Analytical Method to Derive Overstrength of Dowel-Type Connections

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1 Introduction

Currently, limited values are available for overstrength factors of timber connections in design standards. In capacity design of timber structures, the potential for connection overstrength needs to be taken into account to ensure ductile response of the system under seismic loading. However, overly conservative overstrength factors can lead to uneconomic design and increase building cost unnecessarily.

In the past, the overstrength factor, γ_{Rd}, has often been derived for a whole connection or structural system, although several authors have also identified individual overstrength components (Gavric et al. 2015, Vogt et al. 2014, Dickof et al. 2014, Brühl et al. 2014). Figure 1 shows the desired strength hierarchy and overstrength components modified from Jorissen and Fragiacomo (2011):

$$\gamma_{Rd}*F_{Rd} = \gamma_M* \gamma_{an}* \gamma_{0.95}*F_{Rd} = \gamma_{an}* \gamma_{0.95}*F_{Rk} \leq F_{BRd} \quad (1)$$

where γ_M is the material safety factor, γ_{an} accounts for conservatism of analytical models, and $\gamma_{0.95}$ quantifies the difference between the 5th and 95th percentile of the strength distribution, ($F_{0.05}$ and $F_{0.95}$, respectively). F_{Rk} and F_{Rd} denote the connection's characteristic and design strength, and F_{BRd} is the design strength of the brittle member or connection failure mode. The individual components are calculated as:

$$\gamma_M = F_{Rk} / F_{Rd} \quad (2.1)$$

$$\gamma_{an} = F_{0.05} / F_{Rk} \quad (2.2)$$

$$\gamma_{0.95} = F_{0.95} / F_{0.05} \quad (2.2)$$

This paper presents a detailed procedure on how to analytically derive and quantify these individual overstrength components for dowel-type connections in the context of Eurocode 5.

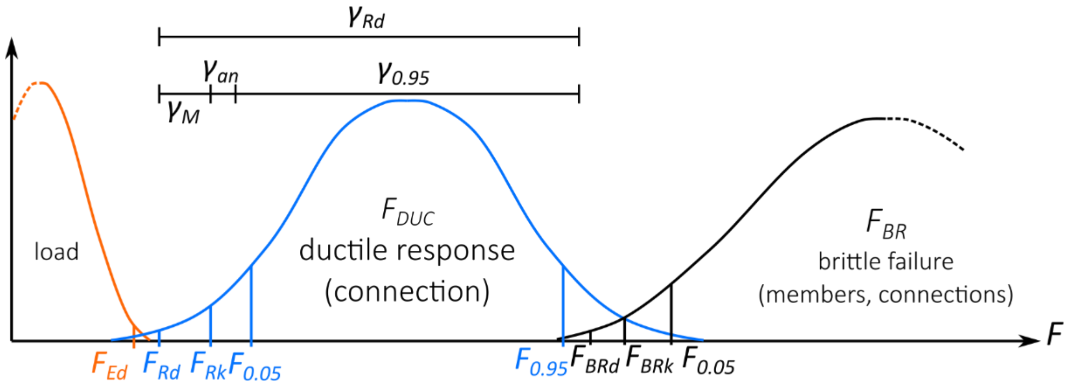

Figure 1. Overstrength concept (modified from Jorissen and Fragiacomo 2011)

2 Overstrength components

The break-down of overstrength components is shown in Figure 2 and explained in the following paragraphs. The European Yield Model (EYM, based on the Johansen's Equations) presented in Eurocode 5 is based on timber material embedment strength, f_h, which is correlated to timber density, ρ, and the fastener yield moment, M_y, which in turn depends on the steel yield strength, f_y. Each overstrength component can thus be subdivided into a contribution from the timber and steel material.

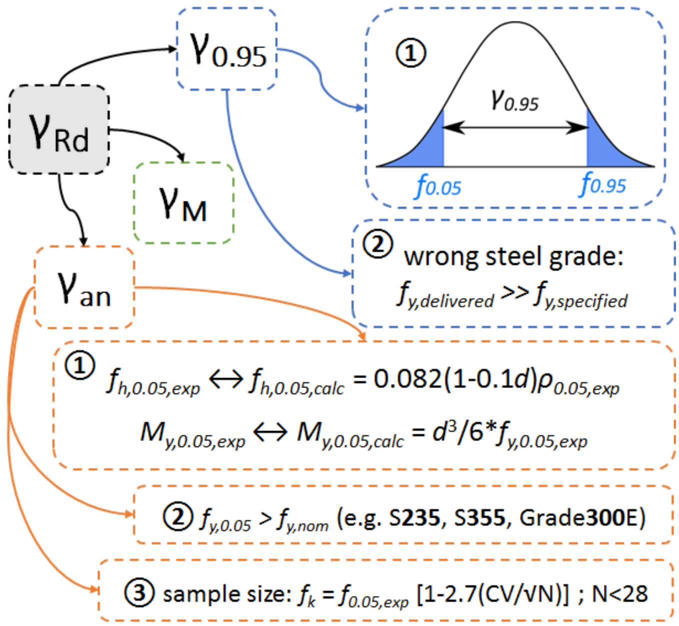

Figure 2. Overstrength components.

2.1 Material safety factor γ_M

The material safety factor, γ_M, is given as 1.30 in Eurocode 5 and therefore known to the designer.

2.2 Analytical model overstrength γ_{an}

Analytical model overstrength, γ_{an}, stems from the difference between the 5th percentile of material strength, $f_{0.05}$, and the calculated characteristic value, f_k. Generally speaking, analytical model overstrength is introduced by conservatism in semi-empirical formulas as shown in Figure 3. $F_{0.05}$ and F_{Rk} can be calculated by inserting the respective embedment strength values ($f_{h,0.05}$ and $f_{h,k}$) and yield moment values ($M_{y,0.05}$ and $M_{y,k}$) into the EYM.

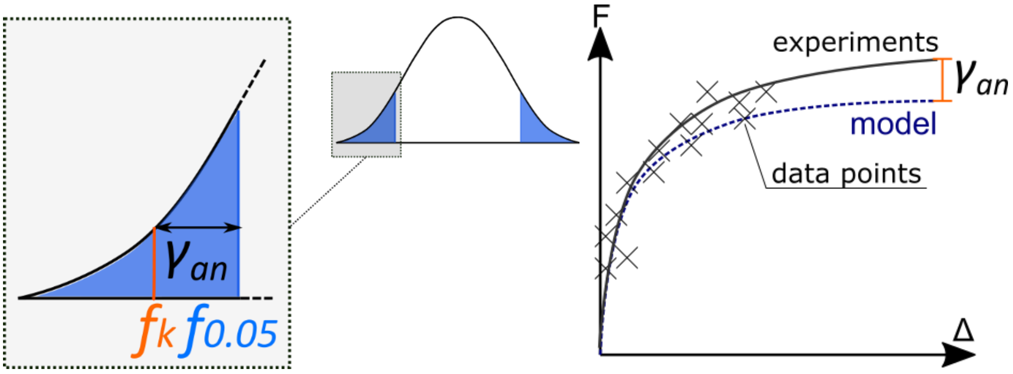

Figure 3. Analytical model overstrength.

2.2.1 *Difference between semi-empirical model and data $\gamma_{an,1}$*

Semi-empirical models are usually established by conservatively calibrating an analytical model to a dataset. As discussed in Ottenhaus et al. 2017, due to this calibration method the difference between calculated and experimental embedment strength is rather small which leads to small overstrength factors $\gamma_{an,fh}$:

$$\gamma_{an,fh} = F(f_{h,0.05,exp}) / F(f_{h,0.05}(\rho_{0.05,exp})) \approx 1.06 \tag{3}$$

$F(f_{h,0.05,exp})$ is obtained by inserting the experimentally established 5th percentile of embedment strength, $f_{h,0.05}$, into the EYM equations.

$F(f_{h,0.05}(\rho_{0.05,exp}))$ is obtained if the 5th percentile of the measured density, $\rho_{0.05,exp}$, is used to calculate the embedment strength $f_{h,0.05} = 0.082(1\text{-}0.1d)\rho_{0.05,exp}$ (Eurocode 5).

As discussed in Ottenhaus et al. 2016, little overstrength is introduced by the difference between $M_{y,exp}$ and $M_{y,calc}$:

$$\gamma_{an,My} \approx 1.0. \tag{4}$$

The total analytical overstrength factor, $\gamma_{an,1}$, is calculated as $\gamma_{an,1} = \gamma_{an,fh} * \gamma_{an,My}$.

2.2.2 *Nominal values of steel grades $\gamma_{an,2}$*

Karmazínová and Melcher (2012) conducted experiments on grade S235 and S355 steel (grades according to Eurocode 3). Their results for the mean value for yield strength, $f_{y,mean}$, coefficient of variation, CV, as well as sample size, N, are given in Table 1. Based on their findings, it is possible to calculate the 5th percentile, $f_{y,0.05}$, for a log-normal distribution with $f_{y,0.05} = f_{y,mean} * e^{-k*CV}$, where k depends on the sample size, N, confidence interval (75%) and percentile (5th percentile). Table 1 shows that the 5th percentile of steel yield strength, $f_{y,0.05}$, was higher than the nominal strength, $f_{y,nom}$, which is used instead of a characteristic value and indicated by the grade's name (e.g. S235 with $f_{y,nom}$ = 235MPa). This difference introduces overstrength:

$$\gamma_{an,2} = \gamma_{an,fy,nom} = F(f_{y,0.05}) / F(f_{y,nom}) \quad (5)$$

Furthermore, a New Zealand steel supplier made their tensile test data available for New Zealand steel grades Grade300 and Grade500 (AS/NZS 4671:2001), providing N, $f_{y,mean}$, $f_{y,0.05}$, and $f_{y,0.95}$.

Table 1. Overstrength from steel yield strength distribution (Karmazínová and Melcher 2012).

	$f_{y,nom}$ [MPa]	$f_{y,mean}$ [MPa]	CV	N	k	$f_{y,0.05}$ [MPa]	$\gamma_{an,fy,nom}$ [(4)]
S235	235	327 [(1)]	0.075 [(1)]	26 [(1)]	1.890 [(2)]	284	1.10
S355	355	452 [(1)]	0.05 [(1)]	19 [(1)]	1.942 [(2)]	410	1.08
Grade300	300	339		320		320	1.03
Grade500	500	554		230		523	1.02

[(1)] Karmazínová and Melcher (2012), [(2)] Guttmann (1970), [(3)] tensile test data provided by Pacific Steel, [(4)] holds for ρ ϵ [300,600] kg/m^3, d ϵ [6,30] mm, t/d ϵ [2,10]

2.2.3 Sample size $\gamma_{an,3}$

The accuracy of a model is limited by the sample size and variability of the underlying dataset. Therefore, the characteristic strength value needs to take the sample size, N, and coefficient of variation, CV, of a dataset into account: $f_k = f_{mean}$ (1 - k CV). k is thus dependent on N, and k = 1.645 only holds for a normal distribution with N→∞.

Generally speaking, a dataset should consist of at least 28 samples in order to derive a characteristic value or 5th percentile with 75% confidence. However, this may not always feasible or possible. For small sample sizes Leicester (1986) proposes:

$$F_k = F_{0.05,exp}\,[1-2.7(CV/\sqrt{N})];\ \text{for } 10 \le N < 28 \quad (6.1)$$

$$F_k = F_{min}\,(N/27)^{CV}\ \text{for } N<10 \quad (6.2)$$

where $f_{0.05,exp}$ is estimated from the data (e.g. with linear interpolation and nearest rank method), and f_{min} is the minimum value of the sample. As Equation (6.1) delivers reasonably accurate results (Smith and Foliente 2002), an estimated overstrength can be computed for 10≤N<28 as

$$\gamma_{an,3} = \gamma_{an,sample} = F_{0.05,exp} / F_k = 1 / [1-2.7(CV/\sqrt{N})] \quad (7)$$

For sample sizes N<10, γ_{an} should only be treated as a rough estimate.

2.3 Material Overstrength $\gamma_{0.95}$

This overstrength component is caused by the variability of material properties which can be expressed as the difference of the 95th and 5th percentile of the strength distribution ($f_{h,0.95}$ and $f_{h,0.05}$, respectively) as shown in Figure 4. However, it is also sometimes the case that steel suppliers deliver a stronger grade than specified, thinking that they act in the best interest of the client (Sandhaas and v. d. Kuilen 2017, Ottenhaus et al. 2018). This can lead to undesirable failure modes and impair seismic safety.

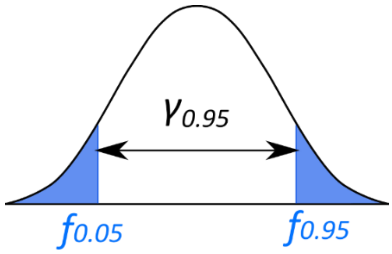

Figure 4. Material overstrength.

2.3.1 Difference between $F_{0.95}$ and $F_{0.05}$ ($\gamma_{0.95,1}$)

This overstrength component can be easily calculated if the mean value and coefficient of variation of the strength distribution are known. In many cases, the timber supplier can either provide data of the timber density distribution, or the material is stress-graded in which case the expected density range is defined by the strength grade given in the respective standard, e.g. EN338:2016.

For a given density distribution, the overstrength, $\gamma_{0.95,fh}$, can be calculated by inserting $\rho_{0.05}$ and $\rho_{0.95}$ into the embedment formula applicable for the timber material, and inserting the obtained values for f_h into the EYM:

$$\gamma_{0.95,fh} = F(f_{h,0.95}) / F(f_{h,0.05}) \tag{8}$$

As shown in Table 2, $\gamma_{0.95}$ decreases if the material is stress-graded or if the material is relatively homogenous as is the case with Laminated Veneer Lumber (LVL). This should encourage designers and suppliers to use stress-graded timber material with less variability.

Table 2. Overstrength from timber density distribution.

	$\rho_{0.05}$ [kg/m^3]	$\rho_{0.95}$ [kg/m^3]	$\gamma_{0.95,fh}$*
NZ ungraded *radiata pine* (1)	402	608	1.51
LVL10(2)	470	558	1.19
LVL11(2)	480	564	1.18
LVL13(2)	500	585	1.17
C18(3)	320	440	1.38
C24(3)	350	490	1.40

(1) provided by XLam Ltd, (2) provided by Nelson Pine Industries Ltd grades according to AS/NZS 4357.0:2005, (3) EN336:2016 with assumed standard distribution * holds for $d \in [6,30]$ mm, $t/d \in [2,10]$

Although steel has relatively consistent material properties, there is still a certain variability in the yield strength distribution. An allowable range for f_y is given in AS/NZS 4671:2001 but not in Eurocode 3. For instance for Grade300 steel, the allowable range of 300-380 MPa leads to an overstrength factor of $\gamma_{0.95,fy}$ = 1.13 (Ottenhaus et al. 2017). Additionally, experimental results were available for grade S235 and S355 (Karmazínová and Melcher 2012) and supplier tensile test data was provided for Grade300 and Grade500. The overstrength factor, $\gamma_{0.95,fy}$, was obtained with Equation (9) by inserting $f_{y,0.05}$ and $f_{y,0.95}$ into Equations 8.9 through 8.13 of Eurocode 5 Part 1-1:

$$\gamma_{0.95,fy} = F(f_{y,0.95}) \,/\, F(f_{y,0.05}) \qquad (9)$$

The total material overstrength factor, $\gamma_{0.95,1}$, is calculated as $\gamma_{0.95,1} = \gamma_{0.95,fh} * \gamma_{0.95,fy}$.

Table 3. Overstrength from steel yield strength distribution and allowable range.

	$f_{y,0.05}$ [MPa]	$f_{y,0.95}$ [MPa]	$\gamma_{0.95,fy}$*
S235(1)	284	377	1.15
S355(1)	410	498	1.10
Grade300(2)	320	358	1.06
Grade500(2)	523	584	1.06
Grade300(3)	300	380	1.13
Grade500(3)	500	600	1.10

* holds for ρ ϵ [300,600] kg/m3, d ϵ [6,30] mm, t/d ϵ [2,10], (1)Karmazínová and Melcher 2012, (2) tensile test data provided by Pacific Steel, (3)allowable range AS/NZS 4671:2001

2.3.2 Unexpected overstrength due to delivery of stronger grade ($\gamma_{0.95,2}$)

Unexpected overstrength is introduced if the supplied steel grade is significantly stronger than the specified grade. Often, suppliers think that they are acting in the best interest of the client, as the material is stronger and thus “better”. In the case of recently conducted connection tests at the University of Canterbury, Grade500 steel dowels instead of Grade300 dowels (grades according to AS/NZS 4671:2011) were delivered which introduced additional overstrength of up to 1.50 (Ottenhaus et al. 2018). This issue was discovered during three-point bending and tensile yield tests. In practice, in-situ material testing is uncommon and hence this sort of mistake often goes unnoticed which likely leads to unsafe seismic design. Therefore, this issue needs to be addressed with suppliers, and if necessary material samples need to be strength tested.

As several timber material properties such as bending and embedment strength are correlated with density, delivery of a higher timber densities also causes unexpected overstrength and needs to be avoided. Shear strength and tensile splitting strength on the other hand are independent of the density of commonly used softwoods and brittle capacity remains unchanged for higher timber densities (Ranta-Maunus 2007).

3 Methodology to analytically derive overstrength

In order to design timber structures with Eurocode 5, designers need to know the characteristic material values. If the timber material is stress-graded, the characteristic timber density, ρ_k, is defined by the strength class and the nominal steel yield strength, $f_{y,nom}$, is defined by the steel grade (e.g. S235, S335, Grade300, Grade500). The connections can then be designed to withstand all load cases (including seismic loading) using the EYM as given in Eurocode 5 (Figure 5):

$$F_{Ed} \leq F_{Rd} = F_{Rk} / \gamma_M \quad (10)$$

where F_{Ed} designates the design demand, F_{Rd} is the connection's ductile design strength calculated with the EYM, γ_M is the material safety factor, and F_{Rk} is the connection's characteristic strength using ρ_k and $f_{y,nom}$.

The next step is to ensure that ductility can be achieved in the connection by protecting all brittle members and brittle connection failure modes from the connection's overstrength γ_{Rd} using Equation 1: $\gamma_{Rd}*F_{Rd} = \gamma_{an}* \gamma_{0.95}*F_{Rk} \leq F_{BR,d}$

The overstrength components can be calculated as follows:

1) Determine the 5th and 95th percentile of the density and yield strength distribution ($\rho_{0.05}$, $\rho_{0.95}$, $f_{y,0.05}$, and $f_{y,0.95}$). These can be obtained from the timber and steel supplier. Alternatively, acceptable limits for these values may be derived from the strength class definitions.
2) Calculate the yield moment and embedment strength $M_{y,0.95}$, $M_{y,0.05}$, $f_{h,0.95}$, $f_{h,0.05}$ for the given dowel diameter d. M_y should be calculated as $M_y = d^3/6*f_y$ to avoid introducing analytical overstrength (Ottenhaus et al. 2017).
3) From the design, the governing EYM mode is known for F_{Rk}. Now calculate $F_{0.95}$ and $F_{0.05}$ for this mode.
4) The material overstrength can be calculated as $\gamma_{0.95} = F_{0.95} / F_{0.05}$
5) The model overstrength can now be calculated as $\gamma_{an} = F_{0.05} / F_{Rk}$
6) The entire overstrength is $\gamma_{Rd} = \gamma_M * \gamma_{an} * \gamma_{0.95}$ with $\gamma_M = 1.3$.

Calculation formulas for γ_{Rd} for dowel-type steel-to-timber connections are outlined in the following. The failure modes according to Eurocode 5 are depicted in Figure 5.

(a) $F_R = 0.4 * f_h * t * d \rightarrow \gamma_{Rd,a} = F_{0.95} / F_{Rd} = 1.3 * f_{h,0.95} / f_{h,k}$

(b) $F_R = 1.15\sqrt{(2M_y * f_h * d)} + F_{ax}/4$
$\rightarrow \gamma_{Rd,b} = 1.3 * [\sqrt{(2M_{y,0.95} * f_{h,0.95} * d)} + F_{ax,0.95}/4.6] / [\sqrt{(2M_{y,k} * f_{h,k} * d)} + F_{ax,k}/4.6]$

(c) $F_R = f_h * t * d \rightarrow \gamma_{Rd,c} = \gamma_{Rd,a} = 1.3 * f_{h,0.95} / f_{h,k}$

(d) $F_R = f_h * t * d[\sqrt{(2+4M_y/(f_h * d * t^2))} - 1] + F_{ax}/4$
$\rightarrow \gamma_{Rd,d} = 1.3 * [f_{h,0.95} * t * d[\sqrt{(2+4M_{y,0.95}/(f_{h,0.95} * d * t^2))} - 1] + F_{ax,0.95}/4]$ /
$[f_{h,k} * t * d[\sqrt{(2+4M_{y,k}/(f_{h,k} * d * t^2))} - 1] + F_{ax,k}/4]$

(e) $F_R = 2.3\sqrt{(2M_y * f_h * d)} + F_{ax}/4$
$\rightarrow \gamma_{Rd,e} = 1.3 * [\sqrt{(2M_{y,0.95} * f_{h,0.95} * d)} + F_{ax,0.95}/9.2] / [\sqrt{(2M_{y,k} * f_{h,k} * d)} + F_{ax,k}/9.2]$

(f) $F_R = f_h * t * d \rightarrow \gamma_{Rd,f} = \gamma_{Rd,a}$

(g) $F_R = f_h * t * d[\sqrt{(2+4M_y/(f_h * d * t^2))} - 1] + F_{ax}/4 \rightarrow \gamma_{Rd,g} = \gamma_{Rd,d}$

(h) $F_R = 2.3\sqrt{(2M_y * f_h * d)} + F_{ax}/4 \rightarrow \gamma_{Rd,h} = \gamma_{Rd,e}$

(j) $F_R = 0.5 f_h * t * d \rightarrow \gamma_{Rd,j} = \gamma_{Rd,a}$

(k) $F_R = 1.15\sqrt{(2M_y * f_h * d)} + F_{ax}/4 \rightarrow \gamma_{Rd,k} = \gamma_{Rd,b}$

(l) $F_R = 0.5 f_h * t * d \rightarrow \gamma_{Rd,l} = \gamma_{Rd,a}$

(m) $F_R = 2.3\sqrt{(M_y * f_h * d)} + F_{ax}/4 \rightarrow \gamma_{Rd,m} = \gamma_{Rd,e}$

For dowelled steel-to-timber connections, the rope effect reduces to zero: $F_{ax}/4 = 0$. The overstrength factors can then be simplified as follows:

$\gamma_{Rd,a} = \gamma_{Rd,c} = \gamma_{Rd,f} = \gamma_{Rd,j} = \gamma_{Rd,l} = 1.3 * f_{h,0.95} / f_{h,k}$

$\gamma_{Rd,b} = \gamma_{Rd,e} = \gamma_{Rd,h} = \gamma_{Rd,k} = \gamma_{Rd,m} = 1.3 * \sqrt{[(M_{y,0.95} * f_{h,0.95}) / (M_{y,k} * f_{h,k})]}$

$\gamma_{Rd,d} = \gamma_{Rd,g} = 1.3 * (f_{h,0.95} [\sqrt{(2+4M_{y,0.95}/(f_{h,0.95} * d * t^2))} - 1]) / (f_{h,k} [\sqrt{(2+4M_{y,k}/(f_{h,k} * d * t^2))} - 1])$

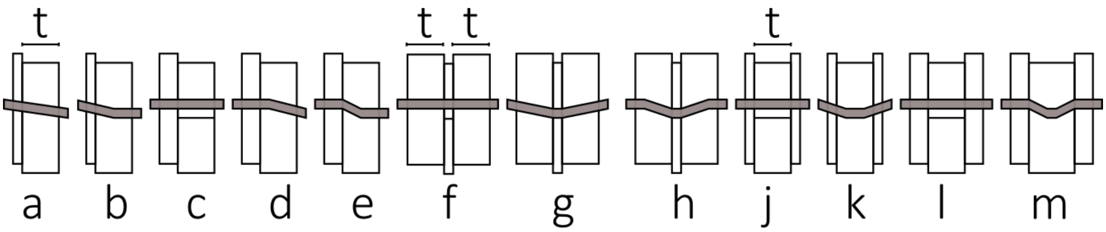

Figure 5. Failure modes for steel-to-timber connections (modified from Eurocode 5).

4 Example calculation

Let's consider a single-sided dowelled connection in ungraded sawn New Zealand *radiata pine* and New Zealand LVL11 under seismic loading. The connection is made of d = 12 mm Grade300 steel dowels with a thin steel plate and side member thickness of $t = 5.5d$ = 66 mm. The connection is loaded parallel to the grain (dowel in shear) and the design load is F_{Ed} = 52.0 kN. The possible failure modes are mode (a) and (b) depicted in Figure 5.

For a single dowel in shear and a thin steel plate, Eurocode 5 gives the characteristic ductile strength as:

$$F_k = \min[0.4*f_{h,k}*t*d;\ 1.15\sqrt{(2M_{y,k}*f_{h,k}*d)}]\ ;\ F_{ax,k} = 0 \text{ for dowels} \quad (11)$$

with $M_{y,k} = d^3/6*f_{y.nom}$ and $f_{h,k} = 0.082(1\text{-}0.1d)\rho_k$ for sawn timber. Note that different embedment formulas can be used in this step, e.g. Franke and Quenneville (2011) or Uibel and Blaß (2014).

Table 4 gives the material strength values. As timber grades in New Zealand are based on the Young's Modulus and no characteristic embedment strength values are available, $\rho_{0.05}$ is used to calculate $f_{h,0.05}$, and an overstrength factor, γ_{an}, is applied.

Table 4. Input values for example calculation.

	ρ_{sawn} [kg/m³]	ρ_{LVL} [kg/m³]	$f_{h,sawn}$ [MPa]	$f_{h,LVL}$ [MPa]	$f_{y,Grade300}$ [MPa]	$M_{y,Grade300}$ [Nmm]
X_k (X_{nom})					300	86 400
$X_{0.05}$	402(1)	480(2)	29.01	34.66	320(3)	92 160
$X_{0.95}$	608(1)	564(2)	43.87	40.73	358(3)	103 104

(1) provided by XLam Ltd, (2) provided by Nelson Pine Industries Ltd, (3) provided by Pacific Steel for Grade300 round bar

Inserted into Equation 11, we find that mode (b) is governing. Note that mode (a) relies solely on timber crushing for ductility and should therefore be avoided in seismic design. In order to satisfy Equation 10 ($F_{Ed} \leq F_{Rd}$), we require 8 dowels in sawn timber and 7 dowels in LVL:

$$F_{Rd,sawn} = F_{Rk,sawn} / \gamma_M = \min\{9.19,8.92\}/1.3 = 6.86 \text{ kN} \rightarrow 52.0 \text{ kN} < 8\text{x}6.86 = 54.89 \text{ kN}$$

$$F_{Rd,LVL} = F_{k,LVL} / \gamma_M = \min\{10.98,9.75\}/1.3 = 7.50 \text{ kN} \rightarrow 52.0 \text{ kN} < 7\text{x}7.50 = 52.50 \text{ kN}$$

Furthermore, we assume that the dowels are arranged in such a manner that their full capacity can be utilized (n_{eff} = 1.0).

Let's first consider the analytical model overstrength, γ_{an}. From Ottenhaus et al. (2017) it is known that $\gamma_{an,My}$ = 1.00 and $\gamma_{an,fh}$ = 1.06. The latter was derived as an upper bound for CLT and can serve as a conservative estimate for LVL. Furthermore, Table 1 gives $\gamma_{an,fy,nom}$ = 1.03 for Grade300 which results in γ_{an} = 1.00*1.06*1.03 = 1.09.

As outlined above, $\gamma_{0.95}$ can be calculated as $\gamma_{0.95} = \sqrt{[(M_{y,0.95}*f_{h,0.95}) / (M_{y,0.05}*f_{h,0.05})]}$ = {1.30 sawn, 1.15 LVL}.

The resulting overstrength can now be calculated:
$\gamma_{Rd,sawn} = \gamma_M * \gamma_{an} * \gamma_{an} * \gamma_{0.95,sawn}$ = 1.3*1.09*1.30 = 1.85 for ungraded sawn *radiata pine* and $\gamma_{Rd,LVL} = \gamma_M * \gamma_{an} * \gamma_{0.95,LVL}$ = 1.3*1.09*1.15 = 1.63 for LVL.

The resulting strength hierarchy is:

F_{Ed} = 52.0 kN $\leq F_{Rd,sawn}$ = 8x6.86 = 54.89 kN

$\leq \gamma_{Rd,sawn} * F_{d,sawn}$ = 1.85*54.89 kN = 101.61 kN

$\leq F_{BR,d}$

F_{Ed} = 52.0 kN $\leq F_{Rd,LVL}$ = 7x7.50 = 52.50 kN

$\leq \gamma_{Rd,LVL} * F_{d,LVL}$ = 1.63*52.50 kN = 85.66 kN

$\leq F_{BR,d}$

This example illustrates how designers can estimate the expected overstrength, γ_{Rd}, of any timber material and connection layout with relatively high confidence by using a combination of stress-graded material, accurate material modelling, and the analytical overstrength derivation method presented in this paper.

5 Conclusions

- A simple procedure to analytically determine overstrength was presented.
- Analytical model overstrength is caused by conservative model assumptions and the difference between the nominal steel strength, $f_{y,nom}$, and the 5th percentile steel strength, $f_{y,0.05}$. As semi-empirical analytical models in design codes are calibrated on test data, γ_{an} is generally small. The overstrength factor was calculated for dowelled connections making use of timber materials with $\rho_k \in [300,600]$ kg/m^3, dowel diameters $d \in [6,30]$ mm, and side-member thickness to dowel diameter ratios $t/d \in [2,10]$ as: $\gamma_{an,S235}$ = 1.10, $\gamma_{an,S355}$ = 1.08, $\gamma_{an,Grade300}$= 1.03, and $\gamma_{an,Grade500}$ = 1.02 for S235, S355, Grade300 and Grade500 steel, respectively.
- Material overstrength factors for different steel grades were derived:
  $\gamma_{0.95,S235}$ = 1.15, $\gamma_{0.95,S355}$ = 1.10, $\gamma_{0.95,Grade300}$ = 1.13, $\gamma_{0.95,Grade500}$ = 1.10
  While the value for Grade300 and Grade500 were based on an allowable range of f_y given in AS/NZS 4671:2001, the values for S235 and S355 are based on experimental data with small sample numbers (26 and 19, respectively). These should therefore be validated with larger sample sizes.
- The upper bound material overstrength factors for New Zealand ungraded sawn *radiata pine* and graded LVL were calculated based on the information provided by suppliers: $\gamma_{0.95,sawn}$ = 1.51 and $\gamma_{0.95,LVL}$ = 1.17-1.19. For European timber grades, the overstrength factors were calculated based on the density ranges given in EN 338:2016: $\gamma_{0.95,C20}$ = 1.36 for C20 and $\gamma_{0.95,C24}$ = 1.24 for C24.

It is apparent that timber strength grading results in lower variability in material strength and smaller overstrength factors.

- Significant unexpected overstrength can be introduced from delivery of material that is stronger than specified – both for the steel fasteners and the timber itself. This is an issue that designers need to be aware of and that needs to be raised with suppliers.

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7 References

Australian/New Zealand Standard, AS/NZS 4357.0:2005. (2005): Structural Laminated Veneer Lumber. Standards Australia, GPO Box 476, Sydney, NSW 2001 and Standards New Zealand, Private Bag 2439, Wellington 6140

Australian/New Zealand Standard, AS/NZS 4671:2001. (2001): Steel reinforcing materials. Standards Australia, GPO Box 476, Sydney, NSW 2001 and Standards New Zealand, Private Bag 2439, Wellington 6140

Brühl, F., Schänzlin, J., & Kuhlmann, U. (2014): Ductility in Timber Structures: Investigations on Over-Strength Factors. In Materials and Joints in Timber Structures (pp. 181-190). Springer, Dordrecht.

Dickof, C., Stiemer, S. F., Bezabeh, M. A., & Tesfamariam, S. (2014). CLT–Steel Hybrid System: Ductility and Overstrength Values Based on Static Pushover Analysis. Journal of Performance of Constructed Facilities, 28(6)

EN 338:2016 (2016). Structural timber. Strength classes. European Committee for Standardization, Brussels.

Eurocode 3 (2008): Design of steel structures - Part 1-1: General and rules for buildings. CEN. (EN 1993-1-1).

Eurocode 5 (2004/2014): Design of timber structures - Part 1-1: General and rules for buildings. CEN. (EN 1995-1-1).

Franke, S., & Quenneville, P. (2011): Bolted and dowelled connections in radiate pine and laminated veneer lumber using the European yield model. Australian Journal of Structural Engineering.

Gavric, I., Fragiacomo, M., & Ceccotti, A. (2015): Cyclic behavior of typical screwed connections for cross-laminated (CLT) structures. European Journal of Wood and Wood Products, 73(2), 179–191

Guttmann, I. (1970): Statistical tolerance regions: classical and Bayesian.

Jorissen, A., & Fragiacomo, M. (2011): General notes on ductility in timber structures. Engineering Structures, 33(11), 2987–2997.

Karmazínová, M. & Melcher, J. (2012): Influence of steel yield strength value on structural reliability. In Proceedings of the 7th WSEAS International Conference on Continuum Mechanics "CM'12 (pp. 441-446).

Leicester, R.H. (1986): Confidence in estimates of characteristic values. Proceedings of 19th CIB W18 Meeting, Firenze, Italy

Ottenhaus, L.-M., Li, M., Smith, T., & Quenneville, P. (2016): Ductility and Overstrength of Dowelled LVL and CLT Connections Under Cyclic Loading. In Proceedings of the WCTE 2016 World Conference on Timber Engineering, Vienna / Austria, August 22-25, 2016

Ottenhaus, L.-M., Li, M., Smith, T., & Quenneville, P. (2017): Overstrength of dowelled CLT connections under monotonic and cyclic loading. Bulletin of Earthquake Engineering.

Ottenhaus, L.-M., Li, M., Smith, T. (2018): Overstrength of large-scale dowelled connections in CLT. Proceedings of the 2018 NZSEE Conference, New Zealand Society of Earthquake Engineering, Auckland, April 13-15, 2018.

Ranta-Maunus, A. (2007): Strength of Finnish grown timber. VTT Technical Research Centre of Finland.

Sandhaas, C., & van de Kuilen, J. W. G. (2017): Strength and stiffness of timber joints with very high strength steel dowels. Engineering Structures, 131, 394–404.

Smith, I., & Foliente, G. (2002): Load and Resistance Factor Design of Timber Joints: International Practice and Future Direction. Journal of Structural Engineering, 128(1), 48–59.

Uibel, T., & Blaß, H. J. (2014): Joints with Dowel Type Fasteners in CLT Structures Test material. Focus Solid Timber Solutions - European Conference on Cross Laminated Timber (CLT), 119–134.

Vogt, T., Hummel, J., Schick, M., & Seim, W. (2014): Experimentelle Untersuchungen für innovative erdbebensichere Konstruktionen im Holzbau. Bautechnik, 91(1), 1–14.