Numerical Analysis of Wood-Concrete Skyscraper by Using Spring Elements to Simulate Mechanical Connections

Report of a Short Term Scientific Mission

Within the Frame of COST Action FP1004
“Enhance mechanical properties of timber, engineered wood products and timber structures”

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Category

1. Purpose of the Short Term Scientific Mission (STSM)

Cross laminated timber (CLT) as a lightweight, prefabricated and environmental friendly new structural material is becoming popular in the markets of North America and Middle Europe during the past years. The research of cross laminated timber utilized in multi-story buildings started many years ago, and therefore has already obtained many experiences and achievements. Nowadays, the highest wooden residential building which is built by CLT panels including elevator shafts and stairwells is Murray Grove with nine stories located in London[1].

However, a wooden skyscraper is rarely to be imagined because of some conservative impressions of wooden such as insufficient strength, easy to deform, etc. Nevertheless, a feasibility of wooden skyscraper is researched. This skyscraper is a hybrid structure with central located concrete core and CLT exterior structural elements[2]. An essential difficulty of skyscraper is control of its deflection under horizontal forces which can be caused by wind load or seismic effect. Different from multi-story wooden buildings, primary attention of wooden skyscraper is paid to the wind load rather than seismic load. Although major lateral forces are carried by concrete core, wooden part of the building still has to undertake the rest lateral forces. However, wooden elements are not like concrete elements which can be casted into an integrated piece, they must be assembled on-site story to story by mechanical fasteners. As comparison to CLT panels, the mechanical connectors are quite flexible, the shear and flexural deflection of CLT panel become negligible, and most of the flexibility is concentrated in the connections. Hence, the strength and stability of wooden part of the skyscraper depend heavily on the mechanical connections.

Therefore, the stiffness and load bearing capacity of these mechanical connections need to be defined, so that they are replaceable to be simulated as spring elements with accurate and rational values in a mathematical model. Such mathematical model is developed by using of the FEM structural software SAP2000 to carry out static analyses where the wooden-concrete building is loaded with horizontal load. These are also the main work of this Short Term Scientific Mission at institute of National Research Council of Italy Trees and Timber Institute (CNR- Ivalsa), in Italy.

2. Description of the Work

2.1 Cross Laminated Timber

Cross laminated timber (CLT) as an industrially engineered wood product has already existed for decades, and it is now providing an innovative solution for construction system for buildings because of its large dimension, appropriate strength, seismic resistant ability and etc. CLT panels are fabricated by several orthogonally glued layers on top of each other over entire surface. Their dimension basically depends on the technical condition of manufacturers. Its thickness can reach to approximately 500mm, length of up to 16-20m and width of up to 3.5m [3]. Openings for windows or doors are easy to be prefabricated in these large dimensional panels, it means, they are quite suitable for one-piece story walls which are designed to be applied in the wood-concrete skyscraper.

2.2 Structure of Wood-Concrete Skyscraper

The designed wood-concrete skyscraper with 40 stories represents a hybrid building where the core walls are made by high-strength concrete (HSC) and locate in the central, and cross laminated timber panels are used as walls, floors, roofs and other structures elements of the building. The central located concrete core is the primary component for resisting both horizontal and gravity loading in tall building structures[4]. Concrete floor or story is planned to be settled in every ten story if necessary. On the one hand, they are used to isolate fire accident in each individual wooden part; on the other hand, they perform as horizontal cantilevers to connect the core shear walls and exterior CLT walls, as well as help the structure resisting the rotation of the core, causing the lateral deflections and moments in the core to be smaller than if the free-standing core alone resisted the loading[4]. CTL panels compose exterior shear walls and also provide important resistance to horizontal loading.

2.2.1 Structural Details

The layout of main construction skeleton shown in Figure 1 is 33.7m (X-direction) by 15.7m (Y-direction) in plan, and 132m in height of the building. Blue and yellow lines respectively represent concrete elements and CLT elements, the fine grey lines mean openings such as windows in exterior walls and elevator doors in interior walls. In the ideal case without considering transport limitations, exterior CLT walls of one story are supposed to be comprised by six panels which are 3.05m in height. Two panels without opening of span of 10.6m are arranged in the short side (Y-axis) of the building, and other four panels are 13.2m in width, assembled in the long side (X-axis) with prefabricated window openings. Windows are assumed in the middle of walls with a uniform height of 1.65m (Figure 2), and elevators are designed to be situated at the concrete core, and their doors are about 2.5m high.

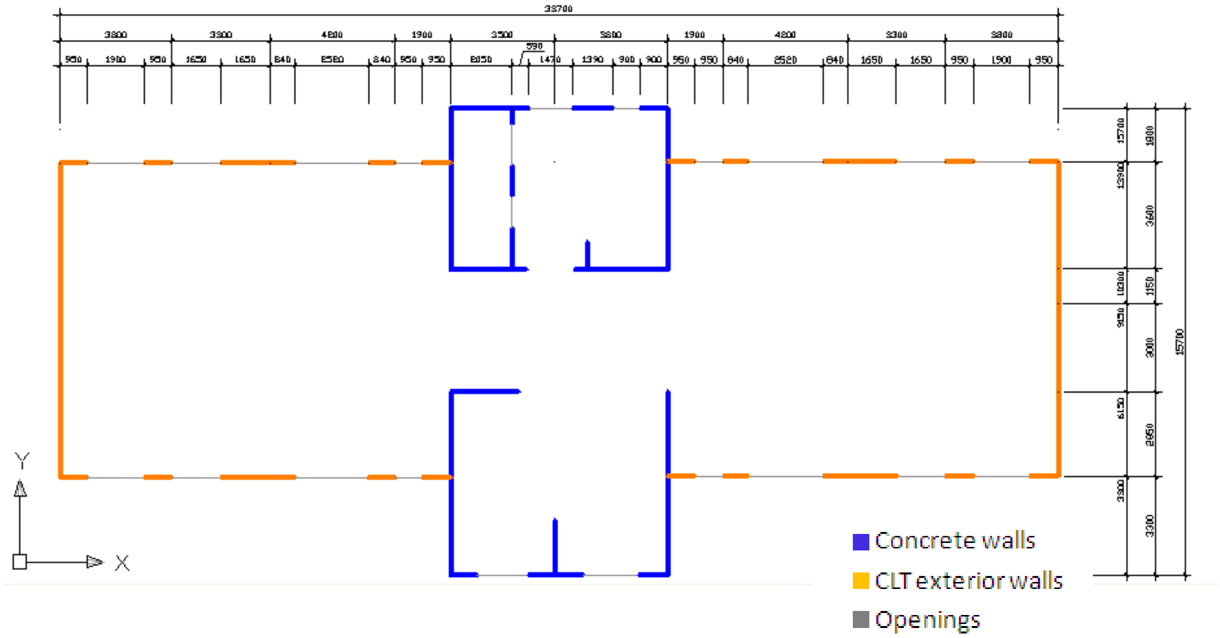


Figure 1: Layout of the main skeleton of the skyscraper

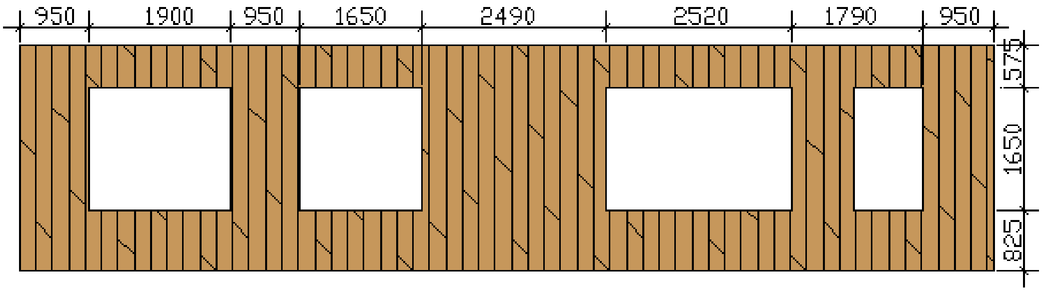


Figure 2: CLT wall panel with window openings

As mentioned before, for the whole 40-story building, there is one concrete floor for every ten stories. Except these four concrete floors, all the other floors are composed by cross laminated timber slabs. These horizontal floors are designed as diaphragms to tie the structure together, and to transfer lateral forces to vertical resisting elements. The horizontal floor diaphragm is much stiffer than the wall when they are under the lateral load. Deflection in diaphragm is so small compared with those in main lateral loading resisting structure so that the CLT floor diaphragm is assumed rigid.

Steel bars, screws and nails are used as basic mechanical connectors for linking CLT panels into a whole structure, and also for resisting uplifting and shear forces that acting on the structure under lateral load. The dimension and number of steel bars, screws and nails are needed to be defined by calculation where the wind forces govern the design.

2.2.2 Characteristic of Structural Materials

In order to get an accurate result and creditable engineering judgment, a mathematical simulation model was developed in the FEM structural software SAP2000 to carry out static analyses where the wood-concrete building is loaded with equivalent horizontal forces.

The high-strength concrete is treated as an isotropic material, and its mechanical properties are easy to be defined in FEM program. In contrast, the mechanical characteristics of CLT panels are more complicated due to the nature properties of wood and the orthogonal glued layers. In order to simplify the analysis and calculation, Blaß (2004) proposed homogenization of solid wood panels with cross layers into one layer homogeneous orthotropic material by the approach using composition factors[5]. Furthermore, E-moduli and shear modulus of CLT panels were estimated on the basis of experimental tests by Dujic, 2008[6].

The modeling of CLT panels in numerical model where they are simulated as orthotropic shell elements is established based on these researches. Basic parameters of two main structure materials used in the model are presented in Table 1.

Table 1: Input data for simplified model

Properties	Cross-Laminated Timber	High-Strength Concrete
Density [kg/m³]	400	2300
E-Modulus [Mpa]	8000(in load-bearing direction E_0)	4000
	4000(in the perpendicular direction E_{90})	
Shear Molulus [Mpa]	500	1670

2.3 Design of Mechanical Connectors

An important aspect of developing the numerical model is defining stiffnesses of links which simulate mechanical connectors between CLT elements. Therefore, the type, dimension and number of steel bars, nails and screws should be identified at first, and the design of these connectors must be able to carry the lateral wind forces. However, the wind forces are distributed on various parts of the structure, how many forces acting on the concrete core or CLT shear walls are unknown and difficult to be calculated by hand. In order to obtain a correct and reliable design of mechanical connectors (spring links in FEM), a simplified temporary numerical model for identifying the distributed force on various parts especially on CLT shear walls is implemented in SAP2000.

2.3.1 Temporary Model

Figure 3 shows first two stories of the designed temporary model. The most important simplification in this model is that CLT wall panels are assumed to be glued together as an entire piece which continues from the base to the top of the whole building. The floor diaphragms connect also directly with these walls, i.e. the model is without any links, and all elements are directly and rigid connected. Other simplifications are:

- Concrete wall thickness is uniform along the height of the building;
- CLT wall thickness is uniform along the height direction;
- Thickness of all the floor slabs are the same;
- Concrete floor slabs are not considered in this temporary model;
- Interior walls are neglected.

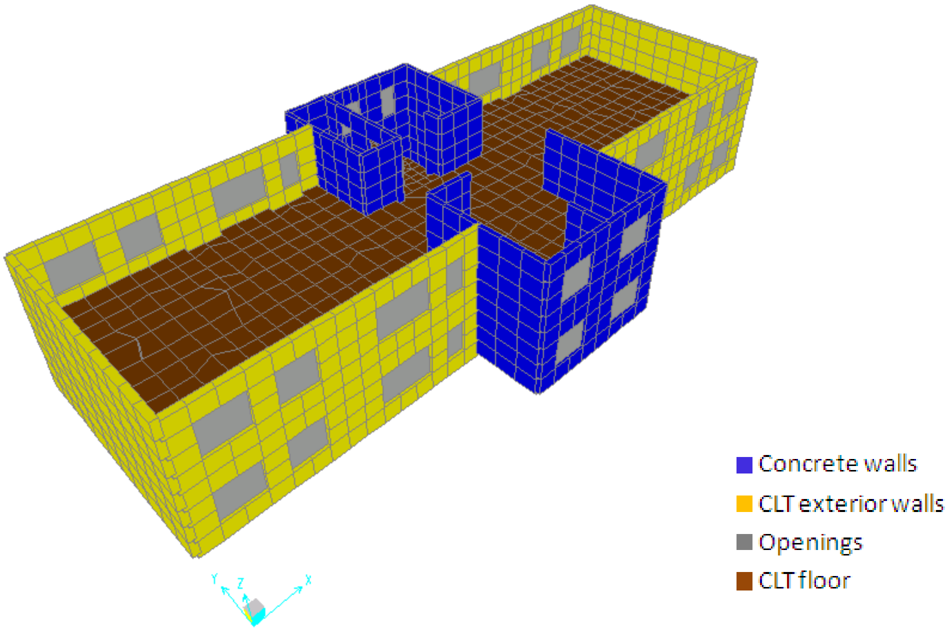


Figure 3: Simplified temporary model of first two stories

The thicknesses of concrete walls, CLT walls and CLT floors are defined as 250mm, 300mm and 250mm. Uniform wind load of 1.84kN/m^2 and live load of 2kN/m^2 are taken into account in the static analysis. The analysis results of FEM have been checked by manual calculation, such as the maximum shear force V_{max} at the base and maximum deflection δ_{max} at the top of the building. They are calculated according to the following equations:

$$V_{max} = qH \tag{1}$$

$$\delta_{max} = \frac{qH^4}{8EI} \tag{2}$$

where q (kN/m) is the wind load uniformly distributed over the height, H (m) is the total height of the building and EI (kN.m^2) is the bending stiffness. The results derived from different methods are quite similar, which proof the feasibility and reliability of present simulation environment. Hence, the following analysis will adopt the calculation results of this temporary model from SAP2000.

The distributed wind forces on various parts are exhibited in the Table 2. V_{tot} and V_{core} represent the total shear forces acting on the corresponding story and core; $V_{CLT\text{-}yl}$ and $V_{CLT\text{-}yr}$ mean the lateral forces carried by CLT shear walls which locate respectively on left and right hand side in Y-direction. It should be emphasized here that $V_{CLT\text{-}yl}$ and $V_{CLT\text{-}yr}$ are the crucial factor of the design of mechanical connectors.

Table 2: Distributed wind load on various parts of the skyscraper

Story Nr.	V_{tot} KN	V_{core} kN	$V_{CLT\text{-}yl}$ kN	$V_{CLT\text{-}yr}$ kN
01	7980	7425	264	286
02	7776	6795	473	537
03	7571	6254	620	695
04	7367	5813	731	810
05	7162	5450	809	891
06	6957	4702	1066	1141
07	6753	4563	1051	1125
08	6548	4386	1036	1109
09	6343	4195	1029	1101
10	6139	3997	1026	1096
11	5934	3796	1024	1092
12	5730	3598	1022	1087
13	5525	3404	1017	1080
14	5320	3216	1010	1070
15	5116	3048	999	1057
16	4911	2859	986	1041
17	4706	2689	971	1023
18	4502	2525	952	1001
19	4297	2367	931	977
20	4093	2213	909	952
21	3888	2062	884	924
22	3683	1915	858	895
23	3479	1771	830	864
24	3274	1629	802	833
25	3069	1489	773	800
26	2865	1350	743	768
27	2660	1212	712	735
28	2456	1075	682	701
29	2251	937	652	668
30	2046	799	622	635
31	1842	660	592	603
32	1637	520	563	571
33	1432	378	534	540
34	1228	234	506	510
35	1023	89	479	480
36	819	-57	453	452
37	614	-205	426	424
38	409	-353	400	396
39	205	-496	372	367
40	0	-644	334	329

2.3.2 Stiffness of Spring Links

For implementing the numerical model, type of fasteners is not crucial, but the stiffness of these fasteners should be seriously considered. At present, shear anchor bracket with nails of ∅4/60 mm is assumed to connect the CLT wall and CLT floor panels. The corner connection of CLT walls is equipped with screws of ∅8/600 mm. The design lateral load-carrying capacity per nail of ∅4/60 mm is 1.4kN and per screw of ∅8/600 mm is 7.7kN according to the Eurocode5 (EC5). On the basis of this data, the necessary number of nails and screws used in every story is determined. Furthermore, the slip moduli K_{ser} of each nail without pre-drilling and screw are also calculated by the following equations according to the EC5:

nail without pre-drilling: $$K_{ser-n} = \rho_m^{1.5} d^{0.8}/30 \quad (3)$$

screw: $$K_{ser-s} = \rho_m^{1.5} d/23 \quad (4)$$

where ρ_m (kg/m³) is the mean density of the material, d (mm)is the diameter of the fastener. The slip moduli of each nail of 1600N/mm and screw of 2700N/mm are then taken into account. The number and stiffness of nails and screws for connecting CLT wall and floor panels in every story are shown in Table 3.

Besides the horizontal deflection, the lateral load also causes tensile forces which could push structure elements upwards. It may lead to failure of the materials. The core material, high-strength-concrete which has been applied in high-rise buildings over hundreds years, already has quite a mature technology, so here only the CLT panels' behavior is discussed. The resistance of uplift force in CLT shear walls substantially depends on their connection system. In order to help the CLT elements to resist the tensile force, steel bars are designed to be integrated in structure (Figure 4). Vertical leer channels are formed in the middle layer of CLT panels, steel bars will be assembled into theses channels and connect CLT wall panels in several stories together through horizontal floor slabs. If necessary, couplers can be used to tie and extended these steel bars. At the head and end positions, they are fastened by holding end anchors, so that CLT wall panels are combined firmly together.

As bending moments of various parts of the building are also computed in the numerical analysis of temporary model, the tensile forces F_t acting on CLT wall panels are calculated according to the following equation:

$$F_t = \frac{M_b}{l} \quad (5)$$

where M_b (kN.m) is the bending moment and l (m) is the length of the panel where the bending moment effects on.

Table 3: Number and stiffness of nails and screws in connecting the CLT wall and floor panels

Story number	Nr. of nails		Stiffness of nails [N/mm]		Nr. of screws		Stiffness of screws [N/mm]	
	yl	yr	yl	yr	yl	yr	yl	yr
01	189	205	302400	328000	35	38	94500	102600
02	338	384	540800	614400	62	70	167400	189000
03	443	497	708800	795200	81	91	218700	245700
04	522	579	835200	926400	95	106	256500	286200
05	578	637	924800	1019200	106	116	286200	313200
06	762	816	1219200	1305600	139	149	375300	402300
07	751	804	1201600	1286400	137	147	369900	396900
08	740	793	1184000	1268800	135	145	364500	391500
09	736	787	1177600	1259200	134	143	361800	386100
10	734	783	1174400	1252800	134	143	361800	386100
11	732	781	1171200	1249600	134	142	361800	383400
12	730	777	1168000	1243200	133	142	359100	383400
13	727	772	1163200	1235200	133	141	359100	380700
14	722	765	1155200	1224000	132	139	356400	375300
15	714	756	1142400	1209600	130	138	351000	372600
16	705	744	1128000	1190400	129	136	348300	367200
17	694	731	1110400	1169600	127	133	342900	359100
18	681	716	1089600	1145600	124	131	334800	353700
19	666	699	1065600	1118400	121	127	326700	342900
20	650	680	1040000	1088000	119	124	321300	334800
21	632	660	1011200	1056000	115	120	310500	324000
22	613	640	980800	1024000	112	117	302400	315900
23	594	618	950400	988800	108	113	291600	305100
24	573	595	916800	952000	105	109	283500	294300
25	552	572	883200	915200	101	104	272700	280800
26	531	549	849600	878400	97	100	261900	270000
27	509	525	814400	840000	93	96	251100	259200
28	488	501	780800	801600	89	92	240300	248400
29	466	478	745600	764800	85	87	229500	234900
30	444	454	710400	726400	81	83	218700	224100
31	423	431	676800	689600	77	79	207900	213300
32	402	408	643200	652800	74	75	199800	202500
33	382	386	611200	617600	70	71	189000	191700
34	362	365	579200	584000	66	67	178200	180900
35	343	344	548800	550400	63	63	170100	170100
36	324	323	518400	516800	59	59	159300	159300
37	305	303	488000	484800	56	56	151200	151200
38	286	283	457600	452800	52	52	140400	140400
39	266	263	425600	420800	49	48	132300	129600
40	239	235	382400	376000	44	43	118800	116100

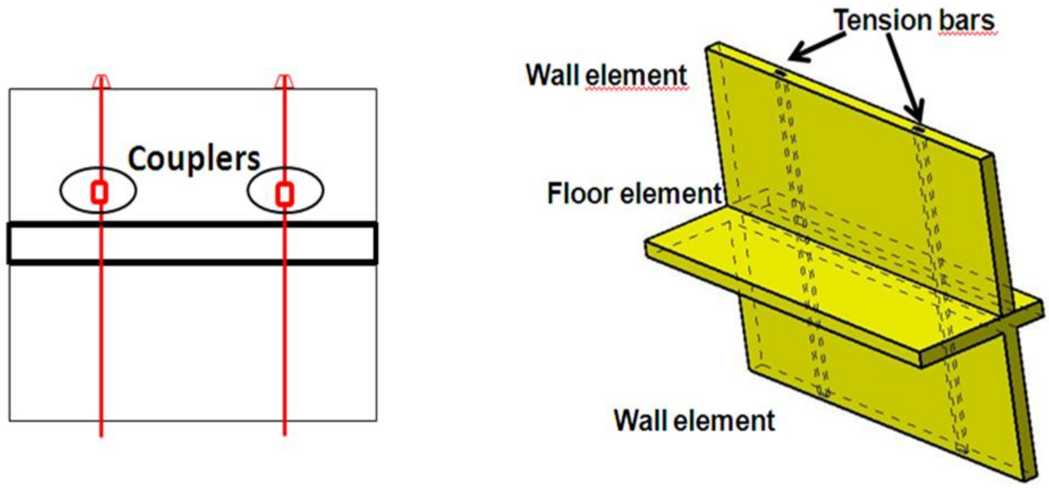


Figure 4: Steel bars planted in CLT elements

Uplift forces are evaluated only at stories where they are presented as tension force values. From the calculation of the temporary model, the tension forces exist only from foundation to 20th story, so that the steel bars should also be used in these stories. It is assumed that there are only two steel bars in one CLT wall which is located in Y-direction as shear wall (Figure 5(a)). One of these two bars is used to carry tension force in windward side and the other one at the meanwhile is taking compression force in the leeward side.

As yield strength and elastic modulus of steel are set as 600N/mm^2, the minimum diameter of one steel bar which is assumed to take all the uplift force in one story can be decided according to the following equation:

$$D = \frac{4F_t}{\pi\sigma_y} \tag{6}$$

where D (mm) is the steel bar diameter, T (N) is tension force, and σ_y (N/mm^2) is yield strength of steel.

Table 4 shows the diameter of steel bars which should be used at left and right side in Y-direction in every story, only tension force values are presented in the table. As Young's modulus of steel is set as 210000N/mm^2, the elongation of steel bar under tension force and its stiffness can be determined according to the Hooke's law.

Table 4: Calculated diameter of steel bars

Story Number	Left side			Right side		
	Bending moment KN-m	Tension force kN	diameter of steel bar mm	Bending moment KN-m	Tension force kN	diameter of steel bar mm
01	16074	1516	56.7	16611	1567	57.7
02	15288	1442	55.3	15720	1483	56.1
03	14369	1356	53.6	14706	1387	54.3
04	13377	1262	51.7	13624	1285	52.2
05	12359	1166	49.7	12523	1181	50.1
06	10973	1035	46.9	11076	1045	47.1
07	9854	930	44.4	9900	934	44.5
08	8799	830	42.0	8795	830	42.0
09	7814	737	39.6	7769	733	39.4
10	6894	650	37.2	6813	643	36.9
11	6033	569	34.8	5921	559	34.4
12	5223	493	32.3	5087	480	31.9
13	4462	421	29.9	4305	406	29.4
14	3747	354	27.4	3573	337	26.7
15	3078	290	24.8	2891	273	24.1
16	2453	231	22.2	2255	213	21.2
17	1871	177	19.4	1666	157	18.3
18	1333	126	16.3	1123	106	15.0
19	838	79	13.0	625	59	11.2
20	386	36	8.8	171	16	5.9

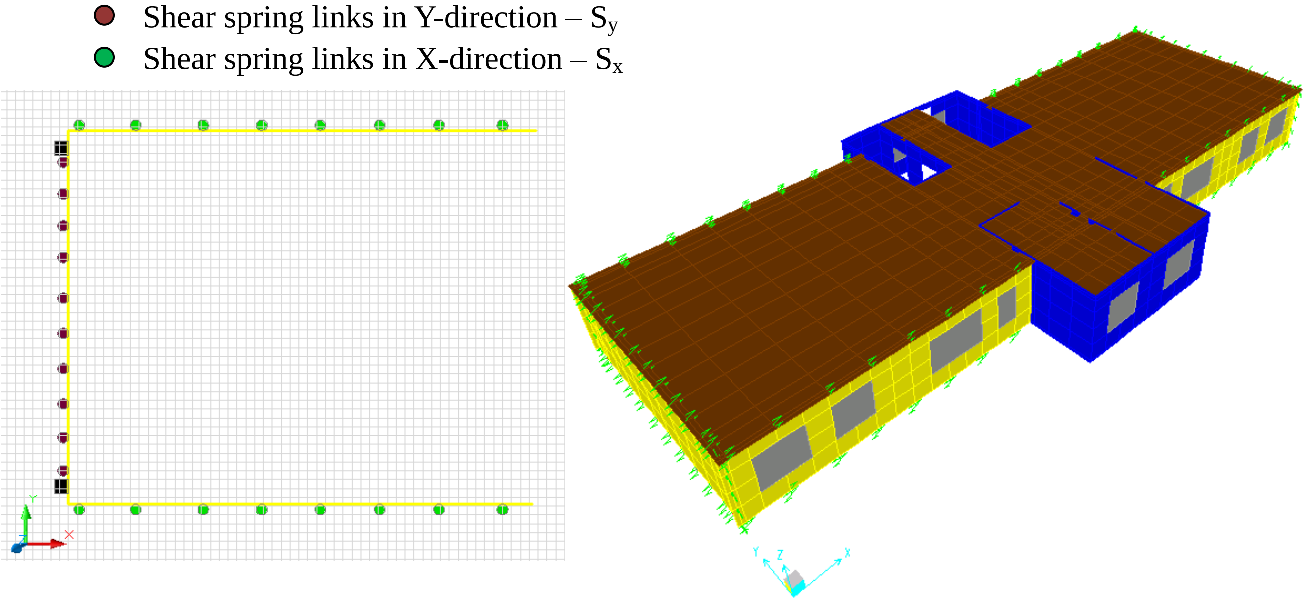
2.3.3 Simplification of Spring Links in the Model

With these stiffness values of mechanical connectors, a formal finite element model was developed for simulating the full-scale test. The model is composed of isotropic shell-thin elements of concrete walls, orthotropic shell-thin elements of CLT panels and connections represented by springs. The concrete walls are assumed as rigid, which continuous from base to the top of the building, and CLT floor diaphragm is also assumed as rigid so that floor element is an entire piece without spring links. CLT wall panels are connected by the linear spring links, the tension spring link “T” represents the steel bar from 0 to 20th level, shear springs used for connecting CLT wall and floor element are separately defined as “S_y” in Y-direction and “S_x” in X-direction (Figure 5), as well as the corner connectors of CLT walls is defined as “C” in the model.

As every story has hundreds of nails and screws in the real structure, it will lead to excessive workload and huge cost of gradient computation if the numerical model including every structural element. Therefore, simplification is necessary and inevitable. Because the static analysis of full-scale model focuses on the overall characteristics and deformation of the entire wood-concrete skyscraper, but not on each structural member, the number of connectors is then reduced, on the base of ensuring the equivalent total stiffness.

According to this assumption, the number of spring links is defined correspondingly in the numerical model as following: 2 tensile links and 10 shear links for each CLT shear wall which is located in Y-axis, 8 shear links for each in X-axis located CLT wall, and 5 corner links to connect every two orthogonal intersected walls. Their positions are the same in every story. In Figure 5 (a) and (b), arrangement of spring links is presented:

- ■ Tensile spring links – T
- ● Shear spring links in Y-direction – S_y
- ● Shear spring links in X-direction – S_x



(a) Layout of springs' placement of left half part (b) Elevation of one story

Figure 5: placement of spring links in numerical model

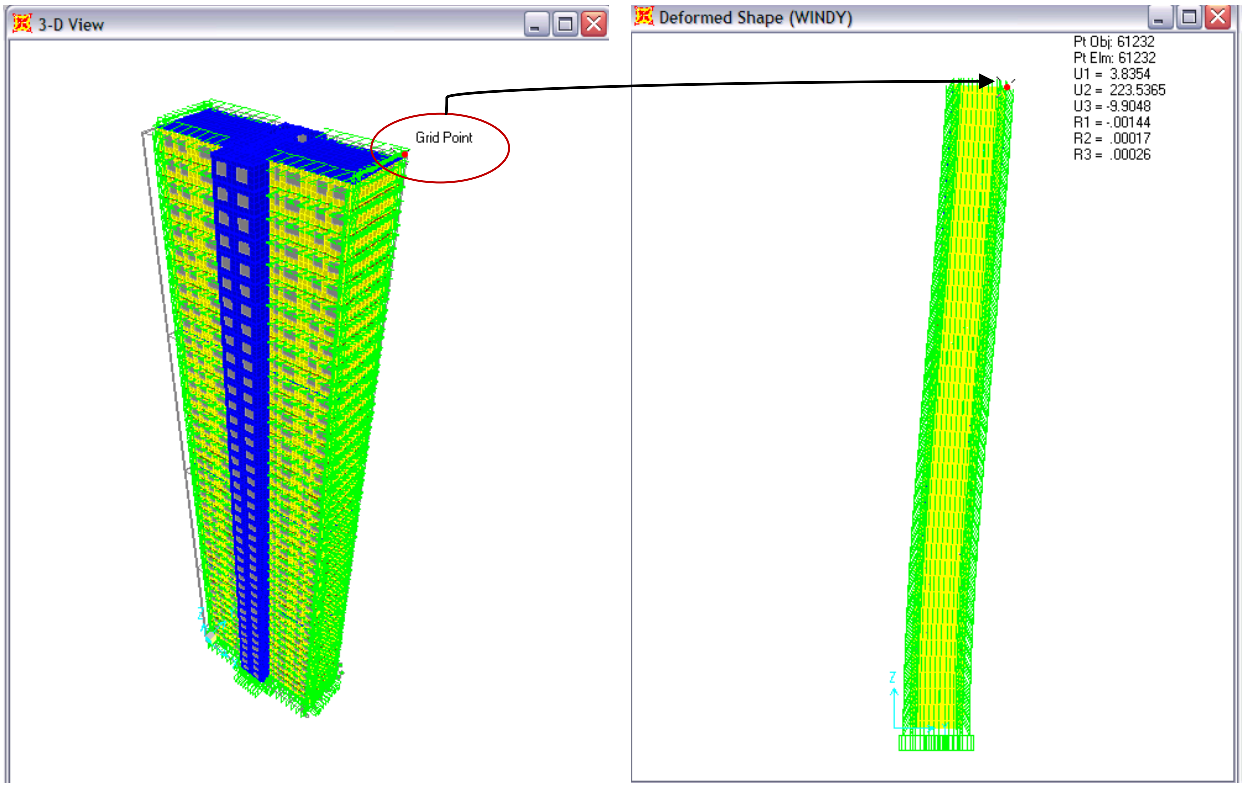
Based on the number of links, the stiffness of each link can be analytically determined. However, in order to further simplify the implementation of the model, every 10 story is regarded as one unit, the maximum value was utilized to define the shear and corner springs which are used in these 10 stories. In actual scenario, every ten stories should apply individual diameter of steel bars. But in simulation to consider the most adverse situation, tensile stiffness applies the maximum value of the calculated results. Table 4 shows the relevant stiffness data of spring links which are defined in the model.

Table 5: Values of stiffness of steel bars and shear connections taken into account as linear springs in the model

Story	T	S_y	S_x	C
	N/mm per link			
0-10	180000	130560	37000	80460
11-20	180000	124960	36700	76680
21-30	--	105600	29400	64800
31-40	--	68960	17500	42660

3. Analysis Results and Discussion

Static analysis of implemented model was executed in SAP2000. In order to compare the results, a control point of the location of X,Y,Z(33550, 13900, 131750 mm) is set at the top of the building (Figure 6(a)). One of the most important control factors of a skyscraper is the sway index. It is limited to certain maximum value which is given by building codes and regulations. In most codes, the sway index must remain smaller than H/500 [7]. Thus, the maximum deflection of designed building of 132m height should be less than 264mm.



(a) Control point (b) Deflection of the control point

Figure 6: Control point and its deflection under wind load

A concrete building with such structure was analyzed at first to be taken as comparison of results. The maximum deflection of this control point is 183mm under single wind load. It means that this designed structure can confirm the sway requirement if a building is constructed all by concrete. However, the deflection of the control point under same wind load condition is 313mm in the temporary model, and 436mm in the formal model which is developed with equivalent spring links, both of them violate the previous mentioned standard. And at the meanwhile, it also showed that the former model assumption is too loose when the second model with spring links reflects the reality better with bigger lateral deflection. Therefore, stiffness of the wooden-concrete skyscraper must be increased, and two methods were firstly attempted: doubled the value of springs' stiffness; and substituted concrete floor for CLT floor in 10th, 20th, 30th and 40th story. The deflection of the control point respectively reduced to 419mm and 391mm. It is obviously that the substituted concrete floor is much more effective than doubled stiffness, but these two deflections are still beyond the requirement.

Considering these two depressed results, an essential factor, or better, a general regulation of structure design should be sincerely taken into account at the moment: shear walls of a tall building should be typically comprised of discontinued thicknesses along the direction of the height. The effects of such variations can be a complex redistribution of the moments and shears between the walls[4].

Therefore, thickness-varying shear walls were introduced into the model to replace the original concrete walls which with uniformed thickness of 250mm. They were defined as 400, 300, 200 and 100mm thick, and respectively utilized in 0-10, 11-20, 21-30 and 31-40 story of the skyscraper. Having applied this modification, the sway of the control point was reduced to 224mm (Figure 6(b)) under single wind load, and 238mm under combined load of dead load, live load and wind load where the dead load was automatically calculated by program and live load was set as $2kN/m^2$.

Consequently, the sway ratio of whole building is represented of H/589, is controlled within the standard of H/500. If additional CLT interior walls, fasteners which can also resist the lateral load and increment stiffness of the construction are considered, the maximum deflection and sway ratio of the wooden-concrete skyscraper would even be furthermore limited.

4. Conclusions

As cross laminated timber (CLT) panels have extremely high in-plane stiffness and load carrying capacity, mechanical connectors become the relatively weak elements and their performance plays a decisive role in the control of wooden structure's deflection under wind load. During this short mission, several finite element models where linear spring links equivalently simulate the mechanical connectors have been implemented in SAP2000. Static analysis results of these models have given an insight into the wind load response of wood-concrete skyscraper. One of the most important conclusions is that, with concrete floors and variant thicknesses of concrete shear walls, the wood-concrete skyscraper would have the possibility to satisfy the sway requirement of high-rise building. And other conclusions through comparison of the results are as following:

- The designed construction skeleton could be properly applied for the wood-concrete skyscraper and accurately be analyzed for lateral load.

- Model where the CLT panels are connected by spring links has larger deflection than the model where the CLT panels are assumed to be glued together.

- Increasing the value of spring stiffness (i.e. add more fasteners in real structure) has no much help to reduce the deflection. Comparatively, four concrete floors which are located in every ten story are much more effective to resist lateral load than doubling the spring stiffness.

- Variation of thickness of concrete shear walls has performed a high capability to resist the lateral load through redistributing the moments and shears forces. It is quite a promising aspect which should be seriously considered in following research.

5. Future Cooperation and Research

Future cooperation between CNR-Ivalsa and Holzforschung München would focus on dynamic response of the wooden skyscraper. Findings and analyses from this work will be used to define further research needs. And the results will be presented in a scientific paper in the near future.

Certainly, the referring research of wood-concrete skyscraper hasn't completed yet, more details need to be concerned and analyzed. For example, in this stage, the connection between CLT panels and concrete members is still set as rigid in the model, but in the real situation, they joint also by mechanical fasteners where the shear deformation will be happened due to lateral load. Therefore, the model should be correspondingly further modified in the next step. Besides, dynamic analysis such as modal, seismic and time history analysis should be carried out in the mathematical full-scale model to validate and certify the feasibility of wood-concrete skyscraper.

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