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Industry CPD

Designing LVL beams with connections

This CPD module, sponsored by Metsä, introduces readers to laminated veneer lumber (LVL), before summarising the design of an LVL beam with connections.

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1 hour of verifiable CPD

Why timber is an optimal choice

Timber stands out as one of the few natural building materials, offering many advantages. Generally non-toxic, timber does not emit chemical vapours within buildings and is safe to handle and touch. As timber ages, it does so naturally, avoiding breakdown into environmentally harmful substances. Always check the Environmental Product Declaration (EPD) for CO_2 emissions at the end of the product's life. Ensure the EPD is verified by an accredited body, such as the International EPD System[1], for compliance with ISO 14025 and EN 15804 standards.

When grown responsibly, timber is a sustainable resource. It has been used as a building material for thousands of years and continues to be cultivated in forests and plantations. With responsible replanting practices, timber will remain a renewable material. Always check whether the timber comes from sustainable forests. There are certification programmes such as the Forest Stewardship Council (FSC)[2] and Programme for the Endorsement of Forest Certification (PEFC)[3].

As trees grow, they draw carbon from the atmosphere, storing it within their structure instead of allowing it to contribute to the greenhouse effect. Incorporating timber in buildings captures this carbon for as long as the structure stands. Additionally, manufacturing engineered wood products (EWPs) requires minimal energy. When sourcing timber products, always look for an EPD, including a life cycle analysis, approved by an accredited body. For instance, Metsä Wood Kerto meets these standards[4], as verified by Environdec.

↗FIGURE 1: LVL was used for roof of Hurlingham Club, London

Timber is also energy-efficient during installation, given its light weight, which can reduce foundation requirements. Using timber in prefabricated panels or modular elements decreases on-site waste and significantly accelerates installation. Off-site construction methods are increasingly vital as the industry faces a skilled labour shortage.

Timber's insulation properties surpass those of many other building materials, making it a common choice for Passivhaus construction. For example, Kerto LVL (laminated veneer lumber) boasts a thermal conductivity of just $\lambda = 0.13$W/(mK).

Sustainable strength and modern innovation

The resurgence of timber use is driven not only by its sustainability as a material, but also by recent innovations and a deeper understanding of EWP design. Advances in glueing techniques now allow for the production of larger and more robust timber sections, with rigorous testing to ensure that adhesives are safe for both people and the environment, and that they do not affect fire resistance design or spread of flame.

Our understanding of timber's fire behaviour has also evolved, enabling us to design with fire resistance in mind, using charring rates to predict performance. For spread of flame, we can apply a range of fire retardants on site or use plasterboard as a protective layer.

Designing any structure for fire performance requires consideration of the spread of

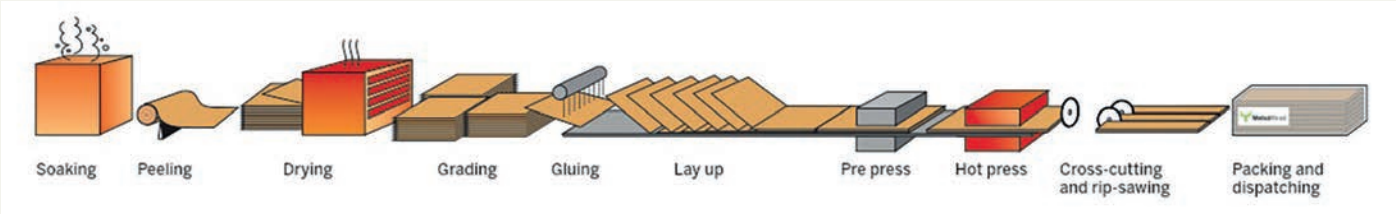


FIGURE 2: LVL manufacturing process

flame, insulation, and structural resistance to ensure that the design complies with building regulations in these three areas. Compliance can be demonstrated through testing or by designing to the Eurocodes.

EWPs can achieve greater spans than previously thought. For instance, the Hurlingham Club roof **(Figure 1)** spans 12.5m, incorporating a green roof and LVL stress skin panels with perforated ceilings for enhanced sound absorption[5].

What is LVL?

LVL is one of the strongest EWPs on the market, owing to a unique production process that disperses knots and fissures throughout the material. It was developed in the 1970s.

The manufacturing process **(Figure 2)** begins with logs being soaked in 40°C water for 24 hours, then debarked and cut into sections approx. 2.5m in length. These sections are rotary peeled into veneers about 2.5m wide and 3.0m long. The veneers are then dried, stress-graded, and layered to the desired thickness, with phenol-formaldehyde glue applied between each sheet. Once glued, all layers are hot-pressed to create a dense, durable product. Finally, the LVL is cut to size, packed and shipped.

LVL is manufactured worldwide using various wood species. Metsä Wood, for instance, produces Kerto LVL from spruce[6].

Kerto is manufactured according to BS EN 14374:2004, either with all veneers in the same direction (Kerto S, LVL 48p; **Figure 3a**) or with 20% perpendicular veneers (Kerto Q, LVL 36c; **Figure 3b**). Grade specification is available in the *LVL Handbook Europe*[7].

Characteristic values for strength and stiffness can be found in the Declarations of Performance (DOP) available on the Metsä Wood website[8,9].

The strength and stiffness of Kerto LVL depend on the grain direction of the material. **Table 1** presents the minimum characteristic values for Kerto S, necessary for designing beams according to BS EN 1995-1-1 2004[10].

The high strength and flexural rigidity of Kerto S LVL 48p make it an ideal material for beam applications. Its characteristic bending strength edgewise, at 44N/mm², is nearly double that of C24 timber, which has a bending strength of 24N/mm² (BS EN 338:2016[11]).

Kerto Q LVL 36c is more homogeneous and stable in terms of shrinkage, although its bending stiffness is lower than that of LVL 48p. However, Kerto Q's compressive strength matches that of C30 concrete at 26MPa. This makes Kerto Q suitable for hybrid wall structures with concrete[12] or as racking steel support, potentially reducing the need for steel bracing[13].

Table 1: Characteristic strength and rigidity of Kerto S LVL 48p

Bending strength	
Edgewise	44.0
Flatwise	50.0
Tension strength	
Parallel	35.0
Perp-edge	0.8
Compression strength	
Parallel	35.0
Perp-edge	6.0
Perp-flat	1.8
Shear strength	
Edgewise	4.2
Flatwise	2.3
Modulus of elasticity	
Parallel-5th per	11 600
Mean value	13 800
Modulus of rigidity	
Mean value	600
Density	510

Example calculation: Single-spanning exposed LVL beam with concealed hanger connection

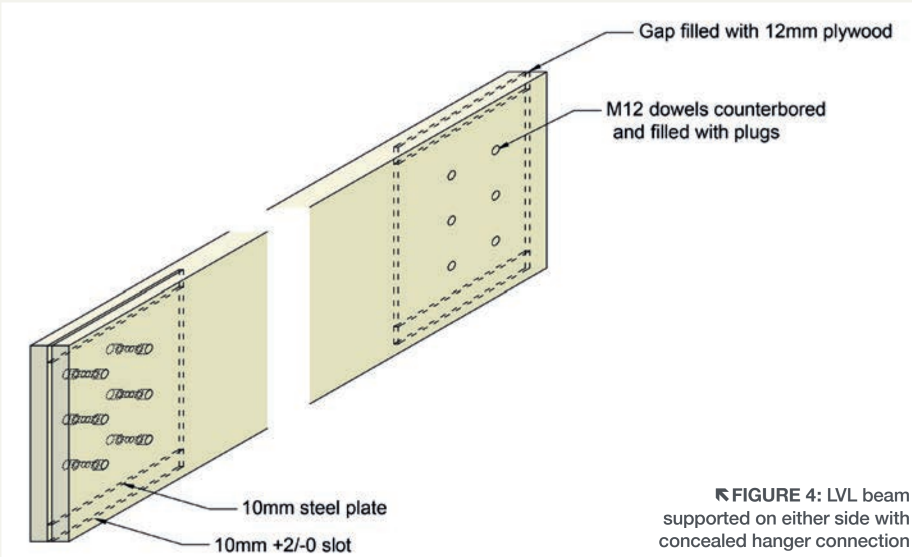
The following example demonstrates the design of a single LVL beam supported on both sides with a concealed hanger connection **(Figure 4)** according to BS EN 1995-1-1:2004[10,14]. The beam is designed for 30 minutes' fire resistance as per BS EN 1995-1-2:2004[15].

The example also includes the connection design, with the connection based on a concealed hanger design which features a 10mm embedded metal plate. Bolts are counterbored with additional Kerto plugs to achieve 30 minutes' fire resistance.

It is important to note that fire resistance in minutes should always be specified by the fire engineer, who can assess the height of the building and the means of escape for the number of people occupying the building. However, this does not take away the need for the structural engineer to exercise the due diligence, especially if there is no fire engineer in place.

The open-plan, two storey office building,

FIGURE 3: Kerto LVL products
a) Kerto S
b) Kerto Q



↖ **FIGURE 4:** LVL beam supported on either side with concealed hanger connection

with a ground-floor area less than 1000m^2, should have a minimum fire resistance of 30 minutes, as per Approved Document B[16]. BS 9999:2017[17] gives a value of 15 minutes even without sprinklers.

Beam design, including fire resistance calculations, can be carried out using Finnwood software[18].

In this scenario, the beam is assumed to support a typical office load of 2.5kN/m^2 (category B1) as per BS EN 1991-1:2002 and its corresponding National Annex[19,20]. The permanent load of the floor includes:

- →| Chipboard deck (22mm thickness, capacity excluded): 0.16kN/m^2
- →| Plasterboard (15mm thickness): 0.14kN/m^2
- →| Self-weight of Kerto LVL 48p (90mm × 400mm, 18.36kg/m for spacing of 0.6m): 0.31kN/m

Loading and geometry are given in **Figure 5**.

Acting forces as per NA to BS EN 1990:2002 Equation (6.10a and 6.10b):

Permanent combination

$$Q_{per} = \gamma_{Gj} \cdot G_{kj} = 0.778\text{kN/m}$$

Medium combination

$$Q_{med1} = \gamma_{Gj} \cdot G_{kj} + \psi_0 \cdot \gamma_{Q1} \cdot Q_{k1} = 2.353\text{kN/m}$$

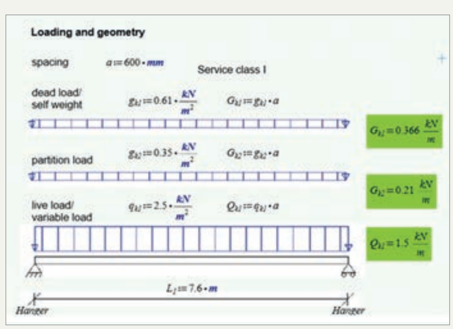
$$Q_{med2} = \xi \cdot \gamma_{Gj} \cdot G_{kj} + \gamma_{Q1} \cdot Q_{k1} = 2.969\text{kN/m}$$

$$Q_{med} = \max\left(Q_{med1}, Q_{med2}\right) = 2.969\text{kN/m}$$

The floor beam is fully restrained against torsional buckling – this is because the chipboard is connected directly to the beam on the compressive side and creates a diaphragm.

The calculation can be divided into two limit states:

1) Ultimate limit state (ULS) as per BS EN 1990:2023[21], Section 5.3. This limit state represents a failure of the structure. The checks in the example include bearing, shear, bending and connection design.



↗ **FIGURE 5: Loading and geometry of example beam**

2) Serviceability limit state (SLS) as per BS EN 1990:2023, Section 5.3. This is a limit state which concerns the functionality of the structure, comfort of the people and appearance of the construction. For timber, this includes instantaneous deflection, final deflection (with the creep) and deformation of the connection with slip modulus (excluded from this example).

Each limit state is checked according to BS EN 1995-1-1 as a comparison between the design resistance and design action.

ULS design

Design acting forces.

Design bending moment.

Permanent

$$M_{per} = \frac{Q_{per} \cdot L_1^2}{8} = 5.614\text{kN} \cdot \text{m}$$

Medium

$$M_{med} = \frac{Q_{med} \cdot L_1^2}{8} = 21.438\text{kN} \cdot \text{m}$$

Design shear

According to BS EN 1995-1-1, Section 6.1.7, the design shear value is defined at distance *h* from inside edge of support. As this is the maximum, shear occurs in solid section.

Permanent

$$V_{per} = Q_{per} \cdot \left(\frac{L_1}{2} - \frac{b}{2} - h\right) = 2.609\text{kN}$$

Medium

$$V_{med} = Q_{med} \cdot \left(\frac{L_1}{2} - \frac{b}{2} - h\right) = 9.962\text{kN}$$

Design shear for connection

Permanent

$$V_{b.perm} = Q_{per} \cdot \left(\frac{L_1}{2}\right) = 2.955\text{kN}$$

Medium

$$V_{b.med} = Q_{med} \cdot \left(\frac{L_1}{2}\right) = 11.283\text{kN}$$

Table 2: k_{mod} for LVL

Service class	Permanent action	Long-term action	Medium-term action	Short-term action	Instantaneous action
1	0.60	0.70	0.80	0.90	1.10
2	0.60	0.70	0.80	0.90	1.10
3	0.50	0.55	0.65	0.70	0.90

Design resistance is calculated with γ_M, the partial factor for material properties, also accounting for model uncertainties and dimensional variations, and with k_{mod}, modification factor, for the duration of load and moisture content **(Table 2)**.

Where γ_M is 1.2 for LVL material, k_{mod} is dependent on material, load duration and moisture content (service class), as per BS EN 1995-1-1, Table 3.1. In this example, we assumed k_{sys} =1 (the beam is considered as acting alone not as a system; see BS EN 1995-1-1, Section 6.6 *System strength*).

Shear design resistance
Characteristic shear resistance
BS EN 1995-1-1, Section 2.4.1, Eqn 2.14

$$V_{ky} = \frac{2}{3} \cdot f_{v0k} \cdot A_{total}$$

Permanent

$$V_{ryperm} = \frac{\left(k_{modperm} \cdot V_{ky}\right)}{\gamma_M} \cdot k_{sys} = 50.4\text{kN}$$

Medium

$$V_{rymed} = \frac{\left(k_{modmed} \cdot V_{ky}\right)}{\gamma_M} \cdot k_{sys} = 67.2\text{kN}$$

Bending design resistance
Characteristic bending resistance including size effect; see DOP.
BS EN 1995-1-1, Section 2.4.1, Eqn 2.14

$$M_{ky} = \min\left(1.2, \left(\frac{300}{h_1}\right)^{0.12}\right) \cdot f_{m0k} \cdot Z_y = 102.017\text{m} \cdot \text{kN}$$

Permanent

$$M_{ryperm} = \frac{\left(k_{modperm} \cdot M_{ky}\right)}{\gamma_M} \cdot k_{sys} = 51.008\text{kN} \cdot \text{m}$$

Medium

$$M_{rymed} = \frac{\left(k_{modmed} \cdot M_{ky}\right)}{\gamma_M} \cdot k_{sys} = 68.011\text{kN} \cdot \text{m}$$

Shear utility rate
BS EN 1995-1-1, Section 6.1.7, Eqn 6.13

$$\frac{V_{per}}{V_{ryperm}} = 5.176\%$$

$$\frac{V_{med}}{V_{rymed}} = 14.824\%$$

$$\max\left(\frac{V_{per}}{V_{ryperm}}, \frac{V_{med}}{V_{rymed}}\right) = 14.824\%$$

Bending utility rate
BS EN 1995-1-1, Section 6.1.6, Eqn 6.11

$$\frac{M_{per}}{M_{ryperm}} = 11.007\%$$

$$\frac{M_{med}}{M_{rymed}} = 31.522\%$$

$$\max\left(\frac{M_{per}}{M_{ryperm}}, \frac{M_{med}}{M_{rymed}}\right) = 31.522\%$$

SLS design
Design acting forces as per BS EN 1990:2023, Section 5.3

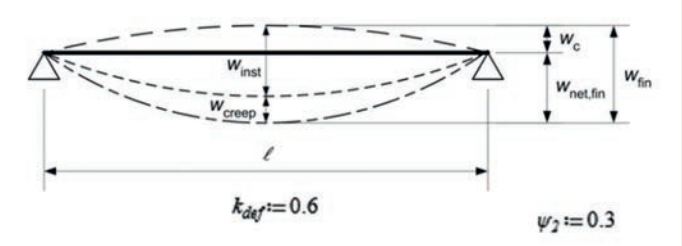
$$G_k = G_{k1} + G_{k2} = 0.576\text{kN/m}$$

$$Q_k = Q_{k1} = 1.5\text{kN/m}$$

According to BS EN 1990, the deflection limit should be established by agreement between the designer and the client. The NA to BS EN 1995 does not specify a limit for instantaneous deflection, but does outline requirements for final deflection and vibration. In this example, we have applied the traditional limit from the 'old BS 5268' standard of *L*/333.

Final deflection
Final deflection accounts for both immediate and long-term deflection due to creep, representing the deflection of timber over time.

To design it, you need k_{def}, deformation factor, Table 3.2 of BS EN 1995-1-1 and ψ_2, factor for the quasi-permanent value of a variable action.



UK NA to BS EN 1990, Table NA.A1.1

$$w_{finalperm} = \left(\frac{5 \cdot G_k \cdot L_1^4}{384 \cdot E_{mean} \cdot I_y} + \frac{G_k \cdot L_1^2}{8 \cdot G_{mean} \cdot A_{total}}\right) \cdot \left(1 + k_{def}\right)$$
$$= 6.352\text{mm}$$

$$w_{finalinst} = \left(\frac{5 \cdot Q_k \cdot L_1^4}{384 \cdot E_{mean} \cdot I_y} + \frac{Q_k \cdot L_1^2}{8 \cdot G_{mean} \cdot A_{total}}\right) \cdot \left(1 + \psi_2 \cdot k_{def}\right)$$
$$= 12.199\text{mm}$$

$$\frac{L_1}{250} = 30.4\text{mm}$$

$$w_{final} = w_{finalperm} + w_{finalinst} = 18.551\text{mm}$$

Note: deflection has one part bending and one part shear deflection.

Vibration analysis
UK NA to BS EN 1995-1-1, Table NA.6, NA 2.7

Plate stiffness: $EI_b = 8500 \cdot 10^6$

Floor width in [m]: $b_2 = 5$

Damping factor
As per BS EN 1995-1-1:2004, Section 7.3.1(3), a value of $\xi = 0.02$ has been found appropriate for typical UK floors.

Floor mass (self-weight only): $m_a = 40\text{kg/m}^2$

$$EI_l = \frac{E_{mean} \cdot I_y}{a} = \left(1.104 \cdot 10^{13}\right)\frac{\text{N} \cdot \text{mm}^2}{\text{m}}$$

$$k_{amp} = \min\left(1.2, 1 + \frac{12 \cdot E_{mean} \cdot I_y}{L_1^2 \cdot G_{mean} \cdot A_{total}}\right) = 1.064$$

$$k_{strut} = 1.0$$

Fundamental frequency
The fundamental frequency f of a floor must be above 8Hz.

BS EN 1995-1-1, Section 7.3.3, Eqn 7.5

$$f_1 = \left|\frac{\pi}{2 \cdot L_1^2 \cdot \text{m}^2} \cdot \sqrt{\frac{EI_l}{m_a}}\right| = 14.287\text{Hz}$$

Point load deflection, *a*, under 1kN
UK NA to BS EN 1995-1-1, NA. 2.6, Table NA.6

$$a_{req} = \text{if}\left(L_1 > 4, \frac{16500}{\left(L_1 \cdot 1000\right)^{1.1}}, 1.8\right) = 0.888$$

$$k_{dist} = \max\left(k_{strut} \cdot \left(0.38 - 0.08 \cdot \ln\left(\frac{14 \cdot EI_b}{s^4}\right)\right), 0.30\right)$$
$$= 0.387$$

$$b_v = \text{if}\left(a_{req} \le 1, 180 - 60 \cdot a_{req}, 160 - 40 \cdot a_{req}\right)$$
$$= 126.698$$

$$a_{actual} = \left|\frac{1000 \cdot k_{dist} \cdot L_1^3 \cdot k_{amp} \cdot \text{kN} \cdot \text{m}^2}{48 \cdot E_{mean} \cdot I_y}\right|$$
$$= 0.568$$

Velocity response, *v*

$$v_{req} = b_v^{\left(\frac{f_1 \cdot \zeta}{\text{Hz}}\right) - 1} = 0.0315$$

$$n_{40} = \left(\left(\left(\frac{40 \cdot \text{Hz}}{f_1}\right)^2 - 1\right) \cdot \left(\frac{b_2}{L_1}\right)^4 \cdot \frac{EI_l \cdot \text{m}}{EI_b \cdot \text{N} \cdot \text{mm}^2}\right)^{0.25}$$
$$= 6.38678$$

$$v_{actual} = \frac{4 \cdot \left(0.4 + 0.6 \cdot n_{40}\right)}{m_a \cdot b_2 \cdot L_1 \cdot \frac{\text{m}^2}{\text{kg}} + 200} = 0.0098$$

Fire design
Due to its good thermal insulation properties, a layer of char is created when timber burns. This helps to protect and maintain the strength and structural integrity of the wood inside. Timber can, therefore, often be used in large sections in unprotected situations where non-combustible materials such as steel would require special fire protection.

The fire design for timber is based on the actions in fire as per BS EN 1991-1-2-2002[22]. Charring and cross-sectional properties in fire should be determined using the reduced cross-section method of BS EN 1995-1-2[15], Section 4.2.2. and the accompanying annex NA to BS EN 1995-1-2:2004[23].

Where indirect fire actions need not be explicitly considered, effects of actions may be determined by analysing the structure for combined actions according to Section 4.3.1 for $t = 0$ only. These effects of actions $E_{fi,d}$ may be applied as constant throughout fire exposure.

$$E_{fi,d,t} = E_{fi,d} = \eta_{fi} \cdot E_d$$

η_{fi} is a reduction factor which should be evaluated using expression 2.9 of BS EN 1995-1-2, but is not permitted to have a value less than 0.4, for load combinations (6.10a) and (6.10b) in EN 1990:2002, as the smallest value given by the following two expressions 2.9a and 2.9b as per BS EN 1995-1-2 2004.

$$\eta_{Fi1} = \frac{G_{kj} + \Psi_{1.1} \cdot Q_{k1}}{\gamma_{Gj} \cdot G_{kj} + \gamma_Q \cdot Q_{k1}} = 0.438$$

$$\eta_{Fi2} = \frac{G_{kj} + \Psi_{1.1} \cdot Q_{k1}}{\xi \cdot \gamma_{Gj} \cdot G_{kj} + \gamma_Q \cdot Q_{k1}} = 0.447$$

$$\eta_{Fi} = \min\left(\eta_{Fi1}, \eta_{Fi2}\right) = 0.438$$

$Q_{k,1}$ is the characteristic value of the leading variable action.
G_k is the characteristic value of the permanent action.
γ_G is the partial factor for permanent actions.
$\gamma_{Q,1}$ is the partial factor for variable action 1.
ξ is a reduction factor for unfavourable permanent actions G.
ψ_{fi} is the combination factor for frequent values of variable actions in the fire situation, given by $\psi_{1,1}$ (see BS EN 1991-1-1).

The driving action in our design is medium action

$$E_{dfi} = \eta_{Fi} \cdot Q_{med} = 1.3\text{kN/m}$$

The cross-section properties must be calculated using the notional charring rate, the magnitude of which accounts for the effect of corner roundings and fissures, and should be taken as constant with time. The notional design charring depth should be calculated as:

$$d_{char,n} = \beta_n t$$

As per Certificate No EUFI29-20000676-C/EN point 8, β_n for Kerto products is 0.70mm/min.

$$b_f = 90\text{mm} - 2 \cdot d_{charn} = 48\text{mm}$$

$$h_f = 400\text{mm} - d_{charn} = 379\text{mm}$$

$$A_{totalf} = b \cdot h = \left(3.6 \cdot 10^4\right)\text{mm}^2$$

$$Z_{yf} = \frac{b_f \cdot h_f^2}{6} = 1149.128\text{m} \cdot \text{mm}^2$$

$$I_{yf} = \frac{b_f \cdot h_f^3}{12} = 217.76\text{m}^2 \cdot \text{mm}^2$$

Acting forces in fire

Bending moment in fire

$$M_{\text{fire}} = M_{\text{med}} \cdot \eta_{\text{Fi}} = 9.389\text{kN} \cdot \text{m}$$

Shear in fire

$$V_{\text{fire}} = V_{\text{med}} \cdot \eta_{\text{Fi}} = 4.363\text{kN}$$

Design values of material properties and resistances in fire

For verification of mechanical resistance, the design values of strength and stiffness properties shall be determined from:

$$f_{\text{d,fi}} = k_{\text{mod,fi}} \frac{f_{20}}{\gamma_{\text{M,fi}}}$$

$$S_{\text{d,fi}} = k_{\text{mod,fi}} \frac{S_{20}}{\gamma_{\text{M,fi}}}$$

where:
$f_{d,fi}$ is the design strength in fire
$S_{d,fi}$ is the design stiffness property (modulus of elasticity $E_{d,fi}$ or shear modulus $G_{d,fi}$) in fire
f_{20} is the 20% fractile of a strength property at normal temperature
S_{20} is the 20% fractile of a stiffness property (modulus of elasticity or shear modulus) at normal temperature
$k_{mod,fi}$ is the modification factor for fire
$\gamma_{M,fi}$ is the partial safety factor for timber in fire.

The recommended partial safety factor for material properties in fire is $\gamma_{M,fi}$ = 1.0 as per the NA to BS EN 1995-1-2.

The 20% fractile of a strength or a stiffness property should be calculated as:

$$f_{20} = k_{\text{fi}} f_{\text{k}}$$

$$S_{20} = k_{\text{fi}} S_{05}$$

where:
f_{20} is the 20% fractile of a strength property at normal temperature
S_{20} is the 20% fractile of a stiffness property (modulus of elasticity or shear modulus) at normal temperature
S_{05} is the 5% fractile of a stiffness property (modulus of elasticity or shear modulus) at normal temperature
k_{fi} = 1.1 as per Table 2.1 of BS EN 1995-1-2
$k_{mod,fi}$ =1.0 for the reduced cross-section method as per the NA to BS EN 1995-1-2.

Shear design resistance in fire

$$V_{\text{dfi}} = \frac{2}{3} \cdot \frac{k_{\text{fi}} \cdot k_{\text{mod fi}} \cdot f_{\text{v0k}}}{\gamma_{\text{Mfi}}} \cdot A_{\text{fi}} = 38.956\text{kN}$$

Bending design resistance in fire

$$M_{\text{dfi}} = \min\left(1.2, \left(\frac{300}{h_1}\right)^{0.12}\right) \cdot \frac{k_{\text{fi}} \cdot k_{\text{mod fi}} \cdot f_{\text{m0k}}}{\gamma_{\text{Mfi}}} \cdot Z_{\text{y}}$$
$$= 112.218\text{m} \cdot \text{kN}$$

Shear utility rate in fire

$$\frac{V_{\text{fire}}}{V_{\text{dfi}}} = 11.2\%$$

Bending utility rate in fire

$$\frac{M_{\text{fire}}}{M_{\text{dfi}}} = 8.367\%$$

Connection design according to BS EN 1995-1-1, Section 8.2.3

Steel to timber connections

To allow for the fire resistance design consideration, we assume that the connectors are countersunk and a Kerto plug-in is used to provide 30 minutes of fire resistance. In this case, as the element will be completely covered, we need to use the acting forces for the cold design. The charring rate for one side of Kerto is β_{char0} = 0.65mm/min.

$$d_{\text{char0}} = \beta_0 \cdot t = 19.5\text{mm}$$

A steel plate is inserted into the Kerto S, with dowels offset by 19.5mm to provide fire resistance. Additionally, we account for the washer and bolt heads, allowing approx. 10mm on each side.

$$t_1 = b - 10\text{mm} - d_{\text{char0}} \cdot 2 - 20\text{mm}$$
$$= 21\text{mm}$$

Fastener diameter, d = 12
Thickness of steel plate, t_s = 10
Fastener strength class, B = 4.6

See BS EN 1995-1-1:2004, Annex 1, Figure 8.3.

BS EN mechanical connection design

The BS EN standard for mechanical connection design is based on plastic values for embedment strength and yielding moment, originally published by Johansen in 1949. Each equation corresponds to a specific failure mode. In our case, we apply failure modes *f*, *g* and *h* for plates of any thickness in double shear.

For failure modes involving the rope effect, the withdrawal capacity of the connectors is considered; however, it is limited to a specific percentage of the Johansen value (rope effect), depending on the type of connector. For example, screws allow for 100% of the rope effect, while bolts are limited to 25% (refer to table in BS EN 1995-1-1).

In our example, the internal diameter of the washer = 12mm; outer diameter of the washer = 22mm; Ac_{90} = 267mm²; A_{eff} = 118mm².

$$f_{\text{c90k}} = 6\text{N/mm}^2$$

$$F_{\text{axRk}} = \min\left(3 \cdot A_{\text{c90}} \cdot f_{\text{c90k}}, A_{\text{eff}} \cdot f_{\text{uk}}\right)$$
$$= 4.807\text{kN}$$

$$k_{90} = 1.30 + 0.015 \cdot d = 1.48$$

$$f_{\text{h0k1}} = 0.082 \cdot (1 - 0.01 \cdot d) \cdot \rho_{\text{k1}}$$
$$= 34.637\text{MPa}$$

$$f_{\text{hk1}} = \frac{f_{\text{h0k1}}}{k_{90} \cdot \sin(\alpha)^2 + \cos(\alpha)^2}$$
$$= 23.403\text{N/mm}^2$$

$$f_{h0k2} = 0.082 \cdot (1 - 0.01 \cdot d) \cdot \rho_{k2}$$
$$= 34.637\text{MPa}$$

Steel plate of any thickness as a central member of double shear connection

Expression 8.11f

$$F_{vRk1} = f_{hk1} \cdot t_1 \cdot d$$

Expression 8.11g

$$F_{vRk2} = f_{hk} \cdot t_1 \cdot d \cdot \left(\sqrt{2 + 4 \cdot \frac{M_{yRk} \cdot \text{mm}^3}{f_{hk} \cdot d \cdot t_1^2}} - 1 \right)$$
$$+ \min\left(\frac{F_{axRk}}{4}, 25\% \cdot \left(f_{hk} \cdot t_1 \cdot d \cdot \left(\sqrt{2 + 4 \cdot \frac{M_{yRk} \cdot \text{mm}^3}{f_{hk} \cdot d \cdot t_1^2}} - 1 \right) \right) \right)$$
$$= 7.785\text{kN}$$

Expression 8.11h

$$F_{vRk3} = 2.3 \cdot \sqrt{M_{yRk} \cdot f_{hk} \cdot d \cdot \text{mm}^3}$$
$$+ \min\left(\frac{F_{axRk}}{4}, 25\% \cdot \left(2.3 \cdot \sqrt{M_{yRk} \cdot f_{hk} \cdot d \cdot \text{mm}^3} \right) \right)$$
$$= 11.879\text{kN}$$

The characteristic load bearing capacity per one dowel is

$$F_{vRk} = \min(F_{vRk1}, F_{vRk2}, F_{vRk3}) = 5.898\text{kN}$$

The recommended partial factor for connections is

$$\gamma_m = 1.3$$

The design capacity for the medium load case is

$$F_{vRdmed} = \frac{k_{modmed}}{\gamma_m} \cdot F_{vRk} = 3.629\text{kN}$$

The design capacity for the permanent load case is

$$F_{vRdperm} = \frac{k_{modperm}}{\gamma_m} \cdot F_{vRk} = 2.722\text{kN}$$

Medium-term acting force

$$V_{b.med} = 11.283\text{kN}$$

Permanent reaction

$$V_{b.perm} = 2.955\text{kN}$$

Minimum required connectors

$$n_{req} = \left(\max\left(\frac{V_{b.med}}{F_{vRdmed}}, \frac{V_{b.perm}}{F_{vRdperm}} \right) \right) = 3.109$$

Active dowels

As per BS EN 1995-1-1:2004, Annex 1, Point 8.5 and 8.6.

Minimum distance, a1

$$a_1 = (4 + 3 \cdot \cos(\alpha)) \cdot d = 48\text{mm}$$

Rows of connectors perp to the grain

$$m_r = 2$$

Rows of connectors parallel to the grain

$$n_r = 3$$

For one row of *n* bolts parallel to the grain direction, the load-carrying capacity parallel to the grain should be calculated using the effective number of bolts n_{ef}

$$n_{ef} = \min\left(n_r, n_r^{0.9} \cdot \sqrt[4]{\frac{a_1}{13 \cdot d}} \right) = 2.002$$

Resisting force for the selected bolts group

$$F_{act} = F_{vRdmed} \cdot n_{ef} \cdot m_r = 14.531\text{kN}$$

Minimum required bolts must be two rows of 3No. M12 grade 4.6.

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