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Design of a long span Belfast truss using UK home-grown timber (ECCM –ECFD 2018 Conference)

D. Johnstone¹, R. Hairstans² and *A. Livingstone³

¹ Edinburgh Napier University, 10 Colinton Rd, Edinburgh EH10 5DT, cocis@napier.ac.uk
www.napier.ac.uk

² Edinburgh Napier University, Unit 1, 7 Hills Business Park, Bankhead Crossway South,
Edinburgh EH11 4EP, R.Hairstans@napier.ac.uk www.napier.ac.uk

³ Edinburgh Napier University, Unit 1, 7 Hills Business Park, Bankhead Crossway South,
Edinburgh EH11 4EP, A.Livingstone@napier.ac.uk www.napier.ac.uk

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1 INTRODUCTION

Industry partners of Edinburgh Napier University presented the desire for a long span timber roof system capable of delivering a clear span of 30m for industrial building projects, utilising UK Home-grown British spruce (WPCS). The Belfast truss design was identified for historical and aesthetic reasons, Figure 1.

Design of the members and the connections were investigated. We found that although home-grown timber is often perceived as being inferior to imported timber it can have greater density, which is important in the design of timber connections. This research investigates the advantages of designing this structural roofing system using the recently defined for market bespoke strength class C16+, which with greater characteristic strength and density values, better fits the properties of UK grown spruce.

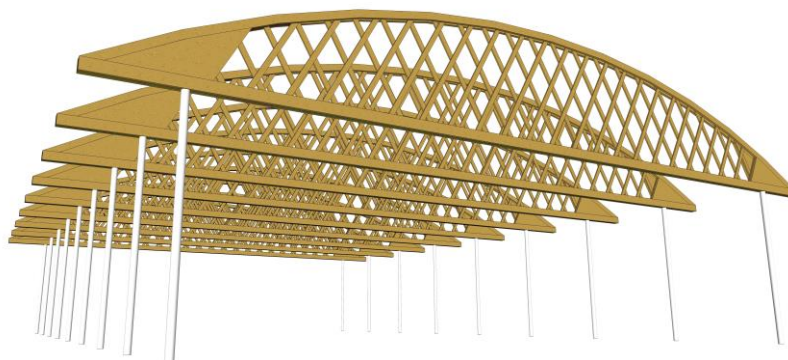


Figure 1 Belfast truss design

2 UK HOME-GROWN TIMBER

At present, roughly 53 million cubic metres of raw wood material is used in the UK each year; 81% of that is imported timber [1, 2]. In 2014 the UK was the third highest importer of timber products in the world, behind China and Japan. 13% of the land area in the UK is covered by forest. However the European average is 38%, so the UK is still behind its neighbours. This UK percentage of forest cover was as low as 5% at the turn of the 20th century, so the UK timber industry is making progress (information from Forestry Commission website [3]).

38% of sawn timber and 52% of wood panels used in the UK are from home-grown sources, and it is predicted that the production of timber in the UK is to increase by 50% by 2025 [1]. As the UK timber industry increases, we must look to rely less on imported timber and start using more home-grown. From the latest Forestry Commission forecasts, we can see the current spread of species within the UK, Figure 2; the information is from [4, 5]. The results show that British spruce (WPCS) is dominant in terms of availability and cost. The two Species that makeup WPCS are Sitka spruce (*Picea sitchensis*) (~90%) and Norway spruce (*Picea abies*, PCAB). For the remainder of this research, we will be focusing on utilisation of British spruce (WPCS).

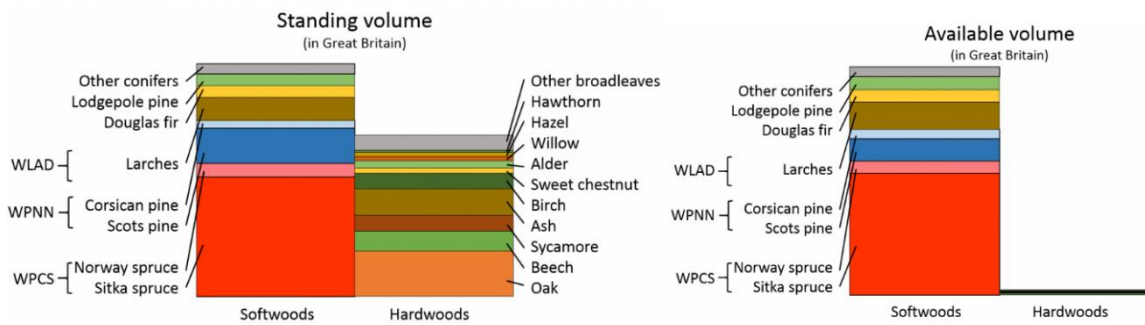


Figure 2 Standing and Available over bark volume of home-grown wood in the trees (in Great Britain) [4-6]

Home-grown timber is often seen as inferior to imported timber because a relatively small amount of the UK spruce can be graded to C24 and the industry grades to C16 / reject as routine. But with perfect grading machines, the same material can achieve the results shown in Table 1.

Table 1 UK Home-grown timber C24 grading potential

Percentage achieving C24 grade	Timber species
~30%	British spruce
~75%	UK larch
~90%	UK IE Douglas fir

The reason British spruce cannot currently achieve high yields of C24 is because not enough of it can reach the requirement for strength, stiffness and density. The main limiting property is stiffness, rather than strength and density. The timber grading is determined by whichever of the three “grade determining properties” (strength, stiffness and density) is the one closest to the requirement. So the limit is the timber, not the grade/class. These limits have been defined as they correspond fairly well with properties of the majority of species normally used for construction [7]. Although the number in the grade corresponds with the bending strength, the actual bending strength of the timber can be much higher, and this is often the case with UK timber [8]. The strength classes in BS EN 338 [9] are not completely separate from each other; as property distributions quite often overlap

between classes [7]. UK timber is generally said to grow too quickly, meaning that it has a low density. However, studies at Edinburgh Napier University on one of the most popular UK timbers, Sitka spruce, have shown that the density is the least limiting factor of the timber. It is correct that it achieves saw log size with a short rotation; the drawback to this is not low density but low stiffness due to the large ratio of juvenile wood [8].

For example, UK Spruce is often graded C16 but as shown in Table 2 the bending strength and density are much higher than the grading limits; it is the stiffness that is limiting it to C16. Recently UK sawmills have been looking into marketing a bespoke strength class C16+ strength class, which with greater characteristic strength and density values, better fits the properties of UK grown spruce [10].

Table 2 Some strength class requirements [10]

Class	Bending strength $f_{m,k}$ kN/m ²	Bending stiffness $E_{m,0,mean}$ kN/m ²	Density ρ_k kg/m ³
C14	14.0	7.0	290
C16	16.0	8.0	310
C18	18.0	9.0	320
C16+	18.5	8.0	330
C20	20.0	9.5	330
C24	24.0	11.0	350

Note: $f_{m,k}$ and ρ_k are 5th percentiles

$E_{m,0,mean}$ is mean

As timber comes from trees, the maximum length of sawn timber for construction is restricted by how high trees grow. Standard timber sizes available commercially are usually 100×50mm and 150×50mm and around 3.6m or 4.2m in length, as anything longer or larger in cross-section can be more difficult and expensive to source. However, industry partners to the project, BSW Timber, have indicated that their large section mill in Fort William can produce section sizes up to 450mm × 400mm and up to 8m in length.

3 TIMBER CONNECTIONS AND THE EFFECT OF TIMBER DENSITY

It is commonly stated that “a structure is a constructed assembly of joints separated by members” [11] and in timber engineering, the joint is generally the critical factor in the design of the structure. The strength of the connectors in the joint will normally dictate the strength of the structure; their stiffness will greatly influence its overall behaviour, and member sizes will generally be determined by the numbers and physical characteristics of the connector rather than by the strength requirements of the member material.

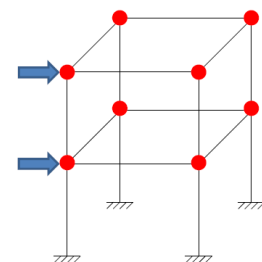


Figure 4 Structure and connections

The equations used in EC5 rely upon three main parameters of influence for the load carrying capacity and behaviour of joints with dowel type fasteners, which are:

1. The bending capacity of the dowel or yield moment.
2. The withdrawal strength of the dowel.

3. The embedding strength of the timber or wood-based material.

Note: the timber density is the only timber property that is used in the calculation of lateral load-carrying capacity.

Bending capacity or yield moment is theoretically the maximum bending moment that the dowel type fastener can resist before going into a plastic deformation. Figure 5 shows a metal dowel connection that has been tested into plastic deformation. The possible strength increase due to the plastic deformation is disregarded.



Figure 5 Metal dowel shear test

Characteristic withdrawal capacity within EC5 for nails comes from the minimum of the point-side axle withdrawal and the head-side pull-through. Once the characteristic withdrawal strengths are identified, either from the manufacturer's test data

or from the EC5 equations, you may still be required to multiply by a reduction factor depending upon the penetration depth.

The embedment strength of the timber, or wood-based product, f_h , is the average compressive strength at maximum load under the action of a stiff straight dowel. According to BS EN 383 [12], see Figure 6.

$$f_h = \frac{F_{max}}{d \cdot t} \quad (BS EN 383 eq.2)$$

Where:

- F_{max} is maximum load of the test, or load at which a 5mm deformation occurred
- d is the fastener diameter
- t is the thickness of the timber

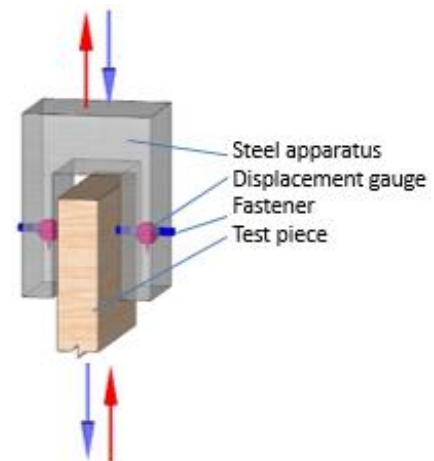


Figure 6 Embedment test

It's been reported that from experimental tests, the influence factors for the embedment strength of timber are [13, 14]:

- Density: embedment strength increases in a linear manner with respect to timber density.
- Moisture content: as this increases the bending strength decreases, and this is independent of timber species and dowel diameter.
- Diameter of the fixing or the predrilled hole: the embedment strength decreases with increasing fastener diameter.
- Reinforcement of timber perpendicular to grain: for example, installing self-tapping wood screws either side of a bolted connection will increase embedment strength within that timber member.
- Friction between the fixing and timber will increase embedment strength.

The embedment strength calculation method within EC5 is as follows: For connections in timber and LVL using bolts up to 30mm diameter.

$$f_{h,\alpha,k} = \frac{f_{h,0,k}}{k_{90} \sin^2 \alpha + \cos^2 \alpha} \quad \text{EC5 eq. (8.31)}$$

$$f_{h,0,k} = 0.082 (1 - 0.01 d) \rho_k \quad \text{EC5 eq. (8.32)}$$

Where:

$$k_{mod} = \begin{cases} 1.35 + 0.015 d & \text{for softwoods} \\ 1.30 + 0.015 d & \text{for LVL} \\ 0.90 + 0.015 d & \text{for hardwoods} \end{cases} \quad \text{EC5 eq. (8.33)}$$

and:

- $f_{h,0,k}$ is the characteristic embedment strength parallel to grain, in N/mm²
- ρ_k is the characteristic density of the timber in kg/m³,
- α is the angle of the load to the grain
- d is the diameter of the bolt in mm

Figure 7 shows the increase in lateral shear resistance as density increases for a standard M6 bolt acting in single shear for all 6 of the failure modes.

The calculation approach used within EC5 is based upon Johansen’s general theory [15, 16], and predicts the method of failure for the timber-to-timber and steel-to-timber connections using dowel type fixings. This calculates each possible failure mode in turn and so identifies the failure mode with the lowest resistance. The general format for these equations can be summarised as: the result from the Johansen’s yield load, multiplied by any effect from friction, plus the rope effect. See Figure 8.

$$F_{v,Rk} = \text{friction factor} * \text{johansen yield load} + \text{Rope effect}$$

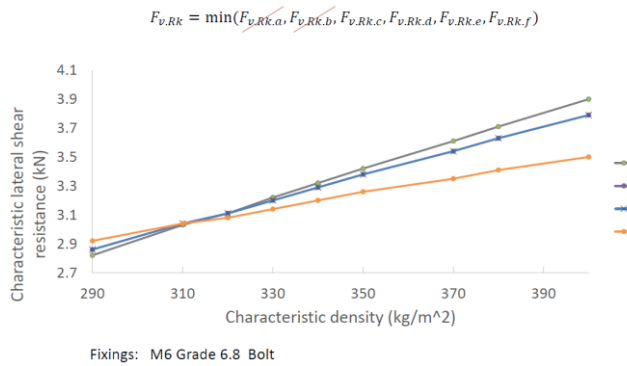


Figure 7 Density effects on resistance of a fastener

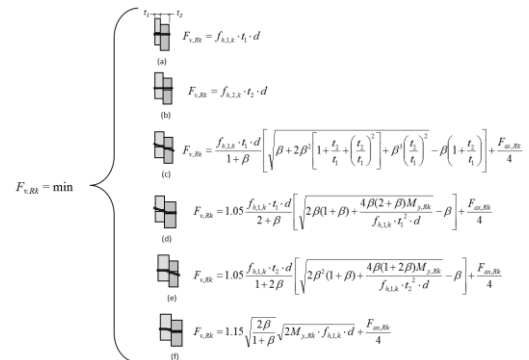


Figure 8 Single shear - failure modes and equations

There are two types of friction effect that can arise between the two timber members in a connection. The first will develop if the members are in direct contact when assembled; this friction will be eliminated either if there is no direct contact on assembly or if there is shrinkage of the timber or wood products in service. As a result of this it is conservatively not considered in EC5. The other will arise when the fasteners yield, pulling the members together as the fasteners deform (see Figure 9). This type of friction will always arise in failure modes that include yielding of the fasteners, relating to such modes.

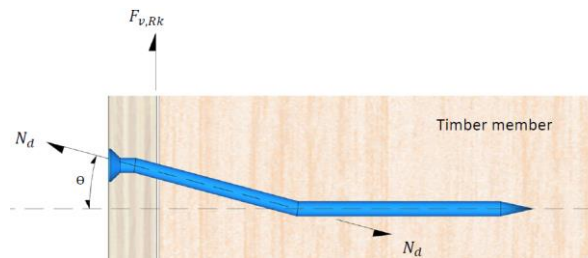


Figure 9 Dowel in single shear, fastener fully yields

The rope effect is the contribution to the lateral load carrying capacity of the point side withdrawal of dowel type fastener. In order to calculate and optimise the structural timber connection designs, this research utilised an automated structural timber connection calculation software, that was created as an output of previous research work and is available on the Tekla Tedds platform, Figure 10 shows an output example.

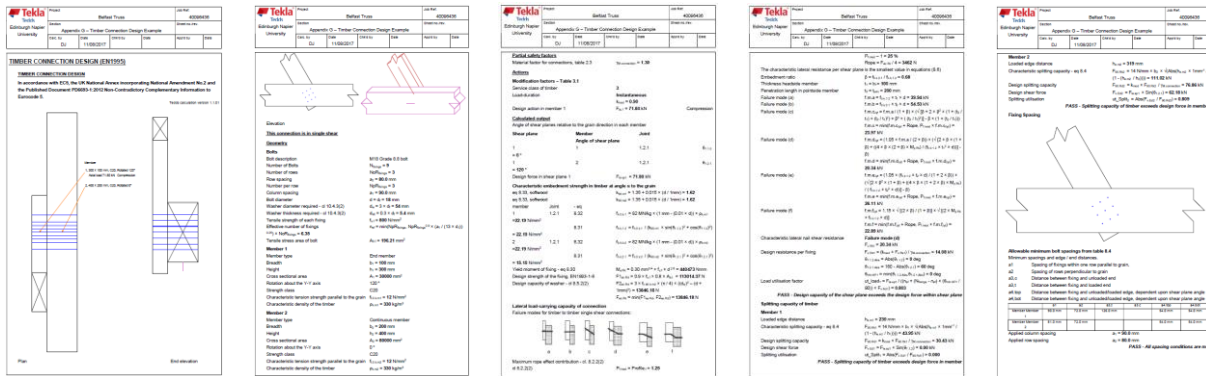


Figure 10 Output from the newly created automated connection calculation software

4 BELFAST TRUSS DESIGNS

A Belfast Truss is a variation of the bowstring truss with a curved top chord and horizontal bottom chord. The chords are linked together with a lattice bracing which can be of various arrangements. Older descriptions define a Belfast truss when the lattice bracing connecting the top and bottom chord is designed to meet at right angles at uniform spacing on the top chord [17]. However a lot of examples do not fit this specific arrangement; an example of a Belfast truss roof at Leuchars Airfield is shown in Figure 11. The Belfast truss is very efficient at taking uniformly distributed loads because the shape of truss is very similar to the shape of the bending moment diagram [18].

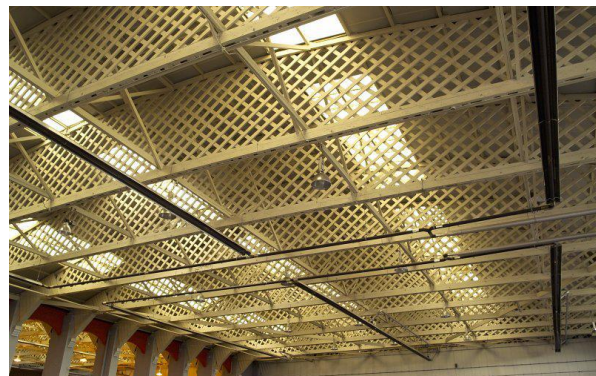


Figure 11 Belfast Truss Roof of a General Service Shed at Leuchars Airfield

The Belfast truss was first referenced in 1866 when McTear & co. advertised it as a ‘durable, cheap and handsome roof for felt’ [18]. Soon after that several other companies were offering the same product, such as D. Anderson & Sons Ltd. The use of the Belfast truss roof was fairly common throughout Great Britain and Ireland, most prominently near ports as the required timber was easily accessible. They were mostly used to construct shipbuilding sheds, farm buildings and general-purpose hangars for spans ranging from 6m to 20m [19], until the need for increased spans saw the use of long-span timber truss construction decline. However they were still often used for small and medium span building; photographic records by Harland & Wolff held by the National Museums of Northern Ireland show that many of the buildings of the Belfast Shipyards in the 1920’s and 1930’s had Belfast roofs, but most of these buildings no longer exist [17].

Different companies made different variations of the design with the same basic shape:

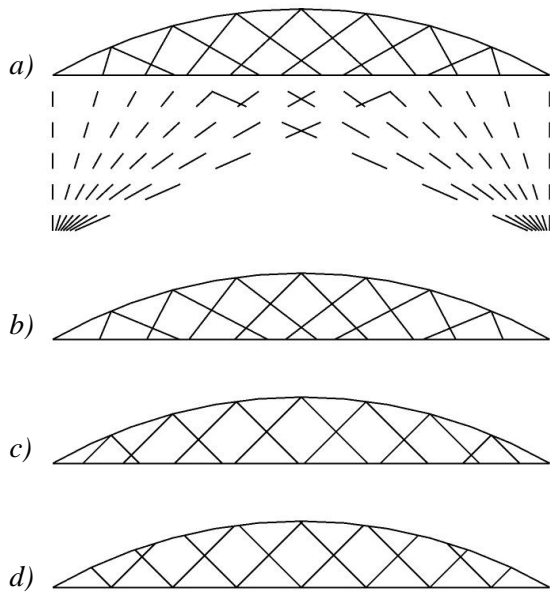


Figure 12 Different configurations of Belfast Truss

a) For the original McTear truss each member of the lattice bracing, if projected below the bottom chord, would meet at one of two points about one-third of the span below the end of the bottom chord as shown in Figure 6.20a.

b) Figure 6.20b shows the Anderson design of the truss where the bracing members are positioned at 45° to the top chord and uniformly spaced along the top chord.

c) The modern Belfast truss (Figure 6.20c) has the lattice bracing members at 45° to the bottom chord and uniformly spaced along the top chord.

d) The more conventional bowstring truss shown in Figure 6.20d has the lattice bracing members at 45° to the bottom chord and uniformly spaced along the bottom chord. Gould et al indicated that this is probably not justified as being a Belfast truss (Gould, Jennings, & Montgomery, *The Belfast Roof Truss*, 1992). However there are examples of this design being called a Belfast truss.

Modern examples of Belfast truss roof designs Case studies.

In 2010 Wynstones School completed the construction of their new school hall (Figure 13), which incorporated the Belfast roof truss. Wynstones is a hands-on school, so with the help of experienced carpenters, the pupils helped to construct nine 18.6m long Belfast trusses. However, unsure about the all original timber design, the architects and engineer on the project revised the design to include steelwork at either end of the building 'ensuring the timber frame won't wrack in either direction'. From pictures, it looks as though the Anderson design has been used for the Wynstones truss, it is also noted that the ends of the truss are supported by haunched columns, which will provide a moment resisting connection.

The sawmill building of Esgair Timber in Wales is constructed with a Belfast Truss Roof. These trusses are 24m long and were designed by Barratt Associates, it can be seen in Figure 14 that the lattice is spaced evenly along the bottom chord and the angles are set from the bottom chord, which means they would be considered as conventional bowstring trusses. It is also clear that the angle of the lattice bracing is not of the traditional 45° ; this could possibly provide a structural benefit to the truss, but at this stage that is unclear. Discussions with Barratt Associates indicated that the success of this building was as much to do with the skills of the carpenter as it was the engineering design.



Figure 13 Wynstone School, Belfast trusses (photo courtesy of David Gregory architects)



Figure 14 Belfast trusses at Esgair Timber (photo courtesy of Barratt Associates)

The standard height to length ratio of the trusses were generally between 1:5 and 1:10, and it has been found that as the ratio increases it reduces the angle at the eaves and increases the forces at this point [19]. For modern trusses it is useful to set the radius of the arched top chord equal to the length of the truss. Therefore the height of the truss is roughly 13.4% of the truss length [20].

Example:

Span	$C = 30 \text{ m}$
Arc_radius	$R = 30 \text{ m}$
Height	$H = R - \sqrt{(R^2 - (C^2 / 4))} = 4.019 \text{ m}$
	$(100 / C) \times H = 13.4 \%$

2D analysis of the truss frame was undertaken for both the standard characteristic properties of C16 to EN 338 [9] and the newly defined home-grown C16+ for British spruce (WPCS) [1, 8, 10]. Then it was identified that the maximum bending moment in the bottom chord was between the eaves supports and the quarter span position as shown in the bending moment diagram in Figure 15 and Figure 16 This confirms what was established in the review of previous work regarding the maximum bending moment position. This is quite different from a standard beam where the maximum bending moment is generally found in the centre of the beam. The bow-string type Belfast truss was the only configuration investigated in this project, replicating what was found in the case study of the truss at Esgair Timber. The review of previous work suggested that different configurations of the latticework within the Belfast truss make little difference to the structural behaviour of the truss. However, it was found that changing the angle of the latticework had an impact on forces in the lattice members. The resulting angle chosen was 60° as this reduced the length of the lattice member and gave a suitable connection design. While undertaking the connection design, problems with the splitting capacity of the bottom chord led to trying a truss with a lattice bracing at an angle of 45°, which gave a worse splitting capacity utilisation. It may be the case that because we are looking at a larger span, this may be magnifying the effect of the different configurations. Part of the configuration is the spacing of the lattice bracing; initially a 2m spacing was applied but the review of the 2D analysis identified that the loads in the lattice members were quite high, so this spacing was reduced to 1m. This effectively doubles the number of lattice members, reducing the load in each member, which reduces the connection design. The member size can be reduced only if the adjusted size still conforms to the EC5 member design calculations. A suitable balance must be struck between these in order to get the most efficient design.

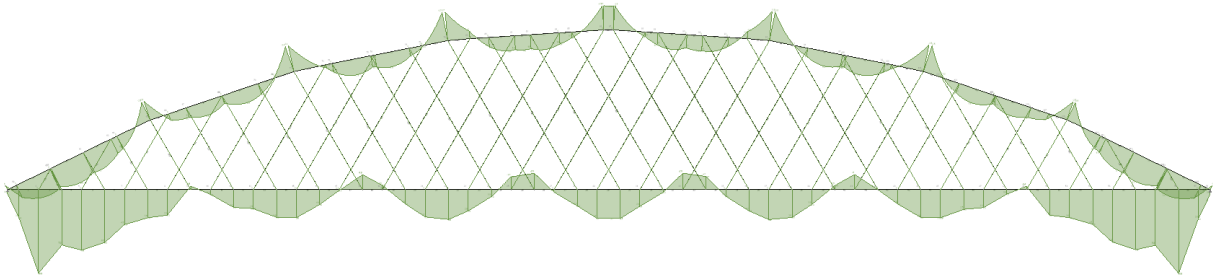


Figure 15 Bending moment example diagram

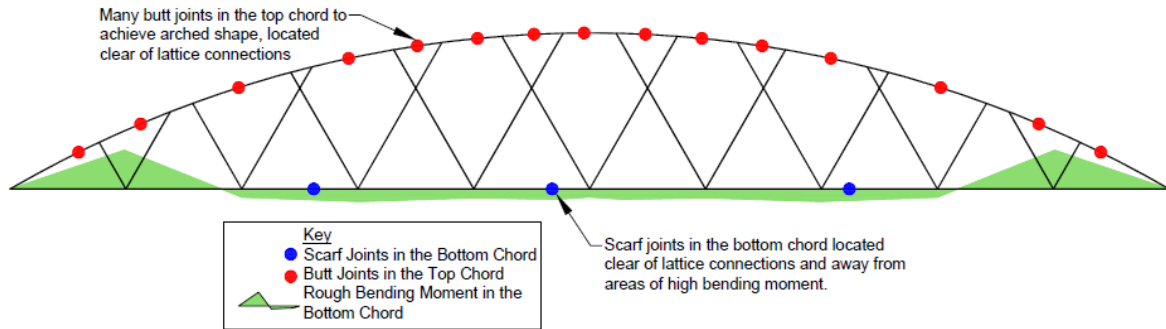


Figure 16 Appropriately located butt and scarf joints

Member design before final optimisation: the top chord is comprised of two 200mm x 250mm sections on either side of the lattice bracing with a 600mm x 50mm top piece to join the two side pieces of the top chord together. When the top chord is in compression then the full cross-section can be utilised but due to the staggered butt joints (See Figure 16), when the top chord is in tension then only one side section and the top piece is utilised. Then the bottom chord is comprised of two 200mm x 400mm sections on either side of the lattice bracing, as the bottom chord is jointed with scarf joints (Figure 18) Then the full cross-section can be utilised in both compression and tension. However the bending strength of the bottom chord will need to be reduced in the locations of any scarf joint to a third of its equivalent solid timber section; Figure 16 shows appropriate locations of main member joints. Lattice members are 100mm x 250mm single pieces of timber. Figure 17 contains the cross-sectional dimensions for the member design before final optimisation with respect to timber density.

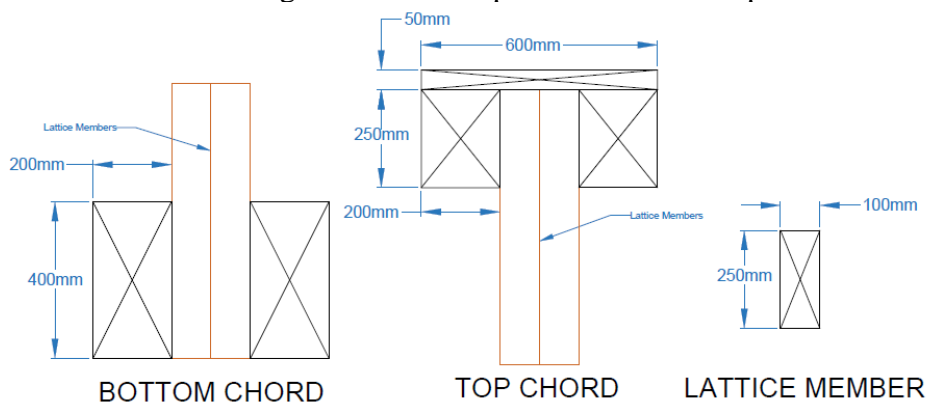


Figure 17 Cross-Section of truss members

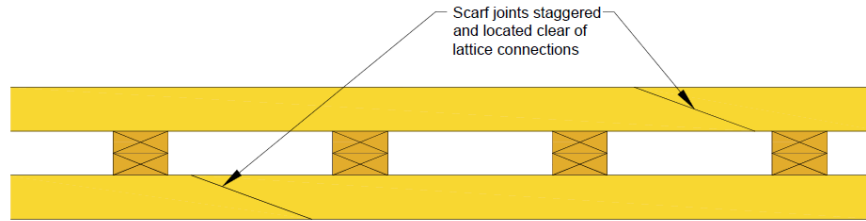


Figure 18 Plan view of bottom chord, Scarf joint

5 CONCLUSIONS

Home-grown timber is often perceived as inferior to its imported equivalent but from the results of this research, it has been demonstrated that home-grown timber can satisfy all the Eurocode 5 structural checks for this Belfast truss design. When designing the connection, the only timber property that the calculation takes into account is the density of the timber. The new created C16+ grade of UK home-grown timber has a greater density than that of standard graded C16.

Most C16 timber on the UK market is home-grown, but when designing this Belfast truss example, using the density properties of the C16+ in comparison to standard C16, we observe that the calculated minimum size of the timber members and the fixings can be reduced. For illustration: The calculated size of the bottom chord timbers and the size and number of bolts in the connection were able to be reduced for the C16+ design, see Table 1 and Table 3. But when calculating for the standard C16 grade, we find failure in shear capacity for this reduced design. This proves that utilising the greater density characteristics of the C16+ graded home-grown timber can provide stronger connections with fewer fasteners, thus allowing section sizes of the members to be reduced.

Table 3 Connection Design for C16 and C16+ timber members

Timber	Density ρ_k kg/m^3	Shear Capacity Utilisation	Splitting Capacity Utilisation	Connection
C16	310	0.843	0.809	9 no. M18 bolts, 400x200mm chord
C16+	330	0.803	0.809	9 no. M18 bolts, 400x200mm chord
C16	310	1.037	0.998	6 no. M16 bolts, 350x150mm chord
C16+	330	0.977	0.998	6 no. M16 bolts, 350x150mm chord

In conclusion, this example of a 30m clear span Belfast truss roof system using UK home-grown timber graded to C16+ as opposed to standard C16 grade timber generated savings of:

- Timber section dimensions savings of 34.4%
- Metal fixings (bolts) savings of 47.3%

6 FUTURE WORK

This project was ideal for demonstrating a computational timber connection calculation software previously created within the research group. This software allowed for parametric optimisation of the Belfast truss design. Continued work will focus on the digitization design capacity for timber-rich houses, with a focus on home-grown timber.

ACKNOWLEDGEMENTS

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REFERENCES

1. Davies, I., *Sustainable construction timber: sourcing and specifying local timber*. 2016: Forestry Commission.
2. *Forestry Commission, Statistics and Forestry Facts & Figures for 2017*. 2018 [cited 2018 14/04/2018]; Forestry Statistics and Forestry Facts & Figures]. Available from: <https://www.forestry.gov.uk/forestry/inf-d-7aqdgc>.
3. *Forestry Commission website* 2018 [cited 2018 14/04/2018]; Forestry Commission website]. Available from: <https://www.forestry.gov.uk/>.
4. Brewer, A., *50-year forecast of softwood timber availability*. 2014, Forestry Commission, National Forest Inventory: <http://www.forestry.gov.uk/inventory>.
5. Brewer, A., *50-year forecast of Hardwood timber availability*. 2014, Forestry Commission, National Forest Inventory: <http://www.forestry.gov.uk/inventory>.
6. Ridley-Ellis, D., *Grade in Britain*. 2016: <http://blogs.napier.ac.uk/cwst/grade-in-britain/>.
7. Ridley-Ellis, D., P. Stapel, and V. Baño, *Strength grading of sawn timber in Europe: an explanation for engineers and researchers*. European Journal of Wood and Wood Products, 2016. **74**(3): p. 291-306.
8. Ridley-Ellis, D., S. Adams, and S. Lehneke, *Thinking beyond the usual strength grades-with examples of British spruce and larch*. 2016.
9. BSi-BS-EN-338, *BS EN 338:2016 Structural Timber - Strength Classes*. 2016, BSI.
10. Ridley-Ellis, D., *Derivation of GoldenEye-702 grading machine settings for British Spruce*. 2014, Report for CEN TC124/WG2/TG1: Edinburgh Napier University.
11. McLain, T.E. *Connectors and fasteners: research needs and goals*. in *Wood Engineering in the 21st Century: Research Needs and Goals*. 1998. ASCE.
12. BSi-BS-EN-383, *BS EN 383:2007 Timber Structures- Test methods _ Determination of embedment strength and foundation values for dowel type fasteners*. 2007.
13. Thelandersson, S. and H.J. Larsen, *Timber engineering*. 2003: John Wiley & Sons.
14. Rammer, D.R. and S.G. Winistorfer, *Effect of moisture content on dowel-bearing strength*. Wood and fiber science, 2007. **33**(1): p. 126-139.
15. Johansen, K., *Forsøg med træ for bindelser (Experiments with wood for connections)*. Bygningsstatistiske meddelelser 1941. **2**.
16. Johansen, K. *Theory of timber connections*. in *International Association of Bridge and Structural Engineering*. 1949.
17. GOULD, M.H., *A Historical Perspective on the Belfast Truss Roof Construction History*, 2001. **17**(2001): p. 75 - 87.
18. Gould, M., A. Jennings, and R. Montgomery, *Belfast roof truss*. Structural Engineer, 1992. **70**: p. 127-9.
19. Gilfillan, R. and S. Gilbert, *The 'Belfast' Roof Truss—Worth Conserving?* Journal of Architectural Conservation, 2003. **9**(1): p. 45-57.
20. Gilbert, J.G.S., *The Historic Belfast Timber Truss-A Way To Promote Sustainable Roof Construction*. 2002.