Which Seismic Behaviour Factor for Multi-Storey Buildings made of Cross-Laminated Wooden Panels?

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1 Abstract

Day after day, multi-storey buildings made of cross-laminated wooden panels are becoming a stronger and economically valid alternative to their counteracts built with concrete and masonry. Throughout Europe and even in seismic prone zones this construction type is gaining a broader acceptance.

However, until now, in Eurocode 8 this constructive system is not yet included and no recommendations are given regarding constructive details. Especially regarding the value of the seismic behaviour factor to be used in seismic design of this new typology of wooden buildings, no comprehensive investigations have yet been undertaken.

In this paper results from shaking table tests on a 3 story cross-laminated wooden building are presented and the value of the seismic behaviour factor is found taking into account the real response of the building for 2 ground motion records of 2 well known historical earthquakes such as the 1995 Great Hanshin Earthquake (Kobe Earthquake) and 1997 Umbria-Marche Earthquake (Nocera Umbra record).

2 Introduction

The SOFIE Project is a cooperative research project patronised by the Trento Province, Italy and coordinated and conducted by the CNR-IVALSA (Italian National Research Council – Trees and Timber Institute).

The main purpose of this project is to analyse a multi-storey building built with solid wooden panels with cross interlayers considering every single aspect of the building behaviour such as static, fire, acoustic, thermal and, particularly, seismic performance.

Especially with regard to the last one, a comprehensive testing program have been undertaken, consisting in the following stages:

- tests on connections,
- in-plane cyclic tests on wall panels with different connection and opening layouts and with different dimensions and amount of vertical load,

- pseudo-dynamic tests on one storey specimen in 3 different opening layouts in the external walls parallel to the shaking direction and without vertical loads,
- shaking table tests a 3 storey building of about 7m x 7m in plan and 10m of total height with a pitched roof n 3 different configurations (3 different openings layout, A, B and C) and with 3 different earthquakes (Kobe, El Centro and Nocera Umbra) at 2 growing levels of PGA (0.15g and 0.5g)

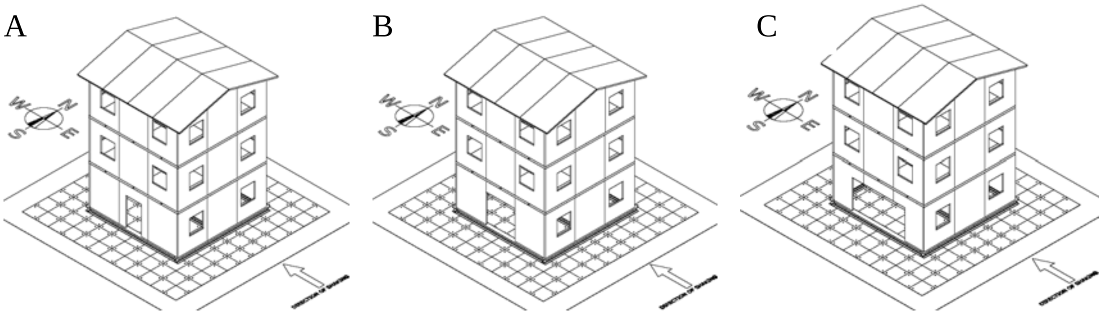

Figure 1: Three different configurations in which the building has been tested. While Configuration A and B are symmetric, Configuration C is asymmetric as the opening ot the other external wall parallel to the shaking direction is equal to the one in Configuration B.

Only in Configuration C the building has been tested also with Kobe and Nocera Umbra with growing level of excitations up to the reaching of the “near collapse” status.

2.1 How to determine the q value

Most seismic design codes contain action reduction factors (ARF) to be used to evaluate the forces to be accounted for when designing the structure using a simple elastic global analysis. ARF then reflects the capability of a structure to dissipate energy through inelastic behaviour, and survive even exceptional earthquakes without complete collapse.

In fact any code's objective is the building to resist the foreseen quake for that area. It is evident that behind this idea there is an assumption of acceptable risk for the community. Because resistance against earthquakes results from a combination of hazard and vulnerability, to take into account the relevant uncertainties (according to semi-probabilistic approach philosophy), appropriate safety coefficients are considered in the codes both for the design action and the design resistance.

This philosophy is the same of Eurocode 8 in which the ARF is called “seismic behaviour factor *q*” which according to the definition is *the factor used for design purposes to reduce the forces obtained from a linear analysis, in order to account for the non-linear response of a structure, associated with the material, the structural system and the design procedures.* Or in other words q is *the ratio between the PGAu that produces the ultimate displacement or rotation and the PGAy that produces the yielding of the first joint*.

Therefore under these assumptions and starting from the test results, the easiest way to find the q value for a particular building type and for a particular ground motion record is just to refer to the definition of q and apply the following procedure:

- Define an appropriate “near collapse” criterion (for example based on a maximum inter-storey drift, or a failure in joints or in timber elements);
- Design the structure using q=1 according to the seismic code for a given design PGA (which in this case is both the PGA leading the building to collapse and the PGA which cause the first yielding), and the resistant system according to the relevant codes (seismic and “static” codes) with the design values for seismic actions;

- Analyze the test results and apply the definition of q founding it by the ratio between the PGA value that caused the “real” collapse of the building and the design value of the PGA.

There’s also another pattern to follow in order to assess the correct q value for a given structural system which is based on mathematical calculations using an appropriate computer model capable of giving the non-linear response of the structure under a certain number of real earthquakes excitations, by applying the following procedure:

- Design the structure for a given q value, and the resistant system according to the relevant codes (seismic and “static” codes). At the end of this step the resistant system will be completely anticipated.
- Model the building mechanical behaviour on the base of its mechanical characteristic (obtained by tests, and scaled to 5% percentile based on COV and test mean value, using additional safety coefficients eventually provided by the code for the earthquake load combination).
- Using a suitable non-linear analysis programme capable of following the displacement history of the building under a quake in the time domain (calibrated on the results of shaking table tests), determine the PGAu that the building will survive without exceeding a given “near collapse” failure limit (for example based on a maximum inter-storey drift, or a rupture in joints or in timber elements).
- Compare this PGAu against PGAcode prescribed by the code.
- Finally, if PGAu > PGAcode the previously chosen design ARF value is adequate.
- This procedure must be repeated for a series of earthquakes suitable for the design site, in order to have a global picture according to different possible inputs.

Anyway this is a longer procedure which is still undergoing and whose outcomes will be published short after this paper.

2.2 Chosen strategy

The strategy used in this paper to evaluate the q value at least for the ground motion records used for the shaking tale tests is therefore the following:

- Design the structure using q=1 according to the seismic code for a given design PGA (0.35g which is the design ground acceleration corresponding to the more hazardous seismic zone of Italy)
- Define as “near collapse” criterion the overcome of the design strength value in holdown anchors;
- Analyze the test results and apply the definition of q founding it by the ratio between the PGA value that caused the “real” collapse of the building and the design value of the PGA.

3 Design of the test building according to Eurocode 8

The reference building considered is the one showed in Fig. 1 which have been tested in June and July 2006 at the NIED shaking table facility in Tsukuba, Japan. As above explained the procedure to follow in order to asses the q value starting from test results needs by definition the reaching of a near collapse status. Therefore, as this condition have been fulfilled only for the Configuration C, plans and elevations of the building are referred only to this configuration and are showed in Fig. 2.

Figure 2: Plans and elevation of the test building.

The distribution of dead and additional loads at each floor in the test house is the following:

Floor	Dead [kN]	Additional [kN]	Total [kN]
1^{st}	60	150	210
2^{nd}	60	150	210
Roof	45	0	45
Total	165	300	465

Table 1 Load distribution at each floor

According to Eurocode 8 the base shear force is calculated according to the following equation:

$$F_b(T_1) = S_d(T_1) \times m$$

Where S_d (T_1) is is the ordinate of the design spectrum at period T_1 and m is the total mass of the building.

From the outcomes of the tests the period T_1 of the building is 0,20 s, therefore the ordinate of the design spectrum is

$$S_d(T_1) = a_g \times S \times \frac{2,5}{q}$$

where:

a_g is the design ground acceleration corresponding to the seismic zone. According to the Italian Seismic Building Code is taken equal to 0,35g, corresponding to the most hazardous value of the Italian territory)

S is the soil factor (taken equal to 1,25 accounting for type B soil, e.g. deposits of very dense sand, gravel, or very stiff clay)

q is the behaviour factor taken equal to 1.

Therefore the calculation of the seismic forces and of the shear at each floor is the following:

Total Weight

roof		45	kN
2° floor		210	kN
1° floor		210	kN
	TOT	**465**	**kN**

seismic action

base shear			
zone 1; a_g =		0.35	
T1		0.20	
soil B S=		1.25	
q		1	
$\mathbf{F_b}$ = 2,5*(m*S*a_g)/q		**509**	**kN**
distribution on storeys			
height			
	Hr (roof) =	9.50	m
	H2 (2nd floor) =	6.12	m
	H1 (1st floor) =	3.02	m
horizontal forces at each floor			
	Fr =	93	kN
	F2 =	279	kN
	F1 =	137	kN
shear at each floor			
	Tr =	93	kN
	T2 =	371	kN
	T1 =	509	kN

3.1 Design of holdown anchors at ground level

The holdown anchors used to connect the building at ground floor are SIMPSON STRONG-TIE holdown anchors HTT22, connected to the basement by means of 8.8 Class M16 anchor bolts and with ϕ 4 annular ringed nails to the cross-laminated walls.

HTT22

Figure 3: Holdown anchors HTT22 and ϕ 4 annular ringed nails used to connect the holdown anchor to the cross-laminated wall

The distribution of holdown anchors at the ground floor and of the seismic forces at each floor is showed in Fig. 4

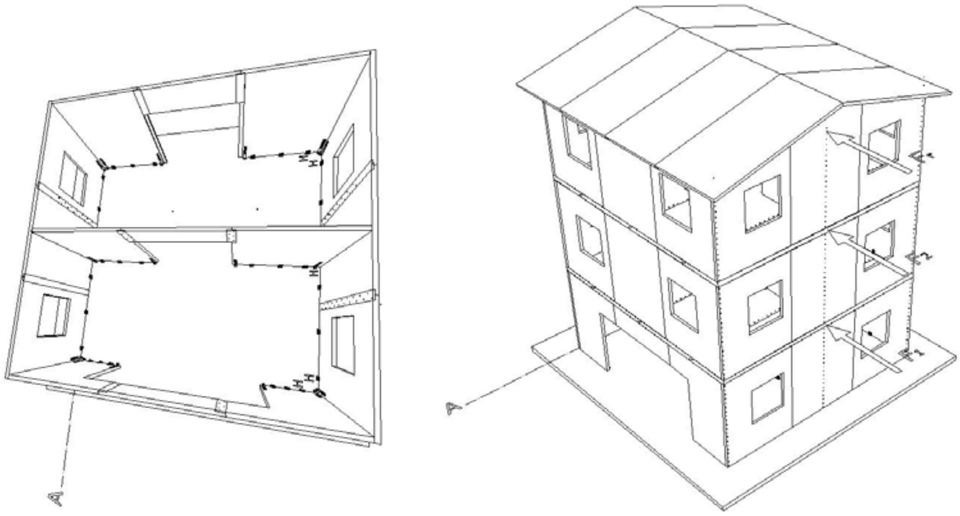

Figure 4: Distribution of holdown anchors and steel angles at ground floor and distribution of seismic forces at each floor. In the left side picture the holdown anchors marked with H are those taken into account in the design.

Considering only the design of the holdown anchors at the ground floor and considering also the contribution of the holdowns in the walls perpendicular to the shaking direction the calculation gives the following results (moment equilibrium around the A line and neglecting the contribution of holdown at openings):

$F_r x h_r + F_2 x h_2 + F_1 x h_1 - m x 6.93/2 - 5 x H x 6.93 = 0$

93x9.50+279x6.12+137x3.02-465x3.47-5xHx6.93=0 ⇨ H =39.75 kN

From the results of the experimental tests on the steel to timber connections using annular ringed nails, each nail has an ultimate shear resistance of 4 kN, which is taken as the 5-percentile value of strength. Therefore, according to Eurocode 5 and 8, the strength design value of each nail taking into account a non-dissipative structural behaviour is:

$$R_d = \frac{R_k \times k_{\mathrm{mod}}}{\gamma_M} = \frac{4 \times 1{,}1}{1{,}3} = 3{,}38\,kN$$

Hence to resist the uplift force each holdown anchor shall be connected using 12 nails.

H_r = 12x3,38 = 40.56 kN > H = 39.75 kN

Note that the design tensile strength of the Class 8.8 ϕ16 anchoring bolt, considering the effective cross section is:

$$N_r = \frac{A_{res} \times f_y}{\gamma_M} = \frac{157 \times 640}{1000 \times 1.1} = 91.35\,kN$$ which is greater than H_r

3.2 Test results

The test specimen have been instrumented with 8 load cells respectively corresponding to the 8 holdown anchors placed in the 4 corners of the building as showed in Fig. 5.

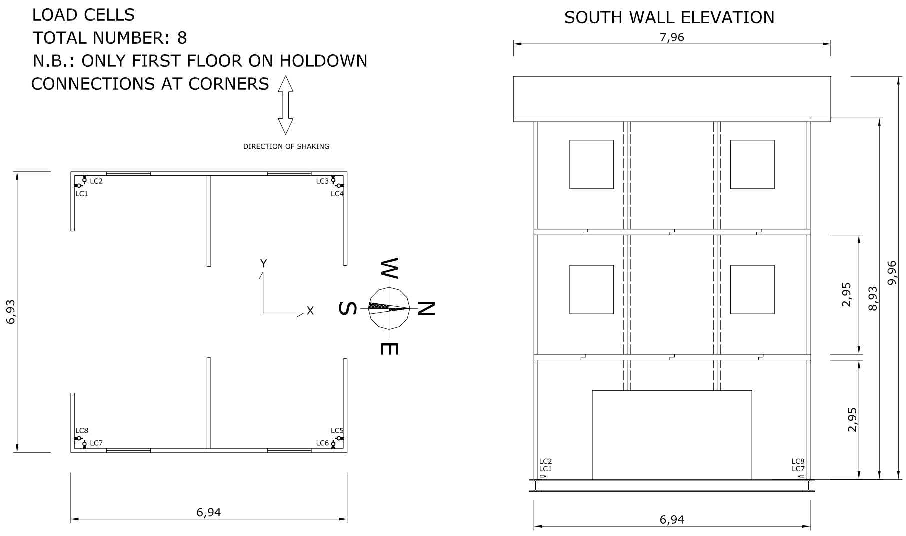

Figure 5: Distribution of load cells at ground floor

The test results in terms of maximum load recorded for each holdown anchor for configuration C are summarized in Table 2

Record	PGA [g]	LC1 [kN]	LC2 [kN]	LC3 [kN]	LC4 [kN]	LC5 [kN]	LC6 [kN]	LC7 [kN]	LC8 [kN]
Nocera Umbra	0.50	21.7	34.5	34.8	30.9	22.5	36.0	33.5	27.7
El Centro	0.50	13.9	26.3	25.1	24.3	17.1	28.3	29.9	26.0
Kobe	0.50	20.6	32.9	30.7	29.9	15.5	27.2	31.5	29.7
Kobe	0.80	**43.4**	**51.1**	**53.9**	**45.9**	35.3	**47.1**	**48.9**	**44.3**
Kobe	0.50	23.8	35.6	35.7	29.5	15.8	24.0	25.6	25.4
Kobe	0.50	24.1	38.5	**41.3**	35.1	9.8	17.5	24.4	23.8
Kobe	0.80	**49.0**	**48.7**	**51.3**	**49.1**	37.5	**42.4**	33.6	38.0
Nocera Umbra	1.20	29.4	35.9	**46.2**	37.2	**48.5**	**51.6**	**41.9**	**45.1**
Kobe	0.90	**60.6**	**67.4**	**72.7**	**56.6**	**43.0**	**50.5**	**40.3**	**42.0**

Table 2 Results of shaking table tests for Configuration C in terms of maximum load reached in holdown anchors.

Considering the design strength of each holdown anchor H_r = 40.56 kN and the results showed in Table 2, it can be clearly observed that in all 0.80g and 1.2g tests the design strength of holdown anchors was exceeded as highlighted by the bold numbers.

4 Outcomes and Conclusion

Analyzing the results showed in Table 2 it could be observed that for all the 0.50 g tests with the 3 ground motion records (Kobe, El Centro and Nocera Umbra) the design strength value of holdown anchors was not exceeded.

Therefore, considering the the design ground acceleration equal to 0.35g, by applying the definition of seismic behaviour factor given in 2.1 it could be found that at least, for the 3 ground motion records used, the q value for this system is not lower than

$$q = \frac{0.50}{0.35} = 1.43$$

5 Acknowledgements

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6 References

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