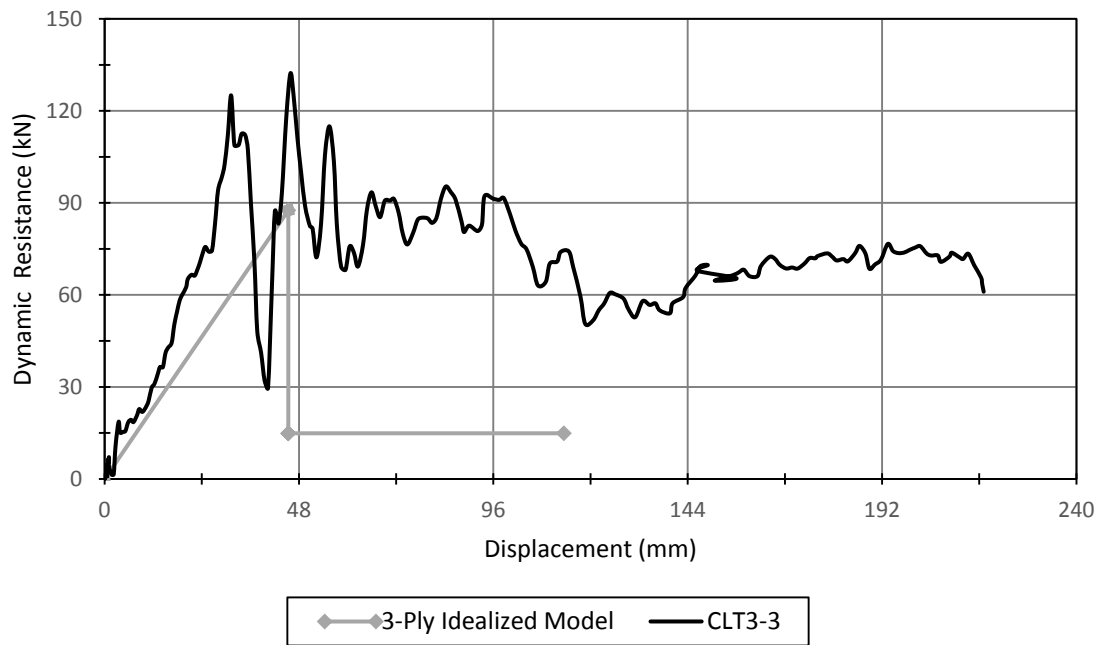


Figure 5.6: U-Shape Configuration Resistance Curve

5.4.3 Effect of Multiaxial GFRP Reinforcement Configurations

Providing two biaxial GFRP confinement layers increased the peak resistance and stiffness by a factor, on average, of 1.49, and 1.32, respectively, when compared to the average dynamic results obtained by Poulin et al. (2017). The contribution of longitudinal and transverse fibres, as shown in Figure 5.7(a), greatly improved the ductility of the system by a factor of 1.81 and 1.67 when compared to the average values reported by

Poulin et al. (2017), and CLT3-4, respectively. GFRP with fibres in the $\pm 45^\circ$ direction, as illustrated in Figure 5.7 (b), resulted in an increase in peak capacity and stiffness of 33%, and 17 %, respectively, relative to GFRP with fibres in the 0/90°. The post peak capacity was found to be less than that obtained for a U-shaped configuration; however the biaxial confinement layers resulted in greater post peak displacements.

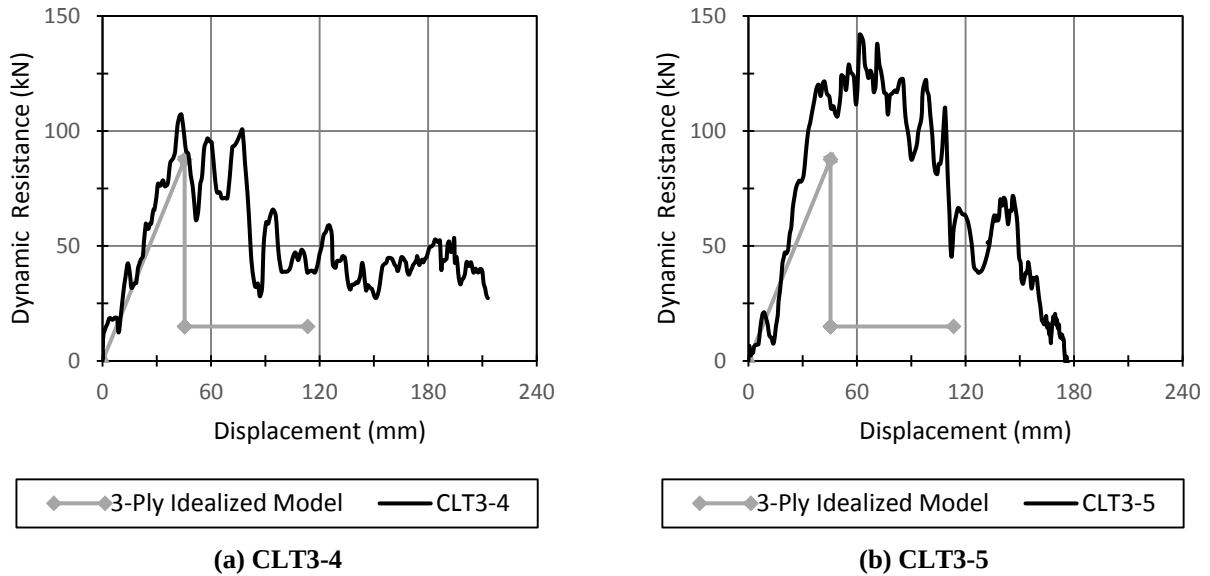


Figure 5.7: Biaxial Fabric Confinement Resistance Curves

Two triaxial GFRP confinement layers also enhanced the performance of the 5-ply CLT panel shown in Figure 5.8. Increases in capacity, ductility, and stiffness by factors of 1.52, 2.07, and 1.58, respectively, were observed relative to an unreinforced panel. A very large increase in post peak capacity was attained and in turn promoted ductile behaviour. CLT5-6 experienced a similar failure mode seen in panels reinforced with biaxial fabric. Flexural failure was the predominant failure mode, with slight delamination between the longitudinal and transverse wood laminations. The confinement layers reduced the tension perpendicular to grain failure.

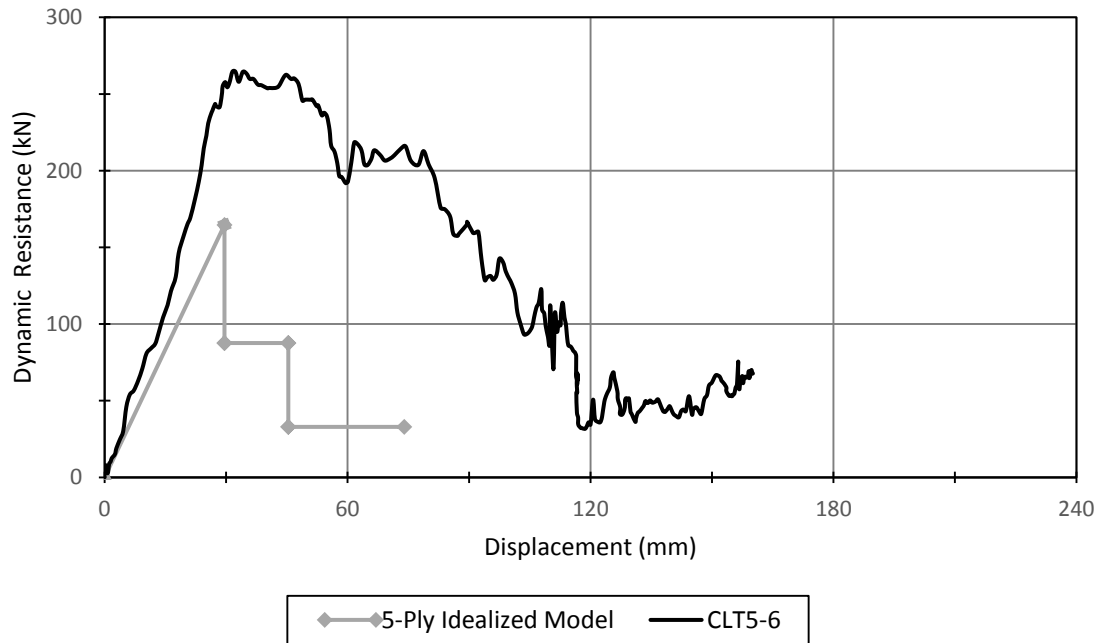


Figure 5.8: Triaxial Fabric Confinement Configuration Resistance Curve

The previously mentioned biaxial fabrics were utilized to create a stacked quadraxial lay-up configuration. The peak resistance of specimen CLT3-6 increased by a factor of 1.46 relative to the results obtained by Poulin et al. (2017). There was significant improvement on the post peak capacity; and hence ductility, as shown in Figure 5.9. Compared to the aforementioned panels this particular configuration resulted in the greatest post peak resistance. The observed behaviour was anticipated as the quadraxial layers consisted of fibres in the 0° , $\pm 45^\circ$, and 90° direction. Unlike the panels reinforced with biaxial fabric, CLT3-6 did not exhibit any flexural failure despite the distressed GFRP and rolling shear failure was limited.

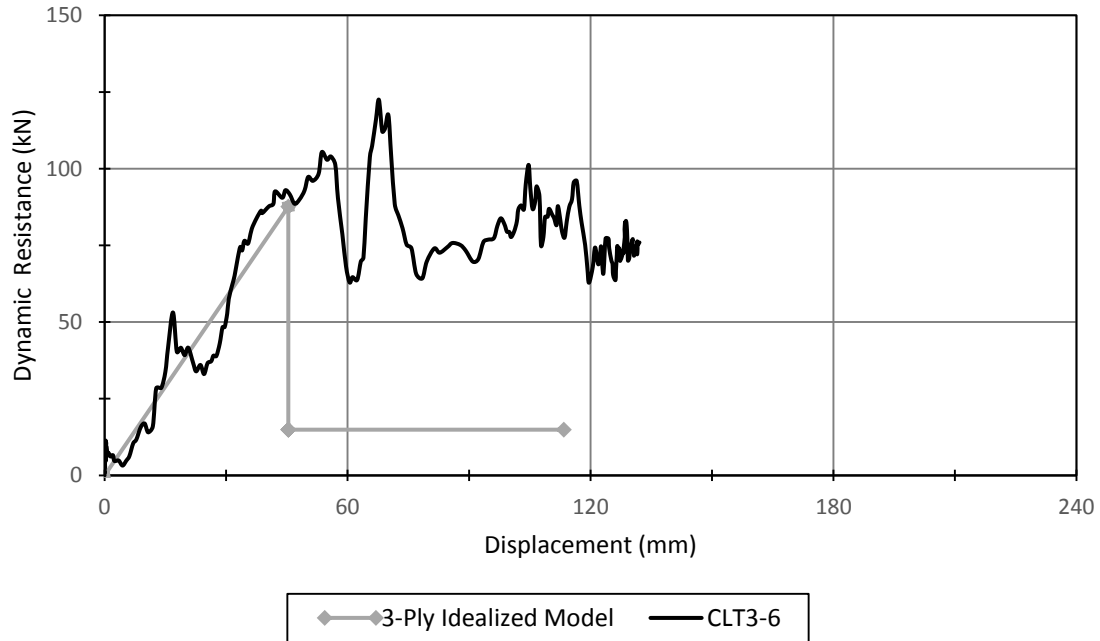


Figure 5.9: Quadraxial Fabric Confinement Configuration Resistance Curve

5.5 Effect of Reinforcement on Damaged Panels

In addition to the twelve aforementioned undamaged panels, previously tested specimens were restored and reinforced with similar retrofitting techniques. The primary objective for conducting these tests, as part of the experimental program, was to investigate the potential in temporary support and to ultimately determine if a confinement alternative would restore the initial capacity of CLT panels. A total of five damaged panels were selected and included in the current study; of which three were reinforced with steel straps while the remaining two were retrofitted with GFRP. Table 5.5 summarizes the stiffness values obtained from the current study and compares them to the average values reported by Poulin et al. (2017). Table 5.6 compares the peak resistance, post peak resistance, and ductility ratio between the damaged retrofitted and undamaged unreinforced panels (Poulin et al., 2017).

Table 5.5: Comparison in Stiffness of Damaged Panels

Panel ID	<i>K</i>							
	<i>kN/mm</i>							
	[1] ¹	[2] ²	[3] ³	[4] ⁴	[2] / [1]	[3] / [2]	[4] / [1]	[4] / [3]
CLT3-D1	2.0	2.1	1.9	1.5	1.05	0.90	0.75	0.79
CLT5-D1	5.4	5.9	5.3	1.1	1.09	0.90	0.20	0.21
CLT5-D2.1				1.0			0.19	0.19
CLT5-D2.2				0.8			0.15	0.15
AVG	-	-	-	1.0	-	-	0.18	0.18
CLT7-D1	7.1	-	-	1.2	-	-	0.17	-
CLT7-D2.1				8.2			1.15	-
CLT7-D2.2				7.7			1.08	-
AVG	-	-	-	5.7	-	-	0.80	-

¹Unretrofitted Static Value Obtained in the Current Study²Unretrofitted Static Value Obtained by Poulin et al. (2017)³Unretrofitted Dynamic Value Obtained by Poulin et al. (2017)⁴Retrofitted Dynamic Value Obtained in the Current Study**Table 5.6: Comparison of Material Properties for Reinforced Damaged Panels**

Panel ID	<i>R_{max}</i>			<i>R_f</i>			<i>μ</i>		
	<i>kN</i>			<i>kN</i>			-		
	[1] ¹	[2] ²	[2] / [1]	[1] ¹	[2] ²	[2] / [1]	[1] ¹	[2] ²	[2] / [1]
CLT3-D1	83.7	109.3	1.31	14.9	28.4	1.91	1.0	1.3	1.30
CLT5-D1	174.2	89.6	0.51	87.6	89.6	1.02	1.5	1.4	0.93
CLT5-D2.1		53.8	0.31		53.8	0.61		1.0	0.67
CLT5-D2.2		62.0	0.36		62.0	0.71		2.2	1.47
AVG	-	68.5	0.39	-	68.8	0.78	-	1.5	1.02
CLT7-D1	-	96.5	-	-	96.5	-	-	1.3	-
CLT7-D2.1		364.0	-		364.0	-		0.8	-
CLT7-D2.2		420.5	-		420.5	-		1.1	-
AVG	-	293.7	-	-	293.7	-	-	1.1	-

¹Unretrofitted Dynamic Value Obtained by Poulin et al. (2017)²Retrofitted Dynamic Value Obtained in the Current Study

CLT5-D1 was retrofitted identically to CLT3-1, with horizontal straps installed at every transverse layer. The results from the test on this specimen showed a decrease in maximum resistance by 49 %, and stiffness by 79 % when compared to the dynamic average values observed by Poulin et al. (2017). There was also a significant increase in maximum displacement of 240 % but a decrease in ductility ratio by a factor of 0.93.

CLT5-D2 was reinforced similarly to CLT5-2 with horizontal straps installed on top of the vertical straps. The horizontal and vertical straps provided confinement to the wood specimen, Figure 5.10 compares the dynamic resistance-displacement curve of an unretrofitted undamaged panel (Poulin et al., 2017) to the curves generated by the retrofitted panels.

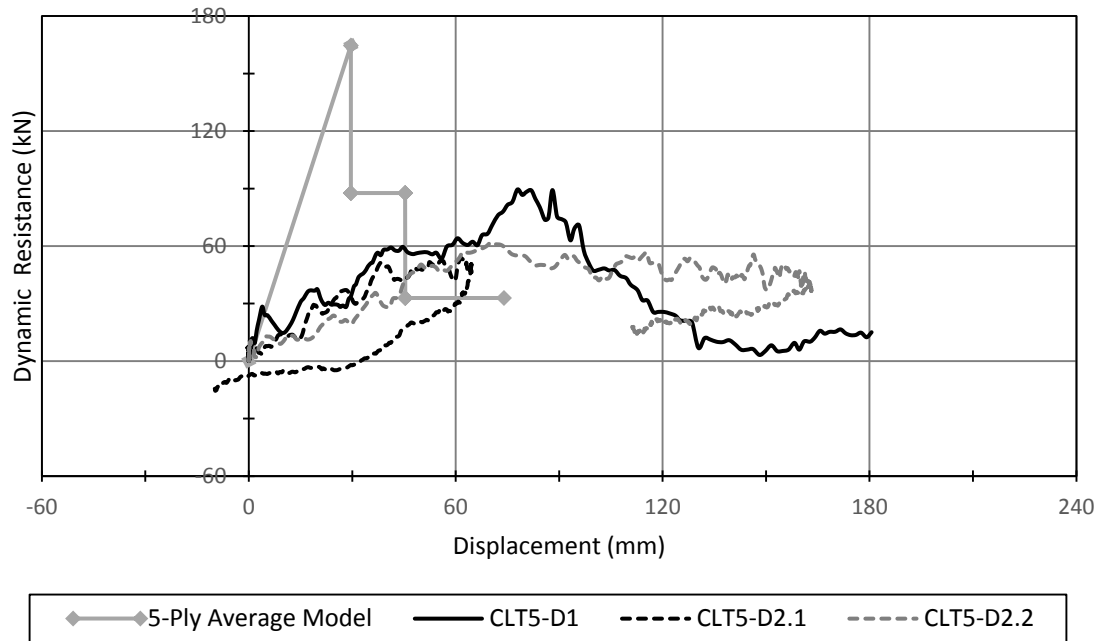


Figure 5.10: Resistance Curve Comparison between Unreinforced and Reinforced 5-Ply Panels

When compared to the dynamic averages of unretrofitted panels, the panel reinforced with two triaxial full confinement GFRP layers, CLT3-D.1, experienced an increase in peak resistance, by a factor of 1.31, and exhibited better post peak performance compared to the unretrofitted and undamaged counterpart, as can be seen in Figure 5.11. Although the specimen had been previously tested, the failure mode resembled that of CLT3-6. No new damage was observed (i.e. rolling shear) and all laminations remained intact. Clearly the specimen was not too damaged in the initial shot, however it is still remarkable that initial conditions not only could be restored but also enhanced using the reinforcement techniques presented in this study.

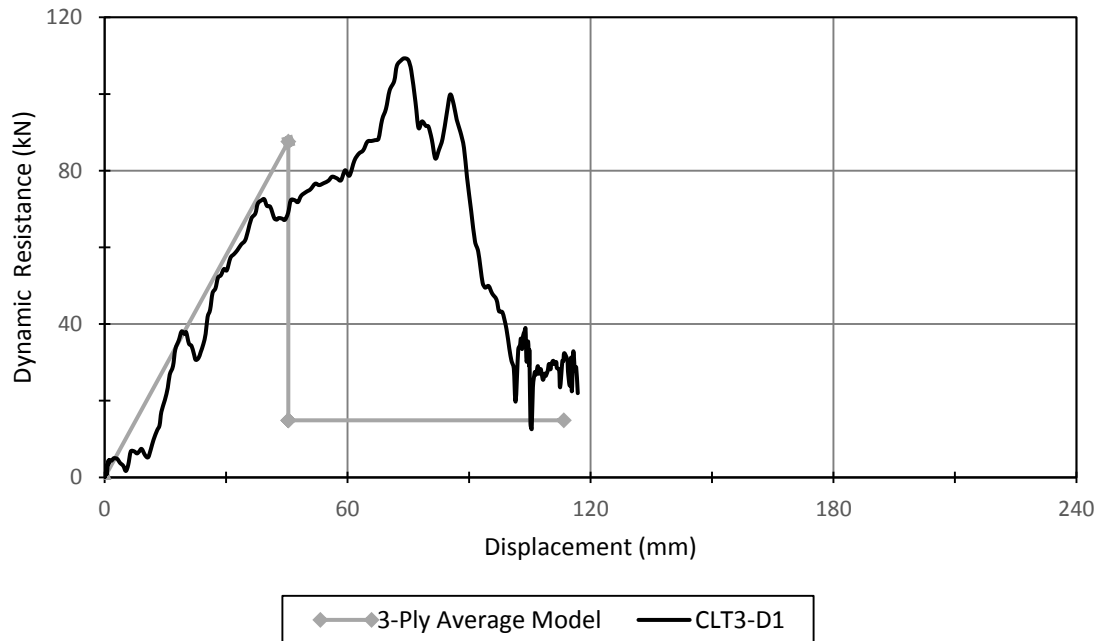


Figure 5.11: Resistance Curve Comparison between Unreinforced and Reinforced 3-Ply Panels

CLT7-D1 and CLT7-D2 were reinforced with horizontal steel straps, and two layers of stacked quadraxial GFRP, respectively. The horizontal straps were able to constrain the wood laminations but delamination between the transverse and longitudinal layers was observed, leading to a significant amount of displacement and a reduction in stiffness, as illustrated in Figure 5.12 (a). Due to the lack of 7-ply unretrofitted specimens, the tested configurations could not be compared with a baseline measure of performance. The ductility was lacking but the confinement returned the specimen to its original shape, as shown in Figure 5.12 (b).

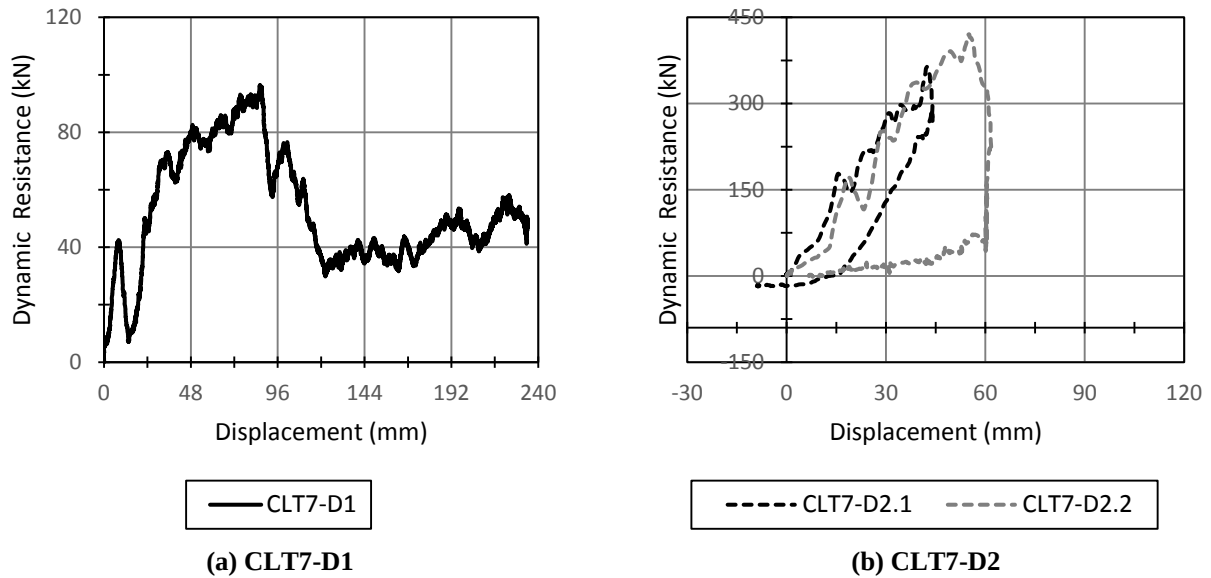


Figure 5.12: Resistance Curve Comparison between Unreinforced and Reinforced 7-Ply Panels

It should be noted that it is very difficult to quantify the initial condition of each damaged slab which makes it difficult to correlate the data to an undamaged panel. In terms of failure modes, both retrofitting alternatives proved to promote flexural failure and confined the failure in a small region. While shear failure was observed, it was more evident in panels reinforced with steel straps. The addition of straps did not improve the peak resistance or the stiffness. GFRP was better capable of partially restoring the panels' initial stiffness and peak resistance.

CHAPTER 6 – Conclusions and Future Recommendations

6.1 General

In the current study, a total of seventeen retrofitted cross-laminated timber (CLT) panels were investigated and the results were discussed and compared to published data from similar but unretrofitted specimens.

6.2 Conclusions

The following conclusions can be drawn from the current study:

- Retrofitting CLT panels with steel straps or GFRP provided some increase in the panels' initial stiffness in the range of 10 % to 28 % compared to the unretrofitted panels. It was also found that no increase in stiffness is observed due to high strain rate effects.
- Reinforcing the panels with only horizontal steel straps at every wood ply had minimal effect on the ultimate strength, but significant levels of post peak resistance was achieved. The post peak resistance increased by factors ranging from 2.99 to 5.86. The ductility ratio also increased by a factor of 2.0. Installing the straps at only the top and bottom thirds of the panel, resulted in almost no enhancement in the behaviour of the CLT panels tested. Regardless of their location, it can be concluded that horizontal straps were able to restrict the failure to small regions and to promote flexural failure by preventing rolling shear failure.
- The addition of vertical straps on top of the horizontal straps improved the post peak resistance significantly by a factor of 1.77. Installing the horizontal straps on top of the vertical ones reduced the rolling shear failure and completely eliminated flying debris. It also further enhanced the ultimate strength, stiffness as well as ductility. Reinforcing every second ply rather than every wood ply had almost no negative effect on the behaviour of the CLT panels.
- Applying two U-shaped uniaxial GFRP layers enhanced the overall behaviour of the slab resulting in a significant increase in peak resistance, ductility, and stiffness when compared to the dynamic results of an unretrofitted panel. The post peak resistance was also greatly improved. Providing two biaxial GFRP

confinement layers increased the peak resistance and stiffness and greatly improved the ductility of the system even when compared to the panels retrofitted with steel straps. Two triaxial GFRP confinement layers also enhanced the performance of the CLT panel. Particularly, the post peak capacity was enhanced and ductile behaviour was evident. Finally a stacked quadraxial lay-up configuration also significantly improved the ductility, and resulted in the greatest post peak resistance.

- Previously tested specimens were restored and successfully retrofitted with both steel straps and GFRP. Only partial recovery of the resistance and the stiffness was achieved. When horizontal straps were installed on top of the vertical straps, the retrofit provided more confinement to the wood specimen. The horizontal straps were able to constrain the wood laminations but delamination between the transverse and longitudinal layers was observed, leading to a significant amount of displacement and a reduction in stiffness. Two triaxial full confinement GFRP layers yielded better performance compared to the unretrofitted and undamaged counterpart. This finding highlights the potential for GFRP to be used in retrofitting damaged CLT elements. The data presented in the current study is however limited and more work is needed to quantify the benefits of using GFRP in these applications.

6.3 Recommendations for Future Work

Based on the observations and results described in the preceding chapters, several areas have been identified as recommended future work. These include the following:

- The complexity of the CLT material necessitate more advanced modeling techniques to be developed in order to capture the interaction between the CLT panel and the retrofit mechanism.
- The CLT slabs that are likely to be subjected to blast loading are wall panels which likely support gravity loads. As such, the effects of axial loads and realistic boundary conditions on the performance of reinforced CLT panels need to be investigated and incorporated into a predictive model.

- More work is required in order to identify damage levels and suitable retrofit options for damaged CLT panels.

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APPENDIX A – CLT Coupons Static Test Results

Compression Coupon Test Results

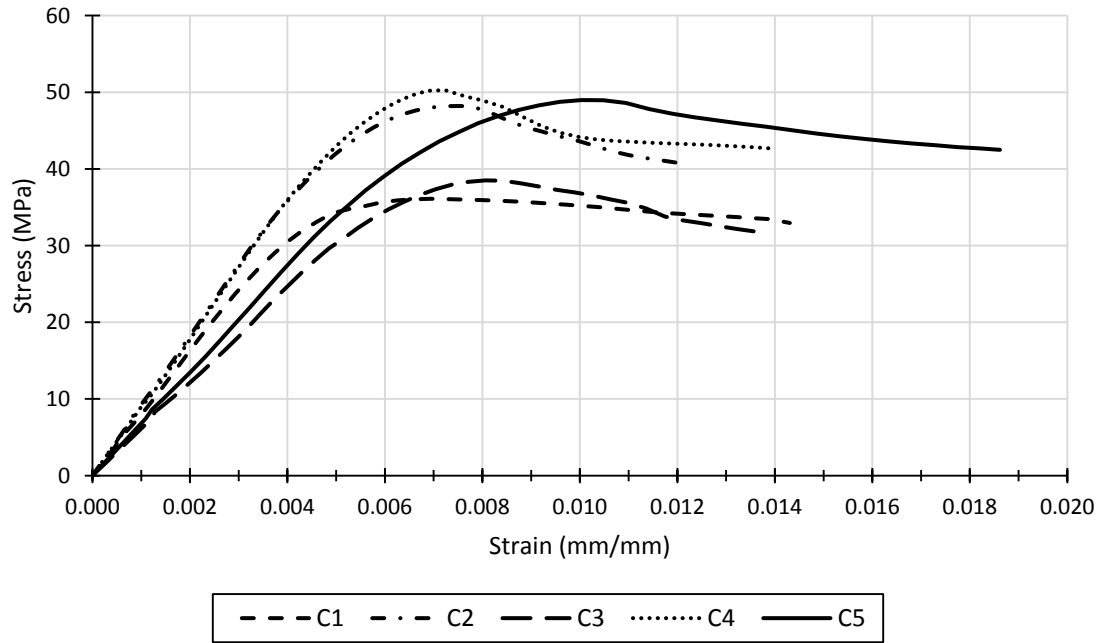


Figure A.1: Experimental Stress-Strain Curve for Compression Coupons

Table A.1: Individual Compression Coupon Test Results

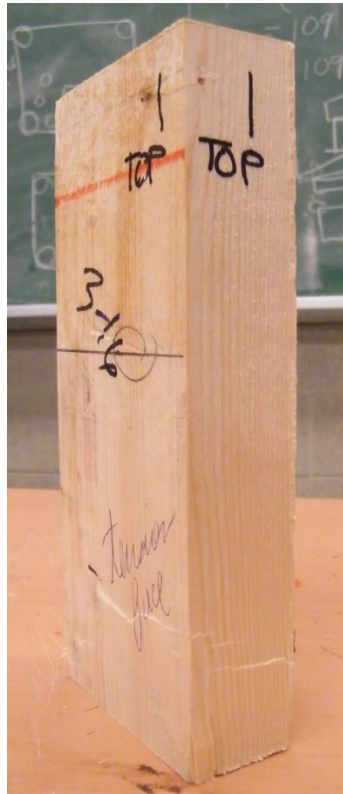
Coupon	E_{wc}^1	f_c^2	f_{cu}^3	$\epsilon_{cu}^4 \times 10^{-4}$
	MPa	MPa	MPa	mm/mm
C1	7,936	34.7	33.0	143.1
C2	9,077	47.6	40.7	120.6
C3	6,126	35.7	31.8	136.3
C4	8,882	49.6	42.6	139.5
C5	6,774	44.8	42.5	186.2
Average	7,759	42.5	38.1	145.1
St. Dev.	1,291	6.9	5.3	24.5
COV	0.17	0.16	0.14	0.17

¹Compression Modulus of Elasticity

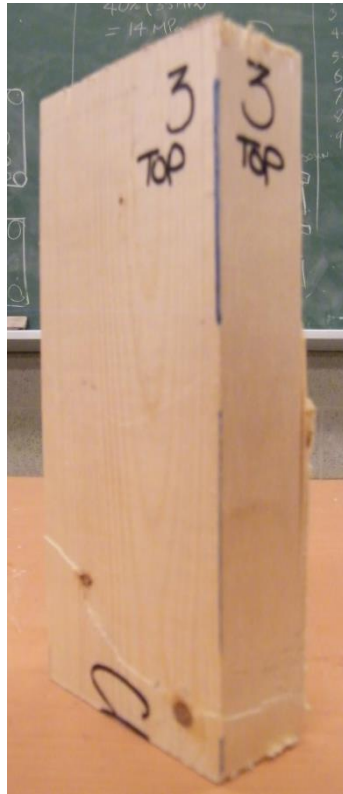
²Compressive Yield Strength

³Ultimate Compressive Strength

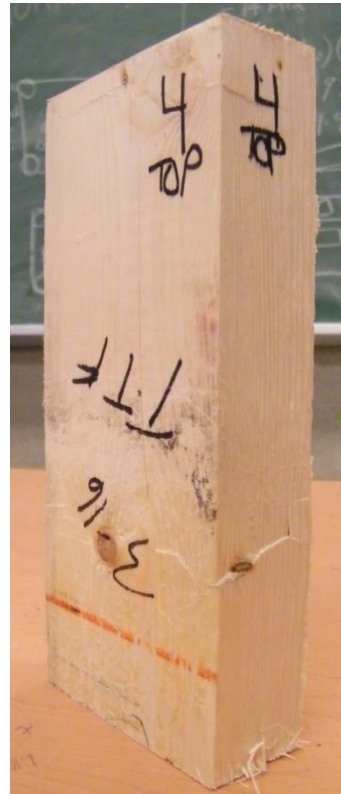
⁴Ultimate Compressive Strain



(a) C1



(b) C2



(c) C3



(d) C4



(e) C5

Figure A.2: Typical Individual Failure Modes for Compression Coupons

Tension Coupon Test Results

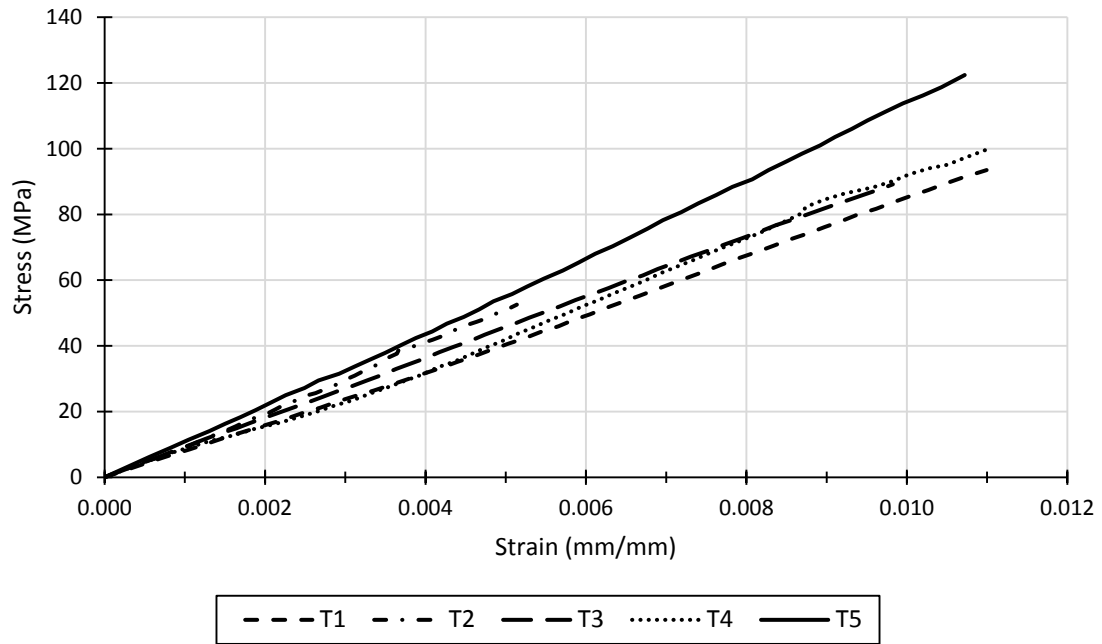


Figure A.3: Experimental Stress-Strain Curve for Tension Coupons

Table A.2: Individual Tension Coupon Test Results

Coupon	E_{wt}^1	f_t^2	$\epsilon_t^3 \times 10^{-4}$
	MPa	MPa	mm/mm
T1	8,387	93.5	110.0
T2	10,111	52.5	51.4
T3	9,133	89.1	98.3
T4	8,981	99.7	110.0
T5	11,261	122.4	107.2
Average	9,574	91.0	95.4
St. Dev.	1,128	25.3	25.0
COV	0.12	0.28	0.26

¹Tensile Modulus of Elasticity

²Ultimate Tensile Strength

³Tensile Failure Strain



(a) T1



(b) T2



(c) T3



(d) T4



(e) T5

Figure A.4: Typical Individual Failure Modes for Tension Coupons

APPENDIX B – Steel Strap Coupons Static Test Results

Simpson Strong-Tie – Magnus Steel Strap – No Seal

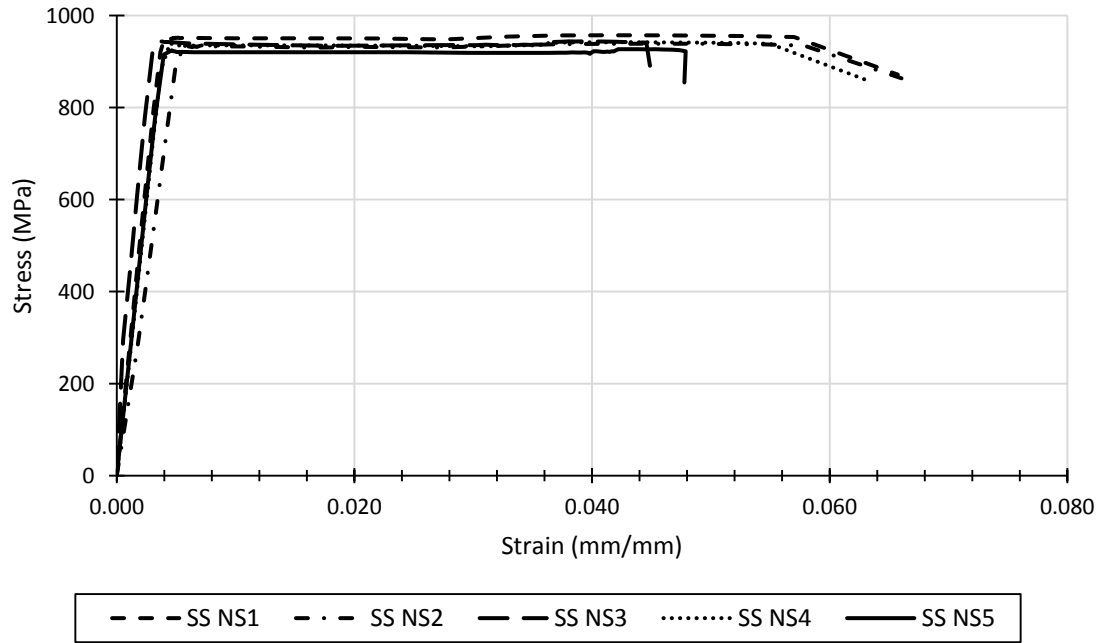


Figure B.1: Experimental Stress-Strain Curve for Magnus Steel Strap without Seal Coupons

Table B.1: Individual Magnus Steel Strap without Seal Coupon Test Results

Coupon	E_{ss}^1	f_{ss}^2	f_{ssu}^3	$\epsilon_{ss}^4 \times 10^{-4}$
	MPa	MPa	MPa	mm/mm
SS _{NS1}	249,181	952.3	953.2	570.1
SS _{NS2}	172,233	938.6	934.5	578.7
SS _{NS3}	301,464	944.4	941.2	445.7
SS _{NS4}	226,728	940.3	938.6	550.0
SS _{NS5}	223,586	924.6	922.5	478.8
Average	234,638	940.1	938.0	524.6
St. Dev.	46,788	10.1	11.1	59.1
COV	0.20	0.01	0.01	0.11

¹Magnus Steel Strap Modulus of Elasticity

²Yield Strength

³Ultimate Tensile Strength

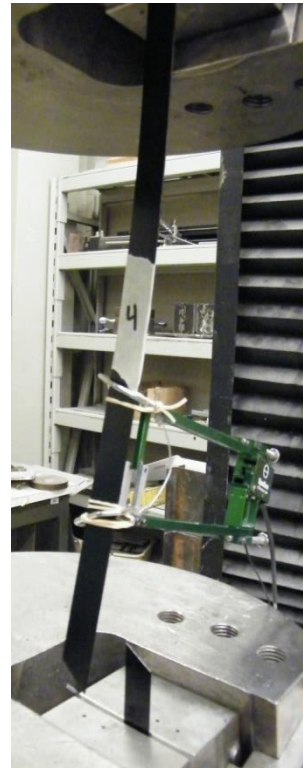
⁴Tensile Failure Strain



(a) SS_{NS1}



(b) SS_{NS2}



(c) SS_{NS3}



(d) SS_{NS4}



(e) SS_{NS5}

Figure B.2: Typical Individual Failure Modes for the Magnus Steel Strap without Seal Coupons

Simpson Strong-Tie – Magnus Steel Strap – Seal

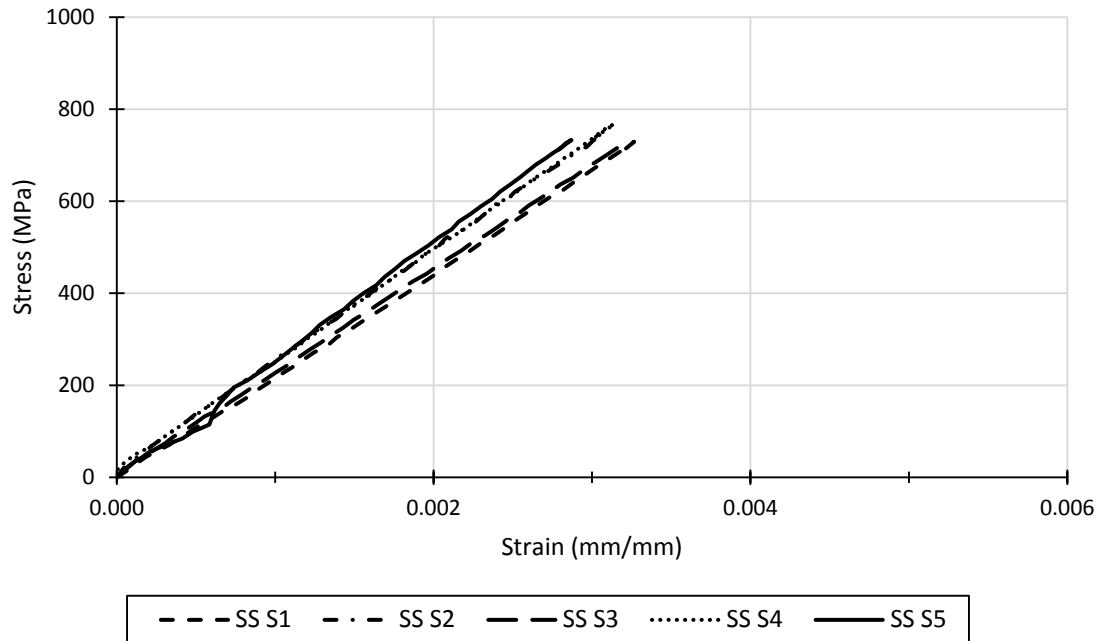


Figure B.3: Experimental Stress-Strain Curve for Magnus Steel Strap with Seal Coupons

Table B.2: Individual Magnus Steel Strap with Seal Coupon Test Results

Coupon	E_{ss}^1	f_{ss}^2	$\epsilon_{ss}^3 \times 10^{-4}$
	MPa	MPa	mm/mm
SS _s 1	221,366	729.2	32.7
SS _s 2	246,187	736.5	30.2
SS _s 3	226,920	723.4	31.9
SS _s 4	246,775	767.5	31.4
SS _s 5	255,692	733.3	28.7
Average	239,388	738.0	31.0
St. Dev.	14,550	17.2	1.6
COV	0.06	0.02	0.05

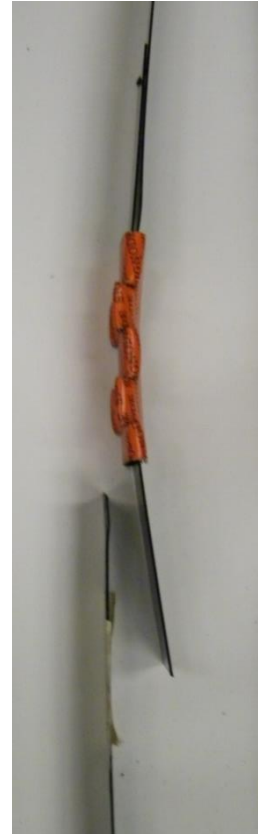
¹Magnus Steel Strap Modulus of Elasticity

²Ultimate Tensile Strength

³Tensile Failure Strain



(a) Top View



(b) Side View

Figure B.4: Typical Failure Mode for the Magnus Steel Strap with Seal Coupons

APPENDIX C – GFRP Coupons Static Test Results

GFRP – [0°]

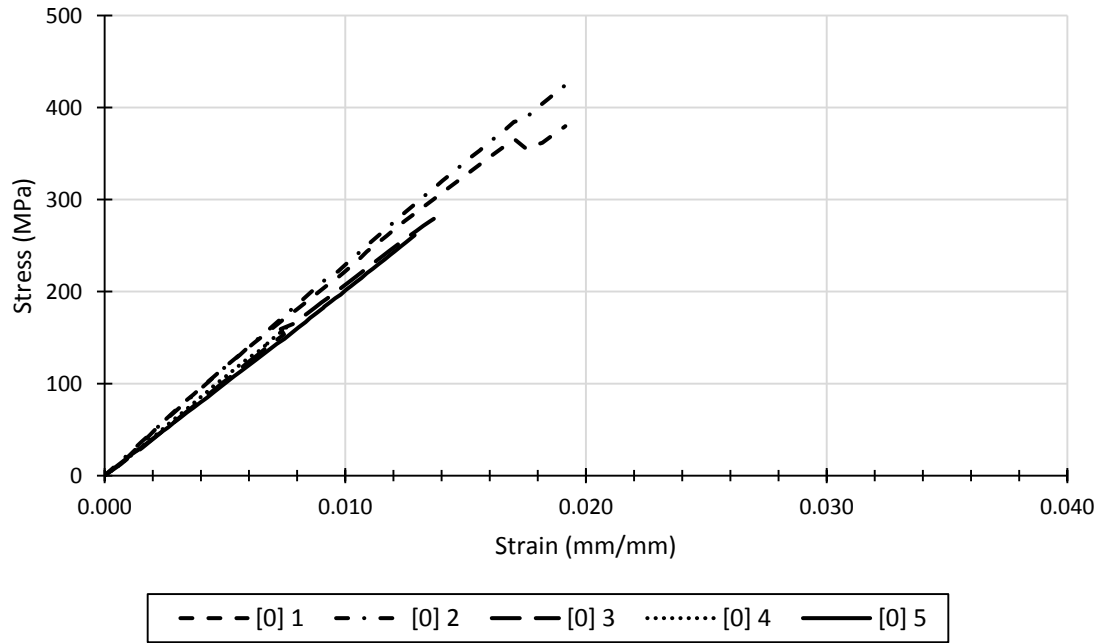


Figure C.1: Experimental Stress-Strain Curve for GFRP [0°]

Table C.1: Individual GFRP [0°] Coupon Test Results

Coupon	E_{GFRP}^1	f_{GFRP}^2	$\epsilon_{f GFRP}^3 \times 10^{-3}$	t
	MPa	MPa	mm/mm	mm
[0°] 1	21,378	379.8	19.2	0.96
[0°] 2	22,672	425.2	19.2	0.95
[0°] 3	20,629	286.3	14.1	1.27
[0°] 4	21,282	162.7	7.6	1.19
[0°] 5	20,128	261.2	12.9	1.21
Average	21,218	303.1	14.6	1.12
St. Dev.	959	103.1	4.8	0.15
COV	0.05	0.34	0.33	0.13

¹GFRP Modulus of Elasticity

²Ultimate Strength

³Failure Strain

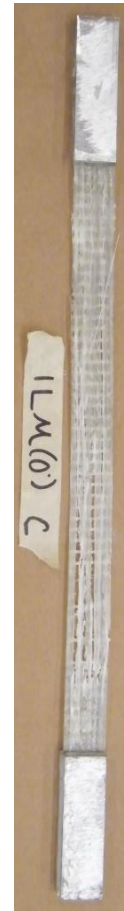
⁴Specimen Thickness



(a) [0°] 1



(b) [0°] 2



(c) [0°] 3



(d) [0°] 4



(e) [0°] 5

Figure C.2: Typical Individual Failure Modes for GFRP [0°] Coupons

GFRP – $[0^\circ]_2$

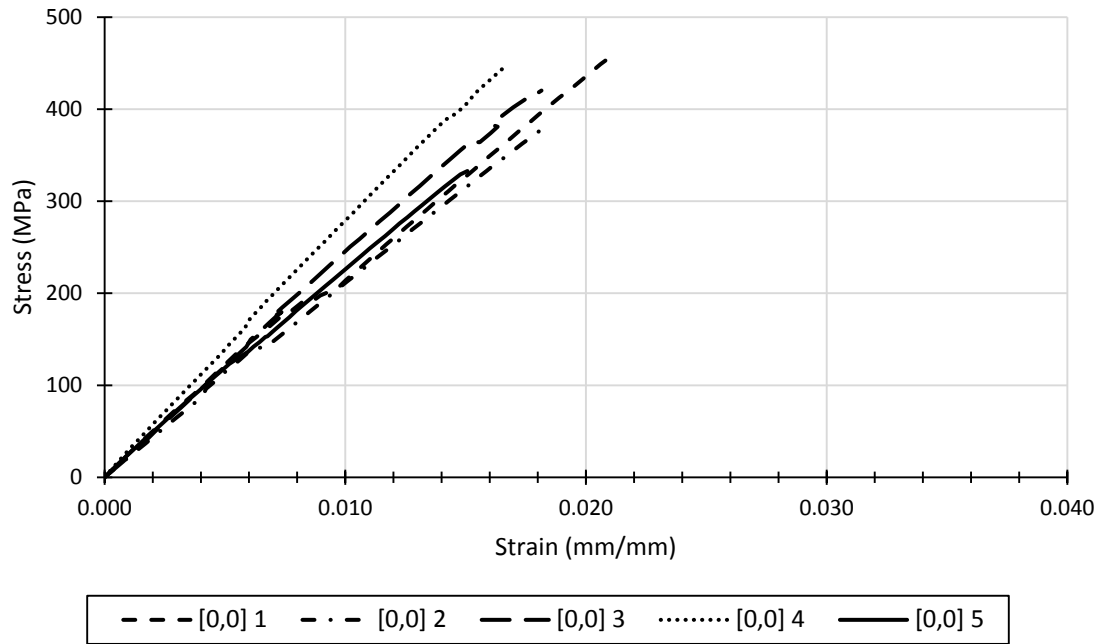


Figure C.3: Experimental Stress-Strain Curve for GFRP $[0^\circ]_2$

Table C.2: Individual GFRP $[0^\circ]_2$ Coupon Test Results

Coupon	E_{GFRP}^1	f_{GFRP}^2	$\epsilon_{f GFRP}^3 \times 10^{-3}$	t
	MPa	MPa	mm/mm	mm
$[0^\circ]_2$ 1	21,914	458.5	21.1	1.74
$[0^\circ]_2$ 2	21,063	386.0	18.5	1.85
$[0^\circ]_2$ 3	23,932	420.4	18.2	1.98
$[0^\circ]_2$ 4	27,507	448.6	16.7	1.89
$[0^\circ]_2$ 5	22,559	332.8	15.1	2.17
Average	23,395	409.2	17.9	1.93
St. Dev.	2,526	51.2	2.2	0.16
COV	0.11	0.13	0.13	0.08

¹GFRP Modulus of Elasticity

²Ultimate Strength

³Failure Strain

⁴Specimen Thickness



(a) $[0^\circ]_2$ 1



(b) $[0^\circ]_2$ 2



(c) $[0^\circ]_2$ 3



(d) $[0^\circ]_2$ 4



(e) $[0^\circ]_2$ 5

Figure C.4: Typical Individual Failure Modes for GFRP $[0^\circ]_2$ Coupons

GFRP – [0/90°]

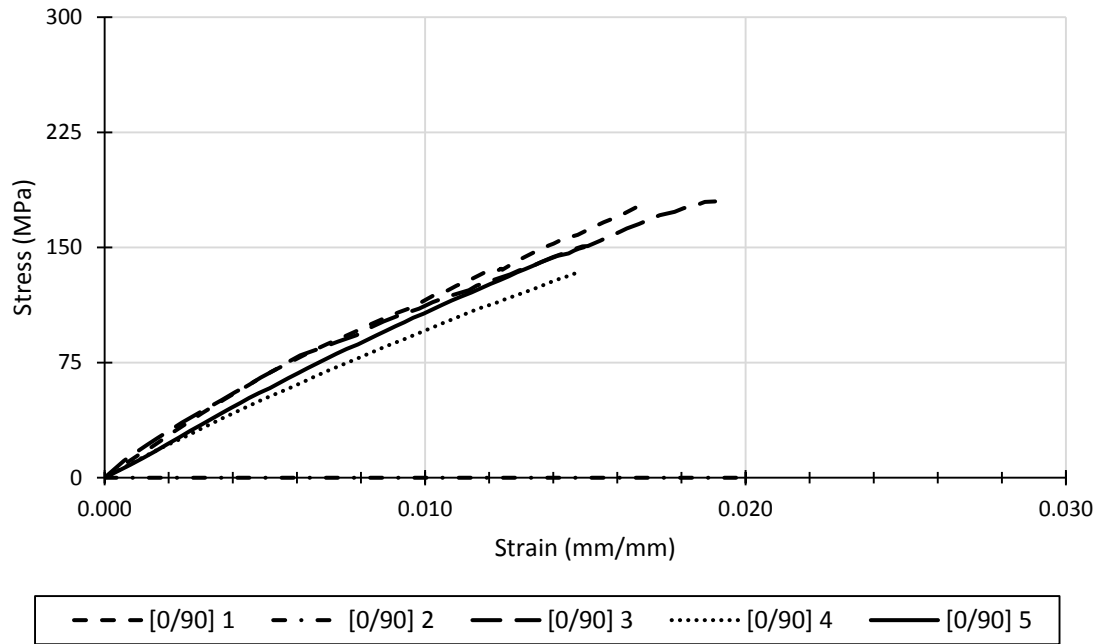


Figure C.5: Experimental Stress-Strain Curve for GFRP [0/90°]

Table C.3: Individual GFRP [0/90°] Coupon Test Results

Coupon	E_{GFRP}^1	f_{GFRP}^2	$\epsilon_{f GFRP}^3 \times 10^{-3}$	t
	MPa	MPa	mm/mm	mm
[0/90°] 1	11,137	176.0	16.6	1.39
[0/90°] 2	10,506	196.5	19.7	1.32
[0/90°] 3	10,254	180.0	19.1	1.40
[0/90°] 4	9,497	134.0	14.8	1.85
[0/90°] 5	10,525	150.1	15.0	1.86
Average	10,384	167.3	17.0	1.57
St. Dev.	593	25.0	2.3	0.27
COV	0.06	0.15	0.13	0.17

¹GFRP Modulus of Elasticity

²Ultimate Strength

³Failure Strain

⁴Specimen Thickness



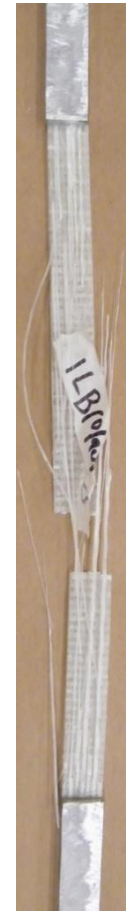
(a) [0/90°] 1



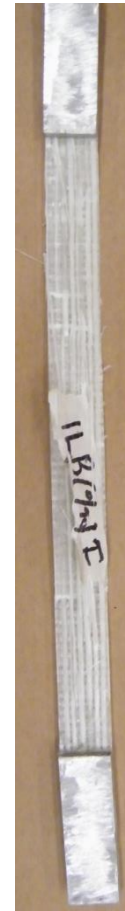
(b) [0/90°] 2



(c) [0/90°] 3



(d) [0/90°] 4



(e) [0/90°] 5

Figure C.6: Typical Individual Failure Modes for GFRP [0/90°] Coupons

GFRP – [0/90°]₂

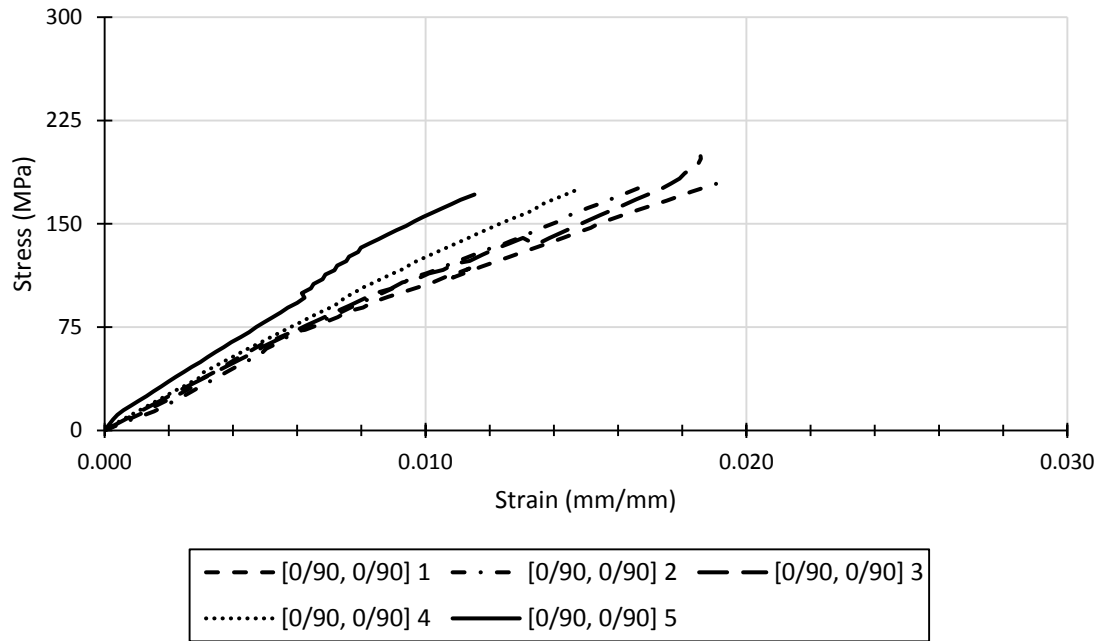


Figure C.7: Experimental Stress-Strain Curve for GFRP [0/90°]₂

Table C.4: Individual GFRP [0/90°]₂ Coupon Test Results

Coupon	E_{GFRP}^1	f_{GFRP}^2	$\epsilon_{f GFRP}^3 \times 10^{-3}$	t
	MPa	MPa	mm/mm	mm
[0/90°] ₂ 1	9,971	179.0	19.1	2.72
[0/90°] ₂ 2	10,950	177.7	16.8	2.65
[0/90°] ₂ 3	10,535	199.1	18.6	2.68
[0/90°] ₂ 4	12,372	174.4	14.7	3.00
[0/90°] ₂ 5	15,855	171.3	11.5	2.95
Average	11,937	180.3	16.1	2.80
St. Dev.	2,364	10.9	3.1	0.16
COV	0.20	0.06	0.19	0.06

¹GFRP Modulus of Elasticity

²Ultimate Strength

³Failure Strain

⁴Specimen Thickness



(a) $[0/90^\circ]_2$ 1



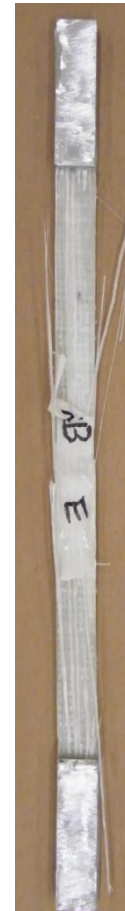
(b) $[0/90^\circ]_2$ 2



(c) $[0/90^\circ]_2$ 3



(d) $[0/90^\circ]_2$ 4



(e) $[0/90^\circ]_2$ 5

Figure C.8: Typical Individual Failure Modes for GFRP $[0/90^\circ]_2$ Coupons

GFRP – [0/90°]₄

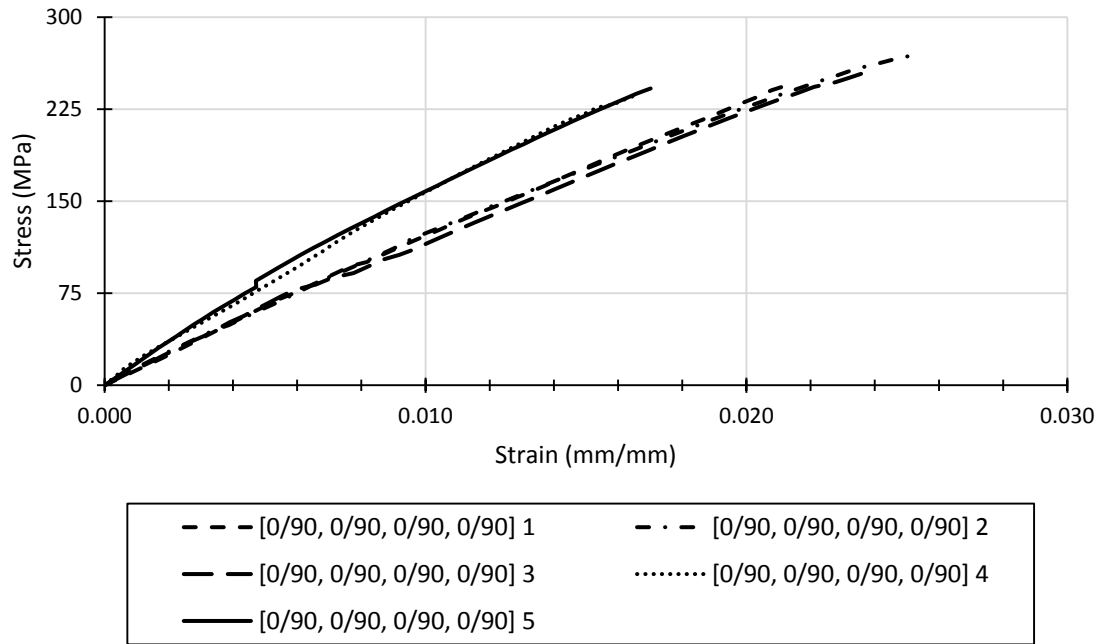


Figure C.9: Experimental Stress-Strain Curve for GFRP [0/90°]₄

Table C.5: Individual GFRP [0/90°]₄ Coupon Test Results

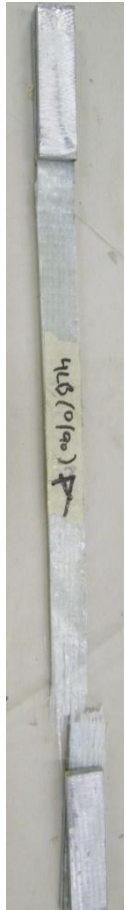
Coupon	E_{GFRP}^1	f_{GFRP}^2	$\epsilon_{f GFRP}^3 \times 10^{-3}$	t
	MPa	MPa	mm/mm	mm
[0/90°] ₄ 1	11,776	245.5	21.4	4.35
[0/90°] ₄ 2	11,428	269.3	25.3	4.43
[0/90°] ₄ 3	11,236	254.5	23.7	4.30
[0/90°] ₄ 4	15,239	236.3	16.5	4.63
[0/90°] ₄ 5	15,233	241.9	17.0	4.50
Average	12,982	249.5	20.8	4.44
St. Dev.	2,066	12.9	3.9	0.13
COV	0.16	0.05	0.19	0.03

¹GFRP Modulus of Elasticity

²Ultimate Strength

³Failure Strain

⁴Specimen Thickness



(a) $[0/90^\circ]_4$ 1



(b) $[0/90^\circ]_4$ 2



(c) $[0/90^\circ]_4$ 3



(d) $[0/90^\circ]_4$ 4



(e) $[0/90^\circ]_4$ 5

Figure C.10: Typical Individual Failure Modes for GFRP $[0/90^\circ]_4$ Coupons

GFRP – [± 45°]

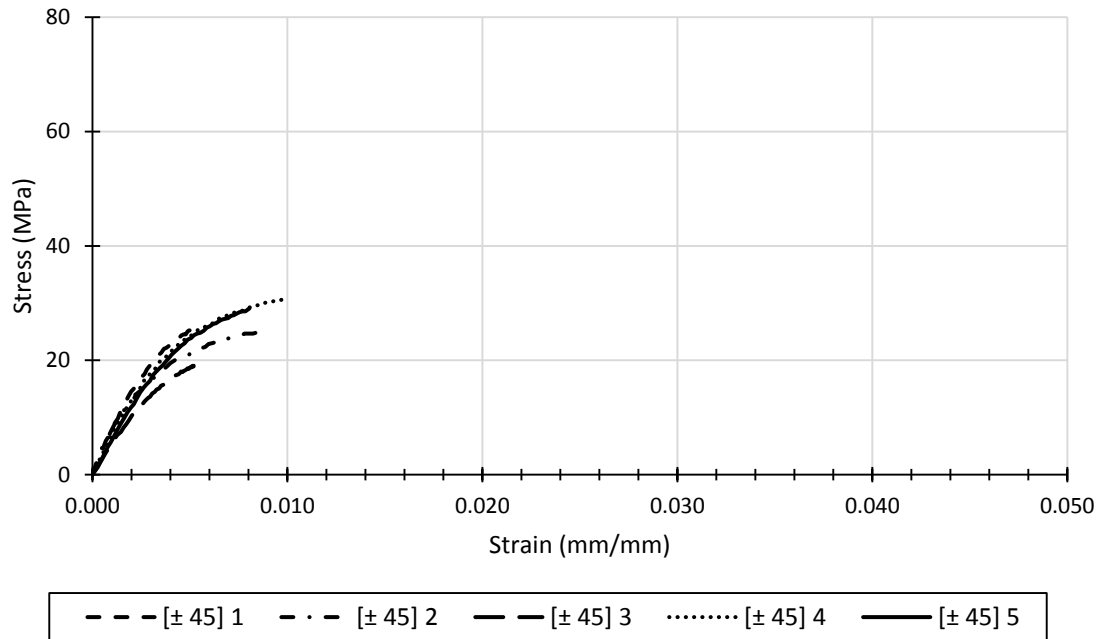


Figure C.11: Experimental Stress-Strain Curve for GFRP [± 45°]

Table C.6: Individual GFRP [± 45°] Coupon Test Results

Coupon	E_{GFRP}^1	f_{GFRP}^2	$\epsilon_{f GFRP}^3 \times 10^{-3}$	t
	MPa	MPa	mm/mm	mm
[± 45°] 1	7,638	25.4	5.1	0.95
[± 45°] 2	5,001	24.8	8.4	0.97
[± 45°] 3	5,182	19.0	5.1	1.78
[± 45°] 4	6,474	31.0	9.9	1.05
[± 45°] 5	5,806	29.1	8.0	1.26
Average	6,020	25.9	7.3	1.20
St. Dev.	1,074	4.6	2.1	0.35
COV	0.18	0.18	0.29	0.29

¹GFRP Modulus of Elasticity

²Ultimate Strength

³Failure Strain

⁴Specimen Thickness



(a) $[\pm 45^\circ]$ 1



(b) $[\pm 45^\circ]$ 2



(c) $[\pm 45^\circ]$ 3



(d) $[\pm 45^\circ]$ 4



(e) $[\pm 45^\circ]$ 5

Figure C.12: Typical Individual Failure Modes for GFRP $[\pm 45^\circ]$ Coupons

GFRP – $[\pm 45^\circ]_2$

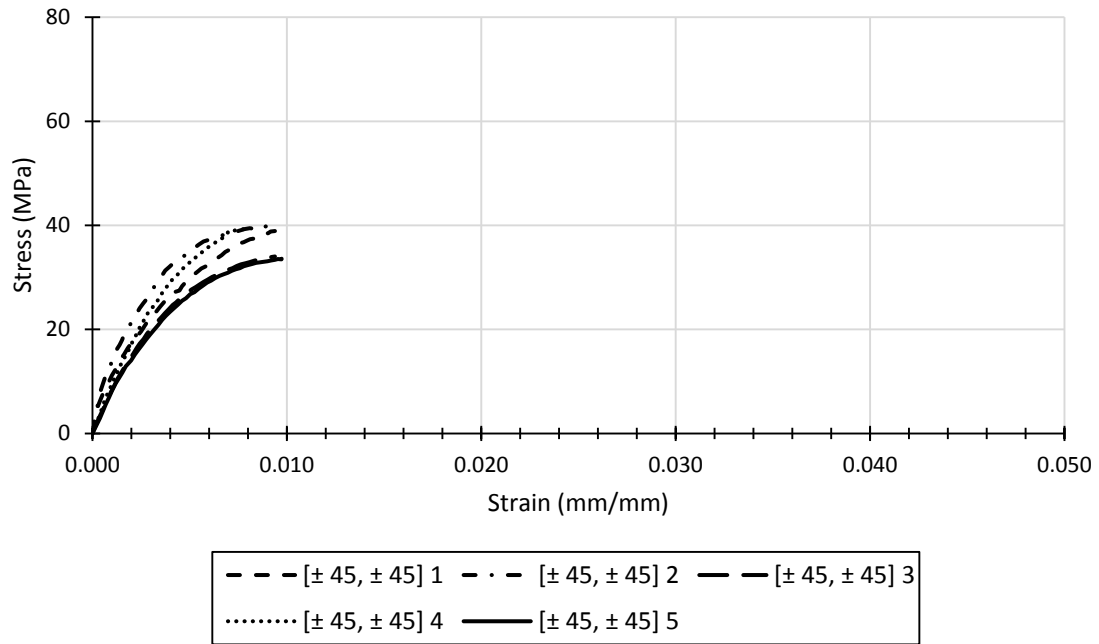


Figure C.13: Experimental Stress-Strain Curve for GFRP $[\pm 45^\circ]_2$

Table C.7: Individual GFRP $[\pm 45^\circ]_2$ Coupon Test Results

Coupon	E_{GFRP}^1	f_{GFRP}^2	$\epsilon_{f GFRP}^3 \times 10^{-3}$	t
	MPa	MPa	mm/mm	mm
$[\pm 45^\circ]_2 1$	6,131	38.9	9.4	1.89
$[\pm 45^\circ]_2 2$	7,533	40.0	9.2	1.89
$[\pm 45^\circ]_2 3$	7,536	34.0	9.4	2.29
$[\pm 45^\circ]_2 4$	8,389	39.5	8.5	2.10
$[\pm 45^\circ]_2 5$	7,300	33.6	9.7	2.19
Average	7,378	37.2	9.3	2.10
St. Dev.	811	3.1	0.5	0.20
COV	0.11	0.08	0.05	0.09

¹GFRP Modulus of Elasticity

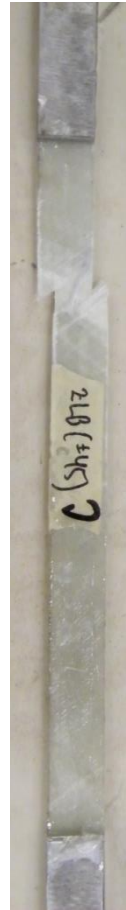
²Ultimate Strength

³Failure Strain

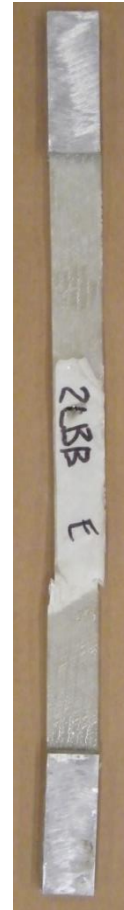
⁴Specimen Thickness



(a) $[\pm 45^\circ]_2$ 1



(b) $[\pm 45^\circ]_2$ 2



(c) $[\pm 45^\circ]_2$ 3



(d) $[\pm 45^\circ]_2$ 4



(e) $[\pm 45^\circ]_2$ 5

Figure C.14: Typical Individual Failure Modes for GFRP $[\pm 45^\circ]_2$ Coupons

GFRP – $[\pm 45^\circ]_4$

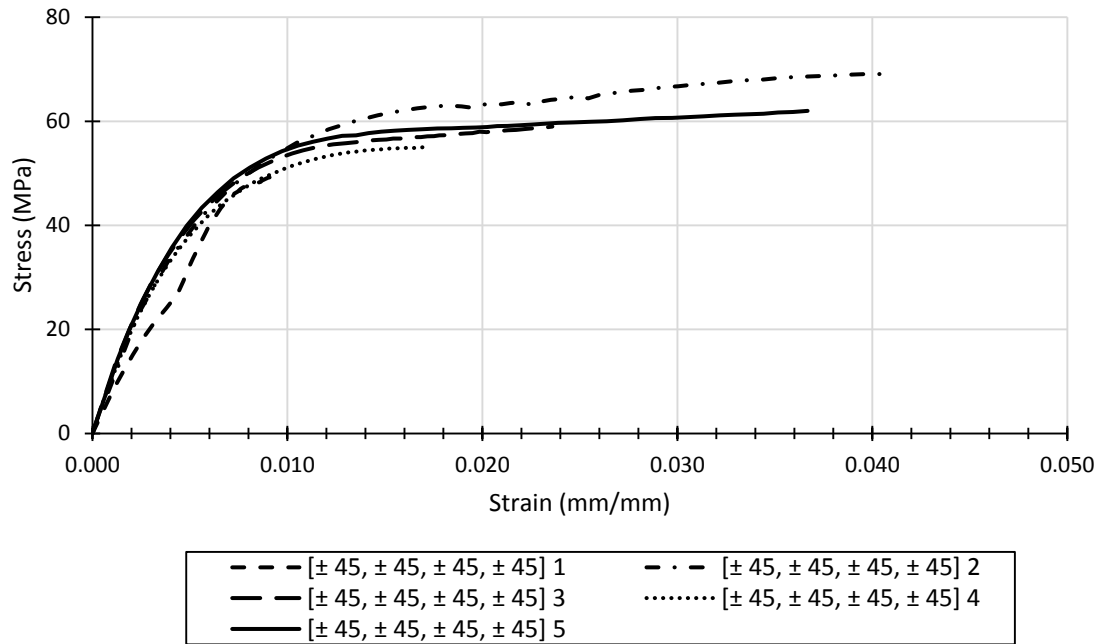


Figure C.15: Experimental Stress-Strain Curve for GFRP $[\pm 45^\circ]_4$

Table C.8: Individual GFRP $[\pm 45^\circ]_4$ Coupon Test Results

Coupon	E_{GFRP}^1	f_{GFRP}^2	$\epsilon_{f\ GFRP}^3 \times 10^{-3}$	t
	MPa	MPa	mm/mm	mm
$[\pm 45^\circ]_4$ 1	6,782	50.0	9.7	3.08
$[\pm 45^\circ]_4$ 2	7,346	69.1	37.0	3.23
$[\pm 45^\circ]_4$ 3	6,799	59.0	23.6	3.44
$[\pm 45^\circ]_4$ 4	9,389	55.1	17.1	3.34
$[\pm 45^\circ]_4$ 5	8,145	62.0	36.7	3.47
Average	7,692	59.0	25.3	3.31
St. Dev.	1,099	7.2	12.8	0.16
COV	0.14	0.12	0.50	0.05

¹GFRP Modulus of Elasticity

²Ultimate Strength

³Failure Strain

⁴Specimen Thickness



(a) $[\pm 45^\circ]_4$ 1



(b) $[\pm 45^\circ]_4$ 2



(c) $[\pm 45^\circ]_4$ 3



(d) $[\pm 45^\circ]_4$ 4



(e) $[\pm 45^\circ]_4$ 5

Figure C.16: Typical Individual Failure Modes for GFRP $[\pm 45^\circ]_4$ Coupons

GFRP – $[0, \pm 45^\circ]$

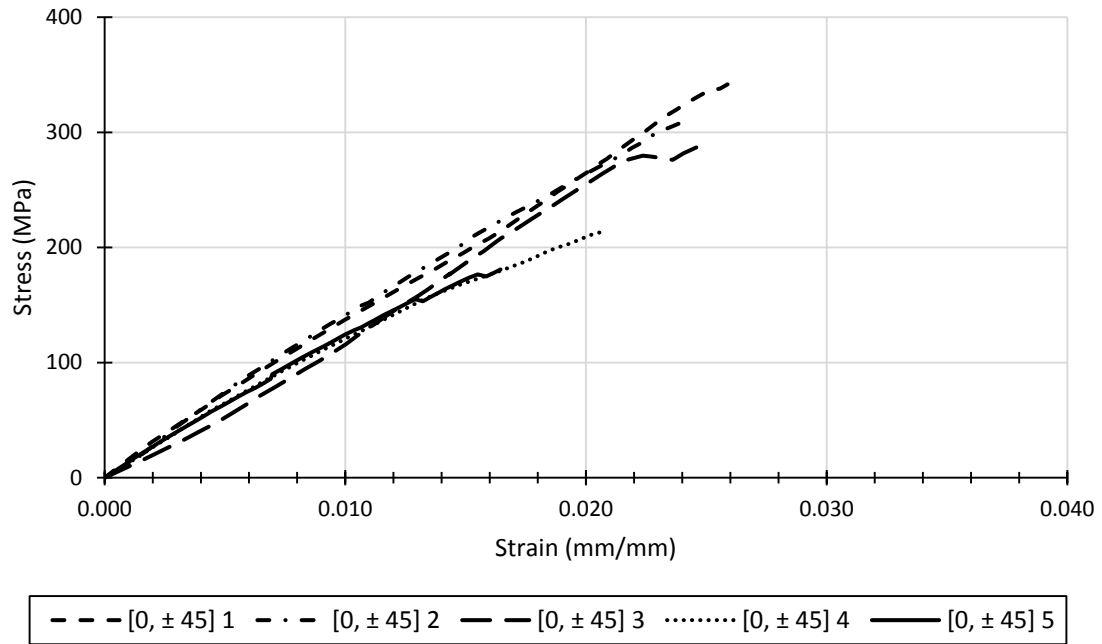


Figure C.17: Experimental Stress-Strain Curve for GFRP $[0, \pm 45^\circ]$

Table C.9: Individual GFRP $[0, \pm 45^\circ]$ Coupon Test Results

Coupon	E_{GFRP}^1	f_{GFRP}^2	$\epsilon_{f GFRP}^3 \times 10^{-3}$	t
	MPa	MPa	mm/mm	mm
$[0, \pm 45^\circ]$ 1	13,318	341.8	25.9	0.92
$[0, \pm 45^\circ]$ 2	13,386	308.1	23.9	1.07
$[0, \pm 45^\circ]$ 3	12,286	291.1	25.0	1.01
$[0, \pm 45^\circ]$ 4	11,192	213.9	20.7	1.44
$[0, \pm 45^\circ]$ 5	11,860	181.0	16.5	1.54
Average	12,408	267.2	22.4	1.20
St. Dev.	946	67.2	3.9	0.28
COV	0.08	0.25	0.17	0.23

¹GFRP Modulus of Elasticity

²Ultimate Strength

³Failure Strain

⁴Specimen Thickness



(a) $[0, \pm 45^\circ]$ 1



(b) $[0, \pm 45^\circ]$ 2



(c) $[0, \pm 45^\circ]$ 3



(d) $[0, \pm 45^\circ]$ 4



(e) $[0, \pm 45^\circ]$ 5

Figure C.18: Typical Individual Failure Modes for GFRP $[0, \pm 45^\circ]$ Coupons

GFRP – $[0, \pm 45^\circ]_2$

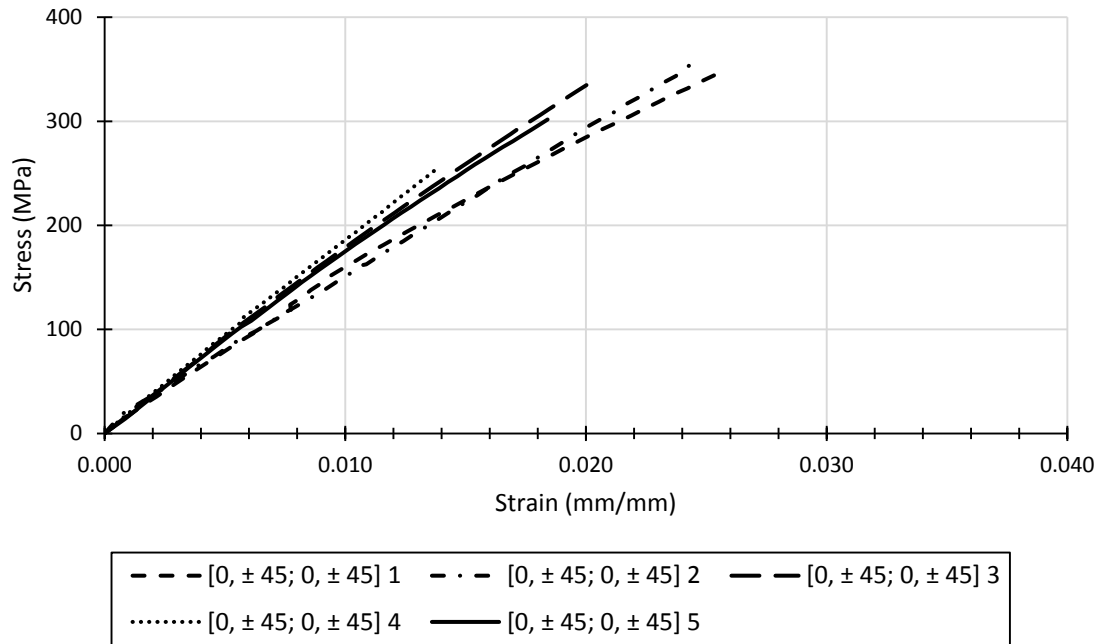


Figure C.19: Experimental Stress-Strain Curve for GFRP $[0, \pm 45^\circ]_2$

Table C.10: Individual GFRP $[0, \pm 45^\circ]_2$ Coupon Test Results

Coupon	E_{GFRP}^1	f_{GFRP}^2	$\epsilon_{f GFRP}^3 \times 10^{-3}$	t
	MPa	MPa	mm/mm	mm
$[0, \pm 45^\circ]_2$ 1	14,424	346.0	25.5	1.88
$[0, \pm 45^\circ]_2$ 2	14,720	358.2	24.8	1.90
$[0, \pm 45^\circ]_2$ 3	17,261	334.8	20.0	1.98
$[0, \pm 45^\circ]_2$ 4	18,627	254.0	13.8	2.08
$[0, \pm 45^\circ]_2$ 5	16,961	301.7	18.4	2.22
Average	16,339	319.0	20.5	2.01
St. Dev.	1,785	41.9	4.8	0.14
COV	0.11	0.13	0.24	0.07

¹GFRP Modulus of Elasticity

²Ultimate Strength

³Failure Strain

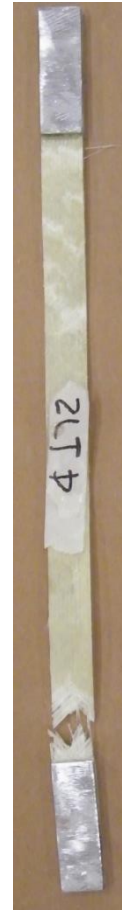
⁴Specimen Thickness



(a) $[0, \pm 45^\circ]_2$ 1



(b) $[0, \pm 45^\circ]_2$ 2



(c) $[0, \pm 45^\circ]_2$ 3



(d) $[0, \pm 45^\circ]_2$ 4



(e) $[0, \pm 45^\circ]_2$ 5

Figure C.20: Typical Individual Failure Modes for GFRP $[0, \pm 45^\circ]_2$ Coupons

GFRP – [0/90°, ± 45°]

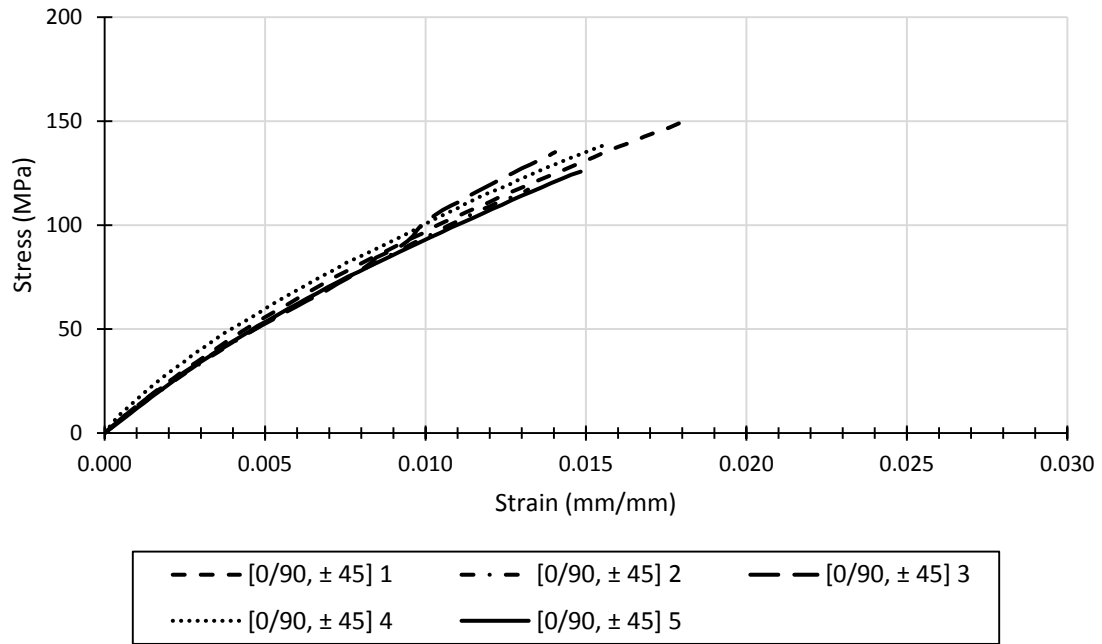


Figure C.21: Experimental Stress-Strain Curve for GFRP [0/90°, ± 45°]

Table C.11: Individual GFRP [0/90°, ± 45°] Coupon Test Results

Coupon	E_{GFRP}^1	f_{GFRP}^2	$\epsilon_{f GFRP}^3 \times 10^{-3}$	t
	MPa	MPa	mm/mm	mm
[0/90°, ± 45°] 1	9,017	150.2	18.2	2.18
[0/90°, ± 45°] 2	9,454	117.1	13.2	1.92
[0/90°, ± 45°] 3	9,933	135.1	14.0	2.14
[0/90°, ± 45°] 4	9,737	139.0	15.7	2.27
[0/90°, ± 45°] 5	9,152	125.7	14.8	2.45
Average	9,459	133.4	15.2	2.19
St. Dev.	385	12.7	1.9	0.19
COV	0.04	0.10	0.13	0.09

¹GFRP Modulus of Elasticity

²Ultimate Strength

³Failure Strain

⁴Specimen Thickness



(a) $[0/90^\circ, \pm 45^\circ]$ 1



(b) $[0/90^\circ, \pm 45^\circ]$ 2



(c) $[0/90^\circ, \pm 45^\circ]$ 3



(d) $[0/90^\circ, \pm 45^\circ]$ 4



(e) $[0/90^\circ, \pm 45^\circ]$ 5

Figure C.22: Typical Individual Failure Modes for GFRP $[0/90^\circ, \pm 45^\circ]$ Coupons

GFRP – $[0/90^\circ, \pm 45^\circ]_2$

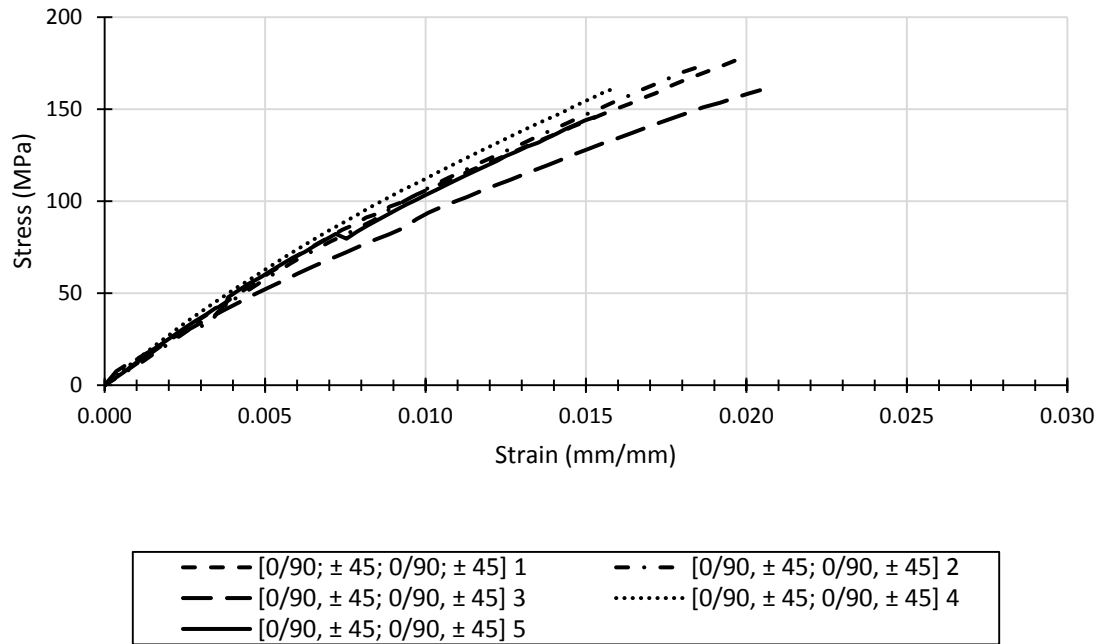


Figure C.23: Experimental Stress-Strain Curve for GFRP $[0/90^\circ, \pm 45^\circ]_2$

Table C.12: Individual GFRP $[0/90^\circ, \pm 45^\circ]_2$ Coupon Test Results

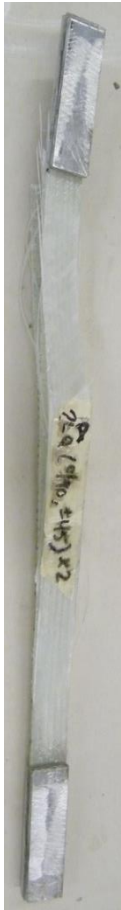
Coupon	E_{GFRP}^1	f_{GFRP}^2	$\epsilon_{f GFRP}^3 \times 10^{-3}$	t
	MPa	MPa	mm/mm	mm
$[0/90^\circ, \pm 45^\circ]_2$ 1	9,712	178.1	19.9	3.92
$[0/90^\circ, \pm 45^\circ]_2$ 2	10,008	173.7	18.6	4.10
$[0/90^\circ, \pm 45^\circ]_2$ 3	8,523	160.3	20.4	4.21
$[0/90^\circ, \pm 45^\circ]_2$ 4	10,901	161.8	16.0	4.22
$[0/90^\circ, \pm 45^\circ]_2$ 5	10,170	145.7	15.3	4.43
Average	9,863	163.9	18.0	4.18
St. Dev.	867	12.7	2.3	0.19
COV	0.09	0.08	0.13	0.04

¹GFRP Modulus of Elasticity

²Ultimate Strength

³Failure Strain

⁴Specimen Thickness



(a) $[0/90^\circ, \pm 45^\circ]_2$ 1



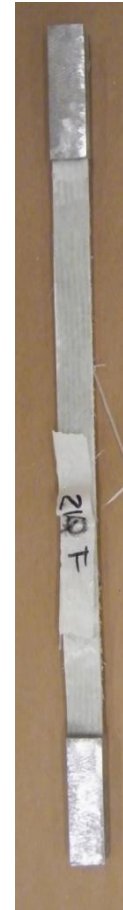
(b) $[0/90^\circ, \pm 45^\circ]_2$ 2



(c) $[0/90^\circ, \pm 45^\circ]_2$ 3



(d) $[0/90^\circ, \pm 45^\circ]_2$ 4



(e) $[0/90^\circ, \pm 45^\circ]_2$ 5

Figure C.24: Typical Individual Failure Modes for GFRP $[0/90^\circ, \pm 45^\circ]_2$ Coupons

GFRP – $[0/90^\circ]_2 [\pm 45^\circ]_2$

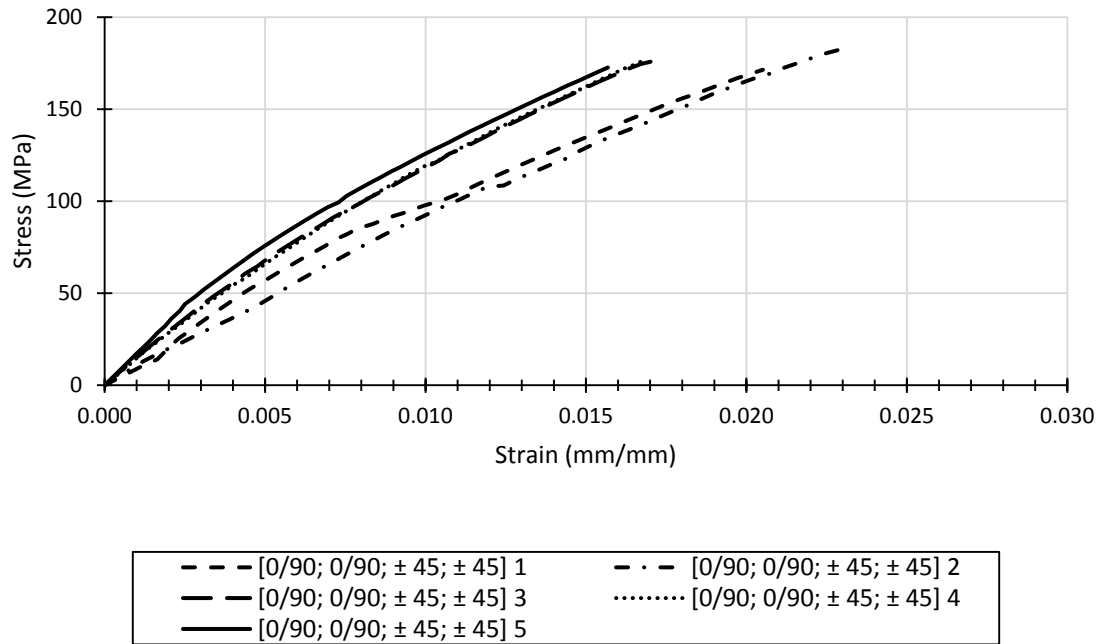


Figure C.25: Experimental Stress-Strain Curve for GFRP $[0/90^\circ]_2 [\pm 45^\circ]_2$

Table C.13: Individual GFRP $[0/90^\circ]_2 [\pm 45^\circ]_2$ Coupon Test Results

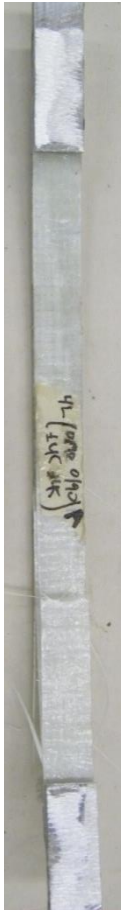
Coupon	E_{GFRP}^1	f_{GFRP}^2	$\epsilon_{f GFRP}^3 \times 10^{-3}$	t
	MPa	MPa	mm/mm	mm
$[0/90^\circ]_2 [\pm 45^\circ]_2 1$	9,077	171.4	20.5	3.72
$[0/90^\circ]_2 [\pm 45^\circ]_2 2$	8,452	182.5	22.9	3.57
$[0/90^\circ]_2 [\pm 45^\circ]_2 3$	11,266	175.8	17.0	3.91
$[0/90^\circ]_2 [\pm 45^\circ]_2 4$	11,344	176.3	16.8	3.97
$[0/90^\circ]_2 [\pm 45^\circ]_2 5$	12,163	172.7	15.7	3.98
Average	10,461	175.8	18.6	3.83
St. Dev.	1,603	4.3	3.0	0.18
COV	0.15	0.02	0.16	0.05

¹GFRP Modulus of Elasticity

²Ultimate Strength

³Failure Strain

⁴Specimen Thickness



(a) $[0/90^\circ]_2 [\pm 45^\circ]_2$ 1



(b) $[0/90^\circ]_2 [\pm 45^\circ]_2$ 2



(c) $[0/90^\circ]_2 [\pm 45^\circ]_2$ 3



(d) $[0/90^\circ]_2 [\pm 45^\circ]_2$ 4



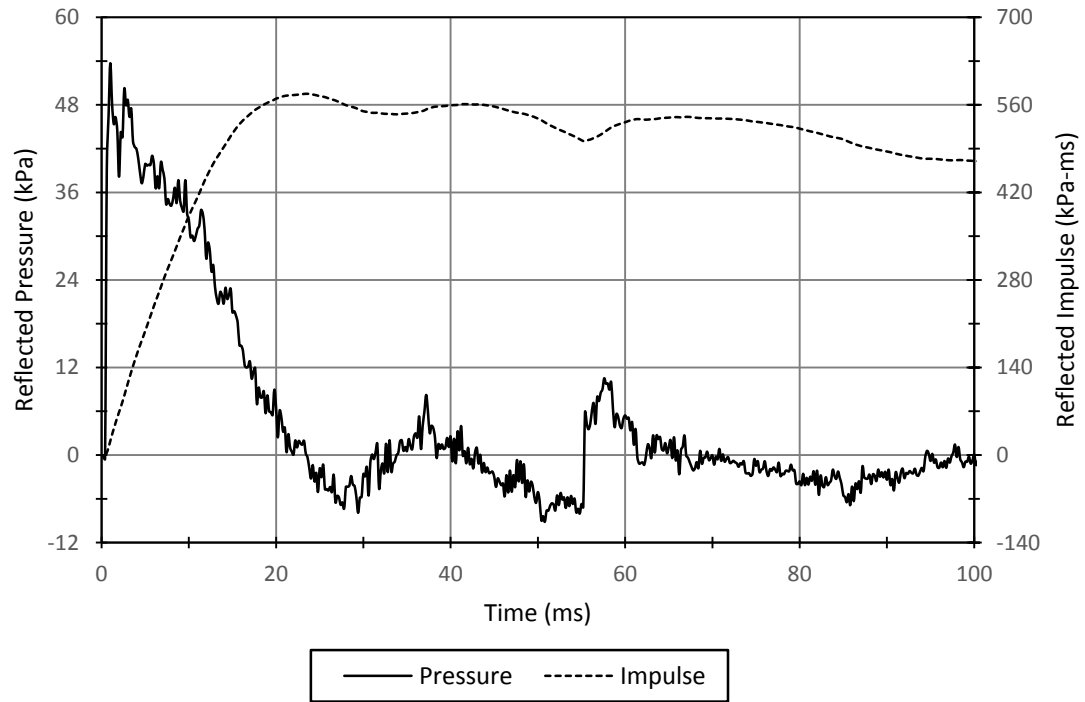
(e) $[0/90^\circ]_2 [\pm 45^\circ]_2$ 5

Figure C.26: Typical Individual Failure Modes for GFRP $[0/90^\circ]_2 [\pm 45^\circ]_2$ Coupons

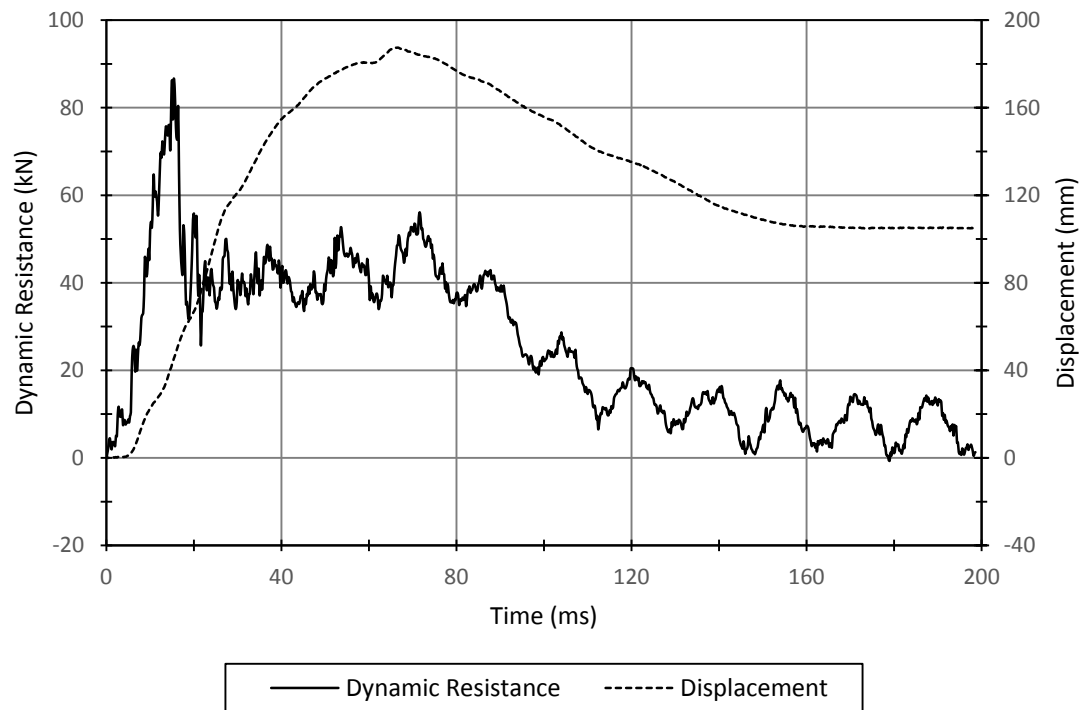
APPENDIX D – Phase 1 Dynamic Test Results

CLT3-1

Test Date:	June 14, 2016	Slab Mass:	56.8	<i>kg</i>
Specimen No.:	1	Volume:	0.1168125	<i>m³</i>
Test Name:	CLT3-1	Density:	544.46	<i>kg/m³</i>
Shot No.:	1	Moisture Content:	13.0	<i>%</i>
Total Slab Height:	2,500		<i>mm</i>	
Slab Clear Span:	2,235		<i>mm</i>	
Configuration:	H [24]			
Approximate Length of Straps Used:	27.62		<i>m</i>	
Driver Length:	2,745		<i>mm</i>	
Driver Pressure:	424.0		<i>kPa</i>	
Maximum Reflected Pressure:	53.7		<i>kPa</i>	
Maximum Reflected Impulse:	577.7		<i>kPa – ms</i>	
Positive Phase Duration:	23.4		<i>ms</i>	
Maximum Mid-Span Deflection:	187.5		<i>mm</i>	
Time to Mid-Span Deflection:	66.0		<i>ms</i>	
Maximum Dynamic Resistance:	86.4		<i>kN</i>	
Deflection at Dynamic Resistance:	45.3		<i>mm</i>	

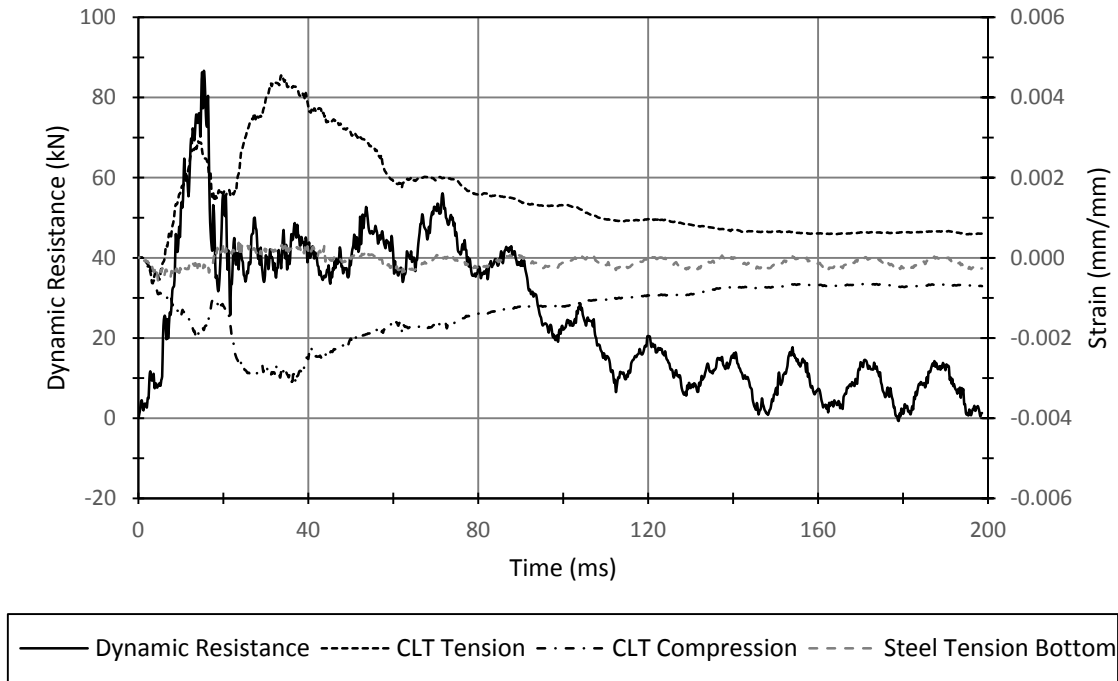


(a) Pressure and Impulse

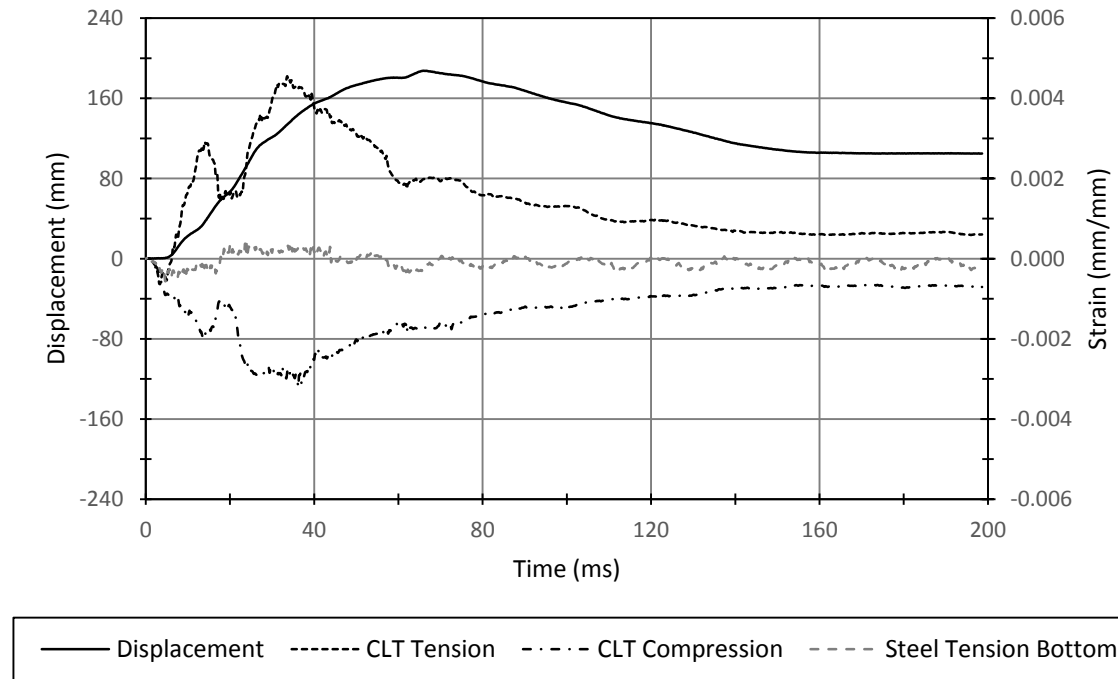


(b) Dynamic Resistance and Displacement

Figure D.1: Test Results for CLT3-1

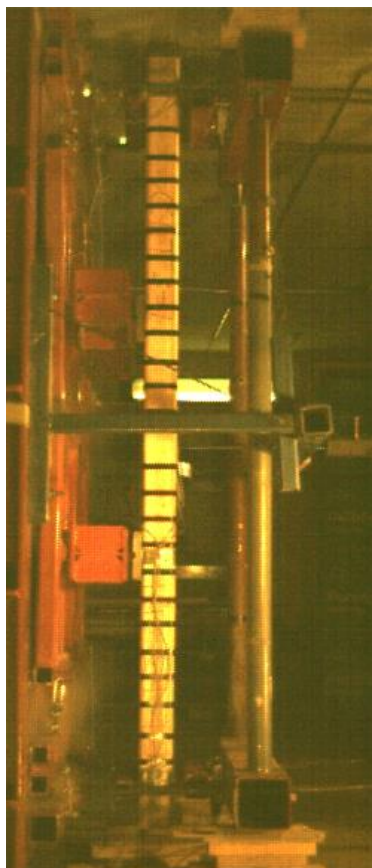


(a) Dynamic Resistance and Strain

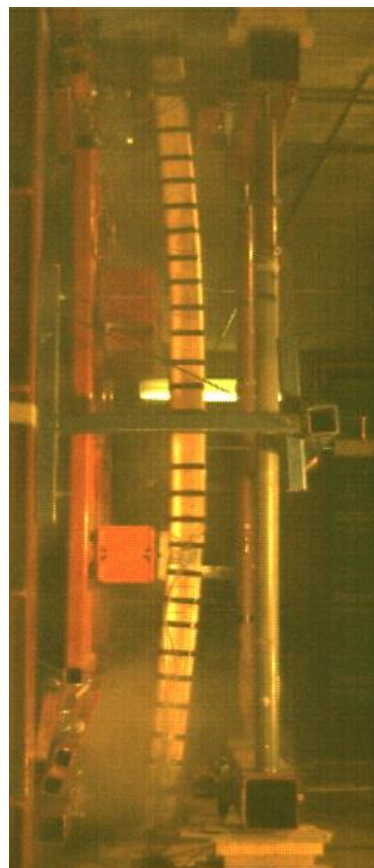


(b) Displacement and Strain

Figure D.2: Test Results for CLT3-1 Continued



(a) Initial Position



(b) Maximum Dynamic Resistance

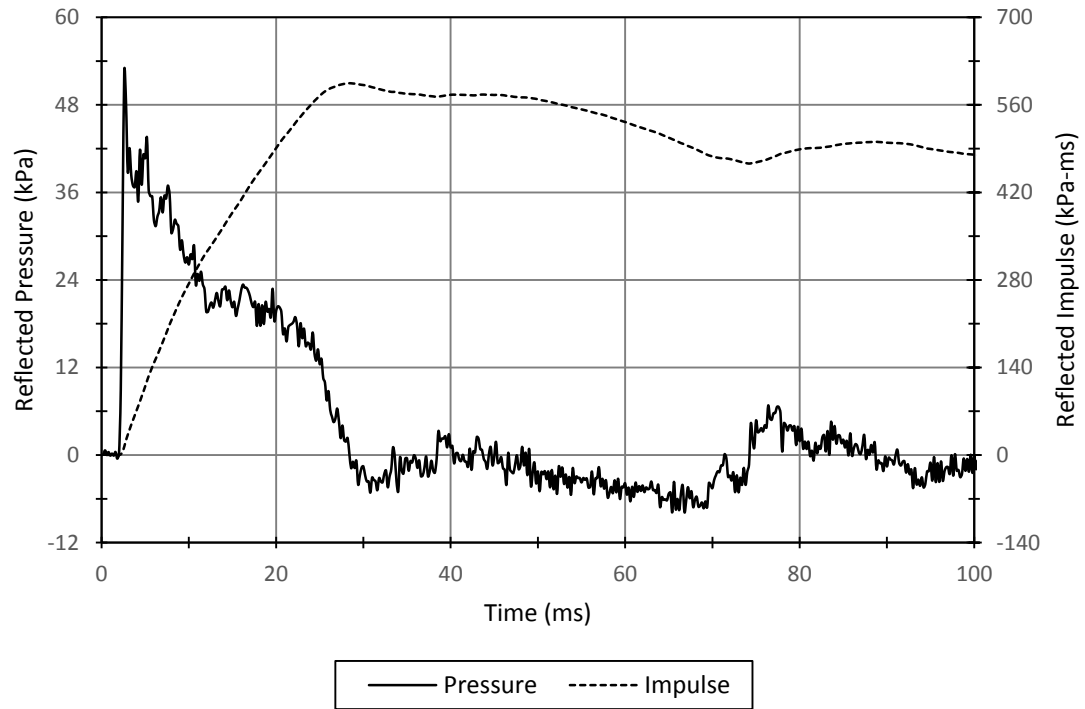


(c) Maximum Displacement

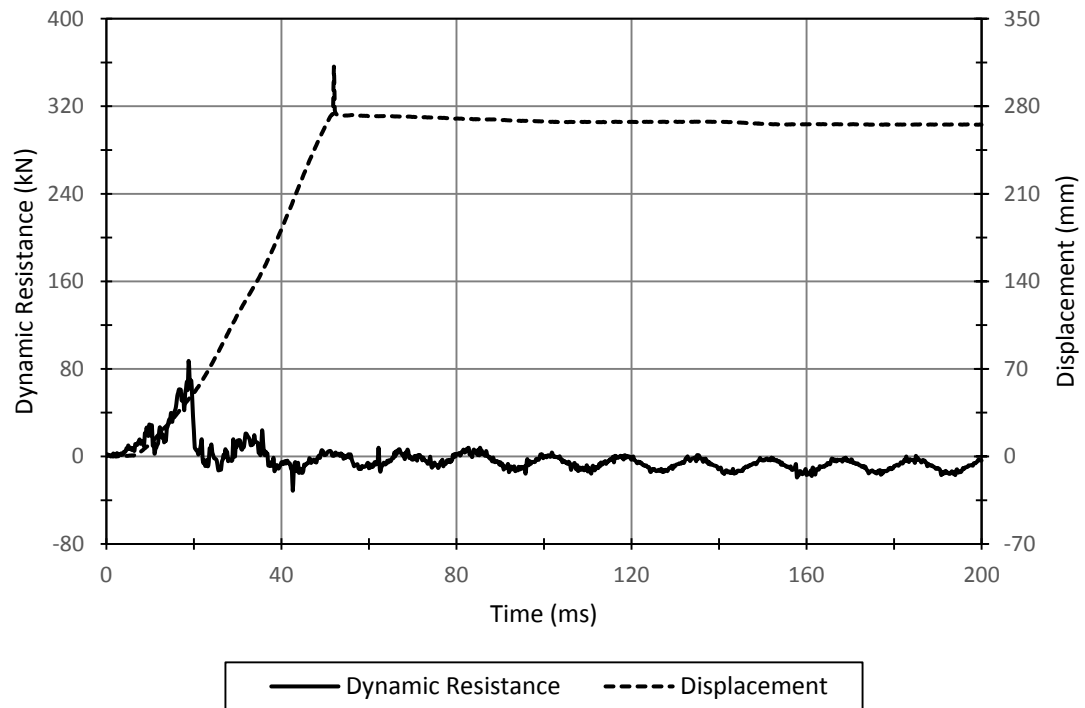
Figure D.3: Progression of Failure of CLT3-1

CLT3-2

Test Date:	June 22, 2016	Slab Mass:	62.6	<i>kg</i>
Specimen No.:	2	Volume:	0.1168125	<i>m</i> ³
Test Name:	CLT3-2	Density:	536.2	<i>kg/m</i> ³
Shot No.:	1	Moisture Content:	13.0	%
Total Slab Height:	2,500		<i>mm</i>	
Slab Clear Span:	2,235		<i>mm</i>	
Configuration:	H [16]			
Approximate Length of Straps Used:	18.4		<i>m</i>	
Driver Length:	4,268		<i>mm</i>	
Driver Pressure:	353.7		<i>kPa</i>	
Maximum Reflected Pressure:	52.5		<i>kPa</i>	
Maximum Reflected Impulse:	594.6		<i>kPa – ms</i>	
Positive Phase Duration:	28.4		<i>ms</i>	
Maximum Mid-Span Deflection:	311.7		<i>mm</i>	
Time to Mid-Span Deflection:	52.0		<i>ms</i>	
Maximum Dynamic Resistance:	366.8		<i>kN</i>	
Deflection at Dynamic Resistance:	58.9		<i>mm</i>	

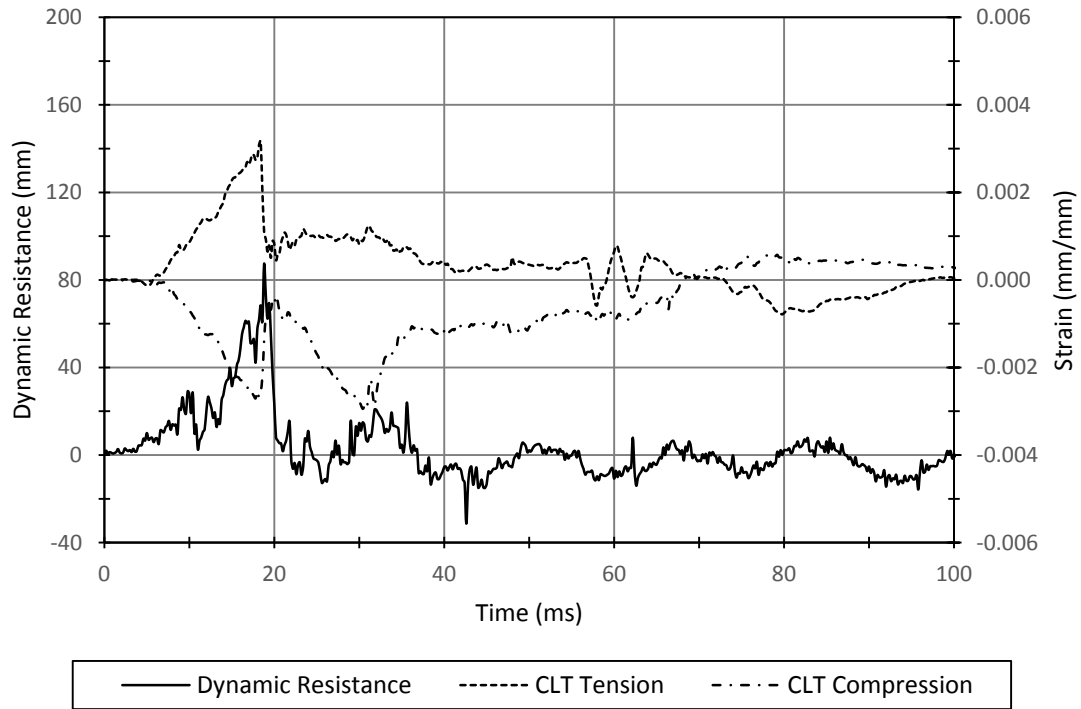


(a) Pressure and Impulse

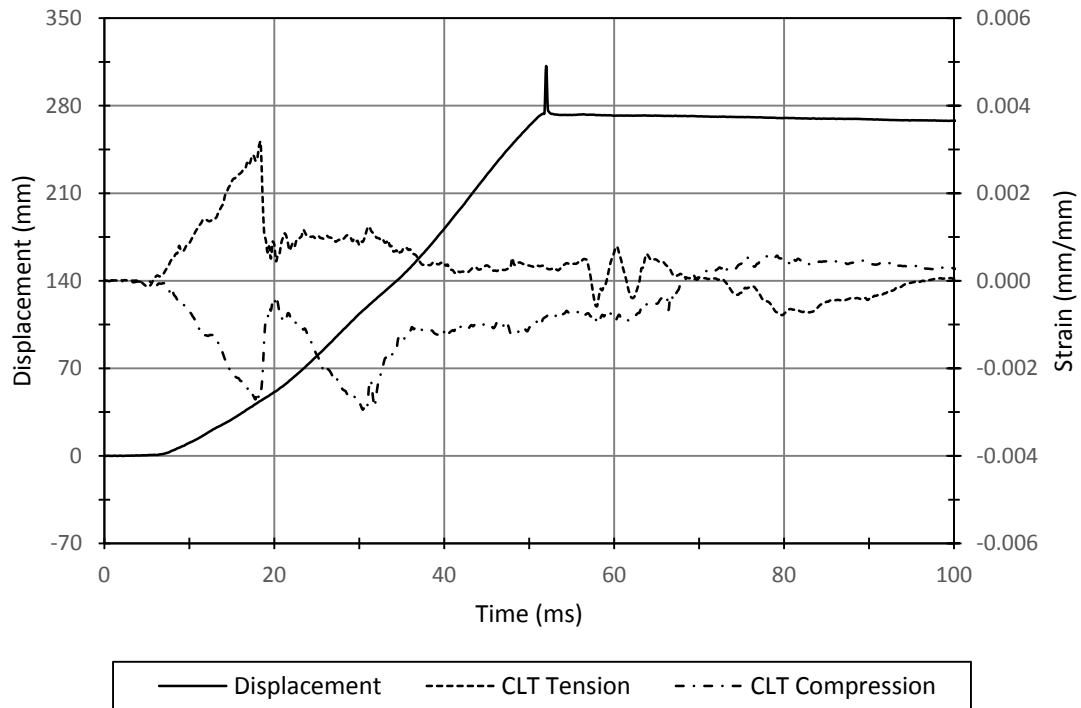


(b) Dynamic Resistance and Displacement

Figure D.4: Test Results for CLT3-2

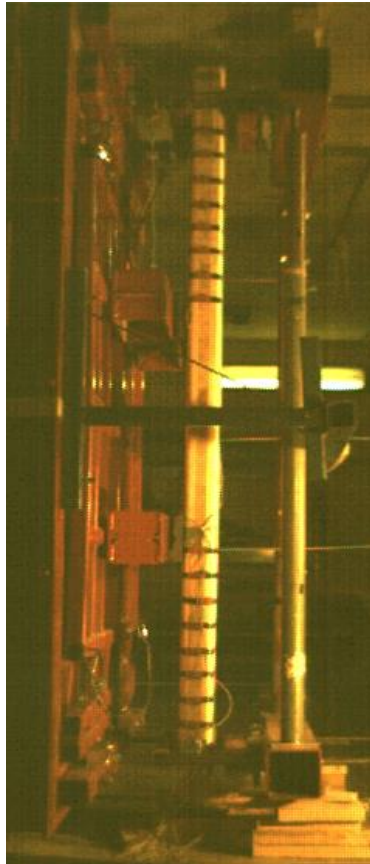


(a) Dynamic Resistance and Strain



(b) Displacement and Strain

Figure D.5: Test Results for CLT3-2 Continued



(a) Initial Position



(b) Maximum Dynamic Resistance



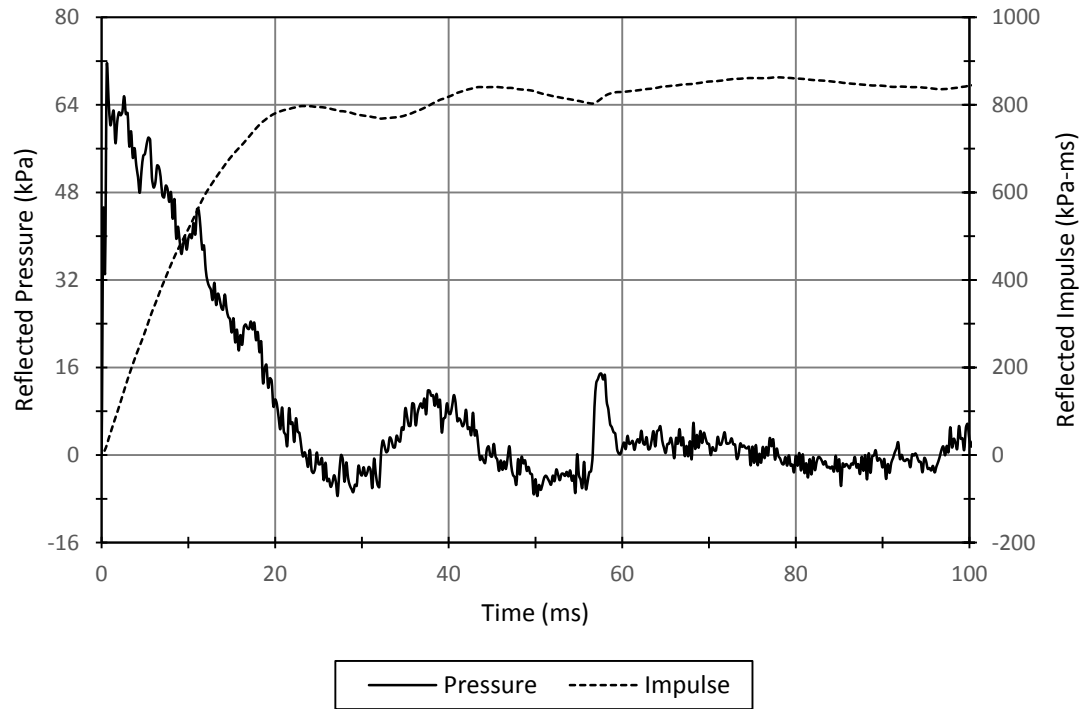
(c) Maximum Displacement

Figure D.6: Progression of Failure of CLT3-2

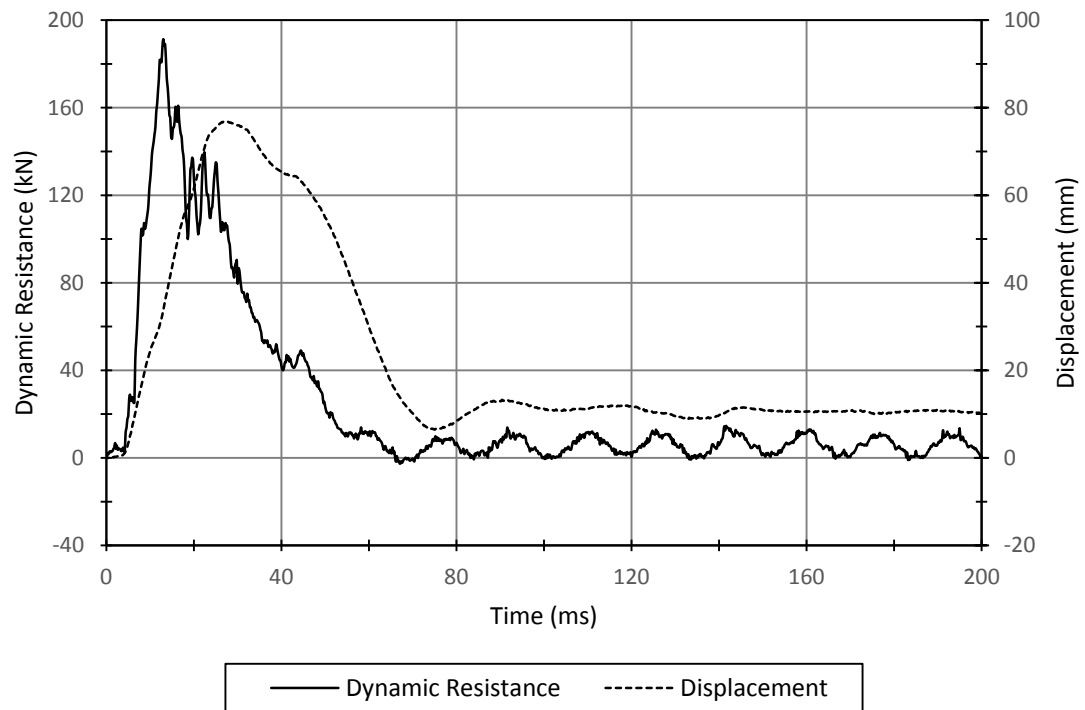
CLT5-1.1

Test Date:	June 15, 2016	Slab Mass:	100.3	<i>kg</i>
Specimen No.:	3	Volume:	0.1946875	<i>m³</i>
Test Name:	CLT5-1.1	Density:	548.47	<i>kg/m³</i>
Shot No.:	1	Moisture Content:	12.5	%

Total Slab Height:	2,500	<i>mm</i>
Slab Clear Span:	2,235	<i>mm</i>
Configuration:	H [24] + V [5]	
Approximate Length of Straps Used:	57.98	<i>m</i>
Driver Length:	4,268	<i>mm</i>
Driver Pressure:	597.1	<i>kPa</i>
Maximum Reflected Pressure:	70.9	<i>kPa</i>
Maximum Reflected Impulse:	880.9	<i>kPa – ms</i>
Positive Phase Duration:	23.2	<i>ms</i>
Maximum Mid-Span Deflection:	76.9	<i>mm</i>
Time to Mid-Span Deflection:	26.8	<i>ms</i>
Maximum Dynamic Resistance:	191.3	<i>kN</i>
Deflection at Dynamic Resistance:	33.7	<i>mm</i>

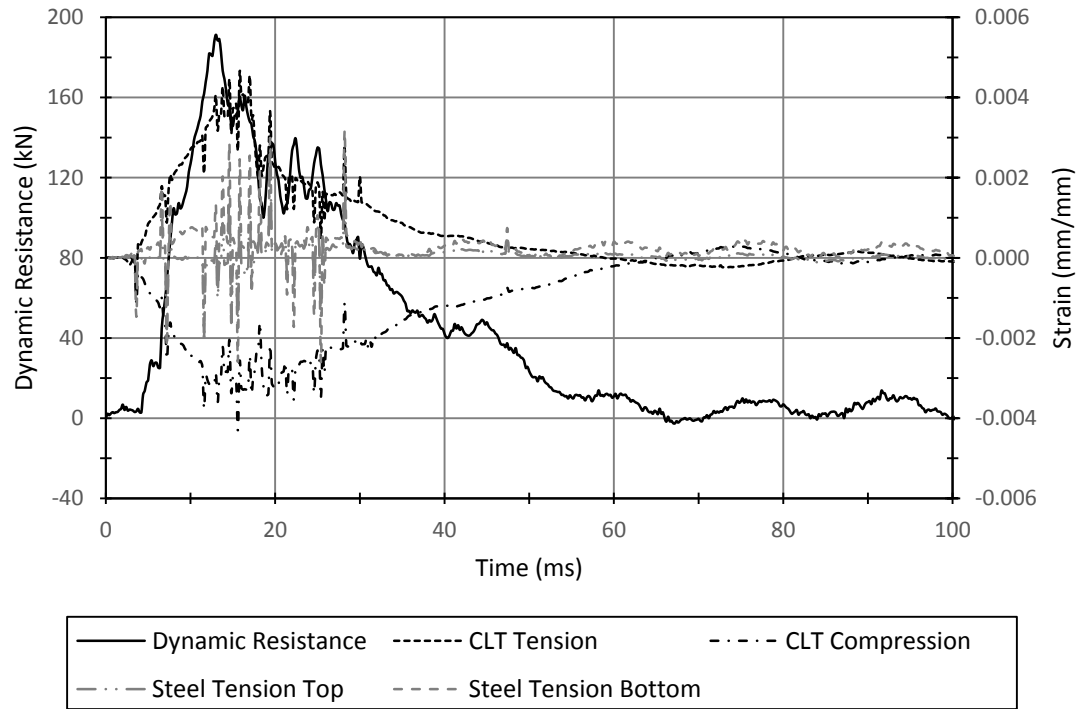


(a) Pressure and Impulse

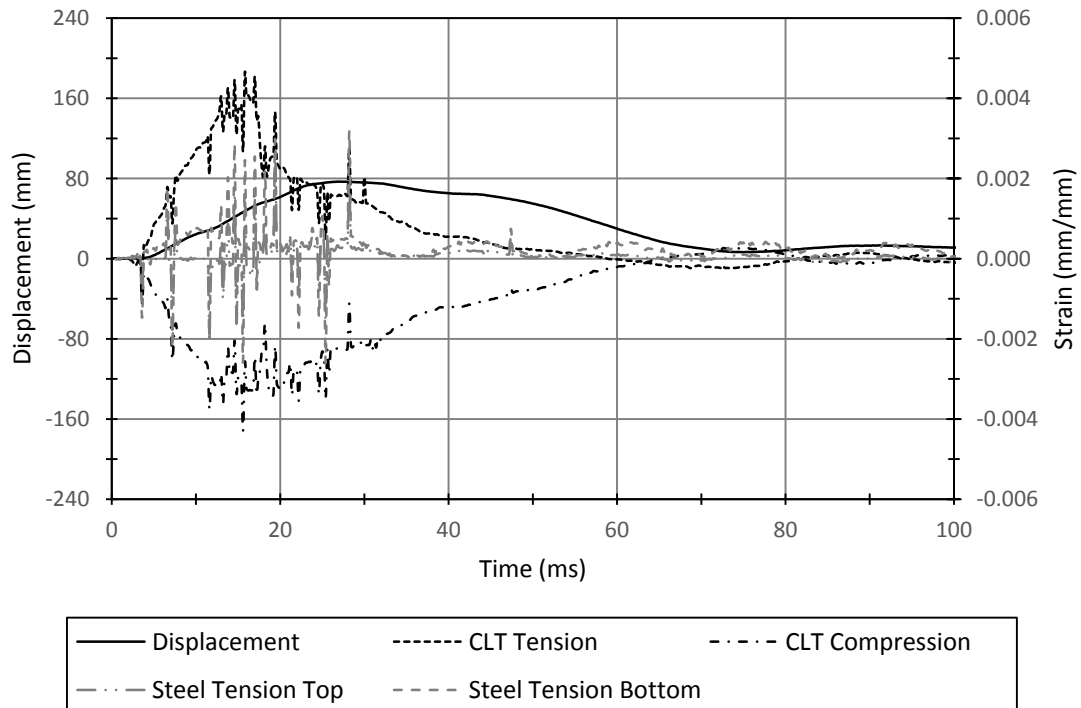


(b) Dynamic Resistance and Displacement

Figure D.7: Test Results for CLT5-1.1

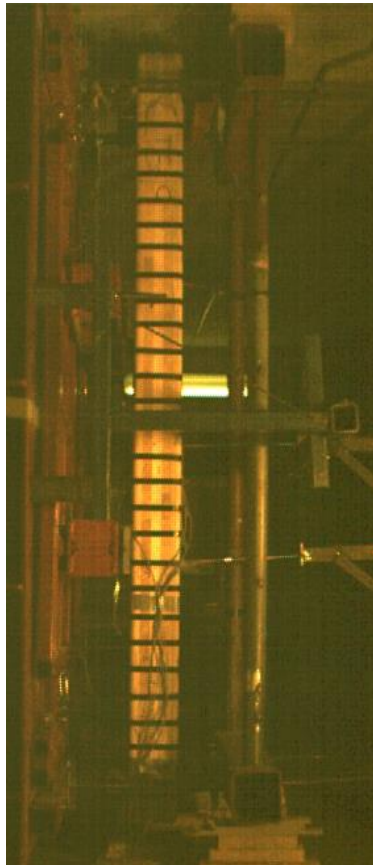


(a) Dynamic Resistance and Strain

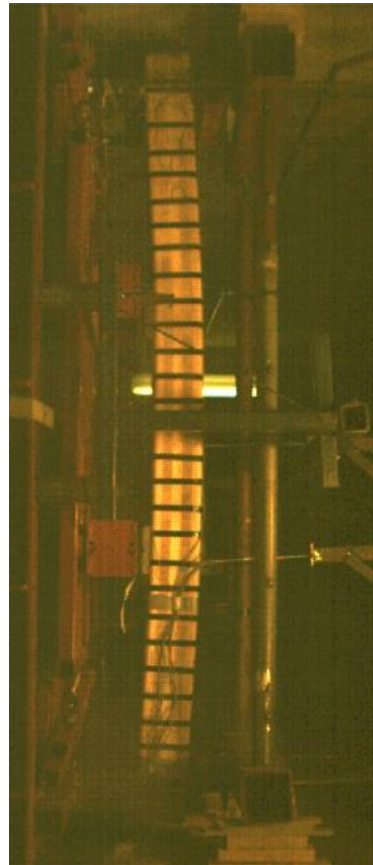


(b) Displacement and Strain

Figure D.8: Test Results for CLT5-1.1 Continued



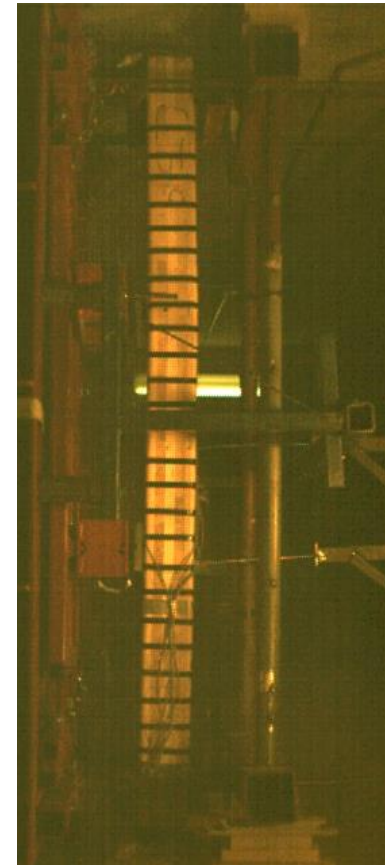
(a) Initial Position



(b) Maximum Dynamic Resistance



(c) Maximum Displacement

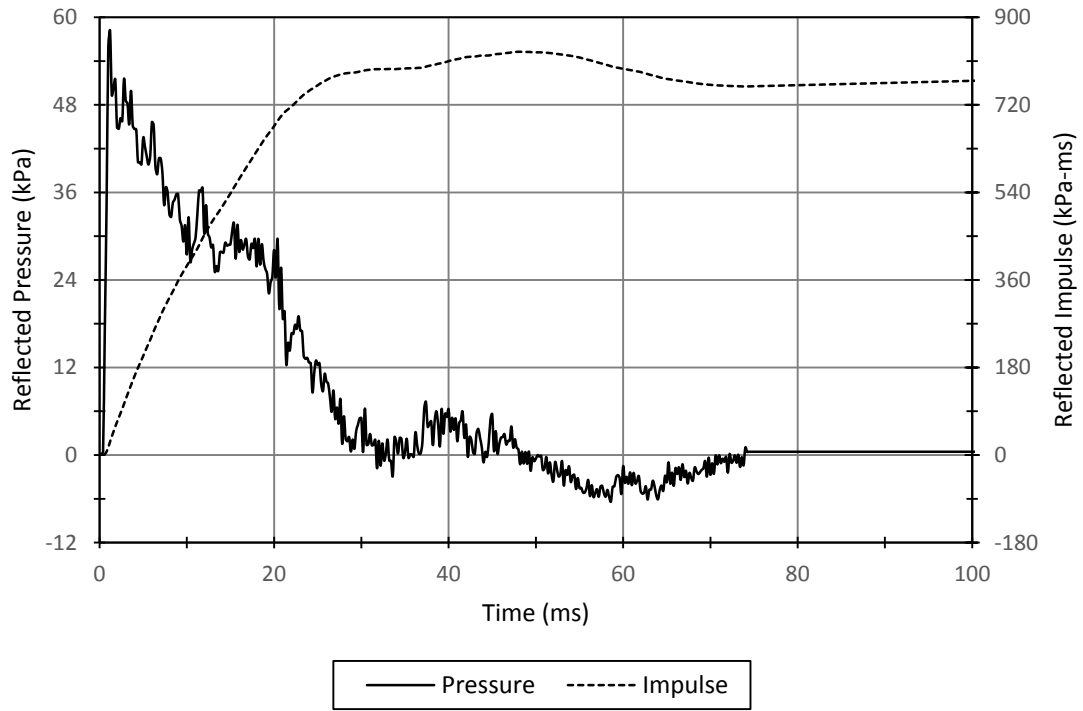


(d) Final Position

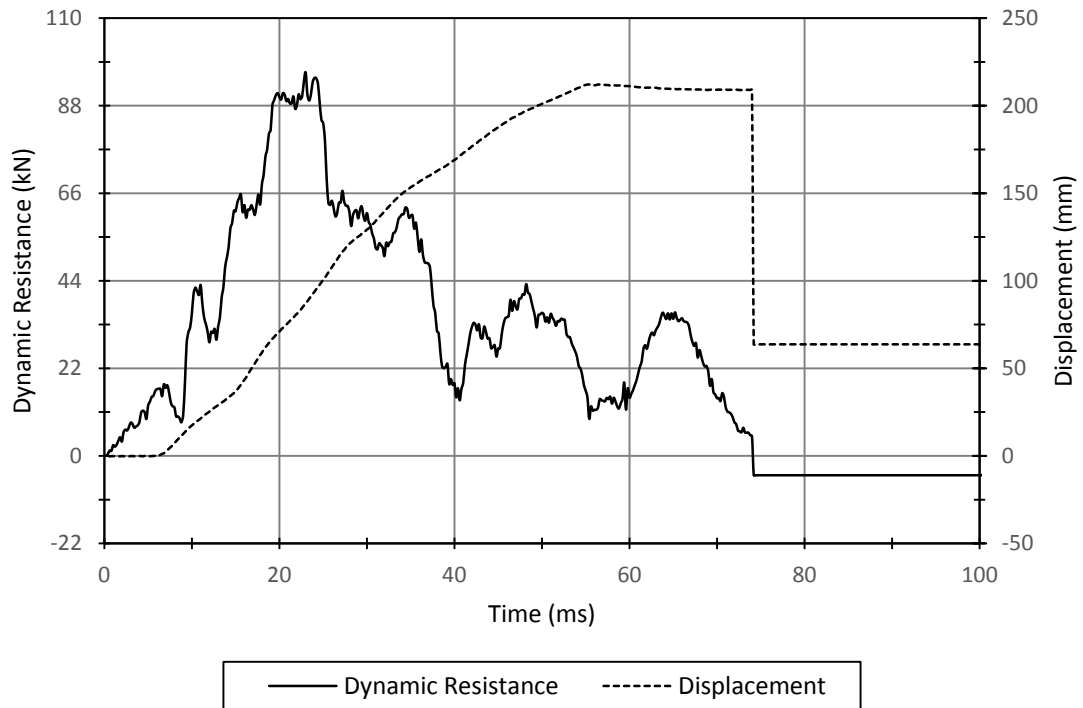
Figure D.9: Slab Movement of CLT5-1.1

CLT5-1.2

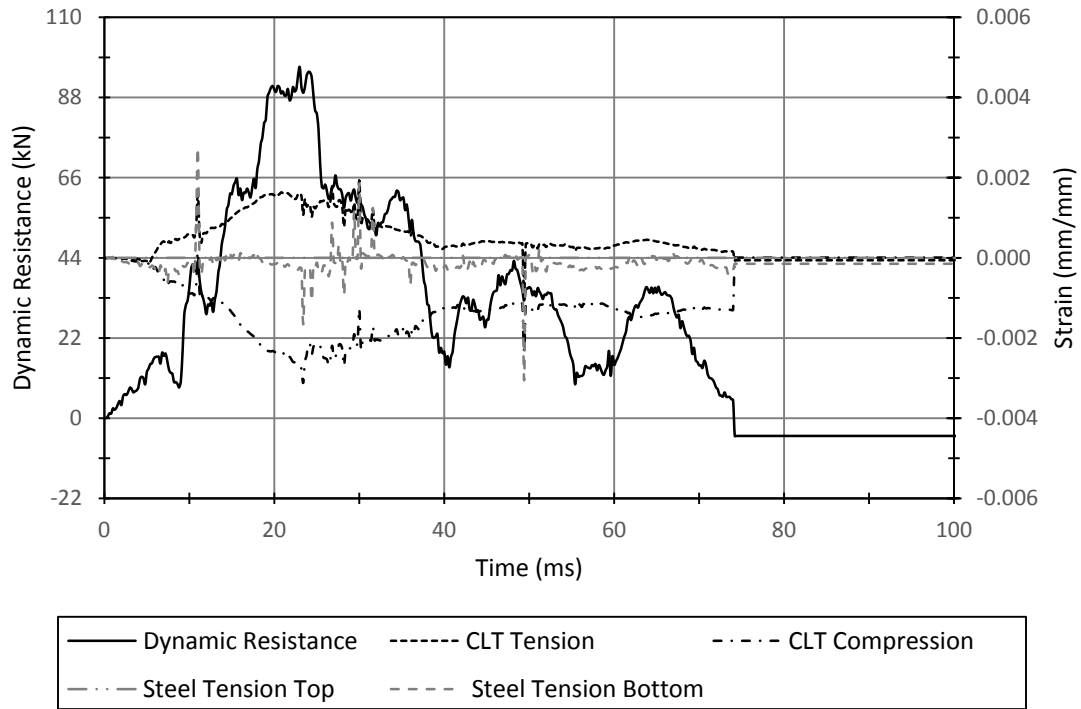
Test Date:	June 15, 2016	Slab Mass:	100.3	<i>kg</i>
Specimen No.:	3	Volume:	0.1946875	<i>m</i> ³
Test Name:	CLT5-1.2	Density:	548.47	<i>kg/m</i> ³
Shot No.:	2	Moisture Content:	12.5	%
Total Slab Height:		2,500		<i>mm</i>
Slab Clear Span:		2,235		<i>mm</i>
Configuration:		H [24] + V [5]		
Approximate Length of Straps Used:		57.98		<i>m</i>
Driver Length:		4,268		<i>mm</i>
Driver Pressure:		455.1		<i>kPa</i>
Maximum Reflected Pressure:		58.2		<i>kPa</i>
Maximum Reflected Impulse:		829.2		<i>kPa – ms</i>
Positive Phase Duration:		29.2		<i>ms</i>
Maximum Mid-Span Deflection:		212.1		<i>mm</i>
Time to Mid-Span Deflection:		56.4		<i>ms</i>
Maximum Dynamic Resistance:		96.3		<i>kN</i>
Deflection at Dynamic Resistance:		87.0		<i>mm</i>



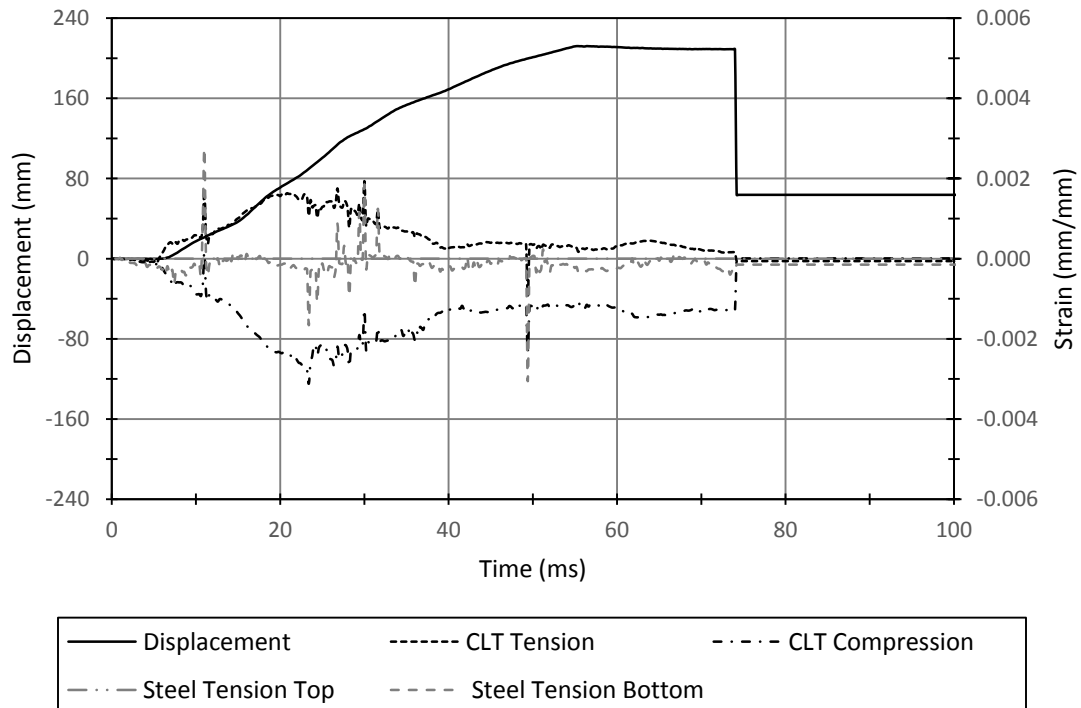
(a) Pressure and Impulse



(b) Dynamic Resistance and Displacement
Figure D.10: Test Results for CLT5-1.2

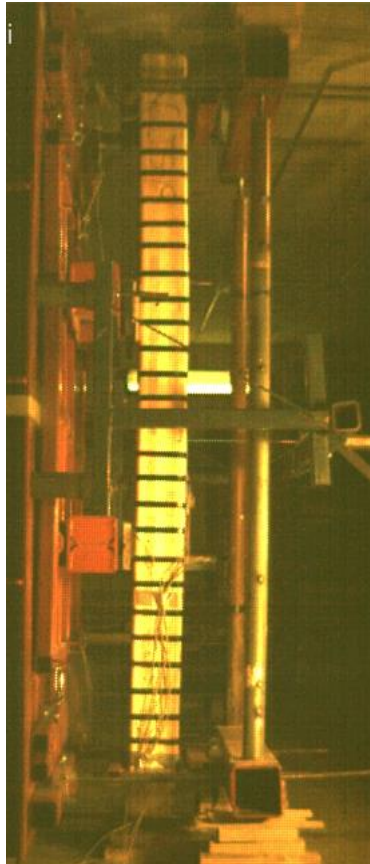


(a) Dynamic Resistance and Strain

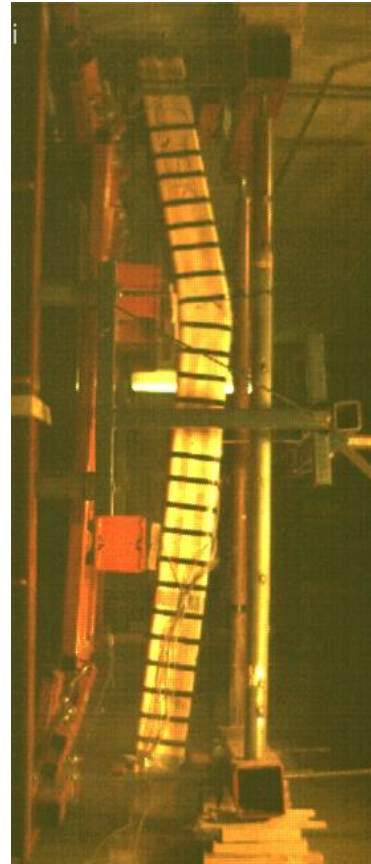


(b) Displacement and Strain

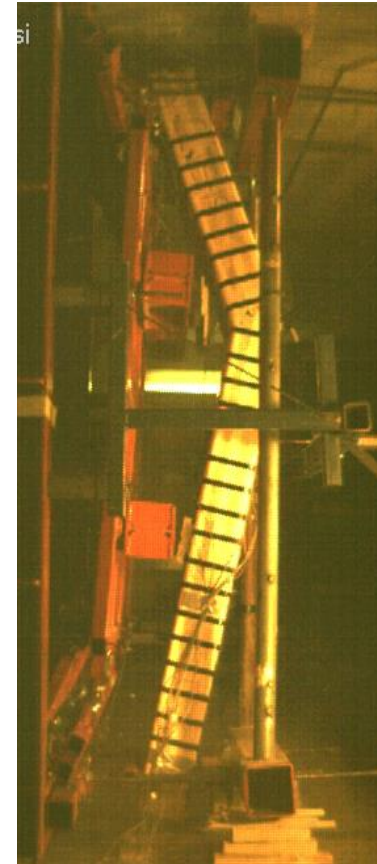
Figure D.11: Test Results for CLT5-1.2 Continued



(a) Initial Position



(b) Maximum Dynamic Resistance

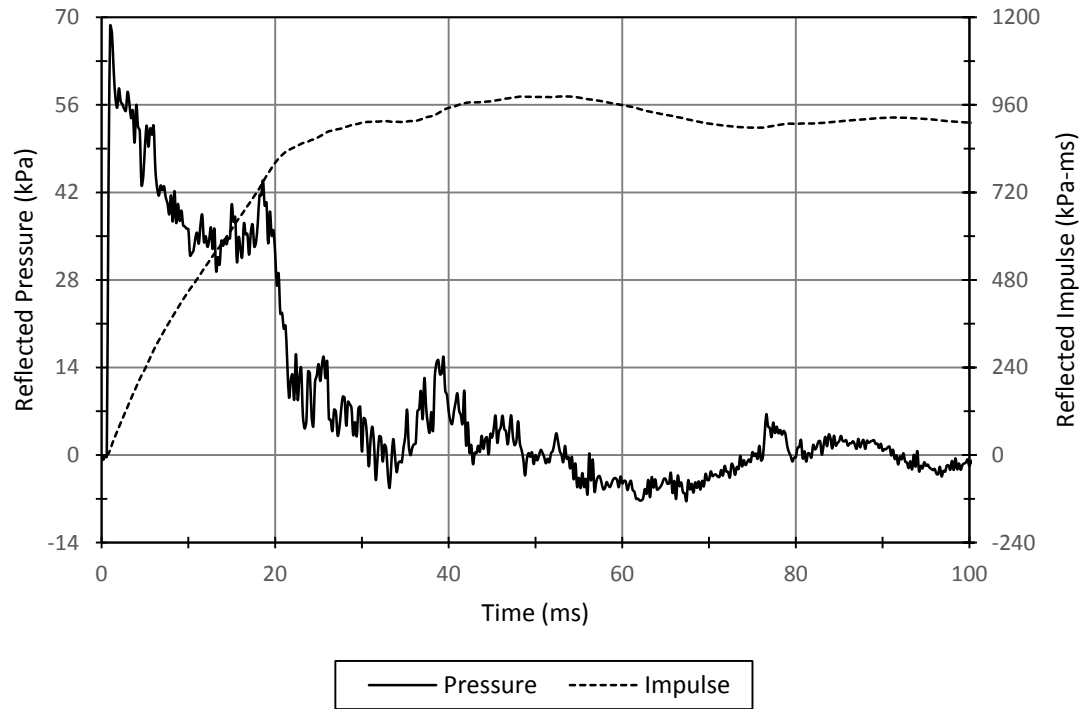


(c) Maximum Displacement

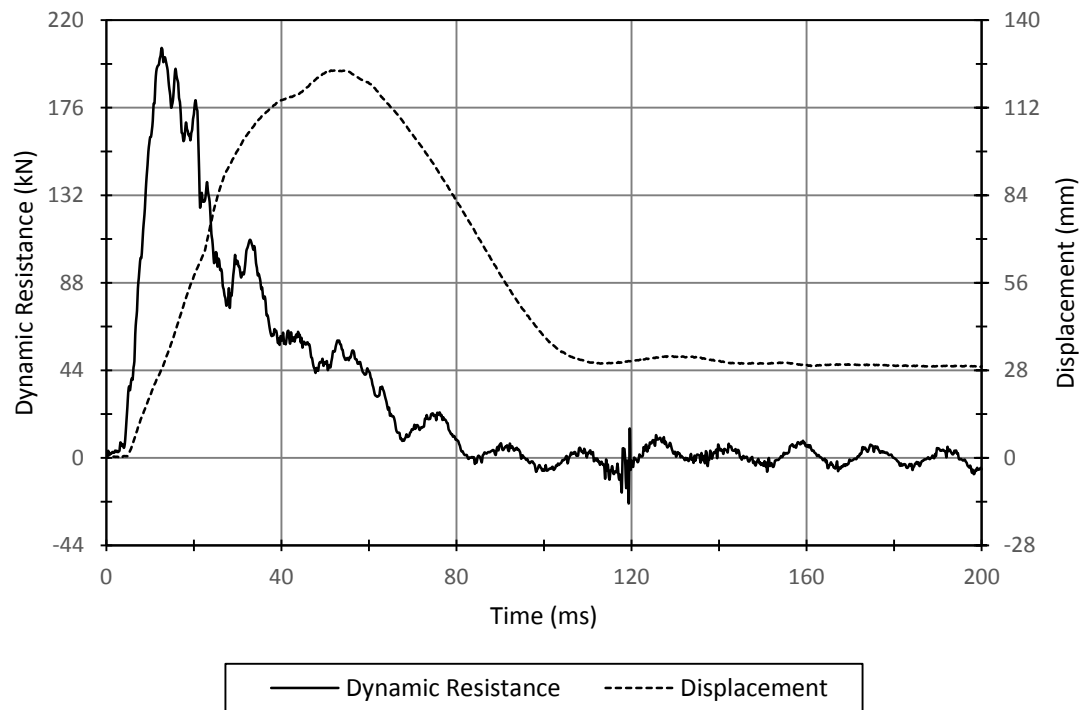
Figure D.12: Progression of Failure of CLT5-1.2

CLT5-2.1

Test Date:	June 23, 2016	Slab Mass:	105.7	<i>kg</i>
Specimen No.:	4	Volume:	0.1946875	<i>m³</i>
Test Name:	CLT5-2.1	Density:	543.1	<i>kg/m³</i>
Shot No.:	1	Moisture Content:	12.0	%
Total Slab Height:		2,500		<i>mm</i>
Slab Clear Span:		2,235		<i>mm</i>
Configuration:		V [5] + H [25]		
Approximate Length of Straps Used:		59.27		<i>m</i>
Driver Length:		4,268		<i>mm</i>
Driver Pressure:		537.1		<i>kPa</i>
Maximum Reflected Pressure:		68.6		<i>kPa</i>
Maximum Reflected Impulse:		983.1		<i>kPa – ms</i>
Positive Phase Duration:		30.0		<i>ms</i>
Maximum Mid-Span Deflection:		124.0		<i>mm</i>
Time to Mid-Span Deflection:		50.8		<i>ms</i>
Maximum Dynamic Resistance:		206.0		<i>kN</i>
Deflection at Dynamic Resistance:		28.5		<i>mm</i>

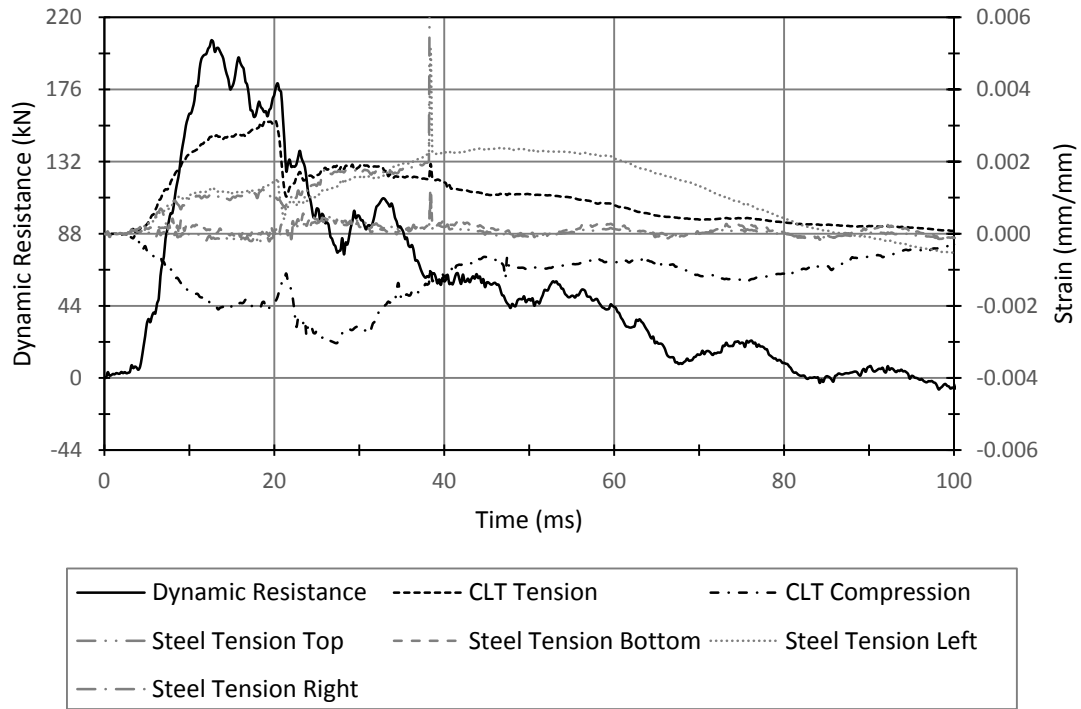


(a) Pressure and Impulse

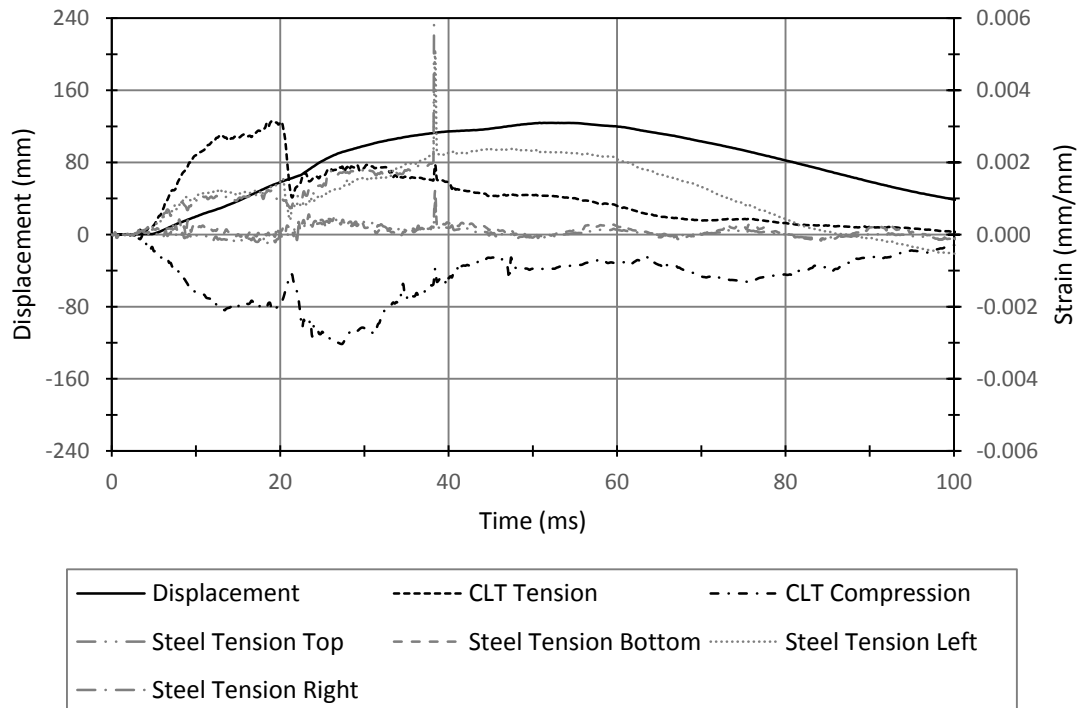


(b) Dynamic Resistance and Displacement

Figure D.13: Test Results for CLT5-2.1

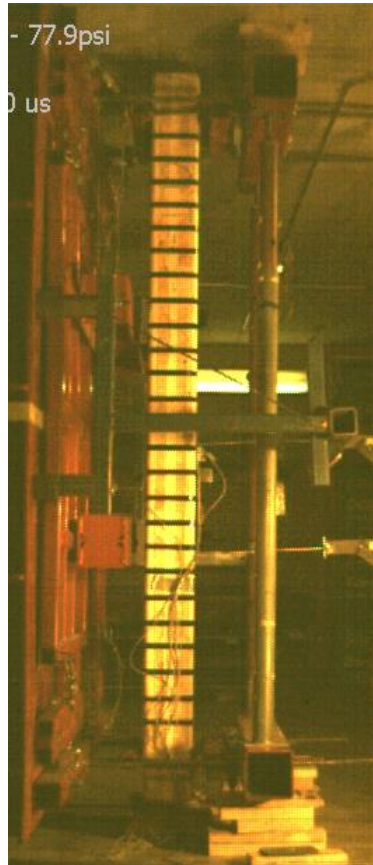


(a) Dynamic Resistance and Strain

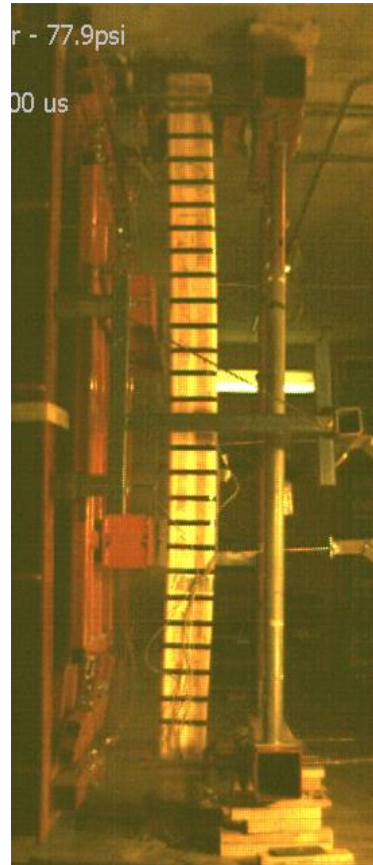


(b) Displacement and Strain

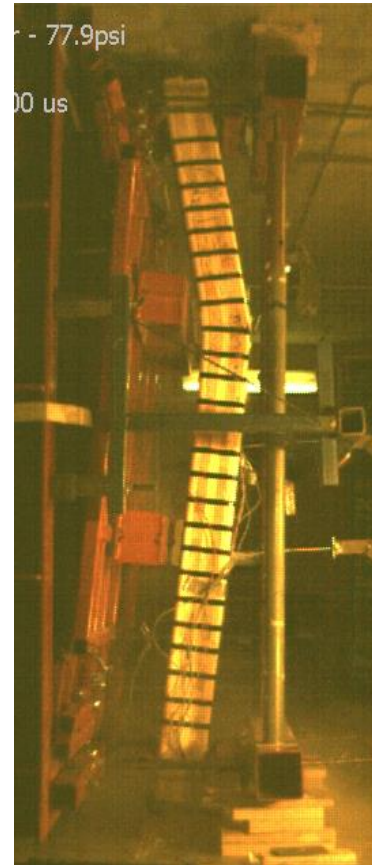
Figure D.14: Test Results for CLT5-2.1 Continued



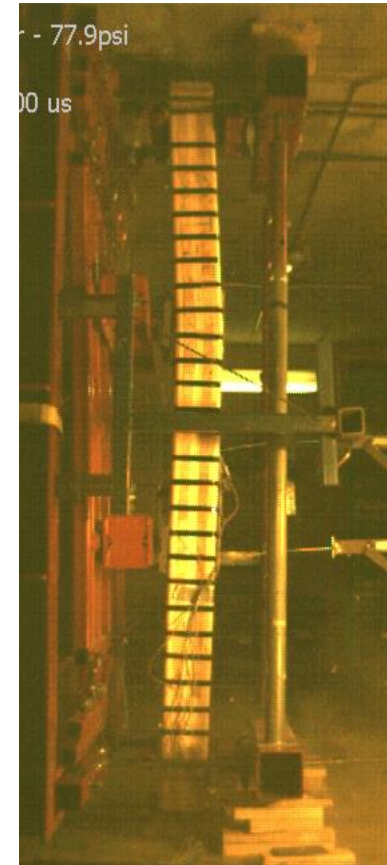
(a) Initial Position



(b) Maximum Dynamic Resistance



(c) Maximum Displacement

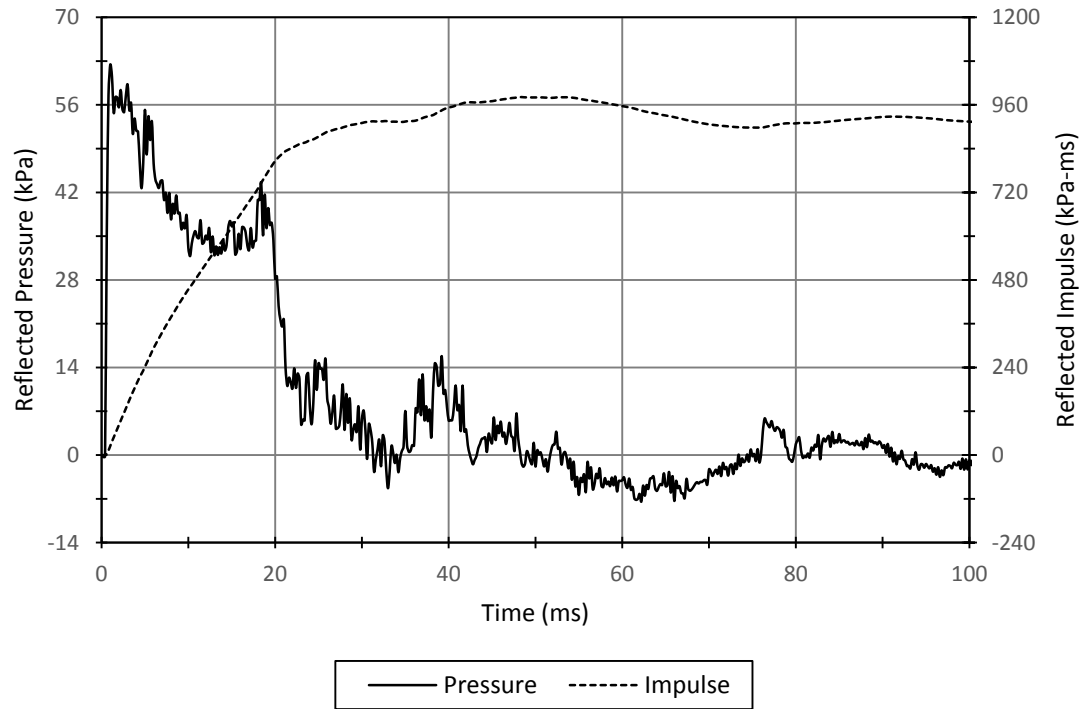


(d) Final Position

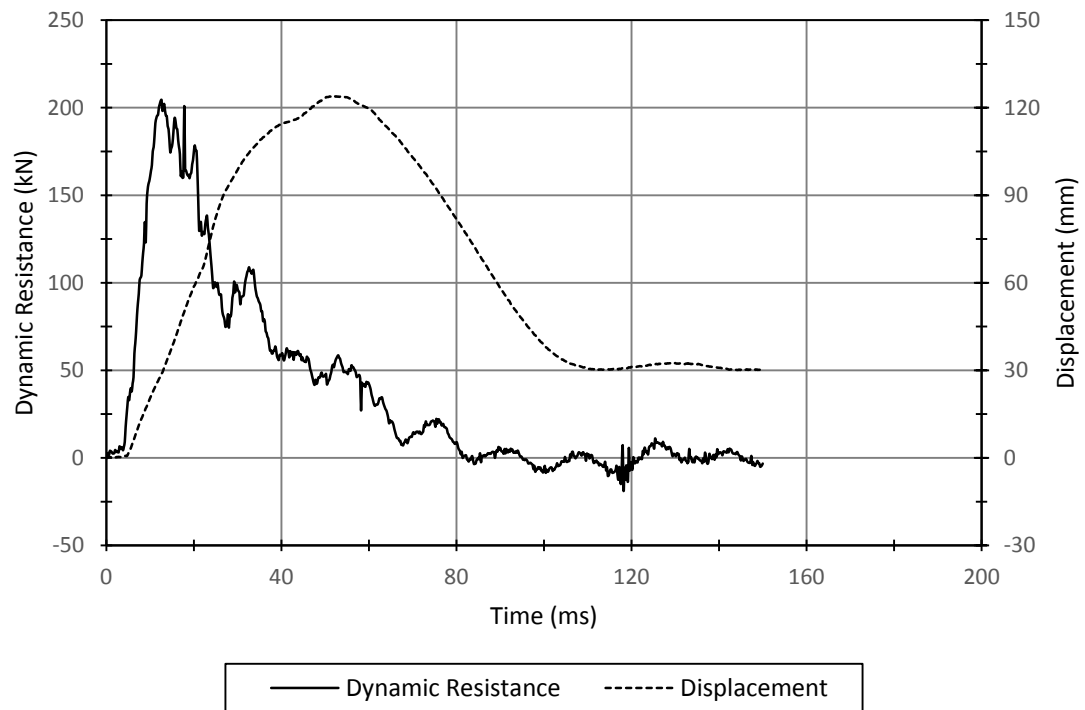
Figure D.15: Slab Movement of CLT5-2.1

CLT5-2.2

Test Date:	June 23, 2016	Slab Mass:	105.7	<i>kg</i>
Specimen No.:	4	Volume:	0.1946875	<i>m</i> ³
Test Name:	CLT5-2.2	Density:	543.1	<i>kg/m</i> ³
Shot No.:	2	Moisture Content:	12.0	%
Total Slab Height:		2,500		<i>mm</i>
Slab Clear Span:		2,235		<i>mm</i>
Configuration:		V [5] + H [25]		
Approximate Length of Straps Used:		59.27		<i>m</i>
Driver Length:		4,268		<i>mm</i>
Driver Pressure:		595.0		<i>kPa</i>
Maximum Reflected Pressure:		62.4		<i>kPa</i>
Maximum Reflected Impulse:		981.4		<i>kPa – ms</i>
Positive Phase Duration:		29.8		<i>ms</i>
Maximum Mid-Span Deflection:		123.9		<i>mm</i>
Time to Mid-Span Deflection:		52.2		<i>ms</i>
Maximum Dynamic Resistance:		204.5		<i>kN</i>
Deflection at Dynamic Resistance:		28.9		<i>mm</i>

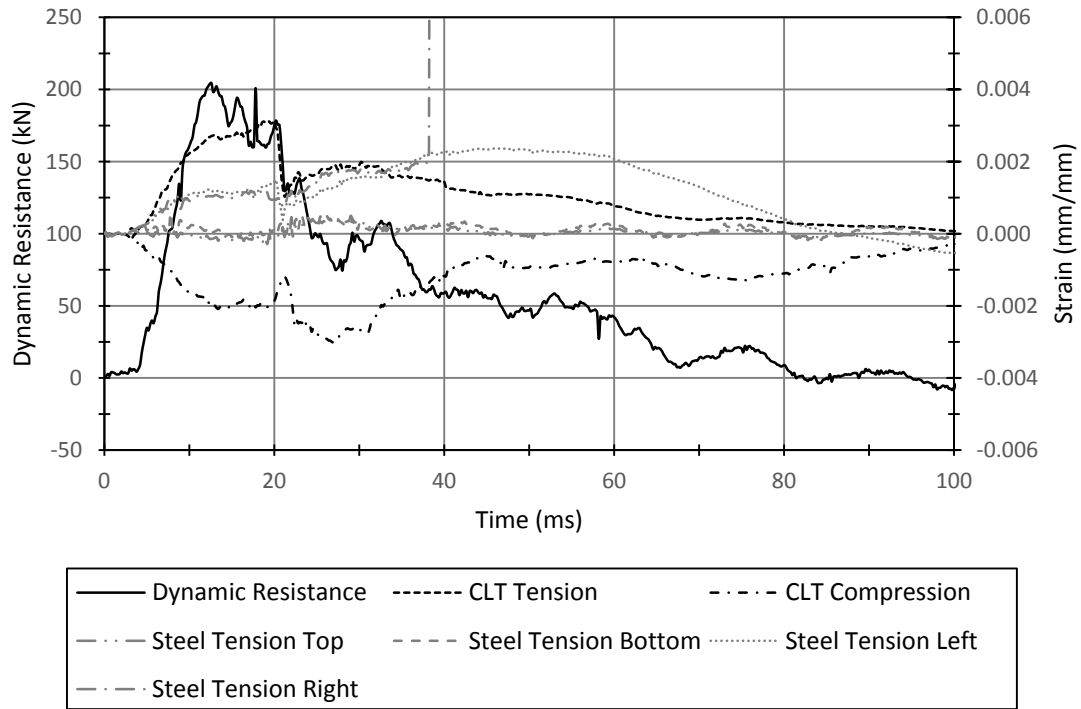


(a) Pressure and Impulse

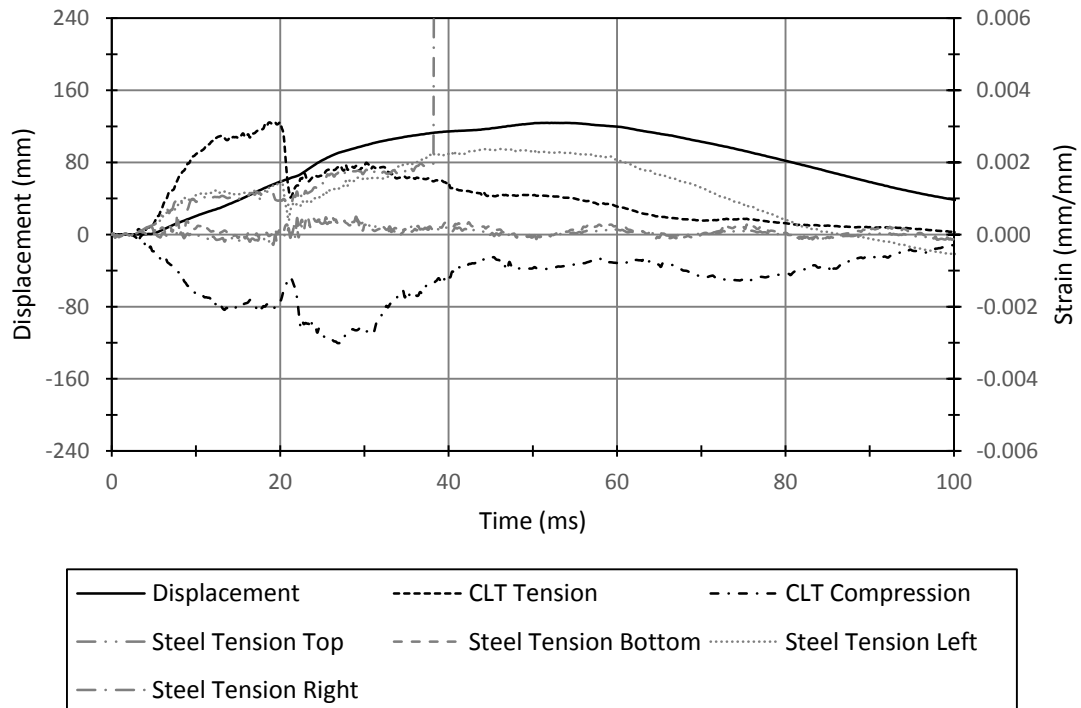


(b) Dynamic Resistance and Displacement

Figure D.16: Test Results for CLT5-2.2

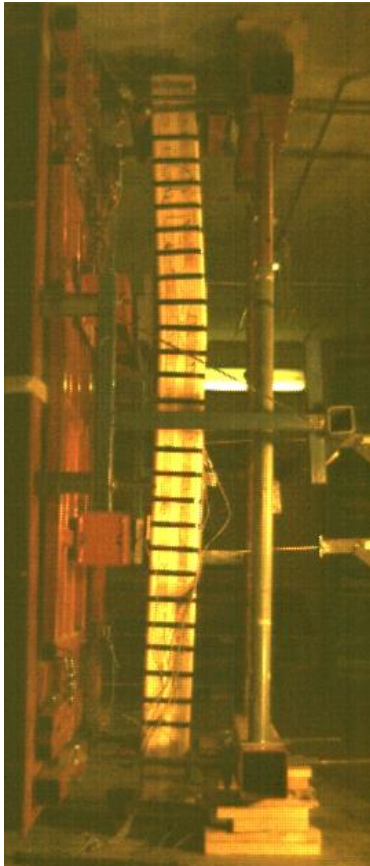


(a) Dynamic Resistance and Strain

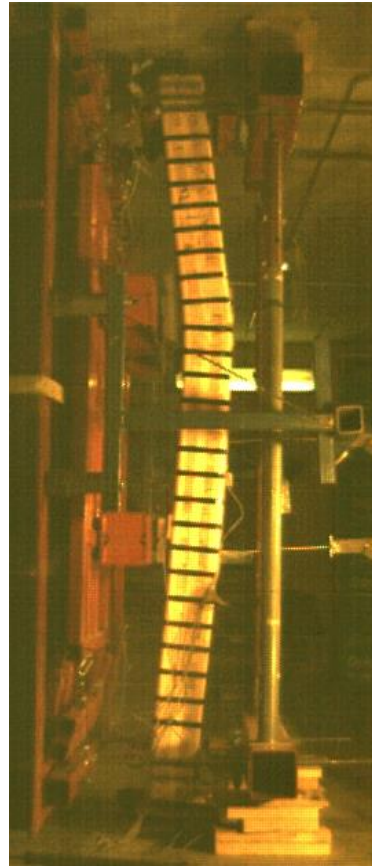


(b) Displacement and Strain

Figure D.17: Test Results for CLT5-2.2 Continued



(a) Initial Position



(b) Maximum Dynamic Resistance

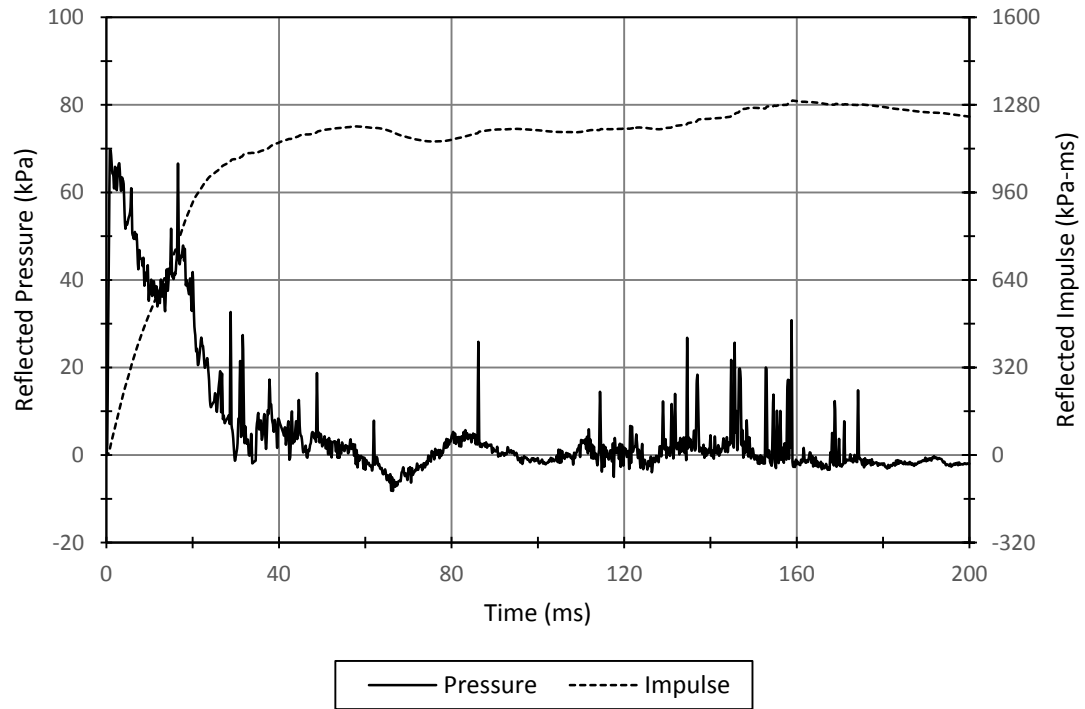


(c) Maximum Displacement

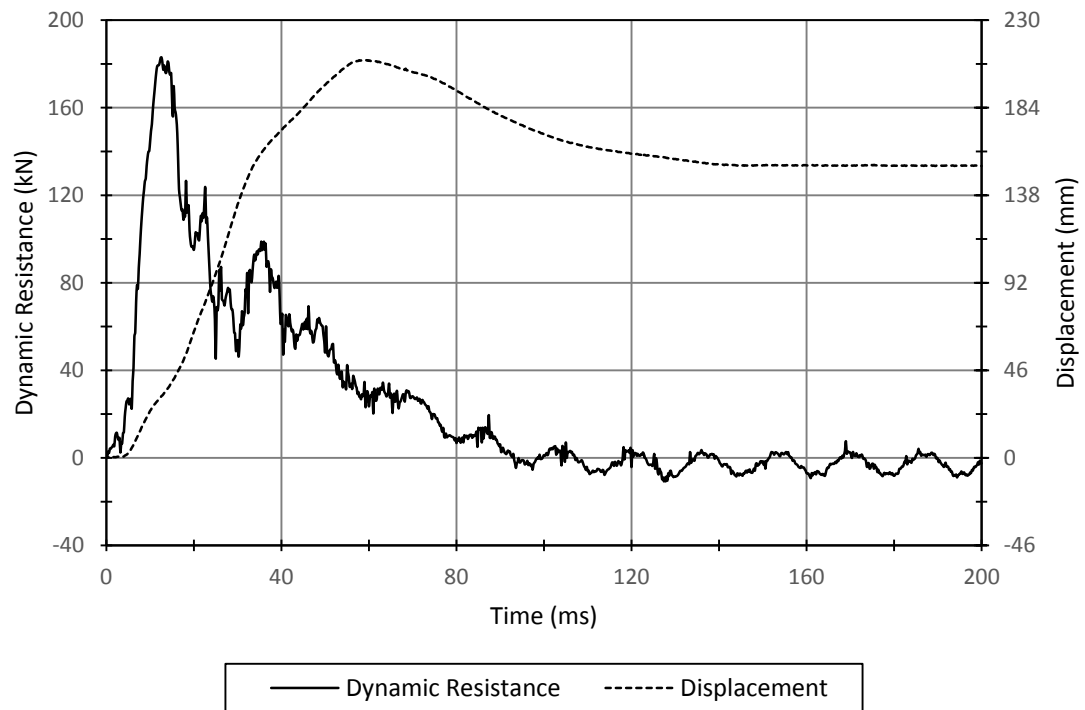
Figure D.18: Progression of Failure of CLT5-2.2

CLT5-3

Test Date:	June 27, 2016	Slab Mass:	102.9	<i>kg</i>
Specimen No.:	5	Volume:	0.1946875	<i>m³</i>
Test Name:	CLT5-3	Density:	528.5	<i>kg/m³</i>
Shot No.:	1	Moisture Content:	13.0	%
Total Slab Height:		2,500		<i>mm</i>
Slab Clear Span:		2,235		<i>mm</i>
Configuration:		V [5] + H [17]		
Approximate Length of Straps Used:		48.95		<i>m</i>
Driver Length:		4,268		<i>mm</i>
Driver Pressure:		623.3		<i>kPa</i>
Maximum Reflected Pressure:		69.6		<i>kPa</i>
Maximum Reflected Impulse:		1,295.7		<i>kPa – ms</i>
Positive Phase Duration:		29.6		<i>ms</i>
Maximum Mid-Span Deflection:		208.9		<i>mm</i>
Time to Mid-Span Deflection:		57.8		<i>ms</i>
Maximum Dynamic Resistance:		182.9		<i>kN</i>
Deflection at Dynamic Resistance:		32.5		<i>mm</i>

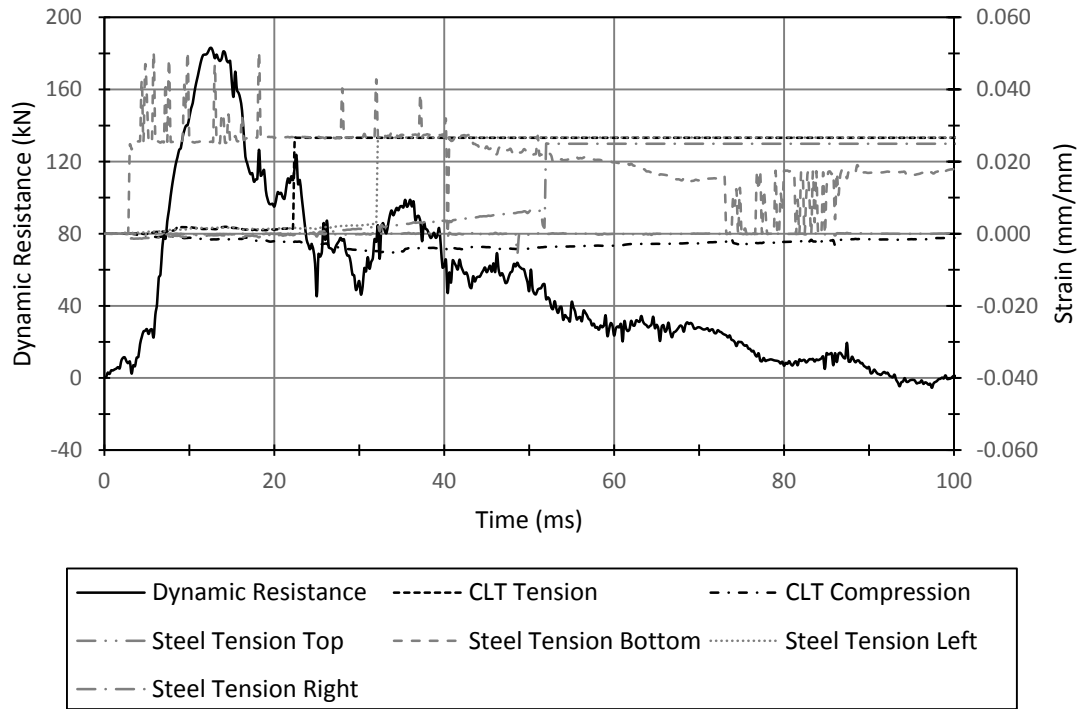


(a) Pressure and Impulse

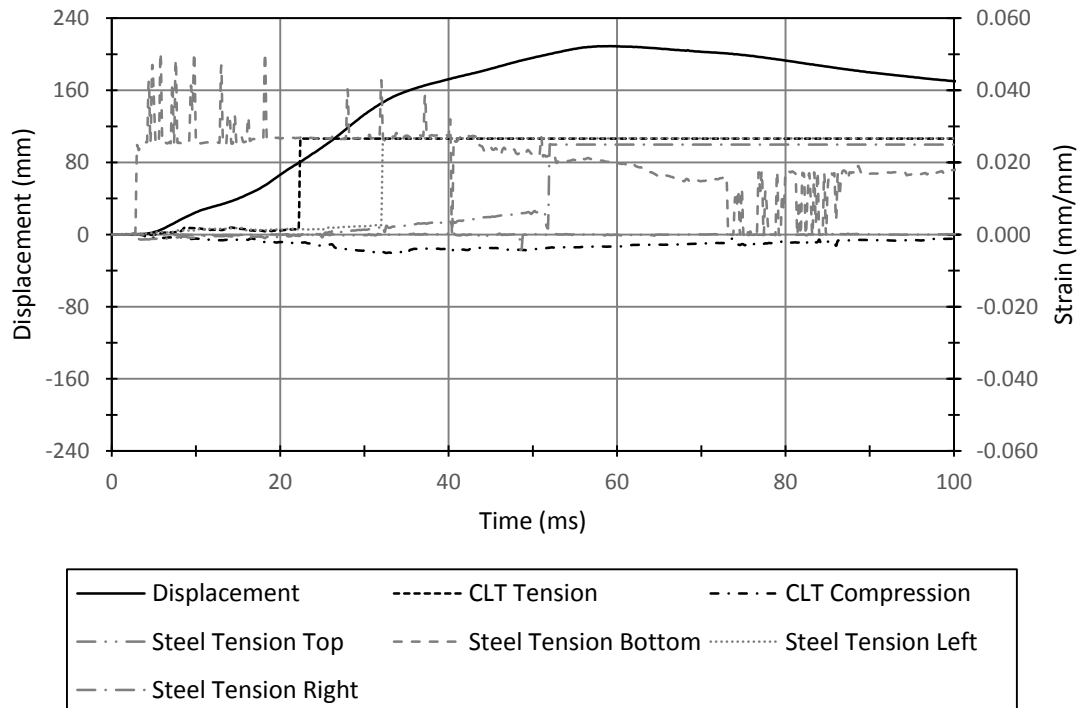


(b) Dynamic Resistance and Displacement

Figure D.19: Test Results for CLT5-3



(a) Dynamic Resistance and Strain

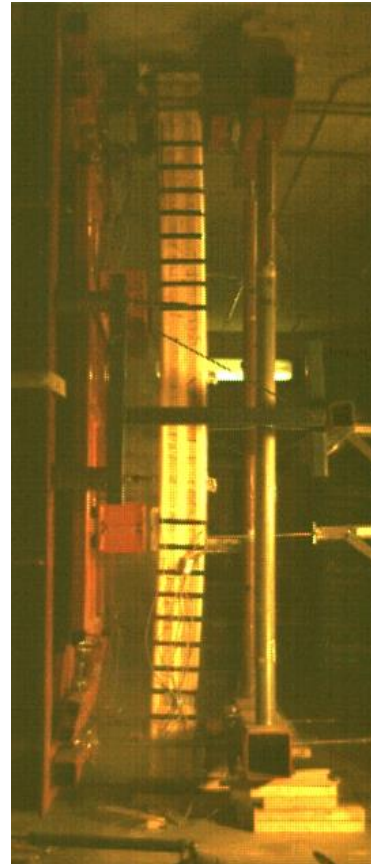


(b) Displacement and Strain

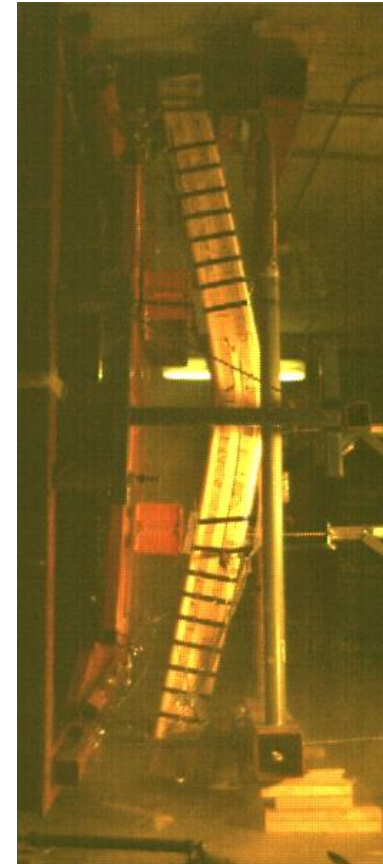
Figure D.20: Test Results for CLT5-3 Continued



(a) Initial Position



(b) Maximum Dynamic Resistance

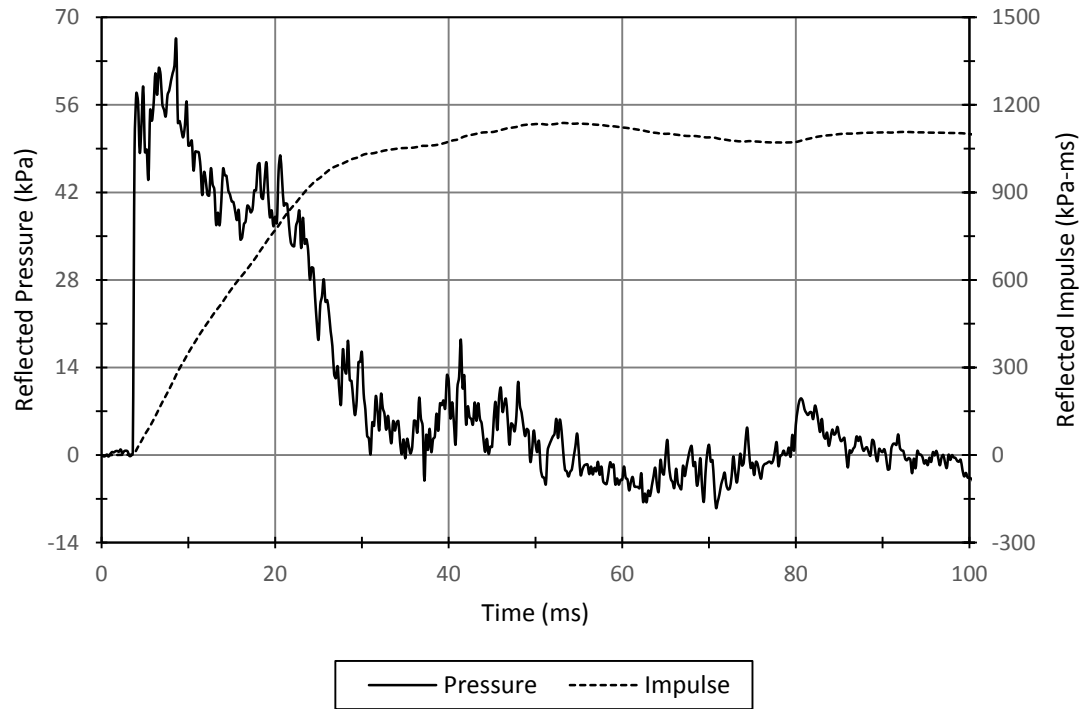


(c) Maximum Displacement

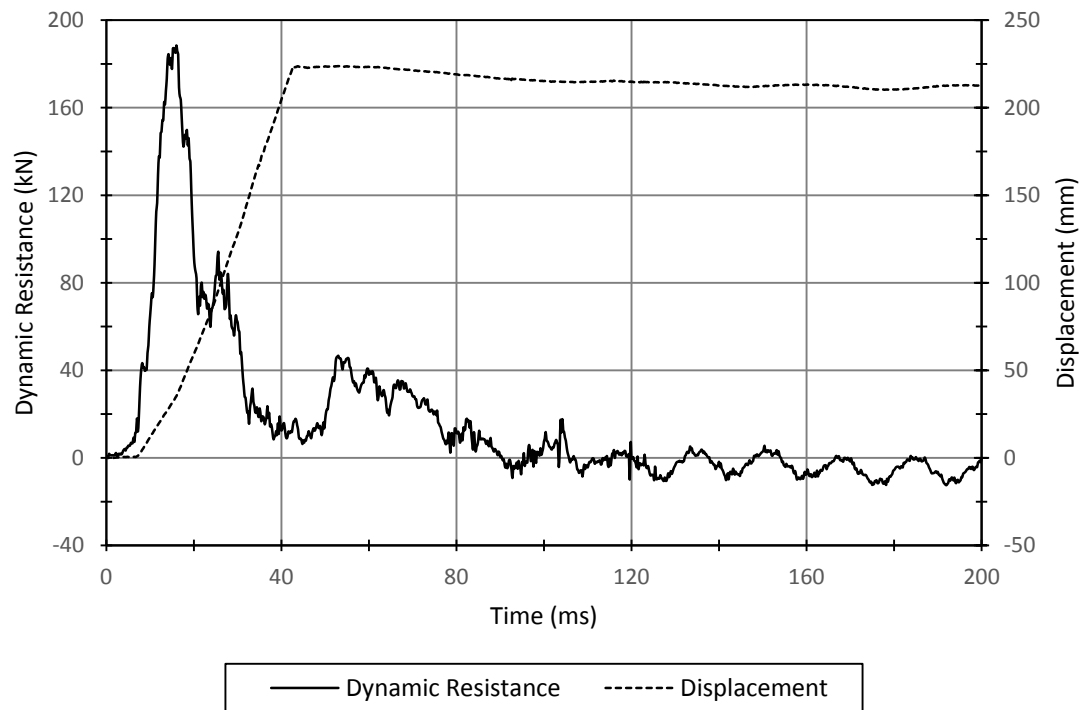
Figure D.21: Progression of Failure of CLT5-3

CLT5-4

Test Date:	June 27, 2016	Slab Mass:	101.0	<i>kg</i>
Specimen No.:	6	Volume:	0.1946875	<i>m³</i>
Test Name:	CLT5-4	Density:	518.8	<i>kg/m³</i>
Shot No.:	1	Moisture Content:	14.0	%
Total Slab Height:		2,500	<i>mm</i>	
Slab Clear Span:		2,235	<i>mm</i>	
Configuration:		V [5] + H [13]		
Approximate Length of Straps Used:		43.78	<i>m</i>	
Driver Length:		4,268	<i>mm</i>	
Driver Pressure:		650.9	<i>kPa</i>	
Maximum Reflected Pressure:		66.4	<i>kPa</i>	
Maximum Reflected Impulse:		1,138.1	<i>kPa – ms</i>	
Positive Phase Duration:		31.0	<i>ms</i>	
Maximum Mid-Span Deflection:		223.8	<i>mm</i>	
Time to Mid-Span Deflection:		42.8	<i>ms</i>	
Maximum Dynamic Resistance:		188.4	<i>kN</i>	
Deflection at Dynamic Resistance:		35.2	<i>mm</i>	

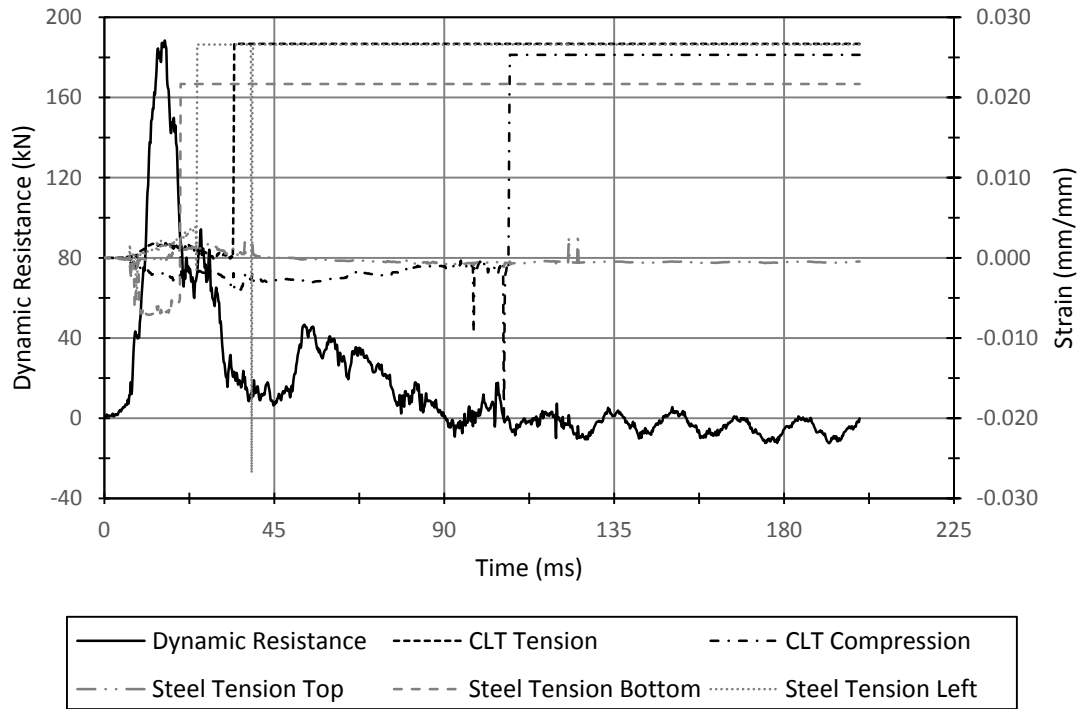


(a) Pressure and Impulse

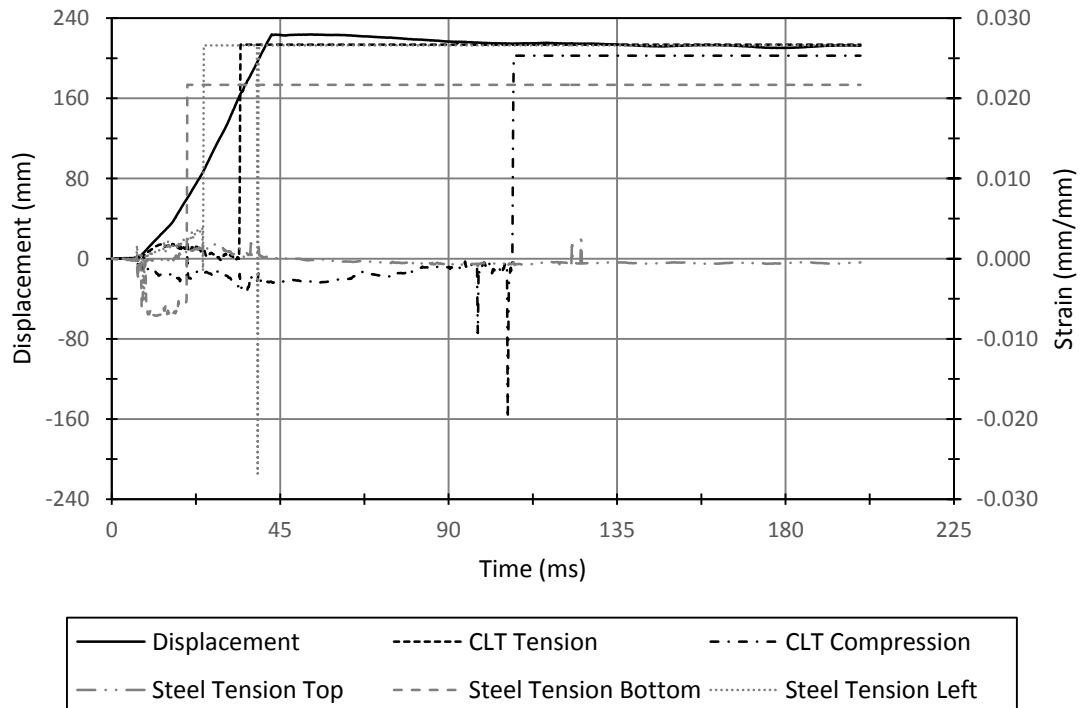


(b) Dynamic Resistance and Displacement

Figure D.22: Test Results for CLT5-4



(a) Dynamic Resistance and Strain



(b) Displacement and Strain

Figure D.23: Test Results for CLT5-4 Continued



(a) Initial Position



(b) Final Position

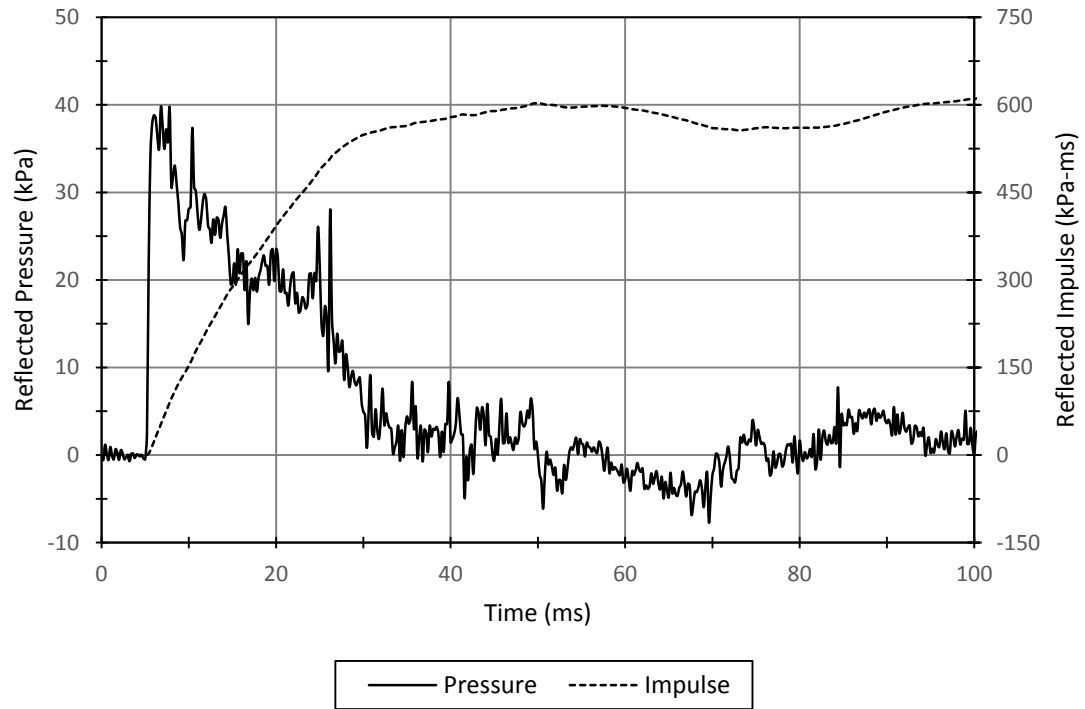


(c) Displaced Shape

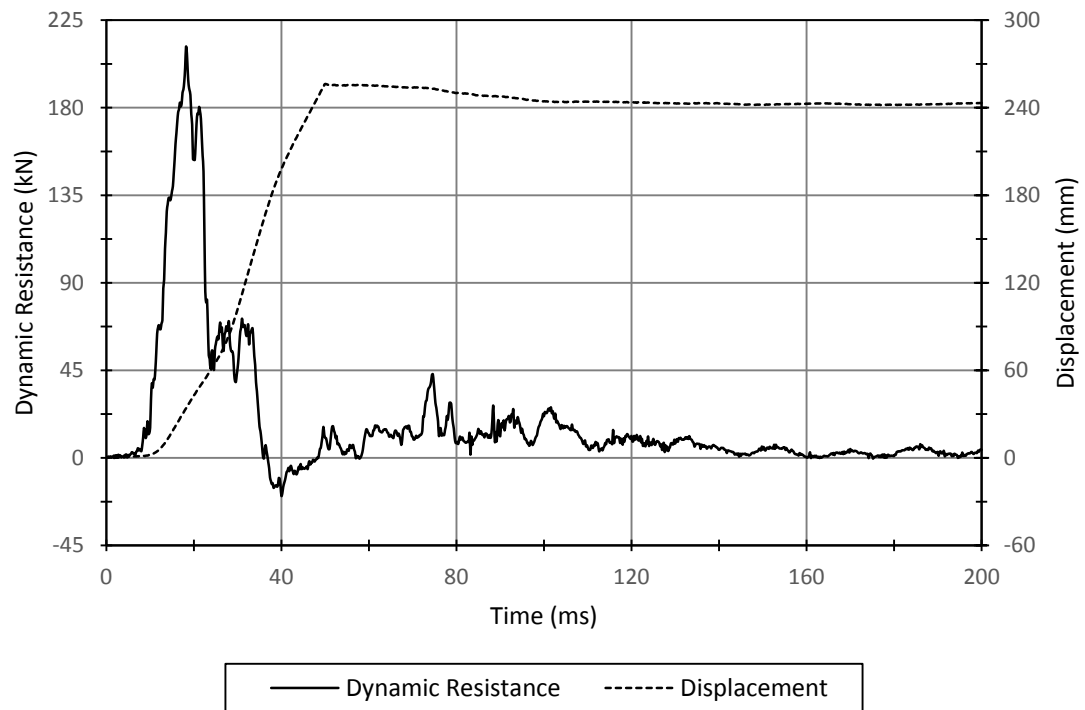
Figure D.24: Slab Failure of CLT5-4

CLT5-5

Test Date:	June 30, 2016	Slab Mass:	109.22	<i>kg</i>
Specimen No.:	7	Volume:	0.1946875	<i>m</i> ³
Test Name:	CLT5-5	Density:	561.0	<i>kg/m</i> ³
Shot No.:	1	Moisture Content:	12.0	%
Total Slab Height:		2,500		<i>mm</i>
Slab Clear Span:		2,235		<i>mm</i>
Configuration:		V [9] + H [8]		
Approximate Length of Straps Used:		13.03		<i>m</i>
Driver Length:		4,268		<i>mm</i>
Driver Pressure:		632.9		<i>kPa</i>
Maximum Reflected Pressure:		65.7		<i>kPa</i>
Maximum Reflected Impulse:		871.1		<i>kPa – ms</i>
Positive Phase Duration:		33.4		<i>ms</i>
Maximum Mid-Span Deflection:		256.2		<i>mm</i>
Time to Mid-Span Deflection:		50.0		<i>ms</i>
Maximum Dynamic Resistance:		211.3		<i>kN</i>
Deflection at Dynamic Resistance:		33.9		<i>mm</i>

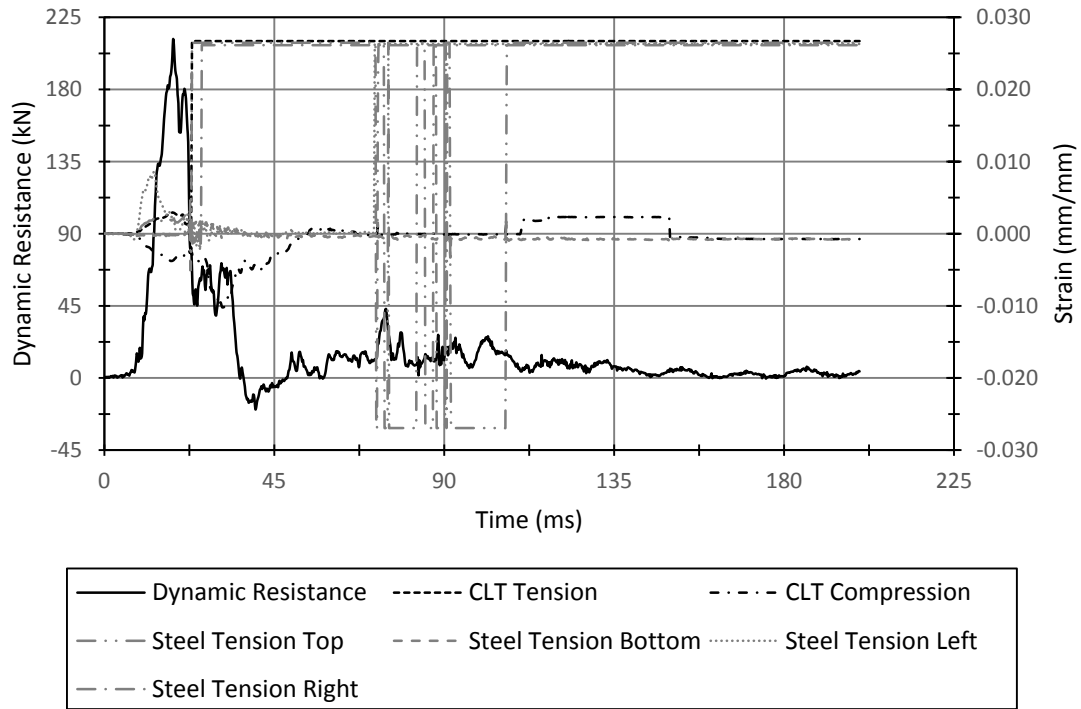


(a) Pressure and Impulse

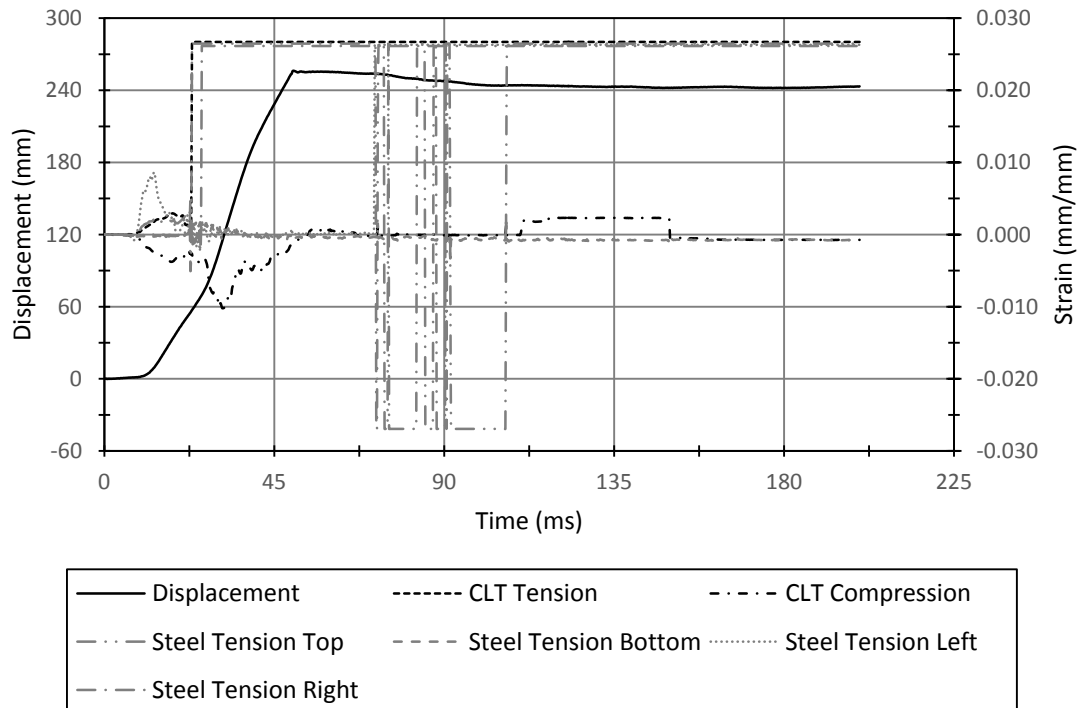


(b) Dynamic Resistance and Displacement

Figure D.25: Test Results for CLT5-5



(a) Dynamic Resistance and Strain



(b) Displacement and Strain

Figure D.26: Test Results for CLT5-5 Continued



(a) Initial Position



(b) Maximum Dynamic Resistance

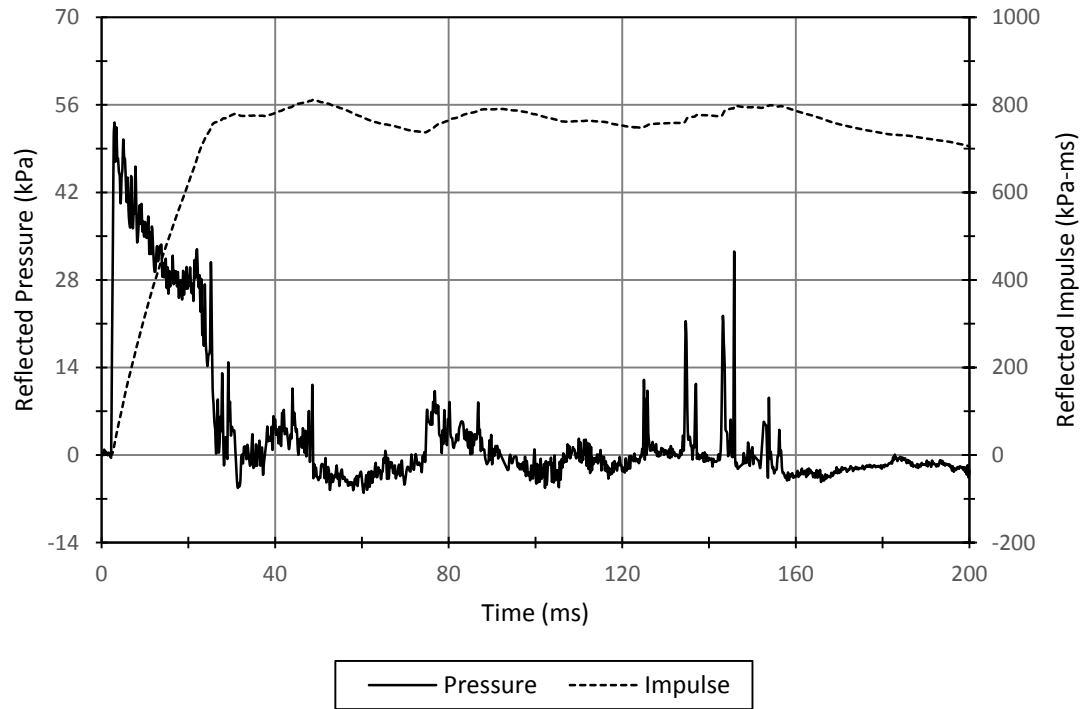


(c) Maximum Displacement

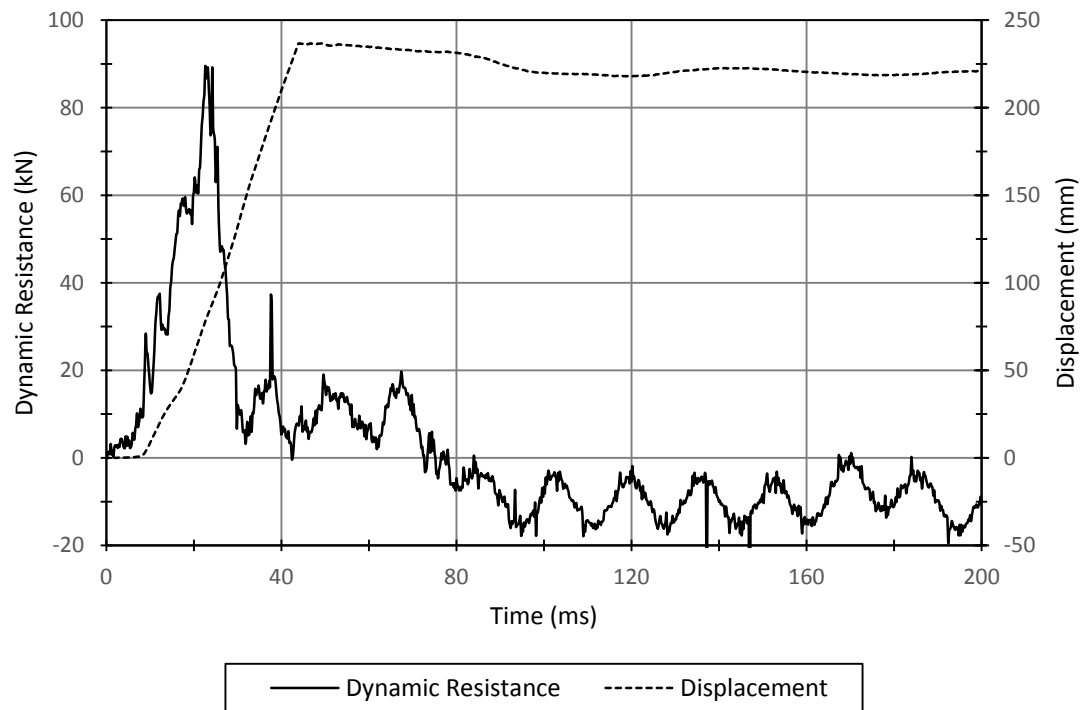
Figure D.27: Progression of Failure of CLT5-5

CLT5-D1

Test Date:	June 21, 2016	Slab Mass:	101.4	<i>kg</i>
Specimen No.:	8	Volume:	0.1946875	<i>m³</i>
Test Name:	CLT5-D1	Density:	520.8	<i>kg/m³</i>
Shot No.:	1	Moisture Content:	14.5	%
Total Slab Height:		2,500	<i>mm</i>	
Slab Clear Span:		2,235	<i>mm</i>	
Configuration:		H [24]		
Approximate Length of Straps Used:		30.98	<i>m</i>	
Driver Length:		4,268	<i>mm</i>	
Driver Pressure:		439.9	<i>kPa</i>	
Maximum Reflected Pressure:		53.2	<i>kPa</i>	
Maximum Reflected Impulse:		811.8	<i>kPa – ms</i>	
Positive Phase Duration:		26.4	<i>ms</i>	
Maximum Mid-Span Deflection:		236.8	<i>mm</i>	
Time to Mid-Span Deflection:		46.6	<i>ms</i>	
Maximum Dynamic Resistance:		89.55	<i>kN</i>	
Deflection at Dynamic Resistance:		77.9	<i>mm</i>	

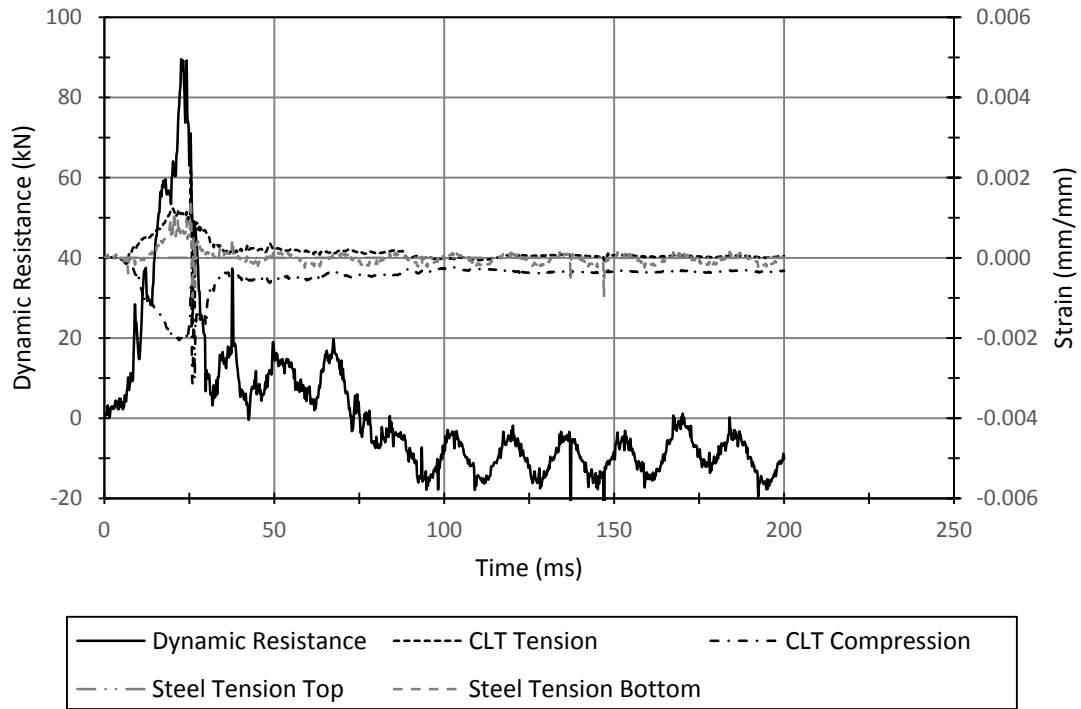


(a) Pressure and Impulse

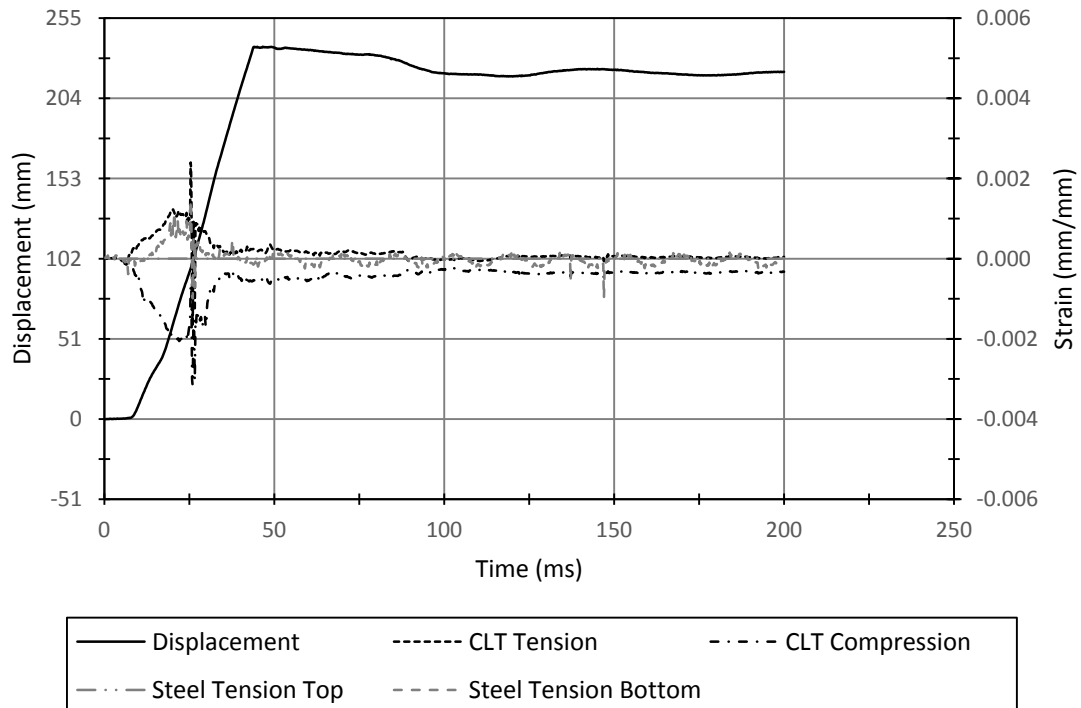


(b) Dynamic Resistance and Displacement

Figure D.28: Test Results for CLT5-D1

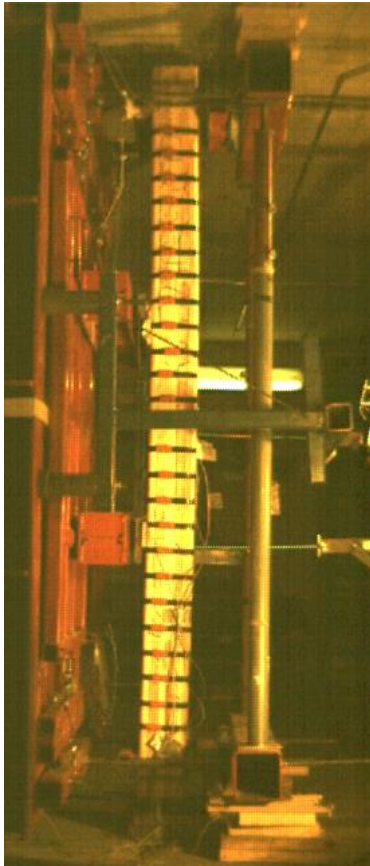


(a) Dynamic Resistance and Strain

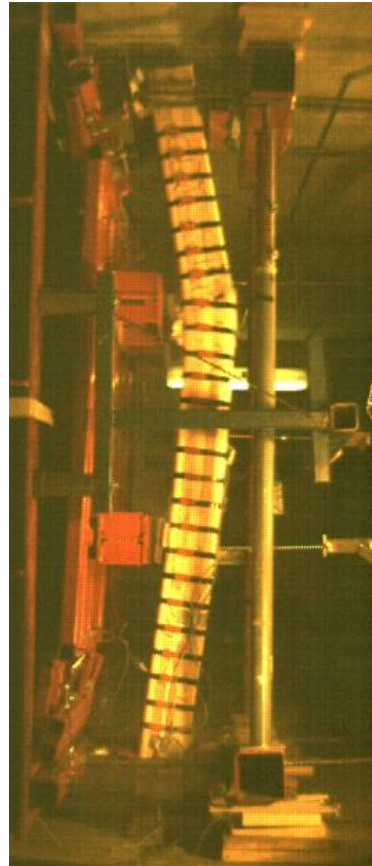


(b) Displacement and Strain

Figure D.29: Test Results for CLT5-D1 Continued



(a) Initial Position



(b) Maximum Dynamic Resistance



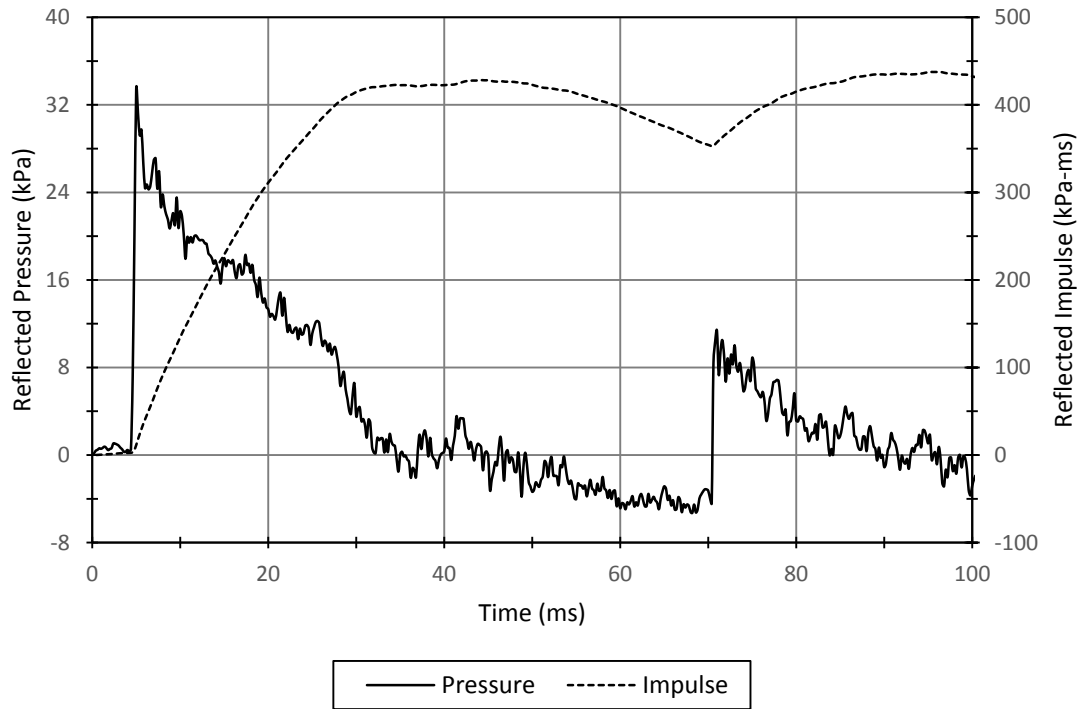
(c) Maximum Displacement

Figure D.30: Progression of Failure of CLT5-D1

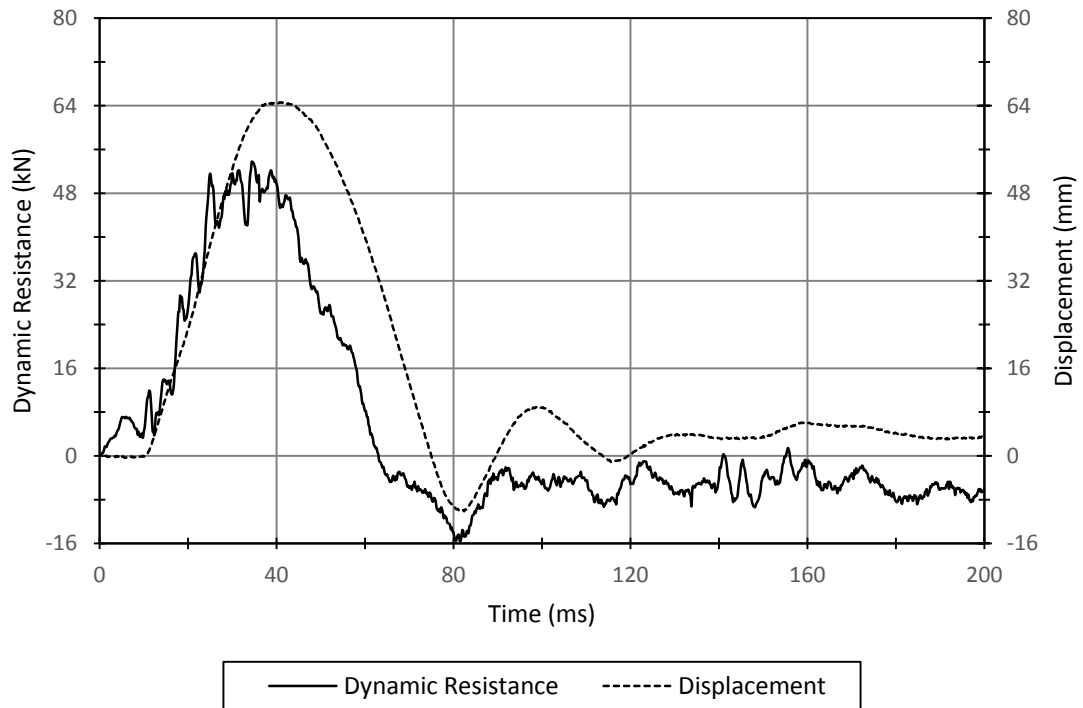
CLT5-D2.1

Test Date:	March 14, 2017	Slab Mass:	101.6	<i>kg</i>
Specimen No.:	9	Volume:	0.1946875	<i>m³</i>
Test Name:	CLT5-D2.1	Density:	521.9	<i>kg/m³</i>
Shot No.:	1	Moisture Content:	10.0	%

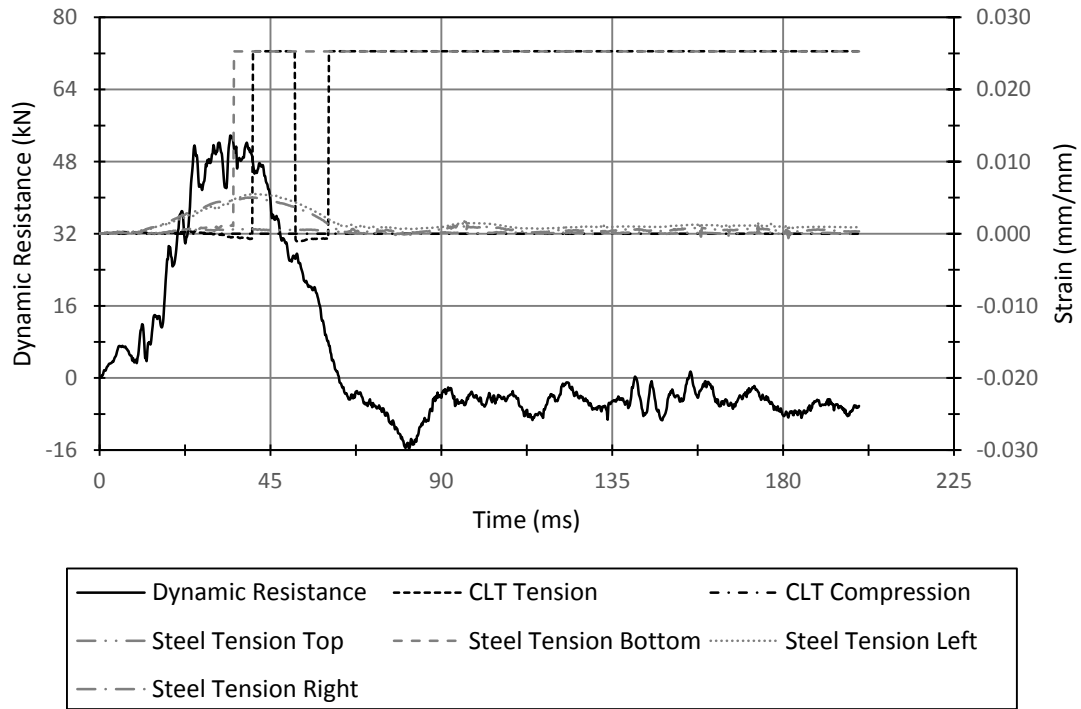
Total Slab Height:	2,500	<i>mm</i>
Slab Clear Span:	2,235	<i>mm</i>
Configuration:	V [5] + H [24]	
Approximate Length of Straps Used:	57.98	<i>m</i>
Driver Length:	4,572	<i>mm</i>
Driver Pressure:	217.9	<i>kPa</i>
Maximum Reflected Pressure:	33.5	<i>kPa</i>
Maximum Reflected Impulse:	437.6	<i>kPa – ms</i>
Positive Phase Duration:	32.0	<i>ms</i>
Maximum Mid-Span Deflection:	64.6	<i>mm</i>
Time to Mid-Span Deflection:	41.0	<i>ms</i>
Maximum Dynamic Resistance:	53.8	<i>kN</i>
Deflection at Dynamic Resistance:	61.3	<i>mm</i>



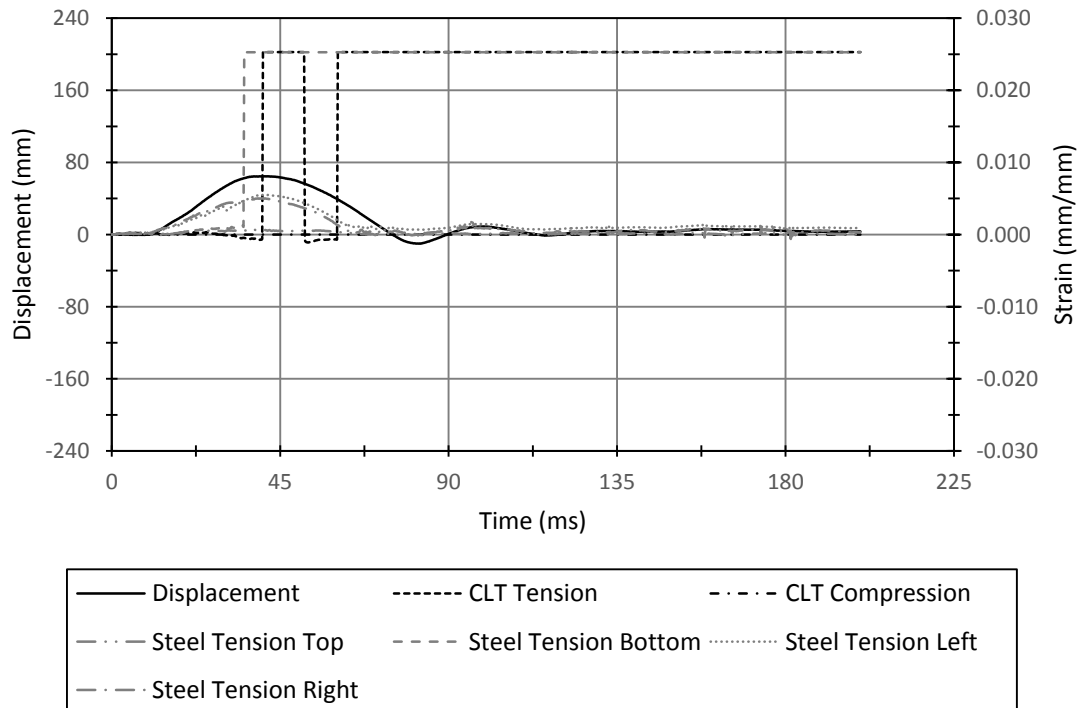
(a) Pressure and Impulse



(b) Dynamic Resistance and Displacement
Figure D.31: Test Results for CLT5-D2.1



(a) Dynamic Resistance and Strain

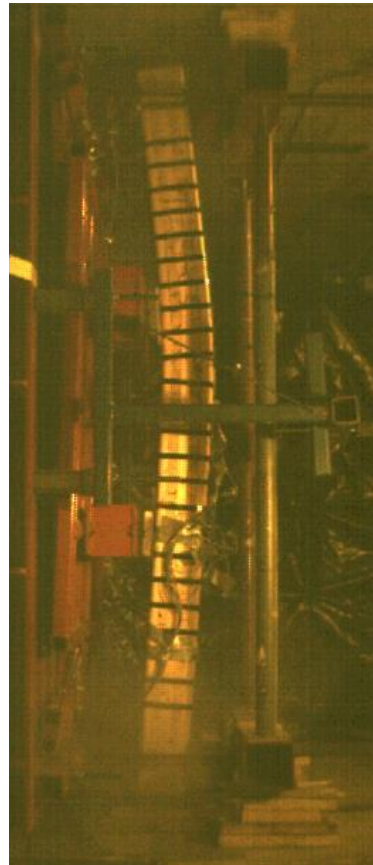


(b) Displacement and Strain

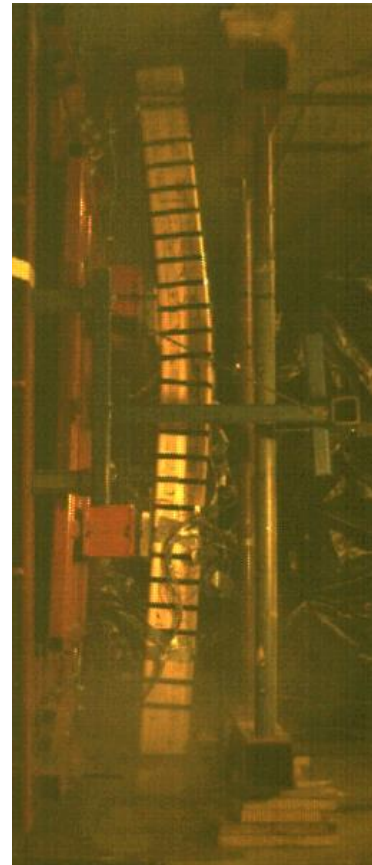
Figure D.32: Test Results for CLT5-D2.1 Continued



(a) Initial Position



(b) Maximum Dynamic Resistance



(c) Maximum Displacement



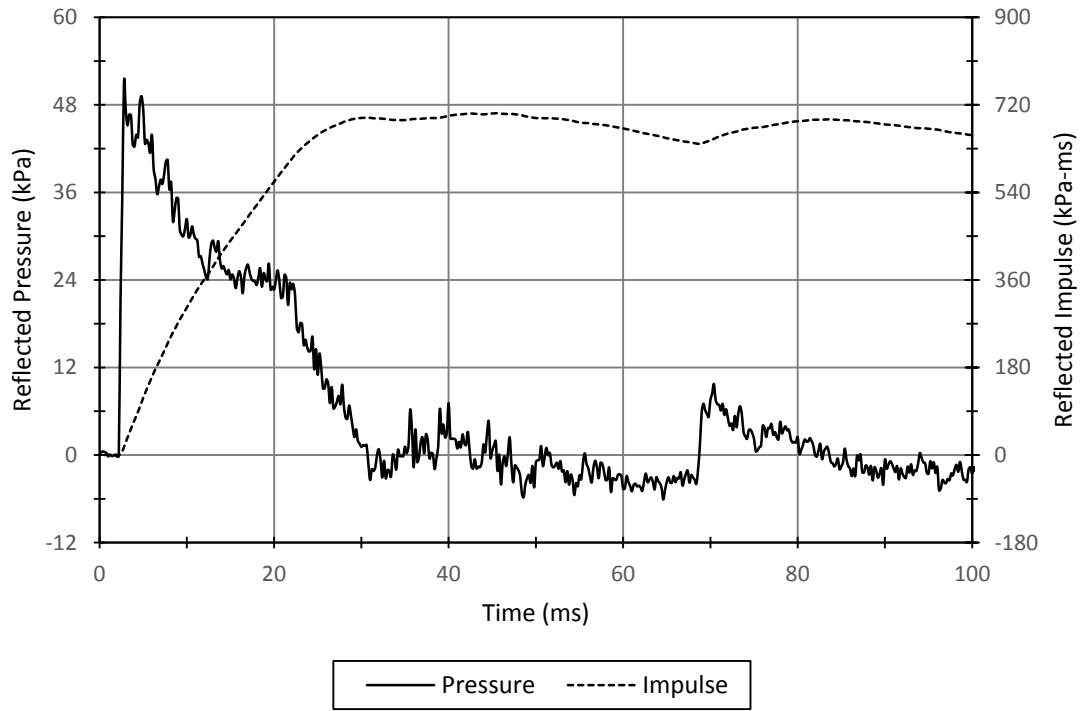
(d) Final Position

Figure D.33: Slab Movement of CLT5-D2.1

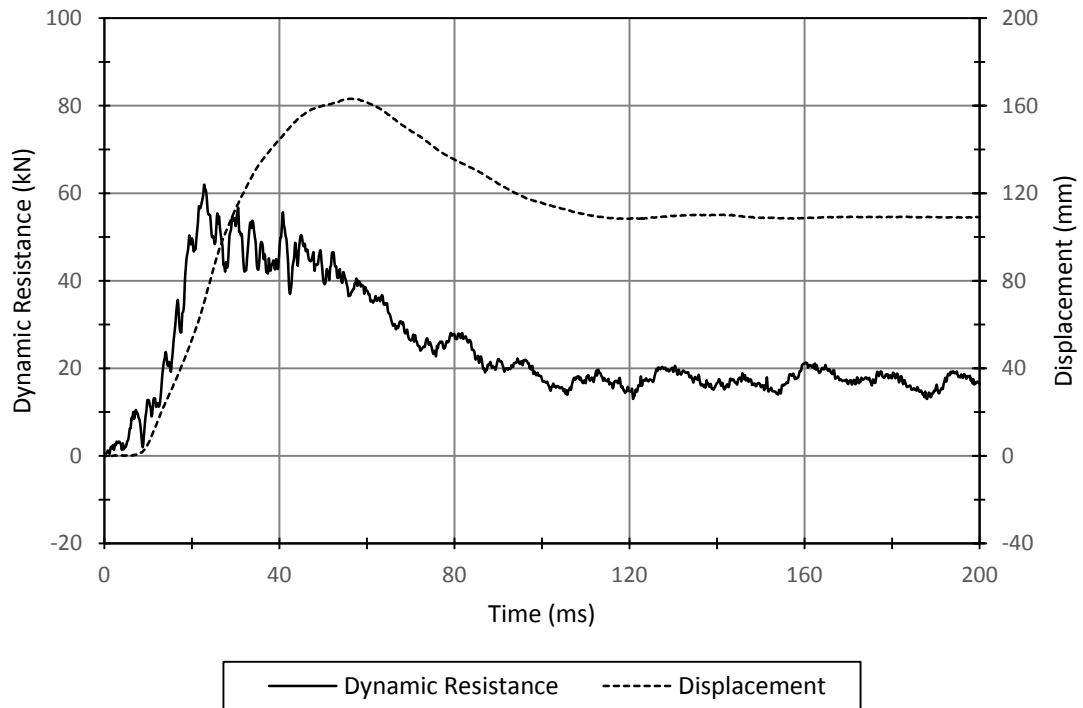
CLT5-D2.2

Test Date:	March 14, 2017	Slab Mass:	101.6	<i>kg</i>
Specimen No.:	9	Volume:	0.1946875	<i>m³</i>
Test Name:	CLT5-D2.2	Density:	521.9	<i>kg/m³</i>
Shot No.:	2	Moisture Content:	10.0	%

Total Slab Height:	2,500	<i>mm</i>
Slab Clear Span:	2,235	<i>mm</i>
Configuration:	V [5] + H [24]	
Approximate Length of Straps Used:	57.98	<i>m</i>
Driver Length:	4,572	<i>mm</i>
Driver Pressure:	380.6	<i>kPa</i>
Maximum Reflected Pressure:	51.2	<i>kPa</i>
Maximum Reflected Impulse:	702.8	<i>kPa – ms</i>
Positive Phase Duration:	31.8	<i>ms</i>
Maximum Mid-Span Deflection:	163.2	<i>mm</i>
Time to Mid-Span Deflection:	56.6	<i>ms</i>
Maximum Dynamic Resistance:	61.9	<i>kN</i>
Deflection at Dynamic Resistance:	70.3	<i>mm</i>

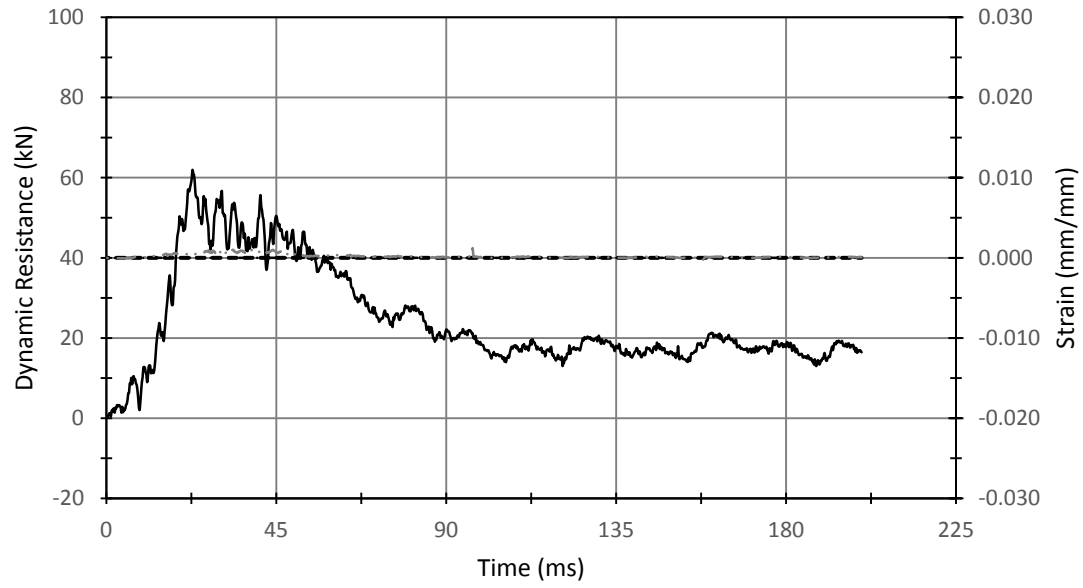


(a) Pressure and Impulse

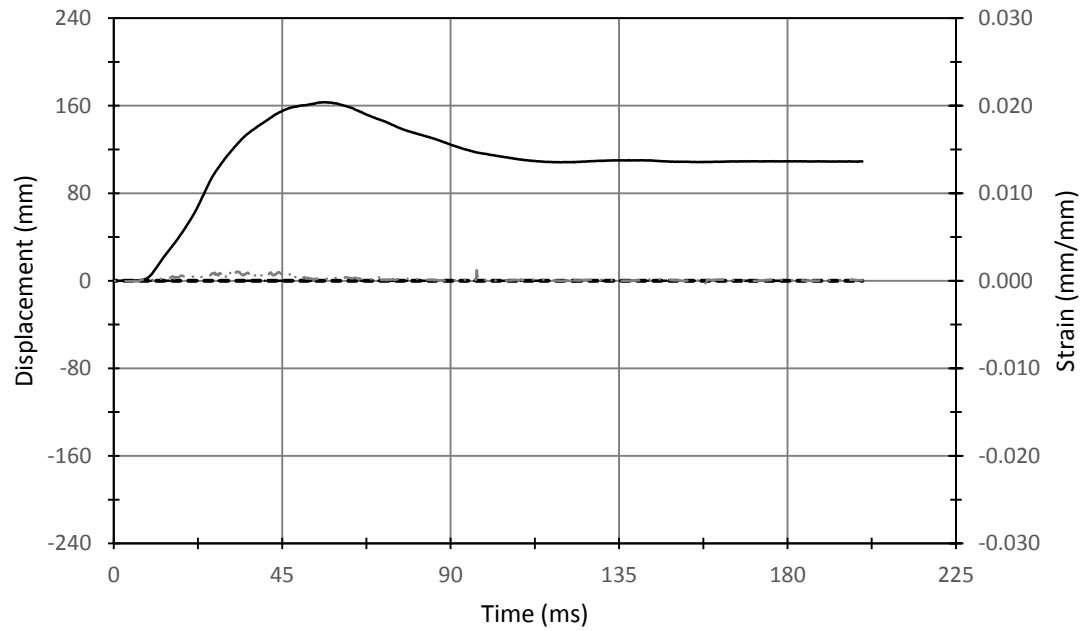


(b) Dynamic Resistance and Displacement

Figure D.34: Test Results for CLT5-D2.2



(a) Dynamic Resistance and Strain

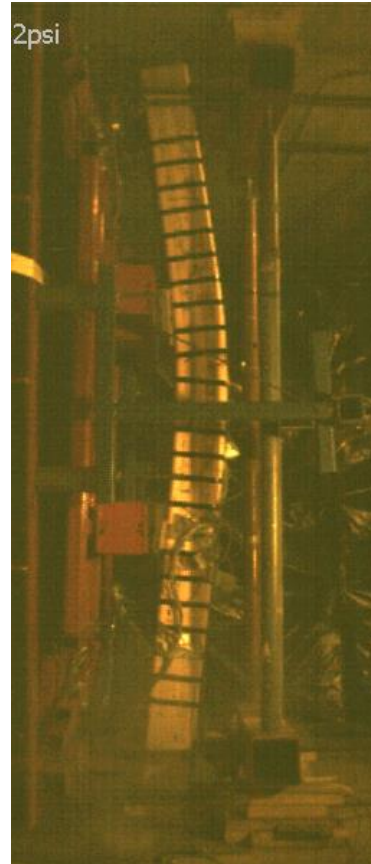


(b) Displacement and Strain

Figure D.35: Test Results for CLT5-D2.2 Continued



(a) Initial Position



(b) Maximum Dynamic Resistance

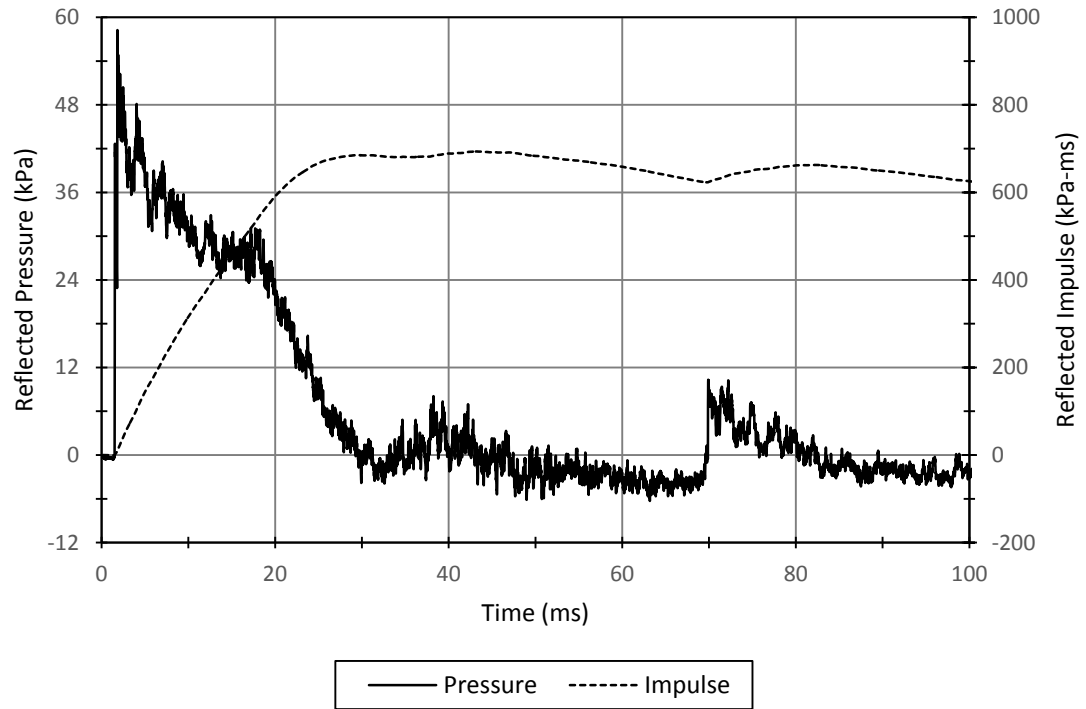


(c) Maximum Displacement

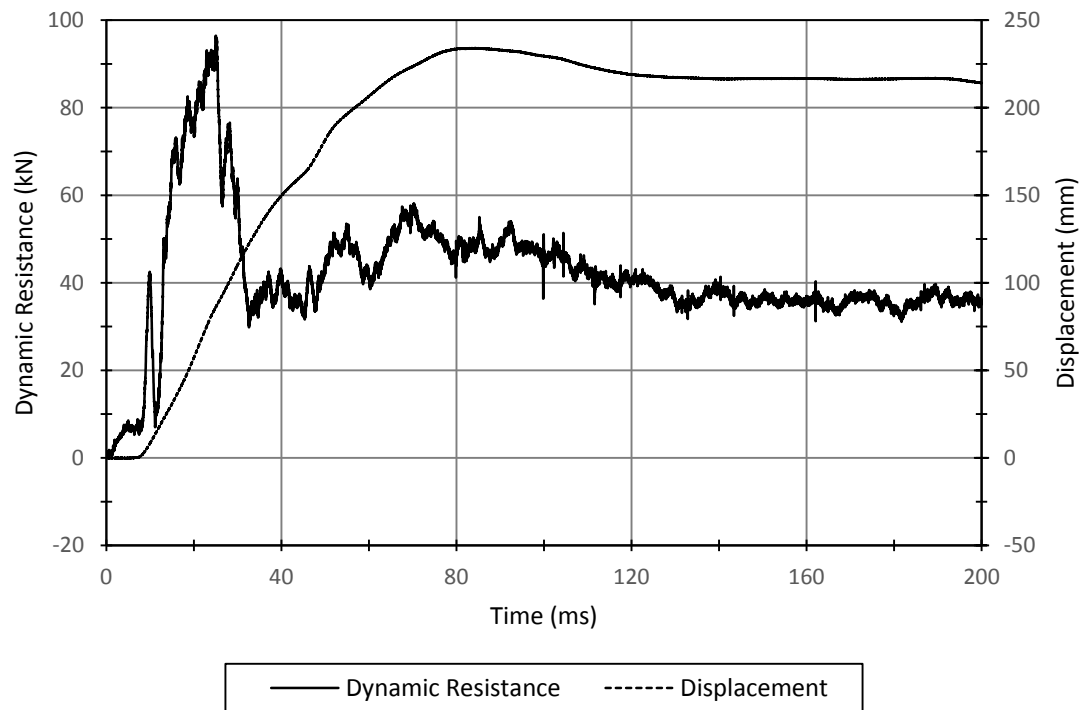
Figure D.36: Progression of Failure of CLT5-D2.2

CLT7-D1

Test Date:	March 17, 2017	Slab Mass:	101.6	<i>kg</i>
Specimen No.:	10	Volume:	0.1800000	<i>m</i> ³
Test Name:	CLT7-D1	Density:	564.4	<i>kg/m</i> ³
Shot No.:	1	Moisture Content:	10.0	%
Total Slab Height:	2,500		<i>mm</i>	
Slab Clear Span:	2,235		<i>mm</i>	
Configuration:	H [24]			
Approximate Length of Straps Used:	27.14		<i>m</i>	
Driver Length:	4,572		<i>mm</i>	
Driver Pressure:	413.0		<i>kPa</i>	
Maximum Reflected Pressure:	58.3		<i>kPa</i>	
Maximum Reflected Impulse:	693.6		<i>kPa – ms</i>	
Positive Phase Duration:	27.9		<i>ms</i>	
Maximum Mid-Span Deflection:	234.1		<i>mm</i>	
Time to Mid-Span Deflection:	84.2		<i>ms</i>	
Maximum Dynamic Resistance:	94.5		<i>kN</i>	
Deflection at Dynamic Resistance:	86.3		<i>mm</i>	

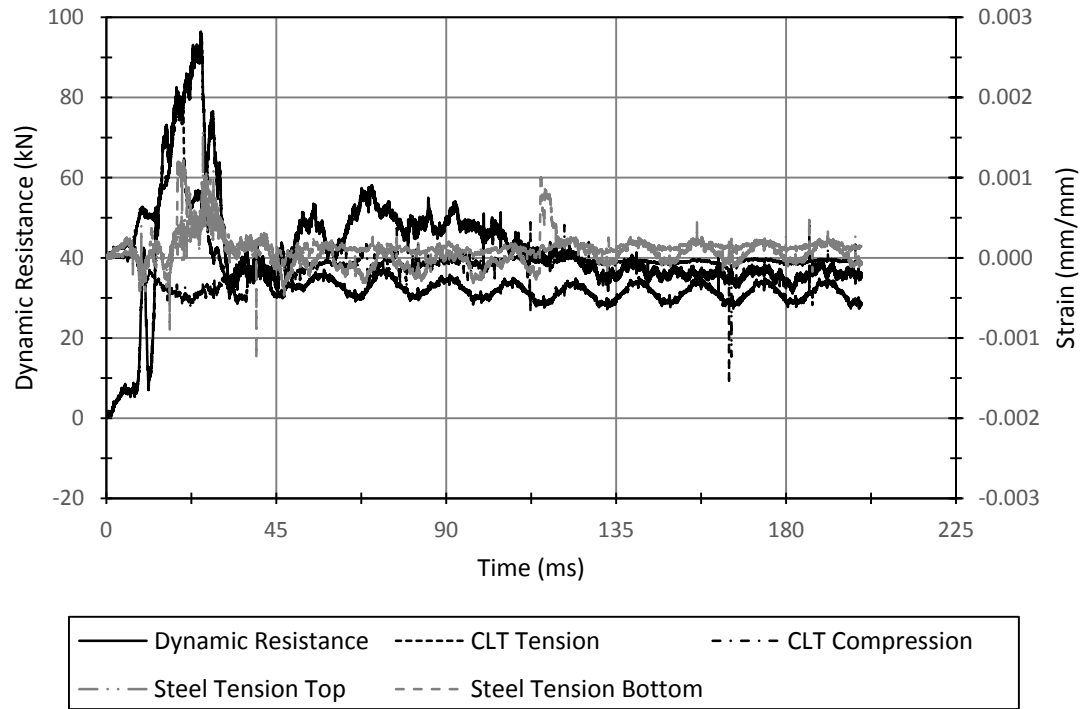


(a) Pressure and Impulse

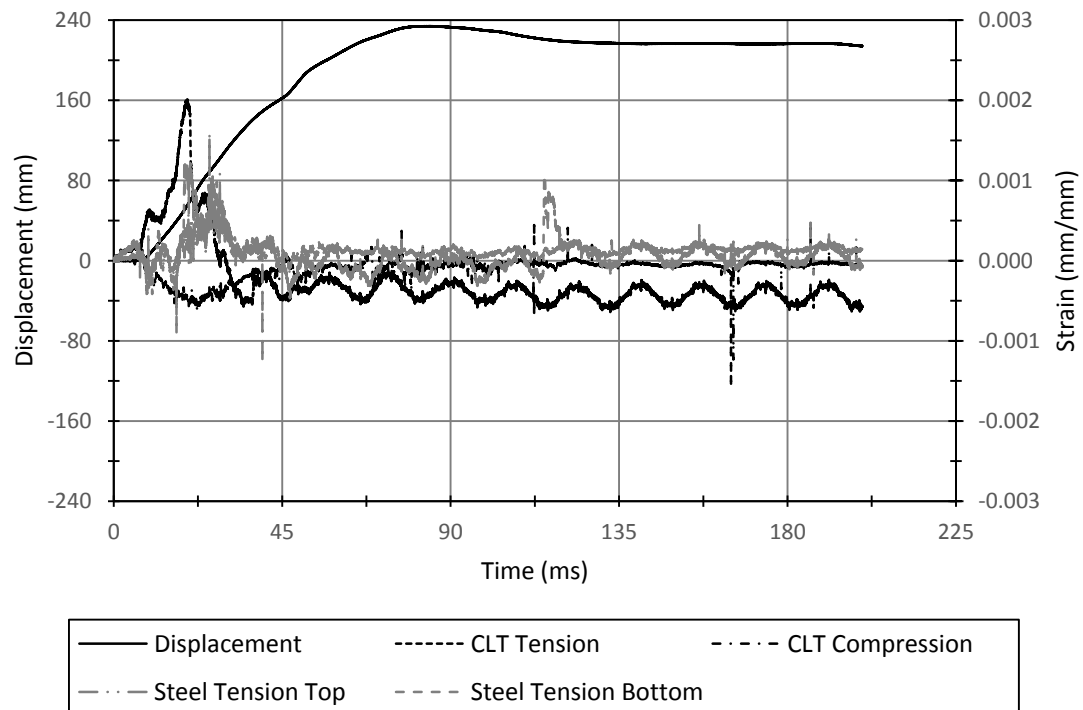


(b) Dynamic Resistance and Displacement

Figure D.37: Test Results for CLT7-D1



(a) Dynamic Resistance and Strain

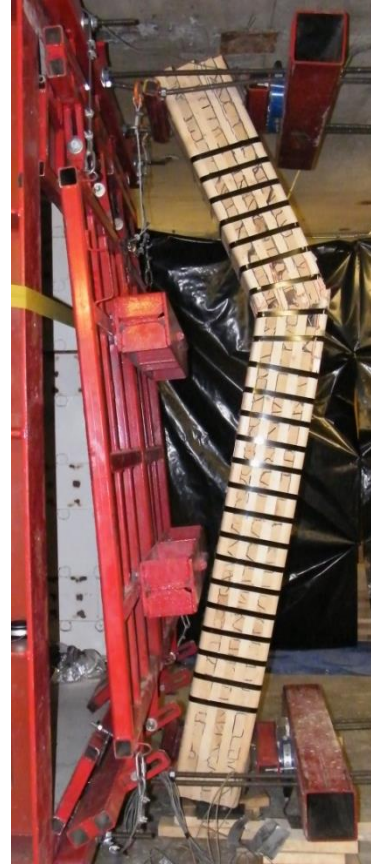


(b) Displacement and Strain

Figure D.38: Test Results for CLT7-D1 Continued



(a) Initial Position



(b) Final Position

Figure D.39: Slab Failure of CLT7-D1



(c) Displaced Shape

APPENDIX E – Phase 2 Dynamic Test Results

CLT3-3

Test Date:	June 14, 2016	Slab Mass:	67.7	<i>kg</i>
Specimen No.:	11	Volume:	0.1168125	<i>m³</i>
Test Name:	CLT3-3	Density:	579.9	<i>kg/m³</i>
Shot No.:	1	Moisture Content:	11.0	%

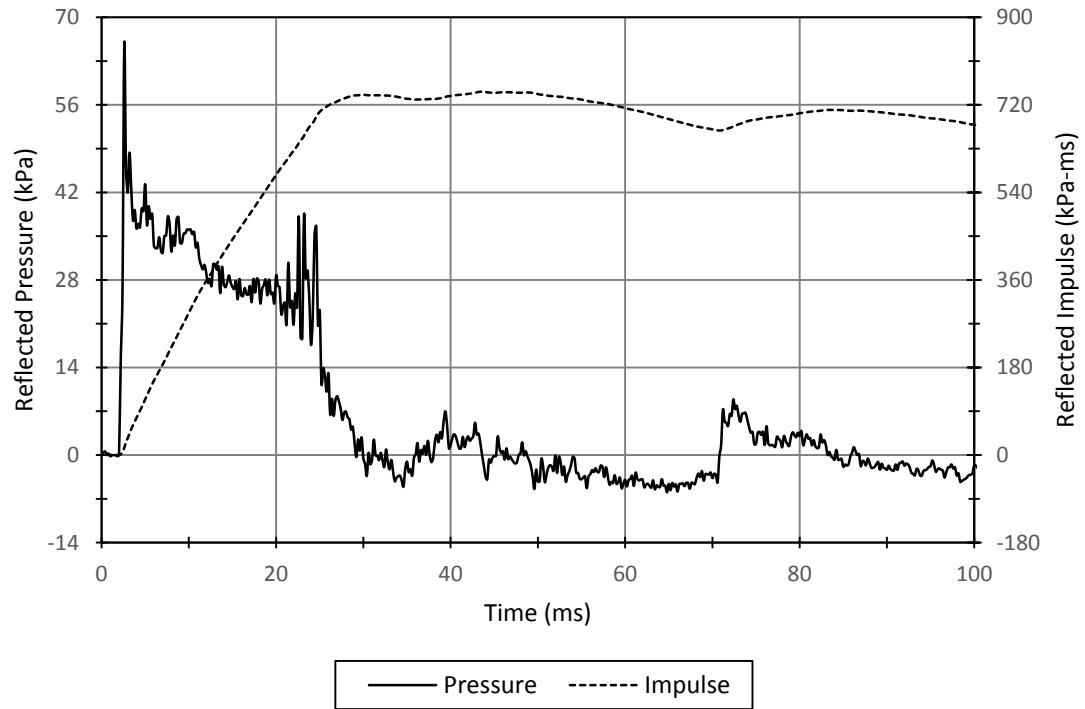
Total Slab Height:	2,500	<i>mm</i>
Slab Clear Span:	2,235	<i>mm</i>

Configuration:	2 U [0°]	
Approximate Area of GFRP Used:	2.71	<i>m²</i>

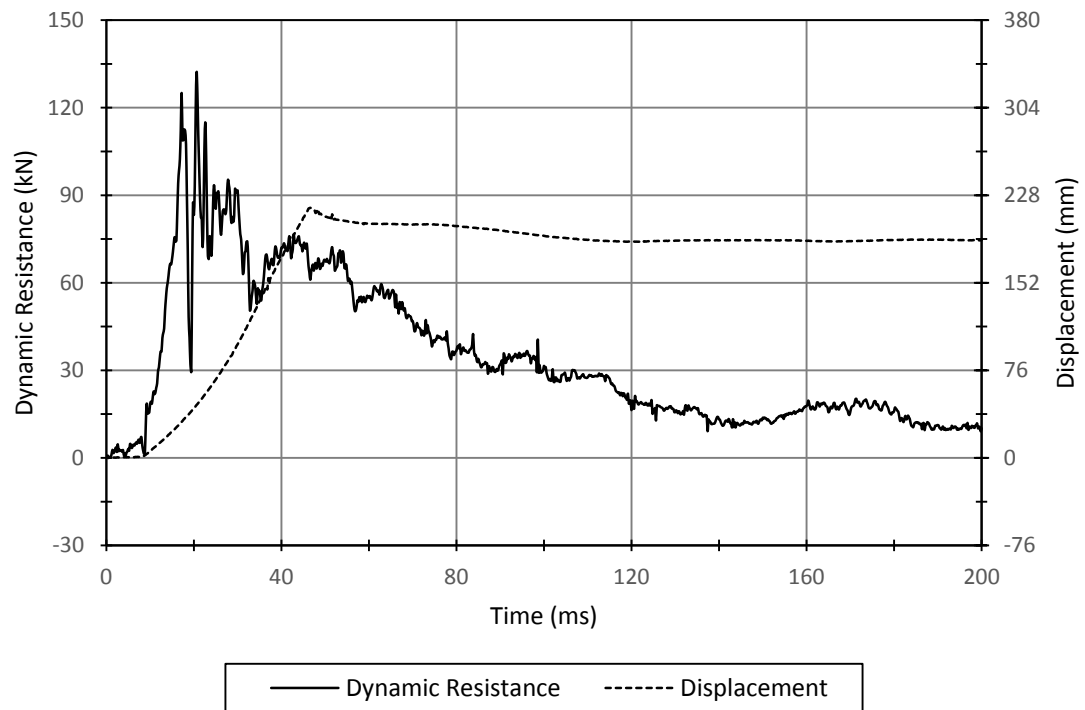
Driver Length:	4,572	<i>mm</i>
Driver Pressure:	424.0	<i>kPa</i>
Maximum Reflected Pressure:	65.8	<i>kPa</i>
Maximum Reflected Impulse:	747.2	<i>kPa – ms</i>
Positive Phase Duration:	29.2	<i>ms</i>

Maximum Mid-Span Deflection:	217.1	<i>mm</i>
Time to Mid-Span Deflection:	46.6	<i>ms</i>

Maximum Dynamic Resistance:	132.2	<i>kN</i>
Deflection at Dynamic Resistance:	45.9	<i>mm</i>

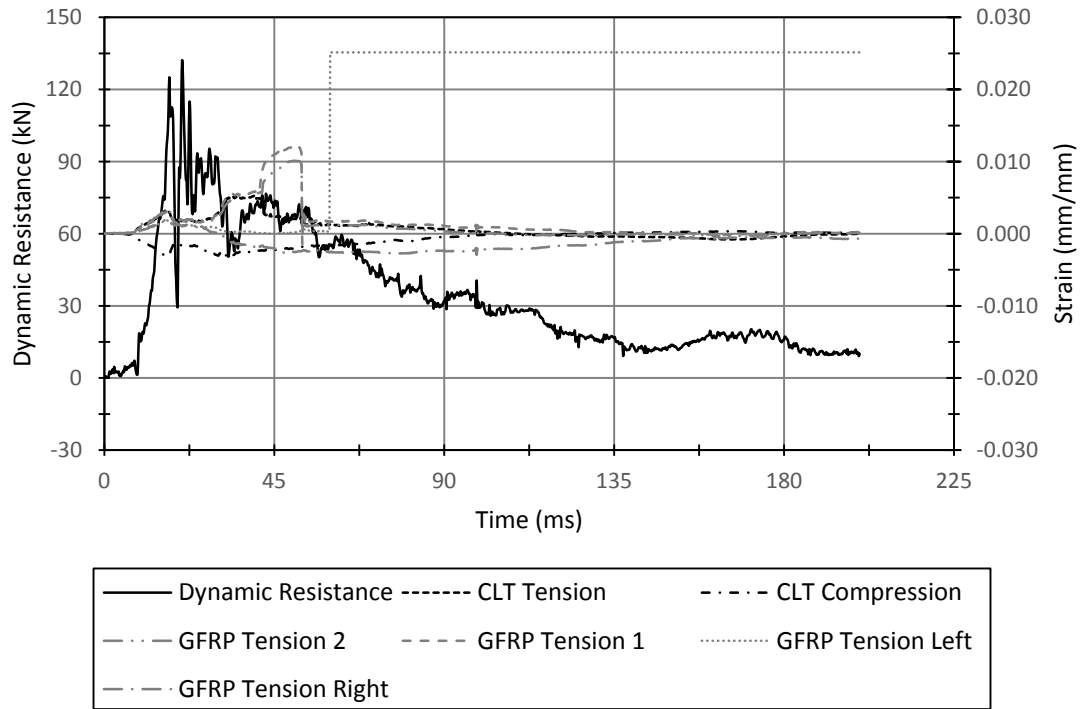


(a) Pressure and Impulse

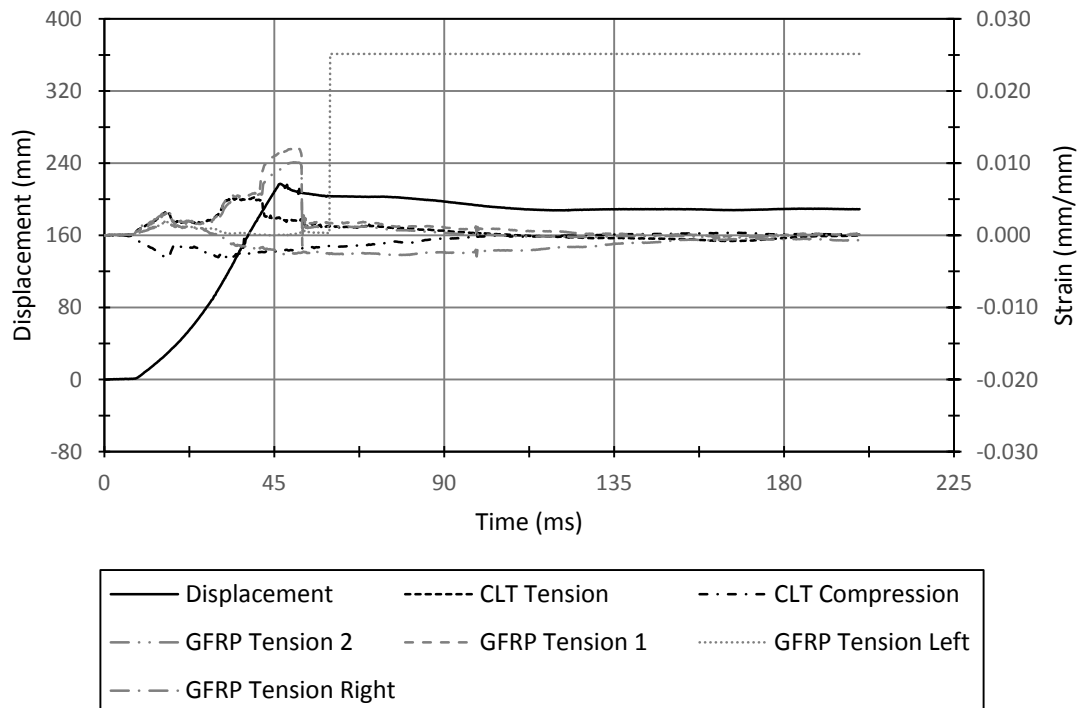


(b) Dynamic Resistance and Displacement

Figure E.1: Test Results for CLT3-3

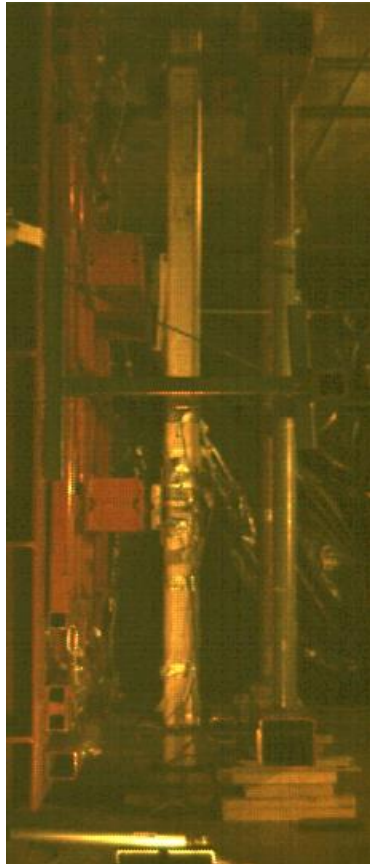


(a) Dynamic Resistance and Strain



(b) Displacement and Strain

Figure E.2: Test Results for CLT3-3 Continued



(a) Initial Position



(b) Maximum Dynamic Resistance

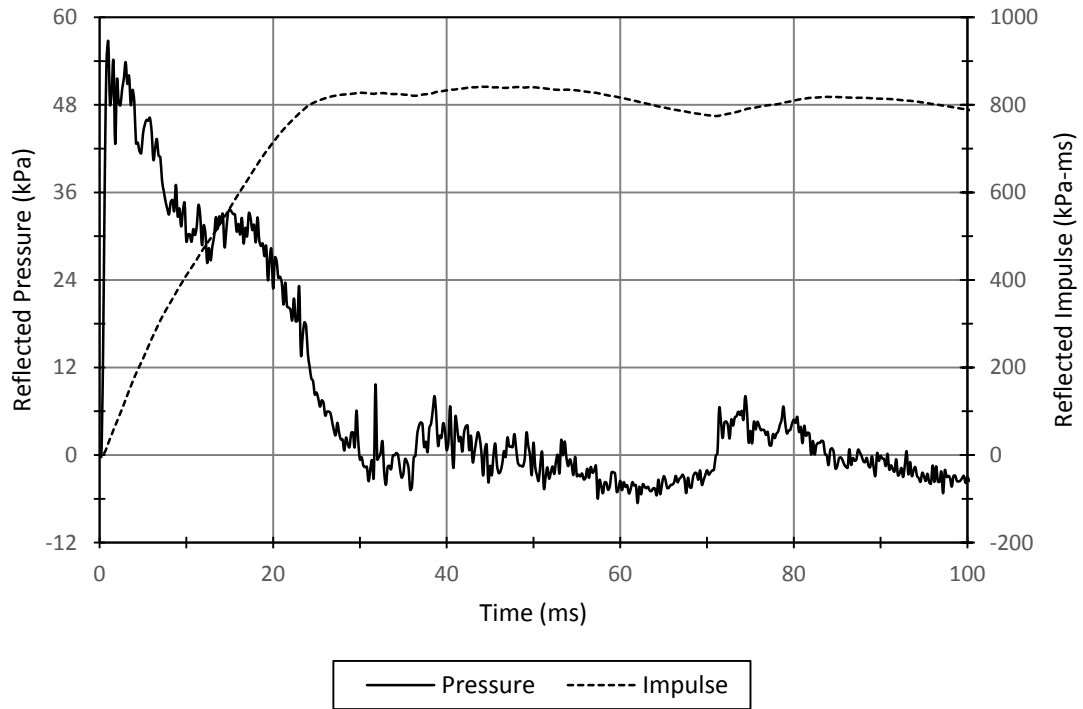


(c) Maximum Displacement

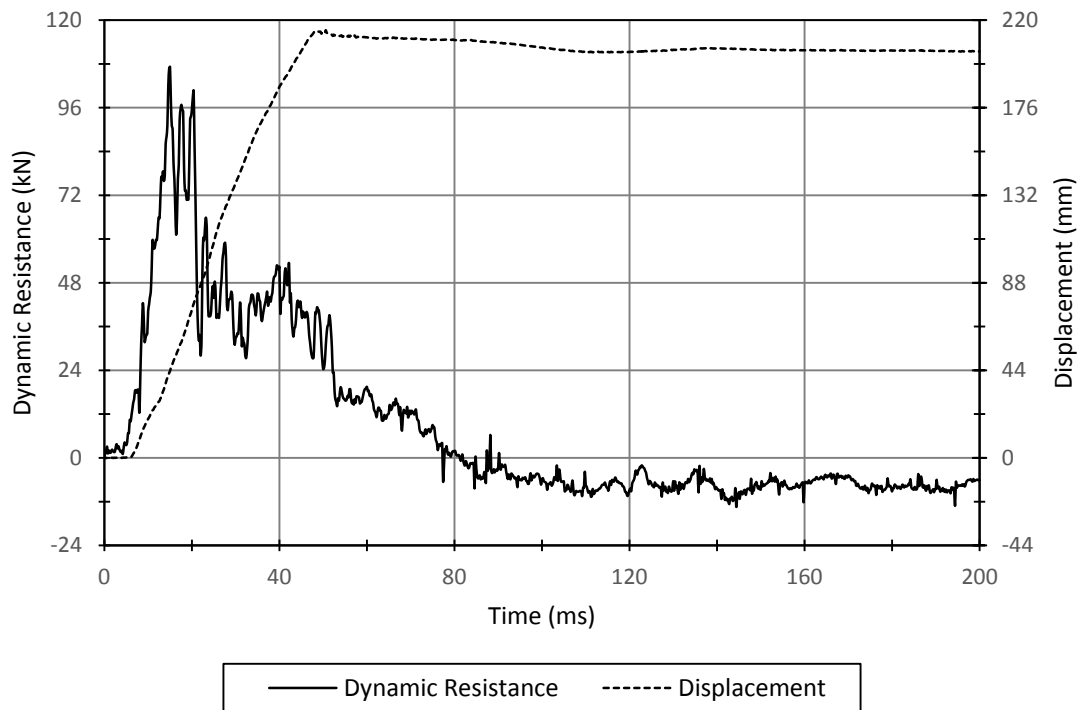
Figure E.3: Progression of Failure of CLT3-3

CLT3-4

Test Date:	April 3, 2017	Slab Mass:	72.4	<i>kg</i>
Specimen No.:	12	Volume:	0.1168125	<i>m³</i>
Test Name:	CLT3-4	Density:	620.1	<i>kg/m³</i>
Shot No.:	1	Moisture Content:	11.0	%
Total Slab Height:		2,500		<i>mm</i>
Slab Clear Span:		2,235		<i>mm</i>
Configuration:		2 CM [0/90°]		
Approximate Area of GFRP Used:		5.97		<i>m²</i>
Driver Length:		4,572		<i>mm</i>
Driver Pressure:		430.9		<i>kPa</i>
Maximum Reflected Pressure:		56.7		<i>kPa</i>
Maximum Reflected Impulse:		841.6		<i>kPa – ms</i>
Positive Phase Duration:		30.0		<i>ms</i>
Maximum Mid-Span Deflection:		214.9		<i>mm</i>
Time to Mid-Span Deflection:		50.6		<i>ms</i>
Maximum Dynamic Resistance:		107.2		<i>kN</i>
Deflection at Dynamic Resistance:		43.7		<i>mm</i>

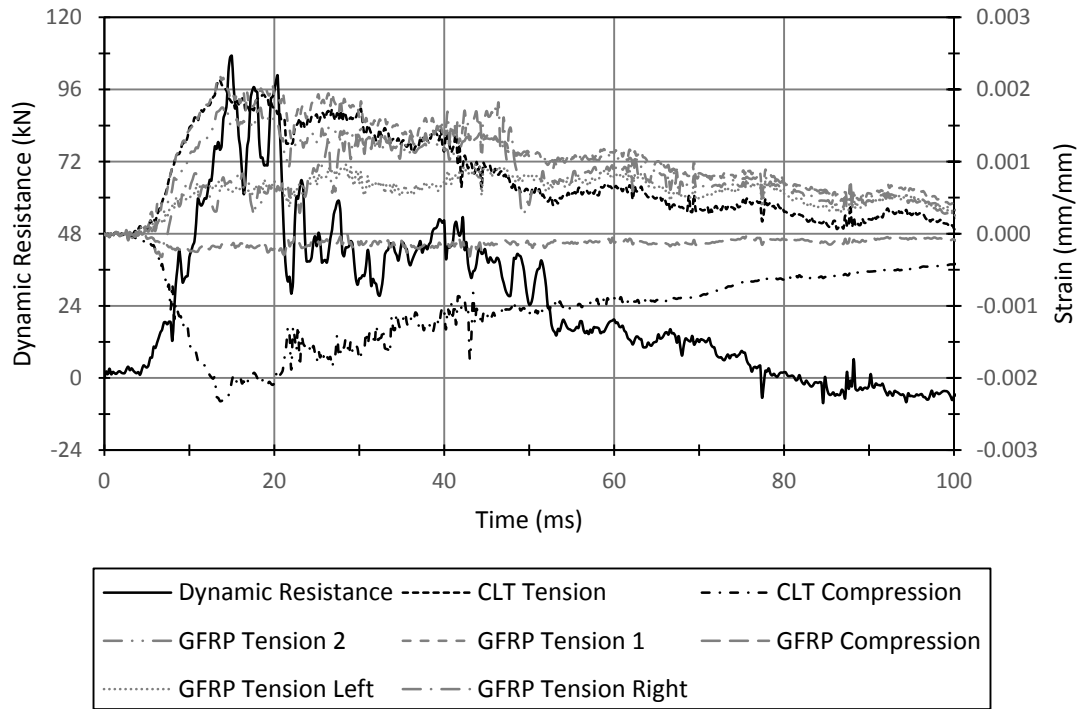


(a) Pressure and Impulse

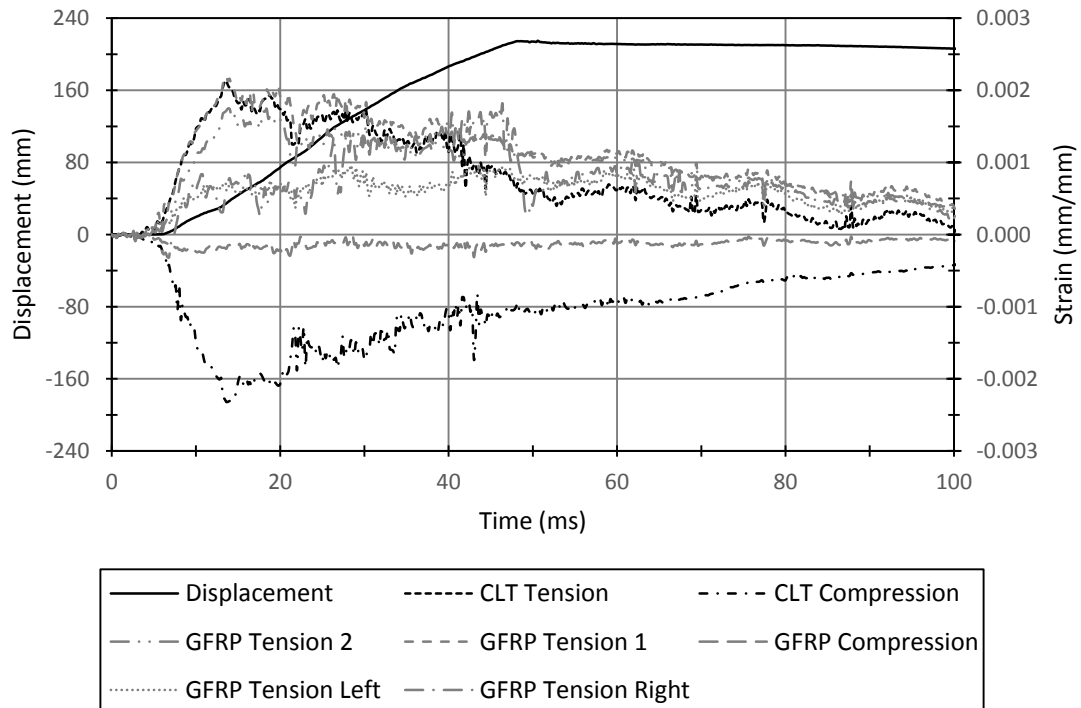


(b) Dynamic Resistance and Displacement

Figure E.4: Test Results for CLT3-4



(a) Dynamic Resistance and Strain



(b) Displacement and Strain

Figure E.5: Test Results for CLT3-4 Continued



(a) Initial Position



(b) Maximum Displacement

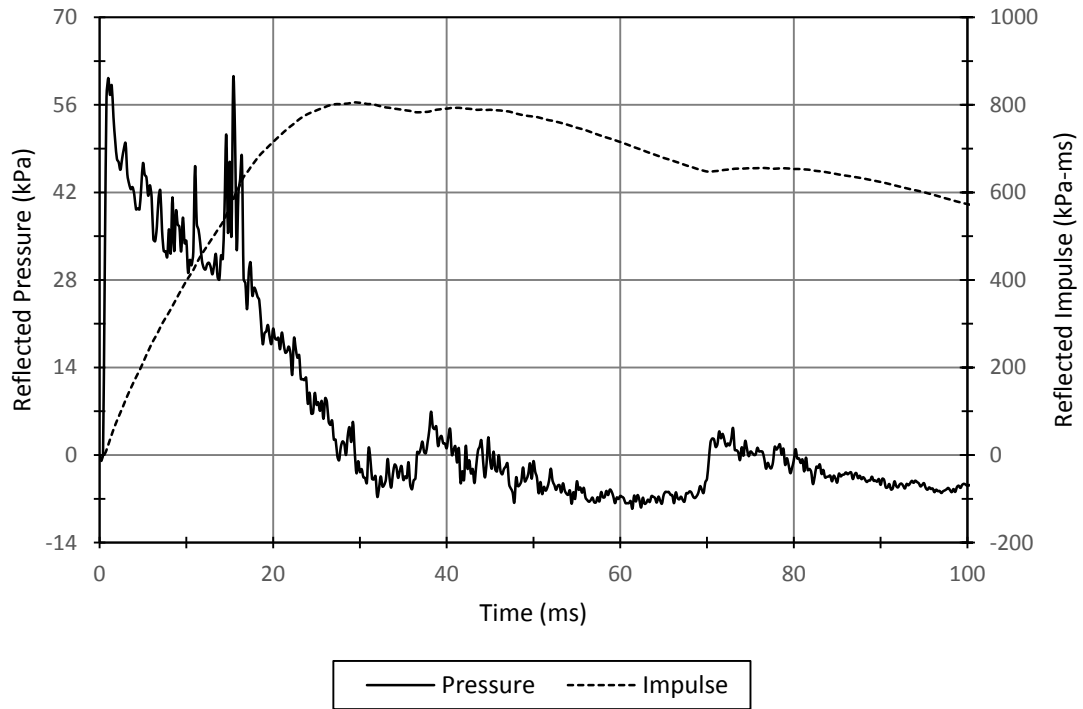


(c) Final Position

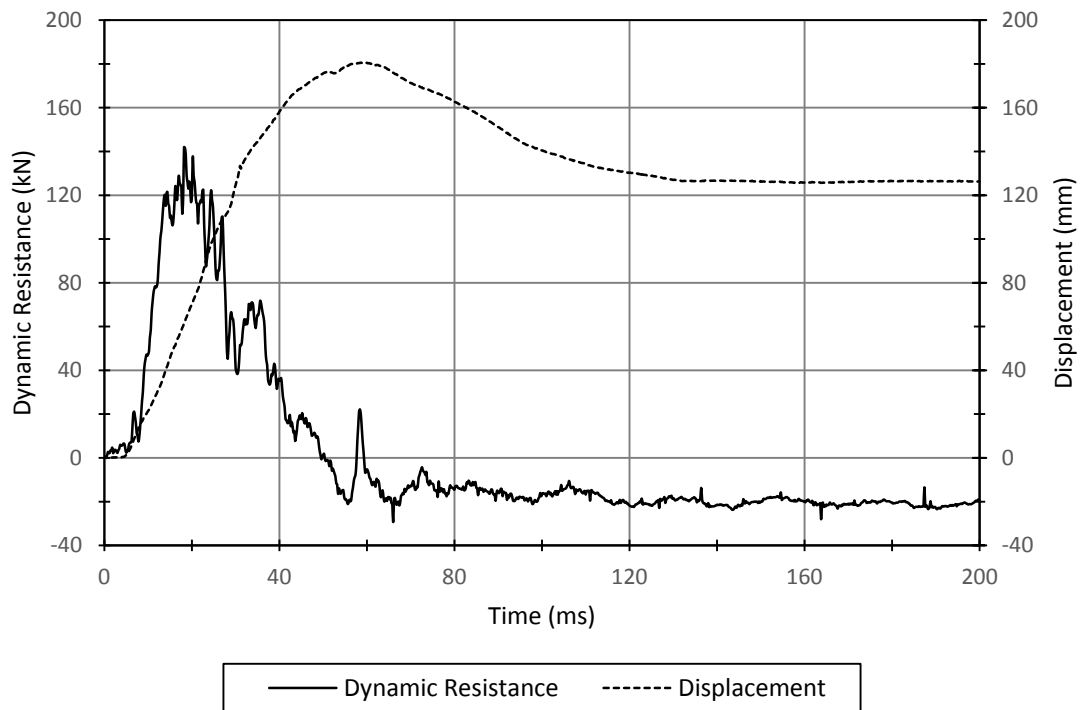
Figure E.6: Progression of Failure of CLT3-4

CLT3-5

Test Date:	April 11, 2017	Slab Mass:	72.6	<i>kg</i>
Specimen No.:	13	Volume:	0.1168125	<i>m³</i>
Test Name:	CLT3-5	Density:	621.3	<i>kg/m³</i>
Shot No.:	1	Moisture Content:	11.0	%
Total Slab Height:		2,500		<i>mm</i>
Slab Clear Span:		2,235		<i>mm</i>
Configuration:		2 CM [$\pm 45^\circ$]		
Approximate Area of GFRP Used:		5.97		<i>m²</i>
Driver Length:		4,572		<i>mm</i>
Driver Pressure:		481.9		<i>kPa</i>
Maximum Reflected Pressure:		60.3		<i>kPa</i>
Maximum Reflected Impulse:		805.7		<i>kPa – ms</i>
Positive Phase Duration:		26.7		<i>ms</i>
Maximum Mid-Span Deflection:		180.6		<i>mm</i>
Time to Mid-Span Deflection:		59.4		<i>ms</i>
Maximum Dynamic Resistance:		141.9		<i>kN</i>
Deflection at Dynamic Resistance:		61.5		<i>mm</i>

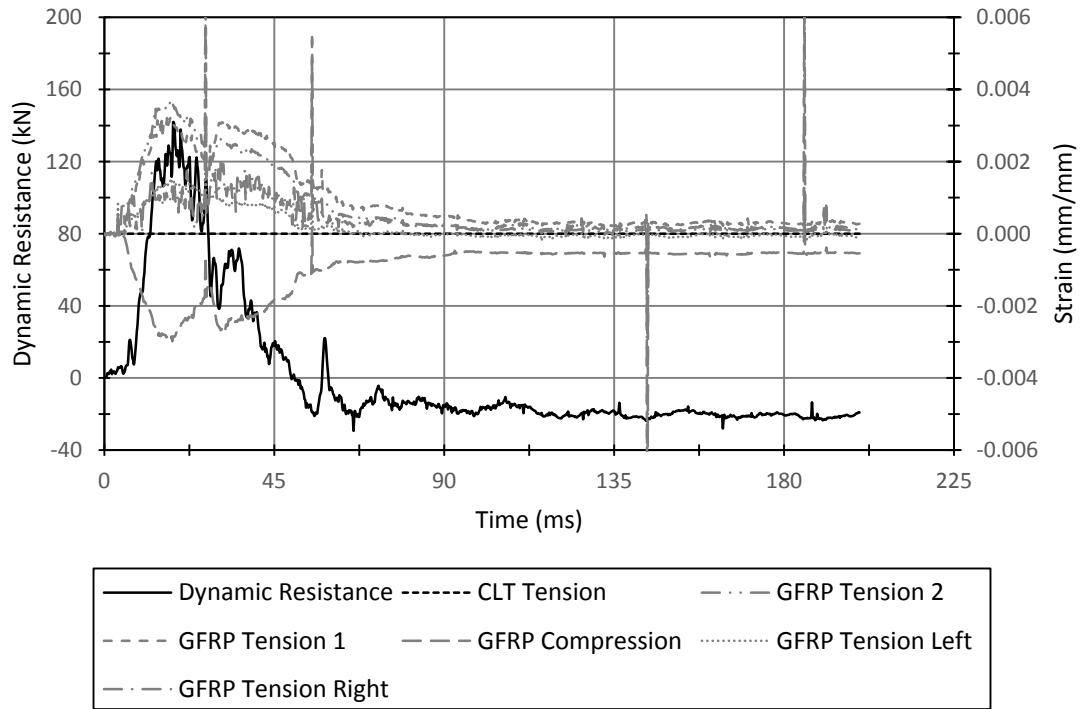


(a) Pressure and Impulse

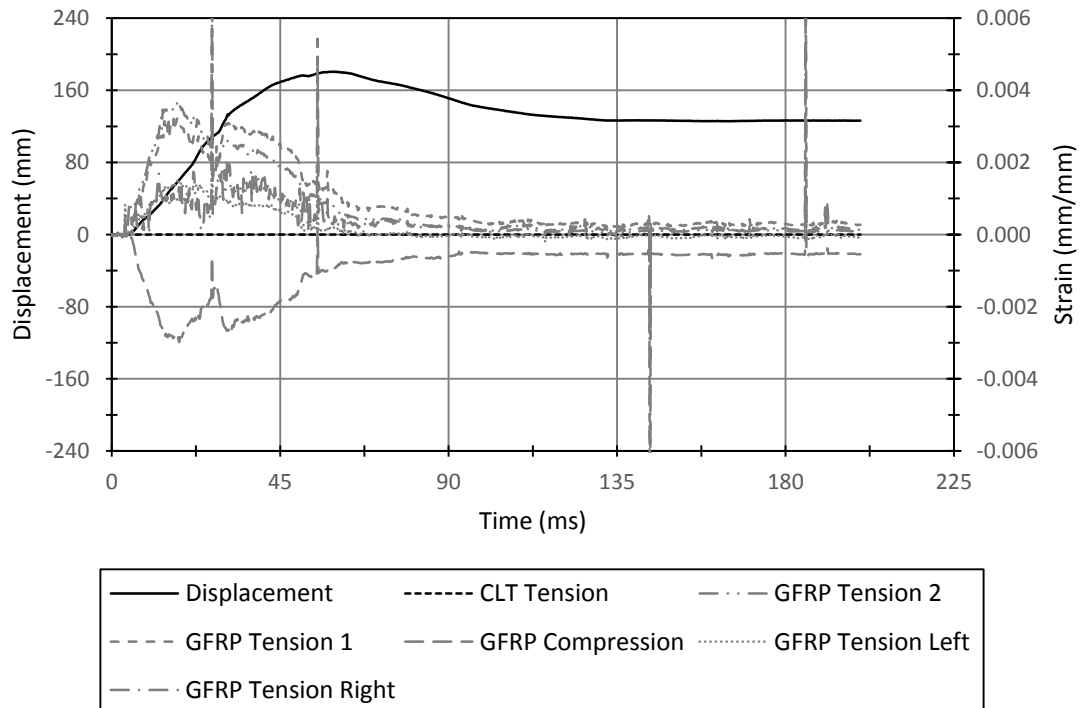


(b) Dynamic Resistance and Displacement

Figure E.7: Test Results for CLT3-5



(a) Dynamic Resistance and Strain



(b) Displacement and Strain

Figure E.8: Test Results for CLT3-5 Continued



(a) Initial Position



(b) Maximum Displacement

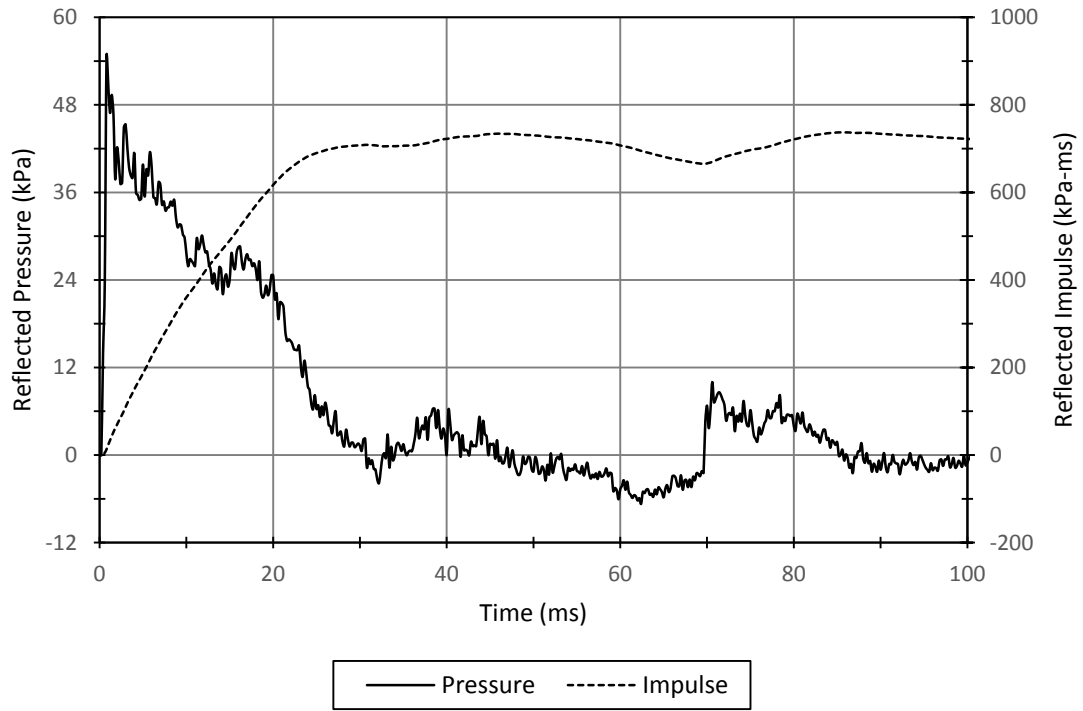


(c) Final Position

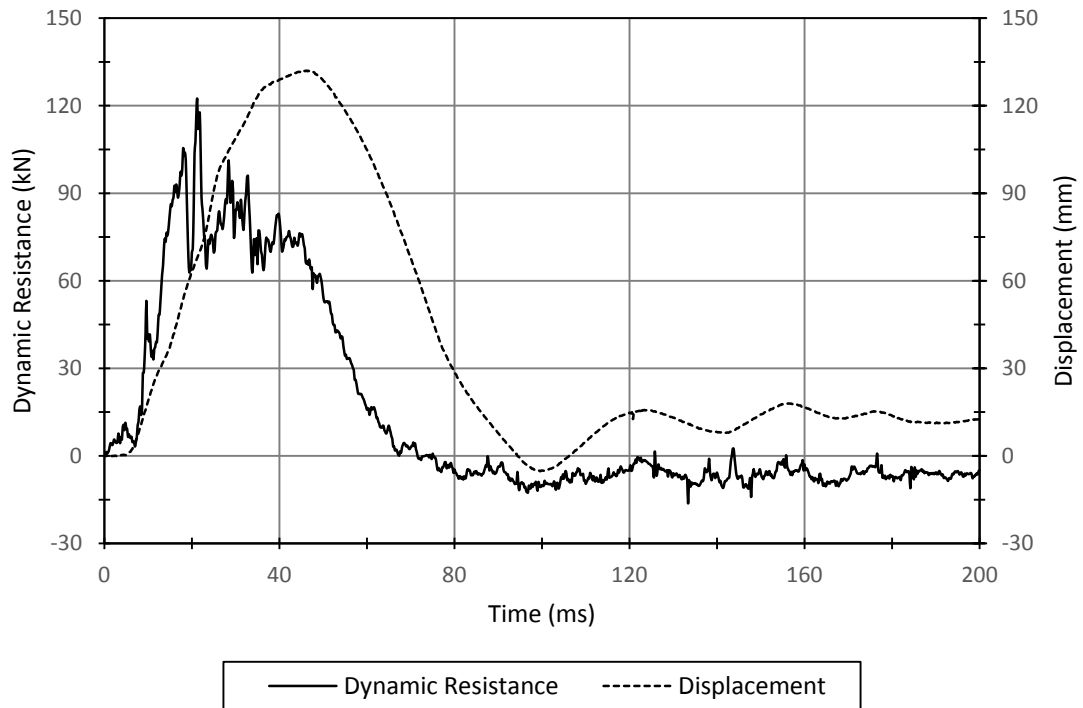
Figure E.9: Progression of Failure of CLT3-5

CLT3-6

Test Date:	March 31, 2017	Slab Mass:	71.7	<i>kg</i>
Specimen No.:	14	Volume:	0.1168125	<i>m³</i>
Test Name:	CLT3-6	Density:	614.1	<i>kg/m³</i>
Shot No.:	1	Moisture Content:	10.0	%
Total Slab Height:		2,500		<i>mm</i>
Slab Clear Span:		2,235		<i>mm</i>
Configuration:	CM [0/90°] + CM [± 45°]			
Approximate Area of GFRP Used:		5.97		<i>m²</i>
Driver Length:		4,572		<i>mm</i>
Driver Pressure:		418.5		<i>kPa</i>
Maximum Reflected Pressure:		54.6		<i>kPa</i>
Maximum Reflected Impulse:		737.2		<i>kPa – ms</i>
Positive Phase Duration:		30.2		<i>ms</i>
Maximum Mid-Span Deflection:		132.1		<i>mm</i>
Time to Mid-Span Deflection:		45.2		<i>ms</i>
Maximum Dynamic Resistance:		122.5		<i>kN</i>
Deflection at Dynamic Resistance:		67.7		<i>mm</i>

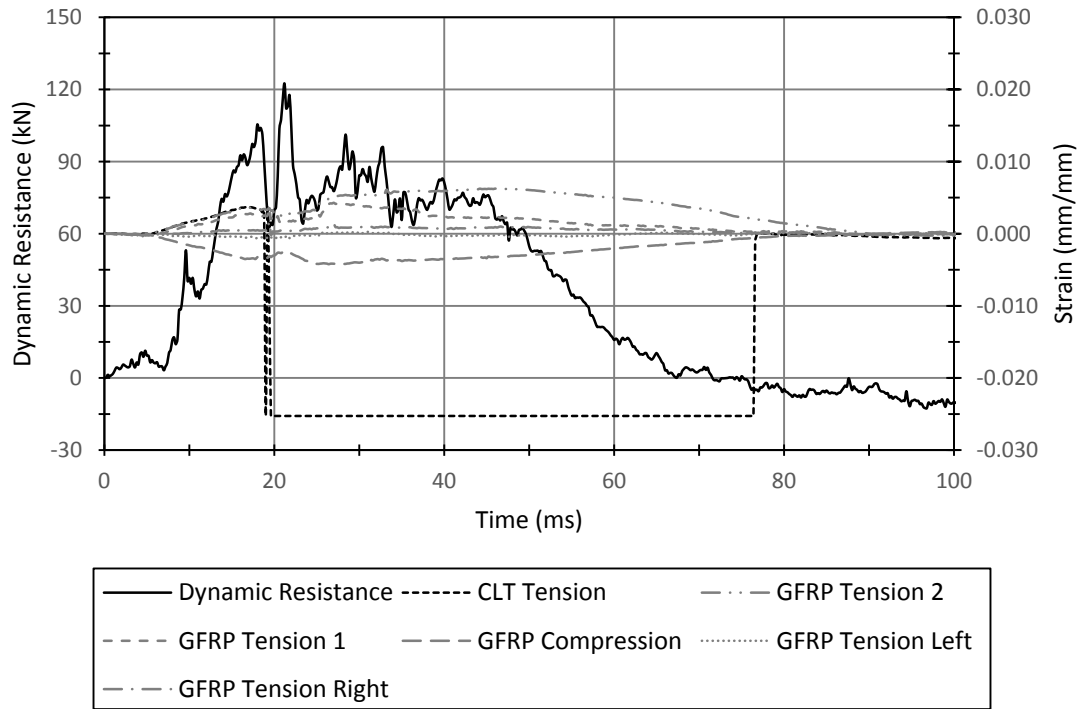


(a) Pressure and Impulse

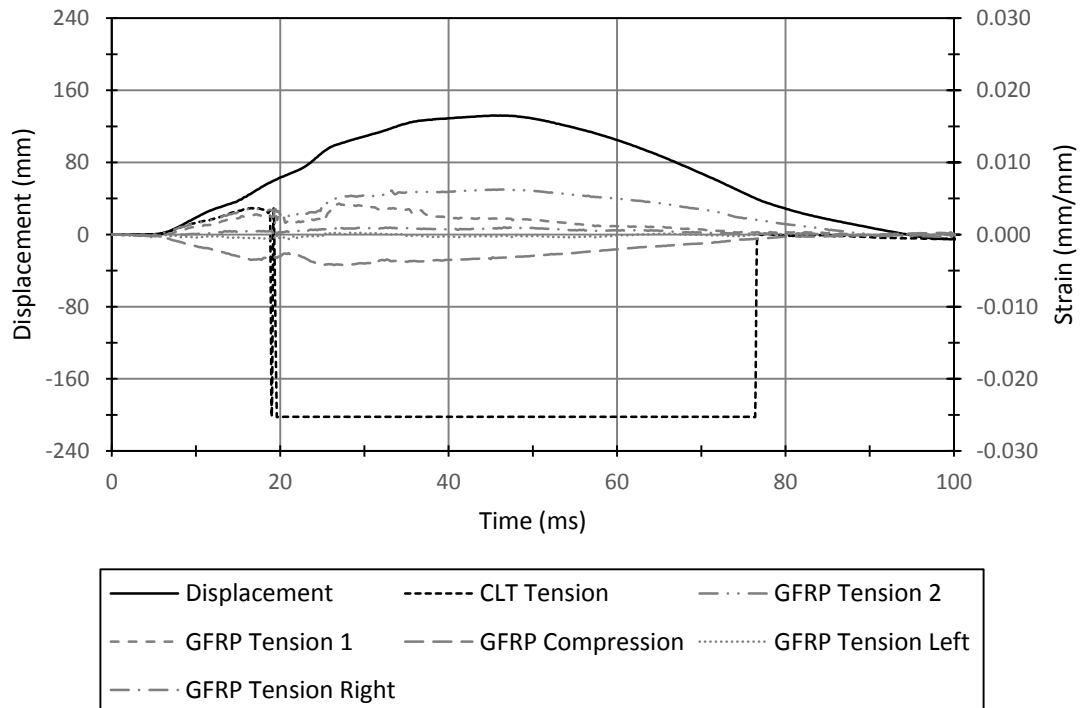


(b) Dynamic Resistance and Displacement

Figure E.10: Test Results for CLT3-6



(a) Dynamic Resistance and Strain



(b) Displacement and Strain

Figure E.11: Test Results for CLT3-6 Continued



(a) Initial Position



(b) Maximum Displacement

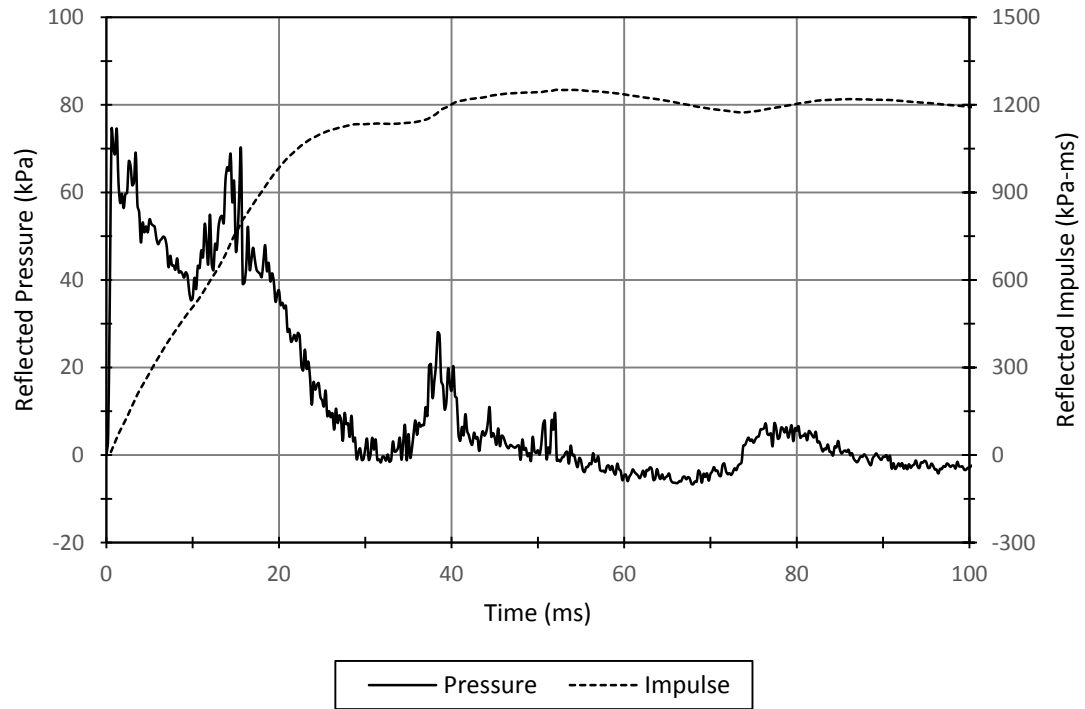


(c) Final Position

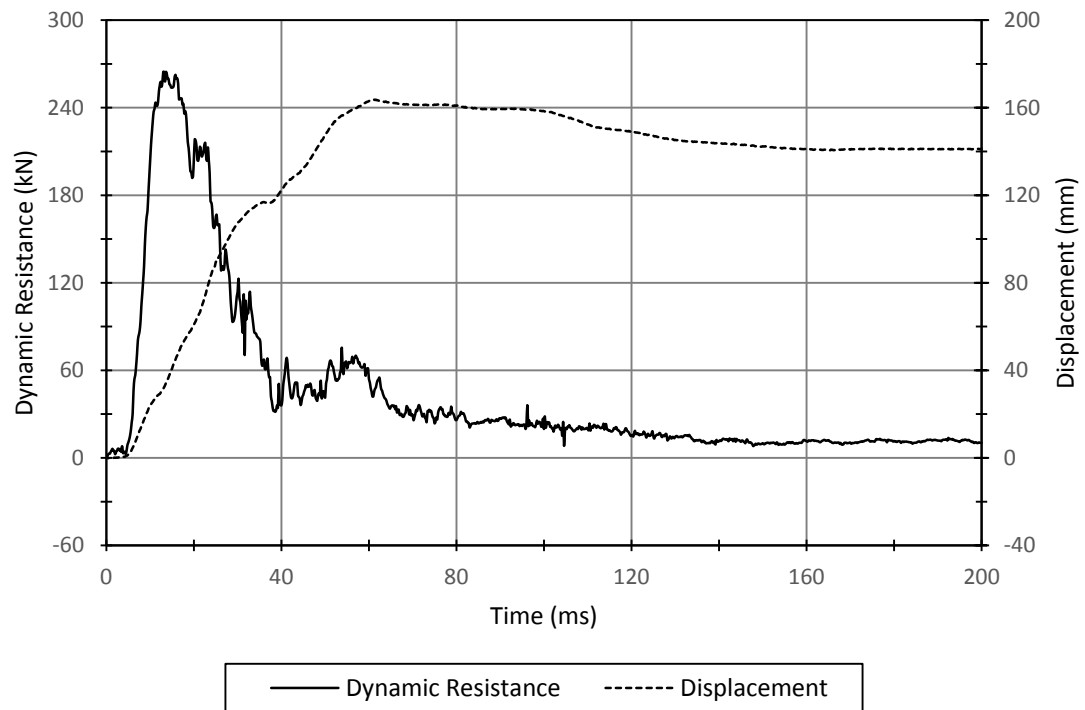
Figure E.12: Progression of Failure of CLT3-6

CLT5-6

Test Date:	April 10, 2017	Slab Mass:	107.9	<i>kg</i>
Specimen No.:	15	Volume:	0.1946875	<i>m³</i>
Test Name:	CLT5-6	Density:	554.5	<i>kg/m³</i>
Shot No.:	1	Moisture Content:	11.0	%
Total Slab Height:		2,500		<i>mm</i>
Slab Clear Span:		2,235		<i>mm</i>
Configuration:		2 CL [0, ± 45°]		
Approximate Area of GFRP Used:		5.42		<i>m²</i>
Driver Length:		4,572		<i>mm</i>
Driver Pressure:		650.9		<i>kPa</i>
Maximum Reflected Pressure:		92.9		<i>kPa</i>
Maximum Reflected Impulse:		1,258.4		<i>kPa – ms</i>
Positive Phase Duration:		29.2		<i>ms</i>
Maximum Mid-Span Deflection:		163.7		<i>mm</i>
Time to Mid-Span Deflection:		61.2		<i>ms</i>
Maximum Dynamic Resistance:		264.5		<i>kN</i>
Deflection at Dynamic Resistance:		31.6		<i>mm</i>

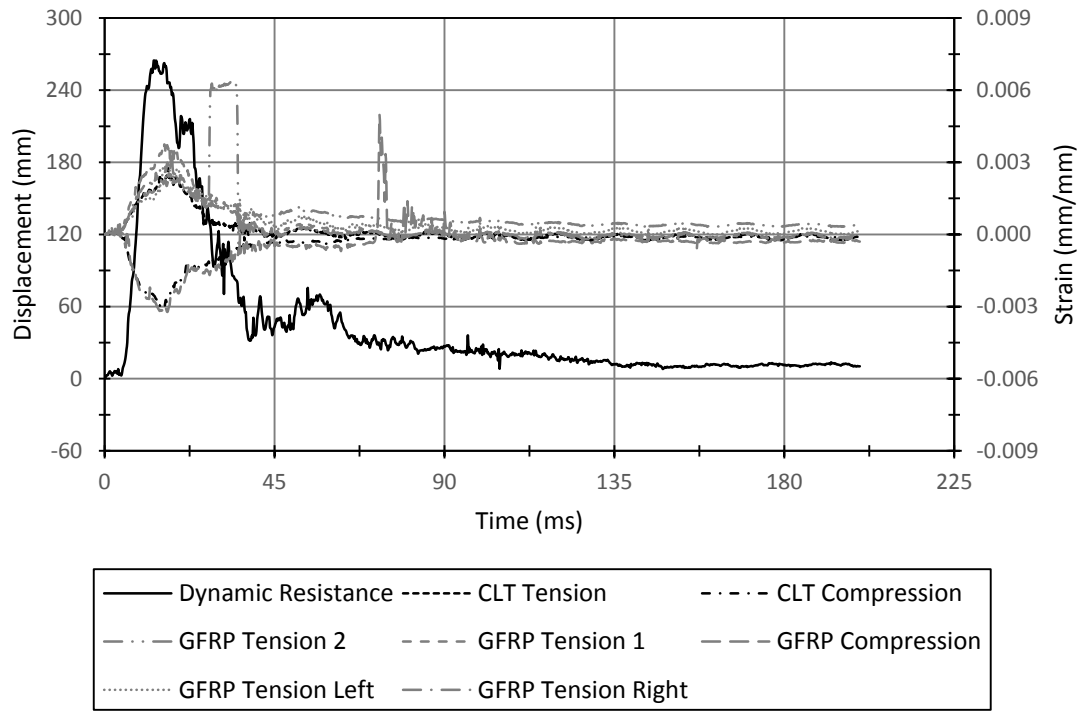


(a) Pressure and Impulse

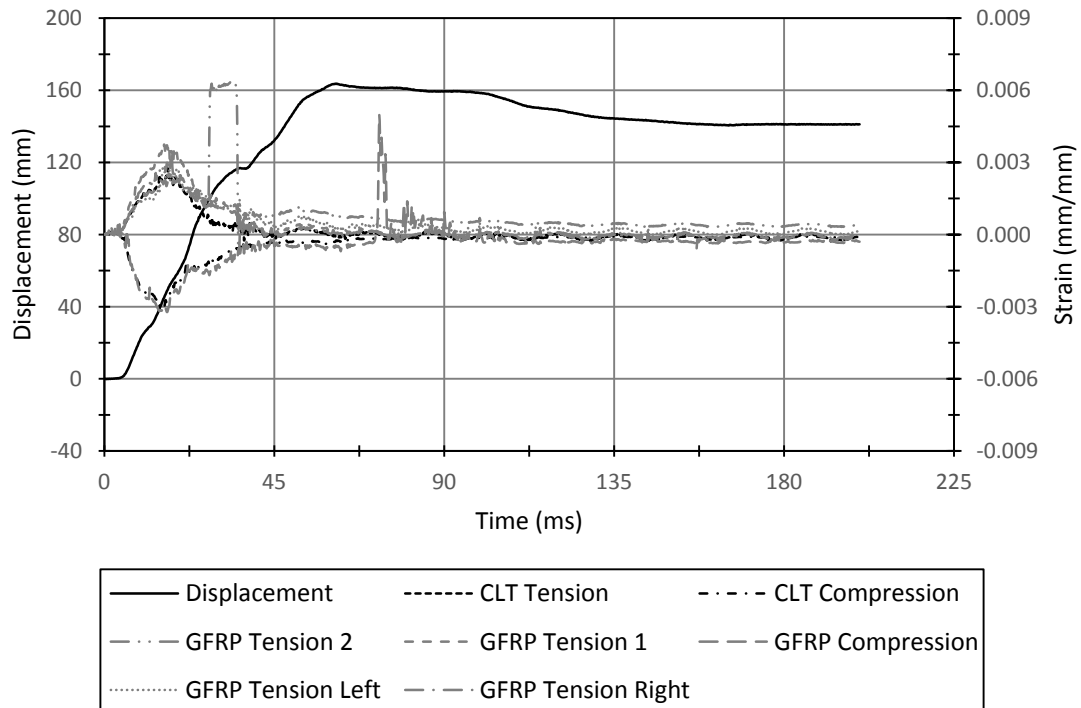


(b) Dynamic Resistance and Displacement

Figure E.13: Test Results for CLT5-6



(a) Dynamic Resistance and Strain



(b) Displacement and Strain

Figure E.14: Test Results for CLT5-6 Continued



(a) Initial Position



(b) Maximum Dynamic Resistance

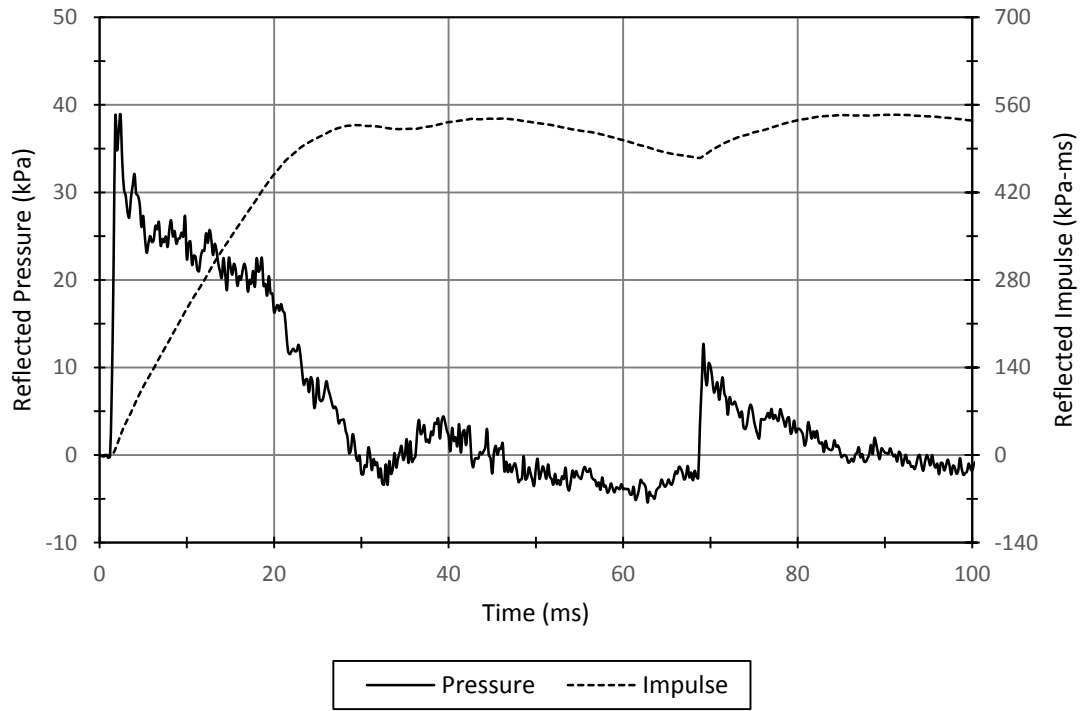


(c) Maximum Displacement

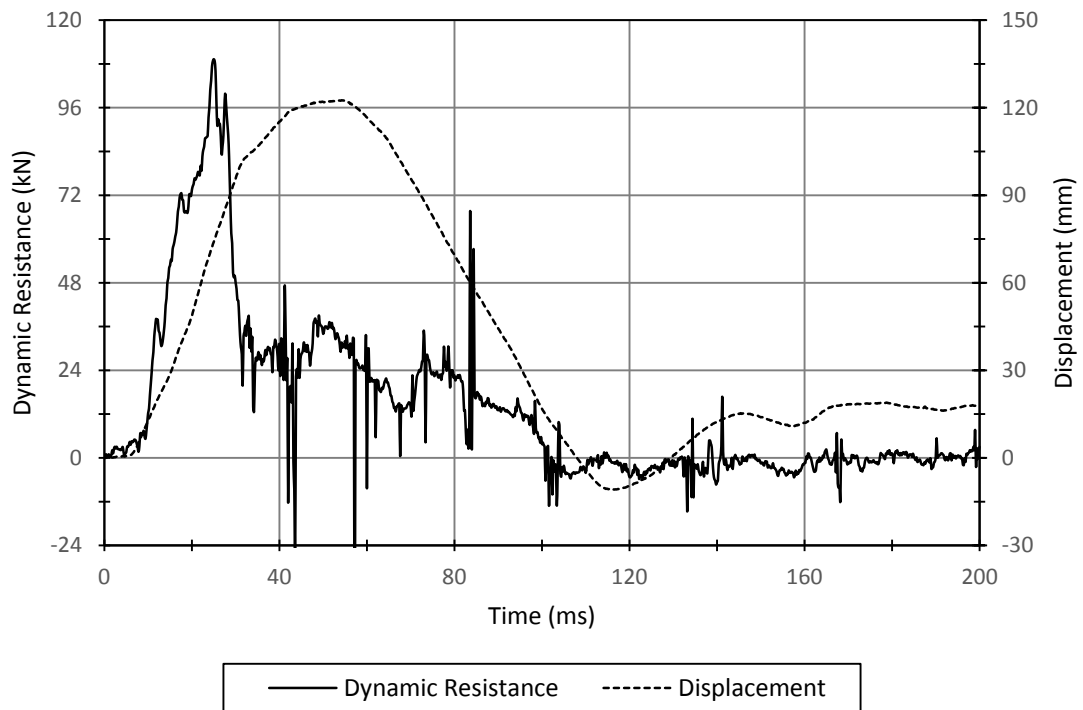
Figure E.15: Progression of Failure of CLT5-6

CLT3-D1

Test Date:	March 29, 2017	Slab Mass:	71.8	<i>kg</i>
Specimen No.:	16	Volume:	0.1168125	<i>m³</i>
Test Name:	CLT3-D1	Density:	614.3	<i>kg/m³</i>
Shot No.:	1	Moisture Content:	11.0	%
Total Slab Height:		2,500		<i>mm</i>
Slab Clear Span:		2,235		<i>mm</i>
Configuration:		2 CL [0, ± 45°]		
Approximate Area of GFRP Used:		5.42		<i>m²</i>
Driver Length:		4,572		<i>mm</i>
Driver Pressure:		321.3		<i>kPa</i>
Maximum Reflected Pressure:		38.8		<i>kPa</i>
Maximum Reflected Impulse:		544.3		<i>kPa – ms</i>
Positive Phase Duration:		28.6		<i>ms</i>
Maximum Mid-Span Deflection:		122.5		<i>mm</i>
Time to Mid-Span Deflection:		54.2		<i>ms</i>
Maximum Dynamic Resistance:		109.3		<i>kN</i>
Deflection at Dynamic Resistance:		74.0		<i>mm</i>

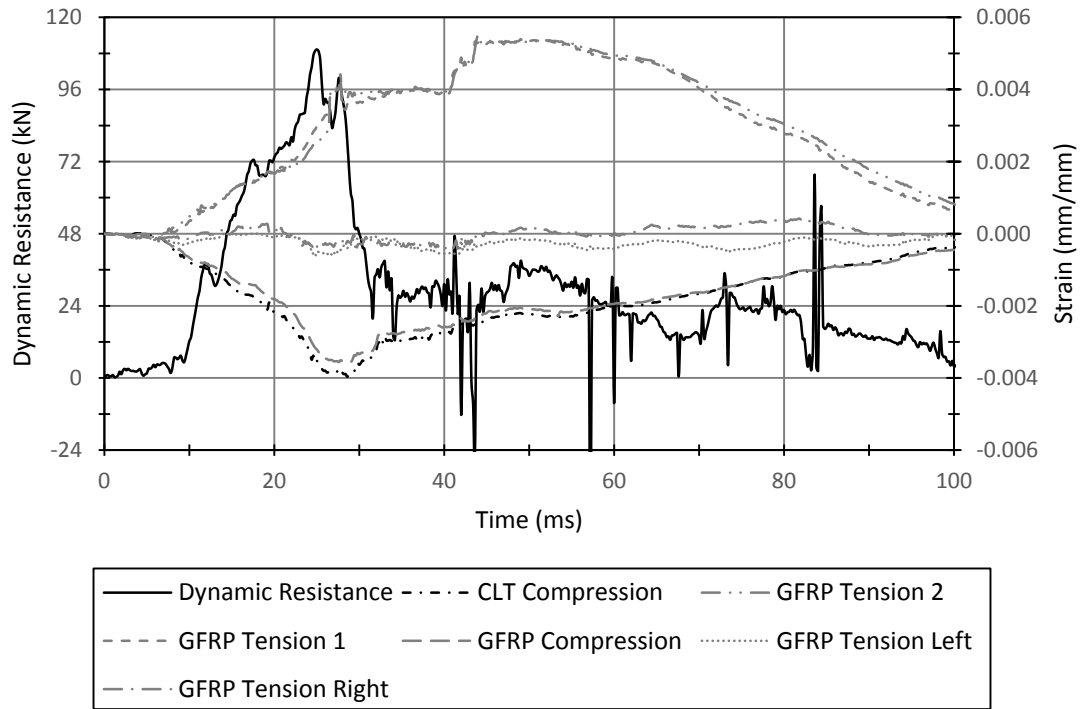


(a) Pressure and Impulse

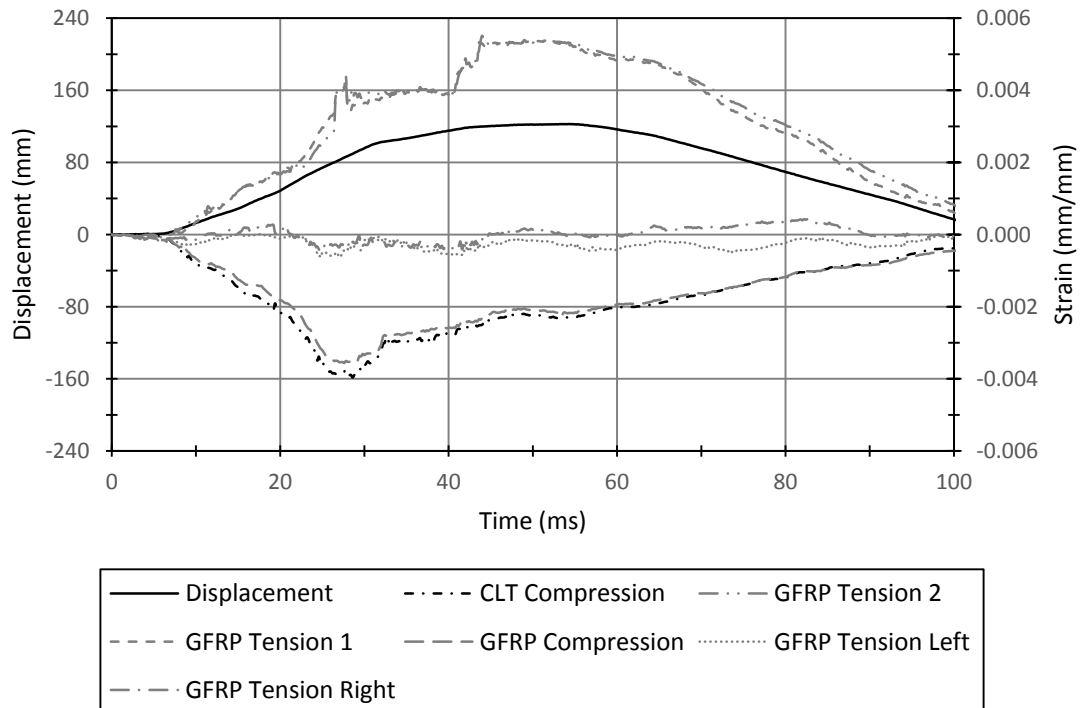


(b) Dynamic Resistance and Displacement

Figure E.16: Test Results for CLT3-2.D



(a) Dynamic Resistance and Strain



(b) Displacement and Strain

Figure E.17: Test Results for CLT3-D1 Continued



(a) Initial Position



(b) Maximum Dynamic Resistance

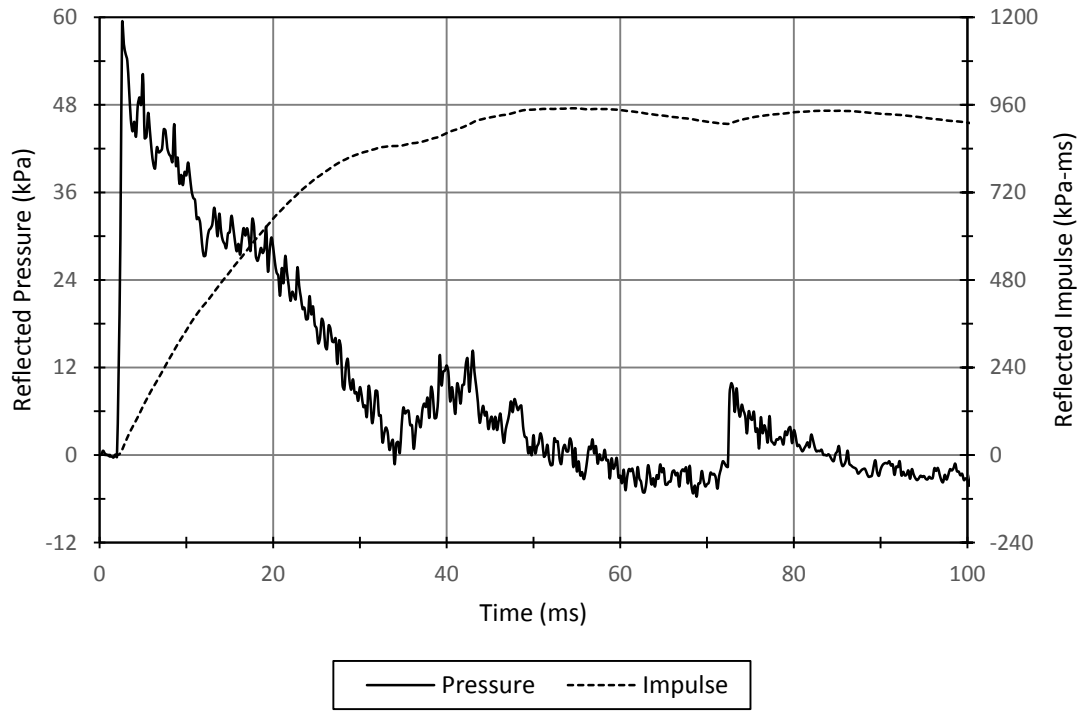


(c) Maximum Displacement

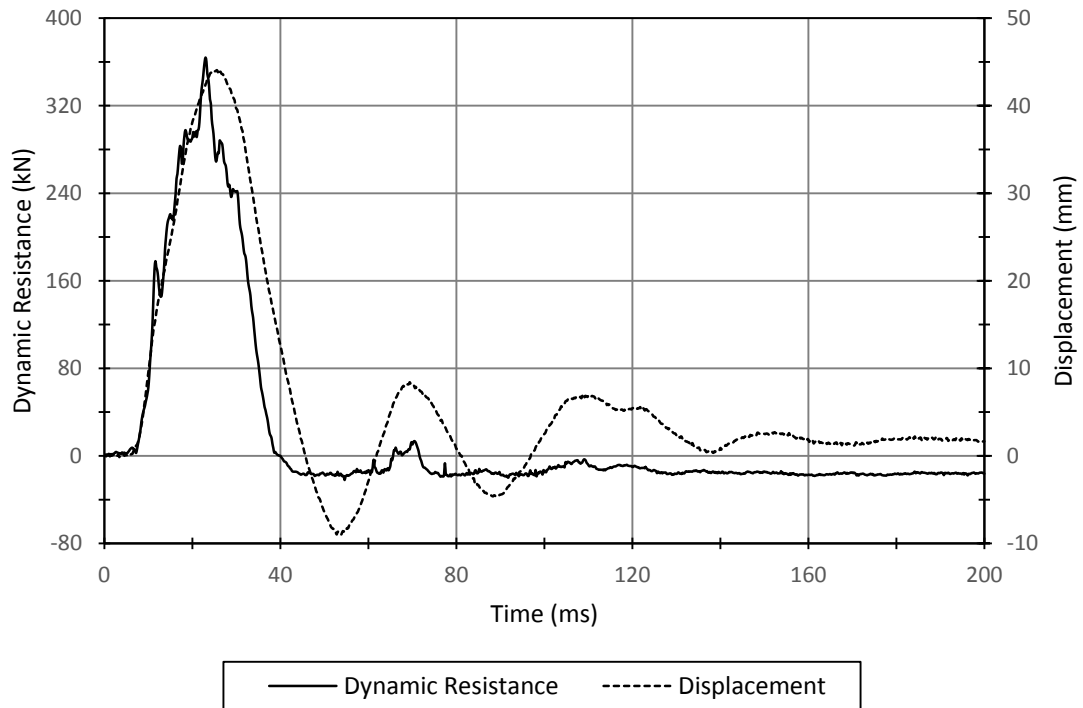
Figure E.18: Progression of Failure of CLT3-D1

CLT7-D2.1

Test Date:	March 21, 2017	Slab Mass:	106.9	<i>kg</i>
Specimen No.:	17	Volume:	0.1800000	<i>m³</i>
Test Name:	CLT7-D2.1	Density:	594.0	<i>kg/m³</i>
Shot No.:	1	Moisture Content:	11.0	%
Total Slab Height:		2,500		<i>mm</i>
Slab Clear Span:		2,235		<i>mm</i>
Configuration:	2 CM [0/90°] + 2 CM [± 45°]			
Approximate Area of GFRP Used:		11.95		<i>m²</i>
Driver Length:		4,572		<i>mm</i>
Driver Pressure:		483.3		<i>kPa</i>
Maximum Reflected Pressure:		59.2		<i>kPa</i>
Maximum Reflected Impulse:		950.7		<i>kPa – ms</i>
Positive Phase Duration:		33.4		<i>ms</i>
Maximum Mid-Span Deflection:		44.1		<i>mm</i>
Time to Mid-Span Deflection:		26.0		<i>ms</i>
Maximum Dynamic Resistance:		364.0		<i>kN</i>
Deflection at Dynamic Resistance:		42.5		<i>mm</i>

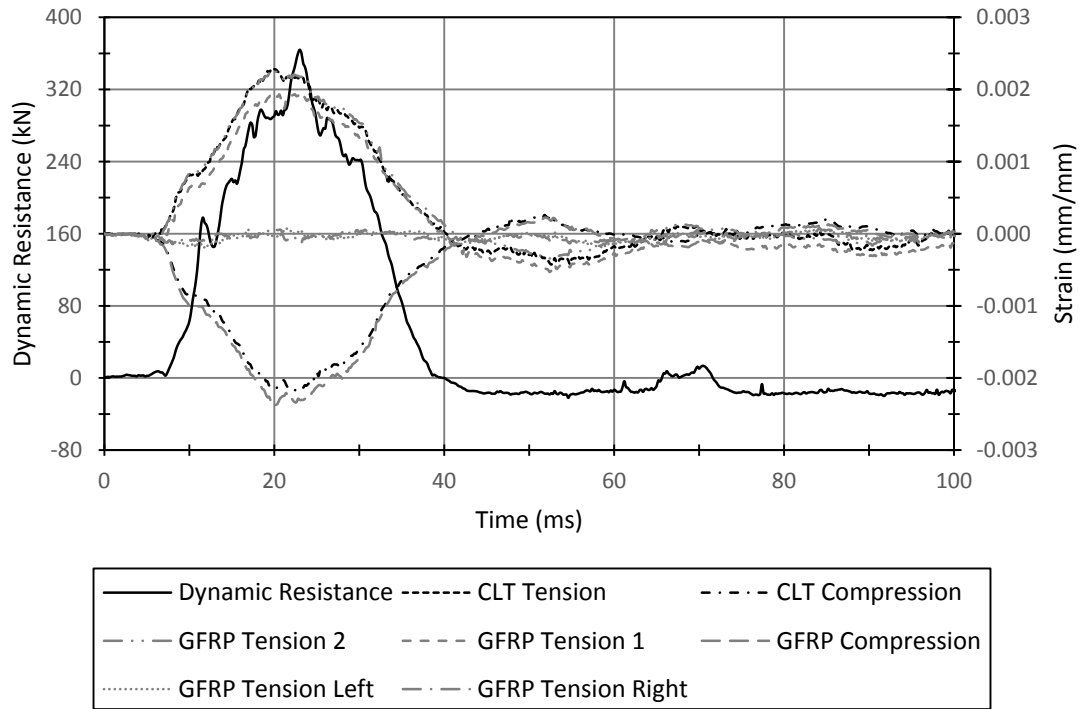


(a) Pressure and Impulse

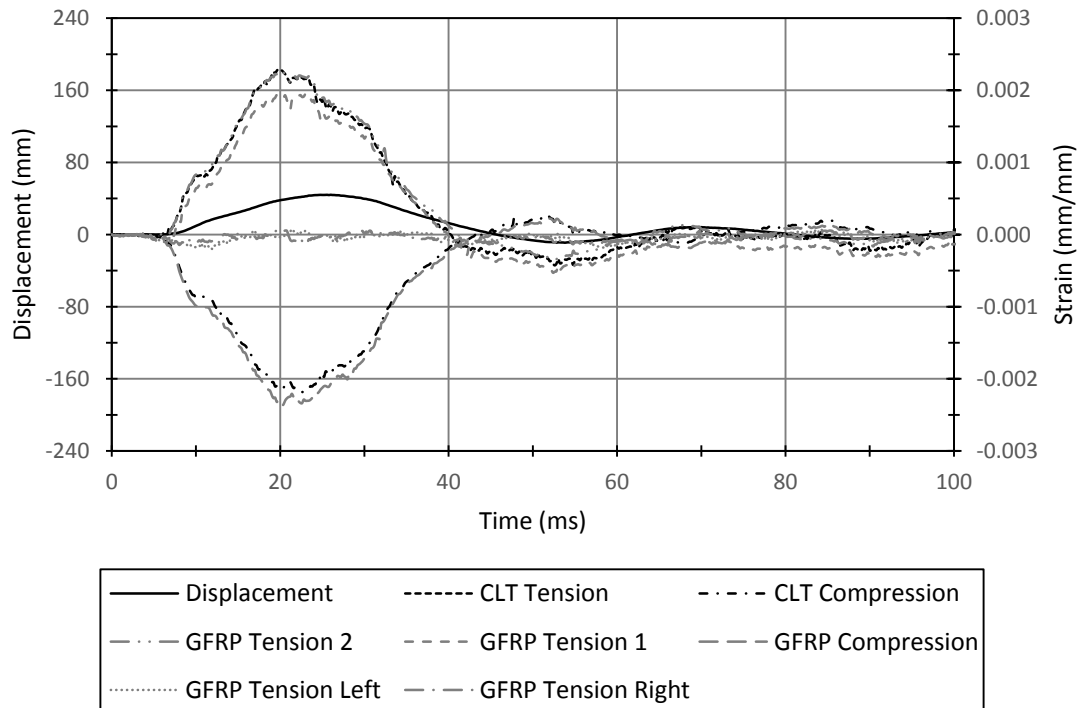


(b) Dynamic Resistance and Displacement

Figure E.19: Test Results for CLT7-D2.1



(a) Dynamic Resistance and Strain



(b) Displacement and Strain

Figure E.20: Test Results for CLT7-D2.1 Continued



(a) Initial Position



(b) Final Position

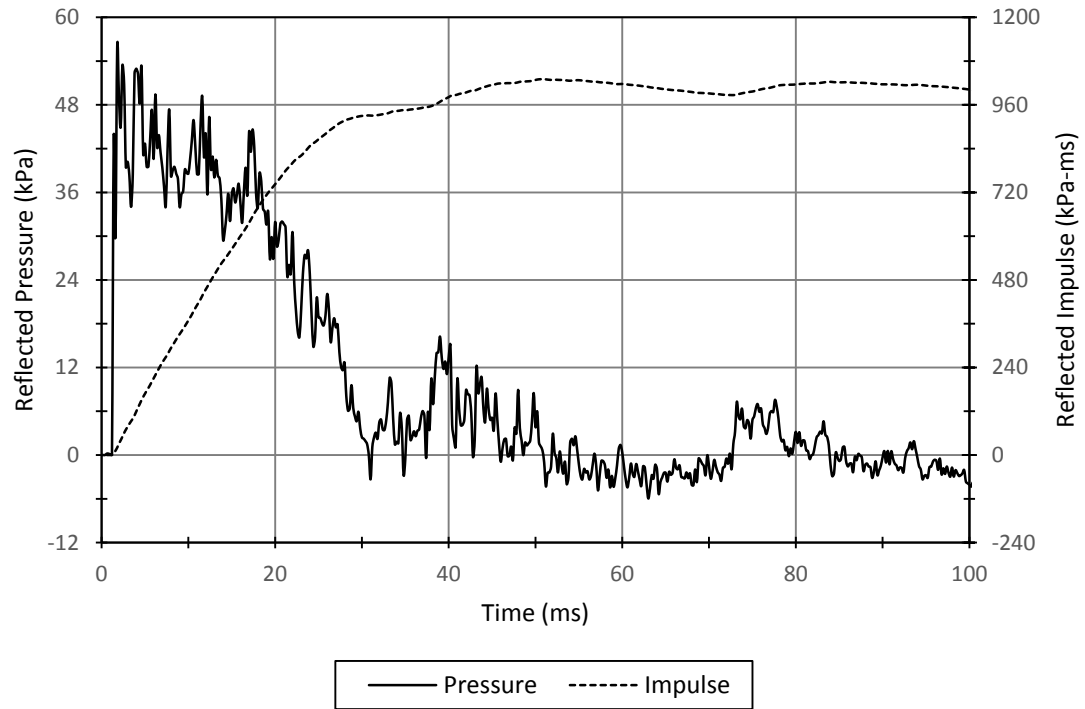


(c) Displaced Shape

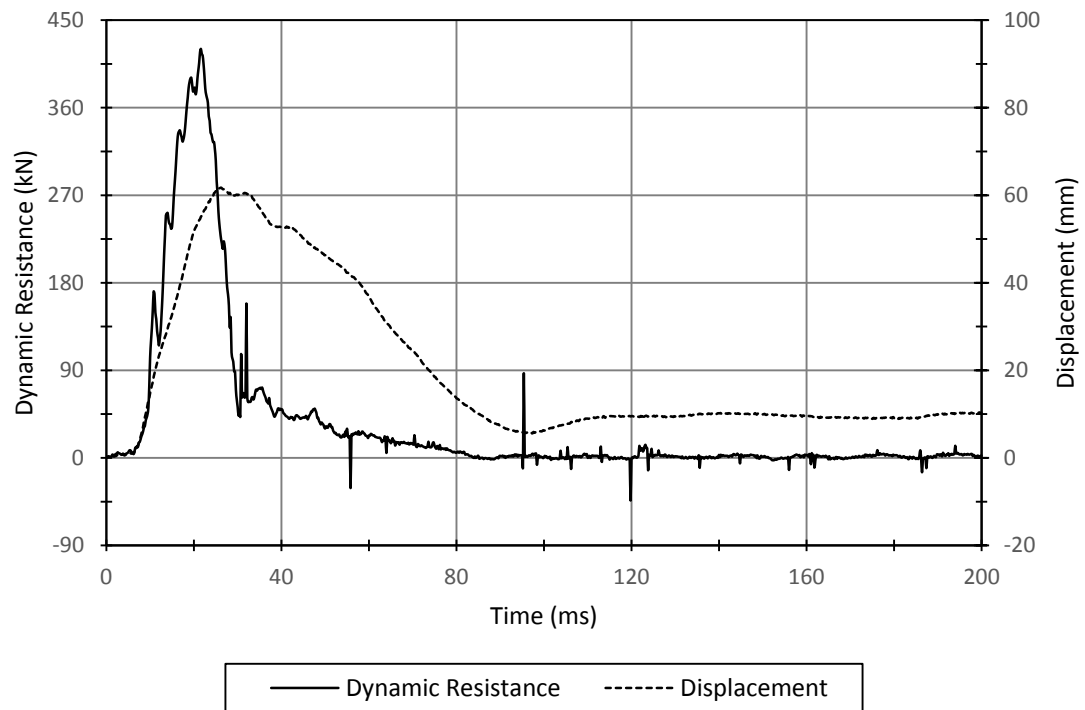
Figure E.21: Slab Movement of CLT7-D2.1

CLT7-D2.2

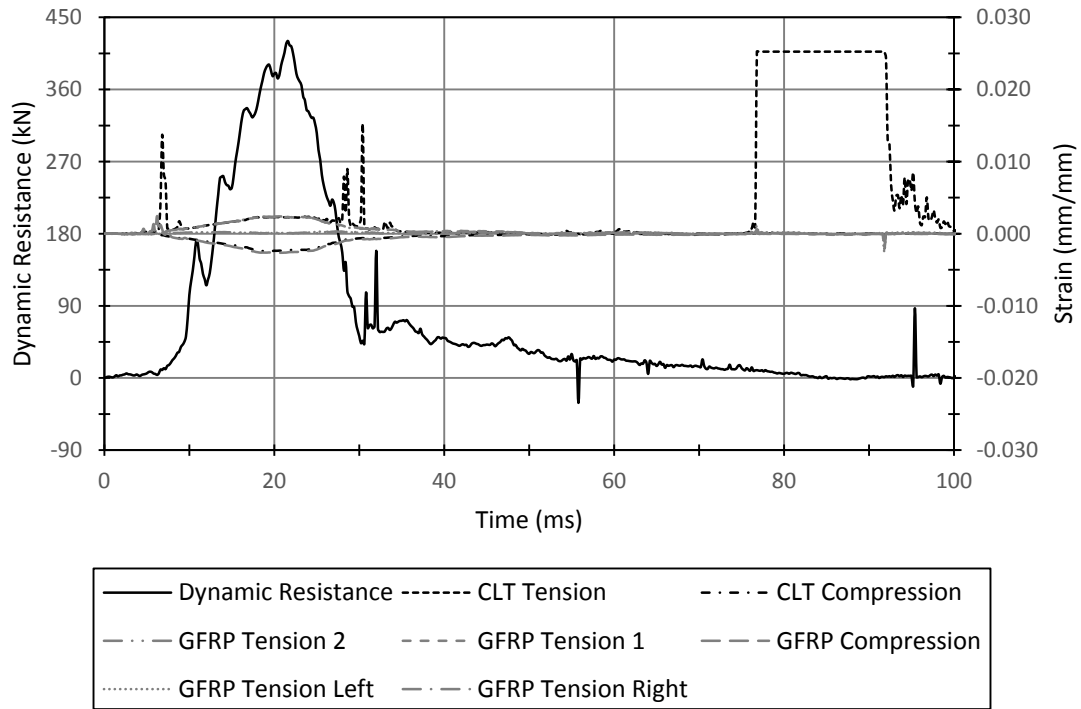
Test Date:	March 28, 2017	Slab Mass:	106.9	<i>kg</i>
Specimen No.:	17	Volume:	0.1800000	<i>m³</i>
Test Name:	CLT7-D2.2	Density:	594.0	<i>kg/m³</i>
Shot No.:	2	Moisture Content:	11.0	<i>%</i>
Total Slab Height:		2,500		<i>mm</i>
Slab Clear Span:		2,235		<i>mm</i>
Configuration:	2 CM [0/90°] + 2 CM [± 45°]			
Approximate Area of GFRP Used:		11.95		<i>m²</i>
Driver Length:		4,572		<i>mm</i>
Driver Pressure:		550.2		<i>kPa</i>
Maximum Reflected Pressure:		55.9		<i>kPa</i>
Maximum Reflected Impulse:		1,030.5		<i>kPa – ms</i>
Positive Phase Duration:		30.8		<i>ms</i>
Maximum Mid-Span Deflection:		61.7		<i>mm</i>
Time to Mid-Span Deflection:		26.2		<i>ms</i>
Maximum Dynamic Resistance:		420.5		<i>kN</i>
Deflection at Dynamic Resistance:		55.1		<i>mm</i>



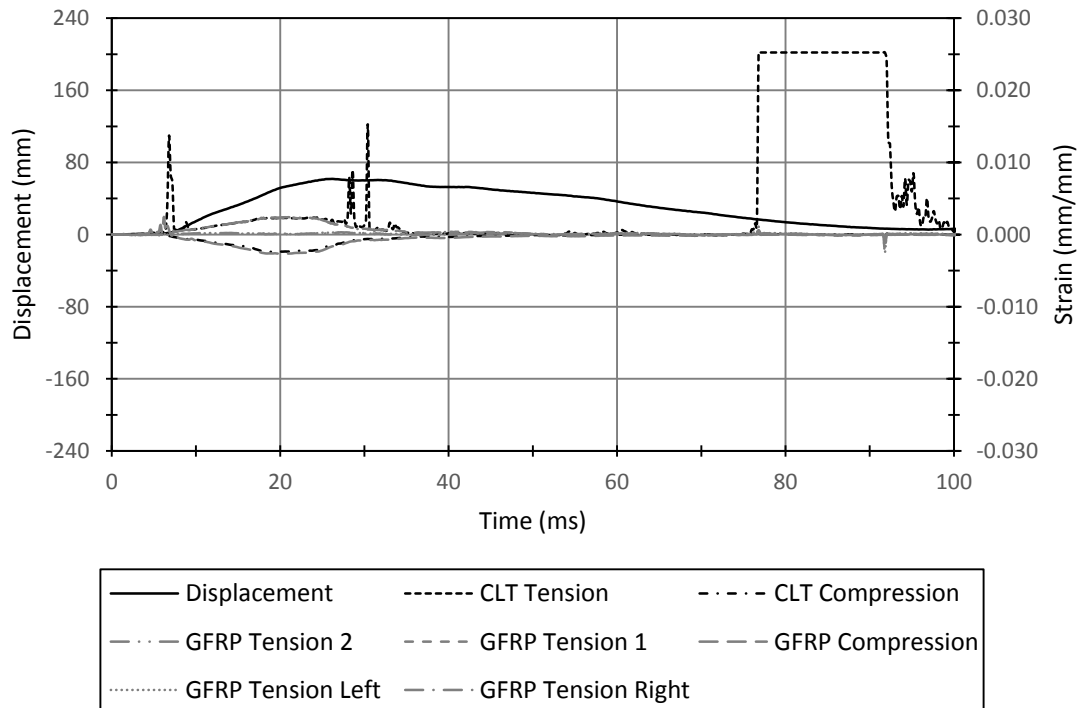
(a) Pressure and Impulse



(b) Dynamic Resistance and Displacement
Figure E.22 Test Results for CLT7-D2.2



(a) Dynamic Resistance and Strain



(b) Displacement and Strain

Figure E.23: Test Results for CLT7-D2.2 Continued



(a) Initial Position



(b) Maximum Displacement



(c) Final Position

Figure E.24: Progression of Failure of CLT7-D2.2