Overstrength of large-scale dowelled connections in CLT

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ABSTRACT: This paper presents an evaluation of overstrength based on an experimental study on large-scale dowelled connections in Cross Laminated Timber (CLT). In order to avoid brittle failure and ensure that ductile system behaviour and energy dissipation can be achieved under seismic loading, the overstrength of specified ductile components needs to be well understood. In timber structures, ductility is often achieved through plastic deformation of steel fasteners in connections.

Overstrength is generally defined as the difference between the analytical strength, based on design-codes using characteristic material strengths, and the 95th percentile of the true strength distribution. Two main contributing factors to overstrength are the conservatism of analytical strength predictions, and the overstrength due to variability of material property distribution. In dowelled connections, further overstrength can be introduced if the yield strength of the supplied fasteners exceeds the yield strength of the specified grade. This is often not picked up during supply as the erroneous assumption is made that the stronger material performs better, and therefore acting in the best interest of the client. While this assumption is generally true for most non-seismic load cases, it can cause problems in capacity design as it introduces unexpected overstrength that is rarely accounted for.

This paper evaluates the individual contributing factors of overstrength and compares experimental findings to theoretical considerations based on previous studies. It was found that unexpected steel fastener overstrength can contribute significantly to overall connection overstrength. However, the previously derived theoretical overstrength factor of 1.68 was safe in all cases.

1 INTRODUCTION

Overstrength is commonly defined as the difference between the calculated design strength in code provisions, F_d, and the 95th percentile of the true strength distribution, $F_{0.95}$. Capacity design ensures that ductility is achieved by protecting the brittle elements from the ductile elements' overstrength as shown in Figure 1. Traditionally, overstrength factors were obtained from experimental testing as $\gamma_{Rd} = F_{max} / F_d$ (Popovski et al. 2002). However, overstrength can be broken down to its contributing factors (Jorissen and Fragiacomo 2011):

$$\gamma_{Rd} = \gamma_M \cdot \gamma_{an} \cdot \gamma_{0.95} = \frac{F_k}{F_d} \frac{F_{0.05}}{F_k} \frac{F_{0.95}}{F_{0.05}} \quad (1)$$

where γ_M is the material safety factor (1.3 in Eurocode 5 / EN 1995-1-1 2004/2008, 1.25 in NZS3603 1993), γ_{an} stems from conservatism in analytical models, and $\gamma_{0.95}$ is the difference between the 5th and 95th percentile of the strength distribution (Ottenhaus et al. 2016a). To simplify calculations, γ_M is set equal to 1.0 in the following considerations.

Mitchell et al. 2003 identified the individual overstrength components for seismic force resisting systems (SFRSs) such as shear walls and moment-resisting frames. However, if connections are not properly detailed, it is possible the brittle failure occurs within an SFRS. Therefore, the recent trend is

to identify overstrength at a connection level as shown in Table 1.

Table 1. Sources of overstrength in timber connections (* due to mode cross-over).

authors	connection	γ_{an} = $F_{0.05}/F_k$	$\gamma_{0.95}$ = $F_{0.95}/F_{0.05}$	$\gamma_{Rd1} = \gamma_{an}\,\gamma_{0.95}$ = $F_{0.95}/F_k$ (1) $\gamma_{Rd2} = F_{max}/F_k$ (2)
Gavric et al. 2014	nailed hold-downs, CLT	1.3-2.8	1.16-1.44	(1) 1.5-4.03
Vogt et al. 2014	nailed hold-downs, CLT	1.33	1.28	(1) 1.70
Schick et al. 2013	nailed hold-downs, CLT	0.99-1.83	1.07-1.35	(1) 1.06-2.47
Popovski et al. 2002	bolted connections, ext. steel plates, Glulam			(2) 1.52-1.95
Ottenhaus et al. 2016a	dowelled connections, internal steel plate, CLT	0.78-0.98*	1.10-1.46	(1) 0.86-1.43 (2) 0.71*-1.39
Ottenhaus et al. 2016b	nailed connections, ext. steel plates, CLT			(2) 0.73-1.31
Ottenhaus et al. 2017a	dowelled connections, int. steel plate, CLT	1.00-1.06	1.17-1.59	(1) 1.17-1.68

For many parts of the world, design is typically based on gravity, snow and wind load cases. In those cases conservatism in design codes is desirable. While the European Yield Model (EYM) itself provides good predictions for connections using dowel-type fasteners, Eurocode 5 (2004/2008) also adopted M_{eff} to account the fact that full plasticisation in dowels is difficult to achieve and to encourage designers to use small-diameter fasteners (Blaß et al. 2001). However, M_{eff} penalizes large-diameter and high-strength fasteners (Blaß et al. 2011, Sandhaas and van de Kuilen 2017) and the lack of full plasticisation in large-diameter fasteners is owed to mode cross-over (Ottenhaus et al. 2017c). Therefore, it is recommended to use $M_{y,p}$ to avoid excessive analytical overstrength (Schick et al. 2016, Ottenhaus et al. 2017a).

There also seems to be a common assumption amongst suppliers that it is acceptable to deliver stronger materials than specified as these perform "better". However, this can cause issues if capacity design is used to ensure seismic safety. Delivery of too strong fasteners was reported by Misconel et al. (2016), and Sandhaas and van de Kuilen (2016). This was also an issue in the given case of large-scale connection testing. Therefore, this paper focusses on overstrength caused by material variability, $\gamma_{0.95}$, and the impact of "unexpected" material overstrength.

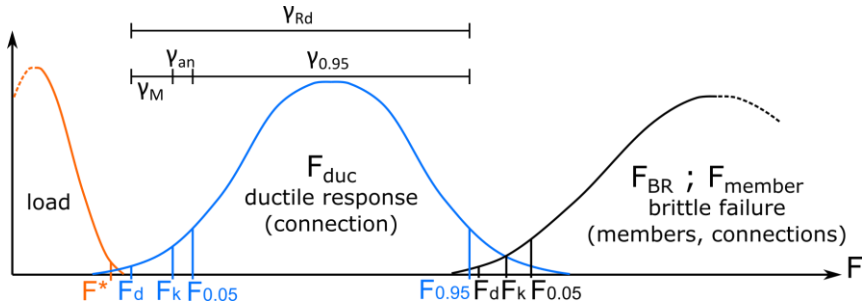

Figure 1. Concept of overstrength (modified from Jorissen and Fragiacomo 2011).

2 THEORETICAL CONSIDERATIONS

In order to understand connection overstrength, it is important to consider how predicted connection strength is calculated in design codes. In the given case the connection strength prediction was based on the EYM as given in Eurocode 5 (2004/2008), material strength values obtained from small-scale connection testing for timber (Ottenhaus et al. 2016a), and specified material grades for steel.

2.1 European Yield Model Equations

The EYM in Eurocode 5 (2004/2008) reads as follows for connections with an internal steel plate per fastener per shear plane (Figure 2):

$$F_{EYM} = \min\begin{cases} t_1 d f_h & Ia \\ 0.5 t_2 d f_h & Ib \\ t_1 d f_h \left(\sqrt{2 + \frac{4M_y}{f_h d t_1^2}} - 1 \right) + \frac{F_{ax}}{4} & II \\ 2.3\sqrt{M_y f_h d} + \frac{F_{ax}}{4} & III \end{cases} \quad (2)$$

Where t_1 is the side member thickness, t_2 is the plate thickness, d is the dowel diameter, f_h is the embedment strength, M_y is the fastener yield moment and $F_{ax}/4$ is the contribution of the rope effect (25% of the Johansen part for bolts).

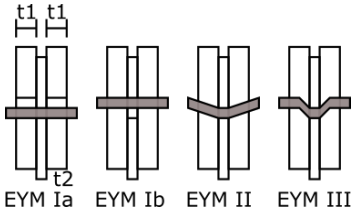

Figure 2. European Yield Model (Mode Ia, Mode Ib, Mode II, Mode III).

In experimental testing, a distinction is made between the yield point, F_y, maximum load, F_{max}, and the ultimate load, $F_u = 0.8\ F_{max}$ for ductile behaviour, and $F_u = F_{max}$ for brittle failure, as shown in Figure 3. The predicted characteristic ultimate load, $F_{u,k,pred}$, is obtained by inserting the characteristic ultimate yield moment, $M_{y,u,k}$, and characteristic ultimate embedment strength, $f_{h,u,k}$, into Equation 2. The characteristic ultimate dowel yield moment is defined as:

$$M_{y,u,k} = \frac{d^3}{6} f_{y,k} \quad (3)$$

Where d is the dowel diameter and $f_{y,k}$ characteristic yield strength of steel. The definition of $f_{h,u,k}$ for different types of timber products and load-to-grain orientations is given in the next section.

The overall connection overstrength can then be calculated as:

$$\gamma_{Rd} = F_{\max,\exp} / F_{u,k,pred} \quad (4)$$

With $F_{max,exp}$ being the maximum strength measured in experiments, and $F_{u,k,pred}$ being the predicted characteristic ultimate connection strength.

The yield point, $F_{y,k,pred}$, was predicted by inserting the elastic yield moment, $M_{y,y,k}$, and characteristic yield embedment strength, $f_{h,y,k}$, into Equation 2:

$$M_{y,y,k} = \frac{\pi d^3}{32} f_{y,k} \quad (5)$$

$$f_{h,y,k} = 0.8 f_{h,u,k} \quad (6)$$

The predicted yield point can be compared to the yield point obtained from experiments according to EN 12512 (2013). Knowledge of F_y and F_u and the respective displacements, Δ_y and Δ_u is important to make predictions about connection ductility (Novis et al. 2016, Ottenhaus et al. 2017c).

Figure 4 illustrates the concept of overstrength on a load-displacement curve. From previous testing it was found that the ratio between F_y and F_{max} ranged from 0.7 to 0.8 for dowelled connections which is similar to the material safety factors used in NZS3606 (1993) and Eurocode 5 (2004/2008).

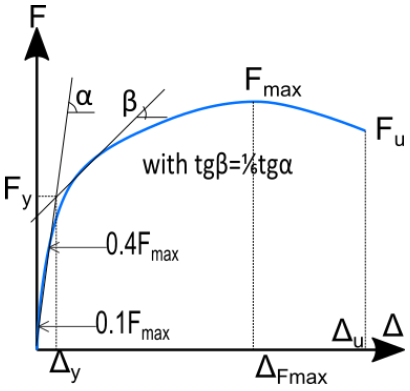

Figure 3. Yield point definition (EN 12512).

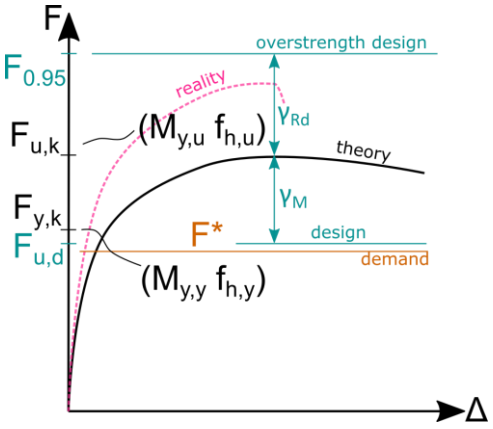

Figure 4. Applied overstrength concept.

3 MATERIAL PROPERTY TESTING

3.1 CLT embedment overstrength

Embedment overstrength from analytical design equations, $\gamma_{an,fh}$, is relatively small since the embedment strength formulas are mostly calibrated using experimental data (Ottenhaus et al. 2017a). Embedment overstrength is thus mainly introduced by the variability of the material strength distribution. As embedment strength is directly related to density, $\gamma_{0.95,fh}$ can be calculated from the density distribution using the embedment strength formula given in Equation 7 (CLT Handbook 2011; Uibel and Blaß 2014).

$$f_{h,u,k,CLT} = 0.031(1-0.015d)\rho_k^{1.16} \quad (7)$$

For New Zealand CLT made out of *radiata pine*, the CLT supplier reported the 5th percentile of the timber density distribution as $\rho_{0.05}$ = 402kg/m^3 and 95th percentile as $\rho_{0.95}$ = 608kg/m^3, respectively at 12% moisture content. Based on this density distribution, $\gamma_{0.95,fh}$ was calculated for the different EYM modes with Equation 8. The results for different dowel diameters, d, side member thicknesses, t_1, and steel yield strengths, f_y, are shown in Figure 5.

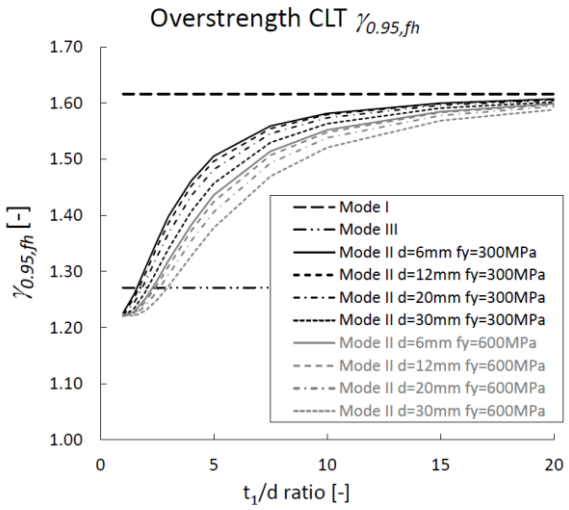

Figure 5. Embedment overstrength in New Zealand CLT from density distribution $\gamma_{0.95,fh}$.

It was found that $\gamma_{0.95,fh}$ ranges from 1.27 to 1.62 depending on the governing EYM mode. As the sample size used by the CLT supplier is larger than the sample size used in the previous study (Ottenhaus et al. 2017a), $\gamma_{0.95,fh}$ = 1.62 is a more conservative upper bound than the previously reported factor of 1.38. However, it should be noted that 1.62 applies for Mode I which is purely embedment failure and should be avoided in ductile design of connections subjected to seismic loading. For $t_1 \leq 10d$, the overstrength factor decreases to 1.60 for Mode II, and 1.27 for Mode III, respectively.

Furthermore, increased density and thereby increased embedment strength is positively correlated with other timber properties, such as bending, shear, or tensile strength (Cown and Hutchison 1983, Burdon et al. 2001). Therefore, overstrength in embedment strength is often less critical as it also increases the strength of brittle failure modes in connections.

$$\gamma_{0.95,fh,CLT} = \frac{F_{EYM\ fh,0.95}}{F_{EYM\ fh,0.05}} = 1.27...1.62 \tag{8}$$

3.2 Fastener overstrength from allowable range within steel grade

Most design codes specify the minimum yield strength for a steel grade, $f_{y,min}$, as well as acceptable f_u/f_y ratios, with f_u being the steel's ultimate strength. AS/NZS 4671 (2001) additionally defines an allowable maximum yield strength, $f_{y,max}$. The difference between the 5th and 95th percentile within one batch of dowels of a specified grade is usually relatively small ($f_{y,k} \approx f_{y,mean}$, Ottenhaus et al. 2016a). However, as $f_{y,min}$ is generally used in design, overstrength can be introduced if the fastener's yield strength exceeds $f_{y,min}$. For dowelled connections in CLT with an internal steel plate and $d \leq 30$ mm, $\rho \leq 600$k g/m^3, $t_1 \geq d$, and Grade 300 steel dowels as specified in AS/NZS 4671 (2001), an overstrength factor of $\gamma_{0.95,My}$ = 1.15 is obtained using Equation 9 (Ottenhaus et al. 2017a). Similarly, $\gamma_{0.95,My}$ can be calculated for other fasteners types and connection configuration by inserting the respective EYM formulas and strength values into Equation 9.

3.3 Fastener overstrength from wrong steel grade

Further overstrength is introduced if the supplied steel grade is significantly stronger than the specified grade. In the present research, ϕ20mm Grade 300 steel dowels were ordered with a specified minimum yield strength of $f_{y,min}$ = 300MPa. The dowel yield moment was obtained in a three-point bending test according to ISO 10984 (2015). Three samples were tested and the average yield strength f_y = 596 MPa and yield moment M_y = 795000 Nmm were determined with the 5% fastener diameter offset method. Three dowels of the same batch were subsequently machined to a 10mm diameter and tested in tension according to ASTM E8/E8M-16a (2016). The average yield strength was determined with the 0.2% offset method as f_y = 535 MPa. The ultimate tensile strength was f_u = 589 MPa and young's

modulus was E = 194.12 GPa, respectively. It was concluded that Grade 500 had been supplied instead of Grade 300. For the given connection configuration and failure mode II, this introduced overstrength of $\gamma_{0.95,My}$ = 1.24 for ϕ20mm dowels and a side member thickness of t_1 = 4.25d with an assumed density of ρ_k = 435 kg/m^3. However, for 400kg/m^3 $\rho \leq$ 600kg/m^3, and $t_1 \geq d$, the overstrength factor is $\gamma_{0.95,My}$ = 1.50 as shown in Equation 9.

AS/NZS 4671 (2001) specifies a minimum f_u/f_y ratio of 1.15 for both Grade 300 and Grade 500. In the given case, the f_u/f_y ratio was about 1.1. While fastener overstrength increases the risk of brittle timber failure, it also decreases fastener ductility as the f_u/f_y ratios are often lower for higher grade steel. If the internal steel plate meets the specified steel grade, some energy can be dissipated through localized yielding of the plate underneath the dowel but this should not be relied upon.

It is also interesting to note that f_y derived from three-point bending tests was higher than that from the tensile tests. This confirms that $M_{y,p}$ can indeed be achieved in larger diameter fasteners. However, due to the nature of the high steel grade, no yield plateau was formed in the three-point bending nor tensile test which required the use of the 5% and 0.2% offset methods, respectively, to determine f_y.

$$\gamma_{0.95,My} = \frac{F_{EYM\,My,0.95}}{F_{EYM\,My,0.05}} = \begin{cases} 1.00\text{K } 1.15 & \text{allowable range Grade 300} \\ 1.35\text{K } 1.50 & \text{Grade 500 instead of Grade 300} \end{cases} \quad (9)$$

4 CONNECTION TESTING

4.1 Test setup and strength prediction

A total of 12 dowelled CLT connection specimens with three different layouts (L1, L2, and L3) were tested under monotonic and cyclic loading. The specimens consisted of 2.5 m×4 m 5-layer CLT panels with a 25 mm thick internal steel plate. The inner layer was 35 mm thick and the cross-layers were 40 mm thick, all made of SG8 (NZS 3603 1993). The outer layer was 45 mm thick for layout L1 and L3. For layout L2 the outer layer was made of 43 mm thick grade 11 Laminated Veneer Lumber (LVL) (AS/NZS 4357.0 2005). The fastener group consisted of 12 smooth ϕ20 mm dowels and 4 additional dowels with threaded ends and hand-tight nuts in the corners, all specified Grade 300 (AS/NZS 4671 2001). The fastener spacing with designations according to Eurocode 5 (2004/2008) is given in Table 3. More information regarding the connection configurations as well as drawings of the test setup can be found in Ottenhaus et al. 2017b. Specimens 01, 05, and 09 were tested under monotonic loading according to EN 26891 (1991) whereas all other specimens were subjected to cyclic loading according to the ISO loading protocol (ISO 16670:2003).

The CLT embedment strength was predicted according to Equation 7. The following formulas were used for the prediction of the embedment strength of the CLT-LVL panels:

$$f_{h,u,90,k} = \frac{0.082(1-0.01d)\rho_k}{(1.35+0.015d)} \quad \text{for sawn radiata pine, perp. to grain loading (EN 1995-1-1 2004/2008)} \quad (10a)$$

$$f_{h,u,k,LVL} = 0.075(1-0.0037d)\rho_k \quad \text{for LVL (Franke and Quenneville 2011)} \quad (10b)$$

$$f_{h,u,k,CLT-LVL} = \frac{40 f_{h,u,90,k} + 43 f_{h,u,0,k,LVL}}{83} \quad \text{for CLT-LVL hybrid used in large-scale experiments} \quad (10c)$$

The connection strength was predicted based on the specified yield strength, f_y = 300MPa, and the characteristic timber densities obtained from previous small-scale connection testing: $\rho_{k,LVL}$=585kg/m^3, $\rho_{k,CLT}$=435kg/m^3 (Ottenhaus et al. 2016a). This was deemed appropriate as the same material grades and suppliers were used. The input material properties are given Table 2. The predicted connection strength and failure mode (BR = brittle, LD = low ductility, MD = moderate ductility, HD = high ductility; classified according to Smith et al. 2006) are given in Table 3.

Table 2. Input material properties.

	$\rho_{k,CLT}$ [kg/m^3]	$\rho_{k,LVL}$ [kg/m^3]	$f_{h,u,k,CLT}$ [MPa]	$f_{h,u,90,k}$ [MPa]	$f_{h,u,0,k,LVL}$ [MPa]	$f_{h,u,k,CLT\text{-}LVL}$ [MPa]	$M_{y,y}$ [Nmm]	$M_{y,u}$ [Nmm]
input	435	585	24.95	17.29	40.63	29.38	235619	400000

Table 3. Ductile strength prediction (considering rope effect of end row bolts).

layout	a_1 [mm]	a_2 [mm]	a_3 [mm]	$F_{Ia,y,k}$ [kN]	$F_{Ia,u,k}$ [kN]	$F_{II,y,k}$ [kN]	$F_{II,u,k}$ [kN]	$F_{III,y,k}$ [kN]	$F_{III,u,k}$ [kN]	μ
L1 CLT dense	140	140	140	1120	1400	**588**	**788**	736	1072	BR
L2 CLT-LVL	140	140	140	1288	1610	**661**	**881**	799	1164	MD
L3 CLT wide	140	240	140	1120	1400	**588**	**788**	736	1072	LD

4.2 **Results**

Figure 6 displays the load-displacement and backbone curves for layout L1 to L3, predicted characteristic ultimate strength, $F_{u,k,pred}$, and overstrength limit, $\gamma_{Rd1} \times F_{u,k,pred}$ (γ_{Rd1} = 1.68 Ottenhaus et al. 2017a).

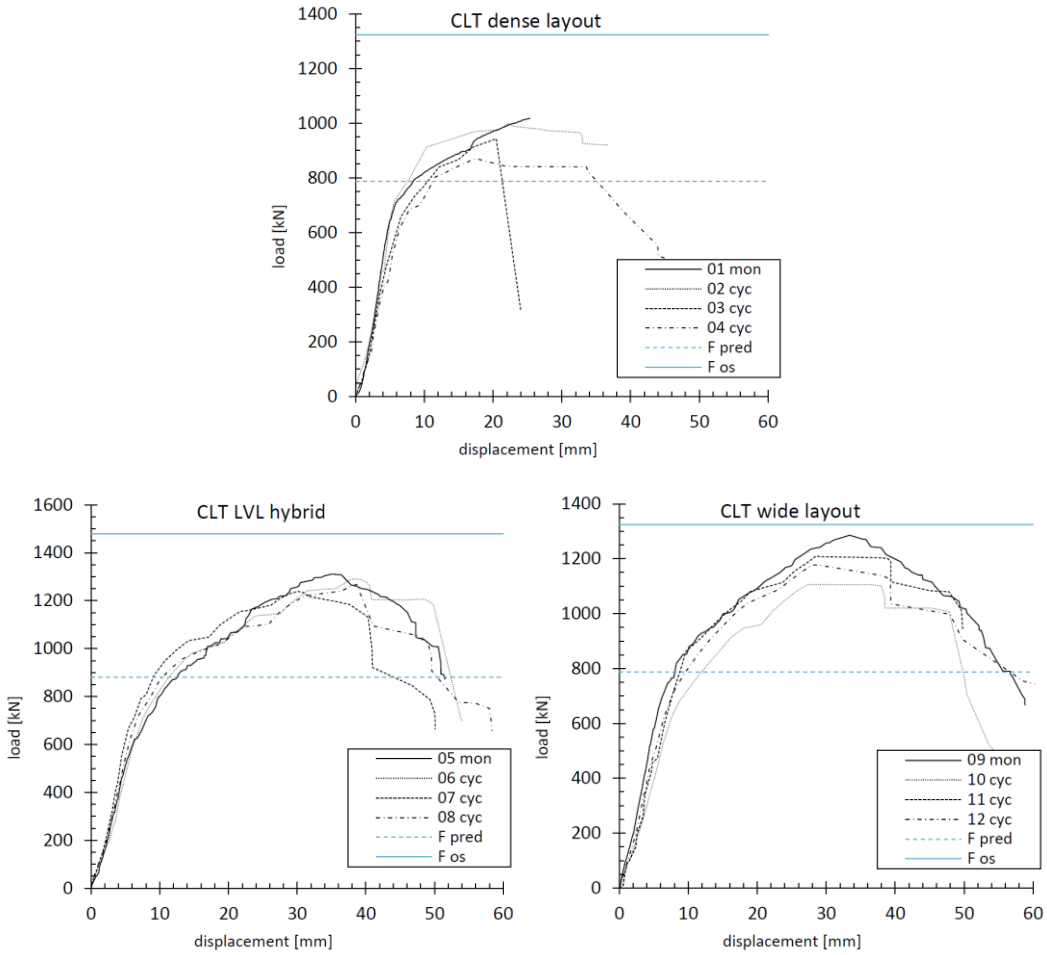

Figure 6. Load displacement curves.

Table 4 shows the connection yield load, F_y, peak load, F_{max}, ultimate load, F_u, overstrength, $\gamma_{Rd2} = F_{max,exp}/F_{u,k,pred}$, yield and ultimate displacements, Δ_y and Δ_u, ductility μ, and failure mode.

Table 4. Test results large-scale monotonic (M) and cyclic (C) connection testing.

layout	panel	F_y [kN]	F_{max} [kN]	F_u [kN]	γ_{Rd2}	Δ_y [mm]	Δ_u [mm]	$\mu = \Delta_u/\Delta_y$
L1 CLT dense	01 M	722	1023	1023	1.30	5.7	25.4	4.5 (MD)
	02 C	812	1000	800	1.27	6.6	37.0	5.6 (MD)
	03 C	701	943	943	1.20	6.1	20.4	3.3 (LD)
	04 C	710	871	696	1.10	6.9	38.0	5.5 (MD)
L2 CLT-LVL	05 M	826	1311	1049	1.49	7.6	47.5	6.3 (HD)
	06 C	870	1292	1033	1.47	7.8	51.0	6.5 (HD)
	07 C	855	1239	991	1.41	6.8	41.0	6.0 (HD)
	08 C	825	1268	1014	1.44	7.8	50.1	6.4 (HD)
L3 CLT wide	09 M	830	1286	1028	1.63	7.0	49.4	7.1 (HD)
	10 C	849	1106	885	1.40	10.1	48.9	4.8 (MD)
	11 C	920	1210	968	1.53	9.9	49.8	5.0 (HD)
	12 C	849	1177	942	1.49	8.2	49.0	6.0 (MD)

5 DISCUSSION

As shown in Figure 6 and Table 4, the strength prediction $F_{u,k,pred}$ was conservative in all cases and the predicted connection overstrength of γ_{Rd1} = 1.68 is also conservatively applicable for the layout containing LVL layers. As New Zealand LVL is performance graded, it has a smaller variability than CLT which means that γ_{Rd1} = 1.68 is also conservatively applicable for the layout containing LVL layers. The overstrength of monotonic experiments was larger than that of cyclic experiments which agrees with previous findings (Ottenhaus et al. 2017a), however a larger sample size is needed to confirm this observation.

After testing, 16 CLT density samples were taken and the 5th percentile was established with the nearest rank method as $\rho_{5\%,CLT}$ = 382kg/m^3 which is both lower than the assumed characteristic density as well as 5th percentile reported by the supplier. If inserted in Equation 2, $F_{II,u,k(\rho 5\%,CLT\ =\ 382\ \mathrm{kg/m3},\ fy\ =\ 300\ \mathrm{MPa})}$ = 706 and γ_{Rd2} = 1286 / 706 = 1.82 are obtained for CLT. After division by $\gamma_{0.95,My}$ = 1.24, $\gamma_{an,My}$ ≈ 1.00 (as $M_{y,p}$ is used instead of $M_{y,eff}$) and $\gamma_{an,fh}$ = 1.06 (Ottenhaus et al. 2017a), $\gamma_{0.95,fh}$ = 1.82 / (1.24x1.00x1.06) = 1.39 is obtained which is very close to the previously reported 1.38. This means that the aforementioned upper bound of the embedment overstrength, $\gamma_{0.95,fh}$ = 1.62, may be too high, as other timber strength properties are also positively correlated with density.

Furthermore, the findings also indicate that the probability of encountering both significantly stronger steel and significantly higher timber strength is low. However, this assumption should be further investigated by probability-based studies, e.g. in Monte-Carlo simulations.

Finally, delivery of the wrong steel grade is an ongoing issue that needs to be addressed with industry in order to prevent unexpected overstrength and ensure that specified ductility levels are met.

6 CONCLUSION

A total of 12 large-scale dowelled CLT connections were subjected to monotonic and cyclic loading and their overstrength was calculated γ_{Rd2} = 1.63 which is smaller than the theoretically established connection overstrength factor of γ_{Rd1} = 1.68 based on previous small-scale testing (Ottenhaus et al. 2017a).

While little analytical overstrength, γ_{an}, is introduced by semi-empirical embedment formulas, the plastic yield moment, $M_{y,p}$, should be used instead of the effective yield moment, $M_{y,eff}$, in order to

avoid artificial analytical overstrength in γ_{an}.

Variability in timber density introduces embedment overstrength, $\gamma_{0.95,fh}$, but it also increases the strength of brittle failure modes and the previously reported $\gamma_{0.95,fh}$ = 1.38 seems to be a good estimate.

Unexpected fastener overstrength caused by delivery of a higher steel grade is an ongoing problem as it can increase fastener overstrength from $\gamma_{0.95,My}$ = 1.15 to $\gamma_{0.95,My}$ = 1.50 and this issue should be addressed properly by the industry.

In conclusion, the findings suggest that it is possible to analytically predict overstrength dowelled connections in CLT and the same approach can be used to derive overstrength factors for other types of fasteners, different connection layouts, and other wood products.

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