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STUDY ON INTEGRAL PERFORMANCE OF A MODERN LOW-RISE LIGHT-WOOD FRAME STRUCTURE BUILDING

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ABSTRACT:
Light-wood frame structure system came into being in North America in middle period of 19th century and has a history of more than one hundred years. At present light-wood frame structure system is widespread in many countries and already became an important structure system. So far, researches on the structure system mainly focus on computing and testing of structure members, and study on the integral performance including seismic performance and shear performance is comparatively few. For the integral structure, modes, story drifts, internal forces of the members under the loads such as seismic loads are important and necessary which can only be obtained through integral analysis of the whole structure instead of computing and testing of a single member. In the paper finite element analysis of integral performance of a modern low-rise light-wood frame structure building in Beijing is carried out using finite element software based on the design criteria in China. Then the modes of the structure are solved by modal analysis and internal forces and deformations in appointed load cases which are the combinations of loads including seismic loads are also solved. Further, seismic performance and shear performance of the structure are analyzed. The load case results show the internal forces and displacements of the structure meet the requirements of design criteria in China and the structural safety margin is ample. It indicates the design of the frames has an appropriate security. Modal analysis results show the structure has a good integrity and potential weak positions of the structure can also be found. The light-wood frame structure system has a good seismic performance because of its light weight and big damping ratio. The results of the paper validate the viewpoint sufficiently. Contours of shear forces of the wall in wind loads and seismic loads are also given in the paper. The results indicate the structural methods of interior walls can meet the requirements of shear resistance, while shear forces of partial exterior walls exceed the design strength of shear resistance. So construction methods such as using thicker wallboards or adding steel members should be adopted to strengthen the positions.

KEYWORDS:
light-wood frame structure system; integral performance; seismic performance; shear performance; finite element analysis

1. INTRODUCTION

Light-wood frame structure system came into being in North America in middle period of 19th century and has a history of more than one hundred years. At present light-wood frame structure system is widespread in many countries and already became an important structure system. In USA, timberwork house takes a dominant place and the percentage of the wastage of wood in the cork output is 60% [1]. In Canada, timber industry is one of the supporting industries, and the industrialization, standardization and building technology are very ripe. In the north Europe such as Finland and Sweden, 90% of the houses are one-storied or two-storied light wood structure [2].

At present, the study on light wood frame structure system is relatively lagged. What is more, the available study is focused on the computations and tests of the light wood members, and is lack of integral performance study [3][4]. As to the whole structure, the mode, story drift, internal forces of the members and the shear forces of the wall are important information, which can only be obtained by integral analysis instead of the computations and tests of the members.

In the paper finite element analysis of integral performance of a modern low-rise light-wood frame structure building in Beijing is carried out using finite element software based on the design criteria in China. Then the

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modes of the structure are solved by modal analysis and internal forces and deformations in appointed load cases which are the combinations of loads including seismic loads are also solved. Further, seismic performance and shear performance of the structure are analyzed. The results and conclusions of the paper can be a supply to the scholars and designers.

2. LIGHT WOOD FRAME STRUCTURE SYSTEM AND THE PROJECT

The light wood frame structure system is different from the log structure system and the beam-column structure system. The system is a space box shape system, which applies the regular wood with little cross section and panels densely and equally covered. The typical shape is shown in Figure 1. The space of the regular wood is 400-610 mm, which is the main bearing member of the structure and supplies workspace for installing the heat preservation and insulation layer and traversing pipes. The bearing capacity, stiffness and integral performance are realized by the cooperation of the frames and the panels. The members of the structure can work together, so the reliability, rationality of the system are both better than the beam-column structure system. The detailed structural methods of the structure including the roof methods, the floor methods and the wall methods can refer to available papers [2][5][6][7].

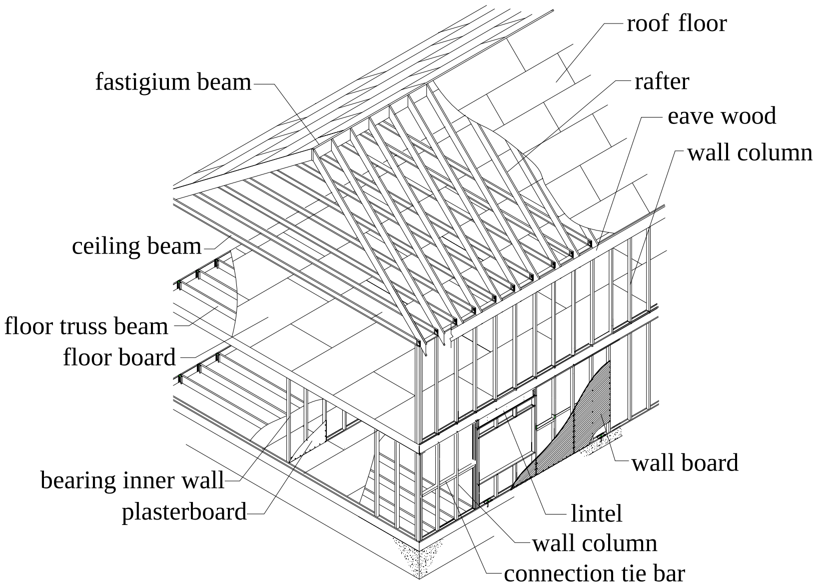


Figure 1 Typical shape of light-wood frame structure system

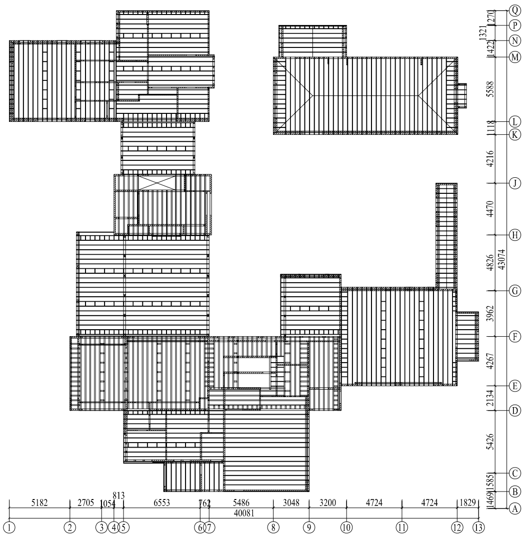
The light wood frame structure building is in the Shunyi District in Beijing. The construction area is about 1300 m^2, which consists of two parts in the space. The first floor includes the kitchen, super market, coffee house and reception hall. The second floor includes the gymnasium and the tea house. The third floor exists in partial position. The story height varies from 2.8 m to 3.4 m. Figure 2 shows the construction plain and architectural elevation. The following three characteristics can be obtained through the Figure 2.

(1) The third floor exists between the axes 4, 7 and H, J, so whipping effect may occur in the positions.

(2) Staggered-floor structure exists. For example, the elevation at the bottom of the first floor between the axes L and Q is -1.524 m, while the value at the other positions is ±0.000 m.

(3) Several rooms with big space exist, such as the room between the axes 10, 12 and E, G (10 m×8.5 m), so local modes may occur at an early time.

The light wood structure building has been completed, as is shown in Figure 3.



(a) Construction plan of first floor

(b) Architectural elevation from axis Q to A

Figure 2 Construction plan of first floor and architectural elevation

Figure 3 The light wood structure building after finished

The standard wood used in the project is imported from Canada, which is a complex material by spruce, pine and fir. So it is classified as the Spruce-Pine-Fir from the degree of species. The wood is mainly used at the positions of wall column, floor beam, lintel and roof frame. The thickness of the SPF cross section is 38 mm and the height of it varies from 64mm to 285 mm. The frames of the structure apply the SPF of 38×89 mm or 38×140 mm mainly.

The mechanical stress and material properties may be set according to the values of the I_c—VII_c degree of the Spruce-Pine-Fir in North America. In the paper, the material properties and design strengths are set according to the references [8] and [9]. The Code for Design of Timber Structures in China (GB 50005-2003) prescribes the size adjusting coefficient should be multiplied for the design values of the strengths and elastic modulus. The material properties used in computations are shown in Table 1, which conservatively apply the size adjusting coefficient of the cross section height of 140 mm.

Table 1 Material properties and design strengths of SPF wood

Density /kg·m^{-3}	Elastic modulus /MPa	Poisson ratio	Damping ratio	Bending strength /MPa	Compressive strength parallel to the grain /MPa	Tensile strength parallel to the grain /MPa
450	9700	0.35	0.05	12.2	13.2	6.2

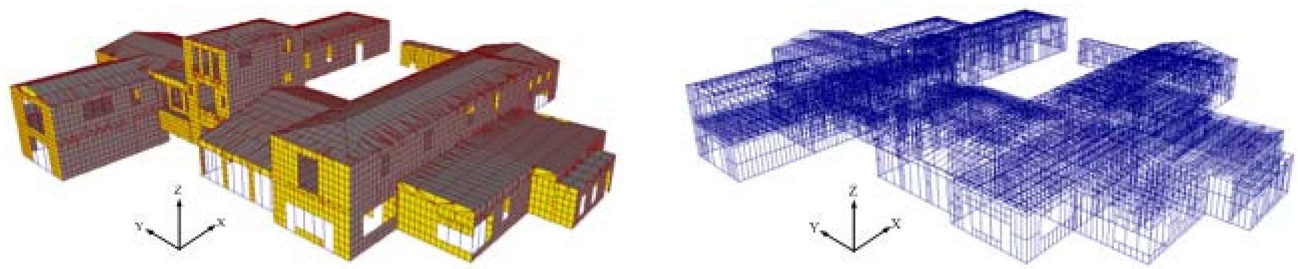
In the project the common wall columns in the inner walls and the outer walls apply the SPF of 38×89(140) mm and 38×140 mm respectively. The corner columns in the inner walls and the outer walls apply the combinations of three SPF wood of 38×89(140) mm and 38×140 mm respectively. The upper and bottom chord bars and web members of the roof apply the SPF of 38×89 mm, and the joist, reinforcing columns at the holes of the door and windows and lintel support column are designed according to the real span and loads. Besides, the plasterboard and OSB board are used in the roof, floor and wall. The material properties of the boards are chosen according to the reference [10], as are shown in the Table 2.

Table 2 Material properties of OSB wallboards and plasterboards

Material	Elastic modulus /MPa	Material strengths /MPa	Poisson ratio
Plasterboard	1124.7	0.66 (Vertical fracture strength)	0.23
OSB board	3500	7.86 (Bending strength perpendicular to the board length)	0.30

3. THE FINITE ELEMENT MODEL AND THE COMPUTATION RESULTS

Integral performance analysis of the structure is performed using the universe finite element software SAP2000. The beam element is applied to simulate the frame members and the plate element is applied to simulate the board members. The slippage between the wall column and the wall is not considered when modeling. The coupling connection is applied between the beam element and the plate element, that is to couple the translation degree of x, y and z and to release the rotational degree. The wall consisted of the wall column, the plasterboard and the OSB board can resist the horizontal forces, while the vertical loads are only resisted by the wall column. So the shear stiffness (the stiffness in plain) is considered, while the axial stiffness is not. In the numerical model, the property modification factor of the vertical forces is set to be zero as to ignore the axial stiffness. The two sides of the corner column are set to be rigid for the joints are relatively reliable and the other wall columns are set to be hinged. The finite element model by the software SAP2000 is shown in the Figure 4.



(a) the finite element model (b) the frames of the structure

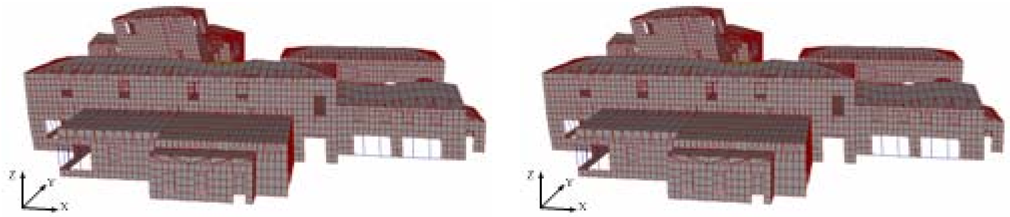
Figure 4 the finite element model of the light wood structure building

3.1. *Mode analysis*

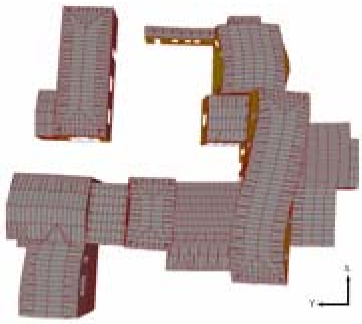
The modes, natural frequency and natural period are the inherent properties of a structure, which can reflect the dynamic properties of it. As the dimension of the structure is relatively big, the first 100 ranks of the modes and the natural periods are solved to ensure the participation of enough integral modes in the finite element analysis. Partial integral modes are shown in the Figure 5.

(a) The second rank mode (the vibration along the same direction of y)

(b) The sixth rank mode (the vibration along the opposite direction of y)



(c) The twelfth rank mode (the vibration along the same direction of x)



(d) The fifteenth rank mode (the integral torsion)

Figure 5 Partial integral modes

The second, sixth, twelfth and fifteenth rank natural periods of the structure are 0.114 s、0.100 s、0.088 s and 0.085 s which correspond to the vibration along the same direction of y, the vibration along the opposite direction of y, the vibration along the same direction of x and integral torsion respectively. The vibration at the third floor is relatively big, so it is indicated that the whipping effect exists in the positions. The influence of the big space room is shown in the modes. For example, the first rank natural period is 0.122 s, which is the vibration up and down between the axes 10, 12 and E, G. What is more, in the other positions with big space room, local modes occur early because of no supports inside.

It is indicated that the integral performance is relatively good as the integral modes obtained from the numerical analysis occur early. At the same time, it can be inferred that the third floor and the big space room are the weak positions, which should be strengthened.

3.2. Internal forces and deformations of the structure in various load cases

The Load Code for the Design of Building Structures in China (GB 50009-2001) prescribes that the structure design should make combinations according to the ultimate bearing capacity and the ultimate normal use respectively. For the former, the following three combinations should be considered.

(1) Load combination S1: 1.2×dead loads+1.4×live loads on the roof and floor; (2) Load combination S2: 1.2×dead loads+1.0×1.4×live loads on the roof and floor+0.6×1.4×wind loads: (3) Load combination S3: 1.2×self weight loads+1.3×earthquake action.

The basic wind pressure is 0.45 kN/m^2 and the wind loads from the four directions are considered respectively. In the load combination S3, response spectrum method is used to take the earthquake action into account and the earthquake actions from the directions of x and y are considered respectively. According to the Code for Seismic Design of Buildings in China (GB 50011-2001), the seismic fortification intensity of Beijing is 8 degree, and the design peak ground acceleration is 0.20 g. The structure is in the first seismic group and the ground is the III degree. So according to the code, the maximum of the horizontal earthquake coefficient $\alpha_{\max}$=0.16 and the initial period T_g=0.45 s. The maximum values of the axial forces of various kinds of wall columns are shown in the Table 3.

The bearing forces in the Table 3 are solved according to the Code for Design of Timber Structures in China (GB 50005-2003) and the reference [8]. Taking the common wall columns of the SPF of 38×89 mm, the solution method is introduced in the above. The maximum story height is 3.4 m, so the maximum height of the inner walls is less than 3m because of the existence of the floor truss beam (the height 407 mm). Set the computational length of the common wall columns of inner walls h=3 m, and set the initial eccentricity distance 0.05h. According to the admitted ultimate axial forces of the SPF of 38×89 mm from the reference [8], the design value of the axial force of the common wall columns of inner walls can be solved to be 9.053 kN. And the design values of the other kinds of the wall columns can be solved in the same way.

For the outer wall columns, the design values of the axial forces are solved according to the stability and the deflection limitation of the compression-flexure members, also considering the lateral wind forces, while the inner wall columns are considered as axial pressed members. So for the same member, the design value of internal forces is less when it is adopted as the outer wall column than the inner wall column. Otherwise, the connection tie bars exist along the middle of the main wall, which can decrease the computational length of the wall columns to some degree. And the wall board has the restriction influence on the wall column, so the design values in the Figure 3 is a little conservative in fact.

Table 3 Maximum axial forces of the columns

Load cases	The common wall columns in the inner wall		The corner columns in the inner wall	The common wall columns in the outer wall	The corner columns in the outer wall
	38×89mm SPF N_{max}/kN	38×140mm SPF N_{max}/kN	three 38×89mm SPFs N_{max}/kN	38×140mm SPF N_{max}/kN	three 38×140mm SPFs N_{max}/kN
S1	-8.14	-6.94	-18.28	-15.33	-32.60
S2 (wind load along the *x*+ direction)	-8.12	-6.97	-18.29	-15.17	-30.70
S2 (wind load along the *x*- direction)	-8.13	-6.84	-19.04	-14.57	-30.95
S2 (wind load along the *y*+ direction)	-8.20	-6.66	-15.32	-14.51	-28.26
S2 (wind load along the *y*- direction)	-7.99	-7.16	-20.82	-15.04	-33.12
S3 (seismic load along the *x* direction)	-7.17	-9.68	-14.26	-15.02	-30.22
S3 (seismic load along the *y* direction)	-6.08	-7.52	-17.72	-11.70	-32.60
Design values of bearing forces	-9	-29	-27	-17	-51

The Table 3 also denotes, the maximum axial forces of the inner wall corner columns and the outer wall corner columns both occur in the load combination S2, the maximum axial forces of the outer wall column occur in the load combination S1, the maximum axial forces of the inner wall columns of the two kinds of cross sections occur in the load combinations S2 and S3 respectively. So the necessity of the solution on many kinds of load combinations is proved. All the numerical results in the Table 3 are less than the design values of corresponding wall column and the safety repertory is ample.

As to the ultimate normal use, the following three standard combinations of loads are adopted:

(1) Load combination D1: 1.0×dead loads+1.0×live loads on the roof and floor; (2) Load combination D2: 1.0×dead loads+1.0×live loads on the roof and floor+0.6×wind loads; (3) As to the deformation under earthquake action, the standard values of the earthquake action is adopted, that is the load combination D3: 1.0×earthquake action.

The maximum displacement values in the three load combinations are demonstrated in the Table 4. Because the load combination D3 is concerned by the unique horizontal earthquake loads and has nothing to do with the other loads, so the values of vertical displacements are relatively small, which can be ignored compared with the other load combinations.

The maximum displacement occurs in the load combination D1, in which the displacement along the x direction is 1.760mm. So 1.760/3000 =1/1705<[w]=1/360, in which the value of [w]=1/360 is chosen according to the deflection limitation of wall columns using rigid decoration materials from the Code for Design of Timber Structures in China (GB 50005-2003). Because the maximum deflection is much less than the maximum horizontal displacement of the structure, the deflection can meet the demand. In the load combination D3 (earthquake action along the y direction), the maximum horizontal displacement is 1.236 mm, so the story drift angle is less than 1.236/3000=1/2427. The elastic story drift of the reinforcement concrete structures is 1/550, and no elastic story drift angle of the wood structure under little earthquake is prescribed. The story drift angle values solved in the paper are much less than 1/550, so we can consider it can meet the demand of story drift angle.

Table 4 Maximum values of the displacement

Load cases	Displacement along the x direction u_x/mm	Displacement along the y direction u_y/mm	Displacement along the z direction u_z/mm
D1	1.760	1.091	-5.844
D2 (wind load along the x+ direction)	1.670	1.088	-5.127
D2 (wind load along the x- direction)	1.442	1.091	-5.119
D2 (wind load along the y+ direction)	1.564	1.151	-5.169
D2 (wind load along the y- direction)	1.574	1.044	-5.170
D3 (seismic load along the x direction)	0.634	0.185	——
D3 (seismic load along the y direction)	0.152	1.236	——

3.3. Seismic performance of the light wood structure

The light wood structure system has the advantages of seismic bearing. Firstly the self weight of the light wood building is light. The dead loads on the floor and roof are about 0.8-1.1 kN/m^2 and the dead loads on the wall are 0.4-1.0 kN/m^2 for the light wood buildings. The values are about 1/5-1/3 of the masonry buildings. Secondly, the damping ratio of timber structure is about 0.05-0.1, which is the maximum among the four main structure systems including the steel structure, the timber structure, the reinforcement concrete structure and the masonry structures, so the timber structure has a strong energy dissipation capacity. The academic institutions had investigated several earthquakes, including the Alaska earthquake in 1964, the Northridge earthquake in California and the Osaka-Kobe earthquake in 1995. The investigations indicated that the timber structure can resist the peak ground acceleration about 0.6g, so the injuries to the inhabitants are relatively little. And the investigations also indicated that the timber structure system has the potentials to meet stricter control criterion.

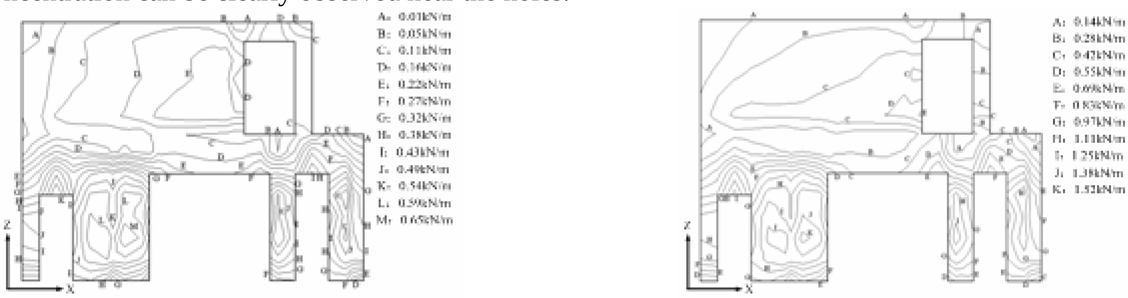
The seismic response of the light wood structure in the area of seismic fortification intensity 8 degree is solved by integral performance analysis. The results indicate the bearing capacity and deformation can meet the demand of the criterion and the safety repertory is ample. In the computation, the damping ratio of wood is conservatively set to be 0.05 and the reinforced effect of the decorating material to the walls is not taken into account. The computation results have ample safety repertories under the preconditions above, so the results in the paper verify the good seismic performance of the light wood structure system.

3.4. The shear performance of the light wood structure system

When the maintenance board has a reliable joint with the frames, it will have the shear stiffness in plain. Finite element analysis can give the shear forces on any wall to analysis the shear performance. Because of the construction process of the building, the wall bear horizontal forces mainly, so unique wind load or unique seismic load should be calculated.

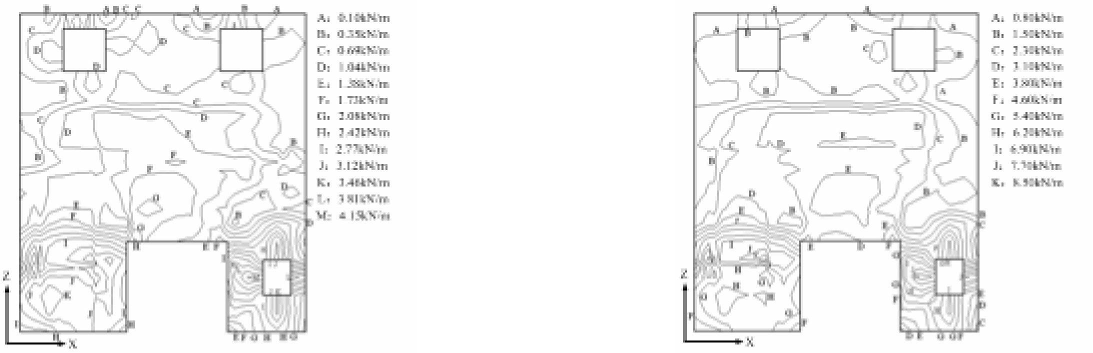
The wall boards for inner walls are two-layer 15 mm plasterboards, and the wall boards for outer walls are single-layer 15 mm plasterboard and 9 mm OSB board. Because the shear construction methods of the inner walls and the outer walls are different, the shear performance of the inner walls and the outer walls will be analyzed respectively. The inner walls below the axis J (two floors) and the outer walls above the axis J (three floors) are illustrated in the following. The shear forces of the two walls under the unique wind load and the unique seismic load are shown in Figure 6 and Figure 7 respectively.

It is indicated that the shear forces under the unique seismic load are greater than those under the unique wind load, and the shear forces of the ground floor are greater than the upper floors. Besides, because of the holes in the wall, the stress concentration can be clearly observed near the holes.



(a) under unique wind load (b) under unique seismic load

Figure 6 Contours of shear forces in wall located under axis J



(a) under unique wind load (b) under unique seismic load

Figure 7 Contours of shear forces in wall located on axis J

For the shear boards of inner walls, the results from the reference [10] indicate that the ultimate shear strength of single-layer plasterboard with the thickness 12 mm is 3.12 kN/m, so it can be concluded that the ultimate shear strength of two-layer plasterboard with the thickness 15 mm is 7.79kN/m. According to the allowable bending stress method [11], when the design of shear walls is carried out, the ultimate shear strength should be divided by the safety coefficient W. Under the wind load W=2.0 and under the seismic load W=2.5. So the design shear strength of the double-layer plasterboard with the thickness 15 mm is 3.90 kN/m and 3.12 kN/m under unique wind load and unique seismic load respectively. It can be indicated from the finite element analysis that the greatest shear force of inner walls is 0.69 kN/m and 1.56 kN/m under unique wind load and seismic load respectively, which can meet the demand of design strength. So it is appropriate to choose the two-layer plasterboard with the thickness 15 mm as the shear material of inner walls.

For the shear boards of outer walls, the results from the reference [10] indicate that the shear strength of complex walls consisting of the plasterboard with the thickness 12 mm and the OSB board with the thickness 9 mm is 12.13 kN/m. The ultimate shear strength of the outer walls applies the value conservatively in the paper. In the same solution way, the design shear strength of the outer walls is 6.07 kN/m and 4.85 kN/m under unique wind load and unique seismic load respectively. It can be indicated from the finite element analysis that the greatest shear force of outer walls is 4.19 kN/m and 8.85 kN/m under unique wind load and seismic load respectively. The Figure 7 illustrates that the shear force of outer walls is less than the design shear strength, while under the seismic load the shear forces of the first floor are relatively great and in partial positions the shear forces are more than the design shear strength. So it is not appropriate to choose the complex walls consisting of the plasterboard with the thickness 12 mm and the OSB board with the thickness 9 mm as the shear material of inner walls. The other methods should be applied to strengthen the walls, such as increasing the thickness of shear boards or add steel drawstrings in partial positions.

4. CONCLUSIONS

In the paper finite element analysis of integral performance of a modern low-rise light-wood frame structure building in Beijing is carried out using finite element software based on the design criteria in China. Then the modes of the structure are solved by modal analysis and internal forces and deformations in appointed load cases which are the combinations of loads including seismic loads are also solved. Further, seismic performance and shear performance of the structure are analyzed.

(1) The load case results show the internal forces and displacements of the structure meet the requirements of design criteria in China and the structural safety margin is ample. It indicates the design of the frames has an appropriate security.

(2) Modal analysis results show the structure has a good integrity and potential weak positions of the structure can also be found. The positions including the partial three-floor and the roofs of the rooms with big space are potentially weak, so the construction method should be applied.

(3) The light-wood frame structure system has a good seismic performance because of its light weight and big damping ratio. The results of the paper validate the viewpoint sufficiently.

(4) The results indicate the structural methods of interior walls can meet the requirements of shear resistance, while shear forces of partial exterior walls exceed the design strength of shear resistance.

REFERENCES

[1] US Department of Housing and Urban Development Office. Steel vs. wood cost and short term energy comparison[R]. Washington, NAHB Research Center, 2001.

[2] Ye Ang. Study on the wood-frame-construction house[D]. Changsha: Hunan University, 2003. (in Chinese)

[3] Wang Yongwei, Long Weiguo, Yang Xuebing. A research on machine grading rule of dimension lumber. Sichuan Building Science, 2005, 31(6): 3-8. (in Chinese)

[4] Xu Xiaoliang, Ma Renle, He Minjuan. Load carrying tests and bearing capacity analysis on light wood truss. Special Structures, 2006, 23(1): 1-4. (in Chinese)

[5] Jiang Xuemei. The research of wood frame construction. Tangshan: Hebei Polytechnic University, 2005. (in Chinese)

[6] He Minjuan, Frank L. Properties of North American wood frame residential constructions. Structural Engineering, 2004(1): 1-5. (in Chinese)

[7] Zhu Jundao, Tan Fusheng, Zhou Chenggong. Structure of Platform-framed wooden residence[J]. Architecture Technology, 2006, 37(2): 142-143. (in Chinese)

[8] The timber structure design handbook[M]. Beijing: China architecture and building press, 2005. (in Chinese)

[9] Liu Jinbgo, Du Xiuli. Structural Dynamics[M]. Beijing: China Machine Press, 2005. (in Chinese)

[10] Zhou Xuhong, Shi Yu, Zhou Tianhua, et al. Study on shear resistance of cold-formed steel stud walls in residential structures[J]. Journal of Building Structures, 2006, 27(3): 42-47. (in Chinese)

[11] Yin Huiguang, Zhang Yuefeng. The establishment background of the technical specification for cold-formed steel framing low-rise buildings. Construction and Design for Project, 2004(7): 24-26. (in Chinese)