

Appendix B: Guidance summaries (formulas, assumptions, parameters)

1. Formulae complementing Case study, Subcase 2.

1.1. Load combinations following the provisions of Finnish design guidance RIL 205-1-2009, applicable during the original design phase. (RIL, 2009)

The loading combination used in definition of static equilibrium of structures, such as under wind load, is defined by RIL 205-1, formula 2.1.3S for residential and office buildings under 8 story:

$$F_{v,Ed} = 1,1F_{g,k,sup} - 0,9F_{g,k,inf} + 1,5F_{w,k} + 1,5 * 0,7 * F_{q,k} \quad (\text{f.1}), \text{ with}$$

$F_{w,k}$ representing the wind load (as governing variable force in static equilibrium), and

lateral forces emerging under vertical permanent and variable loads due to geometric eccentricity (evaluated as 1/150), represented as:

$F_{g,k,sup}$	for horizontal component or permanent force added to the calculated design force
$F_{g,k,inf}$	for horizontal component permanent force subtracted from the calculated design force
$F_{q,k}$	for horizontal component of the secondary variable force

1.2. Load combinations according to the original design solution

In the project documentation obtained from municipal archives (Case project documentation, 2015c, 2015b, 2015d), a set of formulae was applied that diverges from the standard load combinations outlined above. These are presented here using notation consistent with formula f.1 for comparison.

For evaluation of horizontal design shear force and compression design force on the trailing edge of the wall:

$$F_{v1,Ed} = 1,15F_{g,k,sup} + 1,5F_{w,k} + 1,5 * 1 * F_{q,k} \quad (\text{f.2})$$

In the formulae used to calculate uplift design force on the leading edge of the wall:

$$F_{v2,Ed} = 0,9F_{g,k,sup} + 1,5F_{w,k} \quad (f.3)$$

2. Formulae complementing Case study, Subcase 2

2.1. Evaluation of wall element stiffnesses according to RIL 205-1,

9.2.4.3S and Gyproc guidelines

Methods 2 and 3 represent derivation of design method developed by (Leskelä, 2005) and widely adopted in Finland.

Method 2. Design method developed by (Leskelä, 2005) is adopted in RIL 205-1 guidance as General design method (9.2.4.3S). General design method (9.2.4.3S) provides evaluation procedure for the wall element stiffnesses (f4a) and in-plane shear strength (f5a) using preset beta and gamma factors. (RIL, 2009)

$$C = 1 / \left(\frac{\beta s H^2}{K B^3} + \frac{H}{B t G} \right) \quad (f.4a)$$

$$R_d = \frac{R_{vd} B}{\gamma_s} \quad (f.5a), \text{ with}$$

H	as the height of the sheathing panel
B	as the width of the sheathing panel
t	as the thickness of the sheathing panel
s	as the fastener spacing
G	as the modulus of rigidity of the sheathing panel
K	as the slip modulus of the sheathing-to-frame connection

According to the method, shear force is distributed between sides and panels withing the wall element is based on relative stiffnesses. Several editions of the RIL 205-1 guidance were reissued between 2007 and 2019 without changes in the relevant parts. Method assumes full rigid tension anchorage at the leading edge of the racking wall.

Method 3. Design method developed by (Leskelä, 2005) is adopted in Gyproc design guidances (Saint-Gobain, 2017, 2022). The guidelines provide both formulae for directly calculated strength and stiffness of the racking wall panels based on fasteners' positions, and precalculated beta and gamma factors.

With the displacement of the fasteners assumed to have component perpendicular to the edge of the panel, (Leskelä, 2005) suggests minimal edge distance of fasteners as $4d$, however Gyproc design guidances allow 10mm edge distance. In this investigation, *Method 3-1* shows calculation, where fasteners are assumed to be positioned along perimeter of the sheathing panel and with uniform spacing. *Method 3-2* demonstrates results based on fasteners assumed to be positioned with 10mm edge distance and interim stud spacing measuring the lesser of 300mm or double the perimeter spacing. Deviations between the directly calculated and tabulated factors are briefly informed in the Table B.1.

Table B.1. Minimum and maximum values of percentage deviance for beta and gamma values of wall panels with dimensions 1200x2400...3200mm and fastener spacing 100...200mm, calculated using alternative instructions provided and referenced in Gyproc certificates

	M.2	M.2	M.2	M.2	Gyproc 2017	Gyproc 2017	Gyproc 2022	Gyproc 2022
	Gyproc 2017	Gyproc 2022	M.3-1	M.3-2	M.3-1	M.3-2	M.3-1	M.3-2
β	0,1...1,6%	3,8...12,8%	-1,4...-7,1%	6,1...15,6%	-2,8...-7,4%	7,3...14,4%	-7,6...-18,9%	-0,6..4,4%
γ	-0,1...1,4%	4,0...12,9%	-1,5...-7,2%	5,6...15,1%	-2,7...-7,1%	7,2...14,1%	-7,9...-19,3%	-1,4..3,8%

Stiffness of the LTF panel then calculated using formula (4b) and shear resistance using formula (5b) (Saint-Gobain, 2017, 2022):

$$C = 1 / \left(\frac{\beta H^2}{K} + \frac{H}{B t G} \right) \quad (\text{f. 4b})$$

$$R_d = \frac{R_{vd}}{\gamma_H} \quad (\text{f.5b}), \text{ with}$$

- H as the height of the sheathing panel
- B as the width of the sheathing panel
- t as the thickness of the sheathing panel
- G as the modulus of rigidity of the sheathing panel
- K as the slip modulus of the sheathing-to-frame connection

Reconstruction of the load distribution presented in (Case project documentation, 2015b; Case project documentation, 2015d) is based on Finnish industry guidelines for concrete structures (Betoniteollisuus ry, 2010), with stiffnesses calculated in *Appendix C, T6 V.1* according to:

$$K_{x/y} = \frac{3 E G I_{x/y} b L}{(G E H^3 + 3 Z E I_{x/y} H)} \quad (\text{f. 4c}), \text{ with}$$

- $K_{x/y}$ as the calculated stiffness of the LTF wall element,
- E taken as 35000000 as per case project documentation,
- G taken as 15000000 as per case project documentation,
- $I_{x/y}$ taken as 100000 as per case project documentation,
- H taken as 6.4 as per case project documentation,
- L as the length of the racking wall in meters,
- b as the width of the racking wall's bottom rail in meters,
- $Z_{x/y}$ taken as 1.2 as per case project documentation.

3. Formulae and overview of guidance informing Case study, Subcase 4.

3.1. General definition of design tensile reaction force at the leading edge of the racking wall

In general form, design tensile reaction force at the leading edge of the racking wall is defined by

$$F_{t,Ed} = F_{v,Ed} * \frac{H}{L} - F_{d,inf} \quad (\text{f.6}), \text{ with}$$

$F_{t,Ed}$ as the design tensile reaction force at the leading edge,

$F_{v,Ed}$ as the design horizontal force,

H as the height of the sheathing panel,

L as the length of the wall segment,

$F_{d,inf}$ as the design vertical force subtracted from the design tensile reaction force at the leading edge of the wall.

While $F_{d,inf}$ can incorporate point load component subjecting directly to the stud at the leading edge of the wall, ambiguity emerges around design guidelines prescribing utilization of the

3.2. Original design solution

Values of the design tensile reaction force at the leading edge, obtained from the project documentation (Case project documentation, 2015b, 2015c, 2015d), correlate with formula 8:

$$F_{t,Ed} = F_{v,Ed} * \frac{H}{0.9L} - 0.5q_{Ed}L \quad (\text{f.7}), \text{ with}$$

$F_{t,Ed}$ as the design tensile reaction force at the leading edge,

$F_{v,Ed}$ as the design horizontal force,

q_{Ed} as the vertical ultimate limit design load subjecting to the load-bearing walls,

a_r as the stud spacing,

H as the height of the racking wall,

L as the length of the racking wall.

Construction drawings *RD2201 Bracing system detailing* (Case project documentation, 2015a) do not show racking walls' connection to the underlying structure, thus suggesting the design intention to implement tension anchorage via the leading edge connections. The construction detail of the floor elements' connection to underlying structure is included under vertical sections ((Case Project Documentation, 2015) HD21 and HD24, see also Figure 5), but evaluation of its tensile capacity is complicated by non-compliance with the edge distances given in the national guidance RIL 205-1 for the inclined screw or nail connections in tension.

3.3. Comparative design methods

3.3.1. Estimation of full rigid tension anchorage at the leading edge of the racking wall, according to RIL 205-1, 9.2.4.2 and 9.2.4.3S

According to the Finnish national guidance RIL 205-1-2009 (s.150), the tension anchoring demand according to Method A (M.1) is based on corresponding instructions in EC5 (SFS-EN 1995-1-1:2004) and provides design method for the racking walls that are stabilized (anchored) against tension either

1. at the ends of the wall elements, so that edge studs are tension anchored to the underlying structure, or
2. at the bottom rail, so that there is at least one anchoring point per each sheathing panel.

Vertical forces can be transferred to the adjacent wall panel or to the structure above or below. When the tensile force is transferred to the structure below, the panel is anchored with rigid connectors. The compressive force of the frame post due to the permanent load may be subtracted from the vertical tension force. Instructions related to the

tension anchoring for the General method (M.2) are identical to the Method A, with the exception of stabilization via bottom rail which is not mentioned.

3.3.2. Complementing calculated examples from UK guidance

Complementing the somewhat ambiguous instructions of the RIL 205-1, UK localisation by (Porteous & Kermani, 2013, pp. 587-595) provides illustrated calculated example to EC 1995-1-1 Method A, showing that a concentrated force equivalent to the value of distributed load over 0.5 width of a single sheathing panel can be subtracted from the tension anchoring (formula 8.1).

$$F_{d,inf_M1_M2} = q_{Ed} \frac{B}{2} \quad (\text{f.8.1}), \text{ with}$$

q_{Ed} as the vertical ultimate limit design load subjecting to the load-bearing walls,

B as the length of the sheathing panel.

(Porteous & Kermani, 2013, p. 560) also state, that the use of plasterboard is not compatible with Method A.

3.3.3. Simplified calculation method according to 2nd generation of Eurocode 5

Simplified design method informed PrEN 1995-1-1 Version 3e (CEN / TC250, 2024)

(M.4) defines the design vertical force at the leading edge of the effective diaphragm as equivalent to the value of distributed load over 0.5 stud span (formula 8.2).

$$F_{d,inf_M4} = q_{Ed} \frac{a_r}{2} \quad (8.2), \text{ with}$$

q_{Ed} as the vertical ultimate limit design load subjecting to the load-bearing walls,

a_r as the stud spacing

3.3.4. Gyproc guidelines

Gyproc design guidelines do not explicitly provide instructions for calculation of the $F_{d,inf}$. References are provided both to RIL 205-1 (assuming formula 7.2) and licentiate

thesis by (Leskelä, 2005) which includes calculated design example where the design vertical force at the leading edge of the effective diaphragm is taken equivalent to the value of distributed load over 0.5 of the wall length (formula 8.3).

$$F_{d,inf_Leskelä} = q_{Ed} \frac{L}{2} \quad (8.3), \text{ with}$$

q_{Ed} as the vertical ultimate limit design load subjecting to the load-bearing walls,

L as the length of the racking wall.

3.3.5. Simplified calculation method according to informative attachment H.4 to 2nd generation of Eurocode 5

Simplified design method (M.5) informed in informative attachment H4 of the PrEN 1995-1-1 Version 3e (CEN / TC250, 2024) provides instructions for calculation of in-plane shear resistance and deflection of the framed wall element with combined anchorage. Unlike any of the previously covered methods, this method explicitly accounts for the distributed load q_{Ed} and tensile capacities of the wall-to-structure connection and tie-down at the leading edge of the racking wall.

3.4. Comparison of design tensile reaction force at the leading edge of the racking wall

Table 10 exemplifies comparison of tie-down capacities calculated based on construction drawings *RD2201 Bracing system detailing* (Case project documentation, 2015a) for walls Y1, Y4 and X8, with estimations of design tensile reaction force according to f.7 (using load combination according to f.3, values are obtained corresponding with the design report) and f.8.2 (using load combination according to f.1)

For the two walls, Y1 and X8 it seems that the capacity of tension anchoring at the leading edge of the wall is insufficient while the Y4 wall demonstrates normal design

situation. Design calculations are available from design documentation only for wall for Y4(Case project documentation, 2015c). Full comparison is provided in Appendix C, spreadsheet T10 and table 10.

4. *Formulae and overview of guidance informing Case study, Subcase 3.*

The load distribution given in project documentation (Case project documentation, 2015b; Case project documentation, 2015d) is reconstructed in *Appendix C, T6 V.1* using element stiffnesses obtained from f.4c, based on Finnish industry guidelines for concrete structures (Betoniteollisuus ry, 2010). Comparative results in the *Appendix C, T6 V.2*, are obtained using element stiffnesses obtained from f.4a (*Appendix C, T6 V.1-V2*) and f.9-11. In both cases, load distribution formula is used as per f.9-11 with principle presented in f.11 applied uniformly in both loading directions.

In the Appendix 1 of Finnish industry guidelines for and timber structures (VTT, 2006)

$$H_{xi} = W_x \left(\frac{b_{xi}}{\sum_j b_{xj}} + \frac{e_y s_{yi} b_{xi}}{\sum_j b_{xj} s_{yj}^2 + \sum_j b_{yj} s_{xj}^2} \right) \quad (\text{f. 9}), \text{ with}$$

H_{xi} as calculated portion of the total horizontal load in the direction x , resisted by the wall i

W_x as the total horizontal load in the direction x (with corresponding racking walls' properties annotated as b_{xi} and their positional coordinates as y_i),

b_{xi} ambiguously (in absence of notation) as either length or stiffness of the wall,

$\sum_j b_{xj}$ ambiguous (in absence of notation) as summa of either length or stiffness of the walls in the direction x ,

$\sum_j b_{yj}$ ambiguous (in absence of notation) as summa of either lengths or stiffness of the walls in the direction y ,

e_y as distance between y -coordinates of center of rotation of the horizontal bracing system, and y -coordinates of the W_x

s_{yi} as defined in f.10 (note $\sum b_{xi}x_i$ is presented as per referenced source):

$$s_{yi} = y_i - \frac{\sum b_{xi}x_i}{\sum b_{xi}} \quad (\text{f. 10}), \text{ and}$$

s_{xi} as defined in f.11:

$$s_{xi} = x_i - \frac{\sum b_{yi}x_i}{\sum b_{yi}} \quad (\text{f. 11}), \text{ with}$$

y_i/x_i as positional coordinates of the racking walls corresponding to wind load directions W_x / W_y

Comparative results in the *Appendix C, T6 V.3, V4 and V5* are obtained assuming floor diaphragm acting as simply supporting beam and using projection areas.

5. References

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