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EARTHQUAKE RESPONSE ANALYSES OF A TALL BUILDING CONSIDERING IN-PLANE FLOOR STIFFNESS

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Abstract

In the design of a tall building structure, it is generally assumed that the stiffness of the in-plane floor structure is rigid. This is because the floor structure is generally reinforced concrete, and thus, in-plane stiffness of the floor structure is large enough for assuming it as a rigid floor. On the other hand, recently, there have been constructed some buildings with relatively flexible floor panels as a Cross Laminated Timber (CLT) in Japan. In the case that some frames with different horizontal stiffnesses are connected by flexible floor panels, the frames behave differently against earthquake ground motions. It is necessary to grasp the effect of the floor panel's flexibility in such a case. Of course, there are many studies which examined the seismic responses of the structures considering in-plane flexibilities of floor panels. However, there is almost no research which treated general response tendencies systematically.

In this paper, the authors examine the earthquake response characteristics of a tall building with flexible in-plane floor panels by parametric response analyses. Here, the target models are the structures which are composed of flexible frames connected by floor panels with rigid frames, And response characteristics are examined systematically, by conducting earthquake response analyses using 2-DOF models and multi-DOF models. The results of the analyses show the effect of floor flexibility to earthquake response behaviors, and the rigidity which gives the same responses of the flexible frames and the rigid frames. It is confirmed that the timber floor panel with the depth of 210mm behaves almost as same as rigid floor structure. And it is also discussed about the response behaviors affected by the in-plane floor flexibilities by adding dampers.

Keywords: Earthquake response analysis; Tall building; In-plane floor stiffness; MDOF model

1. Introduction

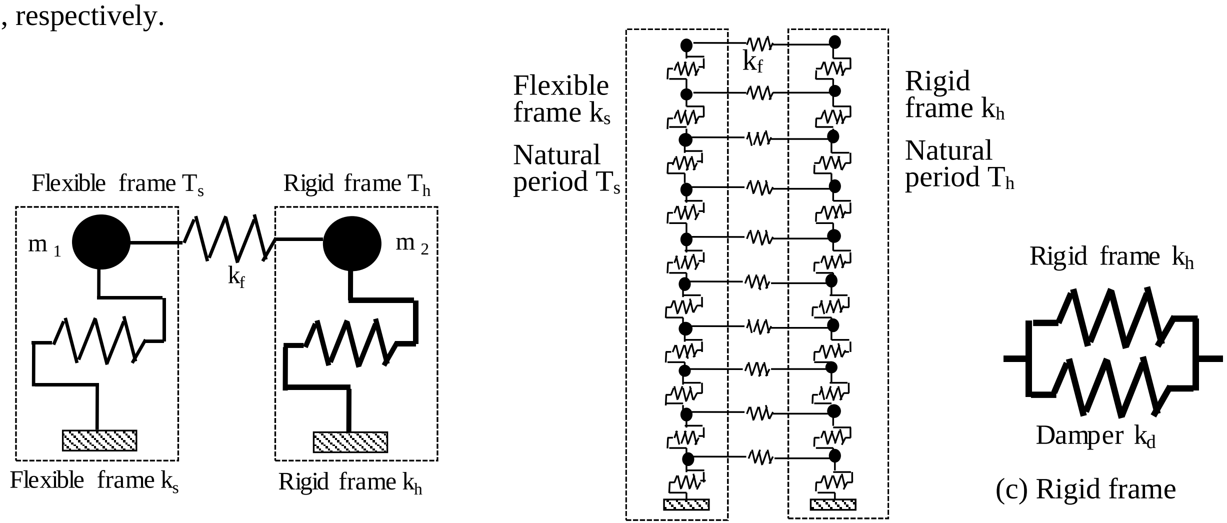
Building structures whose plan shapes are slender, a variety of atriums and low-rise wooden structure often have low in-plane stiffness floors, and they are assumed not to be the rigid floor assumption. For example, in low-rise building steel structures such as factories and gymnasiums, horizontal braces are installed to secure in-plane roof stiffness. Particularly, regarding to earthquake responses of the building structure with large horizontal aspect ratio, the behaviors of central roof structure in longitudinal direction are different from the one of roof edge. On the other hand, recently, there have been constructed some building structures with relatively flexible floor panels such as Cross Laminated Timber (CLT) panels in Japan [1-4]. The response behaviors of the building structure with flexible floors such as CLT panels are considered to be different from the ones with rigid floors. In previous studies, various effects of diaphragm flexibility on dynamic properties and seismic responses of timber structure are reported. Building structures integrated flexible part with rigid part by flexible horizontal floor are also reported [5-8]. Authors also have studied the effect of in-plane stiffness of flexible floors to earthquake response of a tall building structure and reported basic response characteristics affected by parameters such as eigenvalues, weights, high eigenmodes [9,10].

In this paper, the earthquake response characteristics of a tall building with flexible in-plane floor panels are examined by conducting parametric response analyses. The response behaviors affected by in-plane floor flexibilities by adding dampers are also discussed.

2. Outline of analysis

2.1 Analysis models

The target model is a tall building structure which is composed of the flexible frames connected by floor stiffnesses with the rigid frames. Fig. 1 shows earthquake response analysis models. These models are 2-DOF Model and Multi-DOF Model assuming the 10th-layer structure. Floors connected between the rigid frame and the flexible frame are assumed as springs. Dampers are connected in parallel to the rigid frame of 2-DOF Models and Multi-DOF Models. Using these building models, earthquake response analyses are conducted to confirm the response behaviors of the models. Parameters are set as the in-plane floor stiffness k_f and with or without dampers. Natural periods of the flexible frame and the rigid frame are expressed by T_s, T_h, respectively.



(a)2-DOF model (b)Multi-DOF model (c) Rigid frame with damper

Fig. 1 Response analysis models

Masses of 2-DOF model are set as m_1=m_2=2000t (Total mass:4000t) and masses of each layer in Multi-DOF model are set as 300t (Total mass:6000t) respectively. These models with rigidity k_s and k_h in the vertical plane are assumed to be connected by in-plane floor spring k_f. Natural period of the flexible frame is set as T_s=1.2sec and the one of the rigid frame is set as T_h=0.8sec. Lateral stiffness distribution of the vertical frame is defined that the 10th-layer is a half of the 1st-layer assuming to be trapezoid distribution. The base shear coefficient Co of the flexible frame is set as 0.2, and the one of the rigid frame is set as 0.3. The shear strengths of the vertical frame depend on the Ai distribution. Hysteresis characteristics of the springs (k_s, k_h) in these models are assumed as a normal bilinear type and the 2nd-stiffnesses are assumed as 0.02 of the initial stiffnesses. In-plane floor springs k_f are assumed to be the elasticity. Here, two types of the models are taken as the simulations. Type1 indicates the basic model using the flexible frame connected by the floor stiffness to the rigid frame, and Type2 indicates the one adding dampers to the rigid frames. Dampers of Type2 corresponds energy absorption members such as axial hysteretic dampers. Damper stiffness k_d is equal to 200 kN/mm, and damping factor of dampers is set as h=0.001 referring to proportional to initial rigidity.

2.2 Earthquake Input motions

Earthquake Input motions are shown in Table 1. The observed ground motion records are normalized that the peak velocity is 50 cm/s.

3. Results of analysis

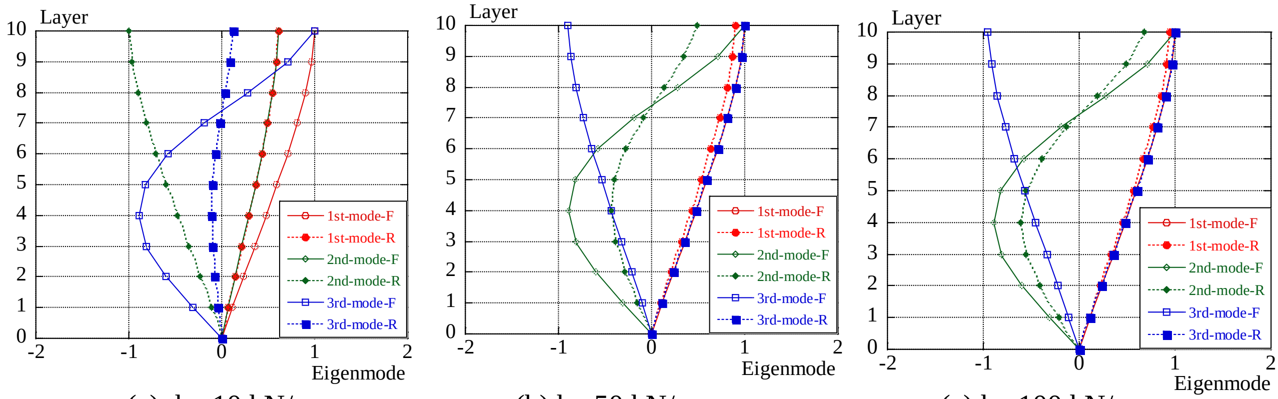
3.1 Eigenmodes of the multi-DOF model

Natural periods of the vertical frame are set as T_s=1.2sec in the flexible frame, and T_h=0.8sec in the rigid frame, respectively. In case of the condition that in-plane floor stiffness (k_f=10 kN/mm, 50 kN/mm, 100 kN/mm) is uniform, the eigenmodes of Type1 are shown in Fig.2. Here, the mark 'mode-F' indicates the flexible frame and 'mode-R' indicates the rigid frame. The high eigenmodes of the multi-DOF model change by increasing floor stiffnesses.

When the floor stiffness increases degree by degree, the eigenmodes of the flexible frame become close to the ones of the rigid frame. In case of k_f=100 kN/mm, 2nd-eigenmode and 3rd-eigenmode of the flexible

Table 1 Earthquake input motions

Earthquake motion	Nomalized velocity cm/s	Nomalized acceleration cm/s^2
EL Centro NS(1940)	50	510.8
Taft NS(1952)		485.7
Hachinohe NS(1968)		330.1

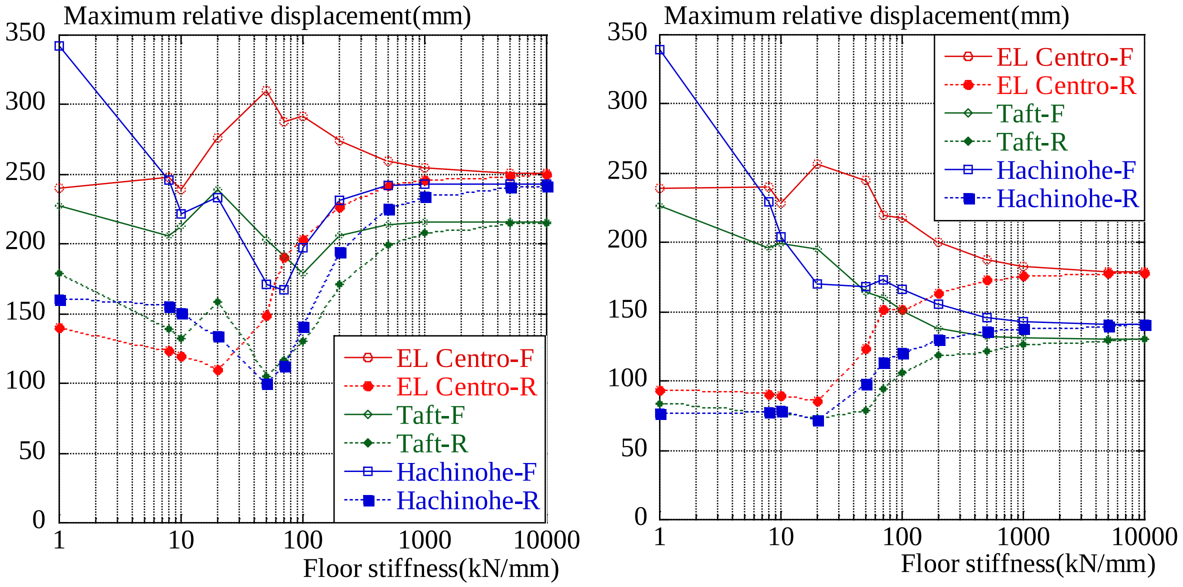


(a) k_f=10 kN/mm (b) k_f=50 kN/mm (c) k_f=100 kN/mm

Fig. 2 In-plane floor stiffnesses and eigenmode frame also change largely.

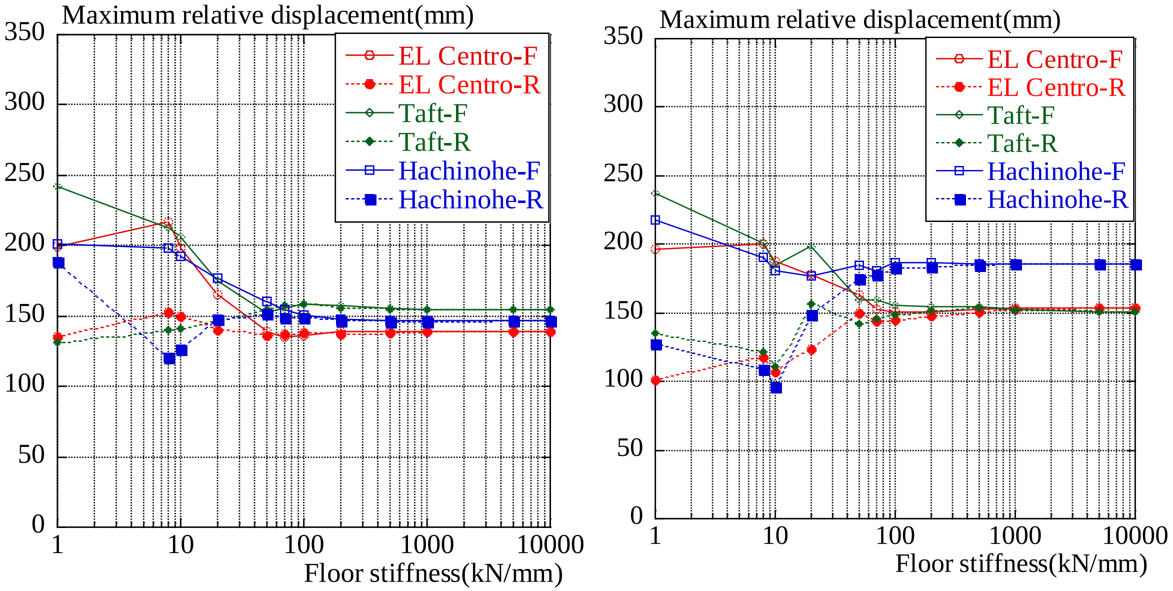
3.2 Maximum response and floor stiffness

Maximum relative response displacements and floor stiffnesses of Type1 and Type2 using 2-DOF model are shown in Fig.3(a),(b), respectively. Here, the mark 'El centro-F' indicates the flexible frame and 'El centro-R' indicates the rigid frame. Maximum relative response displacements of the flexible frame become close to the ones of the rigid frame by increasing the floor stiffness in all earthquake motions, and they approximately match the ones of the rigid frame when the floor stiffness k_f exceeds 1000 kN/mm. Maximum relative response displacements and the floor stiffness of 10th-layer of Type1 and Type2 using multi-DOF model are shown in Fig.4(a),(b), respectively. Maximum relative response displacements of the flexible frame also become close to the ones of the rigid frame by increasing the floor stiffness in all



(a)Type1 (b) Type2

Fig. 3 Maximum response relative displacements of 2-MOD model



(a)Type1 (b) Type2

Fig. 4 Maximum response relative displacements of multi-MOD model at the 10th-layer earthquake input motions. These tendencies of Type1 are approximately similar to the ones of Type2. When the floor stiffness k_f exceeds 50 kN/mm, maximum relative response displacements of the flexible frame of Type1 approximately match the ones of the rigid frame. On the contrary, in Type 2, when the floor stiffness k_f exceed 80 kN/mm, maximum relative response displacements of the flexible frame approximately match the ones of the rigid frame. Namely, though maximum relative response displacements of Type2 of multi-DOF models are slightly smaller than ones of Type1, the range of the rigid floor assumption of Type2 slightly shrinks than that of Type1.

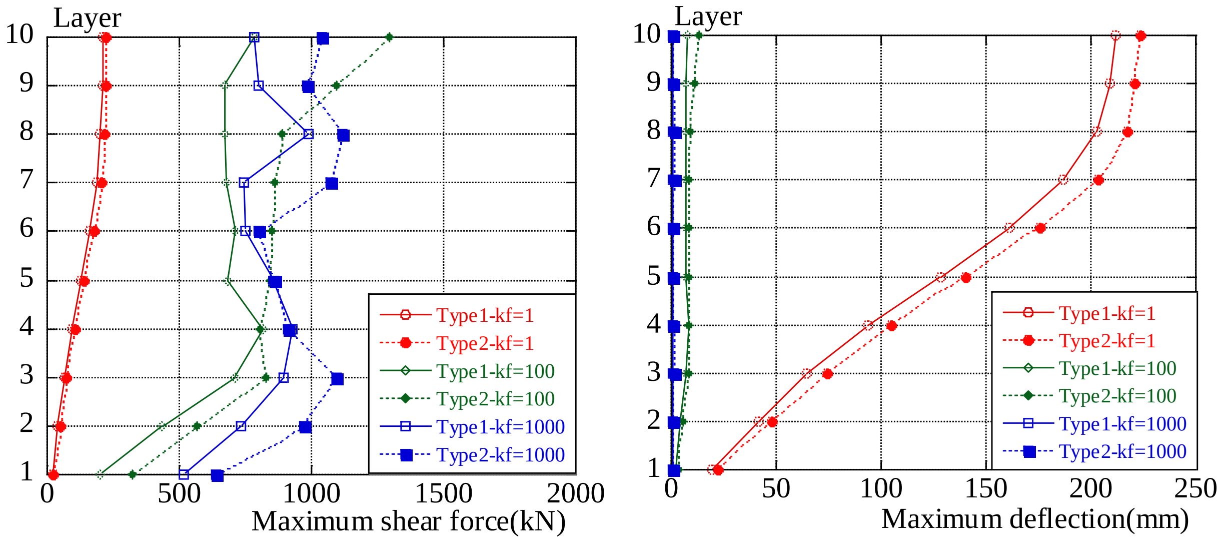
Maximum relative response displacements of multi-DOF model of Type1 and Type2 have smaller fluctuation than the ones of 2-DOF model. However, overall earthquake responses can be approximately confirmed by 2-DOF model.

3.3 Maximum responses of multi-DOF model

Maximum response shear forces and deflections of each floor of Type1 and Type2 of multi-DOF model are shown in Fig.5(a),(b), respectively. Here, the parameters of the floor stiffness k_f is set as 1 to 1000 kN/mm using only El centro NS wave. Maximum response shear forces of Type1 and Type2 fluctuate when the floor stiffness k_f exceed 100 kN/mm, and maximum response shear forces of Type2 are larger than to the ones of Type1. Maximum response shear forces of Type1 are approximately similar to the ones of Type2 when the floor stiffness k_f is equal to 1 kN/mm, and each of them gradually increases on upper floors. Fluctuation of them is smaller than the ones of k_f =100 kN/mm and 1000 kN/mm.

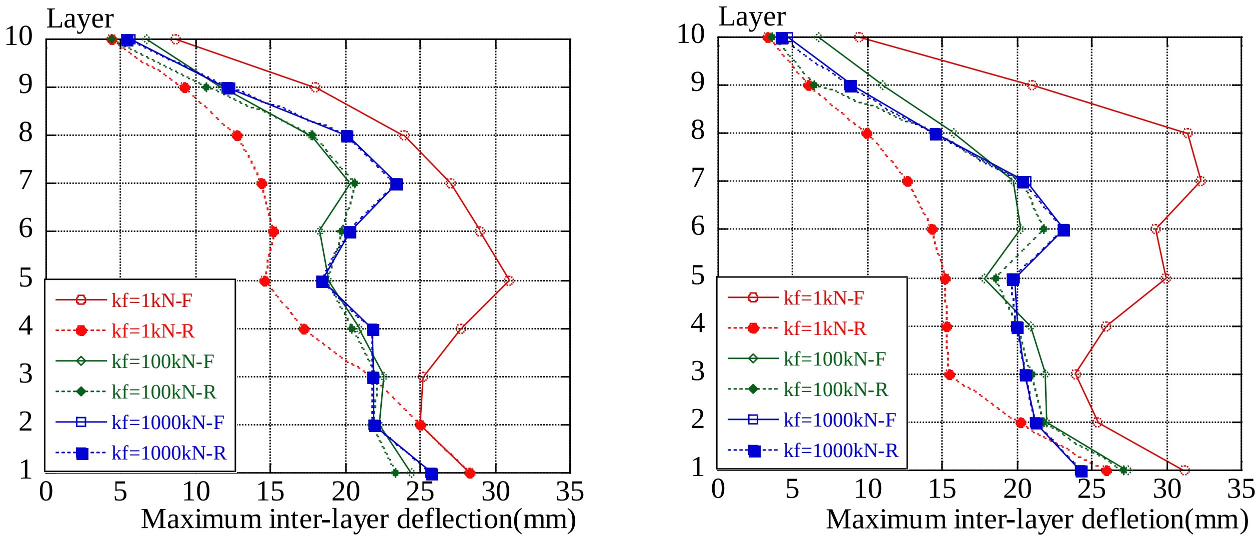
On the contrary, Maximum response deflections of Type1 and Type2 gradually increase on upper floors in case of k_f=1 kN/mm, and maximum response deflections of Type2 are slightly larger than the ones of Type1. Maximum response deflections of Type1 and Type2 hardly fluctuate when the floor stiffness k_f exceeds 100 kN/mm, and maximum response deflections of Type2 are approximately similar to the ones of Type1. Namely, in this case, the condition that the floor stiffness k_f approximately exceeds 100 kN/mm is considered to be the limit range of the rigid floor assumption.

Maximum inter-story deflections of each floor of Type1 and Type2 of multi-DOF model are shown in Fig.6(a),(b), respectively. Maximum inter-story deflections of the flexible frame and the rigid frame with k_f=1 kN/mm fluctuate near mid-layer of Type1 and Type2. Particularly, maximum inter-story deflections of the flexible frame are largely different from the ones of the rigid frame near mid-layer. On the contrary, when the floor stiffness k_f exceeds 100 kN/mm, maximum inter-story deflections of the flexible frame are similar to the ones of the rigid frame.



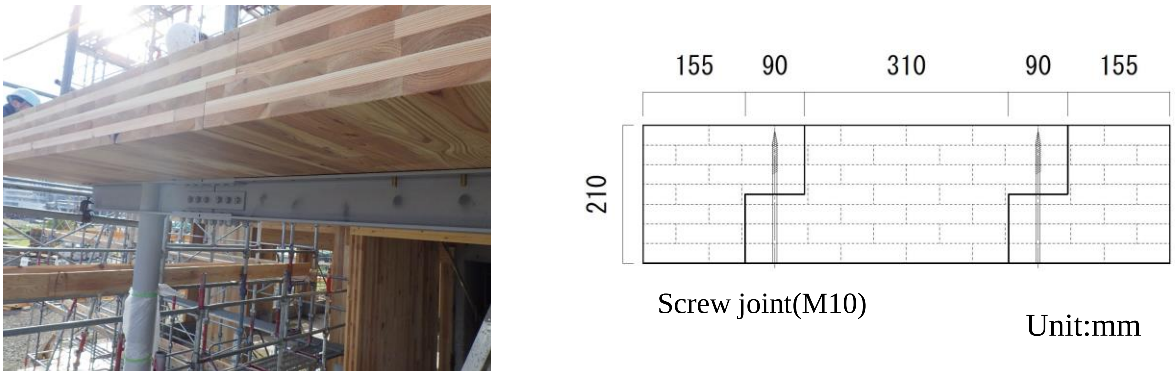
(a)Shear force (b) Deflection

Fig. 5 Maximum responses of floor stiffnesses(El centro NS motion)



(a)Type1 (b) Type2

Fig. 6 Maximum inter-layer deflections of multi-MOD model(El centro NS motion)



(a) Construction (b) Detail

Fig. 7 Screw joint between CLT floor panels

3.4 In-plane floor stiffness of CLT floor panels

In previous studies, shearing modules are reported to be equivalent to 400-500 N/mm^2, and shearing strengths of CLT panels such as cedars, pines are equivalent to 5.4, 6.0 N/mm^2, respectively [11,12]. At the 1/100 shearing displacement in this performance, as the shearing forces of CLT panels are equivalent to 4-5 N/mm^2, they are 10-15% lower than the ultimate shearing strength of CLT. According to Fig.4, when the floor stiffness k_f exceed 50 kN/mm in Type1, 80 kN/mm in Type2 respectively, maximum relative response displacements of the flexible frame match the ones of the rigid frame. When the CLT floor panel with depth of 210 mm fixed to surrounding beams whose the building structure is under the small aspect ratio condition are used, the floor stiffness k_f is approximately equivalent to 80-100 kN/mm using shearing moduluses. Therefore, it is assumed that the CLT floor panels approximately behave as rigid floor structure.

On the other hand, the long screw joint between CLT floor panels is shown in Fig.7. CLT members would be composed with 7layer-7ply Japanese cedar, its thickness is 210mm, grade Mx60 in Japanese Agricultural Standards. Ultimate shear strength and yield strength of screw joint (M10, PX10-200, length:200mm) connected to CLT panels are approximately 11 kN, 6 kN by shearing test (Average of five specimens). These joint pitches both floor panels connected are selected within yield shear strength using allowable stress design. The bolt joint of the CLT floors connected to steel beams should be also selected within yield shear strengths.

4. Conclusions

In this paper, the earthquake response characteristics of a tall building structure are discussed considering in-plane floor stiffness by conducting parametric response analyses. The major findings obtained from the study are as follows:

- When the floor stiffness approximately exceeds 100 kN/mm, maximum inter-story deflections of the flexible frame become similar to the ones of the rigid frame.
- When the CLT floor panels with depth of 200 mm fixed to surrounding beams whose the building structure is under the small aspect ratio condition are used, it can be assumed that the CLT floor panels approximately behave as rigid floor structures.
- The rigid floor zone between the flexible frame and the rigid frame slightly shrinks adding dampers to the structures.

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