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Publication Date:
2017-09

Permanent Link:
https://doi.org/10.3929/ethz-b-000228388

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Brittle Failure of Connections Loaded Perpendicular to Grain

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Summary
Connections loaded perpendicular to grain are prone to brittle failure due to fracture induced by tension perpendicular to grain stresses. Different approaches can be found in design codes and literature to account for the reduction of load-carrying capacity in the design of the structure. In this paper selected design approaches are discussed and their behaviour with regard to different geometrical parameters is analysed. The structural behaviour of connections loaded perpendicular to grain is evaluated on the basis of a test series carried out at ETH Zurich and based on test results from literature. The impact of different geometrical parameters on the load-carrying capacity is demonstrated and the design approaches are benchmarked by the large number of individual test results. Recommendations for a safe design are given at the end of the paper.

1. Introduction
1.1 Failure Modes of Connections and Considerations in Design
Failure in connections can occur either due to local failure of the fasteners or due to failure in the surrounding timber. Design should be aimed at receiving a failure of the fastener in order be able to adjust the load-carrying capacity by the choice of an adequate fastener diameter and in order to achieve a ductile failure mechanism.

The different failure modes of fasteners in shear were described by Johansen [1] and are given in the European design code for timber structures EN 1995-1-1 (Eurocode 5, EC 5) [2]. The failure modes of the so called European Yield Model (EYM) are defined mainly by the properties and diameter of the fastener and the density and thickness of the timber members.

In addition to the resistance of the fastener, the surrounding timber has to allow for sufficient load distribution in order to prevent splitting and brittle failure in the timber. Minimum spacing and minimum end and edge distances of the fastener are specified in EC 5 in order to prevent the premature splitting of the timber. The
effect of splitting of multiple fasteners loaded in a row parallel to grain is accounted for by a reduction factor for the load-carrying capacity that is leading to a reduction of the number of effective fasteners. For fasteners in a row loaded perpendicular to grain, such a reduction in form of an effective number of fasteners is not included. Instead a simple design approach is given to account for the risk of tension perpendicular to grain splitting at connections loaded perpendicular to grain.

In this study the different impacts on the fracture and failure of connections loaded perpendicular to grain are evaluated and discussed and recommendation for a safe design are given.

1.2 Tension Perpendicular to Grain Failure of Connections

Due to its anisotropic material behaviour wood shows very high strength and stiffness when being loaded in direction of the grain but only moderate or low strength and stiffness when being loaded perpendicular to the grain direction. Especially in tension perpendicular to grain, not only low strength and stiffness but also brittle failure behaviour can be observed. That is why an economic design should avoid any loading in tension in direction perpendicular to grain. Since the strength of timber in tension perpendicular to grain is generally low and shows high variability conservative values are specified in different design codes e.g. the Swiss SIA 265 [3]. The values given in the product standards for solid timber EN 338 [4] or glued laminated timber EN 14080 [5] corresponding to EC 5 give representative values that may not be adequate to design the specific case of stress singularities near concentrated loads and cracks.

In connections loaded perpendicular to grain very high tensile stresses perpendicular to grain occur that may initiate cracking and cause failure of the timber member. A typical crack pattern occurring at connections loaded perpendicular to grain is illustrated in Fig. 1.

The magnitude of the tensile stresses perpendicular to grain depends mainly on the relative distance \( h_c/h \) of the most distant row of fasteners from the beam edge loaded in tension (lower beam edge in Fig. 1). The force has to be transferred from the connection into the beam by shear and perpendicular to grain stresses. Depending on the position of the connection along the beam height, the transfer of forces induces compression or tension stresses perpendicular to grain. In order to avoid tension stresses perpendicular to grain and in order to reduce the risk of cracking and failure of members, the connection should be
positioned at sufficient distance from the beam edge loaded in tension. If that is not possible the connection should be reinforced in order to prevent fracture and to bridge possible cracks.

1.3 Reinforcement

In order to prevent cracking and premature failure members loaded in tension perpendicular to grain should be reinforced. Recommendations for the reinforcement of connections loaded perpendicular to grain can be found e.g. in the former German design code for timber structures DIN 1052 [6] or in the German national annex to EC 5 [7] and are currently under development for the next generation of EC 5 [8].

The reinforcement can be distinguished into internal and external reinforcement. Internal reinforcement can be achieved by means of glued-in rods or fully-threaded screws or rods. The reduction of cross-section by this internal reinforcement should be accounted for in the design. External reinforcement can be realized e.g. by means of wood based panels (e.g. plywood or laminated veneer lumber) or boards glued to the timber member or pressed-in punched metal plates. When designing the glued on external reinforcement it needs to be accounted for the unequal stress distribution in the glue line and the resulting unequal distribution of forces in the reinforcing panel. When determining the tensile forces in the reinforcing elements, the tensile capacity of the timber may be neglected.

1.4 Types of Connections Loaded Perpendicular to Grain

Connections where a load component is introduced into a beam at an angle to grain occur e.g. at primary to secondary beam connections, to hang up loads or members to beams. Other examples, although only of temporary use, are mounting joints for lifting and assembling large timber elements or CLT panels.

Connections with slotted in metal steel plates are often made by means of dowels. External steel plates are used e.g. in combination with bolts or screws. Punched metal plates are used e.g. for truss structures made of solid timber elements. Three dimensional nailing plates like joist hangers are often nailed or screwed.

2. Theoretical Description of the Load-Carrying Behaviour of Connections Loaded Perpendicular to Grain

2.1 General

The structural behaviour of connections loaded perpendicular to grain has been extensively studied by several authors in various studies. The studies differ with regard to the intended goal and the development of concise design rules. Different geometrical and material parameters are used depending on the complexity of the approaches.

In DIN 1052 [6] an empirically based design approach was included, that was developed on the basis of tests on nailed connections on full size glulam beams. In
EC5 [2] a theoretically based design approach is given, that is based on the fracture mechanical model of a connection with a single dowel. This approach was developed further in order to account for additional geometrical parameters by Ballerini [9].

These three design approaches were selected for further discussion due to their existing implementation in design codes, the consideration of a wide range of geometrical parameters and the availability of relevant material parameters.

### 2.2 Strength Based Design Approach

A first empirically based design approach was presented by Möhler and Siebert [10,11]. The approach is based on studies and tests described in [10,12]. The size effect of the volume loaded in tension perpendicular to grain is accounted for based on studies by Barret et al. [13] by an exponent of the effectively loaded tension area \( (t_{ef} h)^{0.8} \).

The design approach was further developed by Ehlbeck et al. [14,15] and is included in the DIN 1052 [6].

\[
\frac{F_{90,Ed}}{R_{n,d}} \leq 1
\]  

\[
R_{n,d} = k_e k_k \left( 6.5 + \frac{18h_i^2}{h^2} \right) t_{ef} h^{0.8} f_{cm,d}
\]  

\[
k_e = \max \left\{ 1; 0.7 + \frac{1.4a_i}{h} \right\}
\]  

\[
k_e = \frac{n}{\sum_{i=1}^{n} \left( \frac{h_i}{h} \right)^2}
\]
The individual configuration of the fastener layout is considered by the two factors $k_s$ and $k_r$ in Eq. (3) and (4), respectively. The factor $k_s$ accounts for the height of the connection $h_m$ as well as the number $n$ of fasteners in the connections. The factor is based on the assumption that the contribution of each single fastener to the entire amount of tension perpendicular to grain stresses is reduced with the square of the ratio between the distance $h_1$ of the closest row of fasteners to the unloaded beam edge and the distance $h_i$ of each fastener row $i$ to the unloaded beam edge.

The tests being the basis for this design approach showed that connections with stiffer fasteners allowed for higher load-carrying capacities. The effective length of the fasteners, necessary to determine the value $l_{ef}$, was chosen in dependency of the diameter of the fasteners. For more slender fasteners the effective length of the fasteners decreases in proportion to the width of the beam. For stouter fasteners the full fastener length can be accounted for.

If more than one connection loaded perpendicular to grain is located in a single beam, there can be an interaction between neighbouring connections. If the clear distance $l_l$ between two neighbouring connections is larger than twice the beam height, no interaction has to be accounted for. In contrast, for distances smaller than half the beam height both connections have to be treated as one single connection of greater width. For intermediate clear distances $l_l$ the impact of neighbouring connections may be accounted for by the following reduction factor per connection:

$$k_s = \frac{l_l}{4h} + 0.5$$

For connections located close to the beam end with a clear end distance $a_3 < h$, only half the load-carrying capacity may be accounted for.

The design approach in Eq. (2) is valid only for relative connection height $h_e/h \leq 0.7$. Connections with $h_e/h > 0.7$ showed a minor risk of failure due to cracking, i.e. no special design with regard to tension perpendicular to grain stresses is necessary.

In contrast connections with $h_e/h < 0.2$ should be avoided and – if absolutely necessary – loaded only in short durations of load (e.g. wind loads).

2.3 Fracture Mechanics Based Design Approach

A fracture mechanics based design approach for connections loaded perpendicular to grain was proposed by van der Put [16,17]. Based on the equilibrium of energies during growth of a crack of infinitesimal length, the energy released during crack growth can be calculated from the variation of elastic energies in the beam.

The example of a crack of length $x$ with a crack growth by $\Delta x$ is illustrated in Fig. 3. The resistance of the timber against fracture can be described by the critical fracture energy $G_c$, the shear stiffness $G$ and the modulus of elasticity $E_0$ in direction parallel to grain as well as the beam height, the beam width and the relative connection height. In a general approach proposed by Jensen [18] the crack length $x$ is considered in addition.

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The approach proposed by van der Put [16] is a simplification of Eq. (6) in which the impact of crack length is disregarded ($x = 0$). The resulting very simple approach in Eq. (7) is the maximum of the load-carrying capacity at crack initiation at the connection in midspan.

$$F_{\text{num}} = 2b \sqrt{\frac{G_{c_1} h_c}{0.6 \left( 1 - \frac{h_c}{h} \right) + 1.5 \frac{E}{E_0} \left( 1 - \frac{h_c}{h} \right)^x}}$$  \hspace{1cm} (6)

$$F_{\text{num}} = 2b \sqrt{\frac{G_{c_1} h_c}{0.6 \left( 1 - \frac{h_c}{h} \right)}}$$  \hspace{1cm} (7)

Other geometrical parameters like the width or the configuration of the connection are not included in the fracture mechanical basis of these approaches. Instead the cracks at a connection with one single point-like dowel-type fastener are described. It can be seen from the tests and the design approach in Eq. (2) that this case with $a_r = 0$ and $h_m = 0$ is the most unfavourable configuration of the fastener layout with regard to the load-carrying capacity. Therefore the design approach in Eq. (7) allows for a conservative design.

The material property values used in Eq. (7) can be either calculated from property values determined in standardized tests, or calibrated on the basis of tests on connections loaded perpendicular to grain. When calculating the theoretical values of a material strength parameter $C_1 = (G \cdot G_c/0.6)^{0.5}$ for Eq. (7), pure crack opening failure in fracture mode 1 can be assumed with $G_{c,\text{mean}} = 0.3$ N/mm for spruce softwood [19]. The theoretical value of the material parameter is $C_{1,\text{mean}} = (G \cdot G_{c,\text{mean}}/0.6)^{0.5} = (650 \cdot 0.3/0.6)^{0.5} \approx 18$ N/mm$^{1.5}$. This value is higher than the value proposed by van der Put and Leijten [17] with $C_{1,\text{mean}} \approx 15$ N/mm$^{1.5}$, which was back calculated from tests. Leijten and Jorissen [20] studied the material property values more in detail and made a comparison with different other design models. A characteristic value of the material strength property $C_{1,k} \approx 10$ N/mm$^{1.5}$ was proposed in [20] for consideration in design codes. For the implementation of Eq. (7) in Eurocode 5, $C_{1,k} \approx 14$ N/mm$^{1.5}$ was chosen. This factor overestimates the load-carrying capacity compared to results from tests as discussed in [21].
An additional adaptation of Eq. (7) was done during implementation into Eurocode 5: The verification is not based on the load applied to the connection $F_{90,Ed}$ but it is based on the shear forces reacting $F_{v,Ed}$ on both sides of the connection loaded perpendicular to grain as shown in Eq. (8) and (9), see also Fig. 1.

$$F_{v,Ed} \leq F_{90,Ed}$$  \hspace{1cm} (8)

$$F_{v,Ed} = \max \{F_{v,Ed1}, F_{v,Ed2}\}$$  \hspace{1cm} (9)

The corresponding resistance of the connection is only half the value compared to Eq. (7):

$$F_{90,Rd} = C_{1,d} b w \frac{h_1}{h_2} \sqrt{1 - \frac{h_1}{h}} \text{ with } C_{1,d} = 14 \text{ N/mm}^{1.5}$$  \hspace{1cm} (10)

For connections with punched metal plate fasteners the resistance can be increased by a factor $w$ in dependency of the width parallel to grain $w_{pl}$ of the punched metal plate:

$$w = \max \left( \frac{w_{pl}}{100}, 1.0 \right)$$  \hspace{1cm} (11)

$$w = 1.0$$  \hspace{1cm} (12)

Using the shear force reactions $F_{v,Ed}$ instead of the load applied to the connection $F_{90,Ed}$ for the verification in Eq. (8) allows to account for the effect of neighbouring connections, connections close to the support or connections at a cantilever. The impact on the load-carrying capacity of such configurations is conservative.

### 2.4 Extension of the Fracture Mechanical Approach

Ballerini [9] made further developments of the approach by van der Put [16] and Jensen et al. [18]. Assuming a different force and moment distribution in the reduced cross-section at the cracked connection, Ballerini proposed a different exponent of the relative connection height in his approach compared to Eq. (7). Based on own tests and results from literature, he proposed a design value of the material parameter $C_{1,d} = 8.6 \text{ N/mm}^{1.5}$.

$$R_{90,d} = 2 b C_{1,d} f_e f_r \sqrt{\frac{h_1}{1 - \frac{h_1}{h}}}$$  \hspace{1cm} (13)

where:

$$f_e = \min \left\{ 1 + 0.75 \left( \frac{a + \frac{h}{2}}{h} \right), 2.2 \right\}$$  \hspace{1cm} (14)
The parameters \( f_w \) and \( f_r \) were introduced by Ballerini in order to account for the influence of the width \((a_r)\) and the height \((h_m)\) of the connection, respectively. In addition the clear distance \((l_i)\) between two neighbouring connections is considered. The derivation of the parameters \( f_w \) and \( f_r \) is purely empirical on the basis of a large number of test results.

3. Structural Behaviour of Connections Loaded Perp. to Grain

3.1 Tests Carried out at ETH Zurich

The structural behaviour of connections loaded perpendicular to grain was studied in a test series at ETH Zurich. In this test series the relative connection height \( h_e/h \) was varied between 60 %, 70 % and 80 % of the beam height and two different arrangements of the dowels in the connection with a slotted in metal steel plate were tested: a horizontal configuration with \( m = 4 \) rows and \( n = 2 \) columns (Fig. 4 left) and a vertical configuration with \( m = 2 \) rows and \( n = 4 \) columns (Fig. 4 right) of dowels with diameter \( d = 12 \text{ mm} \). The beam height was \( h = 440 \text{ mm} \) and the width \( b = 140 \text{ mm} \).

![Fig. 4](image)

*Fig. 4  Geometry of the connections tested at ETH Zurich.*

The load-displacement behaviour of the beam, the crack opening and the relative pull-out of the steel plate were measured by means of LVDT. The deformations on the surface of the beam were recorded by means of optical measurements. These deformations were used to study the crack initiation and growth during load application. Results of these measurements are summarized in [22,23].

In Fig. 5 the load-deformation behaviour of three specimen with different relative connection height is shown. The deformation was measured at the steel plate and it can be seen that with increasing relative connection height higher load-carrying capacities and larger deformations were reached. The full load-carrying capacity of the connection with yielding of the fasteners is reached for the connection with

\[
f_i = 1 + 1.75 \frac{\kappa}{1 + \kappa} \quad \text{with} \quad \kappa = \frac{n \cdot h}{1000} \quad \text{and} \quad h, \text{ in } [\text{mm}]
\]

(15)
$h_c/h = 0.8$. In contrast the connection with $h_c/h = 0.6$ fails due to instable crack growth already at small deformations of the fasteners. The connection with $h_c/h = 0.8$ shows a more stable crack growth with increasing load until the full load-carrying capacity of the fasteners is reached. Some of these connections with $h_c/h = 0.8$ failed due to shear failure in the reduced cross-section.

Tests showed that cracking occurs also for relative connection heights $h_c/h$ larger than 70% as well as for connections with reinforcement. Nevertheless, the crack initiation is not followed by an unstable crack growth causing failure of the entire beam, but instead further loading is possible.

3.2 Tests Reported in Literature

The large number of tests reported in literature offers the possibility to analyse the impact of individual parameters on the structural behaviour and to calculate fractile values for the design. For the following evaluations individual results from tests on glulam beams reported in [11,24–31] were used. A summary of the tests is given in [32].

The majority of the tests were performed as 3-point bending tests with the connection in the central position of the beam span. A small number of tests were performed with an eccentric position of the connection [24,30,31] or with the connection at a cantilever beam [24,31].

The majority of the tests were carried out on beams with rather small beam height $h \leq 400$ mm and beam widths $b \leq 80$ mm. The small beam width in combination with relatively large fastener diameter caused mostly an embedment failure mode of the fasteners. This constant load introduction over the entire width of the beam leads to a linear dependency of the load-carrying capacity on the beam width. For more slender fastener an early crack initiation around the fasteners can be expected. Mostly bolts with external steel plates were used in the tests, besides dowels, nails and connectors.

3.3 Comparison of Test Results and Design Approaches

3.3.1 Geometry of the Connection

The impact of the geometry of the connection on the load-carrying capacity is shown in Fig. 6. In the two diagrams on the top the impact of the connection width
$a_r$ on the relative load-carrying capacity in comparison to a reference with $a_r/h = 0.5$ is shown. In the two diagrams below the impact of the number of rows of fasteners on the relative load-carrying capacity in comparison to a reference with $n = 1$ row is shown. The test results are normalized by means of the design approach by Ballerini in Eq. (13) (Figures on the left) and Ehlbeck et al. in Eq. (2) (right), respective, and can be compared with the solid lines representing the corresponding design approach. The impact of the parameter is underestimated by the design approaches if the test results are located in average above the solid lines, whereas the impact of the parameter is overestimated by the design approaches if the test results are located in average below the solid lines.

![Fig. 6 Impact of the connection width $a_r$ (top) and number of rows of fasteners $n$ (bottom) on the relative load-carrying capacity and comparison with design approaches according to Eq. (13) (left) and Eq. (2) (right).](image)

It can be seen that with increasing width $a_r$ of the connection and with increasing number of rows of fasteners the load-carrying capacity is increasing. The different behaviour of the design approaches in dependency of the connection width and number of rows of fasteners can be explained by the empirical background and the
limited number of tests considered in the derivation of the parameters accounting for the impact of the connection geometry in these design approaches.

The design approach given in EC 5 on the basis of [16] was derived for ($h_m = 0$ and $a_r = 0$ together with $n = 1$ and $m = 1$), leading to conservative results with increasing value of these parameters.

3.3.2 Influence of the Position of the Connection

The impact of the position of the connection loaded perpendicular to grain along the beam axis is considered only by the design approach given in Eurocode 5 as explained in Chapter 2.2. The evaluation of the test results shows no considerable impact of the position of the connection along the span of the beam, see Fig. 7. Only in one test series on connections with small end-distances at a cantilever beam, lower load-carrying capacities were reached compared to tests with connections at midspan.

Fig. 7 Impact of the position of the connection along the beam axis on the relative load-carrying capacity according to EC 5 (left) and according to DIN 1052 (right).
3.3.3 Influence of Neighbouring Connections

The load-carrying capacity of neighbouring connections increases with increasing distances between the connections. Two neighbouring connections with a small clear distance \((l_i = a_i/h \leq 0.5)\) show approximately the same resistance as one single connection. For larger distances the load-carrying capacity of each individual connection increases but does not reach the full capacity of one single connection. Due to the limited number of available test results no precise statement about the impact of neighbouring connections is possible.

3.4 Characteristic Values of the Material Property Values

Each of the design approaches presented requires different material property values and an individual calibration for the design, due to the differences in the underlying theory and partially empirical background. The approach based on strength theory in Eq. (2) uses tension perpendicular to grain strength, which is strongly dependent on the tested volume and is topic of various discussions among experts (e.g. [33, 34]). The use of the general strength value \(f_{t,90,k}\) given in the product standards (EN 338 [4] and EN 14080 [5]) should be treated with caution.

The approaches with a fracture mechanical background in Eq. (7) and (13) are based on the fracture energy and the shear modulus of the wood. The fracture with crack opening in mode 1 is the relevant failure mode. Together with the values of shear modulus taken from EN 14080 [5] for the common glulam strength grades, it would be possible to estimate the load-carrying capacity based on fracture energy values given in literature e.g. by Larsen and Gustafsson [19].

With the help of the large number of test results it is possible to back-calculate the material parameters used in the different design approaches. The design approaches can be benchmarked based on the variability of these material parameters and possible dependencies on certain parameters can be determined. In case of an ideal design approach the material parameter back-calculated from tests would show a very low variation, which could be explained by the natural variability of the material timber. However, the existing design approaches lead to material properties with a relatively high variation and with dependencies on certain geometrical properties.
Fig. 9 Material property values in dependency of the relative connection height $h_{\text{rel}}/h$ back-calculated from test results for EC 5 in Eq. (8) (left) and Ballerini in Eq. (13) (right) together with linear and quadratic regression functions.

The material parameters $C_{1,EC5,test}$ and $C_{1,Ba,test}$ back-calculated from the test results by means of the design approaches in Eq. (8) (Eurocode 5) and (13) (Ballerini), respectively, are shown in Fig. 9 in dependency of the connection height $h_{\text{rel}}/h$. All test selected for this evaluation were made on softwood glulam and no dependency on additional material parameters was accounted for.

The material parameter back-calculated from the design approach by Ballerini shows very little dependency on the geometry of the connection, which means that these properties are accounted for in an adequate way by the two parameters $f_w$ and $f_r$ in Eq. (14) and (15).

The approach give in Eurocode 5 does not account for the beneficial impact of larger connection width or connection height and hence, underestimates the load-carrying capacities of such connections.

4. Considerations for the Design of Connections Loaded Perpendicular to Grain

The design approach proposed by Ballerini [9] in Eq. (13) considers various geometrical parameters and configurations of connections and shows the best agreement with test results. Based on the tests under short term duration of load a characteristic material property value of $C_{1,Ba,k} \approx 10 \text{ N/mm}^{1.5}$ can be found for the design approach by Ballerini in Eq. (13) as shown in Fig. 10. In case it is decided to keep the current approach in EC 5, despite its limits in consideration of connection geometry, a value of $C_{1,EC5,k} \approx 9.5 \text{ N/mm}^{1.5}$ is proposed.

For the determination of design values, the characteristic 5 %-fractile values have to be reduced by the partial safety factor for the material $\gamma_M$ and the modification factor $k_{\text{mod}}$ for the consideration of duration of load effects and the impact of
Fig. 10 Cumulative distribution of material parameter $C_1$ calculated from test results.

service classes. It is well known that timber is prone to cracking if loaded in tension perpendicular to grain and if exposed to varying moisture content. Hence, the application of connections loaded perpendicular to grain should be limited to situations with low variation in moisture content. Otherwise reinforcement should be installed in order to avoid cracking and to maintain the load-carrying capacity even after cracks occur [8]. Reinforcement can also help to avoid premature failure in cases of long duration of loads since knowledge about the behaviour of connections loaded perpendicular to grain under long duration of load is scarce.

5. Conclusion

The most relevant geometrical parameters with regard to the load-carrying capacity of connections loaded perpendicular to grain are height $h$ and width $b$ of the beam, relative connection height $h_c/h$, connection width $a$ and connection height $h_m$. In case of multiple connections the distance between them has an important influence as well. The position of the connection along the beam span or at a cantilever beam is of minor relevance with regard to the load-carrying capacity.

The various design approaches from literature can be separated into approaches based on strength criteria (like the approach given in DIN 1052) and into approaches based on fracture mechanics theory (like the approach given in EC 5 or the approach by Ballerini). A good fit between a large number of test results and the design approach by Ballerini was found.

Reinforcement is an easy and efficient possibility to restore the load-carrying capacity of beams with connections loaded perpendicular to grain.

6. Acknowledgement

The work presented in this paper was developed during a Short Term Scientific Mission at Technical University of Munich supported by COST Action FP 1402 (www.costfp1402.tum.de). The students C. Thiede and D. Gisler are thanked for their contributions in the frame of their master thesis.
7. References


