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Seismic resistance of existing buildings with added light timber structure storeys

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Research Paper

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Seismic resistance of existing buildings with added light timber structure storeys

In the presented paper, the problem of vertical expansions is treated in terms of seismic resistance of structures with added storeys. A large parametric study has been performed, confirming the impact of different number of added storeys, and the change of their stiffness, on the seismic response of structures. The paper shows examples of how stiffness in light timber frame and cross laminated timber structures can be easily altered just by changing the type and distribution of fasteners. Known procedures are used to calculate the stiffness of the wall elements of a light timber frame system, and a new procedure is developed for determining the stiffness of cross laminated timber wall elements. The study shows how changing the stiffness of the vertical expansion can have a significant impact on the seismic response and that, in some cases, vertical expansion can have a favourable effect on seismic resistance, despite a minor increase in the mass of the structure.

Key words:
added storeys, seismic resistance, light timber structure

Prethodno priopćenje

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Potresna otpornost postojećih zgrada s dodatnim katovima od laganih drvenih konstrukcija

U ovom se radu problem vertikalne nadogradnje razmatra u okviru analize potresne otpornosti građevina s dodanim katovima. Provedena je opsežna parametarska studija koja potvrđuje utjecaj broja dodanih katova i promjene njihove krutosti na potresni odziv građevina. Primjeri prikazani u radu pokazuju kako se krutost konstrukcija od laganih drvenih okvira i križno lameliranog drva lako može promijeniti mijenjanjem vrste i rasporeda spojnih sredstava. Poznati se postupci primjenjuju za izračunavanje krutosti zidnih elemenata lakog drvenog okvirnog sustava, a novi je postupak razvijen za određivanje krutosti križno lameliranih drvenih zidnih elemenata. U radu je prikazano kako promjena krutosti vertikalne nadogradnje može bitno utjecati na potresni odziv te da u nekim slučajevima takva nadogradnja može povoljno utjecati na potresnu otpornost, i to bez obzira na manje povećanje težine konstrukcije.

Ključne riječi:
dodani katovi, potresna otpornost, laka drvena konstrukcija

1. Introduction

Projects that deal with existing buildings are often encountered in construction practice. These usually deal with renovations, renewals, seismic repairs and upgrades. The lack of new living areas in city centres, the justification of the upgrade price compared to the price of new construction, and the possibility of rehabilitating an existing building through the market value of the upgrade, are the reasons why upgrades of existing buildings are becoming increasingly frequent. In addition to economic reasons, upgrades justify and, above all, enable development of new construction systems that are suitable for upgrades due to their lightness or low weight and speed of construction.

The possibility of upgrading without additional interventions in the existing building is mainly related to original oversizing, or to additional reserves of load-bearing capacity, quality of execution, as well as the damage or condition of the building. From an engineering point of view, the number of new floors depends mainly on structural capacity of the existing structure, as most existing structures were designed according to older regulations, which in comparison to the Eurocode provide lower seismic resistance and less ductile behaviour during earthquakes.

However, when deciding to upgrade, it is also essential to choose the right construction system for the implementation of the upgraded part of the building. Systems that are suitable for upgrades due to their positive properties are characterized by low weight, easy implementation, and the highest possible level of prefabrication in terms of faster construction. In the past, steel structures were considered to be the only suitable superstructure system due to their fast construction and lower mass in comparison to classical construction systems (masonry and reinforced concrete structures). The structural system of steel structures is skeletal, which means that loads are transferred to the existing structure at points that can result in large local loads, which are difficult or even impossible to transfer to the existing structure. In the case of upgrades, it is desirable to use systems that can easily be adapted to architectural requirements and design of an existing building, have a low mass and can transfer their loads in a distributed way. These requirements can fully be met by both systems of modern timber construction: the construction system with cross-laminated timber panels (Xlam) and the timber frame system.

A new segment in the use of timber structures is presented in this paper. This construction system has recently been increasingly used. It opens up a new chapter of hybrid or combined structures, which have been poorly represented so far. Various materials that form composite structures are mostly used at the level of cross-sections (concrete-steel joint, concrete-wood joint) and less often at the level of entire structures, where concrete and steel, or more often, concrete and wood, have been combined in some cases. The so-called hybrid structures are, in most cases, implemented at the level of individual floors, where the concrete core appears as the main stabilizing element, while steel or timber structures are the only elements for transmission of vertical loads.

The paper covers an area of vertical extensions with timber structures, where a certain number of new floors, constructed as a light timber system, are built on an existing reinforced concrete or masonry structure. In general, various studies emphasize the use of various materials with as many identical properties as possible, whereas this article emphasizes the use of materials with different stiffnesses and masses, while also seeking to find the positive effects of different performance.

The investigated impacts mainly deal with cases of steel superstructure [5, 16, 17, 18, 28, 29, 40], where studies find that the upgrade prolongs the natural period of the structure. This usually results in a lower seismic force only at the ground floor, while at the higher existing floors, shear forces and interstorey drifts increase, causing a negative impact of the upgrade which, in many cases, eliminates the possibility of upgrading an existing structure. Due to the lower stiffness of the steel structure, the so-called whip effect can occur, and the upgrade may not be able to meet the criteria of permissible interstorey drifts due to an increased response. In such cases, the upgrade is possible by installing viscous dampers, which consequently reduce shear forces and interstorey drifts in all floors of the structure. A cost-effective method of damping is also the installation of calibrated mass dampers (TMD - tuned mass damper), in which the mass and spring are calibrated in such a way that the natural frequency of the damper is close to the first natural frequency of the building, which means a significant increase in the damping of the oscillation.

The timber frame system and the cross laminated construction system are discussed in this paper as upgrade systems. With both systems, it is possible to easily change the rigidity of the structure by changing the composition of elements and using different fasteners with different arrangement. Since both structural systems have a relatively similar mass, we have found the possible range of variation of the stiffness of both systems and have used these findings for modelling various variants of upgrades.

The purpose of the investigations described in this paper is to determine whether it is possible to ensure seismic safety of both the existing building and the upgrade despite the increase in the number of floors and additional weight. The study involved investigation of the effects of the superstructure on the existing structure and the effects of the response of the existing structure to the superstructure. As systems with significantly different mass and different stiffness along height are used, the effects of changing the stiffness of the superstructure on the response of the entire structure have been investigated. By changing the stiffness and the number of floors of the superstructure, we have checked whether it is possible to exert an influence on the dynamic response of the entire structure, especially the response of the existing structure.

2. Modelling of timber system

As for the structural system of superstructures, it can be a prefabricated structural system in the form of timber frame walls and a structural system with cross-laminated panels. Compared to other timber building systems and other construction systems of other materials where stiffness and load bearing capacity are dictated by cross-section dimensions, the stiffness and partly the load bearing capacity of prefabricated construction systems in the form of timber frame walls and cross-laminated panels can easily be changed while maintaining the same appearance and dimensions of elements. This is made possible by fasteners (screws, nails, shear brackets and hold-downs), which largely define the behaviour of both timber construction systems. Fasteners represent approximately 2-4 % of the value of the execution of the entire timber structure, which represents a negligible cost compared to the investment in the structure.

In the presented study, a possible variation of stiffness was performed on the example of a 3.4 m long and 2.8 m high wall element that was the same for both Xlam (cross laminated timber) and timber frame construction system. The study covered one to three floors of the superstructure.

2.1. Timber frame system

In the case of the timber frame structure, the stiffness of the structure was varied using hold-downs, a number of shear brackets, by varying the spacing at the connection between the timber frame and the sheathing panel (OSB), and by changing the sheathing panel thickness and single and double sheathing. The wall elements were made of timber frame elements measuring 8/10 cm, C 24, where the studs were made on a grid of 62.5 cm, and the timber frame was sheathed with single-sided OSB board 12 mm and 15 mm thick. In the case of TN_1, the wall was anchored with only 4 ABR 90-type angles from Simpson Strong-Tie, which were positioned at every second stud. Type TN_2 was anchored with shear brackets only at the intermediate distance between two studs, and the edge studs were anchored with hold-downs KR 285 manufactured by Simpson Strong-Tie. The anchoring done for TN_2 with 4 ABR 90 brackets was also performed in the case of TN_3. In all cases, staples such as Haubold 1.53 / 11/55 mm with f_u = 880 MPa were used, which in cases of TN_1, TN_2 and TN_3 were installed at a distance of 7.5 cm around the perimeter of the plate and at distance of 5 cm in cases of TN_4, TN_5 and TN_6. A 12 mm thick OSB board was used in cases of TN_1 to TN_4, while the remaining elements were sheathed with a 15 mm thick OSB board, whereas the sheathing in the case of TN_6 was performed on both sides.

The displacement of the wall element Δ is thus the sum of the individual displacements resulting from the deformation of the fasteners Δsh (staples) at the connection of between the timber frame and the sheathing, the rotation of the wall as a rigid body (activation of hold-downs), Δh, the sliding of the wall as a rigid body (deformation of shear brackets), and the shear deformations of the sheathing plates Δp. The principles offered in the literature [3] were used in this paper to calculate different stiffnesses of wall elements. The results are shown in the table below [3]. The parameters in the following equations have the following meaning: F_h – horizontal force, H – height of the wall, k_c – stiffness of the fastener in the frame to sheathing connection, x_i and y_i – horizontal and vertical distance of the fasteners from the centre of the sheathing, L – wall length, q_v – distributed vertical load of the wall, k_d – stiffness of hold-down in tension, $k_{s,I}$ – sliding stiffness of shear bracket, G_p – shear stiffness of sheathing, n – number of sheathed sides of the wall, t_p – thickness of the sheathing.

$$\Delta = \Delta s\square + \Delta\square + \Delta a + \Delta p \tag{1}$$

$$\Delta sh = \frac{F_h \cdot H^2}{k_c} \cdot \left[\frac{1}{\sum_{i=1}^{n} x_i^2} + \frac{1}{\sum_{i=1}^{n} y_i^2} \right] \tag{2}$$

$$\Delta_h = \begin{cases} \frac{H}{L \cdot k_d} \cdot \left(\frac{F_h \cdot H}{L} \right); q_v = 0 \\ \frac{H}{L} \cdot \left(\frac{F_h \cdot H}{L} - \frac{q_v \cdot L}{2} \right) \cdot \frac{1}{k_d}; 0 < q_v < \frac{F_H \cdot H}{2 \cdot L^2} \\ 0; q_v \geq \frac{F_H \cdot H}{2 \cdot L^2} \end{cases} \tag{3}$$

$$\Delta a = \frac{F_h}{\sum_{i=1}^{n} k_{s,i}} \tag{4}$$

$$\Delta p = \frac{F_h \cdot h}{G_p \cdot n \cdot t_p \cdot L} \tag{5}$$

The total stiffness of the wall element can be written as follows:

$$\frac{1}{K_{tot}} = \frac{1}{K_{sh}} + \frac{1}{K_p} + \frac{1}{K_a} + \frac{1}{K_h} \tag{6}$$

The displacement of the wall element can be written as follows:

$$\Delta = \frac{F_H}{K_{tot}} \tag{7}$$

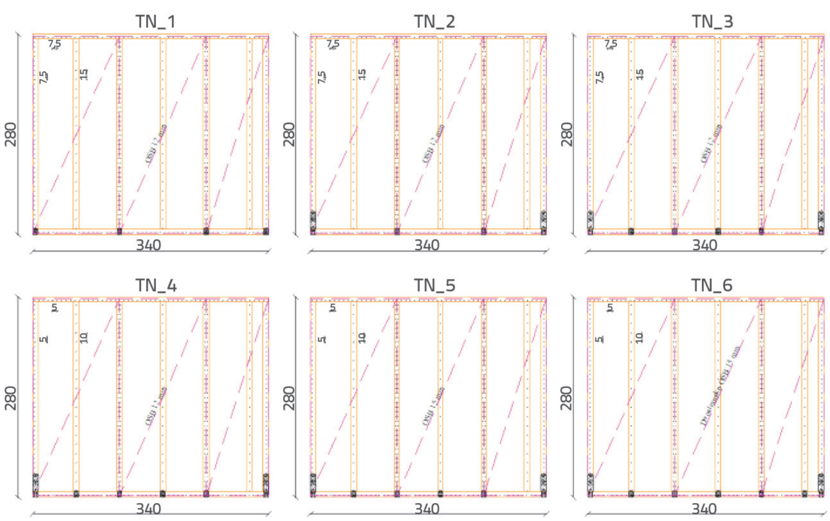

Figure 1. Variation of timber frame shear walls

Table 1. Calculation of timber frame wall sample deformations and stiffness

Deformation of fasteners (frame of cladding)												Wall rotation				Horizontal wall slip				Deformation of cladding				Sum	
Specimen		F [kN]	q [kN/m]	n_{bs}	s_c [cm]	K_c [kN/cm]	α	η	ζ	Δsh [cm]	k_{sh} [kN/cm]	k_h [kN/cm]	r_{HD} [cm]	Δh	k_h [kN/cm]	n_{sk} [kN/cm]	k_{sk} [kN/cm]	Δk_{sk}	K_{sk} [kN/cm]	G_p [kN/cm]	t_p [cm]	Δh_p [cm]	K_p [kN/cm]	Δtot [cm]	K_{tot} [kN/cm]
3_EN	TN_1	27.5	36.3	1.0	7.5	4.2	0.8	0.6	0.5	0.4	104.1	16.0	332.0	0.0	/	4.0	16.0	0.4	93.1	50.0	1.2	0.4	106.0	1.2	23.1
	TN_2	27.5	36.3	1.0	7.5	4.2	0.8	0.6	0.5	0.4	71.5	32.0	332.0	0.0	/	2.0	16.0	0.9	46.5	50.0	1.2	0.4	106.0	1.6	17.0
	TN_3	27.5	36.3	1.0	7.5	4.2	0.8	0.6	0.5	0.4	71.5	32.0	332.0	0.0	/	4.0	16.0	0.4	93.1	50.0	1.2	0.4	106.0	1.2	23.1
	TN_4	27.5	36.3	1.0	5.0	4.2	0.8	0.6	0.5	0.3	107.3	32.0	332.0	0.0	/	4.0	16.0	0.4	93.1	50.0	1.2	0.4	106.0	1.1	25.9
	TN_5	27.5	36.3	1.0	5.0	4.2	0.8	0.6	0.5	0.3	107.3	32.0	332.0	0.0	/	4.0	16.0	0.4	93.1	50.0	1.5	0.3	132.5	1.0	27.8
	TN_6	27.5	36.3	2.0	5.0	4.2	0.8	0.6	0.5	0.3	107.3	32.0	332.0	0.0	/	5.0	16.0	0.3	116.4	50.0	1.5	0.2	264.9	0.8	36.6
2_EN	TN_1	37.5	18.8	1.0	7.5	4.2	0.8	0.6	0.5	0.5	76.3	16.0	332.0	0.0	/	4.0	16.0	0.6	68.3	50.0	1.2	0.5	77.7	1.6	23.1
	TN_2	37.5	18.8	1.0	7.5	4.2	0.8	0.6	0.5	0.5	71.5	32.0	332.0	0.0	/	2.0	16.0	1.2	34.1	50.0	1.2	0.5	77.7	2.2	17.0
	TN_3	37.5	18.8	1.0	7.5	4.2	0.8	0.6	0.5	0.5	71.5	32.0	332.0	0.0	/	4.0	16.0	0.6	68.3	50.0	1.2	0.5	77.7	1.6	23.1
	TN_4	37.5	18.8	1.0	5.0	4.2	0.8	0.6	0.5	0.3	107.3	32.0	332.0	0.0	/	4.0	16.0	0.6	68.3	50.0	1.2	0.5	77.7	1.5	25.9
	TN_5	37.5	18.8	1.0	5.0	4.2	0.8	0.6	0.5	0.3	107.3	32.0	332.0	0.0	/	4.0	16.0	0.6	68.3	50.0	1.5	0.4	97.1	1.3	27.8
	TN_6	37.5	18.8	2.0	5.0	4.2	0.8	0.6	0.5	0.3	107.3	32.0	332.0	0.0	/	5.0	16.0	0.5	85.3	50.0	1.5	0.2	194.3	1.0	36.6
1_EN	TN_1	8.0	8.3	1.0	7.5	4.2	0.8	0.6	0.5	0.1	357.7	16.0	332.0	0.0	/	4.0	16.0	0.1	320.0	50.0	1.2	0.1	364.3	0.3	23.1
	TN_2	8.0	8.3	1.0	7.5	4.2	0.8	0.6	0.5	0.1	71.5	32.0	332.0	0.0	/	2.0	16.0	0.3	160.0	50.0	1.2	0.1	364.3	0.5	17.0
	TN_3	8.0	8.3	1.0	7.5	4.2	0.8	0.6	0.5	0.1	71.5	32.0	332.0	0.0	/	4.0	16.0	0.1	320.0	50.0	1.2	0.1	364.3	0.3	23.1
	TN_4	8.0	8.3	1.0	5.0	4.2	0.8	0.6	0.5	0.1	107.3	32.0	332.0	0.0	/	4.0	16.0	0.1	320.0	50.0	1.2	0.1	364.3	0.3	25.9
	TN_5	8.0	8.3	1.0	5.0	4.2	0.8	0.6	0.5	0.1	107.3	32.0	332.0	0.0	/	4.0	16.0	0.1	320.0	50.0	1.5	0.1	455.4	0.3	27.8
	TN_6	8.0	8.3	2.0	5.0	4.2	0.8	0.6	0.5	0.1	107.3	32.0	332.0	0.0	/	5.0	16.0	0.1	400.0	50.0	1.5	0.0	910.7	0.2	36.6

Table 2. Shear load bearing capacity of timber frame wall samples

Shear wall capacity (Method A. EN 1995-1)								Shear capacity due to shear brackets	Shear capacity due to hold downs	Shear capacity
Specimens		Distance of fasteners [cm]	No. Of cladded sides	Cladding thicknesse [mm]	Fv, Rk [kN]	Fv, Rd [kN] (I)	Fv, Rd [kN/m'] (I)	Fv, Rd [kN] (I)	Fv, Rd [kN] (I)	Fv, Rd_min [kN] (I)
3_EN	TN_1	7.5	1	12	29.4	32.3	9.5	26.0	82.0	9.5
	TN_2	7.5	1	12	29.4	32.3	9.5	21.8	101.5	9.5
	TN_3	7.5	1	12	29.4	32.3	9.5	34.8	101.5	9.5
	TN_4	5.0	1	12	44.1	48.5	14.3	34.8	101.5	14.3
	TN_5	5.0	1	15	44.3	48.7	14.3	34.8	120.3	14.3
	TN_6	5.0	2	15	88.5	97.4	28.6	41.3	120.4	28.6
2_EN	TN_1	7.5	1	12	29.4	32.3	9.5	26.0	55.8	9.5
	TN_2	7.5	1	12	29.4	32.3	9.5	21.8	75.4	9.5
	TN_3	7.5	1	12	29.4	32.3	9.5	34.8	94.1	9.5
	TN_4	5.0	1	12	44.1	48.5	14.3	34.8	94.1	14.3
	TN_5	5.0	1	15	44.3	48.7	14.3	34.8	94.1	14.3
	TN_6	5.0	2	15	88.5	97.4	28.6	41.3	94.2	28.6
1_EN	TN_1	7.5	1	12	29.4	32.3	9.5	26.0	27.8	9.5
	TN_2	7.5	1	12	29.4	32.3	9.5	21.8	47.3	9.5
	TN_3	7.5	1	12	29.4	32.3	9.5	34.8	66.1	9.5
	TN_4	5.0	1	12	44.1	48.5	14.3	34.8	66.1	14.3
	TN_5	5.0	1	15	44.3	48.7	14.3	34.8	66.1	14.3
	TN_6	5.0	2	15	88.5	97.4	28.6	41.3	66.2	28.6

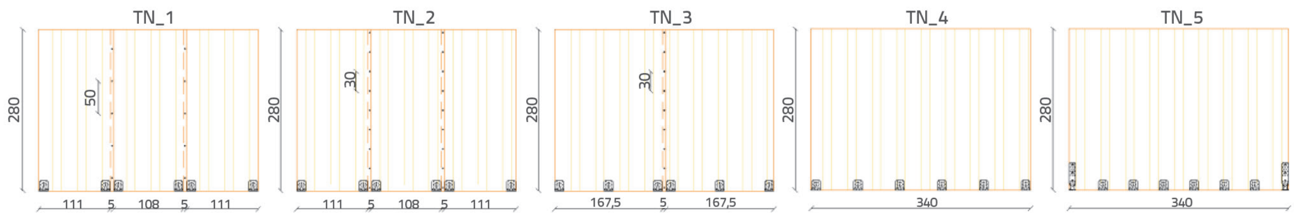

Figure 2. Variation of cross laminated timber shear walls

Wall segments can thus be modelled by substitute diagonals of appropriate stiffness, but it is necessary to be aware that the stiffness of the wall segment (activation of the hold-downs) is also influenced by the vertical load and the horizontal load themselves. The load-bearing capacity of the timber frame walls was calculated in accordance with EC 5 and method A (Table 3), while the load-bearing capacity of the wall anchoring was determined in accordance with the load-bearing capacity of the angles (ETA-07/0285).

Table 3 show the stiffness and the load-bearing capacity of the considered timber frame wall segments, pointing out that the range of stiffness between the wall assemblies TN_1 and TN_6 is between 17.0 kN / cm and 36.6 kN / cm and that the load-bearing range is between 9.5 kN and 28.6 kN in the case of an accidental load combination. With approximately 2-times variation in stiffness of the wall segments, their load-bearing capacity varies by approximately 3-times. The relevant criterion in all cases was the mechanism of the fasteners at the connection between the timber frame and the sheathing panels, while the load-bearing capacity of the brackets in shear and tension was higher than the load-bearing capacity of the wall itself. It is necessary to be aware that, in reality, the friction that occurs between the wall element and the floor structure also has a certain influence on the ductility, but this influence was neglected in the presented calculations. The same applies to the vertical load, which can either prevent or reduce the formation of lifts at the corners of the wall element. In the presented case, the vertical load was large enough so that the hold-downs were not activated at all.

2.2. Cross-laminated timber

In the case of the cross-laminated timber (Xlam) structure or the Xlam wall segment, the rigidity of the structure was varied using hold-downs, a number of overlapping joints in the wall segment and a number of screws in the overlapping joints [2]. In all cases, the wall segments were made of CLT C3s 100 mm in thickness, with external 3 cm thick vertical layers and a 4 cm thick central horizontal layer, and were anchored with brackets type ABR 105 manufactured by Simpson Strong-Tie, while in the case of the element TN_5, additionally with hold-downs type KR 285.

Examples TN_1, TN_2 and TN_3 consisted of two or three wall segments, where the overlapping joint in the cases TN_2 and TN_3 was screwed with self-tapping screws HBS 8x80 mm at a distance of 30 cm and at a distance of 50 cm in the case of TN_1.

The wall segments were modelled as orthotropic with the SAP 2000 program (figures 4 and 5) using stiffness reduction coefficients [2]. Similar to the timber frame system, the displacement of the wall element consisted of displacement due to CLT wall deformation, displacement due to shear deformation in brackets, displacement due to wall rotation or deformation of hold-downs in tension and deformations in overlapping joints of successive wall segments. The properties of Xlam are very well presented in [26].

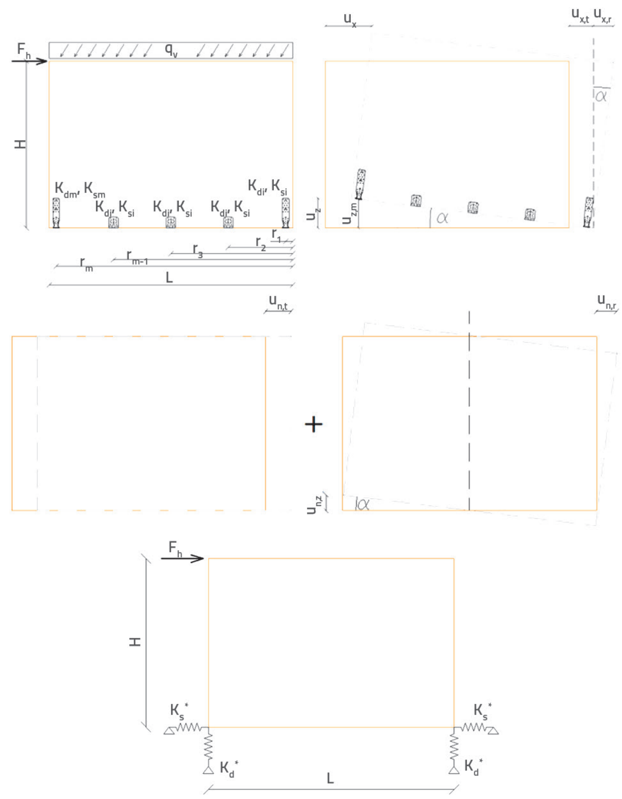

Figure 3. Calculation model

The responses of the screws and shear brackets are symmetrical regardless of the horizontal direction of loading, and the behaviour of the hold-downs in tension is markedly asymmetric at rocking mechanism because, in the case of lifting of the wall segment on one side, the hold-downs in tension were activated, while on the other side of the wall element, pressure forces occurred, in which case, the loads were transmitted through the contact. The latter, however, poses a problem in modelling

Table 3. Calculated stiffness of replacement springs and CLT samples

Load							Shear bracket		Hold down		Stiffness of replacement springs			
Specimen		Lw [m]	Fh [kN]	q [kN/m]	Fdv [kN]	Q [kN]	Ks [kN/m]	Kdv [kN/m]	Ks [kN/m]	Kdv [kN/m]	Ks* [kN/m]	Kd* [kN/m]	Horizontal displacement [cm]	K [kN/cm]
3_EN	TN_1	1.13	18.3	36.4	31.7	41.1	2000.0	2000.0	2000.0	2000.0	2000.0	6931.91	3.81	21.0
	TN_2	1.13	18.3	36.4	31.7	41.1	2000.0	2000.0	2000.0	2000.0	2000.0	6931.91	3.24	24.7
	TN_3	1.70	27.5	36.4	29.9	61.8	2000.0	2000.0	2000.0	2000.0	3000.0	13775.09	2.45	32.7
	TN_4	3.40	55.0	36.4	-45.2	123.7	2000.0	2000.0	2000.0	2000.0	6000.0	/	0.99	80.8
	TN_5	3.40	55.0	36.4	-45.2	123.7	2000.0	2000.0	0.0	3200.0	6000.0	/	0.94	85.1
2_EN	TN_1	1.13	12.5	22.8	22.9	25.8	2000.0	2000.0	2000.0	2000.0	2000.0	6537.8	3.53	19.8
	TN_2	1.13	12.5	22.8	22.9	25.8	2000.0	2000.0	2000.0	2000.0	2000.0	6537.8	3.02	23.2
	TN_3	1.70	35.0	22.8	72.1	38.8	2000.0	2000.0	2000.0	2000.0	3000.0	7286.2	2.41	29.0
	TN_4	3.40	37.5	22.8	-19.3	77.5	2000.0	2000.0	2000.0	2000.0	6000.0	/	1.17	59.8
	TN_5	3.40	37.5	22.8	-19.3	77.5	2000.0	2000.0	0.0	3200.0	6000.0	/	1.01	69.3
1_EN	TN_1	1.13	5.3	8.3	10.6	9.3	2000.0	2000.0	2000.0	2000.0	2000.0	5981.5	1.55	19.4
	TN_2	1.13	5.3	8.3	10.6	9.3	2000.0	2000.0	2000.0	2000.0	2000.0	5981.5	1.32	22.7
	TN_3	1.70	6.0	8.3	6.1	14.0	2000.0	2000.0	2000.0	2000.0	3000.0	14805.7	1.08	27.8
	TN_4	3.40	16.0	8.3	0.3	28.1	2000.0	2000.0	2000.0	2000.0	6000.0	1.3E+06	0.55	54.5
	TN_5	3.40	16.0	8.3	0.3	28.1	2000.0	2000.0	0.0	3200.0	6000.0	2.1E+06	0.47	63.8

structures for the needs of modal analysis, which does not allow nonlinear modelling of elements. For this purpose, we translated the existing system into a system of substitute springs [20] taking into account an appropriate stiffness of the connecting elements (brackets), while the deformation of the wall elements was included in the appropriate consideration of the material characteristics.

The starting point of the transformation was the equality of the horizontal displacements of the real and the alternative model as shown in [21] where the horizontal displacement consisted of translation and rotation of the rigid body:

$$u_x = u_{x,t} + u_{x,r} \tag{8}$$

The following are the replacement shear and "lifting, or vertical" stiffnesses of the replacement springs [21]:

$$K_s^* = 0,5 \cdot \sum_{i=1}^{m} K_{s,i} \tag{9}$$

$$K_d^* = \frac{\dfrac{2}{L^2}\sum_{i=1}^{m} K_i \langle r_i^2}{1-\dfrac{q \cdot L^2}{2 \cdot F \cdot H}} \tag{10}$$

The parameters in equation 10 have the following meaning: H – height of the wall, K_i – tension stiffness of hold-downs and shear brackets, q – distributed vertical load of the wall, F_h – horizontal force acting on the wall, L – length of the wall, r_i – distance of brackets from the compression side of the wall. Timber screws in overlapping joints were modelled by a usable linear spring (2-point link) with a defined shear response for shear-loaded bolts based on the stiffness given in EC 5.

$$K_{ser\_TN\_2\&3} = n \cdot \frac{\rho_m^{1,25} \cdot d}{23} = 9 \cdot \frac{420^{1,5} \cdot 8}{23} = 2694,50\,[\text{N/mm}] \tag{11}$$

$$K_{ser\_TN\_1} = n \cdot \frac{\rho_m^{1,25} \cdot d}{23} = 5 \cdot \frac{420^{1,5} \cdot 8}{23} = 14969,5\ [\text{N/mm}] \tag{12}$$

Wall segments were subjected to concentrated horizontal forces. The recorded responses are given in Table 3, while Figure 5 shows an example of the response of single-storey Xlam wall elements. Given that all the examples of wall elements were of the same length, but executed in different ways, a markedly different behaviour and different stiffness of the wall assemblies could be observed. As expected, the stiffness of the Xlam wall elements of type TN_1 to TN_5 increased, and the response of the elements varied from a markedly bending or "rocking" mechanism in wall elements TN_1 and TN_2 where rotation of wall elements took place, to a predominantly shear response or translation of the wall elements in the response of elements TN_4 and TN_5.

The calculation of the stiffness of the replacement springs and the stiffness of the wall segments is shown in Table 3. Designation 3_EN covers wall segments of the first floor of the superstructure in the case of an upgrade of 3 additional floors, while designation 2_EN covers wall segments of the second floor of the superstructure in the case of a three-storey superstructure, and wall segments of the first floor of the

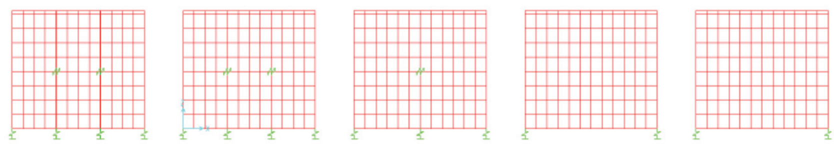

Figure 4. Numerical models of CLT wall with replacement springs (defined in SAP 2000)

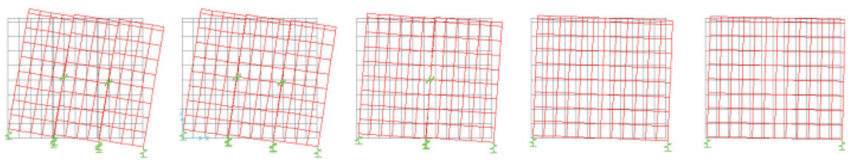

Figure 5. Deformation of numerical models of CLT wall with replacement springs

superstructure in the case of two floors. In all cases of upgrades from one to three additional floors, designation 1_EN indicates the upper floor of the superstructure.

The stiffness of wall elements by individual floors is shown int tables 3 and 4. It can be seen that the wall stiffness on the first floor of the superstructure and in the case of the three-storey superstructure (3_EN) ranges between 21.0 kN/cm and 85.1 kN/cm, while the load-bearing capacity of the wall segment is 33.6 kN, and at the most rigid one, it is 75.4 kN (Table 4). This means about 2-times change in the load-bearing capacity of the wall with 4-times change in the wall stiffness.

3. Parametric study

The purpose of the parametric study was to consider the largest possible set of basic and existing structures, which were further upgraded with additional floors and to monitor the responses of various upgrades and the effects of various upgrades on the existing structures. For this purpose, we varied the stiffness and the number of floors of the basic structures as well as the number of additional floors and their stiffness in individual basic structures, thus creating a set of 18 different basic structures, 15 different superstructure structures, which meant 270 different upgraded structures, as shown in Figure 7.

Structures having 1 to 6 floors were chosen for the set of basic structures. All of them were considered with three different levels of rigidity, namely, flexible (designation "F"), semi-rigid (designation "SR") and rigid (designation "R") basic structures. The criterion for determining rigidity was the natural period or its increase according to the number of floors. The masses of the basic structures were not varied. In the case of flexible basic structures, the natural period was increased by a step of 0.3 s per floor. In the case of semi-rigid structures, it was increased by a step of 0.2 s per floor and in the case of rigid structures, it was increased by a step of 0.1 s per floor. Thus, the natural periods of the 1-storey basic rigid, semi-rigid and flexible structures were 0.1 s, 0.2 s and 0.3 s, and, consequently, natural periods of the 6-storey basic structures were 1.8 s for flexible, 1.2 s for semi-rigid and 0.6 s for rigid basic structures.

Due to the larger range of wall stiffness values and, at the same time, much higher load-bearing capacity, the cross laminated

Table 4. Shear load bearing capacity of CLT wall samples

						Characteristic strenght CLT		Sheat and bending capacity CLT wall		Shear capacity due to shear brackets	Shear capacity due to hold downs	Resulting shear capacity
Specimen		H [m]	L [m]	bef [cm]	n	fv.k [kN/cm²]	fm. k [kN/cm²]	Fv. Rd [kN] (I)	Fm. Rd [kN] (I)	Fv. Rd [kN] (I)	Fv. Rd [kN] (I)	Fv. Rd_min [kN] (I)
	TN_1	3.0	1.13	6.0	3			596.6	337.1	66.6	33.63	33.6
	TN_2	3.0	1.13	6.0	3			596.6	337.1	66.6	33.63	33.6
3_EN	TN_3	3.0	1.7	6.0	2	0.4	2.4	598.4	508.6	66.6	48.66	48.7
	TN_4	3.0	3.4	6.0	1			598.4	1017.3	66.6	93.49	66.6
	TN_5	3.0	3.4	6.0	1			598.4	1017.3	75.4	130.94	75.4
	TN_1	3.0	1.13	6.0	3			596.6	337.1	66.6	24.94	24.9
	TN_2	3.0	1.13	6.0	3			596.6	337.1	66.6	24.94	24.9
2_EN	TN_3	3.0	1.7	6.0	2	0.4	2.4	598.4	508.6	66.6	35.55	35.6
	TN_4	3.0	3.4	6.0	1			598.4	1017.3	66.6	67.28	66.6
	TN_5	3.0	3.4	6.0	1			598.4	1017.3	75.4	104.74	75.4
	TN_1	3.0	1.13	6.0	3			596.6	337.1	66.6	15.66	15.7
	TN_2	3.0	1.13	6.0	3			596.6	337.1	66.6	15.66	15.7
1_EN	TN_3	3.0	1.7	6.0	2	0.4	2.4	598.4	508.6	66.6	21.54	21.5
	TN_4	3.0	3.4	6.0	1			598.4	1017.3	66.6	39.25	39.3
	TN_5	3.0	3.4	6.0	1			598.4	1017.3	75.4	76.71	75.4

Table 5. Masses of the basic and upgraded structures

	OK	OK + 1EN		OK + 2EN		OK + 3EN	
	M ([Ton]	M [Ton]	ΔM [%]	M [Ton]	ΔM [%]	M [Ton]	ΔM [%]
1 E_OK	7.9	13.1	65.8 %	17.7	124.1 %	22.3	182.3 %
2 E_OK	19.9	24.7	24.1 %	29.3	47.2 %	33.9	70.4 %
3 E_OK	31.5	36.3	15.2 %	40.9	29.8 %	45.5	44.4 %
4 E_OK	43.1	47.9	11.1 %	52.5	21.8 %	57.2	32.7 %
5 E_OK	54.7	59.5	8.8 %	64.1	17.2 %	68.8	25.8 %

timber system presented in 2.2 was used in the parametric study of superstructures. Figure 6 shows the natural periods of the superstructures themselves for the 1st-storey, the 2nd-storey and the 3rd-storey superstructures with the stiffnesses of the structures TN_1, TN_2, TN_3, TN_4 and TN_5. For all cases of superstructures with the same number of storeys, the ratio of period times between cases TN_1 and TN_5 was almost twice.

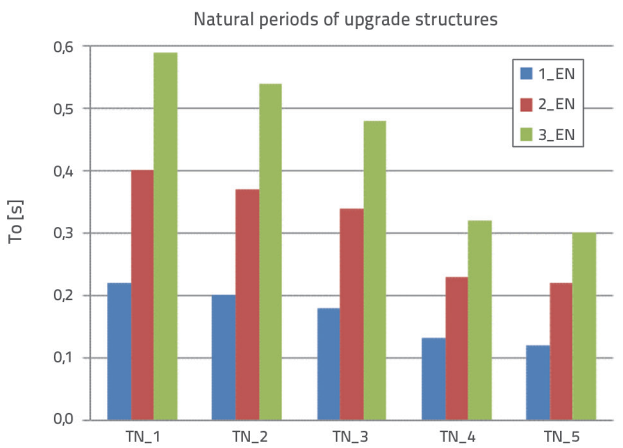

Figure 6. Natural periods of upgrade CLT structures

The meaning of the abbreviations and symbols is illustrated in the example below, while more detailed descriptions of the structures are given in the sections that follow R_OK_5E + 1EN_TN1: Rigid basic 5-storey structure with 1-storey superstructure with stiffness 1

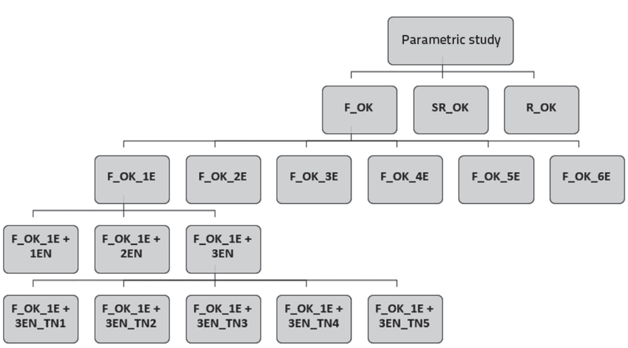

Figure 7. Partial presentation of the set of upgraded structures in the parametric study

Parts of the structure, but not the structure as a whole were considered as part of the parametric study. A part of the 3.4 m long structure was considered just like in the case with the timber wall segments presented in the section on structural modelling. The masses of the existing and upgraded structures were calculated for the case of an impact width of 3,5 m, as shown in the table and graph below.

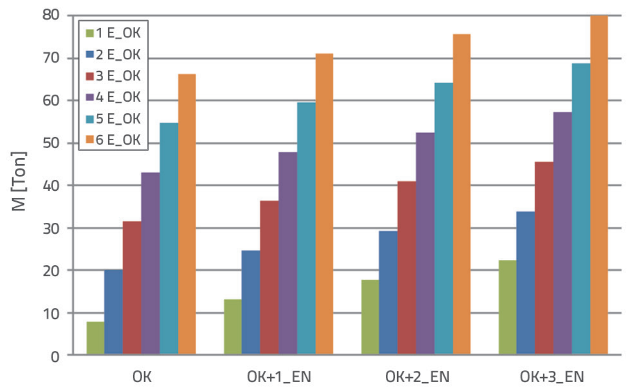

Figure 8. Comparison between the masses of the basic and the upgraded structures

The seismic analyses within the parametric study were performed by means of the SAP 2000 program. To cover all different levels of rigidity of different structural systems and materials, the basic structures were modelled by substitute diagonals corresponding to the desired increase of natural periods per floor. The cross laminated timber upgrade structures were modelled as shown in Section 2.2. The masses were modelled as concentrated at the levels of individual storeys. For all examples of basic and upgraded structures and superstructures, a modal analysis with a response spectrum was performed, considering soil type A (S = 1.0), ground acceleration a_g = 0,25 g (return period: 475 years), and behaviour factor q = 1.5. The response spectrum was chosen for the city of Ljubljana as the capital of Slovenia.

According to EC 8, the sum of effective modal masses for the considered modes had to be at least 90 % of the total seismic mass. However, all modes corresponding to 100 % of the seismic mass were taken into account in the presented study.

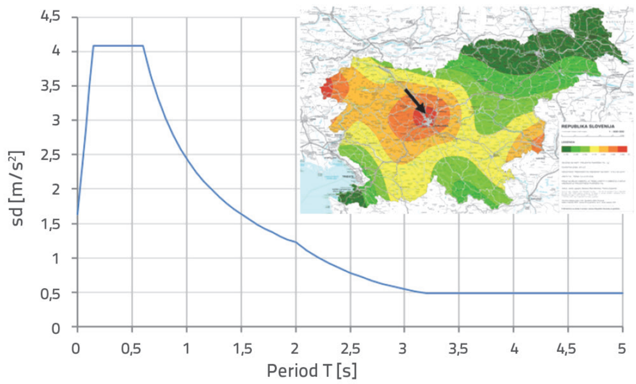

Figure 9. Response spectrum curve

Disregarding the total seismic mass could have caused an inconsistency in comparing the results of the modal response spectrum analysis.

3.2. Results of upgrading flexible basic structures

With the exception of the 1-storey structure, flexible basic structures are located outside the plateau of the acceleration spectrum and therefore have a relatively low level of seismic forces compared to the more rigid structures discussed in the subsequent subsections. The seismic effect decreases outside, or beyond the area of maximum response (the plateau in the response spectrum).

From the results on the total seismic forces and the increase in interstorey drifts, it was observed that the values increased in most of the cases. In the case of upgrading of the 2- and 3-storey basic structures (F_OK_2E and D_OK_3E), a slight decrease of base shear and inter-storey drift was observed at the ground floor at certain rigidities of the superstructure. In general, all results (total seismic forces, floor shear forces and floor displacements) show the impact of the superstructure stiffness on seismic loads. In these two cases, the situation is such that variation in stiffness can mean either favourable or unfavourable impact on the existing structure or its (lower) floors. In general, a more favourable effect of flexible upgrades was observed in all cases.

The reason for the favourable effects that occur in certain cases can be found in the favourable ratio of the decrease of value in the response spectrum, as a consequence of the increase in the natural periods of the structure and the limited increase in mass. The effect is noticeable in cases where the drop in value in the response spectrum is the greatest. In the cases of upgraded basic structures with more existing floors, the upgrade did not have any favourable effect despite a smaller percentage of increase in mass. Such an example is the upgrade of a 6-storey structure, where the change in the value of the influential part of the response spectrum for an upgraded 6-storey structure is very small and thus has a very small effect.

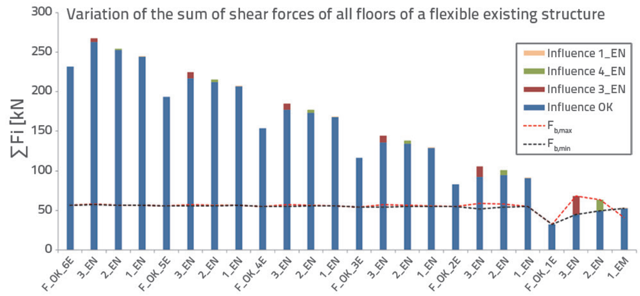

Figure 10. Variation of the sum of shear forces of all floors of a flexible existing structure

3.3. Results of upgrading semi rigid basic structures

All semi rigid basic structures, except 1- and 2-storey structures, have natural periods outside or beyond the plateau of the response spectrum, but they are positioned higher in the acceleration spectrum than flexible structures (F_OK). Consequently, the seismic loads are slightly higher compared to those of the upgraded flexible basic structures.

In the cases of adding storeys to 2, 3, 4, 5 and 6-storey structures, favourable effects of the superstructure are observed, especially at the ground floor, which is also the case with the 1st floor of the 3, 4, 5 and 6-storey basic structures. In the case of added storeys in 2, 3 and 4-storey structures, it can be observed that the impact of an upgrade with two or three additional storeys, despite the greater additional weight, is even more favourable than the impact caused by only one additional storey. The decrease in shear forces in the lower floors can be explained by a decrease in the value in the response spectrum, which clearly predominates in relation to the additional mass due to the multi-storey superstructure. The shear forces understandably increase due to additional mass on the existing structure, on the upper existing floor in all cases, and also in the lower floors in certain cases.

In general, all the observed changes (total seismic forces and floor shear forces and floor displacements) show that the change of rigidity of the superstructure can have a significant impact on the seismic loads. In certain cases, the favourable impact of the change in stiffness of the upgrade structure on the base shear force was up to 25 %. The impact of changing the stiffness of the upgrade decreased with a decrease in the number of additional floors. In all cases of upgrading the 1-storey basic structure, the impact of changing the rigidity of the superstructure was understandably the greatest. As in the previous subsection, a more favourable effect of more flexible upgrades was observed in all cases.

The reason for the favourable effects that occur in the cases of upgrading 2, 3, 4, 5 and 6-storey basic structures can be found in the favourable ratio of decrease in values in the response spectrum, as a consequence of increasing natural periods of the structure, and of a relatively small weight increase. The effect is observed in cases where the drop in the response spectrum values is the greatest. In cases basic structures with more existing floors were upgraded, the upgrade had no favourable effect despite the small percentage increase in mass, which is due to a minimum decrease in the response spectrum.

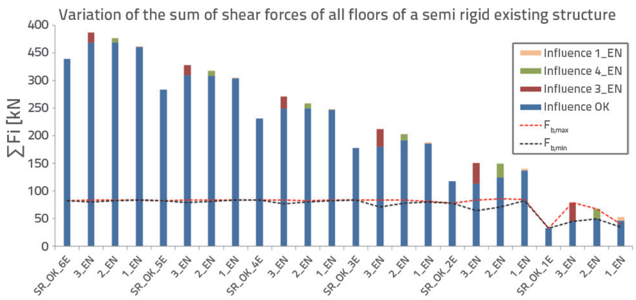

Figure 11. Variation of the sum of shear forces of all floors of an existing semi rigid structure

3.4. Results of upgrading rigid basic structures

With the exception of 1- and 6-storey structures, all rigid basic structures are located in the area of the response spectrum plateau. Consequently, the levels of horizontal loads in these cases are the highest. With the added floors, the natural periods increase depending on the number of floors added, the stiffness of the superstructure, and the additional weight. In the case of an upgrade involving three additional storeys, the impact is such that the new natural periods are, in all cases, outside the range of the maximum seismic action. The impact on the increase in natural periods decreased with a decrease in the number of additional storeys.

In the case of upgrades of 2, 3, 4, 5 and 6-storey structures, favourable effects of upgrades were observed at all floors except at the level of the upper floor of the existing structure. The decrease in shear forces and floor displacements in the lower floors can, as in the previous two sections, be explained by a decrease in the value of the acceleration spectrum, which clearly predominates in relation to the additional mass as a result of the additional floors. The reduction of the total seismic force in all cases, except for the upgrade of the 1-storey structure, was between 15 % and 25 % in the most favourable cases, and more favourable results were in all cases obtained for more flexible superstructures. The impact of the rigidity of the superstructure was the greatest when upgrading rigid basic structures and, in the case of 3-storey superstructures, it meant a change in the total seismic force between 15 % and approx. 50 %. The impact decreased in 2- and 1-storey superstructures.

Graphs of floor displacements show that, in the case of more flexible superstructures, a more pronounced rise in stiffness occurs between the basic structure and the superstructure, while in the case of more rigid superstructures, the deformation shapes are more uniform. As cross-laminated timber structures are generally relatively rigid (in the case of normal proportions of wall segments), deformations at the level of the upper floors do not, in any case, exceed 1 % of the floor height, which corresponds to the strictest requirement of Eurocode 8, prescribing the use of the reduction factor of displacements ν for checking deformations.

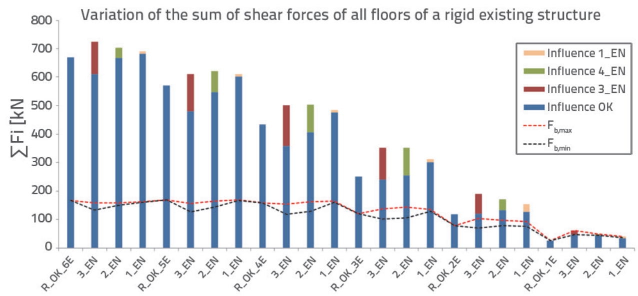

Figure 12. Variation of the sum of shear forces of all floors of a rigid existing structure

4. Analysis of the results of a parametric study

Within the scope of the parametric study, the level of stiffness and the number of storeys were changed to cover the largest possible range of potential structures that could result from upgrades. The structures were upgraded with variations of 1 to 3 floors of the superstructure and 5 different levels of rigidity were determined for all upgrades. The Xlam upgrade system was chosen based on the greater possibility of variation in stiffness and higher load-bearing capacity of wall elements. The study showed that, without changes of their global geometry, the stiffness of the Xlam elements could be changed very easily and cost-effectively only by different composition and configuration of connection elements. By changing the stiffness, the load-bearing capacity of entire wall assemblies also changes, but for more flexible systems with lower load-bearing capacities, as stated by foreign authors, higher seismic reduction factors can be adopted, which consequently means lower horizontal forces. Changes in natural periods, seismic forces, displacements and interstorey drifts were monitored based on modal response spectrum analysis, depending on the number of superstructure floors and variations in the rigidity of the superstructure. Regardless of the rigidity, the share of the structural mass was increased. Figure 10, Figure 11, and Figure 12 show the sums of floor shear forces of the existing part of the structure (S Qi) in examples of the existing and upgraded structures, with brown (influence of 1_EN), green (influence of 2_EN) and red (influence of 3_EN) part of the columns showing the possible difference in the total shear forces with respect to the variation of stiffness. The graphs also display the maximum (F_{b_max}) and minimum (F_{b_min}) values of the total seismic force (base shear), which were achieved based on the variation of the rigidity of the superstructure. From the graphs shown (change in the total seismic force depending on natural periods of the superstructures), it can be observed that the influence of rigidity of superstructures increases with an increase in rigidity of the basic structure. In the case of 3-storey superstructures of flexible basic structures, the maximum range of change in the total shear force was more than 10 %. In the case of medium-rigid basic structures, it was up to 25 %, and in the case of rigid basic structures, it was up to approximately 45 %. The case of upgrading with 1-storey structure was excluded, as there the total seismic forces increased significantly due to a significant increase in mass. The shear forces on the upper existing floor were increased in all cases, which is a logical consequence of the additional mass at the top of the existing structure.

In all cases, the results of the extensive parametric study show a significant influence of the rigidity of the superstructure on the response of the structure. Flexible upgrades had a more favourable effect on changes in seismic forces and displacements of the basic structure, with variations in stiffness with a larger number of additional floors having a greater impact on the responses of the structures compared to a smaller number of floors of the superstructure. In most cases, a change in the rigidity of the superstructure had either favourable or unfavourable effects on the change in seismic loads, pointing to the importance of determining the appropriate rigidity of the superstructure.

The more favourable effect of more flexible upgrades can, in most cases, be explained by a decrease in the value of the response spectrum, as the natural periods increased the most with flexible upgrades. The greatest favourable effects were recorded in structures whose natural periods were initially located in the plateau of the acceleration spectrum and moved past the plateau the most with the upgrades (Figure 13 - area 2). Such structures were mainly semi-rigid 2, 3 and 4-storey basic structures and rigid 3, 4, 5 and 6-storey structures. Figure 11 and Figure 12 referring to the mentioned structures show a decrease of the total seismic forces, while in the case of upgrading rigid basic structures, even reduction can be noted in the sum of all storey shear forces of the existing part of the structure. In these cases, except for the upper existing floor, it can be said that the superstructures had a favourable effect on the seismic loads of the existing structure in the sense that they were generally reduced. In practice, such examples of buildings are usually reinforced concrete structures or masonry structures constructed with a relatively large proportion of wall elements or columns relative to the floor area. In addition to the favourable effect of reducing seismic loads in the lower floors of such structures, the load-bearing capacity of the elements is generally also favourably affected by the increase in vertical load as a result of the upgrade. The bending and shear capacity of RC walls or columns and wall columns increases with an increase in vertical load. The only exceptions are cases when this load is so large that the elements become less ductile, and the cross-sections may fail due to high compressive loads. Such cases are usually characteristic for RC columns and less often for wall elements.

In the case of upgrading flexible basic structures, the favourable effects of upgrades were smaller, as the values based on natural periods in the flat part of the acceleration spectrum decreased with a smaller ratio in respect to the share of the mass increase (Figure 13 - area 3). In most cases, the seismic effects increased significantly in the case of upgrading 1 and 2-storey rigid structures. In the case of 1-storey structures, the natural periods moved to the plateau, while in the case of 2-storey basic structures, the natural periods remained within it. The preservation of the value in the acceleration spectrum and the increase in the mass of the structure resulted in a significant increase of seismic influences (Figure 13 - area 1). Based on the results of the parametric study, the existing structures can be schematically classified into three areas in the acceleration spectrum according to their suitability to be upgraded in terms of increase of seismic loads and influence of superstructure properties (additional weight and rigidity of the superstructure) on seismic loads.

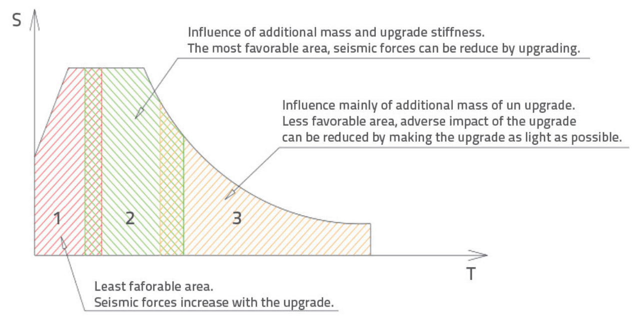

Figure 13. Suitability of structures to be upgraded as related to location in the acceleration spectrum

The results of the parametric study prove that the change of stiffness of the superstructure may have an influence upon the seismic response of the new upgraded structure. It is shown that, in certain cases, or combinations of suitable basic structures and suitable upgrades, the seismic safety of a building can even be increased from the viewpoint of seismic safety, where the greatest favourable effects are observed in the critical lower floors. In most cases, smaller increases in seismic loads were caused by more flexible superstructures and, in some cases, these effects were even favourable, as they reduced seismic loads and displacements of the structure. A similar situation was observed during the shaking table tests, namely in the case of upgrading the rigid basic structure with a light and flexible steel structure [21], where the displacements of the basic structure dropped with the added upgrade. The study shows that the benefit of the upgrade effect is influenced by the additional weight of the upgrade and especially by the change in the value in the acceleration spectrum. Hence, it is necessary to be aware that, in the design practice, each upgrade should be considered separately. Namely, the stiffness and mass ratios of the basic structures and the superstructures differ from case to case, while the shape of the acceleration spectrum is also defined by soil type.

5. Instead of a conclusion - Example from design practice

When an idea of upgrading an existing structure arises, the first questions in the design practice that need to be answered are the following: which material is to be used to construct the upgrade and when was the existing building built. The reasons why timber structures, such as timber frame or Xlam, are suitable for upgrades have already been given in the paper. The answer to the second question gives us at least a rough estimate of how the basic structure is constructed in terms of details and performance related to earthquake resistance.

In Slovenia and in former Yugoslavia, the first regulations related to seismic codes were published in 1963 [37, 38]. The ensuing regulations, in which the requirements already approached the Eurocodes, came into force in 1981 [39], then were amended in 1985, and were applied until 2008, when the current Eurocodes became mandatory. With each change in regulations, seismic loads increased significantly, but the requirements for a more ductile behaviour increased as well. A comparison of seismic regulations in former Yugoslavia, including seismic loads and requirements in terms of ductile behaviour, together with comparison of load-bearing capacity, are presented in

In any case, building plans, inspection of plans corresponding to the as-built condition, inspection of the building condition, identification of possible damage and, if possible, investigation of installed materials and reinforcement of cross-sections, are necessary to obtain sufficient input data on an existing structure for design and analysis. In some cases, ambient vibrations can be measured to determine the condition of a structure. The following analyses are usually performed for existing buildings: detailed analysis of the load-bearing capacity of foundations, analysis of the load-bearing capacity of elements with regard to vertical loads, and seismic analysis. In case of favourable results, the building analysis for the upgraded state are conducted. To obtain the most accurate results regarding seismic resistance, a more complex nonlinear analysis is carried out to verify the seismic resistance. Such analysis also provides a more favourable result in comparison to a simpler analysis, which is what we are looking for in such cases [30]. At the design phase, special attention should be paid to the upgrade, namely maintaining a lower level of additional mass (structure and composition of floors, walls and roofs) and adapting the floor plans to the lower part of the building. Masonry or concrete wall structures have proven to be the most suitable for upgrading. As to the large proportion of wall elements, sufficient seismic resistance can be expected, while the low additional mass represents a very small fraction in comparison to the existing building. In cases of upgrading with wall type structures, the load transfer is normally not critical as the load is transferred (distributed) along the elements, which is not the case with frame type structures. Point load transfer may prove critical in some cases, especially in transmission of horizontal loads where the implementation of suitable joints must be enabled, while a basic structure should prove to have a sufficient local capacity. When dealing with rigid basic structures upgraded with much more flexible structures, such as steel structures with moment resisting frames, different parts of the structure (upgrade and basic structures separately) can be excited by different excitation frequencies [19, 21]. An increased response of the upgrade structure, i.e., the so-called whip effect, can be expected. In practice, each case is unique, and the demands should be treated with great care and caution.

The example of the upgrade of the Terme Hotel in Brežice, Slovenia, which was completed in 1977, is shown below. The building was designed in accordance with the 1963 seismic regulations [37, 38] and the lateral force method of analysis was used. The existing 3 storey (G + 3) structure of the hotel is founded on strip foundations, while the vertical structure is made of 19 cm thick silicate masonry walls enveloped with vertical and horizontal ties. The RC roof and floor plates are 15 cm thick. The central part of the building has numerous RC walls at the ground floor. The structure consists of three parts (left wing, central part and right wing), which are separated by expansion joints along all floors.

The seismic resistance of the existing structure without the superstructure was checked in the design phase by company CBD d.o.o. along with the Civil Engineering Institute ZRMK d.o.o. The analysis was then carried out by a 10 % increase in seismic mass, simulating an Xlam superstructure consisting of two floors and an attic. The SREMB program, developed at the Civil Engineering Institute ZRMK, which is based on the limit state method and considers the nonlinear relationship between loads and deformations, was used to account for the seismic resistance. Based on the layout of the walls and their mechanical characteristics, the program determines the hysteresis

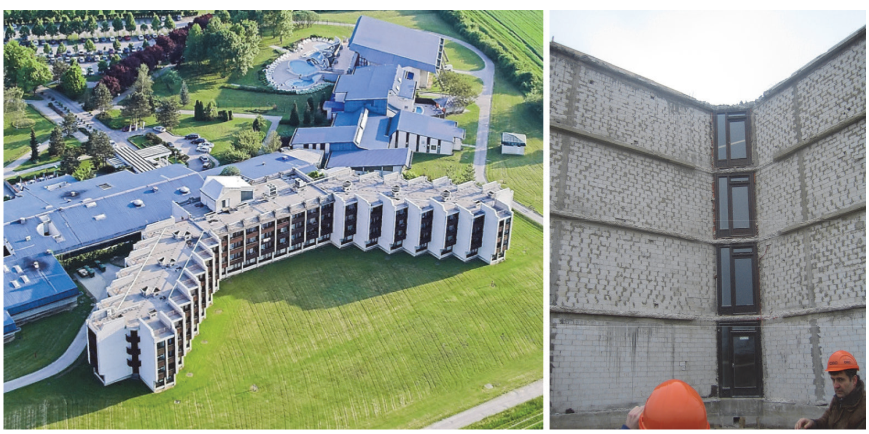

Figure 14. Hotel Terme before the upgrade

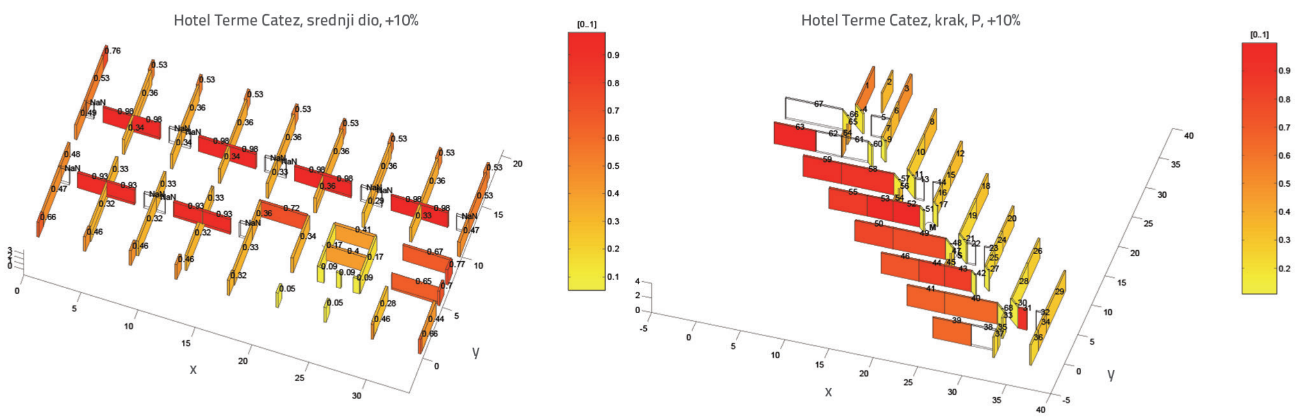

Figure 15. Load capacity of the 1st floor (left) and load capacity of the ground floor (right)

Figure 16. Assembly of Xlam structure

envelope of the critical floor. The calculation showed that both the existing structure, and the structure with an additional 10 % increase in mass, met the required seismic resistance in accordance with the requirements of EC 8. The analysis showed that extensive cracks would form in some walls during the expected earthquake (PGA 0.225 g with a return period of 475 years), but that the building would remain stable and would not collapse. Due to the RC walls on the ground floor of the central part, the 1st floor proved to be the critical floor of the central part, while at the left and the right wing, the ground floor was critical, as expected. The demonstrated sufficient seismic resistance is mainly due to the relatively large number of wall elements, good condition of the structure (no cracks), and good knowledge of the structure and the built-in materials.

Following the decision to upgrade, a modal response spectrum analysis of the entire upgraded structure was performed using the ETABS program. The existing masonry part of the building was modelled by substitute diagonals, while the Xlam elements were modelled by shell elements. The results of the analysis showed an increase of the natural periods and a slight reduction of the seismic forces despite the increase of mass. The change in stiffness caused a shift in the response spectrum from maximum to lower values. During the design, special attention was paid to minimisation of the added mass to the existing building which also included removal of the RC parapet walls on the roof of the existing structure, which represented a significant part compared to a timber superstructure. During the construction phase, it was found that the sloping concrete on the roof of the existing building could not be removed. Its weight was equivalent to the timber attic floor, so it was necessary to decide to carry out the upgrade with only two additional floors. The pictures below show the hotel during the Xlam assembly phase (Figure 16) and after completion (Figure 17).

Figure 17. Finished hotel with two-storey upgrade

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