1. GENERAL

Washers with a side length or an external diameter of at least 3d (where d is the diameter of the bolt) and a thickness of at least 0.3d should be used under the head of bolts and nuts. Washers should have a full bearing area.

Bolts should be tightened so that the members fit closely, and they should be re-tightened if necessary when the timber has reached equilibrium moisture content. If re-tightening cannot be done, and there is a possibility that the timber can dry over 5% of its weight after installation of the bolts, only 80% of the calculated capacity of the bolt connection can be utilised.

Bolt holes in timber should have a diameter no more than 1 mm larger than the bolt. Bolt holes in steel plates should have a diameter no more than 2 mm or 1.1d (whichever is greater). If the connection is designed using thick steel plate (t ≥ d) equations and bolt diameter d < 20 mm, the maximum allowed hole in the steel plate should not be more than 1.1d.

Table 1: Strength modification factors for service classes and load-duration classes k_{mod} and partial factors Y_{M} for material properties and resistances.2

<table>
<thead>
<tr>
<th>MATERIAL</th>
<th>SERVICE CLASS</th>
<th>LOAD-DURATION CLASS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>PERMANENT ACTION</td>
</tr>
<tr>
<td>Solid timber, round timber, glued laminated timber, Kerto LVL plywood</td>
<td>1</td>
<td>0.60</td>
</tr>
<tr>
<td>Particleboard EN 312-4* and -5, OSB/2*, Hard fibreboard</td>
<td>1</td>
<td>0.30</td>
</tr>
<tr>
<td>Particleboard EN 312-6* and -7, OSB/3, OSB/4</td>
<td>2</td>
<td>0.20</td>
</tr>
<tr>
<td>Medium fibreboard: MBH, LA*, MBF.HLS, MDF.LA* and MDF.HLS</td>
<td>2</td>
<td>0.20</td>
</tr>
</tbody>
</table>

Partial factors Y_{M} (EN 1995 recommended values and the Finnish NA values)

| Fundamental combinations: | 1.30 | 1.40 |
| Solid and Round timber in general | 1.30 | 1.25 |
| Softwood structural timber, strength class ≥ C35 | 1.25 | 1.20 |
| Kerto LVL | 1.25 | 1.20 |
| Glued laminated timber | 1.25 | 1.20 |
| Plywood, OSB | 1.20 | 1.25 |
| Particle- and fibreboards | 1.30 | 1.25 |
| Connections | 1.30 | according to timber material |
| Accidental combination | 1.00 | 1.00 |

* Can only be used in service class 1

Bolts and steel plates should, where necessary, either be inherently corrosion-resistant or be protected against corrosion.

Table 2: The minimum specification for material protection against corrosion for fasteners. Electroplated zinc coating Fe/Zn classes are according to ISO 2081 and hot-dip coating Z classes according to EN 10346.3 Stainless steel according to EN 10088-1 (grades 1.4401, 1.4301 and 1.4310).4

<table>
<thead>
<tr>
<th>FASTENER</th>
<th>SERVICE CLASS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
</tr>
<tr>
<td>Bolts</td>
<td>None</td>
</tr>
<tr>
<td>Steel plates up to 3 mm thickness</td>
<td>Fe/Zn 12c, Z275</td>
</tr>
<tr>
<td>Steel plates from 3 mm up to 5 mm in thickness</td>
<td>None</td>
</tr>
<tr>
<td>Steel plates over 5 mm thickness</td>
<td>None</td>
</tr>
</tbody>
</table>

Parts that are according to the Finnish national annex are marked with green text or they are given in the endnote. These rules may not apply outside Finland. The equations by RIL 205-1-2009 are generalized from the Eurocode and are on the safe side. Additional general information about connections is also collected from several sources.
Table 3: In EN 1993-1-1, EN 1993-1-2 and EN 1993-1-8 the following partial factors are used according to EN 1993 recommended values and FI NA for structural members, cross sections and connections.

<table>
<thead>
<tr>
<th>MARKING</th>
<th>VALUE (EN 1993)</th>
<th>VALUE FI/NA</th>
</tr>
</thead>
<tbody>
<tr>
<td>YM0</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>YM1</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>YM2</td>
<td>1.25</td>
<td>1.25</td>
</tr>
<tr>
<td>YM3</td>
<td>1.25</td>
<td>1.25</td>
</tr>
<tr>
<td>YM3,sar</td>
<td>1.10</td>
<td>1.10</td>
</tr>
<tr>
<td>YM4</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>YM5</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>YM6,sar</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>YM8</td>
<td>1.10</td>
<td>1.10</td>
</tr>
<tr>
<td>YM,fi</td>
<td>1.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>

3. LOADING

Bolts can be loaded laterally or axially. The loading can also be combined lateral and axial load.

Reductions in cross section should be taken into account when analysing the capacity of timber members.

In compressed Kerto-to-Kerto joints, 2/3 of the perpendicular compression force can be transferred directly through contact from member to member. If the contact surfaces have been CNC-machined, 3/4 of the perpendicular compression force can be transferred directly through contact from member to member. Splitting of the compressed side in sloped connections, such as ridge connections, should be prevented by shaping the end of the member or installing a hard fibreboard or steel plate with a height of about 3/4 of the total height of the connection.

When a force in a connection acts at an angle to the grain, the bolts of laterally loaded joints should ideally be positioned at the compressed side of the member. In these cases there is generally no need to check the tension capacity perpendicular to the grain. See Figure 5.

4. LATERALLY LOADED BOLTS

When calculating the lateral load-capacity of the connection, the capacity of the fastener and block shear in the timber member should be checked. See Figure 1.

Design capacity of the connection:

$$ R_d = \frac{k_{mod} \cdot R_k}{\gamma_M} \quad (1) $$

where $k_{mod}$ is the modification factor for duration of load and moisture content

$\gamma_M$ is the partial factor for connection resistance

When connecting two different materials the smallest value of $k_{mod} / \gamma_M$ should be used.

4.1 TIMBER-TO-TIMBER CONNECTIONS

The characteristic load-carrying capacity for a fastener per shear plane:

$$ R_k = \min\left\{ \frac{0.4 \cdot f_{h,k} \cdot t_u \cdot d \cdot \sqrt{1 + \frac{3 \cdot M_y}{f_{h,k} \cdot d \cdot t_u}}}{2 \cdot \sqrt{M_y \cdot f_{h,k} \cdot d}} \right\} \quad (2) $$

where

$$ t_u = \min\left\{ \frac{t_1 \cdot f_{h,1,k}}{f_{h,k}}, \frac{t_2 \cdot f_{h,2,k}}{f_{h,k}} \right\} \quad (3) $$

$$ f_{h,k} = \min(f_{h,1,k}, f_{h,2,k}, f_{h,3,k}) \quad (4) $$

$t_1$ and $t_2$ are the thicknesses of the outer timber members

$f_{h,1,k}$ and $f_{h,2,k}$ are the characteristic embedment strengths of outer timber members

$f_{h,3,k}$ is the characteristic embedment strength of inner timber member in two shear plane connection

$d$ is the fastener diameter

The characteristic value for the yield moment:

$$ M_y = 0.3 \cdot f_{u,k} \cdot d^{2.6} \quad \text{[Nmm]} \quad (5) $$

where $f_{u,k}$ is the characteristic tensile strength of the bolt, in N/mm$^2$

$d$ is the fastener diameter, in mm
The characteristic embedment strength, at an angle \( \alpha \) to the grain:

\[
f_{h,k} = \frac{f_{h,0,k}}{k_{90} \cdot \sin^2 \alpha + \cos^2 \alpha} \quad [N/mm^2]
\]

where:

\[
f_{h,0,k} = \begin{cases} 0.082 \cdot (1 - 0.01d) \cdot \rho_k & \text{In general} \\ 37 \cdot k_{Q} \cdot (1 - 0.01d) & \text{for Kerto-Q} \end{cases}
\]

\[
k_{Q} = \begin{cases} 1 & \text{for flatwise connections} \\ 1 - \frac{2}{d} \leq 0.87 & \text{for edgewise connections} \end{cases}
\]

\[
k_{90} = \begin{cases} 1.30 + 0.015d & \text{for Kerto-S and Kerto-T} \\ 1.15 + 0.015d & \text{for Kerto-Q} \\ 1.35 + 0.015d & \text{for softwood} \\ 0.90 + 0.015d & \text{for hardwoods} \end{cases}
\]

For Kerto-Q, \( f_{h,k} = f_{h,0,k} \) when \( 45^\circ \leq \alpha \leq 90^\circ \)

\( \rho_k \) is the characteristic timber density, in kg/m\(^3\)

\( d \) is the fastener diameter, in mm

\( \alpha \) is the angle of the load to the grain

For particleboard and OSB the following embedment strength should be used for all loading directions:

\[
f_{h,k} = 50 \cdot d^{-0.6} \cdot t^{0.2} \quad [N/mm^2]
\]

where:

- \( d \) is the fastener diameter, in mm
- \( t \) is the panel thickness, in mm

### 4.3 STEEL-TO-TIMBER CONNECTIONS

The capacity of a steel plate should be checked according EN 1993.

In compressed steel plate connections the buckling length of \( 0.8L \), can generally be used for outside plates, where \( L \) is the distance between the first fasteners at opposite sides of the connection. The buckling does not need to be taken into account for steel plates installed inside a timber member if the expansion of timber members is prevented, for example, by using tie bolts and limiting the size of the slot for the steel plate to maximum of \( 1.25t_t \).

The drying shrinkage perpendicular to the grain direction should be taken into account with steel-to-timber connections.

It should also be taken into account that the load-carrying capacity of steel-to-timber connections with a loaded end may be reduced by failure along the perimeter of the fastener group. There are two types of loaded end failures: block shear and plug shear failure.

The characteristic load-carrying capacity for a thin steel plate, with \( t_t \leq 0.5d \), in single shear:

\[
R_k = \min \left\{ \frac{0.4 \cdot f_{h,k} \cdot t \cdot d}{2 \cdot \sqrt{M_y \cdot f_{h,k} \cdot d}} \right\}
\]

The characteristic load-carrying capacity for a thick steel plate, with \( t_t \geq d \), in single shear:

\[
R_k = \min \left\{ 1.3 \cdot f_{h,k} \cdot t \cdot d \cdot \left[ \frac{2 + 4 \cdot M_y}{f_{h,k} \cdot d \cdot t^2} - 1 \right] \right\}
\]

where:

- \( f_{h,k} \) is the characteristic embedment strength of the timber member, equation (4)
- \( t \) is the thickness of the timber member
- \( d \) is the fastener diameter
- \( M_y \) is the characteristic fastener yield moment, equation (5)

The characteristic load-carrying capacity of connections with a steel plate thickness between a thin and thick plate, where \( 0.5d < t_t < d \), should be calculated by linear interpolation between equations (13) and (14).
The characteristic load-carrying capacity for a steel plate of any thickness as the central member of a double shear connection should be calculated with equation (14) where \( t \) is the smaller thickness of the timber side member.

The characteristic load-carrying capacity for steel plates as the outer member of double shear connection:

\[
R_k = \begin{cases} 
0.5 \cdot f_{h,k} \cdot t \cdot d / \sqrt{M_y} & \text{for } t_i \leq 0.5d \\
2 \cdot \sqrt{M_y} \cdot f_{h,k} \cdot d / t_i & \text{for } t_i \geq d \\
\left( \frac{t_i}{0.5d} + 1 \right) \cdot \sqrt{M_y} \cdot f_{h,k} \cdot d / t_i & \text{for } 0.5d < t_i < d 
\end{cases}
\]

4.4 EFFECTIVE NUMBER OF FASTENERS

For one row of \( n \) fasteners parallel to the grain direction, the load-carrying capacity parallel to the grain should be calculated using the effective number of fasteners \( n_{ef} \):

\[
R_v,d = 2R_{1,d} \quad R_v,d = 4 \min \{ R_{1,d}; R_{2,d} \}
\]

\[
R_v,d = 6 \min \{ R_{1,d}; R_{2,d}; R_{3,d} \}
\]

Figure 2: Calculating the connection capacity of a multiple shear plane steel plate connection. \( R_{1,d} \) is the capacity per shear of a two shear plane timber-steel-timber \((t_u\text{-steel}-t_u)\) connection, \( R_{2,d} \) is the capacity per shear of a two shear plane steel-timber-steel \((t_u\text{-steel}-t_u)\) connection, and \( R_{3,d} \) represents the capacity per shear of a two shear plane timber-steel-timber \((t_u\text{-steel}-t_u)\) connection.
6. BLOCK SHEAR FAILURE

6.1  TIMBER FAILURE CAPACITY OF THE JOINT AREA

The effective number of fasteners is taken into account in the following equations. This method can be used for Kerto-S, Kerto-Q, Kerto-T used flatwise and glued laminated timber.

To take into account the possibility of splitting or shear or tension failure of the joint caused by the force component parallel to grain $F_{0,Ed}$, the following expression should be satisfied:

$$ F_{0,Ed} \leq F_{0,Rd} = \frac{k_{mod}}{\gamma_M} F_{0,Rk} \quad (19) \quad (20) $$

The characteristic timber failure capacity of the joint area:

$$ F_{0,Rk} = \sum_{i=1}^{m} F_{i,0,Rk} \quad (20) \quad (21) $$

Where $F_{i,0,Rk}$ is the timber failure capacity for lamella $i$ of the timber member calculated according to equation (21) and $m$ is the number of joint lamellas in the timber member.

Timber failure capacity for the lamella $i$ should be taken as:

$$ F_{i,0,Rk} = F_{ip,Rk} + F_{ep,Rk} \quad (21) $$

6.1.1  CAPACITY OF INNER PART LAMELLAS

The capacity of inner part lamellas:

$$ F_{ip,Rk} = \left\{ \begin{array}{ll} \min(A_{h,ip} \cdot f_{h,0,k}, F_{tv,k}) & \text{in tension joints} \\ \min(A_{h,ip} \cdot f_{h,0,k}, F_{cv,k}) & \text{in compression joints} \end{array} \right. \quad (22) $$

where:

- $f_{h,0,k}$ is the embedment strength of timber parallel to grain
- $A_{h,ip} = (n - n_1) \cdot d \cdot t_i$ (23)
- $F_{cv,k} = F_{v,k} + (n_2 - 1) \cdot d \cdot t_{ef,i} \cdot f_{h,0,k}$ (24)

$$ F_{tv,k} = \left\{ \begin{array}{ll} F_{i,k} \left( 1 - 0.3 \cdot \frac{F_{i,k}}{F_{v,k}} \right) , & \text{when } F_{i,k} \leq F_{v,k} \\ F_{v,k} \left( 1 - 0.3 \cdot \frac{F_{v,k}}{F_{i,k}} \right) , & \text{when } F_{i,k} > F_{v,k} \end{array} \right. \quad (25) $$

$$ F_{v,k} = k_v \cdot n_1^{-0.1} \cdot A_{v,ip} \cdot f_{v,k} \quad (27) $$

$\gamma_M$ is the characteristic timber failure capacity of the joint area:

$$ n_1 \text{ is the maximum number of fasteners in the fastener rows perpendicular to grain} $$

- $f_{v,k}$ is the shear strength of the timber member

$$ f_{v,k} = \left\{ \begin{array}{ll} 1.63 \cdot d \cdot \frac{f_{y}}{f_{h,0,k}} \leq t_i & \text{for side lamellas} \\ 0.68 \cdot d \cdot \frac{f_{y}}{f_{h,0,k}} \leq t_i & \text{for middle lamellas} \end{array} \right. \quad (29) $$

$$ A_{ip} = (n_2 - 1)(a_2 - d) \cdot t_i \quad (30) $$

$$ A_{ip} = 2(n_2 - 1)(a_1 + a_3) \cdot t_{ef,i} \quad (31) $$

- $f_y$ is the yield strength of the fastener
- $a_1$ is the fastener spacing parallel to the grain
- $a_2$ is the fastener spacing perpendicular to the grain
- $a_3$ is the fastener end distance
6.1.2 CAPACITY OF THE EDGE PART OF LAMELLAS

The capacity of the edge part of lamellas:

\[
F_{ep,Rk} = \begin{cases} 
\min\left( A_{h,ep} \cdot f_{h,0,k} \cdot F_{iv,k} \cdot F_{sv,k} \cdot F_{se,k} \right) & \text{in tension joints} \\
\min\left( A_{h,ep} \cdot f_{h,0,k} \cdot F_{cv,k} \right) & \text{in compression joints}
\end{cases}
\]

\[
F_{cv,k} = F_{v,k} + d \cdot t_{ef,i} \cdot f_{h,0,k}
\]

\[
F_{sv,k} = \begin{cases} 
F_{s,k} \cdot \left( 1 - 0.3 \cdot \frac{F_{s,k}}{F_{v,k}} \right) & \text{when } F_{s,k} \leq F_{v,k} \\
F_{v,k} \cdot \left( 1 - 0.3 \cdot \frac{F_{v,k}}{F_{s,k}} \right) & \text{when } F_{v,k} \leq F_{s,k}
\end{cases}
\]

\[
F_{se,k} = \frac{14 n_{0.9}^i t_{ef,i} (a_3 - 0.5d) \cdot f_{t,90,k}}{s_{hole}}
\]

where:
\[
A_{h,ep} = n_1 \cdot d \cdot t_i
\]

and \( F_{v,k} \) is calculated according to equations (25) - (27) with substitutions from equation (36):

\[
A_{t,lp} = k_{t,lp} \cdot A_{t,lp} \quad \text{and} \quad A_{v,lp} = A_{v,lp}
\]

where:
\[
A_{t,lp} = (2a_4 - d) \cdot t_i
\]

\[
A_{v,lp} = 2((n_1 - 1)a_1 + a_3) \cdot t_{ef,i}
\]

\[
k_{t,lp} = \frac{1}{1 + \frac{A_{t,lp}}{A_{v,lp}}}
\]

\( a_4 \) is the fastener edge distance

The splitting capacities:

\[
F_{s,k} = \frac{14 n_{0.9}^i t_{ef,i} (a_3 - 0.5d) \cdot f_{t,90,k}}{s_{hole}}
\]

\[
F_{se,k} = \frac{14 n_{0.9}^i t_{ef,i} (a_3 - 0.5d) \cdot f_{t,90,k}}{s_{end}}
\]

\[
s_{hole} = \max\left\{ 1, \frac{0.65 \cdot a_3}{a_4} \right\}
\]

\[
s_{end} = \frac{2.7}{\cosh\left( \frac{a_3}{a_4} - 1.4 \right)}
\]
6.2 CONNECTION FORCES AT AN ANGLE TO THE GRAIN

When a force in a connection acts at an angle to the grain, see Figure 5, the possibility of splitting caused by the tension force component, \((F_{Ed} \cdot \sin \alpha)\), perpendicular to grain, shall be taken into account.

For solid timber, glued laminated timber, Kerto-S, Kerto-T and Kerto-Q edgewise, the following expressions shall be satisfied:

\[
F_{v,Ed} \leq F_{90,d}
\]  \hspace{1cm} (44)

where: \(F_{90,d}\) is the design splitting capacity

\[
F_{v,Ed} = \max\left(F_{v,Ed1}; F_{v,Ed2}\right)
\]  \hspace{1cm} (45)

\(F_{v,Ed1}\) and \(F_{v,Ed2}\) are the design shear forces on either side of the connection caused by the connection force component \((F_{Ed} \cdot \sin \alpha)\) perpendicular to the grain.

For softwood, the characteristic splitting capacity:

\[
F_{90,k} = 14 \cdot b \cdot \sqrt{\frac{h_e}{h_0}} \left[\frac{1}{1 - \frac{h_e}{h}}\right] \quad [N]
\]  \hspace{1cm} (46)

where: \(h_e\) is the loaded edge distance to the centre of the most distant fastener, in mm, see Figure 5

\(b\) is the timber member height, in mm

\(b\) is the member thickness, but not more than the penetration depth, in mm

The equation (46) does not need to be checked for flatwise Kerto-Q connections since Kerto-Q when used flatwise is not sensitive to splitting caused by connection forces at an angle to the grain due to the cross-veneers.
6.3 ALTERNATIVE DIMENSIONING METHOD

Timber failure capacity of the joint area can be calculated by the method shown in RII 205-1-2009 in section 8.2.4S Lohkeamismurto. When using this method, the connection area for splitting and row shear is taken into account by using the effective number of fastener \( n_{ef} \), see equation (16). This method cannot be used for edgewise Kerto connections.

When connection force components are parallel to the grain, the timber failure should be checked at tension loaded member ends. There are two types of timber failure modes: block shear and plug shear. The block and plug shear capacities do not require checking for connections where all the fasteners are in a single row parallel to the grain \( (n_2 = 1) \).

For Steel-to-timber connections with Kerto-Q, both block shear and plug shear capacity should be checked.

For bolted connections, where the amount of fasteners in a row parallel to grain is not more than four and the bolt spacing perpendicular to the grain \( a_2 \geq 5d \), the block shear capacity does not need to be checked.

For bolted timber-to-timber connection the plug shear capacity does not require checking.

The characteristic block shear capacity of a timber member:

\[
F_{bt,k} = L_{net,t} \cdot t_1 \cdot k_{bt} \cdot f_{t,0,k}
\]  

(47)\(^\text{57}\)

where: \( f_{t,0,k} \) is the tension strength of timber member without the size effect

\[
k_{bt} = \begin{cases} 
1.50, & \text{for solid wood and glued laminated timber} \\
1.25, & \text{for Kerto-LVL} 
\end{cases}
\]  

(48)\(^\text{58}\)

\[
L_{net,t} = (n_2 - 1) \cdot (a_2 - D)
\]  

(49)\(^\text{59}\)

\( n_2 \) is the number of rows perpendicular to the grain
\( a_2 \) is the fastener spacing perpendicular to the grain
\( D \) is the hole spacing perpendicular to the grain
\( t_1 \) is the thickness of the timber member \( (t_1 \leq 2t_0) \)

The characteristic block shear capacity of Kerto-Q member:

\[
F_{bt,k} = \max \left\{ L_{net,t} \cdot t_1 \cdot f_{t,0,k} + 0.7 \cdot L_{net,v} \cdot t_1 \cdot f_{v,k} \right\} 
\]  

(50)\(^\text{60}\)

where: \( f_{v,k} \) is the edgewise shear strength \( (f_{v,0, edge, k} = 4.5 \text{ N/mm}^2) \)

\[
L_{net,v} = 2 \cdot \left( a_1 + (n_1 - 1) \cdot (a_1 - D) \right)
\]  

(51)\(^\text{61}\)

\( a_1 \) is the fasteners end distance
\( a_1 \) is the fastener spacing parallel to the grain
\( n_1 \) is the amount of rows parallel to the grain

The characteristic plug shear capacity of a Kerto member:

\[
F_{ps,k} = L_{net,t} \cdot t_{ef} \cdot f_{t,0,k} + (a_3 + (n_1 - 1) \cdot a_1) \cdot f_{v,0,k}
\]  

(52)\(^\text{62}\)

where: \( L_{net,t} = (n_2 - 1) \cdot (a_2 - D) \)  

(53)\(^\text{63}\)

\[
t_{ef} = \frac{R_k}{d \cdot f_{h,0,k}}
\]  

(54)\(^\text{64}\)

\( f_{v,0,k} \) is the shear strength of the timber member

\( f_{h,0,k} \) is the characteristic load-carrying capacity per shear plane per fastener

\( f_{h,0,k} \) is the characteristic embedment strength

| \( f_{v,0,flat,k} \) | 2.3 N/mm\(^2\) | for flatwise Kerto-S connections |
| \( f_{v,0,flat,k} \) | 1.3 N/mm\(^2\) | for flatwise Kerto-Q connections |
| \( f_{v,0,flat,k} \) | 1.3 N/mm\(^2\) | for flatwise Kerto-T connections |

\( R_k \) is the characteristic load-carrying capacity per shear plane per fastener

Kuva 6: a) Block shear  b) Plug shear \(^\text{56}\)
7. STEEL PLATES

7.1 TENSION STRENGTH

\[ \frac{N_{Ed}}{N_{t,Rd}} \leq 1.0 \]  \hspace{1cm} (55)

where: \( N_{Ed} \) is the tension force design value

\[ N_{t,Rd} = \min \left[ N_{pl,Rd}, N_{u,Rd} \right] \]  \hspace{1cm} (56)

the design tension capacity for gross area:

\[ N_{pl,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} \]  \hspace{1cm} (57)

the design tension capacity for net area

\[ N_{u,Rd} = \frac{0.9 \cdot A_{net} \cdot f_u}{\gamma_{M2}} \]  \hspace{1cm} (58)

\( A \) is the gross area of cross-section
\( A_{net} \) is the net area of cross-section
\( f_u \) is the ultimate tensile strength
\( f_y \) is the yield tensile strength
\( \gamma_{M0} \) and \( \gamma_{M2} \) are the partial factors

7.2 EMBEDMENT STRENGTH

The design embedment strength for a single fastener:

\[ F_{b,Rd} = k_1 \cdot a_b \cdot f_u \cdot d \cdot t \]  \hspace{1cm} (59)

where: \( a_b = \min \left[ \alpha_d, \frac{f_{ub}}{f_u}; 1.0 \right] \)  \hspace{1cm} (60)

parallel to force:

- for plate’s end fasteners \( \alpha_d = \frac{e_1}{3d_0} \);
- others \( \alpha_d = \frac{p_1}{3d_0} - \frac{1}{4} \)

perpendicular to force:

- for plate’s end fasteners \( k_1 = \min \left( 2.8 \frac{e_2}{d_0} - 1.7; 2.5 \right) \)
- others \( k_1 = \min \left( 1.4 \frac{p_2}{d_0} - 1.7; 2.5 \right) \)

\( f_u \) is the ultimate tensile strength of the steel plate
\( f_{ub} \) is the ultimate tensile strength of the fastener
\( d \) is the fastener diameter
\( d_0 \) is the hole diameter in steel plate
\( e_1 \) is the end distance of the fastener
\( e_2 \) is the edge distance of the fastener
\( p_1 \) is the fastener spacing parallel to load
\( p_2 \) is the fastener spacing perpendicular to load
\( t \) is the thickness of the steel plate
\( \gamma_{M2} \) is the partial factor of the steel plate (see Table 3)

7.3 BLOCK TEARING

The block tearing design capacity of a steel plate when a symmetrical fastener group has a centric force:

\[ V_{eff,1,Rd} = \frac{f_u \cdot A_{nt}}{\gamma_{M2}} + \frac{f_y \cdot A_{nv}}{\sqrt{3} \cdot \gamma_{M0}} \]  \hspace{1cm} (61)

where: \( A_{nt} \) is the tension stressed net area of cross-section
\( A_{nv} \) is the shear stressed net area of cross-section
\( f_u \) is the ultimate tensile strength
\( f_y \) is the yield tensile strength
\( \gamma_{M0} \) and \( \gamma_{M2} \) are the partial factors

8. AXIALLY LOADED BOLTS

The axial load-bearing capacity and withdrawal capacity of a bolt should be taken as the lower value of: the bolt tensile capacity; the load-bearing capacity of either the washer or (for steel-to-timber connections) the steel plate.

The bearing capacity of a washer should be calculated assuming a characteristic compressive strength on the contact area of \( 3f_{c,90,k} \).

The bearing capacity per bolt of a steel plate should not exceed that of a circular washer with a diameter which is the minimum of: \( 12t_1 \), where \( t_1 \) is the plate thickness; \( 4d \), where \( d \) is the bolt diameter.

Washer with a side length (in the case of square washers) or a diameter of at least \( 3d \) and a thickness of at least \( 0.3d \) should be used under the head and nut. Washers should have a full bearing area.
9. FASTENER SPACINGS AND EDGE AND END DISTANCES

This instruction is property of Metsä Wood.
The instruction has been prepared in cooperation with VTT Expert Services Ltd.
Figure 7: Minimum spacings and end and edge distances
The fastener spacing parallel to the grain $a_1$ and perpendicular to the grain $a_2$:

-90° ≤ $\alpha$ ≤ 90°  
Loaded end

90° ≤ $\alpha$ ≤ 270°  
Unloaded end

0° ≤ $\alpha$ ≤ 180°  
Loaded edge

180° ≤ $\alpha$ ≤ 360°  
Unloaded edge

$\alpha$ is the angle between a force and the grain direction

Figure 8: Fastener spacings and edge and end distances.

Table 4: Bolt minimum spacings and edge and end minimum distances

<table>
<thead>
<tr>
<th>SPACING AND EDGE/END DISTANCE, SEE FIGURE 8</th>
<th>ANGLE</th>
<th>SOLID TIMBER AND GLUED LAMINATED TIMBER</th>
<th>KERTO-S, KERTO-T AND EDGEWISE KERTO-Q</th>
<th>FLATWISE KERTO-Q</th>
</tr>
</thead>
<tbody>
<tr>
<td>$a_1$</td>
<td>0° ≤ $\alpha$ ≤ 360°</td>
<td>(4 +</td>
<td>cos $\alpha$</td>
<td>) $d$</td>
</tr>
<tr>
<td>$a_2$</td>
<td>0° ≤ $\alpha$ ≤ 360°</td>
<td>4 $d^b$</td>
<td>4 $d^b$</td>
<td>4 $d^b$</td>
</tr>
<tr>
<td>$a_3t$</td>
<td>-90° ≤ $\alpha$ ≤ 90°</td>
<td>max(7 $d$, 80 mm)</td>
<td>max(7 $d$, 105 mm)</td>
<td>max(4 $d$, 60 mm)</td>
</tr>
<tr>
<td>$a_3c$</td>
<td>90° ≤ $\alpha$ ≤ 150°</td>
<td>(1 + 6</td>
<td>sin $\alpha$</td>
<td>)$d$</td>
</tr>
<tr>
<td></td>
<td>150° ≤ $\alpha$ ≤ 210°</td>
<td>4 $d$</td>
<td>4 $d$</td>
<td>4 $d$</td>
</tr>
<tr>
<td></td>
<td>210° ≤ $\alpha$ ≤ 270°</td>
<td>(1 + 6</td>
<td>sin $\alpha$</td>
<td>)$d$</td>
</tr>
<tr>
<td>$a_4t$</td>
<td>0° ≤ $\alpha$ ≤ 180°</td>
<td>max(2 + 2</td>
<td>sin $\alpha$</td>
<td>,$d$, 3 $d$)</td>
</tr>
<tr>
<td>$a_4c$</td>
<td>180° ≤ $\alpha$ ≤ 360°</td>
<td>3 $d$</td>
<td>3 $d$</td>
<td>3 $d$</td>
</tr>
</tbody>
</table>

a) Block shear should also be checked in timber connections if $a_2 < 5d$.

b) For bolts with diameter $d < 15$ mm, the minimum end distance may be further reduced to 7$d$, if the embedment strength $f_{h,0,k}$ is reduced by factor $a_3$ / (105 mm).

c) For bolts with diameter $d < 15$ mm, the minimum end distance may be further reduced to 4$d$, if the embedment strength $f_{h,0,k}$ is reduced by factor $a_3,t$ / (60 mm).

d) The minimum spacing may be further reduced to 5$d$ if the embedment strength $f_{h,0,k}$ is reduced by factor $\sqrt{\frac{a_1}{4 + 3|\cos \alpha|d}}$. 
Table 5: For bolted moment resisting multi shear Kerto-to-Kerto flatwise connections with circular patterns of fasteners, the following minimum values of distances and spacings may be used.  

<table>
<thead>
<tr>
<th>SPACING AND EDGE/END DISTANCES</th>
<th>KERTo-S TO KERTo-Q a)</th>
<th>KERTo-S TO KERTo-S</th>
<th>KERTo-Q TO KERTo-Q</th>
</tr>
</thead>
<tbody>
<tr>
<td>End distance</td>
<td>6 d in Kerto-S</td>
<td>7 d</td>
<td>4 d</td>
</tr>
<tr>
<td></td>
<td>4 d in Kerto-Q</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Edge distance</td>
<td>4 d in Kerto-S</td>
<td>4 d</td>
<td>3 d</td>
</tr>
<tr>
<td></td>
<td>3 d in Kerto-Q</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Spacing on a circular</td>
<td>5 d</td>
<td>6 d</td>
<td>4 d</td>
</tr>
<tr>
<td>Spacing between circulars b)</td>
<td>5 d</td>
<td>5 d</td>
<td>4 d</td>
</tr>
<tr>
<td>a) When Kerto-Q is used as outer member</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>b) Between radius of the circulars</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 9: For bolted moment resisting multi shear Kerto-to-Kerto flatwise connections with circular patterns of fasteners.

10. ALLOWED TOLERANCES OF BOLTED CONNECTIONS

Table 6: Allowed tolerances of bolt connections - allowed deviations from designed position, unless structural design otherwise states.  

<table>
<thead>
<tr>
<th>Bolt connection</th>
<th>bolt location</th>
<th>simultaneous drilling a)</th>
<th>separate drilling</th>
<th>parts to contact</th>
<th>tilted gap max. 3 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>hole location tightening</td>
<td>± 5 mm b)</td>
<td>± 1.5 mm c)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

a) Drilling through all the parts without stopping or using a predrilled part as a template.  

b) On rows parallel to the grain, the fasteners can have a maximum tolerance of 5 mm to each other in the parallel direction.

c) Prerequisite that the timber members have 1 mm bigger holes than the bolt diameter and metal plates have 1.5...2.0 mm bigger holes than the bolt diameter.

11. BIBLIOGRAPHY

CALCULATION EXAMPLE

LATERALLY LOADED TIMBER-TO-TIMBER BOLT CONNECTION

Location: Roof truss, lower chord tension joint.

Capacity of laterally loaded bolt group

Checking the possibility of using M12 bolts, \( d = 12 \text{ mm} \)

Bolt grade is \( 8.8, f_{uk} = 800 \text{ N/mm}^2 \)

Timber beams: Kerto-S flatwise connection.

Service class: 2, load-duration class: medium term action.

The thicknesses of connection timbers, double shear plane connection.

\[
\begin{align*}
t_1 &= 51 \text{ mm} > 4d = 48 \text{ mm} \\
t_2 &= 51 \text{ mm} > 4d = 48 \text{ mm} \\
t_s &= 63 \text{ mm} > 5d = 60 \text{ mm} \text{ and } t_s > \min(t_1, t_2) = 51 \text{ mm}
\end{align*}
\]

The thicknesses of the connecting timber members are OK.

The characteristic embedment strength:

\[
\begin{align*}
f_{h,0,k} &= 0.082 \cdot (1-0.01d) \cdot \rho_k = 0.082 \cdot (1-0.01 \cdot 12) \cdot 480 = 34.63 \text{ N/mm}^2 \\
f_{h,1,k} &= f_{h,2,k} = f_{h,s,k} = f_{h,0,k} = 34.63 \text{ N/mm}^2 \\
f_{h,k} &= \min(f_{h,1,k}, f_{h,2,k}, f_{h,s,k}) = 34.63 \text{ N/mm}^2
\end{align*}
\]

\[
t_u = \min\left\{ \frac{t_1 \cdot f_{h,1,k}}{f_{h,k}}, \frac{t_2 \cdot f_{h,2,k}}{f_{h,k}} \right\} = \min\left\{ \frac{51 \text{ mm} \cdot 34.63 \text{ N/mm}^2}{34.63 \text{ N/mm}^2}, \frac{51 \text{ mm} \cdot 34.63 \text{ N/mm}^2}{34.63 \text{ N/mm}^2} \right\} = 51 \text{ mm}
\]
The characteristic value for the yield moment:

\[ M_y = 0.3 \cdot f_{u,k} \cdot d^{2.6} = 0.3 \cdot 800 \cdot 12^{2.6} = 153490 \text{ Nmm} \]

The characteristic load-carrying capacity per shear plane per fastener for single shear:

\[
R_k = \min \left\{ \begin{aligned}
0.4 \cdot f_{u,k} \cdot t_u \cdot d \cdot \sqrt{1 + \frac{3 \cdot M_y}{f_{u,k} \cdot d \cdot t_u^2}} \\
2 \cdot \sqrt{M_y \cdot f_{u,k} \cdot d}
\end{aligned} \right. \]

\[
R_k = \min \left\{ \begin{aligned}
0.4 \cdot 34.63 \text{ N/mm}^2 \cdot 51 \text{ mm} \cdot 12 \text{ mm} \cdot \sqrt{1 + \frac{3 \cdot 153490 \text{ Nmm}}{34.63 \text{ N/mm}^2 \cdot 12 \text{ mm} \cdot (51 \text{ mm})^2}} \\
2 \cdot \sqrt{153490 \text{ Nmm} \cdot 34.63 \text{ N/mm}^2 \cdot 12 \text{ mm}}
\end{aligned} \right. 
\]

\[
R_k = \min \left\{ \begin{aligned}
10123 \text{ N} \\
15973 \text{ N}
\end{aligned} \right. = 10.12 \text{ kN/shear}
\]

The design capacity per bolt per shear plane:

\[ k_{mod} = 0.8 \text{ and } \gamma_M = 1.2 \]

\[
R_{d,1} = \frac{k_{mod} \cdot R_k}{\gamma_M} = \frac{0.8 \cdot 10.12 \text{ kN/shear}}{1.2} = 6.76 \text{ kN/shear}
\]

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 \( n_{ef} \):

\[
a = \min(a_1; a_3) = 85 \text{ mm}
\]

\[
t = \min(2t_1; 2t_2; t_3) = \min(102 \text{ mm}; 102 \text{ mm}; 63 \text{ mm}) = 63 \text{ mm}
\]

\[
n_i = 2
\]

\[
n_{ef} = \min \left\{ \begin{aligned}
n_i \\
n_i^{0.9} \cdot \frac{a \cdot t}{50 \cdot d^2}
\end{aligned} \right. = \min \left\{ \begin{aligned}
2 \\
2^{0.9} \cdot \frac{85 \text{ mm} \cdot 63 \text{ mm}}{50 \cdot (12 \text{ mm})^2}
\end{aligned} \right. = \min \left\{ \begin{aligned}
2 \\
1.73
\end{aligned} \right. = 1.73
\]

The design capacity of timber-to-timber connection:

\[
R_d = \text{amount of bolts \cdot per shear capacity \cdot shears} = (2 \cdot 1.73) \cdot 6.76 \text{ kN/shear} \cdot 2 \text{ shears}
\]

\[= 46.77 \text{ kN} \]

\[F_{v,d} = 40 \text{ kN} \Rightarrow \eta = \frac{F_{v,d}}{R_d} = 86\% \]

Utilization rate against timber-to-timber capacity is 86 %. \( \Rightarrow \text{OK} \)
When the lateral timber-to-timber load-carrying capacity is calculated according to the effective number of bolts in a row parallel to grain, the block shear capacity can be calculated according to section 6.3 using equation (47).

\[ L_{net,t} = (n_2 - 1) \cdot (a_2 - D) = (2 - 1) \cdot (50\text{mm} - 13\text{mm}) = 37\text{mm} \]

The characteristic load-bearing capacity of block shear:

\[ F_{bt,k} = k_{mod} \cdot t_i \cdot k_{bt} \cdot f_{r,0,k} = 37\text{ mm} \cdot 63\text{ mm} \cdot 1.25 \cdot 35\text{ N/mm}^2 = 101.98\text{ kN} \]

The design load-bearing capacity of block shear:

\[ k_{mod} = 0.8 \text{ and } \gamma_M = 1.2 \]

\[ F_{bt,d} = \frac{k_{mod}}{\gamma_M} F_{bt,k} = 67.98\text{kN} \]

\[ F_{v,d} = 40\text{kN} \Rightarrow \eta = \frac{F_{v,d}}{F_{bt,d}} = 59\% \]

Utilization rate against block shear capacity is 59%. -> OK