AN EXPERIMENTAL STUDY OF CLT BEAMS REINFORCED WITH STEEL BARS, COLD FORMED STEEL PLATE AND FRP

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ABSTRACT: Cross-laminated timber is a growing phenomenon and a recent building solution alternative in timber construction in different parts of the world by using massive or solid wood plates as roof, floor, and wall elements. Laminated timber has proven to be a strong material than the conventional wood. However, to use it for higher loads it still needs to be strong enough. Many tall all wooden buildings under construction or proposed for future will definitely require a much stronger timber to ensure safety and durability. Many researches have been carried out in the past and successfully presented, however being a comparatively new material; laminated timber has vast scope for further research and enhancements. Also, being a bio degradable component, this doesn’t pose any environmental threats and this definitely encourages us to work deeper in this area. Not only will timber prove to be an environment friendly material but also an easy one, with easy to use, easy to construct, repair and demolish. So, taking these points into consideration we aim to do our part to add something useful for others to consider. In this research a variation of the conventional CLT is considered by studying the performance of systems reinforced with cold formed and hot rolled steel plates bonded together with the timber using an epoxy resin under the brand name of Araldite®. This study also concerns investigations of beam strength of CLT elements reinforced with steel bars, the bars are placed in the grooves made in the bottom layer and bonded with the same adhesive. FRP reinforcement is also studied to determine its effect on the beam. The same epoxy is used in the bonding of FRP and timber. CLT is very relevant from a practical engineering point of view since the cross layers have a reinforcing effect with respect to stress perpendicular to the beam axis. Due to the general structure of cross laminated timber, the stress state is however very complex and there are many possible modes of failure. The specimens were designed, manufactured in-house, and tested under symmetric four-point bending (also known as third-point loadings). The bending strength is tested and compared with conventional timber beams which in our case are the solid timber beam of equal size and shape as of every other sample. Finally, standard test results are produced to predict the behavior of such beams and scope for further research.

Keywords; CLT, wooden structures, lumber, steel plates, box beams

I. INTRODUCTION
Structural engineering is a classical subject and our general knowledge about performance and capacity of load bearing structures in buildings and infrastructure is nowadays on an advanced level. Modern computer-based methods have made it possible to simulate the behaviour of both simple and advanced structures in a very sophisticated manner. Against this background it is surprising that the frequency of structural failures occurring in practice is still quite high. The truth is however that, structures should be designed to sustain extreme loads. Numerous investigations of structural failures occurring in practice have been performed during the years. Such investigations show convincingly that with few exceptions, structural failures are due to human errors and almost never a result of unfavourable combinations of random events. The present study is focused on studying the behaviour of timber strengthened using steel and fibre reinforced polymer. Timber has many advantages such as high strength to weight ratio, lightweight, material, easy to construct and move and are economic alternatives to concrete and steel. If compared to concrete, timber structures can be constructed easily because there is no formwork required. The actual erection is greatly simplified if all components can be moved by workers instead of heavy machinery. In civil construction, light weight is seldom sought as a design goal, although there are some examples to the contrary. Light weight may prove advantageous when transportation is a problem and where heavy machinery is not available to aid in assembly. Thus, the use of timber will lower the construction cost. In addition, construction time can be shortened because they do not require extra time for hardening which is needed for concrete structures. Due to its insulation from sound and electricity, as well as resistance to corrosion and oxidation, timber is also popular in light construction. However, timber also has disadvantages such as poor mechanical properties with a wide variation and low fire resistance. Although timbers are widely used in construction, their natural durability is often insufficient to ensure confidence to the people. Durability can be achieved by the appropriate selection and application of effective preservative treatments or by processes that modify the timber structure against insect and fungal attacks. The main concerns of using timber for structural members are strength, stiffness and durability. Softwood or softwood to medium hardwood can be strengthened using fibre reinforced polymer (FRP) to improve its mechanical properties. Since the 1930s, FRP was developed for aerospace, automotive and sports equipment industries (Fiorelli and Dias, 2002). But recently, it has become popular in the rehabilitation of civil engineering projects due to its high strength to weight ratio. Other benefits of FRP over conventional materials are high strength, lightweight, corrosion resistance, non-conducting, nonmagnetic characteristics and superior fatigue performance (Nagaraj and Gangara, 1997; Zureick and
Scott, 1997; Harris et al., 1998). These lightweight materials are easy to be used for the strengthening of new and existing structures. Moreover, FRP is also well known as an effective strengthening material with its good durability. Thus, with its advantages over the steel, FRP is believed to be able to substitute the role of steel as the latest strengthening material. The conventional steel can be a beneficial reinforcement as well besides the FRP owing to its lower cost than the FRP. Moreover, the steel is easily available and has many other desirable properties. Besides, FRP is not easily available in the local market, thus studying the reinforcing effects of different steel arrangements can prove handy.

In this research, the focus is on the determination of load carrying capacity and bending strength of the reinforced timber beams.

OBJECTIVES OF PRESENT STUDY
The main objectives of the present study are:

- To study the structural behaviour of timber beams under static loading condition.
- To determine whether strengthening timber beams would change the failure modes of the timber beams.
- To study the structural behaviour of CLT reinforced with different materials.
- To perform a comparative study of strengthened beams and to arrive at the most suitable type.

II. REVIEW OF LITERATURE
In India, particularly in Kashmir, there has not been any significant research on timber strengthening of timber beams and on the CLT. The cost of the strengthening material such as FRP is considered high because it is not produced by local companies. Unlike other countries, the FRP is a common material used for strengthening and although the price of material is quite high but it is still reasonable since the material can be produced in bulk. Strengthening can be used for new structures or repair works. The choice made over the method employed to repair a timber structure is influenced by a number of issues, including:

a) Location of the repair in the structure whether the repair is interior or exterior in the building.

b) The determined structural requirements of the member to be repaired or upgraded.

c) The fire resistance requirement of the repair.

d) Access to the site of the repair. (Wheeler and Hutchinson, 1998)

The structure, properties and characteristics of timber. Derived from the medieval timber trade, “hardwoods” and “softwoods” are well known terms, which have little association with the hardness of wood, Butterfield and Meylan (1980). Some hardwood species are softer than softwoods and the botanical definition is perhaps more comprehensible. Hardwood species (angiosperms) are separable from softwood species (gymnosperms) because the seeds borne by angiosperms are enclosed within the ovary of the flower whereas in contrast, gymnosperms bear seeds that are not enclosed in this way, Miller (1987). Although there are evident micro- and macro-structural differences between the wood from angiosperms and the wood from gymnosperms, the primary constituents and structural arrangements for both are rooted in composite science. The composite structure of wood. The role of a composite is to combine the properties of two or more materials to generate a material that exhibits conjoint properties from all of them. Common arrangements of composite materials include particulate reinforced composites, fibre reinforced composites and laminates. Wood consists of fibre reinforced composite and laminated composite arrangements, which together influence the macro-structural properties of sawn timber. At the molecular level, the three primary constituents of wood are cellulose, the hemicelluloses and lignin. The molecular level backbone of wood is highly ordered crystalline cellulose, a giant polysaccharide made up entirely of glucose units, which are connected by β linkages, Purves et al (1995). Lignin glues the cellulose and the hemicelluloses together to increase the overall mechanical strength of wood, Rowland (1992). The cellulose molecules are very stiff along the chain axis and are helically oriented. The lignin locks into place the cellulose molecules in the form of micro fibrils and effectively stiffens up the woody structure. The elastic modulus of cellulose according to Salmén (2001) is no higher than 134GPa. The structural material in wood at an ultra-structural level is essentially comprised of xylem. Xylem is generally made up of tracheid and ray cells. The tracheid dominates both numerically and volumetrically, and are laminated tubular structures, each lamina in each tracheid cell wall being made up of micro fibrils. The tracheid cell wall density is 1500kgm-3, Walker (1993) and the micro fibril orientation is different in each of the cell wall laminates relative to its neighbouring wall, Wardrop (1963). Figure, taken from Eaton and Hale (1993), shows a diagram representing the cell wall organisation of a mature tracheid. The outer wall represents the primary cell wall layer and surrounds the three secondary layers S1, S2 and S3. The orientation of the cellulose micro fibrils is represented in each layer by lines. The innermost warty layer is insignificant in its contribution to the physical properties of the tracheid, Bodig and Jayne (1993). The S2 layer is considered the micro fibril angle as its contribution to strength and stiffness is more significant than the S1 and S3 layers. Higher micro fibril angles result in lower values for the elastic modulus of wood, Mark (1967), as well as lower values for the tensile strength, Zhang and Zhong(1992).
Cell wall organization of a mature tracheid, from Eaton and Haley (1993).

Xylem cells differentiate from the vascular cambium. The rate of cell production from the cambium is seasonally dependent with faster and slower growth resulting in less dense and denser woody material respectively. The darker denser material is termed latewood and the lighter coloured lower density material is the early wood. The cellular transition from early wood to late wood is more gradual than the transition from latewood to early wood. Together, pairs of early wood and latewood bands represent the growth rings and the growth rings are in essence, macro-structural laminates that critically determine the mechanical properties of sawn timber. Discontinuities and abnormalities in growth rings do not have any adverse effects on the service quality of wood, Tsoumis (1968). Fast grown trees have larger growth rings than slow grown trees and contain a higher volume fraction of juvenile wood compared to mature wood. Juvenile wood has very few differences in chemical composition to mature wood; however, it has highly contrasting physical and mechanical properties. Juvenile wood absorbs more water, is less dense, is neither as stiff nor as strong as mature wood, has shorter tracheid, thinner cell walls and higher micro fibril angles, Bao et al (2001). The mechanical properties of fast grown trees are hence inferior to slow grown counterparts, Kretschmann and Bendtsen (1992). Experimental variability, with regard to the stiffness, is also greater in fast grown trees than in slow grown trees, Bengtsson (1999).

The cross-sectional macro-structure of a tree is summarised in Figure, which is taken from Dinwoodie (2000). The longitudinal axis is parallel to the grain and the radial and tangential directions are therefore perpendicular to the grain. It can be seen from Figure that there is a definite distinction between the central wood of a tree, the heartwood, and the wood closer to the bark, the sapwood. The heartwood is usually darker in colour and is highly impermeable to moisture relative to the neighbouring sapwood. Thus, heartwood is harder to treat than sapwood when liquid-based wood preservatives are used, however, the heartwood is inherently more resistant to decay and fungal attack, Hoffmeyer (1995).

Factors affecting properties and characteristics of wood.
The composite characteristics of timber are hindered by the presence of natural defects such as fungal decay, knots, cracks and fissures. Moreover, chemical, seasoning and conversion defects also hinder the efficiency of timber, Kermani (1999). Xylem vessels grow around knots and gross on-linearity results in the direction of the grain. This significantly reduces the strength and stiffness of timber sections parallel to the grain axis. Xu (2002) has reported on a decreasing parallel to grain stiffness for timber as a function of increasing volume fraction of knots. The decline of parallel-to-grain stiffness was also found to be more radical as the growth angle of the knots in the timber heights. This is a logical outcome, because the grain direction of the knot contributes less to stiffness as the grain of the knot deviates further away from the timber grain. According to Itagaki et al (2001), the timber itself, loaded in tension parallel to the grain axis, experiences high localised deformation near knots in a direction perpendicular to the grain and therefore, normal to the loading direction. The stiffness of wood decreases as a function of increasing distance from the centre of the tree, Bengtsson (1999). Fluctuations in the moisture content of timber sections can result in seasoning defects, which include warping, twisting and bowing. Sudden, major changes in humidity generate significant internal stresses in timber, Svensson and Toratti (2002). The shrinkage caused by moisture loss in the tangential direction of wood is higher than in the radial direction. Many woods have an approximate value of 2 for tangential to radial shrinkage. The shrinkage in the longitudinal direction is significantly lower than in the tangential direction. These differences in the magnitude of shrinkage for the tangential, radial and longitudinal directions are the prime cause of moisture related shape changes in wood, Skaar (1972). The position from which wood is cut from a tree determines the different moisture related shape changing effects, Figure.

Distortions caused by drying of various cross sections cut from different regions of a log, from Hoffmeyer (1995)
The strength and stiffness properties of timber are highly dependent upon the moisture content. DD ENV1995-1-1 (1994), part of Euro code 5 (EC5), has categorised moisture content within three service classes for timber sections. Service classes 1, 2 and 3 refer to moisture contents of less than 12%, 12% to 20% and above 20% respectively.
Decreasing the moisture content results in higher values for the elastic modulus of timber. However, the flexural rigidity, EI, is almost unaffected by a decreasing moisture content. This is a direct result of a reduced second moment of area, which is a function of cross-sectional shrinkage upon drying, Madsen (1992). Figures illustrate this phenomenon well.

Modulus of elasticity as a function of moisture content at different levels from a normal distribution, from Madsen (1992).

Rigidity, EI as a function of moisture content at different levels from a normal distribution, from Madsen (1992).

The strength characteristics in tension are less affected by moisture fluctuation than the strength characteristics in compression. By and large, the tensile strength of timber is unaffected at different moisture contents. However, the compression strength falls dramatically as a function of increasing moisture content, Madsen (1992). Larger volumes of timber contain more defects than smaller volumes and properties can vary according to material dimension. The strength in particular, of tensile loaded timber will decrease as a function of increasing length, Burger and Glos (1996).

The negative effect of defects and moisture variation can be countered by manufacturing structural timber composites, which reduce the inherent heterogeneity of timber members. This is often achieved by gluing and laminating timber and is commercially available as laminated veneer lumber (LVL) and glued laminated timber (glulam).

III. RESEARCH METHODOLOGY, MATERIAL PROPERTIES AND EXPERIMENTAL PROGRAM

Introduction
This chapter begins with the research methodology adopted in this research followed by a detailed description of various materials used and their relevant properties obtained through material testing. In the end sections of this chapter a detailed description of the experimental program is given.

Research methodology
The methodology adopted for the present research in compliance with the research objectives mentioned in the first chapter is outlined here in terms of a flow chart in Fig.3.

Flowchart

Methodology in detail

LITERATURE REVIEW
Before the commencement of any practical work a detailed review of literature on strengthening CLT beams was carried out. A Comprehensive study of various strengthening techniques and strengthening materials was done in order to arrive at the present research. Any previous work on strengthening was thoroughly reviewed.

COLLECTION AND FABRICATION OF MATERIAL
This step involved procurement of the materials required for construction and strengthening of beam samples viz.
- Timber
- steel sheets
TESTING OF MATERIALS
The materials were tested in order to authenticate the quality of material to be used for casting. The tests involve physical tests for timber, reinforcing steel & cold steel.

FABRICATION OF BEAMS
A total of seven timber beams were prepared under similar environmental conditions. The description of each beam is given in detail in the later sections of this chapter.

TESTING OF BEAMS
Each beam was tested to failure under four-point loading. The beams were made to rest on bearing pads of width 10 cm at the ends which in turn were rested on supports. This was done to simulate the simply supported conditions as closely as possible.

TEST RESULTS AND DISCUSSION
The test results obtained from testing of unstrengthened and strengthened beams are presented in terms of various parameters such as ultimate load, cracking load, load deflection behaviour. Besides many indirect results were also calculated from the directly observable parameters such as strength to weight ratios, ductility factors etc.

INTERPRETATION OF RESULTS
The results obtained directly and indirectly through testing are interpreted and the effect of various strengthening parameters on the behaviours of strengthened beams is illustrated. A comparative analysis is done to reflect the improvement in the behaviours of beams by adopting different strengthening materials.

CONCLUSION AND FUTURE SCOPE
This involves brief overview of the data analysis and interpretation. This chapter rounds up the whole project and also contains the highlights of the project. Finally, the recommendations for future research in the present research area are included.

IV. TEST RESULTS AND DISCUSSIONS
Introduction
The objective of the experimental research work is to study the effect of reinforcements on the CLT beams. All the results obtained from the experimental works i.e. the two-point bending test for all beams are discussed in this chapter. The results from laboratory testing for the strengthened beams are compared with control beam (un-strengthened) in order to study the behaviour of strengthened timber beams in terms of load carrying capacity and bending strength. In addition, the modes of failure for all beams have been studied and discussed. All the data obtained are shown in graphs, tables and photos, whichever is suitable to ensure a better and clearer understanding of the test results. The results obtained from the experimental investigation of beams are expressed in terms of the following:
- Cracking load (load at first crack) of the beam.
- Crack pattern of the beam.
- Failure mode of the beam.
- Ultimate load
- Load vs deflection behaviour of the beam

Results of control beam (CB)
Control timber beams were tested for loading and unloading process under two-point loading. This is to determine the initial bending stiffness whilst in the elastic zone. The aim is to determine whether the unstrengthened beam would exhibit an acceptable consistent stiffness (no significant variation in stiffness). The Control beam started to behave non-linearly when the load was 16.6kN and the corresponding deflection was 12.5 mm. The failure load was 25.65KN for Control Beam respectively. Any non-linearity in the behaviour of timber beam was due to plasticity in the compression zone Figure shows load versus deflection at mid-span of two control beams.

![Figure 4.1 Load vs deflection curve for control beam](image1)

Figure 4.1 Load vs deflection curve for control beam

The beam developed small vertical cracks which propagated upwards and turned horizontal near the mid layer. The crack widened and remained horizontal until final failure thus the beam primarily failed in horizontal shear failure.

![Figure 4.2 Cracked control beam](image2)

Figure 4.2 Cracked control beam

Results of cross laminated beam (CLT 0)
The cross laminated beam showed increase in the load carrying capacity. The ultimate failure load was recorded as 28.98 KN at which the ultimate deflection was 14.74 mm. This beam showed less non-linearity as compared to the
control beam. The load deflection curve is shown below.

![Load deflection curve of CLT 0](image)

Figure 4.3 Load deflection curve of CLT 0
The beam failed developed shear cracks near the ends which propagated towards centre. The cracks were diagonal when the beam failed at the end. Thus, the failure was assumed to be cross grain tensile failure.

![Failed CLT beam](image)

Figure 4.4 Failed CLT beam.

Results of beam reinforced with 8mm diameter bars (CLT 1)
The beam started to show non linear behaviour at a load of 14.94 KN corresponding to a deflection of 8.25mm. At a load of 30.71 KN partial crushing occurred at the top of the compression zone. With further increase in the loading tensile cracks started to appear in the bottom layer which propagated upwards and caused a brittle tensile failure at the ultimate load of 33.05 KN when the deflection was 19.4mm. The load deflection curve recorded for the beam is given below:

![Load deflection curve of CLT 1](image)

Load deflection curve of CLT 1
Although the bars increased the strength but the cracks also tend to propagate along the reinforcement line. In this case when the tensile cracks developed in the bottom layer, the beam lost much of its strength as they started to widen up. The behaviour was nearly linear until failure and any non-linearity was assumed to be due to presence of some internal hair cracks.

![Failed CLT 1 Beam](image)

Failed CLT 1 Beam

Results of beam reinforced with CFS (CLT 2)
The beam behaved strong and showed enhanced strength. Small cracks initially developed but the beam was still showing enough capacity. The ultimate load recorded was 43.76 KN and the corresponding ultimate deflection was 36.7mm
The load deflection curve is shown:

![Load deflection curve of CLT 2](image)

Load deflection curve of CLT 2
The deflection was initially higher which later started decreasing. Initially the behaviour was almost same as that of the hot rolled reinforced one but in this case the crushing in the compression zone was significant. The beam showed tensile cracks but the crushing as predominant during the later stages. However, bottom layer tensile cracks and top layer crushing were both significant and it wasn’t clear which the prime cause of the failure was.
As the beam started failing in crushing, the tensile cracks also widened up and caused bottom layer to break diagonally. Although it was easy to assume crushing failure because of large deflections and the crushed loading
locations at the top but still it was doubtful.

Results of beam reinforced with hot rolled steel plate (CLT 3)
The beam reinforced with hot rolled plate showed enhanced ductility. The ultimate load observed in this type was 41.6 KN corresponding to a deflection of 15.5mm. The load deflection curve is shown as;

Load deflection curve of CLT 3
The beam showed a linear behaviour for long. The cracks started to appear at the bottom of the lower lamination. The crack widened and started to propagate upwards however the plate didn’t allow going through. The crack took the form of a diagonal crack and caused tensile failure. Although small cracks also developed in the upper layers but the beam lower layer caused the failure of the beam. It appeared as though the beam failed in the bonding and after developing the crack, the bottom layer behaved individually. The bond however was intact but still the failure was some sort of individual with bottom layer behaving individually and thus causing the failure.

Results of beam reinforced hot rolled steel plate (CLT 3)
The beam reinforced with hot rolled plate showed enhanced ductility. The ultimate load observed in this type was 41.6 KN corresponding to a deflection of 15.5mm.

Load deflection curve of CLT 4 beam
The beam showed enhanced behaviour and strength, initially the cracks developed in the lower layer however the FRP plates prevented their propagation to the above layers. Even though the cracks developed, the beam behaved well. With increasing the load some popping sounds were heard which possibly could be the initiation of bond failure however, once the ultimate load reached the FRP gave some cracking sounds and the beam failed with a vertical crack developed in the lower layer, which propagated vertically up. The failure was thus considered to be bash tensile failure.

Results of beam reinforced with 5mm thick FRP (CLT 4)
The beam showed considerable increase in the load carrying capacity. The ultimate load recorded was 48.5KN and the ultimate deflection was 14.2mm prior to the failure. The load deflection curve is shown below

Load deflection curve of CLT 4 beam
The beam showed enhanced behaviour and strength, initially the cracks developed in the lower layer however the FRP plates prevented their propagation to the above layers. Even though the cracks developed, the beam behaved well. With increasing the load some popping sounds were heard which possibly could be the initiation of bond failure however, once the ultimate load reached the FRP gave some cracking sounds and the beam failed with a vertical crack developed in the lower layer, which propagated vertically up. The failure was thus considered to be bash tensile failure.

Results of beam reinforced with 1mm thick CFRP (CLT 5)
The beam reinforced with 1mm CFRP at the two ends showed increase in the strength than the CLT beam and particularly avoided shear failure. The cracks however started to appear in the diagonal mode and propagated towards centre.

The ultimate failure load was recorded as 33.35 KN corresponding to the deflection of 28.9mm. The load deflection curve is shown as;

Load deflection curve of CLT 5 beam
The beam showed a linear behaviour for long. The cracks started to appear at the bottom of the lower lamination. The crack widened and started to propagate upwards however the plate didn’t allow going through. The crack took the form of a diagonal crack and caused tensile failure. Although small cracks also developed in the upper layers but the beam lower layer caused the failure of the beam. It appeared as though the beam failed in the bonding and after developing the crack, the bottom layer behaved individually. The bond however was intact but still the failure was some sort of individual with bottom layer behaving individually and thus causing the failure.
Deflection curve of CLT 5 beam
The beam resisted shear well enough and restricted the development of shear cracks. The cracks developed near the central region and propagated diagonally. A crack in the bottom lamination, however appeared at the centre region and propagated vertically upwards and ultimately led to tensile failure.

Comparison of various parameters.
Percentage increase in ultimate load carrying capacity.
Showing percentage increase in ultimate loads.

<table>
<thead>
<tr>
<th>BEAM</th>
<th>Ultimate load (KN)</th>
<th>Percentage increase</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control beam</td>
<td>25.65</td>
<td>-</td>
</tr>
<tr>
<td>CLT 0</td>
<td>28.98</td>
<td>12.98%</td>
</tr>
<tr>
<td>CLT 1</td>
<td>33.05</td>
<td>28.85%</td>
</tr>
<tr>
<td>CLT 2</td>
<td>43.76</td>
<td>70.6%</td>
</tr>
<tr>
<td>CLT 3</td>
<td>41.6</td>
<td>62.18%</td>
</tr>
<tr>
<td>CLT 4</td>
<td>48.5</td>
<td>89.08%</td>
</tr>
<tr>
<td>CLT 5</td>
<td>33.35</td>
<td>30.02%</td>
</tr>
</tbody>
</table>

Showing CLT 5 beam
The control beam was not reinforced. It should be noted that the flexural strength determined from the control beam is a representative of the unreinforced strength of all beams. This comparison is valid for the wood specie chosen in the project only and for species within the group. For other timber groups, research needs to be carried out to study their behaviour when it is strengthened.

In general, all control beams and strengthened beams behaved linearly elastic initially and as the load increased, the flexural cracks increased in number, width, and depth, and the beams tend to behave non-linearly until failure. However the propagation of cracks was different in differently reinforced samples. There are some slight drops observed in some of the curves as a result of internal cracking of the beam. All figures clearly indicate that all the strengthened beams have higher ultimate load carrying capacity than control beam. When there is significant failure, the corresponding load will be taken as ultimate load carrying capacity. Also the reinforced beams experienced lower deflection than the un-strengthened control beam at the same load level. This low deflection phenomenon is desirable in the aspect of serviceability limit state in design to ensure comfort ability of timber structures. In timber design, generally the deflection will govern the design since timber shows large deflections as compared to other structural materials like steel and concrete. For all beams the deflection was either very near or exceeding the serviceability limits although the load carrying capacity was increased but the deflection was not as much improved.
Percentage increase in maximum mid span deflection.

<table>
<thead>
<tr>
<th>BEAMS</th>
<th>Maximum mid span deflection</th>
<th>Percentage increase</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control beam</td>
<td>12.5</td>
<td>-</td>
</tr>
<tr>
<td>CLT 0</td>
<td>14.74</td>
<td>17.92%</td>
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<tr>
<td>CLT 1</td>
<td>19.4</td>
<td>55.2%</td>
</tr>
<tr>
<td>CLT 2</td>
<td>36.7</td>
<td>193.6%</td>
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<tr>
<td>CLT 3</td>
<td>15.5</td>
<td>24%</td>
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<tr>
<td>CLT 4</td>
<td>14.2</td>
<td>13.6%</td>
</tr>
<tr>
<td>CLT 5</td>
<td>28.9</td>
<td>131.2%</td>
</tr>
</tbody>
</table>

IV. CONCLUSION

Conclusion

Based on the experimental and theoretical results of strengthened beams the following conclusions are drawn:

- All the strengthened beams exhibited linearly elastic behavior in the first stage followed by non-linear in a short period before the beams failed.
- The strengthened beams showed an increase in cracking load, ultimate load and ultimate deflection capacity compared to the control beams.
- An increase in deflection capacity was achieved by this strengthening procedure however each reinforcement showed different deflection.
- All beams in this study did not fail due to debonding.
- CFRP is advised for maximizing the strength whilst minimizing the density. Mild steel rods are advised for maximizing the strength and stiffness whilst minimizing the cost.
- The reinforcement-adhesive elements act as barriers to crack propagation across the timber.
- Cracks prefer to travel around the reinforcement-adhesive inclusions rather than through them.
- Adhesives are advisable for obtaining higher strength and stiffness properties but are a slower joining technique because they spread throughout the surface and thus create a continuous bonding surface.
- Adhesive bonding coherently bonds the steel plate to the LVL and hence the steel plate fully contributes to the stiffness and moreover the adhesive itself contributes to certain strength as well.
- Adhesively bonding the composite elements together introduces no defects into the LVL and hence the opportunity for cracks to propagate between introduced defects is not an issue.
- All timbers have natural defects but the level of seriousness is different. It is almost impossible to get a perfect timber beam without any defects. Thus the factor of safety for any timber design should always be higher.
- By the experimentation it was clear that the strength increased but the study was restricted to the experimental work and of particular timber specie. Although timber, it is very much difficult to consider the same behaviour for another specie considering the complex behaviour and structure of wood.

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