Structural Design Guide

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1. Introduction & Scope of Manual

1.1. This Design Manual has been prepared as a design aid to Structural Engineers in the structural design of their Structural Insulated Panels structures (SIPby GreenPanel). It is expected that the Structural Engineers shall have a good working knowledge of EN 1990, EN 1991 (parts 1, 3 & 4) and EN 1995-1-1 and EN 1995-1-2.

It is further assumed that the Structural Engineer is experienced in timber frame / SIP frame design.

1.2. SIP by GreenPanelis a composite, sandwich panel manufactured by injecting PIR foam nominal density of 46 Kg/m^3, in a readymade mold with outer skins of OSB3.

1.4 The SIP by GreenPanel section depths that form the integral part of this document are122mm, 152mm, 182mm, 212mm and 242mm. This should not exclude other sizes being added in the future as the calculation methods described within can be readily adapted for any combination of thickness of OSB skins and PIR rigid foam core.

1.5 The SIP panels described within are currently limited to roof and wall elements. SIP lintels are also designed and incorporated as part of the wall design.

The “SIP structure” can also include some or all of the following elements:

1.5.1. Solid timber ceiling ties.

1.5.2. Solid timber stud walls as per structural engineers design and specification.

1.5.3. Solid timber or engineered floor joists (I-Joists or Posi Joists) which are seated on the load-bearing SIP external wall panels and the internal load-bearing walls.

There is typically a rim beam running around perimeter of floor.

Joists may also be hung from side of SIP panels using proprietary joist hangers such as Simpson Strong-tie IUQ and HIUQ hangers or similar. The joist hangers must be allowed by the proprietary hanger manufacturer for this purpose.

1.5.4. Solid timber, engineered timber or steel floor beams and purlins which shall be designed to adequately resist the applied loading as per normal structural design principles.

As items described in 1.5.1 to 1.5.4 are not limited to SIP structure and their design is standard and very well described in numerous publications, this document shall, henceforth, limit itself to structural design specifically relating to the SIP panels.

1.6. The main objectives of this design guide are as follows:

1.6.1. To produce a design guide and accompanying tables to provide a basis for design ofthe Sips by GreenPanel to Eurocode design.

These designs should be carried out by a Structural Engineer experienced in the design of SIP panels.

1.6.2. To provide SIP lintel table.

2. Standards Used and Other References

2.1 EN 1990: 2002 + A1: 2006, Basis for Structural Design.

2.2 EN 1991: 2002 + A4: 2004 - Action on Structures: Part 1-1:General actions – densities, self-weight, imposed loads on Buildings.

2.3 EN 1991-1-3: 2003 - Actions on Structures: Part 1-3:Snow loads.

2.4 EN 1991-1-4:2005 + A1: 2010 - Actions on Structures: Part 1-4:Wind loads.

2.5 EN 1995-1-1:2004 + A1:2008 - Design of Timber Structures: Part1-1: General Common rules and rules for Buildings.

2.6. EN 1998 Design of structures for earthquake resistance: Part 1: General rules, seismic actions and rules for buildings

2.7 EN 300: 2006 - Oriented Strand Boards (OSB) –Definitions, Classifications and Specifications.

2.8 EN 13163 - Thermal insulation products for buildings.

2.9 EN 13165 - Thermal insulation products for buildings, Polyurethanes.

2.10 EN 12369-1: 2001 - Wood-based Panels-Characteristic Value for Structural Design– OSB, particleboard and fiberboards.

2.11 EN 338: 2009 - Structural Timber: Strength

2.12 Non-standard References

2.12.1 TR019 - EOTA Technical Report: calculation models for prefabricated wood based load bearing stressed skin panels for use in roofs.

2.12.2 Manual for the Design of Timber Building Structures to Eurocode 5 by *TRADA and I.Struct.E*.

2.12.3 ETAG 019. Prefabricated wood based loadbearing Stressed Skin panels

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3. Background - TR019: calculation models for prefabricated wood based load bearing stressed skin panels for use in roofs.

3.1 There are several types of stressed skin panels presented in TR019. In relation toSIP panel design, there are 2 of relevance:

3.1.1 Type A - Stressed skin panels, closed box type double skin, without wooden ribs, with load-bearing insulation (i.e. SIP panels with SIP splines only at panel /panel connection).

3.1.2 Type B1 - Stressed skin panels, closed box type double skin, with wooden ribs and load-bearing insulation (although the design methodology for SIP panels with splines ignores the effect of the insulation in this case. Longer spans may be possible in the future with clarity on the TR019 design method of sandwich panels with wooden ribs and load-bearing insulation).

3.2 In all cases, it is critical to the design that the grain / strand direction of timber / OSB is parallel to the span direction.

3.3 Although TR019 is primarily for use for roof panels, the principles of TR019 and EN1995-1-1 can be used to design SIP wall panels as will be described in further detail in Section 9.

3.4 Shear deflection: Due to the relatively low shear modulus (G_c) of the polyisocyanurate core (which transfers shear loads within the sandwich panel), the shear deflections in SIP panel design are significant and must be calculated for both the instantaneous deflection and final deflection stages.

3.5 Creep: The deformation modification factor, k_{def} for both OSB and polyisocyanurate (PIR) are high and hence the additional deflection due to creep over time is very often critical. The effect of creep must be applied to both bending and shear deflections.

In addition, the ultimate limit state design checks (bending, shear, etc) must be carried out for both the instantaneous response of the material to the applied actions and the final (re-distributed) response. These are based on the reduced modulus of elasticity and shear modulus of the outer skin OSB (typically in Service class, SC 2),the Inner skin OSB (typically in Service Class, SC 1) and the PIR core.

Note that the k_{def} value of OSB in SC1 (1.5) and SC2 (2.25) differ. This means that, even when the OSB for inner and outer skins are identical, the neutral axis of the SIP panel at the final (re-distributed) response stage shall not be at mid-depth of the SIP panel.

3.6 TR019 is based on the Kreuzinger model which shall be described in greater detail in Section 7.

4. <u>Material Properties</u>

4.1 Outer and inner wood panel skins - These are both 15mm OSB/3 which shall be incompliance with EN 300 and EN 12369-1. Bonding of the OSB and the foam is secured by injecting the insulation in a readymade mold in a quality controlled factory process.

OSB3 properties have been used as described in Table 1 below, which represents the minimum permitted strength and stiffness values (based on EN12369-1: 2001). Only values of relevance to the design of SIP roof panels, SIP wall panels and SIP lintel have been stated.

Table 1: Properties and Density Values for 15mm OSB outer and inner boards

Strength / Stiffness / Density Property	EC5 notation	Values and units
Characteristic tension stress	$f_{t,0,k}$	9.4N/mm²
	$f_{t,90}$	7N/mm²
Characteristic compression stress	$f_{c,0,k}$	15.4N/mm²
	$f_{c,90}$	12.7N/mm²
Characteristic panel shear stress	$f_{v,0,k}$	6.8N/mm²
Characteristic rolling shear stress	$f_{r,k}$ (or $f_{v,r,k}$)	1.0N/mm²
Characteristic flexural stress	$f_{m,0,k}$	16.4N/mm²
	$f_{m,90}$	8.2N/mm²
Characteristic density	ρ_k	600kg/m³
Mean modulus of elasticity	$E_{t,0,mean}$ & $E_{c,0,mean}$	3800N/mm²
Mean Panel modulus of rigidity(shear modulus)	$G_{v,mean}$	1080N/mm²
	$E_{m,0}$	4930N/mm²
	$E_{m,90}$	1980N/mm²
	$E_{t,90}$	3000N/mm²
	$E_{c,90}$	3000N/mm²
	G_r	50N/mm²
	$K_{mod.perm}$	0.3 N/mm²
	$K_{mod.long}$	0.45 N/mm²
	$K_{mod.med}$	0.65 N/mm²
	$K_{mod.short}$	0.85 N/mm²
	$K_{mod.instant}$	1.1 N/mm²
	$K_{def.perm}$	SC1 = 1.5 SC2 = 2.25
	γ_μ	1.3
	λ(w/mK)	0.13
	μ	200/300
	linear expansion	0.3

Based on Table 3.2 of EN1995-1-1, the values for k_{def} (permanent load deformation modification factor) is as follows:

4.1.1 OSB/3 in Service Class 1 environment = 1.5

4.1.2 OSB/3 in Service Class 2 environment = 2.25.

4.2 Properties of the Polyisocyanurate (PIR) core:

4.2.1 Material properties of PIR have been used in the design as stated in Table 1 below.

Table 2: Rigid polyisocyanurate (min. strength class) Properties

Property	**notation**	**Values and units**
Characteristic panel shear stress	$f_{v,k}$	0.12N/mm²
	$f_{m,0}$	0.19N/mm²
	$f_{m,90}$	0.19N/mm²
	$f_{t,0}$	0.0N/mm²
	$f_{t,90}$	0.0N/mm²
	$f_{c,0}$	0.214N/mm²
	$f_{c,90}$	0.214N/mm²
	$f_{r,0}$	0.0N/mm²
	$E_{m,90}$	6.8N/mm²
	$E_{t,90}$	6.8N/mm²
	$E_{c,90}$	6.8N/mm²
Mean modulus of elasticity	$E_{c,mean}$	6,8N/mm²
Mean Core modulus of rigidity (shear modulus)	$G_{v,mean}$	2.5N/mm²
Deformation modification factor (permanent and long-term loads)	$k_{def,perm}$	3.0 (c)
Deformation modification factor (medium-term loads)	$k_{def,med}$	1.0 (d)
Deformation modification factor (short-term and inst.-term loads)	$k_{def,short}$	0.0 (e)
Characteristic density of core	ρ_k	46kg/m³
	λ	0.021 w/mK
	μ	90

Table Notes:

(a) Characteristic panel shear stress is based on interpolation from table in Section E3.1.1 of TR019.

(b) Mean core modulus of rigidity is based on interpolation from table in Section E3.1.1 of TR019.

(c) Deformation modification factor, $k_{def.med}$ for medium-term loads is based on calculation ($k_{def,perm}$ * Ψ_2), where $\Psi_2 = 0.3$ for floor imposed loading

(d) Deformation modification factor, $k_{def.short}$ for short and instantaneous-term loads is based on calculation ($k_{def,perm}$ * Ψ_2), where $\Psi_2 = 0.0$ for roof imposed, snow and wind loads.

4.3 Properties of the Timber Splines and Rails:

The characteristic strength and stiffness properties and the densities and deformation modification factors for the range of timber grades are as per EN 338: 2009 and EN 1995-1-1.

NO	Wood ratio	fmk (Mpa)	ft0k (Mpa)	ft90k (Mpa)	fc0k (Mpa)	fc90k (Mpa)	fvk (Mpa)	E0m (Mpa)	E05m (Mpa)	E90m (Mpa)	Gm (Mpa)	pk (Kg/M3)
1	C14	14.0	8.0	0.3	16.0	4.3	1.7	7000.0	4700.0	230.0	440.0	290.0
2	C16	16.0	10.0	0.3	17.0	4.6	1.8	8000.0	5400.0	270.0	500.0	310.0
3	C18	18.0	11.0	0.3	18.0	4.8	2.0	9000.0	6000.0	300.0	560.0	320.0
4	C22	22.0	13.0	0.3	20.0	5.1	2.4	10000.0	6700.0	330.0	630.0	340.0
5	C24	24.0	14.0	0.4	21.0	5.3	2.5	11000.0	7400.0	370.0	690.0	350.0
6	C27	27.0	16.0	0.4	22.0	5.7	2.8	12000.0	8000.0	400.0	750.0	370.0
7	C30	30.0	18.0	0.4	23.0	5.7	3.0	12000.0	8000.0	400.0	750.0	380.0
8	C35	35.0	21.0	0.4	25.0	6.0	3.4	13000.0	8700.0	430.0	810.0	400.0
9	C40	40.0	24.0	0.4	26.0	6.3	3.8	14000.0	9400.0	470.0	880.0	420.0

Kmode	0.6
γm	1.3
Kvol	1
Ksys	1.1

5. Loading on structural SIP panel elements and associated deflections (for SLS)

5.1 SIP panels will typically be subjected to some or all of the following loads:

5.1.1 Gravity loading: This is a combination of permanent (G_k) and variable loads (Q_k) applied to the structure based on loads to EN1991. There can be several different types of variable loads, each with different load durations.

Uniformly distributed loads for the SIP panels themselves can be calculated from the combined: (thickness of 2 no. outer skins in meters x 550 + thickness of core in meters x 46) to give a self-weight load in kg/m^2. Clearly if timber ribs / splines are used these should be additional to this based on mean timber density for the timber grade used.

5.1.2 Wind Loads perpendicular to plane of SIP panels, i.e. those loads resulting in compressive stresses on one OSB skin and tensile forces on the other skin. This load is also a type of variable load, $q_{k,wind}$.

5.1.3 Wind loads applied in the plane of the SIP panels, i.e. applying racking / shear forces to the SIP panels.

5.2 Load Cases:

5.2.1 The various loads are applied to the structure in the form of a set of load-cases which are separately applied in accordance with EN 1990 and the related national annex. There are often several load cases for the ultimate limit state and several load cases for the serviceability limit state.

5.2.2 Ultimate limit state load cases:

The basic equation, from which several load cases can be defined at ultimate limit state is:

$$\Sigma\ \gamma_{G,j}.G_{k,j} + \gamma_{Q,1}.\ Q_{k,1} + \Sigma\gamma_{Q,i}.\Psi_{0,i}\ .\ Q_{k,i}$$

j>=1 i>1

where

G_k and Q_k= Characteristic permanent and variable actions (loads) respectively.

γ_G = ULS partial load factor for permanent loads, i.e. 1.35.

γ_Q = ULS partial load factor for variable loads, i.e. 1.5.

Ψ_0 = Factor for combination value of a variable action (See table A1.1 of EN 1990). For domestic floor load and for imposed roof load, the value of Ψ_0 = 0.7, for snow load and for wind load, Ψ_0 = 0.5.

On a SIP roof panel, there would be typically 4 load-cases with each case having a different "leading" variable load, i.e. roof imposed load, snow load, man point load and wind load.

For the purposes of the Load-span tables within this document, a roof imposed load only has been considered, there will be cases where the snow load iscritical and the design engineer must consider the possibility of snow load being more critical than roof imposed load in their design.

On a SIP wall panel, there can be several other load-cases to consider due to the possibility of floor imposed loads, seism etc.

5.2.3 Serviceability limit state load cases:

In relation to SIP panel design of roof and walls only, limiting deflection to acceptable limits is the sole serviceability limit state criteria (vibration not a consideration for these elements). There are 2 parts to the deflection of a SIP panel, i.e. instantaneous deflection, u_{inst} and creep deflection, u_{creep}. These are added together to obtain the final deflection, $\mathbf{u_{fin}}$.

5.2.3.1 The basic equation, from which several load cases can be defined for instantaneous deflection is:

$\Sigma\ G_{k,j} + Q_{k,1} + \Sigma\psi_{0,i}\ .\ Q_{k,i}$ (instantaneous deflection load cases)

j>=1 i>1

This is the basis for similar load cases as set-up for the ultimate limit case, except the partial load factors are omitted for the serviceability limit state cases.

5.2.3.2 The basic equation, from which the load case can be defined for creep deflection is:

$\Sigma G_{k,j} + \Sigma\ \psi_{2,i}\ .\ Q_{k,i}$ (creep deflection load case)

j>=1 i>0

ψ_2 are the factors for the quasi-permanent value of variable actions (Table A1.1 of EN1990).

For all of the common roof applied variable loads, i.e. roof imposed, snow and wind, = 0 and therefore the creep deflection for roof loads is based on permanent (and include long-term loads such as storage also) loads only.

For walls, the main load causing deflection, i.e. wind load normal to the SIPs panel, has a “0” creep associated with it but there will be a creep deflection resulting from axial load induced deflections, i.e. from permanent, long-term and medium-term loads.

N.B. If designing using 5268, note that rafter imposed and snow loads are medium-term loads (short-term to Eurocodes) and floor loads are long-term loads (medium-term in Eurocodes). Therefore there may be a creep deflection to consider from roof panels and a larger creep deflection to consider from floor, snow and rafter imposed loads than would be case for Eurocode design.

5.3 Deflections:

5.3.1 Due to the large shear deflections and the large creep deflections, this is often the limiting criteria for SIPs. The final deflections are best calculated as follows for SIP panels:

$u_{fin,} = u_{fin,G} + u_{fin,Q,1} + \Sigma\, u_{fin,Qi}$ where i>=1 for final part of equation.

$u_{fin,G} = u_{inst,G}\,(1 + k_{def})$ (final defl. resulting from perm. action)

$u_{fin,Q,1} = u_{inst,Q,1}\,(1 + \Psi_{2,1}.\,k_{def})$ (final defl. resulting from lead variable action)

$u_{fin,Q,i} = u_{inst,Q,i}\,(\Psi_{0,i} + \Psi_{2,i}.\,k_{def})$ (final defl. resulting from lead variableaction)

i>1 (i.e. secondary variable actions)

For SIP roof panels to Eurocode designs (for sites < 1000m above sea level), this simplifies to:

$u_{fin} = u_{inst,G}\,(1 + k_{def}) + u_{inst,Q,1} + u_{inst,Q,i}\,.\,\Psi_{0,i}$

SIP wall panels, on the other hand, will generally require a full deflection derivation as ψ_2 will not be equal to 0 for all wall applied loads.

The instantaneous deflection, u_{inst} is comprised of both bending and shear deflection components. The final deflection equations therefore account for both also being a multiplier of the instantaneous deflection.

For the load span tables in Annex A, B and C, the following deflection limits have been assumed:

5.3.1.1. Instantaneous deflection: L/300.

5.3.1.2. Final deflection: L/250.

6. <u>SIP Roof Panels</u>

6.1 Roof of SIP panel buildings can be from loose rafters, ceiling joists, prefabricated roof trusses or SIP roof panels. SIP roof panels are usually supported on a ridge purlin, possibly with intermediate purlins and on wall (or floor member) at eaves level.

6.2 Generally, supports should down stand below SIP roof panels, with the SIP panel supported on triangular wedges which are connected down to supporting structure. The connections from the SIP panel to the wedge and the wedge down to the supports must be able to fully resist the "sliding" force due to the "in-plane component of the gravity roof load (permanent and variable loads) along the slope of the roof, i.e. panel to wedge screws in shear. They must also resist the worst wind load-case which is causing an uplift force on the roof, perpendicular to the plane of the roof, i.e. panel to wedge screws in tension.

6.3 A SIP roof, without additional bracing, will generally provide adequate roof diaphragm action.

6.4 The design of a SIP roof requires the determination of loads perpendicular to the plane of the panel. The length of the span is the slope length.

6.5 The use of SIP panels in flat roofs is permitted; however the designer should be aware of the very significant risk of failure (serviceability limit state and possibly ultimate limit state in extreme cases) if the roof is subjected to water pounding due to inadequate falls. This is particularly high risk for SIP panels due to the additional long-term load from the water, which would not have been accounted for in the creep deflection calculation (as no variable load related creep is accounted for in roofs (see section 5.2.3.2)).

7. <u>Design of SIP roof panels (single span) without timber "spline" reinforcement.</u>

The following steps should be taken to verify the design of a SIP roof panel (the use of a computerized calculation, such as spreadsheet or similar is strongly recommended due to the complexity of the design calculations for these composite panels). The design is based on TR019 and the structural engineer should become familiar with this document before attempting to apply a design to SIPs.

7.1 Calculate the structural span on slope for the SIP panel. This is the span to be used in the design.

7.2. Resolve the dead (G_{DL}) and variable loads (Q_{IL}) in the load diagram shown below so that the vector components of these loads acting perpendicular to the roof are derived. The wind load, Q_{WL} shall already be acting perpendicular to the plane of the roof. See Figure 1 for forces to be resolved.

In figure 1 where: G_{DL} = dead (permanent) load

Q_{IL} = variable gravity loads (various combinations of roof imposed, snow, man point as laid out in section 5.2).

Q_{WL} = variable wind load.

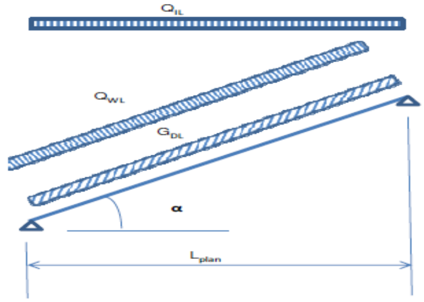


Figure 1

7.3. Calculate the un-factored maximum forces i.e. moment and shear for the various load durations, i.e. permanent (include long-term if applicable), short-term (worst case for man load / roof imposed / snow load) and instantaneous (wind load).

These calculated forces can then be factored using the partial load factors, γ_G and γ_Q in previously stated combinations in which the appropriate ψ_0 factors (as per Section5.2).

7.4 State the geometric and material properties of the 3 layers (due to differences in k_{def} between inner and outer OSB skins) similar to that shown on the page overleaf State the relevant material types, thicknesses, service classes and strength and stiffness properties (as well as densities and calculated relevant areas) for each o the 3 layers.

Figure 2

2. Geometric & Material Properties:

		Layer 1 (external)		Layer 2 (core)		Layer 1 (internal)	Units
Material type		OSB3		PIR90		OSB3	
Depth	d_1 (mm) =	15	d_2 (mm) =	122	d_3 (mm) =	15	(mm)
Effective width	b_1 (mm) =	100	b_2 (mm) =	1000	b_3 (mm) =	1000	(mm)
Service class	SC =	SC2	SC =	SC1	SC =	SC1	
MOE ($E_{t(c),//,o,mean}$) =		4930		6,8		4930	(N/mm^2)
Shear mod. (G_{mean}) =		1980		2,5		1980	(N/mm^2)
Char. Comp. strength ($f_{c,//,k}$) =		15,4		0,214		15,4	(N/mm^2)
Char. Tens. Strength ($f_{t,//,k}$) =		9,4		0,0		9,4	(N/mm^2)
Char. Panel shear strength ($f_{v,o,k}$) =		1		0,12		1	(N/mm^2)
Density		600		46		600	(kg/m^3)
Area		15000		122000		15000	(mm^2)

7.5 Determine the Steiner bending stiffness and shear stiffness of virtual beam B for the instantaneous response period:

From EOTA TR019 for instantaneous response:

(a) Bending stiffness of virtual beam B:

$$(EI)_B = \sum_{I=1}^{3} E_{t(c),//,omean,I} \cdot A_I \cdot Z_I^2 = 3.697E+11 \text{ Nmm}^2 \text{ (steiner bending stiffness)}$$

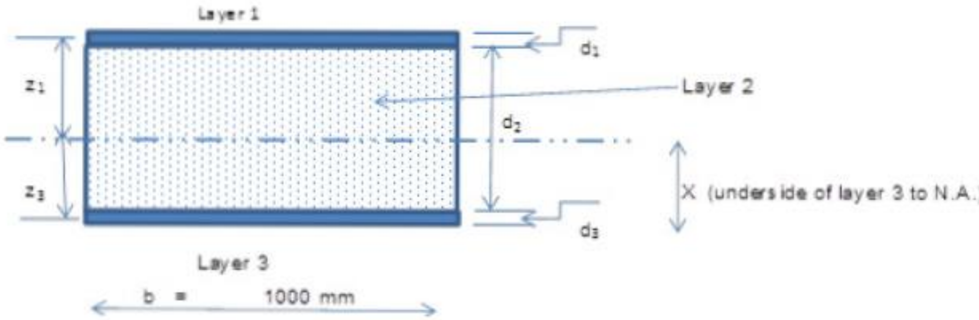
Note: Layer 2 ignored for bending stiffness

(b) Shear stiffness of virtual beam B:

$$1/(GA)_B = 1/S : (1/a^2) * = \frac{d1}{2 \,.\, G_{1,0,mean,1} \,.\, b_1} + \frac{d2}{G_{mean,2} \,.\, b_2} + \frac{d3}{2 \,.\, G_{1,0,mean,3} \,.\, b3}$$

This is based on calculation of the neutral axis position as shown overleaf (diagram is also appropriate for derivation of neutral axis based on final response stiffness's).

Note that the neutral axis will be central at the instantaneous response check where OSB skins are the same but will not be centered at final response check due to the outer skin stiffness being lower than that of the inner skin due to differing k_{def} values (as outer skin in SC2 and inner skin in SC1).



7.6 Ultimate Limit State design checks at the instantaneous response (these checks to be made for all relevant load duration cases).

7.6.1 Calculate the compression stress in the upper OSB layer and the tensile strength in the lower OSB layer resulting from the applied ULS moment on the SIP panel as described in C2.1.3 of TR019. Design to ensure that these are less than the allowable design compression stress (and tension stresses) of the top (and bottom) OSB layers respectively:

i.e. verify that $\sigma_{c,d,1} < f_{c,0,d}$ for compression and
$\sigma_{t,d,1} < f_{t,0,d}$ for tension.

$f_{c,0,d} = f_{c,0,k} \times k_{mod} / \gamma_M$ where:

and $f_{t,0,d} = f_{t,0,k} \times k_{mod} / \gamma_M$ k_{mod} is modification factor for duration of load and moisture content where k_{mod} for various timber members and wood-based panels are defined in Table 3.1 of EN 1995.

γ_M = material modification factor as defined Table 2.3 of EN 1995.

7.6.2 Carry out shear checks for each layer as well as the interface between the layers as per C2.1.4 and C2.1.5 of TR019. The designer should verify that the maximum applied design shear stress be less than the permissible design shear stress of the PIR core.

7.6.3 The local design compressive stress in the PIR core should be checked at each support in accordance with C2.1.6 of TR019.

Verify: $\sigma_{c,d,2} = R_d / A_{eff}$ where the notation is as described in C2.1.6.

7.7 Ultimate limit state design after internal force re-distribution (final response):

At final response stage, the stresses within the layers will be re-distributed due to the changes in relative bending and shear stiffness's of the 3 layers (primarily due to the fact that the outer OSB layer is in service class 2 environment and is subject to a higher k_{def} than the Service class 1 inner OSB layer). The different stiffness's of each layer is due to reduced E_{mean} and G_{mean} values for the layer based on the calculation for each layer:

$E_{mean,fin} = E_{mean,i} / (1 + \psi_2 . k_{def})$ and $G_{mean,fin} = G_{mean} / (1 + \Psi_2 . k_{def})$.

(for roof loads, $\Psi_2 = 1$ for permanent / long-term loads and 0 for short and instantaneous loads. For wall panel designs, derivation may be required for both

$E_{mean,fin,perm}$ and $E_{mean,fin,med}$ as well as $G_{mean,fin,perm}$ and $G_{mean,fin,med}$ to check at medium term response also)

Verify that the design compression, tension and shear stresses, based on the final response re-distribution of stresses are less than the design resistance stresses (similar to checks carried out for instantaneous response as described in 6.4.6.1 and 6.4.6.2 above).

7.8 Verification of serviceability limit states:

7.8.1 The deflection should first be calculated based on instant loading of the panel. This shall be based on the equations stated in Section 5.3 of this document and must include a calculation of shear deflection in addition to bending deflection.

From Annex A (A.2) of TR019, the instantaneous deflection shall be calculated based on the below equation for each of the instantaneous deflections from permanent, lead variable and secondary variable loads:

$$\text{uinst} = \frac{5\ \text{w}\ \text{L}^4}{384\ (EI)_{B,inst}} + \frac{\text{w}\ \text{L}^2}{8\ (GA)_{B,inst}}$$

See equation above and equations in Section 5.3 for calculation of $u_{inst,total}$ and calculation of $u_{fin,total}$.

7.9 For SIP roof panel designs it is critical that the permanent (and long-term if applicable) loads are correctly applied as these will have a significant effect on the critical final deflection calculation.

It is also critical that shear deflections are accounted for and also that the effect of creep is fully calculated. The shear deflection is often much larger than the bending deflection while the creep deflection component is often larger in magnitude to the initial deflection, i.e. the final deflection is > 2 x initial deflection.

7.10 Roof load span tables in Annex A are based on the calculation methodology described in this section (see also load-span table notes in Annex A for guidance on using tables).

7.11 The principles of the design methodology set out above, based on TR019 and EN1995-1-1 could be used to produce a design. This would be based on the working stress principle of timber design.

Alternatively, design tables based can be used to determine allowable spans for SIP roof panels. These are based on short-term loaded tests and it is critical that due account is taken of creep deflection and also reduced permissible stress resistances for permanent / long-term portion of load.

Using the spans as stated in these test-based tables, without accounting for these items, may yield unsafe designs!

7.12 SIP splines are used to connect adjacent SIP panels together. These should not be considered structurally in the design of the SIP roof panel.

7.13 The designers first preference, where possible, should always be to place purlins, etc at centers to allow for unreinforced splines as this avoids repeat thermal bridges. Due to the constraints of some projects this is not always possible and in these cases a timber spline reinforced SIPs panel is required as described below in section 8.

8. Design of SIP roof panels (single span) with timber "spline" reinforcement.

8.1 Timber splines are used to stiffen / strengthen SIP roof and wall panels. They are placed within the 45mm wide rebate at each end of the 1.25m wide SIP panels.

Therefore there are 2 no. 45mm timber splines at 1.25m crs. The depth of the timber splines is: depth of SIP panel – 15mm upper OSB – 15mm lower OSB – 2mm, i.e. a 150mm deep SIP panel will have a 120mm deep timber spline, etc, etc.

The OSB connects to the timber splines with 3.5 x 50mm screws @ 75mm crs. (into each 45mm wide timber member). There are 2 splines within a 1.25m panel so it is appropriate to design for a 1.25m panel with 90mm wide timber splines screwed @ 37.5mm crs. (2 no. screws @ 75mm crs.).

SIP roof panel with reinforced timber splines achieve longer spans, as demonstrated within load-span table in Annex A. On rare occasions the roof spans may dictate an addition full depth or higher (of SIPs panel) timber spline between the "rebated" timber splines.

8.2 The design of a SIP roof panel with timber splines can be undertaken as follows:

8.2.1 The loads are calculated in the same manner as for the unreinforced SIP roof panel (Section 6.4.2) except that design should allow for a 1250mm wide panel and hence 1250mm wide loading.

8.2.2 Additionally, the moments and shear forces are calculated similarly as described in Section 6.4.3.

8.2.3 As the stiffness of the timber is bigger than the stiffness of the PIR core, the core can be ignored for purposes of calculation without significantly effecting the design outcomes. This allows for the design of the reinforced SIP panel as a "mechanically jointed beam" with the upper and lower OSB, the timber spline and the interconnection between the 3 layers as being critical. The design can be based on the method given in *Annex B (Informative): Mechanically jointed beams* in *EN1995-1-1.*

As this Annex lays out clearly the design methodology, a competent design should be readily able to follow the requirements of the ultimate limit state checks for this design. Note that, as in the unreinforced SIP panel design, the effective bending stiffness shall vary between instantaneous response and final response, with a consequent re-distribution of stresses. Ultimate limit state design checks need to be verified for both the instantaneous and final response conditions.

8.2.4 The serviceability limit state design is not explicitly covered and hence some guidance is given here:

The effective bending stiffness as given in equation B.1 of this EN 1995 Annex will form the basis of the calculation of the deflections in a similar manner to that described in Section 5.3 and 6.4.7.

In this case, as the shear stress is resisted by the timber “web” members, the shear deflections will be significantly less but nonetheless we recommend that they are calculated. The effect of creep should also be accounted for by calculation of u_{fin} as previously described.

8.3 Load span tables, for SIPs by GreenPanel, in Annex A are based on the calculation methodology described in this section.

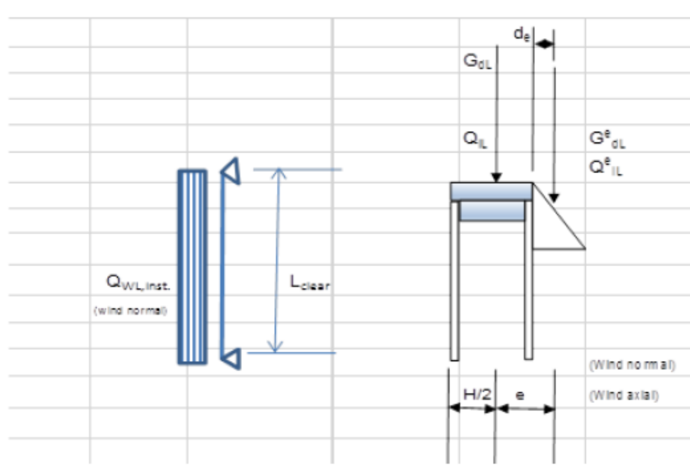
8.4 The principles of the design methodology set out above, based on Annex B of EN1995-1-1 could be used to produce a design calculation if considered appropriate by the designer. This would be based on the working stress principle of timber design to the local national standards.

9. Design of SIP wall panels (single span) without timber “spline” reinforcement.

9.1 The design methodology for unreinforced SIP wall panel, i.e. SIP splines only connecting adjacent panels, is similar to that for roof panels as outlined in section 7 although there are a number of existing checks to deal with the axial forces.

Although the design methodology in TR019 is for roof stressed skin panels, the principles can also be used for wall panels with, as stated previously, a number of additional checks.

9.2 The loads on a SIP wall panel should be set up as shown in the diagram below where G_{eDL} and Q_{eDL} are the eccentrically applied loads from the floor / roof immediately on the wall under consideration. The worst case is where the joists are hung from the side of the wall with proprietary joist hangers as this results in the largest axial load induced moment.



9.3 Clearly, in the case of wall panels the axial force is significant and this should be included in all applicable calculations.

9.4 Determine the Steiner bending stiffness and shear stiffness of the SIP wall panel for all applicable load durations. In particular permanent (to include long-term), medium term (as floor imposed loading on wall panels) and instantaneous stiffness's to allow for distribution of the stresses and these can also be used to determine deflections u_{inst} and u_{fin} as applicable.

9.5 The maximum compression forces (or tension if applicable for instantaneous loading) in the OSB skins shall be calculated based on the formulae in C.2.1.3 of TR019 and discussed in Section 7.6.1 of this document for roof panels.
Note that the axial forces are resisted by the OSB skins only although the load bearing insulation provides lateral restraint to the skins to prevent their buckling.

9.6 Shear checks to be carried out as stated in Section 7.6.2.

9.7 Re-design for final and medium term response re-distributed stresses as per Section7.7 of this document (based on determination of $(EI)_{B,fin}$ and $(GA)_{B,fin}$ and $((EI)_{B,med}$ and $(GA)_{B,med})$.

9.8 Buckling of Wall under Compression
This issue is specific to wall panels and the methodology used is based on the section "Wall Panel Buckling" from Lightweight Sandwich Construction by JM Davies in which the following formula is presented to determine P_{cr} (the critical axial force re buckling):

$$Pcr = \frac{P_E . P_{EF} - P^2_{EF} + P_E . P_C}{P_E - P_{EF} + P_C}$$

where $PE = \pi^2.(EI)_{B,perm} / L^2$

$P_{EF} = \pi^2.(EI)_{OSB\ flangers} / L^2$

$P_c = A_c.G_c$

A_c = area of core

G_c = shear modulus of core

These shall be calculated for each of permanent, medium-term and instantaneous (including short-term) response periods.

The Eurocode 5 instability reduction factor, $k_{c,y}$ is then calculated for the wall panel, for each duration period stated above, by calculation based on Section 6.3.2 of EN1995-1-1 with _$_{rel,y}$ being calculated with the more general formula

$\lambda_{rel,y} = (P_{c,0,k} / P_{CR})^{½}$ where $P_{c,0,k}$ is the allowable axial force in the OSB skins.

This replaces formula 6.21 from EN1995-1-1.

9.9 Combined Axial and Bending
The following equations shall be verified to confirm adequacy of wall panels with regard to combined axial compression and bending;

9.9.1 $$\frac{\sigma_{c,axial,perm,\ perm}}{k_{c,y,perm} \cdot f_{c,0,d,perm}} + \frac{\sigma_{c,axial,perm,,med}}{k_{c,y,med} \cdot f_{c,0,d,perm}} + \frac{\sigma_{c,axial,\ perm,\ short\ /\ inst}}{k_{c,y,short\ /\ inst} \cdot f_{c,0,d,perm}} + \frac{\Sigma\sigma_{c,bending,0,d}}{f_{c,0,d,perm}} < 1.$$

9.9.2 $$\frac{\sigma_{c,axial,med,\ perm}}{k_{c,y,perm} \cdot f_{c,0,d,med}} + \frac{\sigma_{c,axial,med,,,med}}{k_{c,y,med} \cdot f_{c,0,d,med}} + \frac{\sigma_{c,axial,\ med,\ short\ /\ inst}}{k_{c,y,short\ /\ inst} \cdot f_{c,0,d,med}} + \frac{\Sigma\sigma_{c,bending,0,d}}{f_{c,0,d,med}} < 1.$$

9.9.3
$$\frac{\sigma_{c,axial,inst,perm}}{k_{c,y,perm} \cdot f_{c,0,d,inst}} + \frac{\sigma_{c,axial,inst,,med}}{k_{c,y,med} \cdot f_{c,0,d,inst}} + \frac{\sigma_{c,axial,inst,short/inst}}{k_{c,y,short/inst} \cdot f_{c,0,d,inst}} + \frac{\Sigma\sigma_{c,bending,0,d}}{f_{c,0,d,med}} < 1.$$

where the first “perm” or “med” or “inst” term refers to the response period for which the stresses are distributed and the second “perm” or “med” or “inst” refers to the stress from the load (based on its duration type).

9.10. Deflections

Lateral deflections, based on wind loads normal to the SIP wall panel are calculated as per Section 7.8 for roof panels. The only deflection associated with this load is u_{inst} as $k_{def} = 0$ (as this is an instantaneous load).
Deflections based on axial forces at the top are derived from:

$$\text{Uaxial} = \frac{ML^2}{16\,EI}$$

The deflection due to axial loads is generally small and can be neglected except where joists are side hung from walls in which case the eccentric moment is significant with consequent larger induced deflections.

9.11. A further critical check is bearing at the sole-plate and head-plate. This has been found to be the limiting factor for axial loads in unreinforced SIP wall panels. Therefore, based on standard timber design to EN1995-1-1, check the applied bearing stress against the permissible cross-grain bearing stress of the timber soleplate / head-binder.

he bearing forces may be reduced by the capacity of the OSB to top-rail or bottom rail screws if deemed appropriate by the designer.

9.12 Load span tables in Annex B are based on the calculation methodology described in this section (see also table notes in Annex B for guidance on using these tables).

The allowable axial loads, and perpendicular allowable wind loads, can be significantly increased by using timber splines (particularly as these significantly increase the bearing area onto the head-binder / sole-plate).

9.13 The principles of the design methodology set out above, based on TR019 and EN1995-1-1 could be used to produce a design calculation to 5268 if considered appropriate by the designer. This would be based on the working stress principle of timber design to the British Standards.

Alternatively, previously produced design tables based on loading can be used to determine allowable spans for SIP wall panels. These are based on short-term loaded tests and it is critical that due account is taken of creep deflection and also reduced permissible stress resistances for permanent / long-term portion of load. Using the spans as stated in these test-based tables, without accounting for these items, will not yield safe designs!

9.14 SIP splines are used to connect adjacent SIP wall panels together. These should not be considered as structural in the design of the SIP wall panels.

9.15 Point Loads on SIP wall panels

Significant point loads will need to be supported by multiple timber studs within the SIP wall panels in accordance with standard design practice using the preferred standard.

Smaller point loads can be supported by the SIP wall panels with the thickness of the head-binder dictating the magnitude of the point load as follows:

$T = 3 \times (F_d \times b_{joist})$ where T = required min. thickness of head binder.
2 28.3 F_d = design loads on the bearing area (N).
b_{joist} = the width of the joist or beam.

In cases of high loads / small head-binder, the joists may need to be hung from the rim beam.

10. Lintels:

10.1 Lintels may be in the form of solid timber or glulam or engineered timber or steel members. Another option, where permitted by design, is to use a “SIP lintel” as a box beam with timber top flange (in line of top-rail and same cross-section as top rail), bottom flange (rail directly above opening) and OSB webs on each side (outer skins of SIP panels). For very lightly loaded, small span lintels it may be acceptable to screw the OSB to the flanges only.

10.2 The design of “SIP Lintels” is based on design of a glued thin-webbed beam in accordance with Section 9.1.1 of EN 1995-1-1. It is critical that the webs are adequately glued to the timber flanges in order to realize the design strength of a SIP-Lintel. Use a structural adhesive of Type 1 or 2 in accordance with EN 301 (note that the glue used to connect OSB to PIR insulation may not be appropriate for OSB to timber structural adhesion).

The insulation (PIR) is ignored for purposes of calculation of a SIP lintel.

10.3 A comprehensive Table and notes for SIP Lintel is shown in Annex C of this document. For designers who want to do a “first principles" design to EN 1995-1-1, the publication “Structural Design to Eurocode 5” by *Jack Porteous and Abdy Kermani* contains a detailed example.

11. Racking Resistance:

11.1 Applied racking forces are calculated by the designer based on wind loads from EN 1991-1-4.

11.2 To date, racking resistance values are based on test results in accordance with EN 594. A “safe” result is then achieved by applying factors based on section 5.9 of 5268-6.1:1996. The basic racking resistance value to 5268-6.1:1996 is 2.5KN/m.

The racking resistance is reduced for wall sections containing openings based on equation:

$R_b = 2.5.\exp(-0.0365x)$

Where

R_b = test racking resistance per meter run of the wall in KN/m

X = percentage opening in a panel of 2.4m in length.

Where the percentage of openings exceeds 35%, $R_b = 0$.

The racking resistance is subject to 5268 Part 6.l factors K104 (panel height factor); K105 (wall length factor) and K108 (interaction factor)

K107 should be modified as follows: $0.004F_2 + 0.025F + 1$ for values up to a maximum of 10.4KN.

11.3 Racking resistance values for EN1995-1-1 designs can be derived as follows:

11.3.1 Unfactored applied wind loads to be derived to provide an applied racking load factor to the level under consideration.

11.3.2 The racking resistance of 2.5KN/m is a safe racking resistance value (as based on racking stiffness which is a serviceability limit state load. Hence use of unfactored wind loads). This can be multiplied by similar equation to K104 for wall heights above 2.4m. No other factors should be applied. Similarly only full height wall panels between openings are permitted (new tests to EN 594:2011 may allow higher values to be used for EN 1995-1-1 designs).

N.B. Although unfactored wind loads are used for verification of racking resistance, it is critical that factored wind loads are used for the following checks to EN 1995-1-1 design:

(a) Overall stability of the SIP frame structure (at each level).
(b) Sliding of the structure at each level.
(c) Overturning and sliding of racking panels.
(d) Uplift of structure at each level.

When checking these to 5268:6.1, check for factor of safety requirements within this standard (and 5268-3 for roof uplift factor of safety of 1.4).

ANNEX A

Span Tables

(with and without timber splines)

Annex A Contents:

TABLE A.1.1 : 122mm SIP Roof Panels - 0.6KN/m^2 wind load

			SIP Splines		1 no. 90 x90 C16 timber splines		1 no. 90 x90 C24 timber splines		1no. 90 x90 C30 timber splines	
Pitch (degrees)	Dead Load DL (KN/m^2)	Imposed load DL (KN/m^2)	Clear span on plan L_{plan} (m)	Clear span on slope L_{slope} (m)	Clear span on plan L_{plan} (m)	Clear span on slope L_{slope} (m)	Clear span on plan L_{plan} (m)	Clear span on slope L_{slope} (m)	Clear span on plan L_{plan} (m)	Clear span on slope L_{slope} (m)
0										
15										
30										
45										

TABLE A.1.2 : 120mm SIP Roof Panels - 0.9KN/m² wind load

			SIP Splines		1 no. 90 x 90 C16 timber splines		1 no. 90 x90 C24 timber splines		1 no. 90 x90 C30 timber splines	
Pitch (degrees)	Dead Load DL (KN/m^2)	Imposed load DL (KN/m^2)	Clear span on plan L_{plan} (m)	Clear span on slope L_{slope} (m)	Clear span on plan L_{plan} (m)	Clear span on slope L_{slope} (m)	Clear span on plan L_{plan} (m)	Clear span on slope L_{slope} (m)	Clear span on plan L_{plan} (m)	Clear span on slope L_{slope} (m)
0										
15										
30										
45										

GREENPANEL INNOVATIVE HOMES