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LOAD LEVELS AND CRITICAL DESIGN ISSUES IN A MULTI-STOREY RESIDENTIAL TIMBER BUILDING BUILT UP BY PREFABRICATED VOLUMETRIC ELEMENTS

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ABSTRACT: The use of structural timber in residential buildings has increased considerably in Sweden during the last decade. Concomitantly, there has been an emphasis on augmenting the level of prefabrication for such structures. A simultaneous need for education imparted in universities exists, and this paper contributes to this need by outlining a design example of a residential timber building with simple geometry using prefabricated volumetric elements. The serviceability limit state as well as the ultimate limit state have been defined and studied for some critical design situations. The results indicate, for example, that the degree of utilisation in the studied building system for compression perpendicular to the bottom rail in the first storey and buckling of the studs are in the same range - 91 % to 97 % - for the walls separating apartments.

KEYWORDS: Design, Education, Prefabrication, Volumetric structure, Module

1 INTRODUCTION

The number of residential buildings with structural timber in their load-bearing frames, has increased considerably in Sweden over time. The primary reason for this is the increase in the population of urban regions in the country, which has led to a rise in demand for housing stock. Another driver, which has been influencing this trend, is the need to truncate the life-cycle greenhouse gas footprint of buildings, and timber-based structures perform well in this regard [1].

A noticeable trend in the last few years is the practice of increasing the level of prefabrication of the building modules. Some advantages of this practice are the increased speed in which the building can be assembled on-site, see Figure 1, the controlled climate in which the modules are produced and not the least the improvement in working environments for labourers at different stages of the construction.

The rapidity with which developments are happening in this sector warrants and motivates the design of new academic course modules in the universities, which both students and professionals opting for competence enhancement opportunities, can avail of [2]. The primary aims of this paper are:

- to present a typical timber structure produced by volumetric modules,
- to calculate a selection of critical design loads acting on the structure and,

__Figure 1:__ Example of installation of a prefabricated timber module in a four-storey building.

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- to discuss some of the challenging design issues related to such a building according to current harmonized design codes [3-5].

The dual motives are to make these results the foundation on which a new academic course can be designed and to facilitate the evolution of numerical models [6, 7], the results from which can be related to load levels obtained.

2 OBJECT DESCRIPTION

The building studied in the case study is a four-storey building designed using prefabricated volumetric elements, so called “modules”, using a light-weigth stud-and-rail system. The building is designed using 10 volumetric elements in width and four volumetric elements in height. The total length, width and height dimensions of the building are 40, 10.5 and 15.5 m respectively and the exterior corridor is 1.5 m wide.

The prefabricated volumetric elements are approximately 4.0 m wide (in the A direction, Figure 2), 9.0 m long (in the B direction, Figure 2) and 3.0 m in height. The long wall elements of the volumetric elements are made of 45 · 95 mm sawn timber with a centre-to-centre distance of typically 600 mm. The bottom rail and top rail is made up of sawn timber with a dimension of 45 · 95 mm. On the inside of the studs, sheets are screwed to the studs giving horizontal stability. The sheet material can be OSB, plywood and/or gypsum wallboards depending on manufacturer. The load bearing part of the gable (short walls) of the volumetric elements is made up of 45 · 170 mm studs with sheet material on the inside screwed to the studs. The floor and ceiling are made up of sawn timber orientated in the short direction of the module with a board glued and screwed to the sawn boards. In the studied version module the floor and ceiling elements are screwed to the inside of the wall elements to minimise the amount of timber loaded perpendicular to the grain.

The northern and eastern facades of the complete building studied are shown in Figure 3 (*a, b*) together with a section of the building (*c*). The direction north, east etc. is used in this paper to ease the understanding of directions.

The roof trusses with a 28° inclination on the top chord to a horizontal axis are oriented with their span in a north-south direction so that self-weight and the snow load, contribute to the total load, only along the edges A_4-B_4 and E_4-F_4, respectively, for each single volume on the top floor respectively, see Figure 4. The northern and southern walls (the shorter walls of the single volume) are exterior walls requiring a greater degree of insulation, vis-à-vis the eastern and western walls (the longer walls of the single volume) for a typical volume inside the building. Owing to the requirement of extra insulation and the fact that the load attributed to the snow will act primarily on those walls, almost a doubling of the height of the cross-sections of their studs, vis-à-vis those in the interior eastern and western walls, is called for. As depicted in Figure 4, the cross-section (width · height) of the latter is typically 45 mm · 95 mm.

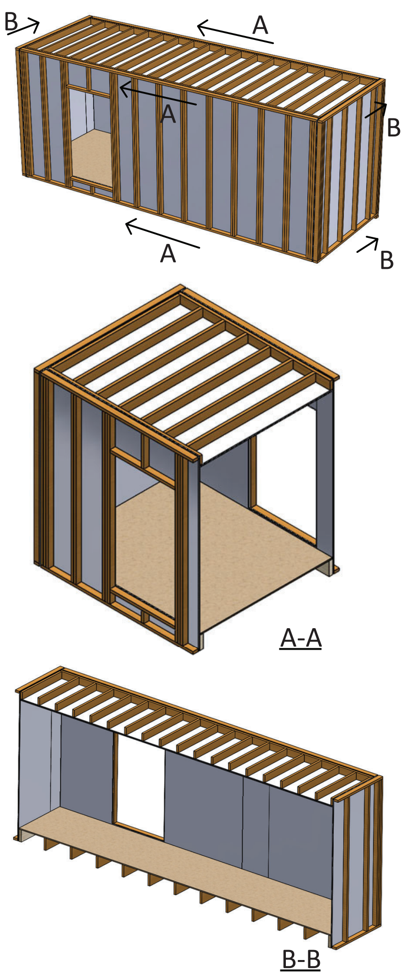

Figure 2: *Illustration of a typical module used for assembling residential buildings.*

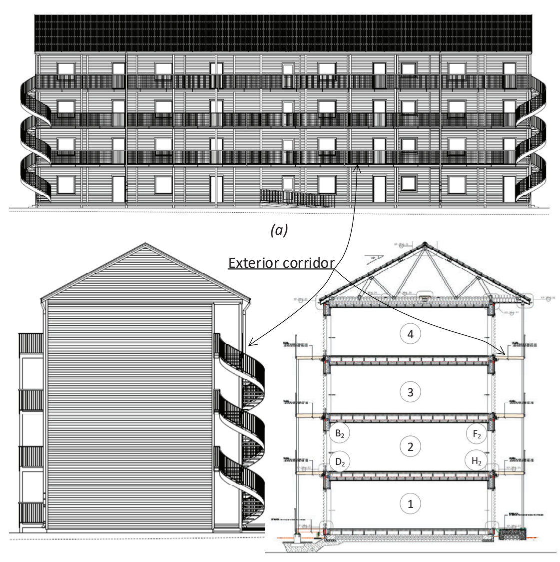

Figure 3: *Illustrations of the studied building: (a) northern façade (b) eastern façade and (c) section of the building.*

The eastern and western walls of the lowest storey are subjected to all the imposed loads in the building by virtue of the orientation of the joists in the flooring for each module. Along the northern façade of the building there is an exterior corridor linking the different apartments on each floor.

How the prefabricated volumetric elements are attached to each other, is a critical design issue, which is complicated by conflicting design requirements. Designing for structural stability calls for strong tie-downs, while designing for good acoustics entails a decrease in number of mechanical connections is desired to reduce the level and transmission of structure-borne noise. In this study, an extreme case devoid of any mechanical connections securing the elements to one another will be considered. The critical issue with load transfer among self-bearing modules will not be discussed.

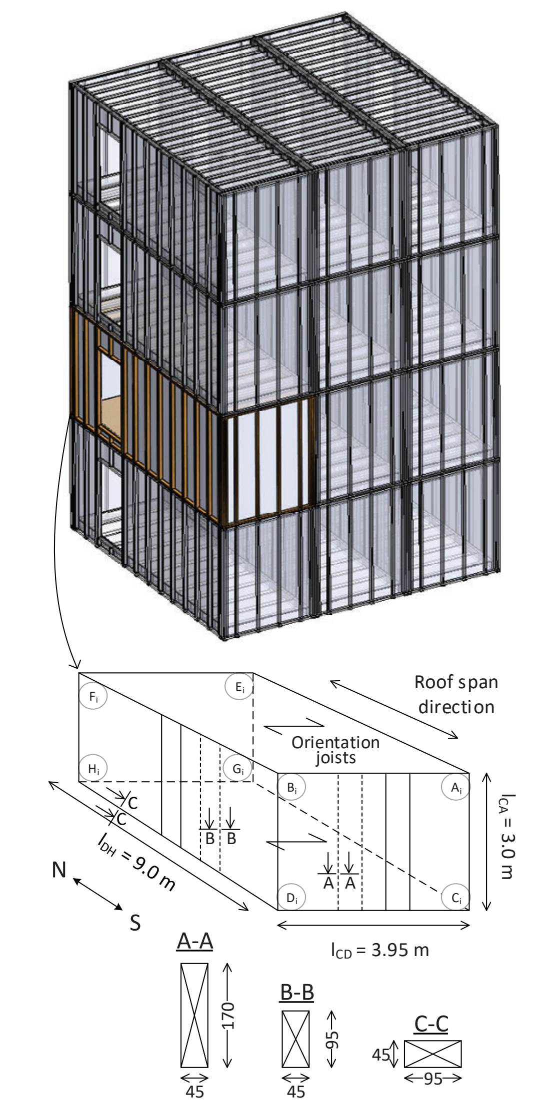

Figure 4: *Example of simplified volumetric element for storey i in the studied structure. Geometrical dimensions as well as the direction of the compass are indicated.*

3 LOAD COMBINATIONS

Design loads acting on a building are calculated in ultimate limit state (ULS) and in serviceability limit state (SLS). For the design cases selected and studied herein the self-weight of the various structural components are unfavourable and thus the load combination for the ULS designed is obtained as

$$E_d = \gamma_d \cdot 0.89 \cdot 1.35 G_k + \gamma_d 1.5 Q_{k,1} + \gamma_d 1.5 \sum_{i>1} \psi_{0,i} Q_{k,i} \tag{1}$$

where E_d is the design load, G_k the self-weight, $Q_{k,1}$ the leading variable action and $Q_{k,i}$ accompanying variable actions acting on the structure being reduced by the reduction factor $\psi_{0,i}$ due to the improbability of simultaneous maximum occurrence of accompanying variable actions. The partial safety factor γ_d is a safety factor defined in the Swedish national annex depending on safety class dependent of the risk of consequences of a failure [5].

In the SLS the characteristic, frequent and quasi-permanent load combinations will be used and they are formulated as

$$E_d = G_k + Q_{k,1} + \sum_{i>1} \psi_{0,i}\, \mathrm{Q_{k,i}} \tag{2}$$

$$E_d = G_k + \psi_{1,1} Q_{k,1} + \sum_{i>1} \psi_{2,i}\, \mathrm{Q_{k,i}} \tag{3}$$

and

$$E_d = G_k + \sum_{i\geq 1} \psi_{2,i} \mathrm{Q_{k,i}} \tag{4}$$

where $\psi_{2,i}\mathrm{Q_{k,i}}$ equates to the average value over a period of time. The characteristic combination (2) is used for calculating the load levels for irreversible permanent actions such as fracture, the frequent combination (3) is used for reversible actions such as vibrations and finally the quasi-permanent combination (4) is used for long term effects, e.g. creep.

The permanent action is considered for the building as a whole. The total self-weight for a single volumetric element of the type used in the studied building ranges between $G_{k.vol}$ = 80 and 90 kN. Permanent loads caused by floor and walls are considered to be distributed loads. The walls range from g_k = 0.35 kN/m^2 for interior load bearing walls to g_k = 0.54 kN/m^2 for exterior load bearing walls and the self-weight of the floor is set to g_k = 1.02 kN/m^2. Beside the permanent action three variable actions add to the total loading of the building; snow load, wind load and imposed load.

The residential building used as an example is assumed to be located in the city of Karlstad (Sweden), where the characteristic snow load is s_k = 2.5 kN/m^2 for the horizontal projection of the area (less than half of extreme values common in some regions of Sweden, reaching 5.5 kN/m^2). The reference wind speed at height z = 16 m for the studied object is v_b = 23 m/s which in terrain category III corresponds to a wind pressure

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$q_p = 0.61$ kN/m^2 (less than half of the maximum wind load of 1.29 kN/m^2 in terrain category 0, in coastal regions; and 64% more than the minimum value of 0.37 kN/m^2 in terrain category IV).

The third substantial variable load is the imposed load (q_k). For a residential building, where the floor area is used as living space, it is a category A load, and thus $q_k = 2.0$ kN/m^2. The imposed loads for the exterior corridors for each floor on the northern façade of the building, and the attic, are set to 3.5 kN/m^2 and 0.5 kN/m^2 respectively. The total imposed load can be multiplied by the reduction factor α_n which in turn is calculated as

$$\alpha_n = \frac{2 + (n - 2)\psi_0}{n} \tag{5}$$

where n denotes the number of storeys in the same load category (>2) above the loaded structural parts, and ψ_0 is a reduction factor.

The horizontal force caused by the initial inclination of interacting structural parts should be considered in the design of the shear walls within one storey. In the structure analysed in this paper, this has been done by calculating a total inclination of such parts consisting of one systematic part, and one random part, so that the total angle $\alpha_{m.d}$ can be determined as

$$\alpha_{m.d} = 0.003 + \frac{0.012}{\sqrt{n_{part}}} \tag{6}$$

Service class 1 is used for all structural parts within the prefabricated volumetric elements, and structural timber of quality C24 is assumed. The partial safety factor is set to $\gamma_d = 1.0$.

4 SELECTED CRITICAL DESIGN CASES

A comprehensive structural design of the building studied, necessitates several calculations. Some of them, being standard and conventional in nature, are not interesting in the context of this paper. Others which are interesting are scrutinized in greater details in this section. The three selected design cases which are related to critical issues the building industry encounters, are illustrated in Figure 4 and discussed thereafter, and degrees of utilisation are determined for them.

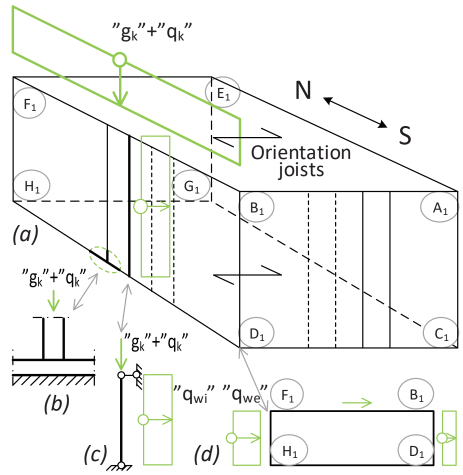

Figure 5: *Illustration of selected design cases a) overview of module in storey 1, b) load perpendicular to the fibres, c) buckling of stud and d) wind load on the structure.*

4.1 Vertical loading in ULS

The design vertical load in the eastern and western walls of the module, see Figure 4, is calculated by including self-weight and the imposed load in the ULS design, equation (1). The combination can be used to calculate the design load for local compression perpendicular to the bottom rail, see Figure 5 (b), and for obtaining the vertical design load in studs in the internal wall which are used in combination with the internal wind load to design for buckling, see Figure 5(c). For the local compression perpendicular to the grains, the design load calculated to $N_{Ec90d} = 17.5$ kN with the imposed action being the leading action. The centre-to-centre distance (cc) of the studs is assumed to be equal to 400 mm. The most critical load in the analysis of buckling of the stud is the one with imposed load as the leading action, and the internal wind load as the accompanying variable action. For that design case, $N_{Ec90d} = 14.1$ kN and $q_{w.int} = 0.033$ kN/m.

In a typical stud in the exterior (northern or southern) wall of the module where a load consisting of snow (leading action), self-weight and imposed loading from the floor- and ceiling elements, and the exterior corridor, is acting, the design load is $N_{Ec90d.N} = 31.9$ kN but due to the smaller cross section in the eastern and western walls focus is put on those walls in the following.

An important detail in the design of compression perpendicular to the fibres is the addition in the EKS11 [5] that allows for assigning the material coefficients $\gamma_M = 1.0$ and $k_{mod} = 1.0$ under specified circumstances. These circumstances are [5] that the "*sole consequence*" of the pressure perpendicular to the grain is *"increased deformations that have no significant impact on the system´s stability and mechanical resistance"*. Cases where this may be applicable include the *"indentation of joists into a sill beam and waling of low buildings*.". Finally, it is also stated that *"where deformations have significant effect on function (e.g. in tall buildings), the recommended partial factors in"* EC5 should be utilized. This implies a necessity for defining whether the building is a tall building or not, *a priori*, which is often challenging. Compared to a situation with $\gamma_M = 1.3$ and $k_{mod} = 0.8$, the reduction in the design material parameter is substantial, 62%, when $\gamma_M = k_{mod} = 1.0$.

4.2 Vertical loading in SLS

Vertical loading leads to instantaneous and time dependent vertical displacements [8]. In a design situation this is particularly interesting, if the elevator shaft is made of another material (concrete, for instance), or if the load bearing structure for the exterior corridor is made of steel, for example, see Figure 3. In such cases, relatively large differences in vertical displacements may be seen; and this may lead to a series of undesired effects.

According to EC5, the loads for design of the long-term effects should be calculated using Equation (4). Quasi-permanent value ($\psi_2 Q_k$) is used for the variable loads so that the load levels are reduced to the long-term average. The variable action to be included in the load case derives purely from the imposed load. The design load on a stud in wall D_1-H_1 at the first storey is calculated to $E_d = 7.9$ kN. Methods are presented in EC5 to take account of the short term (elastic) and long term (creep) effects in the timber material. In addition, the reduction of the moisture content of the timber, from around $mc_{init} = 13\%$ on average, to $mc_{avg\_longterm} = 9\%$ in the long-term results in a reduction in the nominal dimensions of the members, and contributes considerably thereby to the total displacement. Note that only "direct" effects of the *mc* decrease is included here. In the calculations performed herein the total vertical displacement is the sum of the contributions from elastic, time dependent effects, and the shrinkage owing to the change in the moisture content. As far as the physical structure is concerned, deviations from the nominal dimensions such as small length deviations or saw-cuts not perfectly perpendicular to the length direction of the timber member are very likely to play a key role, at least during the initial phase after the installation is completed. However, these effects being difficult to quantify, have not been included in this analysis.

The deformation limit is not defined in EC 5 for example, but should be specified at a project level. For a building assembled using prefabricated modules, a suggested total vertical displacement span of 15 mm < $v_{lim} \leq 20$ mm is recommended. This is based on a rule of thumb that the angle for the exterior corridor should be a minimum of 1:100 so that the floor of the corridor is always inclined away from the building facilitating possible flow of rain water (and meltwater) outwards. The typical width of a corridor lies in the range of 1.5 to 2 metres. For the case-study building with an approximate height $h_{tot} = 15$ m this correlates to $v_{lim.longterm} = h_{tot}/(750 \text{ to } 1000) = 15 – 20$ mm.

4.3 Wind load acting on the northern façade (SLS and ULS)

One of the most challenging loads to design for in prefabricated modules with a load bearing timber structure braced with sheathings mechanically attached to the frame members is the lateral load caused by wind. In the current study, only the effects of the load acting on the northern wall (and suction on the southern) will be analysed. Thus, only the shear walls parallel to F_1-B_1- D_1-H_1 see Figure 5(d), will be subjected to the wind load and therefore analysed. Each "tower" of modules is assumed to stand separate. It follows that 50% of the load on the area B-A-C-D enters along the line A-C, and the other half acts along B-D. The horizontal elements in the ceiling and the flooring are assumed to be stiff, continuous and securely attached to the walls.

In addition to the wind load, the initial inclination of the studs, see equation (6), combined with the acting imposed load and self-weight, contribute to the total horizontal load for any given module. Challenges in ULS arise if the modules are designed with large openings in the walls used as shear walls. Such analyses, however, are straight forward and the load effect can easily be compared with resistance (using method A in EC5).

For the current building, the design horizontal load acting on and in parallel with the eastern wall in the first storey, is $H_{d.w.ULS} = 30.9$ kN of which the contribution due to inclined members is only $H_{d.i.ULS} = 1.4$ kN.

In SLS, there are uncertainties related to the task of manually calculating the horizontal displacement for each storey, as well as for the building as a whole. A recommendation has been given [9] and is formulated as

$$u \approx 4.5\frac{sH_{d.SLS}}{bk} + \frac{H_{d.SLS}h}{G_k bt} \tag{7}$$

where s is the fastener spacing, $H_{d.SLS}$ is the design force (no account of inclined members), b, h and t are the width, height and thickness of the sheet respectively, and G_k is its shear modulus. The largest uncertainty relates to the slip modulus k. Due to the relatively high horizontal load ($H_{d.SLS.char} = 19.7$ kN) in characteristic combination, see equation (2), the secant slip modulus used is that calculated as $k = K_u = 2/3 \cdot K_{ser}$. For the screw with $d = 3.9$ mm in combination with a plywood sheathing material the secant modulus is calculated to $k = 922$ kN/m which is a relatively high value.

The design criteria for the horizontal displacement is the next issue the designer of the building is faced with. Limitations of that displacement are project-specific, though general recommendations suggest that they must be limited – for the building on the whole, as well as for each storey – to $h/500$ [10,11] where h is the total height of the building (when calculated for the building as a whole) or the height of each storey (when calculated for that purpose).

5 RESULTS AND DISCUSSION

The selected critical design cases were:

1. Vertical loading in ULS,
2. Vertical loading in SLS,
3. Wind load acting on the northern façade (SLS and ULS).

As it turns out, the degree of utilisation is relatively high for all the three cases, and they are accounted for, in the sub-sections which follow.

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5.1 Vertical loading in ULS

The degree of utilisation for local compression under each stud along the bottom rail – between points H_1-D_1 and G_1-C_1, is calculated to be 91 %. The corresponding degrees of utilisation for the second, third and fourth storeys are 73 %, 54 % and 32 % respectively. It must be recalled that this applies for the assumed situation with no openings in the long walls on the eastern and western sides of the module.

For the design situation with buckling in the studs around the strong axis, the degree of utilization has been tabulated in Table 1 for two different centre-to-centre distances.

Table 1: *Load levels and degree of utilisation within parenthesis.*

Storey:	1	2	3	4
Local comp., cc 400 mm [kN,(%)]	17.5 (91)	13.9 (73)	10.4 (54)	6.2 (32)
Buckling, cc 400 mm [kN,(%)]	14.1 (97)	10.5 (73)	6.7 (48)	2.5 (20)
Buckling, cc 600 mm [kN,(%)]	21.2 (144)	15.7 (108)	10.1 (70)	3.7 (28)

5.2 Vertical loading in SLS

The storey-wise long-term cumulative vertical displacement is tabulated in Table 2. The total vertical long-term displacement for the top storey thus is calculated to $v_{lim.longterm}$ = 13.7 mm. This corresponds to a value less than the suggested limit of 15 - 20 mm, and thereby the use of $k_{mod} = \gamma_M = 1.0$ for the ULS design of the local compression in section "Vertical loading in ULS" could be adopted here if the suggested methodology is followed. However, since the suggested method is still not common practice the conventional values for k_{mod} and γ_M are used for the design in ULS in the current example. Had the displacement $v_{lim.longterm}$ been greater than the suggested limit, the partial coefficients in the ULS should be set to their actual "ordinary" values.

In Figure 6 the vertical displacements over the building height are illustrated with the long-term component (including immediate elastic displacement), the shrinkage caused by variation in moisture content and the total vertical displacement. Note the considerable contribution of the regions including timber compressed perpendicular to the fibers. It is also worth noticing that the effect of shrinkage due to change in moisture content from 13% to 9% is larger than the effect of the external load acting under long term.

Table 2: *Components contributing to the total vertical long-term displacement for each storey.*

Storey:	1	2	3	4
Elastic [mm]	1.43	1.06	0.68	0.31
Long term [mm]	0.59	0.44	0.30	0.15
Shrinkage [mm]	2.17	2.17	2.17	2.17
Total [mm]	4.19	3.67	3.15	2.63

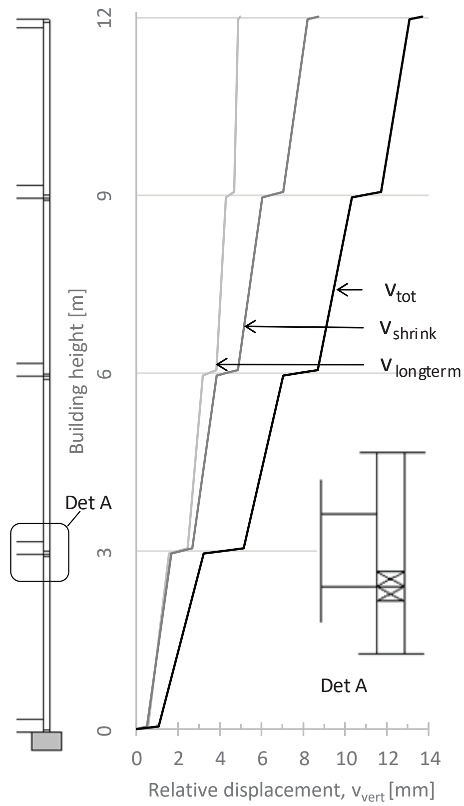

Figure 6: *Relative vertical displacement over the height of the building caused by immediate, long term and shrinkage.*

5.3 Wind load acting on the northern façade (SLS and ULS)

For the first storey, the total horizontal load to be accounted for during the design of the shear walls in ULS, and with wind as the leading action is $H_{d.w}$ = 30.9 kN, which includes both wind load and the horizontal component caused by the initial inclination of the studs, and the vertical load from all the storeys above. Under the assumption of an even load distribution across the top rail, this would correspond to a total load of 3.7 kN/m. The corresponding loads for the 2nd, 3rd and 4th storeys, are tabulated in Table 3. If the SLS is considered, the design loads for characteristic and frequent load combinations are very different from each other. For example, for the first storey, the characteristic combination, Equation (2), gives five times higher load levels compared to the frequent combination, Equation (3). There are reports of repeated cracking of the plasterboard for such buildings, commencing from the interior corners of the doors or window openings. If such cracks are to be regarded as "permanent damage", the horizontal load to be used when calculating the action causing these cracks should be taken from the characteristic load combination. If they instead are regarded as "temporary disturbances" since they normally are not even visible, the load used (frequent combination) for the design is only a fraction of the other. This issue calls for further investigation in future research. For the purpose of this study, conservatively, the

characteristic combination 6.14 is used for calculation of the action in SLS.
The horizontal displacement for each storey is calculated using Equation (7) and used load levels are shown in Table 3, the degree of utilisation shown within parentheses. This gives a total horizontal displacement u_{tot} = 7.0 mm (30%).

Table 3: *Horizontal load ULS and SLS. Degree of utilisation is shown within parentheses.*

Storey:	1	2	3	4
ULS [kN, (%)]	30.9 (100)	23.5 (76)	16.0 (52)	8.5 (27)
SLS, Characteristic [kN, (%)]	19.7 (46)	14.9 (35)	10.2 (24)	5.5 (13)
SLS, Frequent [kN, (%)]	3.9 (9)	3.0 (7)	2.0 (5)	1.1 (3)

6 CONCLUSIONS

In this paper, some critical design issues are identified, selected for further analysis and discussed with respect to load levels and degree of utilisation for a four-storey building using prefabricated volume elements. For the four-storey residential building studied, the degree of utilisation is 97% for the design case with buckling of individual studs in the first storey, and 91% for loading perpendicular to the rail at the same storey, in both cases in the eastern and western walls. It is concluded that this high degree of utilisation indicates the necessity for special attention since minor changes in the design might lead to overutilization.
For the building studied, the calculated maximum long term relative vertical displacement is $v_{lim.longterm}$ = 13.7 mm. In addition to elastic and time-dependent effects the displacement is calculated based on a 4% reduction in moisture content in the timber material. It must be noted that these reductions may be much higher from time to time. Since the design criteria must be project-specific, and very limited guidelines exist, it is, for this building, suggested to limit the displacement based on a minimal inclination of an external corridor with typical width. This results in a limitation being 15 - 20 mm corresponding to h/(750;1000), for the case-study building. It is suggested to assume "no significant impact" for the ULS design as long as this calculated displacement does not exceed h/750 including effects of short- and long term as well as shrinkage. It is concluded that relatively large vertical displacements occur, that limits are set on project basis and that selection of partial coefficients largely influences the analysis in ULS.
The horizontal displacement caused by wind loading for instance, may initiate the formation and propagation of cracks at (and from) the vicinity of the "inner" corners in door openings. Such crack-initiations may occur for low levels of loading, and may not always be visually observed. However, they should be considered as causing permanent damage to the structure. In the current study, the load levels obtained in the characteristic combination (corresponding to permanent damage) is used for obtaining those loads serving as input data for the calculation of horizontal displacement. Also note that there might be other causes for cracks occurring, such as differential vertical displacement.
The use of prefabricated modules in light-weight timber systems is a building system that is common in the Nordic countries as well as in North America. It is, however, a subject rarely covered in the education for structural engineers today. As the building system is frequently utilized it is important that it is also covered in the curriculum for future students and the case study in the current paper is one way to cover the subject. The example include some of the critical aspects when designing buildings using modules such as:

- load paths in both vertical and horizontal directions,
- design calculations for selected elements in ULS and
- critical design calculations for SLS.

7 FURTHER WORK

The current work has proven that there is a high degree of utilisation for some of the selected load cases in the design of residential buildings. Regarding local compression perpendicular to the grain as well as relative vertical displacement, the utilisation may be reduced considerably if the rails are imparted greater stiffness and strength. Waste material from CLT-production is of particular interest in this respect, and will be investigated further in the future.
It will also be interesting to further scrutinise the criteria for long-term vertical displacements and cracking caused by the wind load. These issues are becoming increasingly important due to the tendency to construct taller buildings. As a closing note, it must be mentioned that in spite of the prevailing tradition in Sweden to design timber buildings, the general design knowledge and the comprehension of the challenges associated with buildings constructed using prefabricated elements, leaves a lot to be desired. Load transfer among modules, effects of accidental loads, uplift due to wind load, and other issues related to modules with large openings have to be studied in greater detail in the future.
The example can also be further developed as a design example in education by including parts as design of the floor elements in ULS and SLS as well as horizontal stability and design of connections e.g. between volumes.

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REFERENCES

[1] Dodoo, A. (2019). Lifecycle impacts of structural frame materials for multi-storey building systems. Journal of Sustainable Architecture and Civil Engineering, 24(1), 17-28.
[2] Johansson, M., Mahapatra, K., Vessby, J., Blom, Å., & Olsson, A. (2014). Expert competence for sustainable timber engineering : a master program in

 https://doi.org/10.52202/069179-0578

close cooperation between industry and academia. WCTE 2014 - World Conference on Timber Engineering, Proceedings.

[3] EN-1995-1-1, Eurocode 5: Design of timber structures-part 1-1: General - Common rules and rules for buildings, Brussels (2004).

[4] EN 1991-1-4: (2005). Eurocode 1: Actions on structures - Part 1-4: General actions - Wind actions.

[5] BFS 2011:10 with amendments up to BFS 2019:1 (EKS 11), available at: https://www.boverket.se/globalassets/publikationer/dokument/2019/application-of-the-european-construction-standards---eks-11.pdf

[6] Kuai, L., Ormarsson, S., Vessby, J., & Maharjan, R. (2022). A numerical and experimental investigation of non-linear deformation behaviours in light-frame timber walls. Engineering Structures, 252, 113599.

[7] Ormarsson, S., Vessby, J., Johansson, M., & Kuai, L. (2019). Numerical and experimental study on modular-based timber structures. Modular and Offsite Construction (MOC) Summit Proceedings, 471-478.

[8] Serrano, E., Enquist, B., & Vessby, J. (2014). Long term in-situ measurements of displacement, temperature and relative humidity in a multi storey residential CLT building. In *World Conference on Timber Engineering, Quebuec City, 10-14 August* (pp. 398-405). World Conference on Timber Engineering (WCTE).

[9] Källsner, B., & Girhammar, U. A. (2009). Analysis of fully anchored light-frame timber shear walls—Elastic model. Materials and structures, 42, 301-320.

[10] SBN avd 2A Bärande konstruktioner med kommentarer, Statens Planverks författningssamling 1979:7.

[11] DIN-Taschenbuch 34, Holzbau Normen, Deutsche Institut für Normung e. V., Beuth Verlag GmbH, Deutschland, 1994.