TimberTech Buildings

Verification Manual

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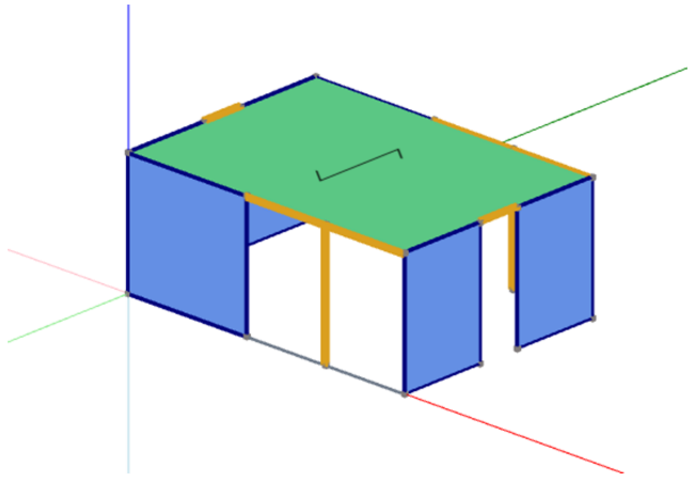
INTRODUCTION

This verification manual provides solved and commented test cases with reference to the input file necessary to reproduce the elaboration.

EXAMPLE 1: CALCULATION OF THE FLOOR SELF WEIGHT G_1



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DESCRIPTION OF THE PROBLEM

The self weight G_1 per unit area of the joists floor is the weight of the joists included in a unit area. The comparison between the value provided by the software and that obtained with a hand calculating is conducted assuming:

- Joist cross section: $160 \times 200\ \mathrm{mm}$
- Volumetric weight γ: $6\ \mathrm{kN/m^3}$
- Spacing i: $500\ \mathrm{mm}$

INDEPENDENT VERIFICATION

Calculation of the joist cross section area:

$$B \cdot h = 160 \times 200 = 32000\ \mathrm{mm^2} = 32 \times 10^{-3}\mathrm{m^2}$$

Calculation of the linear weight per unit length for the joist:

$$A \cdot \gamma = (32 \times 10^{-3}) \times 6 = 0{,}192\ \mathrm{KN/m}$$

Calculation of the weight per unit area:

$$G_1 = \frac{A \cdot \gamma}{i} = \frac{0{,}192}{0{,}5} = 0{,}38\ \mathrm{KN/m^2}$$

RESULTS PROVIDED BY THE SOFTWARE

- Self-weight of the floor G_1: $0{,}38\ \mathrm{kN/m^2}$

RESULTS COMPARISON

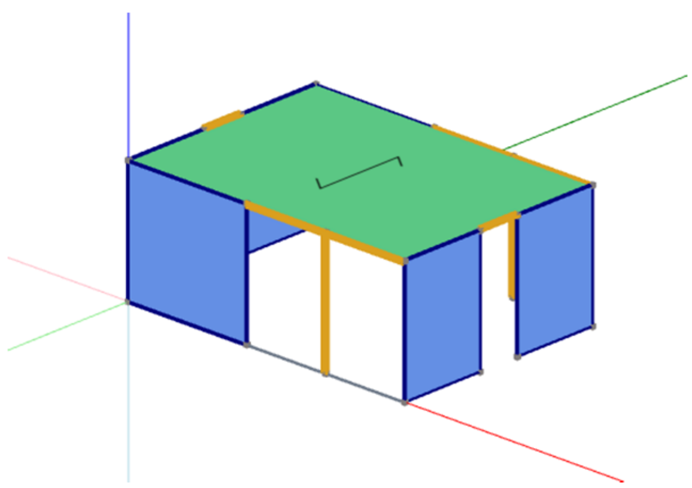
Comparison between the value provided by the software and the hand-calculating value:

Validated parameter	Independent verification	Software	Percentage error
$G_{1,floor}$	$0{,}38\ \mathrm{KN/m^2}$	$0{,}38\ \mathrm{KN/m^2}$	0%

CONCLUSIONS

The comparison shows the coincidence between the value provided by the software and that hand-calculated.

E XAMPLE 2: CALCULATION OF THE BEAM SELF WEIGHT G1



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DESCRIPTION OF THE PROBLEM

The comparison between the value of the beam self weight G_1 provided by the software and that obtained independently is conducted assuming:

- Cross section of the beam: $160 \times 240\ \mathrm{mm}$
- Volumetric weight γ: $6\ \mathrm{kN/m^3}$

INDEPENDENT VERIFICATION

Calculation of the beam cross section area:

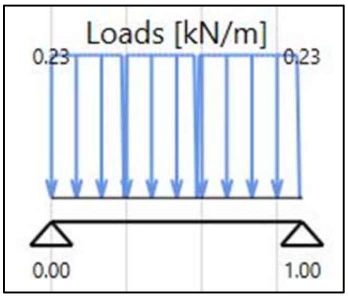
$$B \cdot h = 160 \times 240 = 38400\ \mathrm{mm^2} = 38{,}4 \times 10^{-3}\mathrm{m^2}$$

Calculation of the beam linear weight:

$$A \cdot \gamma = (38{,}4 \times 10^{-3}) \times 6 = 0{,}23\ \mathrm{KN/m}$$

RESULTS PROVIDED BY THE SOFTWARE

- Self-weight of the beam G_1: $0{,}23\ \mathrm{kN/m}$



RESULTS COMPARISON

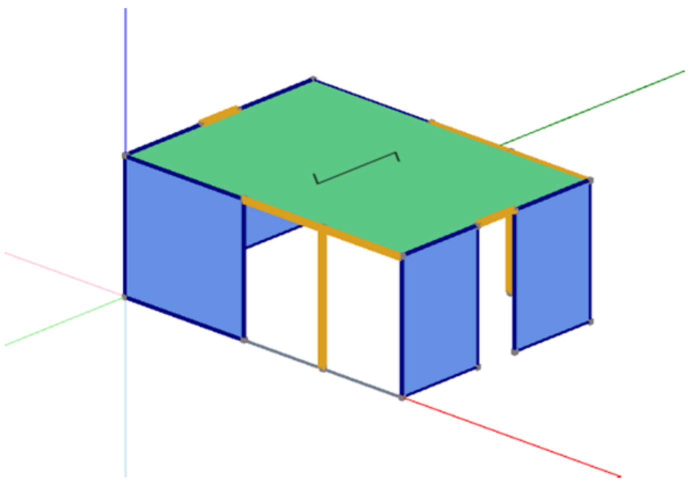
Comparison between the value provided by the software and the hand-calculating value:

Validated parameter	Independent verification	Software	Percentage error
$G_{1,beam}$	$0{,}23\ \mathrm{KN/m}$	$0{,}23\ \mathrm{KN/m}$	0%

CONCLUSIONS

The comparison shows the coincidence between the value provided by the software and that hand-calculated.

EXAMPLE 3: CALCULATION OF THE SNOW LOAD



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EsempioValidazione.bldg

DESCRIPTION OF THE PROBLEM

The calculation of the snow load on the roof is been made with reference to the dispositions provided by the Italian Standard D.M.14-01-2008, assuming:

- Building position: Trento
- Altitude m.a.s.l.: 193 m
- Snow load zone: Zona I – Alpina
- Topographic class: Normale
- Exposure factor: 1
- Thermal coefficient: 1

INDEPENDENT VERIFICATION

The load caused by snow on roof q_s is assessed by the equation:

$$q_s = \mu_i \cdot q_{sk} \cdot C_E \cdot C_t$$

where:

- μ_i is the shape coefficient of the roof equal to 0,8 in the case of flat roof
- q_{sk} is the characteristic value of the reference snow load on the ground

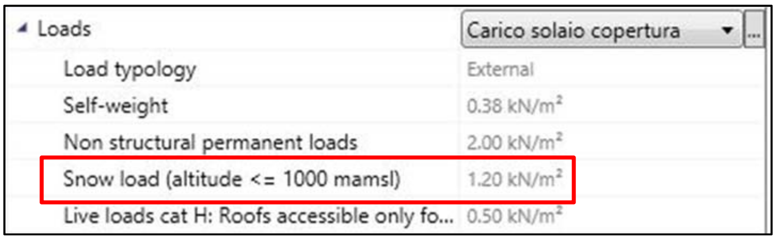
For locations belonging to Zone I – Alpina and for heigth above sea level less than $200\ \mathrm{m}$, q_{sk} takes the value of 1,5.

Load q_s assumes the value:

$$q_s = \mu_i \cdot q_{sk} \cdot C_E \cdot C_t = 0{,}8 \times 1{,}5 \times 1 \times 1 = 1{,}2\ \mathrm{kN/m^2}$$

RESULTS PROVIDED BY THE SOFTWARE

- $q_s = 1{,}2\ \mathrm{kN/m^2}$



RESULTS COMPARISON

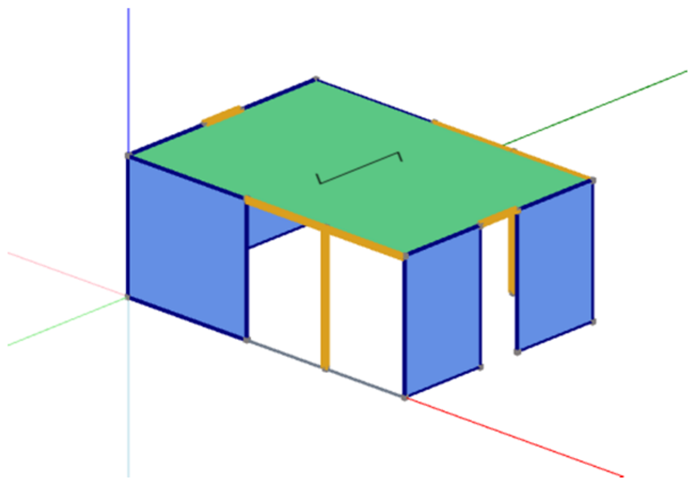
Comparison between the value provided by the software and the value calculated in an independent manner:

Validated parameter	**Independent verification**	**Software**	**Percentage error**
q_s	$1{,}2\ \mathrm{kN}$	$1{,}2\ \mathrm{kN}$	0%

CONCLUSIONS

The comparison shows the coincidence between the value provided by the software and that hand-calculated.

EXAMPLE 4: CALCULATION OF THE WIND LOAD



File name:
EsempioValidazione.bldg

DESCRIPTION OF THE PROBLEM

The calculation of the snow load on the roof is been made with reference to the dispositions provided by the Italian standard D.M.14-01-2008, assuming:

- Province: Trento
- Altitude m.a.s.l.: 193 m
- Wind load zone: Zone 1
- Roughness class: Class A
- Distance from the coast: Hinterland
- Exposure category: V
- Airtight construction

INDEPENDENT VERIFICATION

The wind is considered to act in the horizontal direction and exerts actions on structures that vary in time and space, usually causing dynamic effects. For the usual constructions, actions are conventionally traced to the equivalent static actions. The wind pressure is given by the expression:

$$p = q_b \cdot c_e \cdot c_p \cdot c_d$$

where:

- q_b is the kinetics reference pressure
- c_e is the exposure factor
- c_p is the shape factor
- c_d is the dynamic factor

KINETICS PRESSURE

The kinetics reference pressure q_b (in N/m²) is given by:

$$q_b = \frac{1}{2} \cdot \rho \cdot v_b^2$$

where:

- ρ is the density of the air assumed equal to 1,25 kg/m³
- v_b is the reference velocity (in m/s)

In the absence of specific and appropriate statistics investigations v_b is given by:

$$v_b = v_{b,0} \qquad \text{for } a_s \leq a_0$$

$$v_b = v_{b,0} + k_a \cdot (a_s - a_0) \qquad \text{for } a_0 < a_s \leq 1500 \text{ m}$$

in which $v_{b,0}$, a_0, k_a are parameters provided and depending on the building's region:

- a_s is the altitude a.s.l. (in m)
- $v_{b,0}$ 25 m/s
- a_0 1000 m
- k_a $0{,}010 \text{ 1/s}$

Since $a_s \leq a_0$, the reference velocity is equal to $25{,}00 \text{ m/s}$. The kinetics reference pressure q_b assumes the value:

$$q_b = \frac{1}{2} \cdot \rho \cdot v_b^2 = \frac{1}{2} \times 1{,}25 \times 25^2 = 390{,}63 \text{ N/m}^2$$

EXPOSURE FACTOR

The exposure factor depends on the heigth z on the ground of the considered point, on the topography of the land, and on the exposure category of the construction's site. In absence of specific analyzes that take into account the wind direction, the effective roughness and the topography of the land surrounding the building for heigths on ground not greater than $z = 200 \text{ m}$, the exposure factor is given by the equation:

$$c_e(z) = k_r^2 \cdot c_t \cdot \ln\left(\frac{z}{z_0}\right) \cdot \left[7 + c_t \cdot \ln\left(\frac{z}{z_0}\right)\right] \qquad \text{for } z \geq z_{min}$$

$$c_e(z) = c_e(z_{\min}) \qquad \text{per } z<z_{min}$$

in which

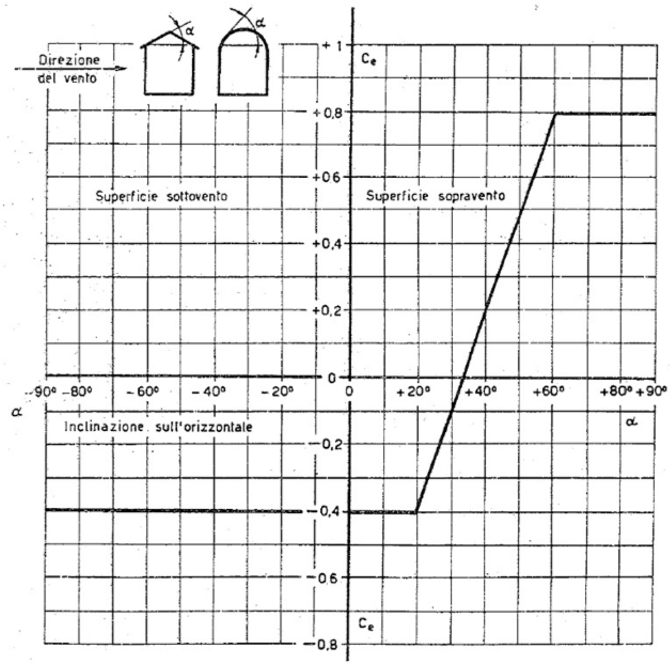
- c_t is the topography factor.

The heigth z_{min} on ground for the category exposure V is $12\ \mathrm{m}$, and, being the heigth of the building equal to $2{,}8\ \mathrm{m}$, the exposure factor is:

$$c_e(z) = c_e(12m) = 0{,}23^2 \cdot 1 \cdot \ln\left(\frac{12{,}0}{0{,}7}\right) \cdot \left[7 + 1 \cdot \ln\left(\frac{12{,}0}{0{,}7}\right)\right] = 1{,}479$$

SHAPE FACTOR

The shape factor (or aerodynamic coefficient) is a function of the typology and the geometry of the building and depends on its orientation respect to the wind direction. The value of the external shape factor can be assessed by the diagram shown in the following figure:



The internal exposure factor c_{pi} is zero value if the construction is airtight while it takes the values ± 0.2 if it is open. Based on the assumptions made, the exposure factor assumes zero value.

Element	α	c_{pe}
Windward wall	90	0,8
Leeward wall	90	−0,4
Leeward pitched roof	-	−0,4
Windward pitched roof 0°	0	−0,4

WIND PRESSURE

The wind pressure, calculated with reference to the two values of c_{pe} reported in the table above, takes the following values:

$$p^+ = q_b \cdot c_e \cdot c_p \cdot c_d = 390{,}63 \times 1{,}479 \times 0{,}8 \times 1 = 0{,}46 \text{ kN/m}^2$$

$$p^- = q_b \cdot c_e \cdot c_p \cdot c_d = 390{,}63 \times 1{,}479 \times (-0{,}4 \times 1) = -0{,}23 \text{ kN/m}^2$$

RESULTS PROVIDED BY THE SOFTWARE

- $p^+ = 0{,}46 \text{ kN/m}^2$
- $p^- = -0{,}23 \text{ kN/m}^2$

RESULTS COMPARISON

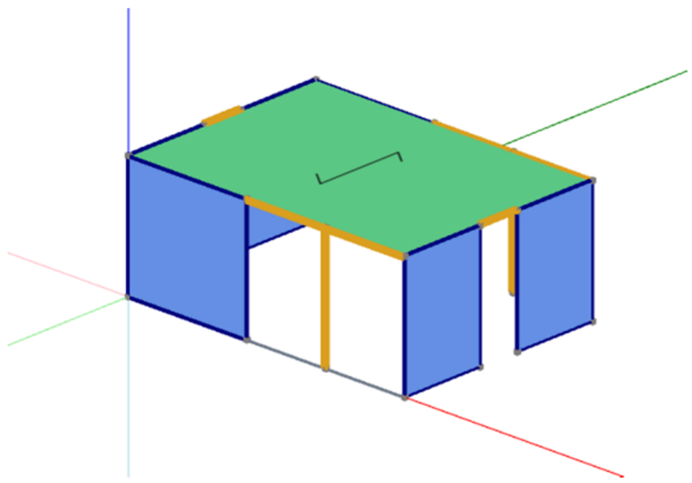
Comparison between the value provided by the software and the value calculated in an independent manner:

Validated parameter	Independent verification	Software	Percentage error
p^+	$0{,}46 \text{ kN/m}^2$	$0{,}46 \text{ kN/m}^2$	0%
p^-	$-0{,}23 \text{ kN/m}^2$	$-0{,}23 \text{ kN/m}^2$	0%

CONCLUSIONS

The comparison shows the coincidence between the value provided by the software and that hand-calculated.

EXAMPLE 5: BEAM'S INERTIA PROPERTIES



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DESCRIPTION OF THE PROBLEM

Inertia properties of the beam cross-section are the following:

- Cross section area A;
- Moment of inertia refers to axis y-y J_{y-y}
- Moment of inertia refers to axis z-z J_{z-z}

The comparison between the values provided by the software and those obtained independently is conducted assuming:

- Cross section of the beam: $160 \times 240 \text{ mm}$

INDEPENDENT VERIFICATION

Calculation of the beam cross section area:

$$A = B \cdot h = 160 \times 240 = 38400 \text{ mm}^2 = 38{,}4 \times 10^{-3}\text{m}^2$$

Calculation of the inertia moment J_{y-y}:

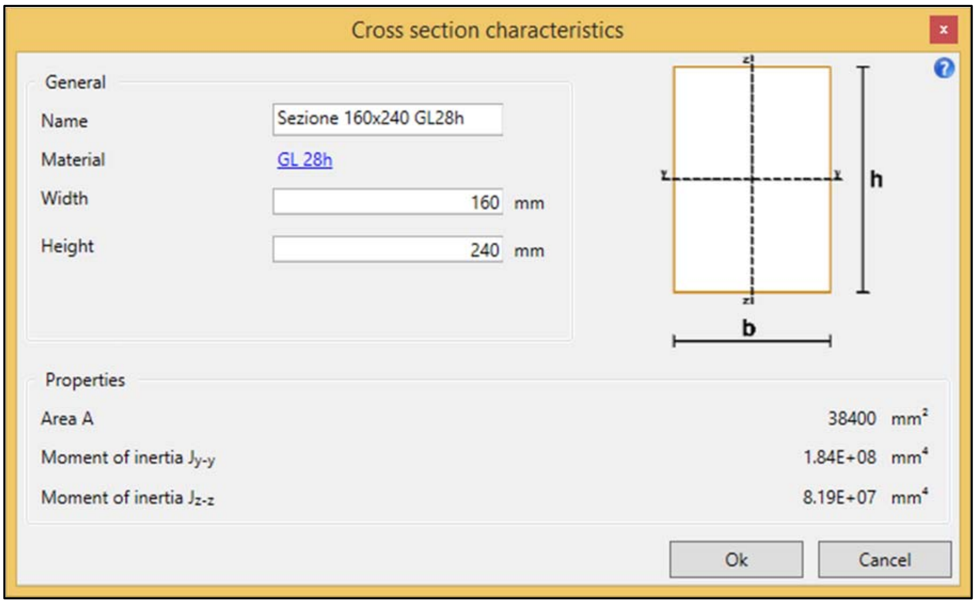
$$J_{y-y} = \frac{B \cdot h^3}{12} = \frac{160 \times 240^3}{12} = 1{,}84 \times 10^8 \text{ mm}^4$$

Calculation of the inertia moment J_{z-z}:

$$J_{z-z} = \frac{B^3 \cdot h}{12} = \frac{160^3 \times 240}{12} = 8{,}19 \times 10^7 \ \text{mm}^4$$

RESULTS PROVIDED BY THE SOFTWARE

- Area *A*: $38400 \ \text{mm}^2$
- Moment of inertia $J_{y\text{-}y}$: $1{,}84 \times 10^8 \ \text{mm}^4$
- Moment of inertia $J_{z\text{-}z}$: $8{,}19 \times 10^7 \ \text{mm}^4$



RESULTS COMPARISON

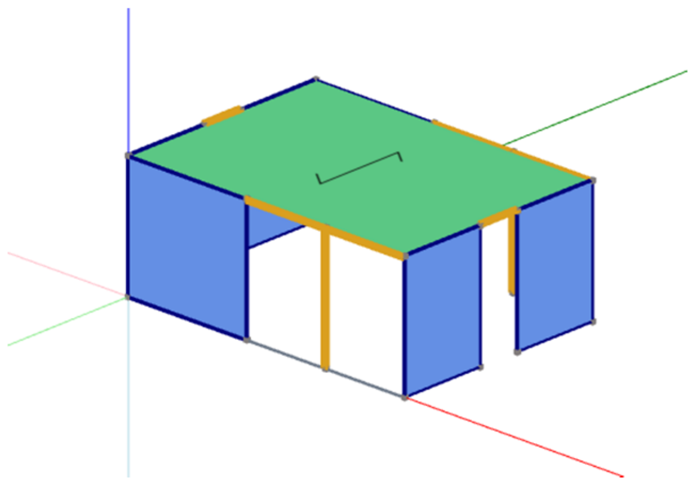
Comparison between the value provided by the software and the value calculated in an independent manner:

Validated parameter	Independent verification	Software	Percentage error
A	$38{,}4 \times 10^3 \text{mm}^2$	$38{,}4 \times 10^3 \text{mm}^2$	0%
$J_{y\text{-}y}$	$1{,}84 \times 10^8 \ \text{mm}^4$	$1{,}84 \times 10^8 \ \text{mm}^4$	0%
$J_{z\text{-}z}$	$8{,}19 \times 10^7 \ \text{mm}^4$	$8{,}19 \times 10^7 \ \text{mm}^4$	0%

CONCLUSIONS

The comparison shows the coincidence between the value provided by the software and that hand-calculated.

EXAMPLE 6: PILLAR'S INERTIA PROPERTIES



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DESCRIPTION OF THE PROBLEM

Inertia properties of the pillar cross-section are the following:

- Cross section area A;
- Moment of inertia refers to axis y-y J_{y-y}
- Moment of inertia refers to axis z-z J_{z-z}

The comparison between the values provided by the software and those obtained independently is conducted assuming:

- Cross section of the beam: $200 \times 200 \text{ mm}$

INDEPENDENT VERIFICATION

Calculation of the pillar cross section area:

$$A = l^2 = 200^2 = 40000\ mm^2 = 40 \times 10^{-3}\text{m}^2$$

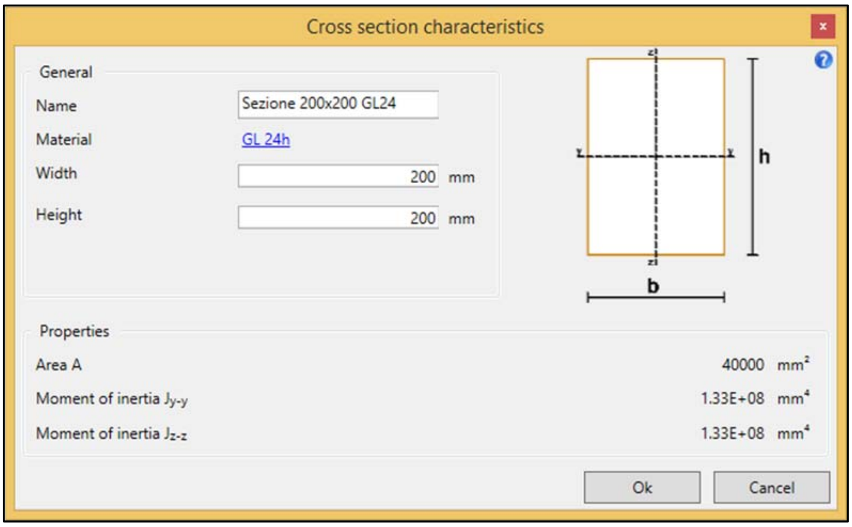
Calculation of the inertia moment $J_{y\text{-}y}$:

$$J_{y-y} = J_{z-z} = \frac{l^4}{12} = \frac{200^4}{12} = 1{,}33 \times 10^8 \text{ mm}^4$$

RESULTS PROVIDED BY THE SOFTWARE

- Area A: 40000 mm^2

- Moment of inertia J_{y-y}: $1{,}33 \times 10^8\ \mathrm{mm}^4$
- Moment of inertia J_{z-z}: $1{,}33 \times 10^8\ \mathrm{mm}^4$



RESULTS COMPARISON

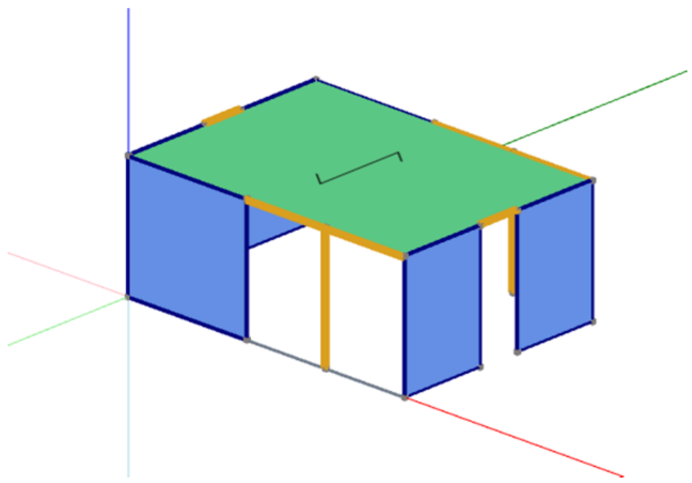
Comparison between the value provided by the software and the value calculated in an independent manner:

Validated parameter	Independent verification	Software	Percentage error
A	$40 \times 10^3 \mathrm{mm}^2$	$40 \times 10^3 \mathrm{mm}^2$	0%
$J_{y\text{-}y}$	$1{,}33 \times 10^8\ \mathrm{mm}^4$	$1{,}33 \times 10^8\ \mathrm{mm}^4$	0%
$J_{z\text{-}z}$	$1{,}33 \times 10^8\ \mathrm{mm}^4$	$1{,}33 \times 10^8\ \mathrm{mm}^4$	0%

CONCLUSIONS

The comparison shows the coincidence between the value provided by the software and that hand-calculated.

EXAMPLE 7: FLOOR LOAD



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DESCRIPTION OF THE PROBLEM

The calculation of the distributed load on the generic joist floor q_{SLU} is conducted with reference to the following ultimate limit load combination:

Load	G_1	G_2	Live load cat. H	Snow	Wind
Partial factors	1,3	1,5	1,5	0	0

Assuming:

- Floor self weight G_1: $0{,}38\ \mathrm{kN/m^2}$
- Permanent load G_2: $2{,}00\ \mathrm{kN/m^2}$
- Live load cat. H Q_H: $0{,}50\ \mathrm{kN/m^2}$
- Spacing between joists i: $500\ \mathrm{mm}$

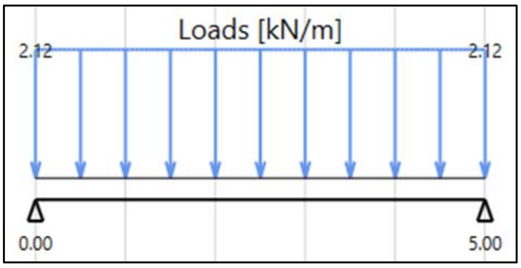
INDEPENDENT VERIFICATION

The distribuited load q_{SLU} per unit length results:

$$q_{SLU} = (1{,}3 \cdot G_1 + 1{,}5 \cdot G_2 + 1{,}5 \cdot Q_H) \cdot i = (1{,}3 \times 0{,}38 + 1{,}5 \times 2{,}00 + 1{,}5 \times 0{,}5) \times 0{,}5 = 2{,}12\ \mathrm{kN/m}$$

RESULTS PROVIDED BY THE SOFTWARE

- q_{SLU}: $2{,}12\ \mathrm{kN/m}$



RESULTS COMPARISON

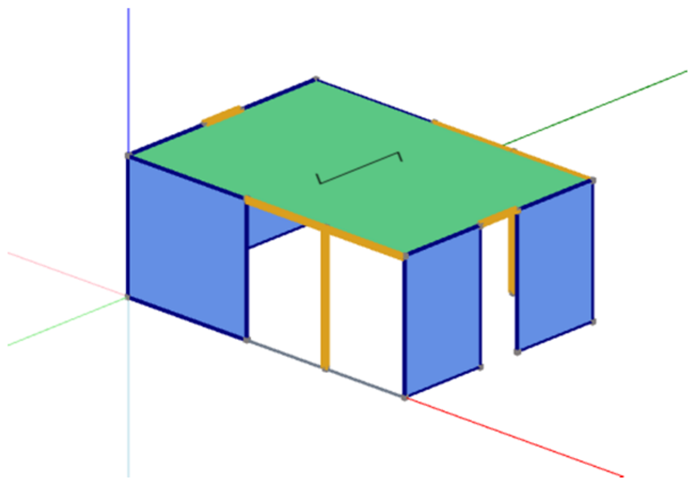
Comparison between the value provided by the software and the value calculated in an independent manner:

Validated parameter	Independent verification	Software	Percentage error
q_{SLU}	$2{,}12\ \mathrm{kN/m}$	$2{,}12\ \mathrm{kN/m}$	0%

CONCLUSIONS

The comparison shows the coincidence between the value provided by the software and that hand-calculated.

EXAMPLE 8: CALCULATION OF FLOOR FORCES



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DESCRIPTION OF THE PROBLEM

The calculation of the forces on the generic joist floor is conducted with reference to a span l equal to $5\ m$, assuming:

- $q_{SLU} = 2{,}12\ \mathrm{kN/m}$

VERIFICATION

Assuming that the joist can be considered simply supported at the ends, the reactions at the ends of the joist are:

$$R_A = R_B = \frac{q_{SLU} \cdot l}{2} = \frac{2{,}12 \cdot 5}{2} = 5{,}3\ \mathrm{kN}$$

The shear force at the ends of the joist is equal to restraint reactions:

$$T_A = 5{,}3\ \mathrm{kN}$$

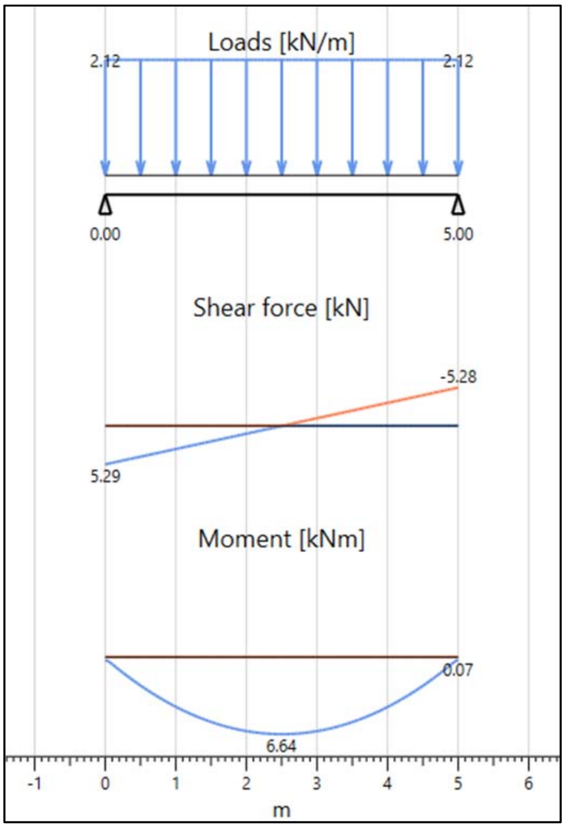
$$T_B = -5{,}3\ \mathrm{kN}$$

The bending moment at the midpoint of the joist is:

$$M_{max} = \frac{q_{SLU} \cdot l^2}{8} = \frac{2{,}12 \cdot 5^2}{8} = 6{,}62\ \mathrm{kNm}$$

RESULTS PROVIDED BY THE SOFTWARE

- Restraint reactions: 5,29 kN
- Shear force A: 5,29 kN
- Shear force B: -5,28 kN
- Maximum bending moment: 6,64 kNm



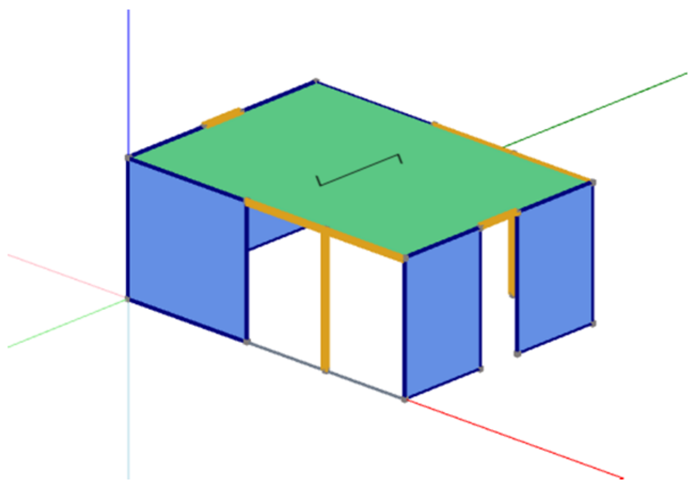
RESULTS COMPARISON

Validated parameter	Independent verification	Software	Percentage error
$R_A,\ R_B$	5,30 kN	5,29 kN	−0,2%
T_A	5,30 kN	5,29 kN	−0,2%
T_B	−5,30 kN	−5,28 kN	+0,4%
M_{max}	6,62 kNm	6,64 kNm	+0,3%

CONCLUSIONS

The comparison shows how results provided by the software are comparable to results obtained with an independent verification. Errors are caused by different approximation between the manual verification and the software.

EXAMPLE 9: FLOOR RESISTANCE CHECKS



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DESCRIPTION OF THE PROBLEM

The resistance checks (bending and shear) are conducted with reference to the dispositions provided by UNI EN 1995-1-1:2009 al § 6.3.3, assuming:

- Joist cross section $b \times h$: $160 \times 200\ \mathrm{mm}$
- Material: solid timber C24
- Bending moment: $M_d = 6{,}64\ \mathrm{kNm}$
- Shear force $T_d = 5{,}29\ \mathrm{kNm}$

INDEPENDENT VERIFICATION

The area *A* and the modulus of resistance *W* are equal to:

$$A = B \cdot h = 160 \times 200 = 32000\ \mathrm{mm}^2$$

$$W = \frac{B \cdot h^2}{6} = \frac{160 \times 200^2}{6} = 1{,}067 \times 10^6\ \mathrm{mm}^3$$

BENDING CHECK

The checks are conducted according to § 6.3.2 of EN 1995-1-1. The following expression shall be satisfied:

$$\frac{\sigma_{m,d}}{k_{crit} \cdot f_{m,d}} \leq 1$$

where:

- $\sigma_{m,d}$ is the design bending stress
- $f_{m,d}$ is the design bending strength
- k_{crit} is a factor which takes into account the reduced bending strength due to lateral buckling

k_{crit} is assumed equal to 1.0 for beams in which the lateral displacement of the compressed edge is prevented over the entire length and the torsional rotation is prevented at the supports.

The design bending stress assumes the value:

$$\sigma_{m,d} = \frac{M_d}{W} = \frac{6{,}64 \times 10^6}{1{,}067 \times 10^6} = 6{,}22 \text{ MPa}$$

The solid timber C24 presents a characteristic value of the bending strength $f_{m,k}$ equal to:

$$f_{m,k} = 24 \text{ MPa}$$

Assuming that the bending moment M_d is calculated with a load combination in which the snow is the load of short duration, the k_{mod} factor assumes the value equal to 0.9, and the design bending strenght is equal to:

$$f_{m,d} = \frac{k_{mod} \cdot f_{m,k}}{\gamma_M} = \frac{0{,}8 \cdot 24}{1{,}5} = 12{,}80 \text{ MPa}$$

The bending check is satisfied, obtaining:

$$\frac{\sigma_{m,d}}{f_{m,d}} = \frac{6{,}22}{12{,}80} = 48{,}59\%$$

SHEAR CHECK

The checks are conducted according to § 6.1.7 of EN 1995-1-1. The following expression shall be satisfied:

$$\frac{\tau_d}{f_{v,d}} \leq 1$$

where:

- τ_d is the design shear stress
- $f_{v,d}$ is the design shear strength for the actual condition

For the verification of shear resistance of members in bending, the influence of cracks should be taken into account using an effective width of the member given as:

$$b_{ef} = k_{cr} \cdot b$$

For solid timber k_{crit} is equal to 0,67.

The maximum design shear stress in a rectangular cross section can be evaluated using the following expression:

$$\tau_d = \frac{3}{2} \cdot \frac{V_d}{k_{cr} \cdot A} = \frac{3}{2} \times \frac{5,29 \times 10^3}{0,67 \times 32 \times 10^3} = 0,37 \text{ MPa}$$

The solid timber C24 presents a characteristic value of shear strenght $f_{v,k}$ equal to:

$$f_{v,k} = 4 \text{ MPa}$$

The design shear strength is:

$$f_{v,d} = \frac{k_{mod} \cdot f_{v,k}}{\gamma_M} = \frac{0,8 \times 4}{1,5} = 2,13 \text{ MPa}$$

The shear check is satisfied, obtaining:

$$\frac{\tau_d}{f_{v,d}} = \frac{0,37}{2,13} = 17,37\%$$

RESULTS PROVIDED BY THE SOFTWARE

- Bending check: 48,67%
- Shear check: 17,36%

RESULTS COMPARISON

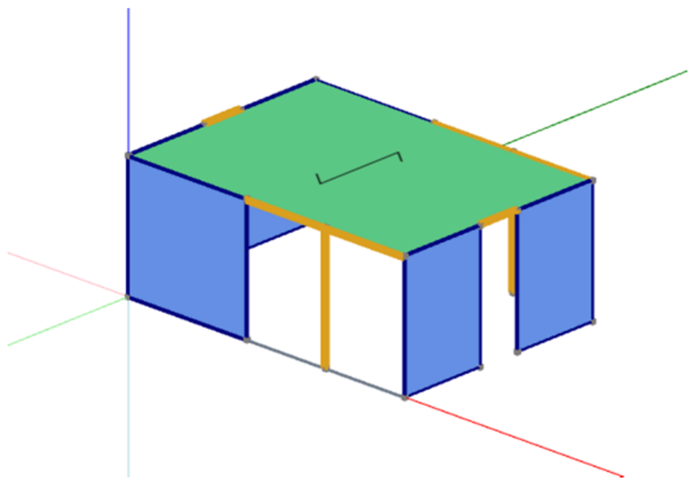
Comparison between the value provided by the software and the value calculated in an independent manner:

Validated parameter	Independent verification	Software
Bending check	48,59%	49%
Shear check	17,37%	17%

CONCLUSIONS

The comparison shows how results provided by the software are comparable to results obtained with an independent verification. Errors are caused by different approximation between the manual verification and the software.

EXAMPLE 10: BEAM LOAD



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DESCRIPTION OF THE PROBLEM

The calculation of the distributed load on the two spans beam q_{SLU} is conducted with reference to the following ultimate limit load combination:

Load	G_1	G_2	Live load cat. H	Snow	Wind
Partial factors	1,3	1,5	1,5	0	0

Assuming:

- Floor self weight G_1: $0{,}38\ \mathrm{kN/m^2}$
- Permanent load G_2: $2{,}00\ \mathrm{kN/m^2}$
- Live load cat. H Q_H: $0{,}50\ \mathrm{kN/m^2}$
- Spacing joists i: $500\ \mathrm{mm}$
- Floor span l: $5\ \mathrm{m}$
- Beam self weight $G_{1,st}$: $0{,}23\ \mathrm{kN/m}$

INDEPENDENT VERIFICATION

The restraint reaction of the generic joist results:

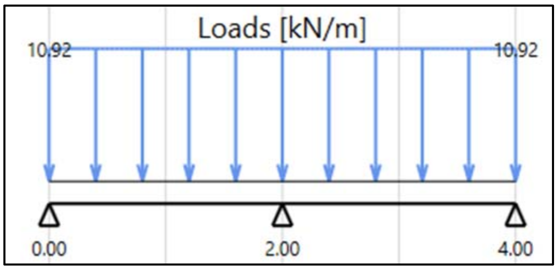
$$R_s = (1{,}3 \cdot G_1 + 1{,}5 \cdot G_2 + 1{,}5 \cdot Q_H) \cdot i \cdot \frac{l}{2} = (1{,}3 \times 0{,}38 + 1{,}5 \times 2{,}00 + 1{,}5 \times 0{,}5) \times 0{,}5 \times 2{,}5 = 5{,}30\ \mathrm{kN}$$

The distributed load per meter q_{SLU} acting on the beam is calculated by adding to the reaction of the joist, redistributed along the spacing of the joists, the weight per meter of the beam multiplied by the appropriate partial factor γ_g:

$$q_{SLU} = \frac{R_s}{i} + 1{,}3 \cdot G_{1,t} = \frac{5{,}30}{0{,}50} + 1{,}3 \times 0{,}23 = 10{,}91 \text{ kN}$$

RESULTS PROVIDED BY THE SOFTWARE

- q_{SLU}: $10{,}92\ kN/m$



RESULTS COMPARISON

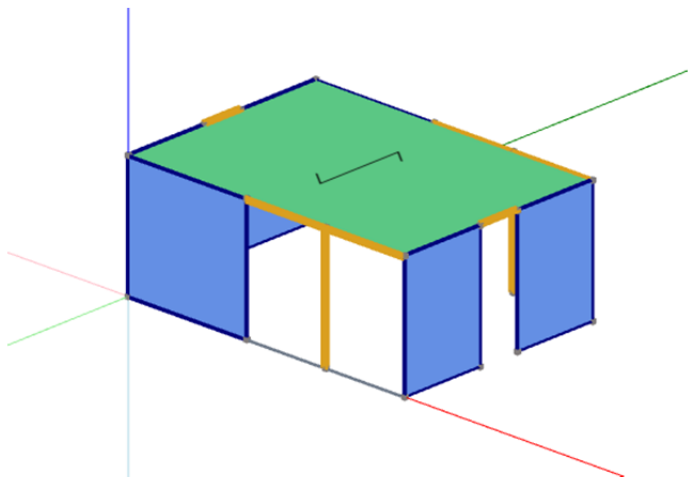
Comparison between the value provided by the software and the value calculated in an independent manner:

Validated parameter	Independent verification	Software	Percentage error
q_{SLU}	$10{,}91$ kN/m	$10{,}92$ kN/m	0,01%

CONCLUSIONS

The comparison shows the coincidence between the value provided by the software and that hand-calculated.

EXAMPLE 11: CALCULATION OF BEAM FORCES



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DESCRIPTION OF THE PROBLEM

In the example we consider a beam consisting of two equal spans (each one has a length of 2 m). The maximum values of the forces in the beam are determined considering various configurations with alternated loads. In this example we consider the following load combinations.

Load	G_1	G_2	Live load cat. H	Snow	Wind
q_{max}	1,3	1,5	1,5	0	0
q_{min}	1	0	0	0	0

The distributed loads q_{max} e q_{min} assume the values:

- $q_{max} = 10{,}96\ \mathrm{kN/m}$
- $q_{min} = 1{,}18\ \mathrm{kN/m}$

INDEPENDENT VERIFICATION

Configuration 1/2 (maximum moment in span)

Given the symmetry of the beam, it is sufficient to solve the beam with reference to one of the two load configurations.

- **Bending moment:** the moment on the support and the maximum span moment assume the values:

$$M_A = M_C = 0$$

$$M_B = -\frac{q_{max} \cdot l_1^3 + q_{min} \cdot l_2^3}{8 \cdot (l_1 + l_2)} = -\frac{10{,}96 \times 2^3 + 1{,}18 \times 2^3}{8 \cdot (2 + 2)} = -3{,}04 \text{ kNm}$$

$$M_{max,campata1} = \frac{q_{max} \cdot l_1^2}{8} + \frac{M_B}{2} \cdot \left[-\frac{M_B}{q_{max} \cdot l_1^2} + 1 \right] =$$

$$= \frac{10{,}96 \times 2^2}{8} - \frac{3{,}04}{2} \times \left[-\frac{3{,}04}{10{,}96 \cdot 2^2} + 1 \right] = 4{,}07 \text{ kNm}$$

- **Reaction forces:** the reaction forces assume the values:

$$V_A = \frac{q_{max} \cdot l_1}{2} + \frac{M_B}{l_1} = \frac{10{,}96 \times 2}{2} - \frac{3{,}04}{2} = 9{,}44 \text{ kN}$$

$$V_B = \frac{q_{max} \cdot l_1}{2} + \frac{q_{min} \cdot l_2}{2} - M_B \cdot \left[\frac{1}{l_1} + \frac{1}{l_2} \right] =$$

$$= \frac{10{,}96 \times 2}{2} + \frac{1{,}19 \times 2}{2} + 3{,}04 \times \left[\frac{1}{2} + \frac{1}{2} \right] = 12{,}91 \text{ kN}$$

$$V_C = \frac{q_{min} \cdot l_2}{2} + \frac{M_B}{l_2} = \frac{1{,}19 \times 2}{2} - \frac{3{,}04}{2} = -0{,}33 \text{ kN}$$

- **Shear:** the shear localised at the end supports is equal to reaction forces while the left and right shear of the central support assumes the values:

$$T_{B,sx} = V_A - q_{max} \cdot l_1 = 9{,}44 - 10{,}96 \times 2 = -12{,}48\ kN$$

$$T_{B,dx} = V_C - q_{max} \cdot l_1 = -0{,}33 + 1{,}19 \times 2 = 2{,}05\ kN$$

Configuration 3 (maximum moment on the central support)

- **Reaction forces:** the reaction forces assume the values:

$$V_A = V_C = 0{,}375 \cdot q_{max} \cdot l = 0{,}375 \times 10{,}96 \times 2 = 8{,}22 \text{ kN/m}$$

$$V_B = 1{,}25 \cdot q_{max} \cdot l = 1{,}25 \times 10{,}96 \times 2 = 27{,}40 \text{ kN/m}$$

- **Shear:** the shear localised at the end supports is equal to reaction forces while the left and right shear of the central support assumes the values:

$$T_{B,sx} = -0{,}625 \cdot q_{max} \cdot l = -0{,}625 \times 10{,}96 \times 2 = -13{,}70 \text{ kN}$$

$$T_{B,dx} = 0{,}625 \cdot q_{max} \cdot l = 0{,}625 \times 10{,}96 \times 2 = 13{,}70 \text{ kN}$$

- **Bending moment:** the moment on the support and the maximum span moment assume the values:

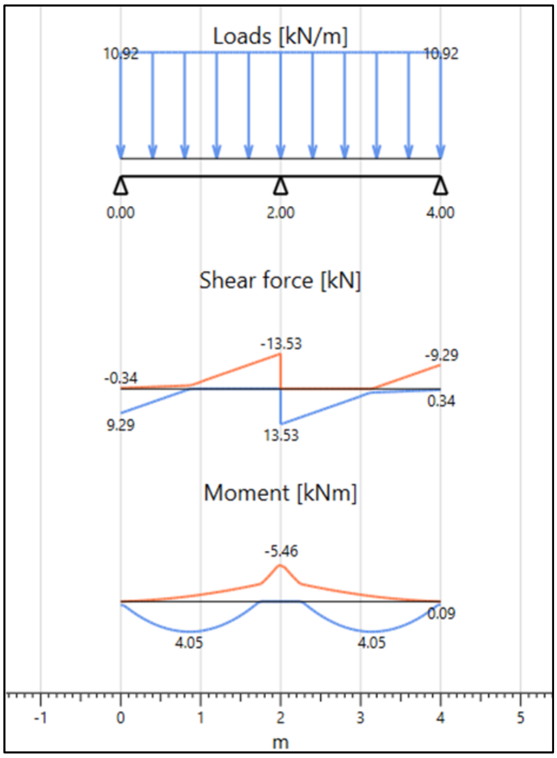
$$M_A = M_C = 0$$

$$M_B = -\frac{1}{8} \cdot q_{max} \cdot l^2 = -\frac{1}{8} \times 10,96 \times 2^2 = -5,48 \text{ kNm}$$

$$M_{max,campata} = \frac{1}{14,3} \cdot q_{max} \cdot l^2 = \frac{1}{14,3} \times 10,96 \times 2^2 = 3,07 \text{ kNm}$$

RESULTS PROVIDED BY THE SOFTWARE

- Maximum positive shear on A support: $9,29$ kN
- Maximum negative shear on A support: $-0,34$ kN
- Maximum positive shear on B support: $13,53$ kN
- Maximum negative shear on B support: $-13,53$ kN
- Maximum span moment: $4,05$ kNm
- Maximum moment on B support: $-5,46$ kNm



RESULTS COMPARISON

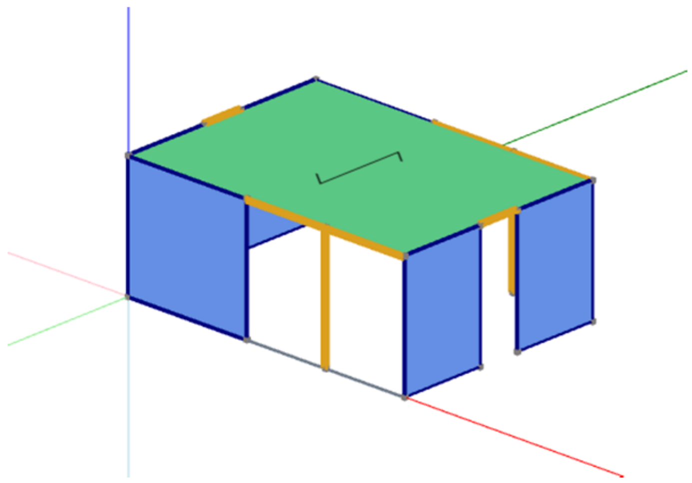
Comparison between the values provided by the software and the values calculated in an independent manner:

Validated parameter	Independent verification	Software	Percentage error
$T_{A,max+}$	9,44 kN	9,29 kN	−1,6%
$T_{A,max-}$	−0,33 kN	−0,34 kN	-3%
$T_{B,sx}$	−13,70 kN	−13,53 kN	-1,2%
$T_{B,dx}$	13,70 kN	13,53 kN	1,2%
$M_{max,span}$	4,07 kNm	4,05 kNm	0,5%
$M_{max,support}$	−5,48 kNm	−5,46 kNm	0,4%

CONCLUSIONS

The comparison shows how results provided by the software are comparable to results obtained with an independent verification. Errors are caused by different approximation between the manual verification and the software.

EXAMPLE 12: RESISTANCE CHECKS OF THE TIMBER BEAM



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DESCRIPTION OF THE PROBLEM

The resistance checks (bending and shear) are conducted with reference to the dispositions provided by UNI EN 1995-1-1:2009 al § 6.3.3, assuming:

- Beam cross section $b \times h$: 160×240 mm
- Material: homogenous glulam timber GL28h
- Bending moment: $M_d = 5{,}48$ kNm

INDEPENDENT VERIFICATION

The area *A* and the modulus of resistance *W* are equal to:

$$A = B \cdot h = 160 \times 240 = 38400 \text{ mm}^2$$

$$W = \frac{B \cdot h^2}{6} = \frac{160 \times 240^2}{6} = 1{,}536 \times 10^6 \text{ mm}^3$$

BENDING CHECK

The checks are conducted according to § 6.3.2 of EN 1995-1-1. The following expression shall be satisfied:

$$\frac{\sigma_{m,d}}{k_{crit} \cdot f_{m,d}} \leq 1$$

where:

- $\sigma_{m,d}$ is the design bending stress
- $f_{m,d}$ is the design bending strength

- k_{crit} is a factor which takes into account the reduced bending strength due to lateral buckling

k_{crit} is assumed equal to 1.0 for beams in which the lateral displacement of the compressed edge is prevented over the entire length and the torsional rotation is prevented at the supports.

The design bending stress assumes the value:

$$\sigma_{m,d} = \frac{M_d}{W} = \frac{5{,}46 \times 10^6}{1{,}536 \times 10^6} = 3{,}55\ \mathrm{MPa}$$

The glulam timber GL28h presents a characteristic value of the bending strength $f_{m,k}$ equal to:

$$f_{m,k} = 28\ \mathrm{MPa}$$

Assuming that the bending moment M_d is calculated with a load combination in which the snow is the load of short duration, the k_{mod} factor assumes the value equal to 0.9, and the design bending strenght is equal to:

$$f_{m,d} = \frac{k_{mod} \cdot f_{m,k}}{\gamma_M} = \frac{0{,}8 \times 28}{1{,}45} = 15{,}45\ \mathrm{MPa}$$

The bending check is satisfied, obtaining:

$$\frac{\sigma_{m,d}}{f_{m,d}} = \frac{3{,}55}{15{,}45} = 22{,}98\%$$

SHEAR CHECK

The checks are conducted according to § 6.1.7 of EN 1995-1-1. The following expression shall be satisfied:

$$\frac{\tau_d}{f_{v,d}} \leq 1$$

where:

- τ_d is the design shear stress
- $f_{v,d}$ is the design shear strength for the actual condition

For the verification of shear resistance of members in bending, the influence of cracks should be taken into account using an effective width of the member given as:

$$b_{ef} = k_{cr} \cdot b$$

For glulam timber k_{crit} is equal to 0,67.

The maximum design shear stress in a rectangular cross section can be evaluated using the following expression:

$$\tau_d = \frac{3}{2} \cdot \frac{V_d}{k_{cr} \cdot A} = \frac{3}{2} \times \frac{13{,}59 \times 10^3}{0{,}67 \times 38{,}4 \times 10^3} = 0{,}79\ \mathrm{MPa}$$

The solid timber GL28h presents a characteristic value of shear strenght $f_{v,k}$ equal to:

$$f_{v,k} = 3{,}5 \text{ MPa}$$

The design shear strength is:

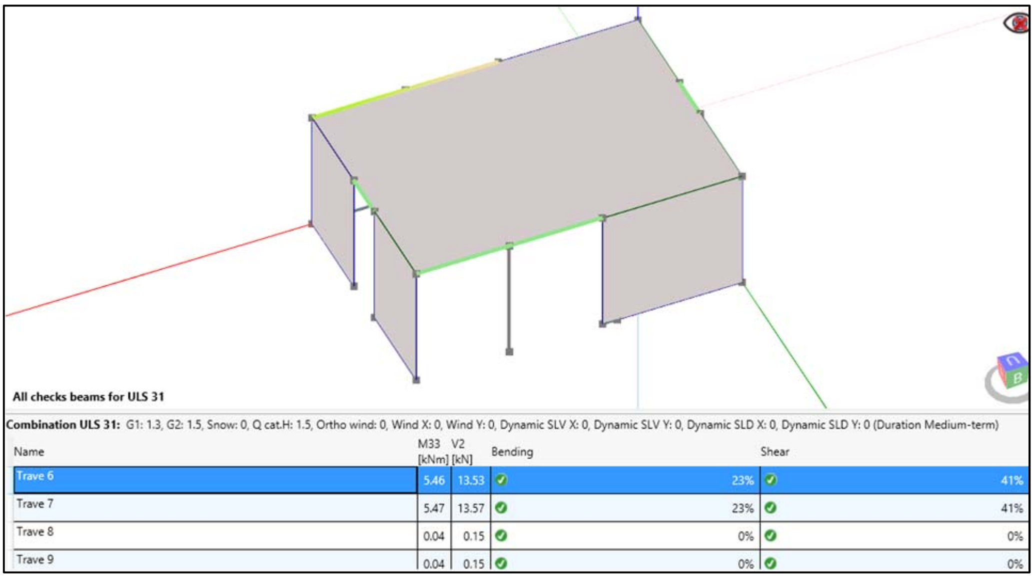
$$f_{v,d} = \frac{k_{mod} \cdot f_{v,k}}{\gamma_M} = \frac{0{,}8 \cdot 3{,}5}{1{,}45} = 1{,}77 \text{ MPa}$$

The shear check is satisfied, obtaining:

$$\frac{\tau_d}{f_{v,d}} = \frac{0{,}79}{1{,}93} = 40{,}93\%$$

RESULTS PROVIDED BY THE SOFTWARE

- Bending check: 23%
- Shear check: 41%



RESULTS COMPARISON

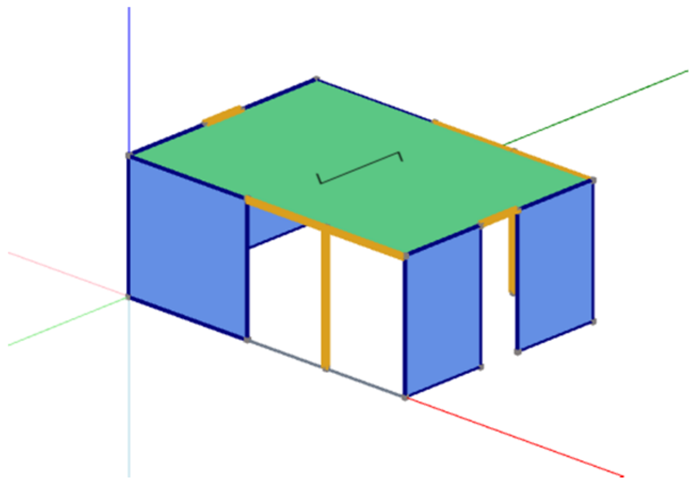
Comparison between the values provided by the software and the values calculated in an independent manner:

Validated parameter	**Independent verification**	**Software**
Bending check	22,98%	23%
Shear check	40,93%	41%

CONCLUSIONS

The comparison shows how results provided by the software are comparable to results obtained with an independent verification. Errors are caused by different approximation between the manual verification and the software.

EXAMPLE 13: RESISTANCE CHECKS OF THE STEEL BEAM

	**File name:** EsempioValidazioneAcciaio.bldg

DESCRIPTION OF THE PROBLEM

The resistance checks (bending and shear) are conducted with reference to the dispositions provided by UNI EN 1993-1-1:2009 al § 6.2., assuming:

- Beam profile IPE 100, Class 1
- Material: steel S235 EN 10025 -2
- Maximum bending moment: $M_{Ed} = 6{,}67\ \mathrm{kNm}$
- Maximum shear: $V_{Ed} = 16{,}55\ \mathrm{kN}$

INDEPENDENT VERIFICATION

In this example we do the bending and shear checks for element “Beam 6”.

When the shear force $\boldsymbol{V_{Ed}}$ is less than half the plastic shear resistance $\boldsymbol{V_{pl,Rd}}$ its effect on the moment resistance may be neglected.

$$V_{Ed} \leq 0.5\, V_{pl,Rd}$$

Otherwise the reduced moment resistance should be taken as the design resistance of the cross-section, calculated using a reduced yield strength for the shear area A_v:

$$(1-\rho)f_{yk}$$

where:

$$\rho = \left(\frac{2V_{Ed}}{V_{pl,Rd}} - 1\right)^2$$

The reduced design plastic resistance moment allowing for the shear force may alternatively be obtained for I-cross-sections (class 1 or 2) with equal flanges and bending about the major axis as follows:

$$M_{c,V,Rd} = \frac{\left[W_{pl,y} - \frac{\rho \cdot A_w^2}{4\, t_w}\right] f_{yk}}{\gamma_{M0}} \leq M_{c,Rd}$$

In this case the design plastic resistance shear is equal to:

$$V_{pl,Rd} = \frac{A_v \cdot f_{yk}}{\sqrt{3} \cdot \gamma_{M0}} = 65{,}64\ kN$$

- A_v is the shear area equal to 5,08 mm² for a profile IPE100;
- f_{yk} is the charateristic yield strenght equal to 235 MPa;
- γ_{M0} is the partial safety factor for the resistance of cross-section equal to 1,05.

In this example the design shear force is lower than $0{,}5\ V_{pl,Rd}$ and therefore the design moment resistance is:

$$M_{c,Rd} = \frac{W_{pl,y} f_{yk}}{\gamma_{M0}} = 8{,}82\ kNm$$

where:

- $W_{pl,y}$ is the plastic section modulus

The bending check results:

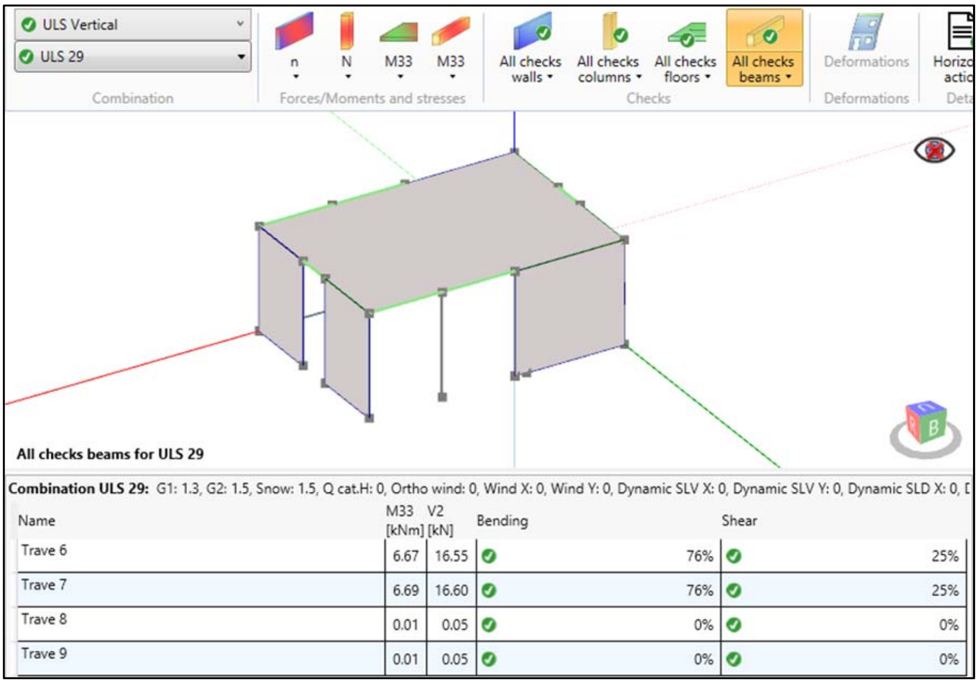
$$\frac{M_{Ed}}{M_{c,Rd}} = \frac{6{,}67\ kNm}{8{,}82\ kNm} = 75{,}62\ \%$$

The shear check results:

$$\frac{V_{Ed}}{V_{pl,Rd}} = \frac{16{,}55\ kN}{65{,}64\ kN} = 25{,}21\ \%$$

RESULTS PROVIDED BY THE SOFTWARE

- Bending check 76 %
- Shear check: 25 %



RESULTS COMPARISON

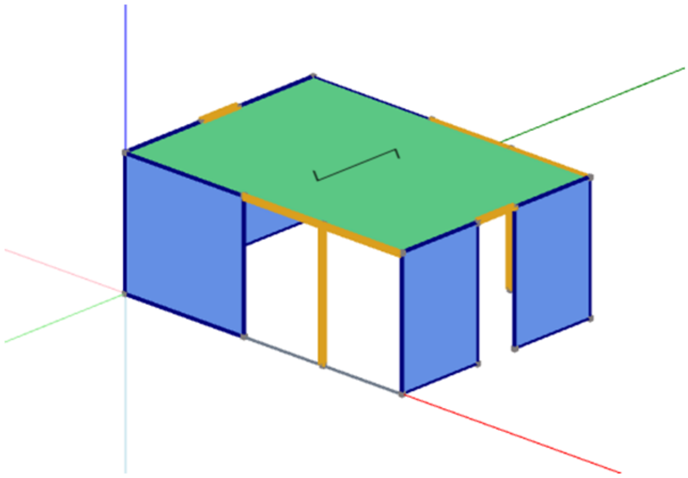
Comparison between the value provided by the software and the value calculated in an independent manner:

Validated parameter	Independent verification	Software
Bending check	75,62 %	76 %
Shear check	25,21%	25%

CONCLUSIONS

The comparison shows how results provided by the software are comparable to results obtained with an independent verification.

EXAMPLE 14: DEFLECTION CHECK OF TIMBER BEAM



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DESCRIPTION OF THE PROBLEM

The deflection checks are conducted with reference to the dispositions provided by UNI EN 1995-1-1:2009 al § 2.2.3, considering a symmetrical beam with 2 spans, assuming:

- Cross section of the beam $b \times h$: 160×240 mm
- Spans length: $2\ m$
- Material: homogenous glulam timber GL28h
- Load q under the combination SLE rare (1): $q = 6{,}43$ kN/m

The code orders that we have to do a check for the instantaneous deflection and another one for the final deflection (considering the creep deflection).

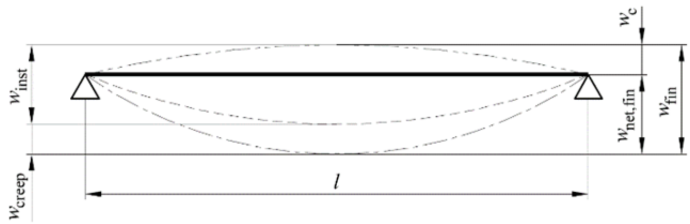
The net deflection below a straight line between the supports, $w_{net,fin}$, is taken as:

$$w_{net,fin} = w_{inst} + w_{creep} - w_c = w_{fin} - w_c$$

where:

- $w_{net,fin}$ is the net final deflection
- w_{inst} is the instantaneous deflection
- w_{creep} is the creep deflection
- w_c is the precamber (if applied)

- w_{fin} is the final deflection



INDEPENDENT VERIFICATION

CALCULATION OF THE INSTANTANEOUS AND THE FINAL DEFLECTION

The inertia moment $J_{y\text{-}y}$ of the cross section is:

$$J_{y-y} = \frac{B \cdot h^3}{12} = \frac{160 \times 240^3}{12} = 1{,}84 \times 10^8 \text{ mm}^4$$

The deflection of a single span can be evaluated considering the fact that, in the present case, there is a symmetry of geometry, load and constraints. Therefore, the single span can be assimilated to a beam simply supported on one end and fixed on the other end.

So the maximum deflection is given by:

$$w_{inst,max} = \frac{2}{384} \cdot \frac{q \cdot l^4}{E_{0,mean} \cdot J_{y-y}} = \frac{2}{384} \times \frac{6{,}43 \times 10^3 \times 2^4}{1{,}26 \times 10^{10} \times 1{,}8432 \times 10^{-4}} = 0{,}23 \text{ mm}$$

For structures consisting of members, components and connections with the same creep behaviour and under the assumption of a linear relationship between the actions and the corresponding deformations the final deformation, wfin, may be taken as:

$$w_{fin} = w_{fin,G} + w_{fin,Q1} + \sum w_{fin,Qi}$$

where:

$w_{fin,G} = w_{inst,G} \cdot \left(1 + k_{def}\right)$ for a permanent action, G

$w_{fin,Q,1} = w_{inst,Q,1} \cdot \left(1 + \Psi_{2,1} \cdot k_{def}\right)$ for the leading variable action, Q_1

$w_{fin,Q,i} = w_{inst,Q,i} \cdot \left(\Psi_{0,i} + \Psi_{2,1} \cdot k_{def}\right)$ for accompanying variable actions, Q_i (i>1)

In this case we have:

$$w_{fin} = w_{fin,G} = 023 \cdot (1 + 0{,}6) = 0{,}37 \text{ mm}$$

The limiting values for deflections are assumed as shown in the following table.

	w_{inst}	$w_{net,fin}$
Beam on two supports	l/300	l/250
Cantilevering beams	l/150	l/125

CHECK OF INSTANTANEOUS DEFLECTION

The maximum value for the instantaneous deflection results:

$$w_{inst,lim} = \frac{l}{300} = \frac{2000}{300} = 6{,}67 \text{ mm}$$

The check is satisfied:

$$\frac{w_{inst}}{w_{inst,lim}} = \frac{0{,}23}{6{,}67} = 3{,}45\%$$

CHECK OF FINAL DEFLECTION

The maximum value for the final deflection results:

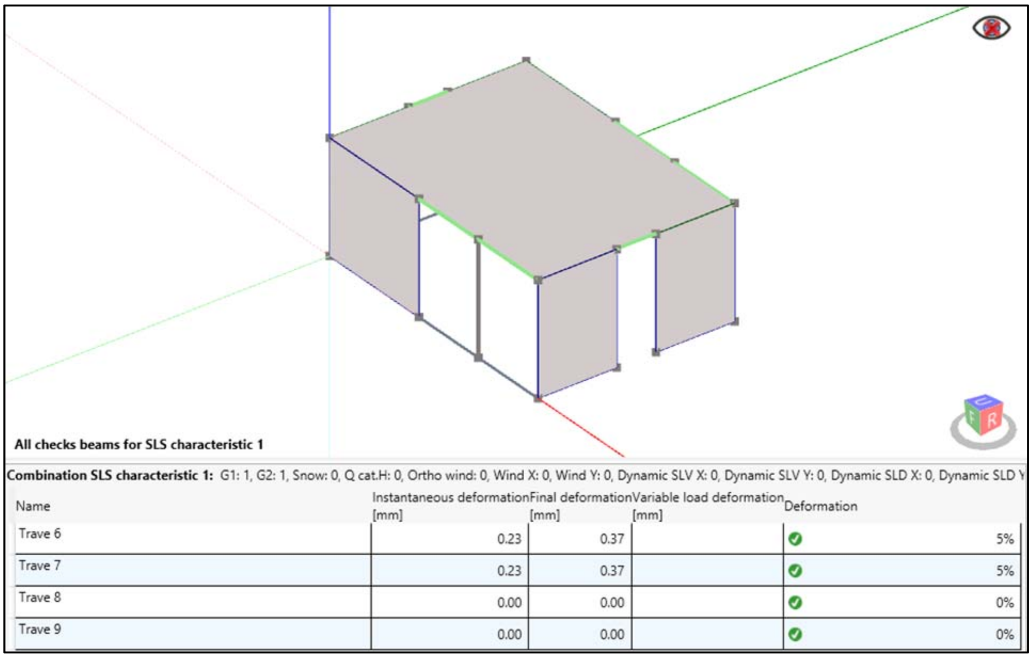
$$w_{fin,lim} = \frac{l}{250} = \frac{2000}{250} = 8{,}00 \text{ mm}$$

The check is satisfied:

$$\frac{w_{fin}}{w_{fin,lim}} = \frac{0{,}37}{8{,}00} = 4{,}62\%$$

RESULTS PROVIDED BY THE SOFTWARE

- Instantaneous deflection: $0{,}23$ mm
- Final deflection: $0{,}37$ mm
- Check of instantaneous deflection: 3%
- Check of final deflection: 5%



Name	Instantaneous deformation [mm]	Final deformation [mm]	Variable load deformation [mm]	Deformation
Trave 6	0.23	0.37		5%
Trave 7	0.23	0.37		5%
Trave 8	0.00	0.00		0%
Trave 9	0.00	0.00		0%

RESULTS COMPARISON

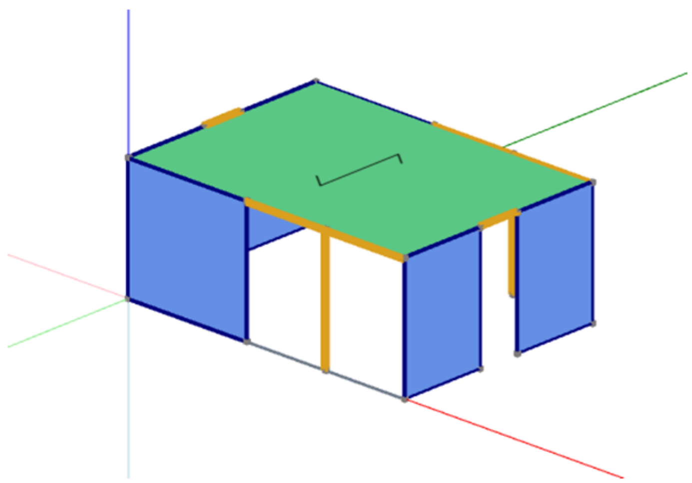
Comparison between the values provided by the software and the values calculated in an independent manner:

Validated parameter	Independent verification	Software	Percentage error
w_{ist}	0,23 mm	0,23 mm	0%
w_{fin}	0,37 mm	0,37 mm	0%
Check w_{ist}	3,45%	3%	
Check w_{fin}	4,62%	5%	

CONCLUSIONS

The comparison shows how results provided by the software are comparable to results obtained with an independent verification.

EXAMPLE 15: AXIAL FORCE IN THE PILLAR



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DESCRIPTION OF THE PROBLEM

The calculation of the axial force N_{SLU} in the pillar is conducted with reference to the following ultimate limit load combination:

Load	G_1	G_2	Live load cat. H	Snow	Wind
Partial factors	1,3	1,5	1,5	0	0

Assuming:

- Pillar's cross section *l* : $200 \times 200\ \mathrm{mm}$
- Pillar's heigth: $l = 2{,}8\ \mathrm{m}$
- Material: Homogenous glulam timber GL24h
- Reaction on the pillar: $R_B = 27{,}19\ \mathrm{kN}$

INDEPENDENT VERIFICATION

The axial force N_{SLU} is calculated adding to the reaction of the beam, applied on the top of the element, the weight of the pillar multiplied by the appropriate partial γ_g:

$$N_{SLU} = R_B + \gamma \cdot h \cdot l^2 = 27{,}19 + 6 \times 2{,}8 \times 0{,}2^2 = 28{,}05\ \mathrm{kN}$$

RESULTS PROVIDED BY THE SOFTWARE

- $N_{SLU} = 28{,}16\ \mathrm{kN}$

RESULTS COMPARISON

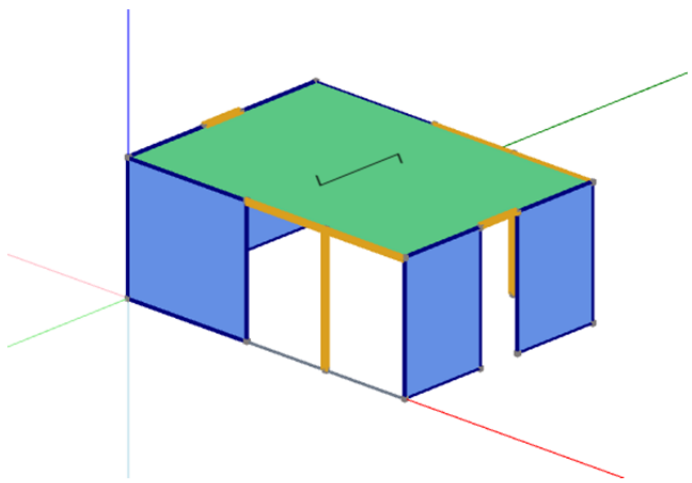
Comparison between the value provided by the software and the value calculated in an independent manner:

Validated parameter	Independent verification	Software	Percentage error
N_{SLU}	28,05 kN	28,16 kN	0,4%

CONCLUSIONS

The comparison shows how results provided by the software are comparable to results obtained with an independent verification. Errors are caused by different approximation between the manual verification and the software.

EXAMPLE 16: STABILITY CHECK IN A TIMBER PILLAR



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DESCRIPTION OF THE PROBLEM

The stability checks are conducted with reference to the dispositions provided by UNI EN 1995-1-1:2009 al § 6.3.2, assuming:

- Pillar's cross section *l* : 200×200 mm
- Pillar's heigth: $l = 2{,}8$ m
- Material: Homogenous glulam timber GL24h
- Reaction on the pillar: $N = 28{,}30$ kN

INDEPENDENT VERIFICATION

Given the symmetry of the cross section of the pillar, the inertial radius of gyration has the same value in the two main directions:

$$i_y = i_z = i = \frac{l}{\sqrt{12}} = \frac{200}{\sqrt{12}} = 57{,}74 \text{ mm}$$

The buckling length is:

$$l_0 = l = 2800 \text{ mm}$$

The slenderness is, for both main directions, equal to:

$$\lambda_y = \lambda_z = \lambda = \frac{l_0}{i} = \frac{2800}{57{,}74} = 48{,}49$$

The relative slenderness ratios should be taken as:

$$\lambda_{rel,y} = \frac{\lambda_y}{\pi} \cdot \sqrt{\frac{f_{c,0,k}}{E_{0,05}}}$$

$$\lambda_{rel,z} = \frac{\lambda_z}{\pi} \cdot \sqrt{\frac{f_{c,0,k}}{E_{0,05}}}$$

where:

- Charateristic compression resistance $f_{c,0,k}$ is equal to 24 kN
- Young modulus parallel to grain $E_{0,05}$ is equal to 9600 MPa

The relative slenderness ratios is:

$$\lambda_{rel} = \frac{48{,}49}{\pi} \cdot \sqrt{\frac{24}{9600}} = 0{,}77$$

The stresses should satisfy the expressions:

$$\frac{\sigma_{c,0,d}}{k_{c,y} \cdot f_{c,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \cdot \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

$$\frac{\sigma_{c,0,d}}{k_{c,z} \cdot f_{c,0,d}} + k_m \cdot \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

where the symbols are defined as follows:

$$k_{c,y} = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{rel,y}^2}}$$

$$k_{c,z} = \frac{1}{k_z + \sqrt{k_z^2 - \lambda_{rel,z}^2}}$$

$$k_y = 0{,}5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel,y} - 0{,}3\right) + \lambda_{rel,y}^2\right)$$

$$k_z = 0{,}5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel,z} - 0{,}3\right) + \lambda_{rel,z}^2\right)$$

β_C is a factor for members within the straightness limits defined in Section 10 of EN 1995-1-1 and assumes the following values:

- $\beta_C = 0{,}2$ for solid timber
- $\beta_C = 0{,}1$ for glued laminated timber and *LVL*

The factors k_y, k_z, k_{cz}, k_{cy} result:

$$k_y = k_z = k = 0{,}5 \times (1 + 0{,}1 \times (0{,}77 - 0{,}3) + 0{,}77^2) = 0{,}82$$

$$k_{c,y} = k_{c,z} = k_c = \frac{1}{0{,}82 + \sqrt{0{,}82^2 - 0{,}77^2}} = 0{,}955$$

The design compressive stress along the grain is:

$$\sigma_{c,0,d} = \frac{N}{A} = \frac{28300}{0{,}2^2} \times 10^{-6} = 0{,}71 \text{ MPa}$$

The design compressive strenght along the grain is,

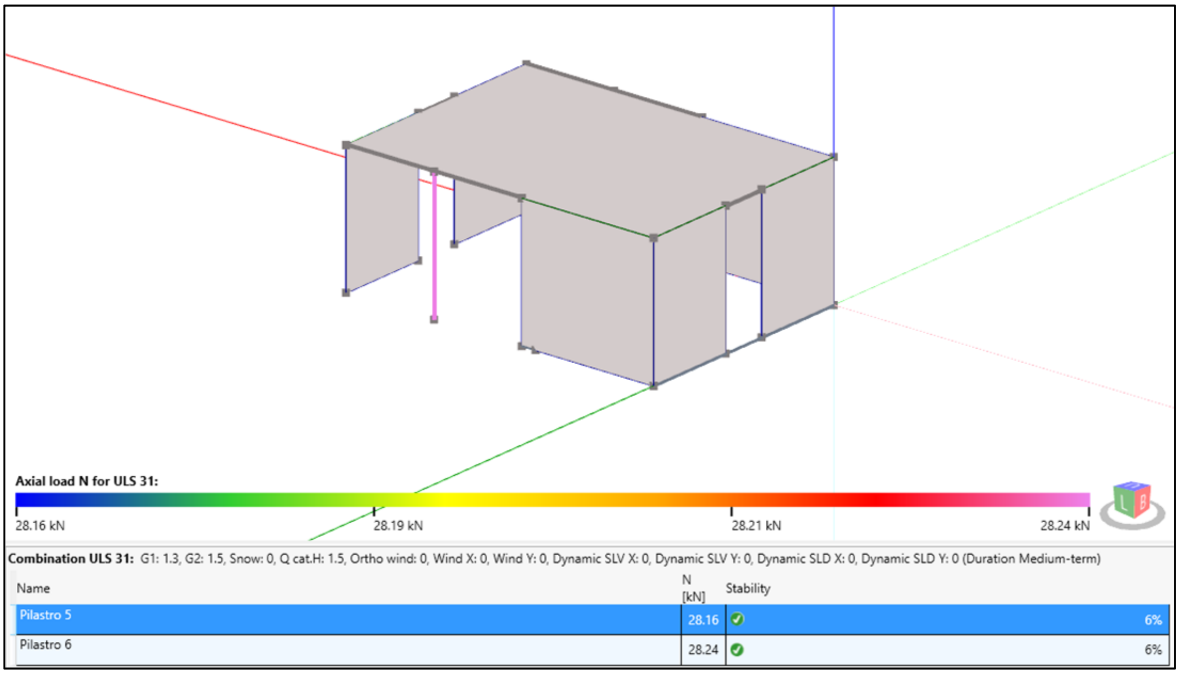
$$\sigma_{c,0,d} = \frac{k_{mod} \cdot \sigma_{c,0,k}}{\gamma_M} = \frac{0{,}8 \cdot 24}{1{,}45} = 13{,}24 \text{ MPa}$$

The check is satisfied, obtaining:

$$\frac{0{,}71}{0{,}955 \cdot 13{,}24} = 5{,}61\%$$

RESULTS PROVIDED BY THE SOFTWARE

- Axial force N: 28,16 kN
- Stability check: 6%



RESULTS COMPARISON

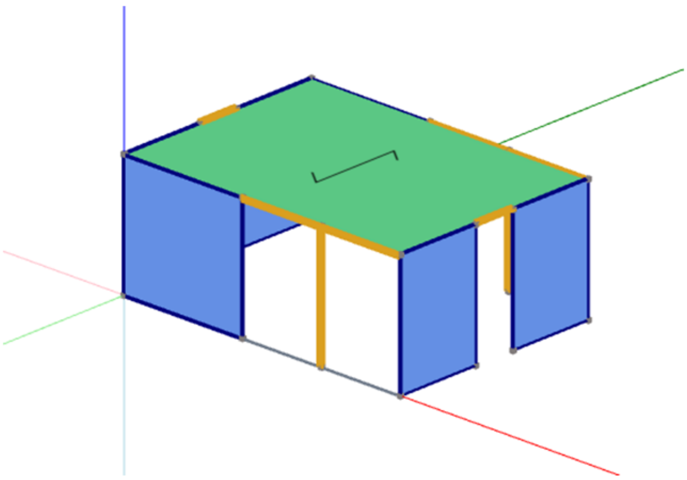
Comparison between the value provided by the software and the value calculated in an independent manner:

Validated parameter	Independent verification	Software
Stability check	5,61%	6%

CONCLUSIONS

The comparison shows how results provided by the software are comparable to results obtained with an independent verification. Errors are caused by different approximation between the manual verification and the software.

EXAMPLE 17: STABILITY CHECK IN A STEEL PILLAR

	**File name:** EsempioValidazioneAcciaio.bldg

DESCRIPTION OF THE PROBLEM

The stability checks are conducted with reference to the dispositions provided by UNI EN 1993-1-1 al § 6.3.1, assuming:

- Steel profile HEA 100, in class 1 (Compression section class)
- Pillar's heigth: $L = 2{,}8\ \mathrm{m}$
- Material S235 – EN 10025 - 2
- Vertical load: $N = 34{,}11\ \mathrm{kN}$

INDEPENDENT VERIFICATION

The inertial properties of the section are deducted from the profile table; the radius of gyration of inertia, in the two main directions have the following values:

$$i_y = 40{,}6\ mm$$

$$i_z = 25{,}1\ mm$$

The buckling length, in both direction, is:

$$L_{\mathrm{cr}} = L = 2800\ \mathrm{mm}$$

The non-dimensional slenderness is given by:

$$\bar{\lambda} = \sqrt{\frac{A \cdot f_{yk}}{N_{cr}}} = \frac{L_{cr}}{i}\frac{1}{\lambda_l} \quad \text{(Class 1, 2 e 3)}$$

$$\bar{\lambda} = \sqrt{\frac{A_{eff} \cdot f_{yk}}{N_{cr}}} = \frac{L_{cr}}{i}\frac{\sqrt{\frac{A_{eff}}{A}}}{\lambda_l} \quad \text{(Class 4)}$$

where:

L_{cr} is the buckling length in the buckling plane considered;

i is the radius of gyration about the relevant axis, determined using the properties of the gross cross-section;

$\lambda_l = \pi\sqrt{\frac{E}{f_{yk}}}$ is the slenderness of proportionality.

The non-dimensional slenderness, for the two main directions, assume the values of:

$$\bar{\lambda}_y = 0{,}735$$

$$\bar{\lambda}_z = 1{,}187$$

A compression member should be verified against buckling as:

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1$$

where $N_{b,Rd}$, is the design buckling resistance of the compression member:

$$N_{b,Rd} = \frac{\chi\, A\, f_{yk}}{\gamma_{M1}} \quad \text{(for class 1,2 e 3)}$$

$$N_{b,Rd} = \frac{\chi\, A_{eff} f_{yk}}{\gamma_{M1}} \quad \text{(for class 4)}$$

where γ_{M1} is the partial factor for resistance of members to instability equal to 1.05, A and A_{eff} are the gross area and the effective area of the cross section (the effective area is calculated in accordance to § 4.4 of UNI EN 1993-1-5) and χ is the reduction factor for the relevant buckling mode, given by:

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \quad con\ \chi \leq 1$$

In this case, for two main directions, result:

$$\Phi_y = 0.5\left[1 + \alpha\left(\bar{\lambda}_y - 0{,}2\right) + \bar{\lambda}_y^2\right] = 0{,}861\ e\ \chi_y = 0{,}764$$

$$\Phi_z = 0.5\left[1 + \alpha\left(\bar{\lambda}_z - 0,2\right) + \bar{\lambda}_z^2\right] = 1,446 \; e \; \chi_z = 0,440$$

The design buckling resistance is:

$$N_{b,Rd,y} = 362,4 \; kN$$

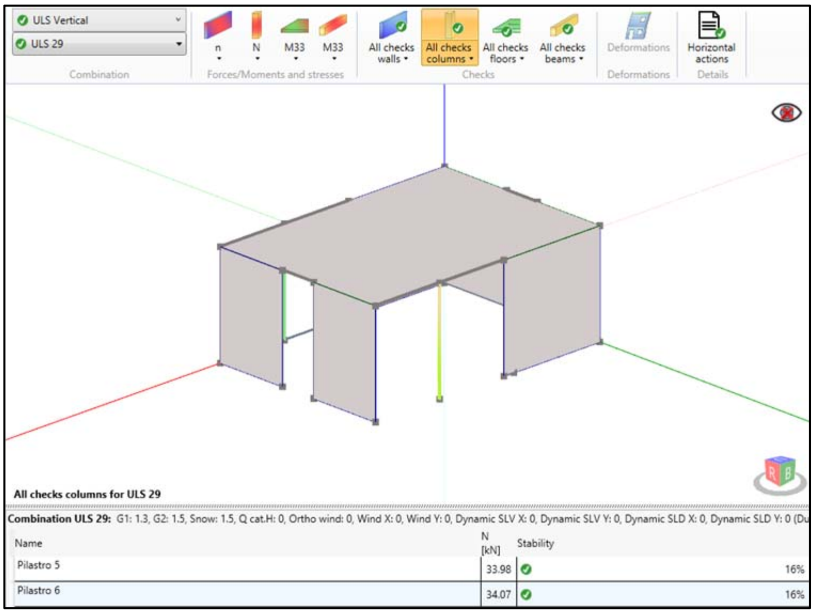
$$N_{b,Rd,z} = 208,8 \; kN$$

The check is satisfied, obtaining:

$$\frac{N_{Ed}}{N_{b,Rd}} = \frac{N_{Ed}}{\min\left(N_{b,Rd,y}, N_{b,Rd,z}\right)} = \frac{34,11 \text{ kN}}{208,8 \; kN} \leq 16 \;\%$$

RESULTS PROVIDED BY THE SOFTWARE

- Axial force N: 34,07 kN
- Stability check: 16 %



RESULTS COMPARISON

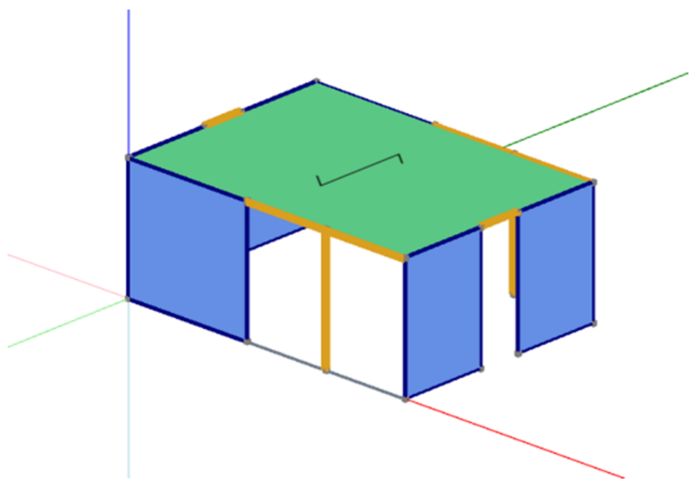
Comparison between the value provided by the software and the value calculated in an independent manner:

Validated parameter	Independent verification	Software
Stability check	16%	16%

CONCLUSIONS

The comparison shows the coincidence between the value provided by the software and that hand-calculated.

EXAMPLE 18: VERTICAL LOAD ON THE WALL



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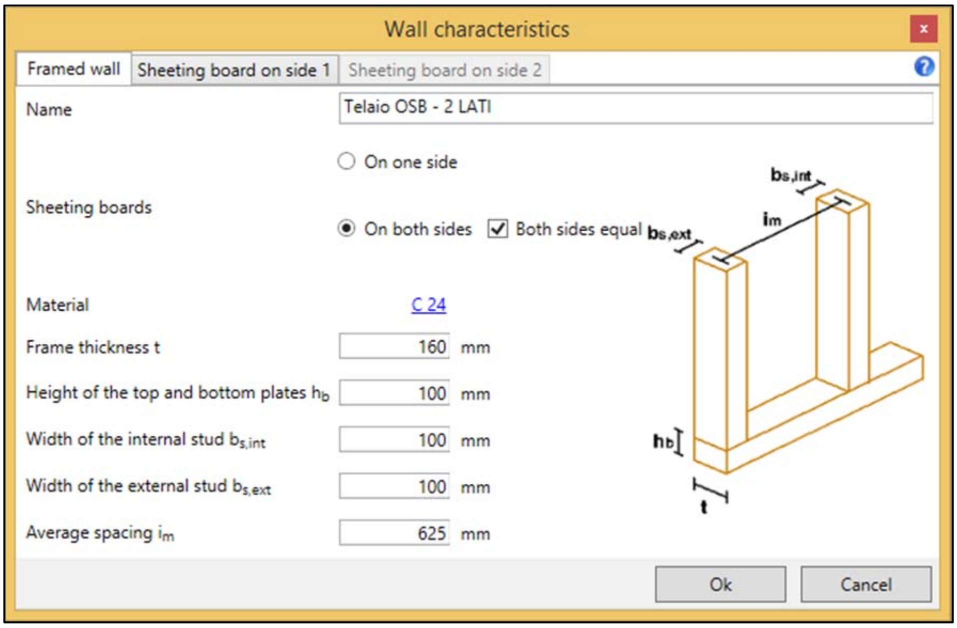
DESCRIPTION OF THE PROBLEM

The calculation of the vertical load is conducted assuming:

- Cross section stud: $b \times h$: 100×160 mm
- Stud spacing: 625 mm
- Material: Solid timber C24
- Heigth of the wall: 2,8 m
- Length of the wall: 3 m
- Wall self weight G_1: 0,46 kN/m^2
- Permanent load G_2: 0,6 kN/m^2
- Distributed load on the top of the wall q_s: 10,61 kN/m
- Beam reaction on the external stud R_b: 8,22 kN (“Configuration 3”)



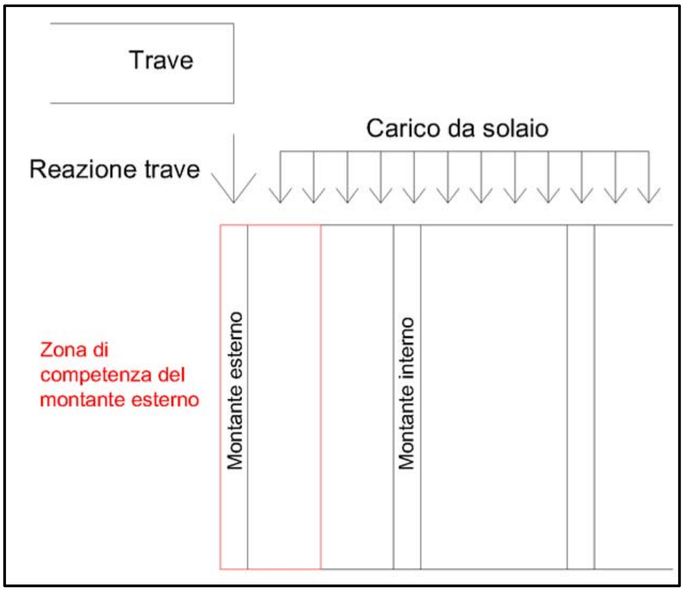
INDEPENDENT VERIFICATION

The distributed load caused by the self-weight and the permanent load is:

$$q_w = (1{,}3 \times 0{,}46 + 1{,}5 \times 0{,}6) \times 2{,}8 \text{ m} = 4{,}19 \text{ kN/m}$$

The external stud takes the beam reaction and the distributed load for a length l_c equal to:

$$l_c = \frac{i}{2} = \frac{625}{2} = 313 \text{ mm}$$



The load per meter length acting at the base of the wall in the external stud influence zone is:

$$q_{bw,1} = q_w + q_f + \frac{R_b}{l_c} = 4{,}19 + 10{,}61 + \frac{8{,}22}{0{,}313} = 41{,}06\ \mathrm{kN/m}$$

The load per meter length acting at the base of the wall out from the external stud influence zone is:

$$q_{bw,2} = q_w + q_f = 4{,}19 + 10{,}61 = 14{,}80\ \mathrm{kN/m}$$

The weighted average of the load *n* is:

$$n = \frac{q_{bw,1} \cdot l_c + q_{bw,2} \cdot (l_w - l_c)}{l_w} = \frac{41{,}06 \times 0{,}313 + 14{,}80 \times 2{,}687}{3} = 17{,}54\ \mathrm{kN/m}$$

The total load of the wall is:

$$N = q_{bw,1} \cdot l_c + q_{bw,2} \cdot (l_p - l_c) = 44{,}60 \times 0{,}313 + 14{,}81 \times 2{,}687 = 52{,}61\ \mathrm{kN}$$

RESULTS PROVIDED BY THE SOFTWARE

- n: 17,52 kN/m
- N: 52,57 kN

n	17.52 kN/m
N	52.57 kN

RESULTS COMPARISON

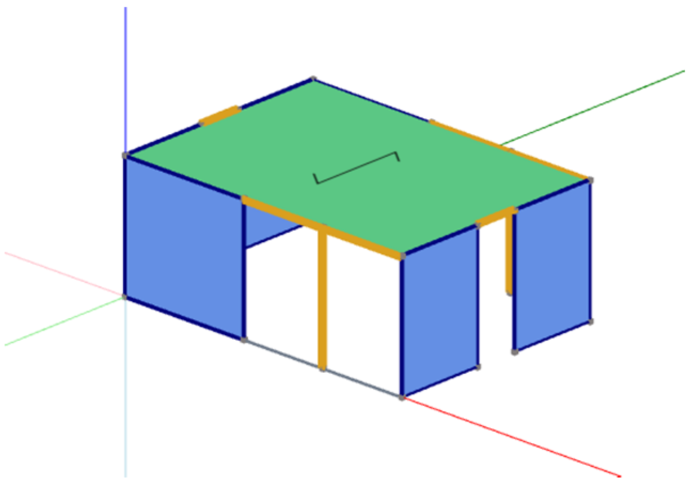
Comparison between the value provided by the software and the value calculated in an independent manner:

Validated parameter	Independent verification	Software	Percentage error
n	17,54 kN/m	17,52 kN/m	−0,1%
N	53,61 kN	52,57 kN	−0,08%

CONCLUSIONS

The comparison shows how results provided by the software are comparable to results obtained with an independent verification. Errors are caused by different approximation between the manual verification and the software.

EXAMPLE 19: WIND LOAD ON THE WALL



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EsempioValidazione.bldg

DESCRIPTION OF THE PROBLEM

The calculation of the wind load is conducted assuming:

- Wall height: h_w: 2,8 m
- Wall length: l_w: 3 m
- Wind secondary load: $\psi_{02} = 0{,}6$;
- Wind load *p* (Windward): $0{,}46\ \mathrm{KN/m^2}$

INDEPENDENT VERIFICATION

The shear force out of plane per meter length at the wall base is:

$$v_3 = p \cdot \gamma_Q \cdot \psi_{02} \cdot \frac{h_w}{2} = 0{,}46 \times 1{,}5 \times 0{,}6 \times \frac{2{,}8}{2} = 0{,}58\ \mathrm{kN/m}$$

The total shear force out of plane is:

$$V_3 = v_3 \cdot l_w = 0{,}58 \times 3 = 1{,}74\ \mathrm{kN}$$

The bending moment per meter length results:

$$m_{2-2} = \frac{p \cdot \gamma_Q \cdot \psi_{02} \cdot h_w^2}{8} = \frac{0{,}46 \times 1{,}5 \times 0{,}6 \times 2{,}8^2}{8} = 0{,}41\ \mathrm{kNm/m}$$

The total bending moment per meter length results:

$$M_{2-2} = m_{2-2} \cdot l_w = 0{,}41 \times 3 = 1{,}23\ \mathrm{kNm}$$

RESULTS PROVIDED BY THE SOFTWARE

- v_3: $0{,}58\ \mathrm{kN/m}$
- V_3: $1{,}75\ \mathrm{kN}$
- m_{22}: $0{,}41\ \mathrm{kNm/m}$
- M_{22}: $1{,}22\ \mathrm{kNm}$

v3 (Out-of-plane)	0.58 kN/m
V3 (out-of-plane)	1.75 kN
m22 (out-of-plane)	0.41 kNm/m
M22 (out-of-plane)	1.22 kNm

RESULTS COMPARISON

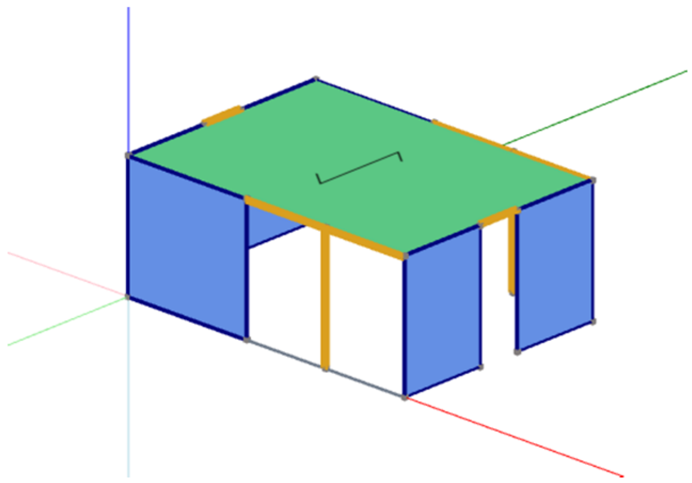
Comparison between the value provided by the software and the value calculated in an independent manner:

Validated parameter	Independent verification	Software	Percentage error
$\boldsymbol{v_3}$	$0{,}58\ \mathrm{kN/m}$	$0{,}58\ \mathrm{kN/m}$	0%
$\boldsymbol{V_3}$	$1{,}75\ \mathrm{kN}$	$1{,}75\ \mathrm{kN}$	0%
$\boldsymbol{m_{22}}$	$0{,}41\ \mathrm{kNm/m}$	$0{,}41\ \mathrm{kNm/m}$	0%
$\boldsymbol{M_{22}}$	$1{,}22\ \mathrm{kNm}$	$1{,}22\ \mathrm{kNm}$	0%

CONCLUSIONS

The comparison shows the coincidence between the value provided by the software and that hand-calculated.

EXAMPLE 20: STABILITY CHECK OF THE WALL



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DESCRIPTION OF THE PROBLEM

The stability of the studs subjected to compression is verified in accordance with § 6.3.2 of EN 1995-1-1. Specifically, the checked elements are the internal and the external studs which are the most loaded. These elements (stud or column in a sheathed wall) are braced against buckling in the in-plane direction therefore checks are performed only in the orthogonal direction. In this example we consider the internal stud and the load combination below (*SLU 33 istantaneous*):

Load	G_1	G_2	Live load cat. H	Snow	Orthogonal wind
Partial factors	1,3	1,5	1,5	0	0,9

Assuming:

- Internal stud cross section of wall 21: $b \times h$: 100×160 mm
- Stud spacing: 625 mm
- Material: Solid timber C24
- Heigth of the wall: 2,8 m
- Length of the wall: 3 m
- Distributed load on the top of the wall q_s: 10,61 kN/m
- Distributed load $q_{bw,2}$: 14,80 kN/m

VALIDAZIONE

The stresses should satisfy the expressions:

$$\frac{\sigma_{c,0,d}}{k_c \cdot f_{c,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} \leq 1$$

where:

- $\sigma_{c,0,d}$ is the design compressive stress along the grain;
- $\sigma_{m,y,d}$ is the design bending stress about the principal axis;
- $f_{c,0,d}$ is the design compressive strength along the grain;
- $f_{m,y,d}$ is the design bending strength about the principal axis;
- k_c is a factor calculated as:

$$k_c = \frac{1}{k + \sqrt{k^2 - \lambda_{rel}^2}}$$

In wich *k* is:

$$k = 0{,}5 \cdot \left(1 + \beta \cdot (\lambda_{rel} - 0{,}3) + \lambda_{rel}^2\right)$$

The total vertical load acting at the base of the internal stud is:

$$N = q_{bw,2} \cdot l_c = 14{,}80 \times 0{,}625 = 9{,}25\ kN$$

The bending moment acting at the wall midpoint is:

$$M = p_v \cdot p \cdot \gamma_Q \cdot \psi_{02} \cdot l_c \cdot \frac{h_p^2}{8} = 0{,}46 \times 1{,}5 \times 0{,}6 \times 0{,}625 \cdot \frac{2{,}8^2}{8} = 0{,}25\ kNm$$

The cross section area of the internal stud is:

$$A = b \cdot h = 0{,}10 \times 0{,}16 = 0{,}016\ m^2$$

The inertia moment in the buckling direction is equal to:

$$J = \frac{b \cdot h}{12} = \frac{0{,}1 \times 0{,}16^3}{12} = 3{,}4133 \times 10^{-5}\ m^4$$

The section modulus is equal to:

$$W = \frac{J}{h/2} = 4{,}2666 \times 10^{-5}\ m^3$$

The gyration radius of inertia *i* is equal to:

$$i = \sqrt{\frac{J}{A}} = 46{,}19\ mm$$

The buckling length results:

$$l_0 = l = 2800\ mm$$

The slenderness of the stud is:

$$\lambda = \frac{l_0}{i} = \frac{2800}{46{,}19} = 60{,}62$$

The relative slenderness is:

$$\lambda_{rel} = \frac{\lambda}{\pi} \cdot \sqrt{\frac{f_{c,0,k}}{E_{0,05}}} = \frac{\lambda}{\pi} \cdot \sqrt{\frac{21}{7400}} = 1{,}028$$

In this case we obtain:

$$k = 0{,}5 \cdot (1 + 0{,}2 \cdot (1{,}028 - 0{,}3) + 1{,}028^2) = 1{,}1$$

$$k_c = \frac{1}{1{,}1 + \sqrt{1{,}1^2 - 1{,}028^2}} = 0{,}67$$

The design stresses are equal to:

$$\sigma_{c,0,d} = \frac{N}{A} = \frac{9{,}25}{0{,}016} \times 10^{-3} = 0{,}58\ \text{MPa}$$

$$\sigma_{m,d} = \frac{M}{W} = \frac{0{,}25}{4{,}2666 \times 10^{-5}} = 0{,}59\ \text{MPa}$$

The design strengths are:

$$f_{c,0,d} = \frac{k_{mod} \cdot f_{c,0,k}}{\gamma_m} = \frac{1 \times 21}{1{,}5} = 14\ \text{MPa}$$

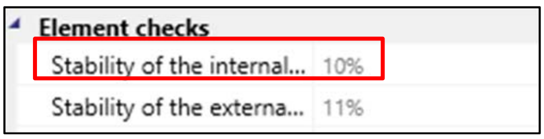
$$f_{m,y,d} = \frac{k_{mod} \cdot f_{c,0,k}}{\gamma_m} = \frac{1 \times 24}{1{,}5} = 16\ \text{MPa}$$

Where the factor k_{mod} is equal to 1 because we consider an instantaneous combination. The stability check is satisfied, obtaining:

$$\frac{\sigma_{c,0,d}}{k_c \cdot f_{c,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} = \frac{0{,}58}{0{,}67 \times 14} + \frac{0{,}59}{16} = 9{,}9\%$$

RESULTS PROVIDED BY THE SOFTWARE

- Internal stud stability check: 10%



RESULTS COMPARISON

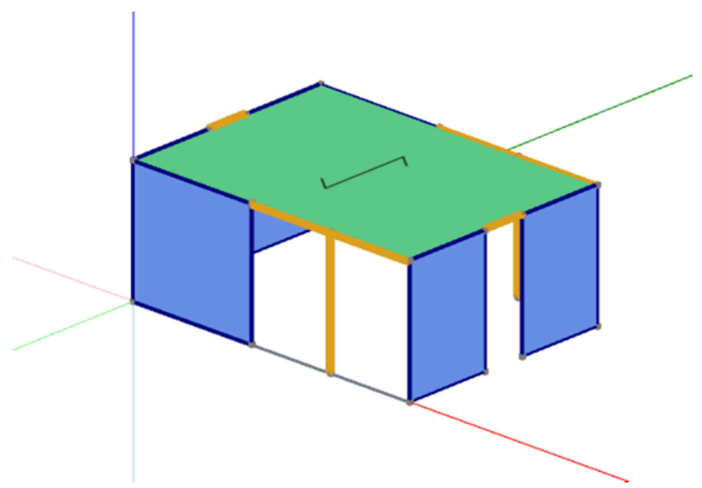
Comparison between the value provided by the software and the value calculated in an independent manner:

Validated parameter	Independent verification	Software
Internal stud stability check	9,9%	10%

CONCLUSIONS

The comparison shows the coincidence between the value provided by the software and that hand-calculated.

EXAMPLE 21: ORTHOGONAL COMPRESSION CHECK



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DESCRIPTION OF THE PROBLEM

The studs are supported at the base by the bottom plate which is stressed by compression forces perpendicular to the grain. The orthogonal compression check is verified in accordance with § 6.1.5 of EN 1995-1-1, assuming:

- Internal stud cross section of wall 21: $b \times h$: $100 \times 160\ \text{mm}$
- Stud spacing: $625\ \text{mm}$
- Material: Solid timber C24
- Distributed load on the top of the wall q_s: $10{,}61\ \text{kN/m}$
- Distributed load $q_{bw,2}$: $14{,}80\ kN/m$

INDEPENDENT VERIFICATION

The following expression shall be satisfied:

$$\sigma_{c,90,d} \leq k_{c,90,d} \cdot f_{c,90,d}$$

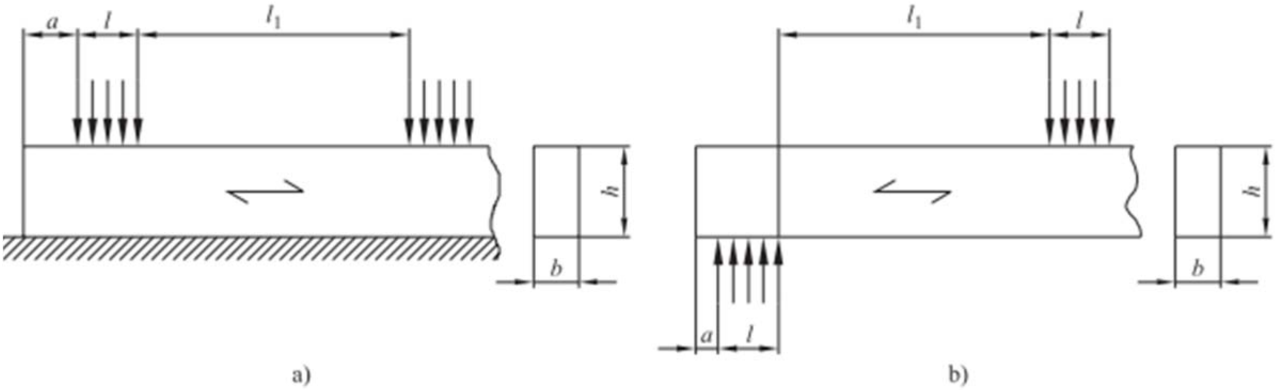
with

$$\sigma_{c,90,d} = \frac{F_{c,90,d}}{A_{ef}}$$

where:

- $\sigma_{c,90,d}$ is the design compressive stress in the effective contact area perpendicular to the grain
- $F_{c,90,d}$ is the design compressive load perpendicular to the grain
- A_{ef} is the effective contact area in compression perpendicular to the grain
- $f_{c,90,d}$ is the design compressive strength perpendicular to the grain
- $k_{c,90,d}$ is a factor taking into account the load configuration, the possibility of splitting and the degree of compressive deformation.

The effective contact area perpendicular to the grain, A_{ef}, should be determined taking into account an effective contact length parallel to the grain, where the actual contact length, *l*, at each side is increased by 30 mm, but not more than $a, l, l_{1/2}$:



The value of $k_{c,90}$ should be taken as 1,0 unless the conditions in the following paragraphs apply. For members on continuous supports, provided that $l_1 \geq 2h$, the value of $k_{c,90}$ should be taken as:

- $k_{c,90} = 1{,}25$ for solid softwood timber;
- $k_{c,90} = 1{,}5$ for glued laminated softwood timber;

The total vertical load on the internal stud is:

$$N = q_{bw2} \cdot l_c = 14{,}80 \times 0{,}625 = 9{,}25 \text{ kN}$$

The effective contact area is:

$$A_{ef} = h \cdot (b + 30 \times 2) = 25600 \text{ mm}^2$$

The stress $\sigma_{c,90,d}$ result:

$$\sigma_{c,90,d} = \frac{F_{c,90,d}}{A_{ef}} = \frac{9{,}25 \times 10^3}{25600} = 0{,}36 \text{ MPa}$$

The design compressive strength perpendicular to the grain $f_{c,90,d}$ is:

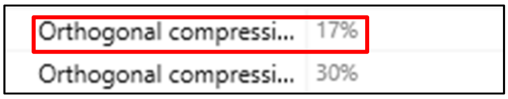
$$f_{c,90,d} = \frac{k_{mod} \cdot f_{c,90,k}}{\gamma_M} = \frac{1 \times 2{,}5}{1{,}5} = 1{,}67\ \text{MPa}$$

The orthogonal compression check is satisfied:

$$\frac{\sigma_{c,90,d}}{k_{c,90,d} \cdot f_{c,90,d}} = \frac{0{,}36}{1{,}25 \cdot 1{,}67} \cdot 100 = 17{,}3\%$$

RESULTS PROVIDED BY THE SOFTWARE

- Orthogonal compression internal stud 17%



RESULTS COMPARISON

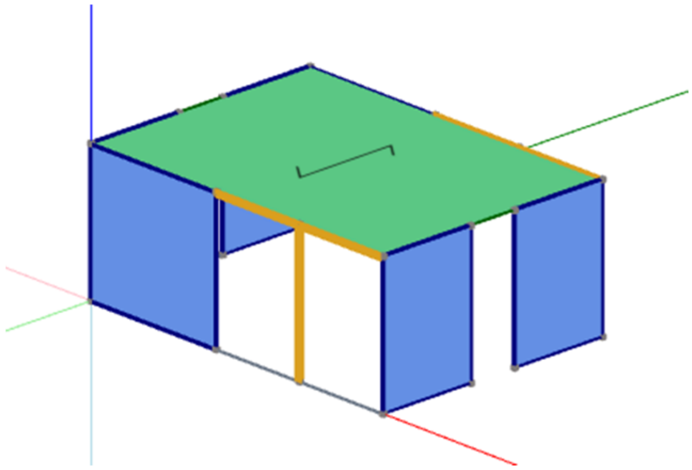
Comparison between the value provided by the software and the value calculated in an independent manner:

Validated parameter	Independent verification	Software
Orthogonal compression internal stud	17,3%	17%

CONCLUSIONS

The comparison shows how results provided by the software are comparable to results obtained with an independent verification. Errors are caused by different approximation between the manual verification and the software.

EXAMPLE 22: CONNECTOR STRENGTH FRAME-PANEL

	**File name:** EsempioValidazione.bldg

DESCRIPTION OF THE PROBLEM

The resistance of each connector is estimated according to the theory of Johansen presented in 8.2.2 EN 1995-1-1 for panel-to-timber connections (Single shear), assuming:

- Connector: Nail with improved adherence RING HH6 2,8/3,1 x 65;
- Panel: OSB/2 with a thickness of 12,5 mm;
- Frame thickness t=160 mm.

INDEPENDENT VERIFICATION

CONNECTOR STRENGTH

The characteristic load-carrying capacity for nails, staples, bolts, dowels and screws per shear plane per fastener, should be taken as the minimum value found from the following expressions:

a) $F_{v,Rk,a} = f_{h,1,k} \cdot t_1 \cdot d$

b) $F_{v,Rk,b} = f_{h,2,k} \cdot t_2 \cdot d$

c) $F_{v,Rk,c} = \frac{f_{h,1,k} \cdot t_1 \cdot d}{1+\beta} \cdot \left[\sqrt{\beta + 2\beta^2 \left[1 + \frac{t_2}{t_1} + \left(\frac{t_2}{t_1}\right)^2\right]} + \beta^3 \left(\frac{t_2}{t_1}\right)^2 - \beta\left(1 + \frac{t_2}{t_1}\right)\right] + \frac{F_{ax,Rk}}{4}$

d) $F_{v,Rk,d} = 1{,}05 \cdot \frac{f_{h,1,k} \cdot t_1 \cdot d}{2+\beta} \cdot \left[\sqrt{2\beta(1+\beta) + \frac{4\beta(2+\beta)M_{y,Rk}}{f_{h,1,k} \cdot d \cdot t_1^2}} - \beta\right] + \frac{F_{ax,Rk}}{4}$

e) $F_{v,Rk,e} = 1{,}05 \cdot \frac{f_{h,1,k} \cdot t_2 \cdot d}{1+2\beta} \cdot \left[\sqrt{2\beta^2(1+\beta) + \frac{4\beta(1+2\beta)M_{y,Rk}}{f_{h,1,k} \cdot d \cdot t_2^2}} - \beta\right] + \frac{F_{ax,Rk}}{4}$

f) $F_{v,Rk,f} = 1{,}15 \cdot \sqrt{\frac{2\beta}{1+\beta}}\sqrt{2 \cdot M_{y,Rk} \cdot f_{h,1,k} \cdot d} + \frac{F_{ax,Rk}}{4}$

with:

$$\beta = \frac{f_{h,2,k}}{f_{h,1,k}}$$

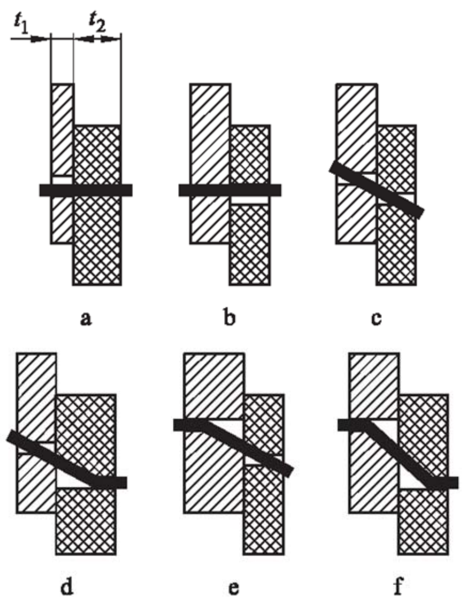
where:

- t_i is the timber or board thickness or penetration depth;
- $f_{h,i,k}$ is the characteristic embedment strength in timber member i;
- d is the fastener diameter;
- $M_{y,Rk}$ is the characteristic fastener yield moment;
- $\frac{F_{ax,Rk}}{4}$ is the contribution from the rope effect. The contribution to the load-carrying capacity due to the rope effect should be limited to 50% of the Johansen part for nail with improved adherence.

In the following table are reported the values used in the calculation of the failure modes.

$f_{h,timber}$ [MPa]	t_{timber} [mm]	t_{pen} [mm]	$f_{h,panel}$ [MPa]	$t_{pannello}$ [mm]	d [mm]	$M_{y,k}$ [Nmm]
21,07	160	52,5	40,70	12,5	2,8	2710

In the figure below there are the failure modes for timber and panel connections.



Characteristic withdrawal capacity

The characteristic withdrawal capacity of nails with improved adherence, $F_{ax,Rk}$ for nailing perpendicular to the grain, should be taken as the smaller of the values found from the following expressions:

$$F_{ax,Rk} = \begin{cases} f_{ax,k}\, d\, t_{pen,frame} \\ f_{head,k}\, d_h^2 \end{cases}$$

where:

$f_{ax,k}$ is the characteristic pointside withdrawal strength;

$f_{head,k}$ is the characteristic headside pull-through strength;

d is the nail diameter;

d_h is the nail head diameter;

$t_{pen,frame}$ is the minimum value between the pointside penetration length and the length of the threaded part in the pointside member.

For threaded nails, the pointside penetration should be at least 6d. For nails with a pointside penetration smaller than 8d the withdrawal capacity should be multiplied by $\frac{t_{pen}}{2d} - 3$.

In the following table are reported the calculations for the characteristic pointside withdrawal strength ($F_{ax,k,point}$) and for the characteristic headside pull-through strength ($F_{ax,k,head}$).

Connector frame-panel	**$f_{ax,k}$ [MPa]**	**d [mm]**	**$t_{pen,frame}$ [mm]**	**$F_{ax,k,point}$ [N]**	**$f_{head,k}$ [MPa]**	**d_h [mm]**	**$F_{ax,k,head}$ [N]**
RING HH6 2,8/3,1 X 65	6,13	2,8	45	772	12,32	4,3	228

Lateral load-carrying capacity

In the table below there are the characteristic load-carryng capacity per shear plane per fastener for each failure mode:

Failure modes	a	b	c	d	e	f
$F_{v,Rk,Johansen}$ (N)	1424	3098	1134	606	1248	747
Rope effect limit (N)	-	-	515	301	570	367
$F_{ax,Rk}/4$ (N)	-	-	57	57	57	57
$F_{v,Rk}$ (N)	1424	3098	1191	663	1305	804

The characteristic load-carryng capacity per shear plane per fastener is:

$$F_{v,Rk} = min\{F_{v,Rk,i}\} = 663 \text{ N}$$

SLIP MODULUS K_{SER}

The slip modulus K_{Ser} per shear plane per fastener for nail with improved adherence is:

$$K_{ser} = \frac{\rho_m^{1,5} \cdot d^{0,8}}{30}$$

- ρ_m is the mean density
- d is the fastener diameter

If the mean densities of the two jointed wood-based members are different then ρ_m in the above expressions should be taken as:

- Panel: $\rho_{m,1} = 650 \text{ kg/m}^2$
- Frame: $\rho_{m,2} = 420 \text{ kg/m}^2$

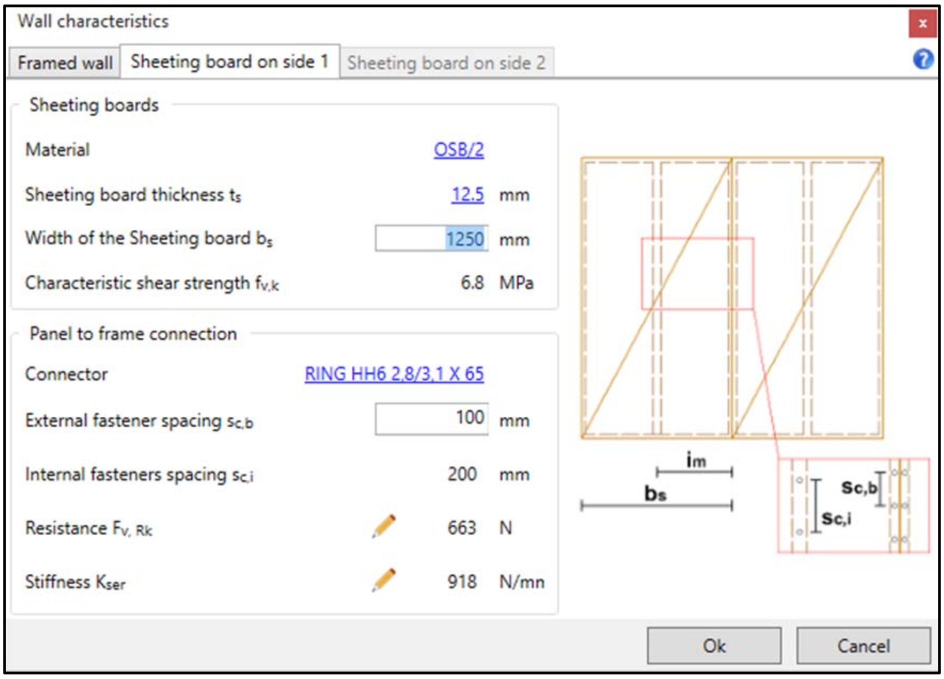
$$\rho_m = \sqrt{\rho_{m,1}\rho_{m,2}} = \sqrt{660 \times 420} = 526 \text{ kg/m}^2$$

The slip modulus K_{Ser} per shear plane per fastener for nail with improved adherence is:

$$K_{ser} = \frac{\rho_m^{1,5} \cdot d^{0,8}}{30} = \frac{522{,}5^{1,5} \times 2{,}8^{0,8}}{30} = 918 \text{ kN/m}$$

RESULTS PROVIDED BY THE SOFTWARE

- $F_{v,Rk} = 663 \text{ N}$
- $K_{ser} = 918 \frac{\text{N}}{\text{mm}}$



RESULTS COMPARISON

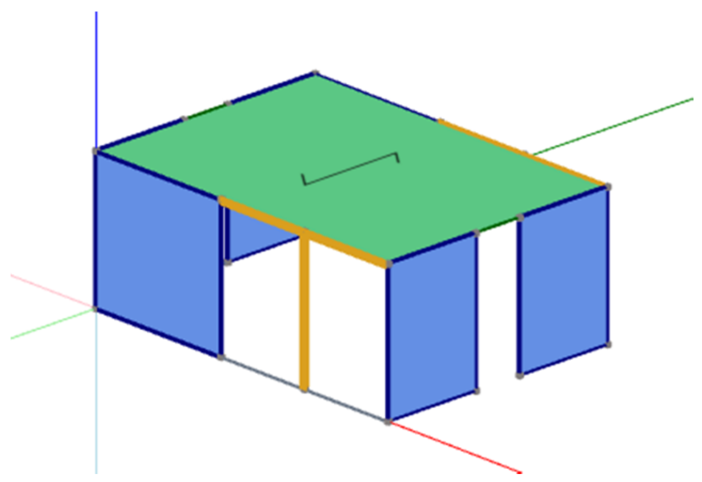
Comparison between the value provided by the software and the value calculated in an independent manner:

Validated parameter	Independent verification	Software	Percentage error
$F_{v,Rk}$	$663\ \mathrm{N}$	$663\ \mathrm{N}$	0%
K_{ser}	$918\ \frac{\mathrm{N}}{\mathrm{mm}}$	$918\ \frac{\mathrm{N}}{\mathrm{mm}}$	0%

CONCLUSIONS

The comparison shows the coincidence between the value provided by the software and that hand-calculated.

EXAMPLE 23: CONNECTOR STRENGTH IN A CLT JOINTED WALL



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DESCRIPTION OF THE PROBLEM

The resistance of each connector is estimated according to the theory of Johansen presented in 8.2.2 EN 1995-1-1 for panel-to-timber connections, assuming:

- CLT panels with a thickness of 120 mm;
- Joint typology:
  - Central board in LVL KertoQ with a thickness of 27 mm
  - Nail with an improved adherence RING HH3 3,8/4,2 x 120;

INDEPENDENT VERIFICATION

CONNECTOR STRENGTH

The characteristic load-carrying capacity for nails, staples, bolts, dowels and screws per shear plane per fastener in double shear, should be taken as the minimum value found from the following expressions:

g) $F_{v,Rk,g} = f_{h,1,k} \cdot t_1 \cdot d$

h) $F_{v,Rk,h} = 0.5 \cdot f_{h,2,k} \cdot t_2 \cdot d$

j) $F_{v,Rk,j} = 1{,}05 \cdot \frac{f_{h,1,k} \cdot t_1 \cdot d}{2+\beta} \cdot \left[\sqrt{2\beta(1+\beta) + \frac{4\beta(2+\beta)M_{y,Rk}}{f_{h,1,k} \cdot d \cdot t_1^2}} - \beta\right] + \frac{F_{ax,Rk}}{4}$

k) $F_{v,Rk,k} = 1{,}15 \cdot \sqrt{\frac{2\beta}{1+\beta}}\sqrt{2 \cdot M_{y,Rk} \cdot f_{h,1,k} \cdot d} + \frac{F_{ax,Rk}}{4}$

with:

$$\beta = \frac{f_{h,2,k}}{f_{h,1,k}}$$

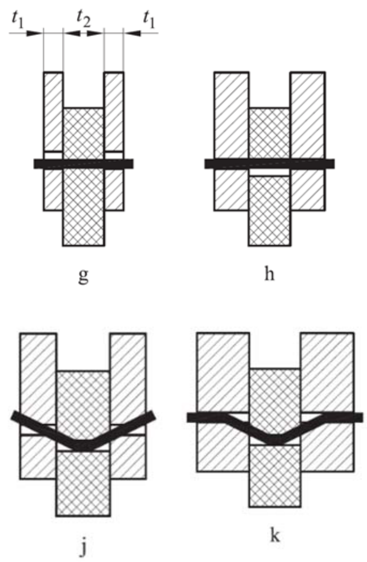
where:

- t_i is the timber or board thickness or penetration depth;
- $f_{h,i,k}$ is the characteristic embedment strength in timber member i;
- d is the fastener diameter;
- $M_{y,Rk}$ is the characteristic fastener yield moment;
- $\frac{F_{ax,Rk}}{4}$ is the contribution from the rope effect. The contribution to the load-carrying capacity due to the rope effect should be limited to 50% of the Johansen part for nail with improved adherence.

In the following table are reported the values used in the calculation of the failure modes.

$f_{h,CLT}$ [MPa]	t_{CLT} [mm]	$t_{pen,CLT}$ [mm]	$f_{h,board}$ [MPa]	t_{board} [mm]	d [mm]	$M_{y,k}$ [Nmm]
19,23	46,5	46,5	26,37	27	3,8	5790

In the figure below there are the failure modes for timber and panel connections.



Characteristic withdrawal capacity

The characteristic withdrawal capacity of nails with improved adherence, $F_{ax,Rk}$ for nailing perpendicular to the grain, should be taken as the smaller of the values found from the following expressions:

$$F_{ax,Rk} = \begin{cases} f_{ax,k}\ d\ t_{pen,frame} \\ f_{head,k}\ d_h^2 \end{cases}$$

where:

$f_{ax,k}$ is the characteristic pointside withdrawal strength;

$f_{head,k}$ is the characteristic headside pull-through strength;

d is the nail diameter;

d_h is the nail head diameter;

$t_{pen,frame}$ is the minimum value between the pointside penetration length and the length of the threaded part in the pointside member.

For threaded nails, the pointside penetration should be at least 6d. For nails with a pointside penetration smaller than 8d the withdrawal capacity should be multiplied by $\frac{t_{pen}}{2d} - 3$.

In the following table are reported the calculations for the characteristic pointside withdrawal strength ($F_{ax,k,point}$) and for the characteristic headside pull-through strength ($F_{ax,k,head}$).

Connector frame-panel	**$f_{ax,k}$ [MPa]**	**d [mm]**	**$t_{pen,punta}$ [mm]**	**$F_{ax,k,punta}$ [N]**	**$f_{head,k}$ [MPa]**	**d_h [mm]**	**$F_{ax,k,testa}$ [N]**
RING HH6 2,8/3,1 X 65	6,13	3,8	46,5	1083	8,58	5,3	241

Lateral load-carrying capacity

In the table below there are the characteristic load-carryng capacity per shear plane per fastener for each failure mode:

Failure modes	g	h	j	k
$F_{v,Rk,Johansen}\ (N)$	3398	1353	1385	1138
Rope effect limit (N)	-	-	693	569
$F_{ax,Rk}/4\ (N)$	-	-	60	60
$F_{v,Rk}\ (N)$	3398	1353	1445	1198

The characteristic load-carryng capacity per double shear plane per fastener is:

$$F_{v,Rk} = 2 \cdot min\{F_{v,Rk,i}\} = 2396\ \text{N}$$

SLIP MODULUS K_{SER}

The slip modulus K_{Ser} per double shear plane per fastener for nail with improved adherence is:

$$K_{ser} = 2 \cdot \frac{\rho_m^{1,5} \cdot d^{0,8}}{30}$$

- ρ_m is the mean density
- d is the fastener diameter

If the mean densities of the two jointed wood-based members are different then ρ_m in the above expressions should be taken as:

- CLT: $\rho_{m,1} = 420 \text{ kg/m}^2$
- Board: $\rho_{m,2} = 510 \text{ kg/m}^2$

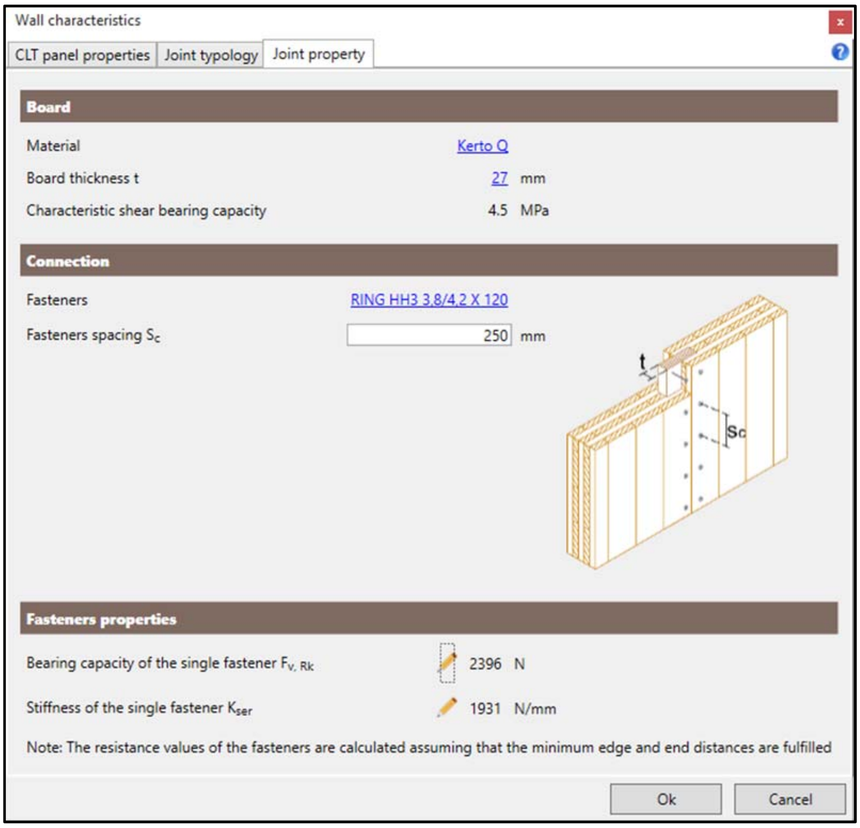
$$\rho_m = \sqrt{\rho_{m,1}\rho_{m,2}} = \sqrt{510 \times 420} = 463 \;\text{ kg/m}^2$$

The slip modulus K_{ser} per double shear plane per fastener for nail with improved adherence is:

$$K_{ser} = 2 \cdot \frac{\rho_m^{1,5} \cdot d^{0,8}}{30} = 2 \cdot \frac{463^{1,5} \times 3{,}8^{0,8}}{30} = 1931 \text{ kN/m}$$

RESULTS PROVIDED BY THE SOFTWARE

- $F_{v,Rk} = 2396 \text{ N}$
- $K_{ser} = 1931 \ \frac{\text{N}}{\text{mm}}$



RESULTS COMPARISON

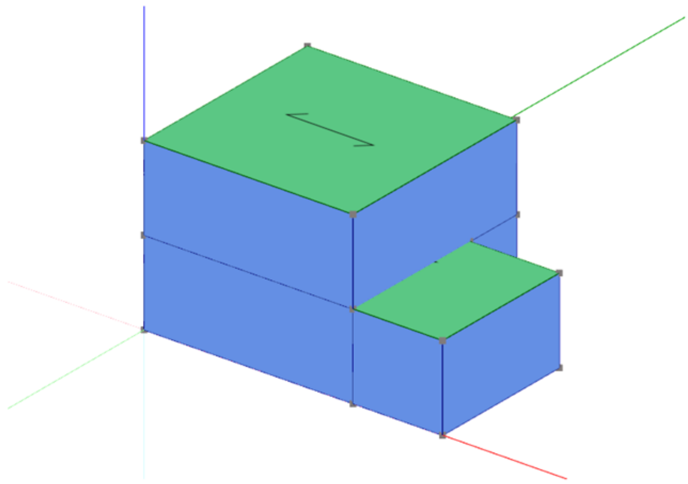
Comparison between the value provided by the software and the value calculated in an independent manner:

Validated parameter	Independent verification	Software	Percentage error
$F_{v,Rk}$	$2396\ \mathrm{N}$	$2396\ \mathrm{N}$	0%
K_{ser}	$1931\ \frac{\mathrm{N}}{\mathrm{mm}}$	$1931\ \frac{\mathrm{N}}{\mathrm{mm}}$	0%

CONCLUSIONS

The comparison shows the coincidence between the value provided by the software and that hand-calculated.

EXAMPLE 24: MODAL ANALYSIS



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DESCRIPTION OF THE PROBLEM

The modal analysis involves the solution of the generalized eigenvalue problem:

$$([\boldsymbol{K}] - \omega^2[\boldsymbol{M}])[\boldsymbol{\Phi}] = 0$$

where:

$[K]$ is the stiffness matrix;

$[M]$ the mass matrix;

ω^2 e $[\Phi]$ are the eigenvalues and the eigenvectors.

For the calculation we consider an irregular building with two levels (figure above).

VERIFICATION WITH SAP2000

In order to validate the solution obtained with the TimberTech Buildings software a modeling in SAP2000 was performed.

It shows a comparison between the periods of the structure and the participating masses along the two main directions "X " and "Y "

Modes	***TimberTech Buildings***			***SAP2000***		
	Periods [s]	***Participating mass X*** [%]	***Participating mass Y*** [%]	**Periods** [s]	***Participating mass X*** [%]	***Participating mass Y*** [%]
1	0,18	2,67	72,35	0,18	2,67	72,35
2	0,18	70,23	2,81	0,18	70,23	2,81
3	0,11	0,01	0,72	0,11	0,01	0,71
4	0,08	0,00	24,06	0,08	0,00	24,06
5	0,08	26,81	0,00	0,08	26,81	0,00
6	0,06	0,28	0,06	0,06	0,28	0,06

CONCLUSIONS

The comparison shows how results provided by the software are comparable to results obtained with a verification with SAP2000.