MULTI-LEVEL EARTHQUAKE PERFORMANCE OF ISOLATED PREFABRICATED CONCRETE SHEAR WALL STRUCTURES WITH HORIZONTAL SEMI-RIGID JOINTS

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Abstract

The connection performance of the prefabricated concrete shear wall (PCSW) structure is the main factor that affects the structural integrity and mechanical performance. Therefore, how to improve the performance of the connection has become the key to structural design. The isolation system is a highly efficient energy dissipation system, which makes the original structure form a flexible structure to isolate the seismic energy and reduce the seismic response of the superstructure, thus achieving the purpose of energy dissipation and seismic reduction. Therefore, the combination of isolation technology and PCSW structure can significantly improve the seismic performance of a PCSW structure. However, at present, there are few kinds of research on the overall seismic design methods for isolated PCSW structures and the characteristics of isolated PCSW structures are not considered, which limits the application of isolated PCSW structures in high-rise buildings and areas with high seismic intensity. In this paper, considering that the connection of prefabricated structures has a great impact on the integrity of the structure, the semi-rigidity of horizontal joints will be considered. Then based on the continuous link method and the response spectrum, the equivalent bending stiffness of the PCSW and the equivalent stiffness and equivalent damping ratio of the isolation story will be determined, respectively. Finally, the two-mass points direct design method will be used to preliminarily design an isolated PCSW structure to achieve a more reasonable design of the isolated PCSW structure. In ETABS 2016, the rubber isolator will be used to simulate the isolated bearings, and a 3D model of the isolated PCSW structure will be established to verify the two-mass points design method of the isolated PCSW structure with the semi-rigid horizontal joints. At the same time, the same method will be adopted to design an isolated equivalent cast-in-place shear wall (ECSW) structure, which will be compared with the isolated PCSW structure under multi-level earthquakes. The results show that the proposed design method is consistent with the calculation results of the finite element model and has high reliability and this simplified method can be used to make a reasonable preliminary design of the isolated PCSW structure. Under the different earthquake levels, compared to the isolated ECSW structure, the presence of semi-rigid horizontal joints changes the maximum acceleration response distribution and story shear distribution of the isolated PCSW structure. The distribution of the maximum acceleration response and the story shear are a fold line and an inverted *S*-shape, respectively.

Keywords: semi-rigid horizontal joints, prefabricated shear wall isolation structure, two-mass points model, direct design analysis method, multi-level seismic action

1. Introduction

In this paper, a PCSW is combined with isolation devices, and isolation devices is provided between the superstructure and the foundation to form an isolated PCSW structure. In this way, the natural vibration period of the building is extended, the damping ratio of the building is increased, and the excellent period of the site is avoided. The deformation and energy consumption of the isolation device are used to reduce the seismic energy and the seismic response of the superstructure[1], and finally improve the seismic performance of the PCSW structure under multi-level earthquakes[2]. However, at the current stage of the isolated design method adopted in China[3], the structural design model does not match the actual model and cannot accurately reflect the boundary conditions of the isolated structure. In the overall design method directly based on time-history analysis, the specific input seismic waves are not given in the code, resulting in the design of isolated structures with non-unique and discrete results, which makes it difficult to assess the reliability of the calculated results and guarantee the safety of structures.

PCSW structures are different from ECSW structures because of their horizontal and vertical joints. Horizontal joints transfer vertical load and bear horizontal shear force, and vertical joints transfer the interaction of adjacent PCSW. The performance of joints is the main factor affecting the integrity and mechanical performance of the PCSW structure. ACI 550[4] connections of precast concrete can be divided into equivalent cast-in-place connections and fabricated connections, the connection method of equivalent cast-in-place in engineering practice has some unavoidable disadvantages: exposed connecting bars are easily damaged during the lifting of the shear wall, complicated grouting operation, difficult to detect grouting quality, unfavorable to shorten the construction period, etc., moreover, the prefabricated concrete structure with equivalent cast-in-place connections can be designed by traditional seismic design methods. In contrast, the using of fabricated connections does not require maintenance time, which can effectively shorten the construction period, and has simpler connection construction and easier connection quality detection. However, related researches were mainly focused on experimental studies, lacking theoretical derivation and corresponding overall structure seismic design method. Compared with the ECSW structure, fabricated horizontal joints produce relative shear displacements under horizontal loads, and the cumulative shear displacement of the horizontal joints has a significant influence on the overall top displacement of the shear wall. Therefore, the relative shear displacement of the horizontal joints should be added when calculating the equivalent flexural stiffness of the shear wall. Based on the continuous link method[5], this paper will consider the semi-rigid horizontal joints of the PCSW structure to determine the relative shear displacement of horizontal joints and propose a design method for the PCSW structure. Then, through the direct design method of two-mass points[6][7][8], the PCSW structure will be preliminarily designed for isolation, and the overall design method of isolated PCSW structure will be formed. Then under multi-level earthquakes, the differences between the isolated PCSW structure and the isolated ECSW structure will be compared and analyzed.

2. Preliminary design of isolated PCSW structure with semi-rigid horizontal joints

2.1 Non-isolated PCSW structure with semi-rigid horizontal joints

This structure adopts a PCSW structure system with Class B of the seismic fortification category, 8 degrees of the seismic fortification intensity, Group 3 of the design earthquake group, 0.3g of the design earthquake acceleration, Class II of the site soil category, and 0.40s of the characteristic period错误!未找到引用源。. The superstructure has 15 floors, with the first to second floors being 3.6 m high and the third to fifteenth floors being 3.3 m high. The building has a total length of 16.6 m, a total width of 16.6 m, a height of 50.1m and a height to width ratio of 3.02. The dead load is 4.5 kN/m^2, and the live load is 2.0 kN/m^2. The structural space model, floor plan and horizontal joints connection are shown in Fig.1.

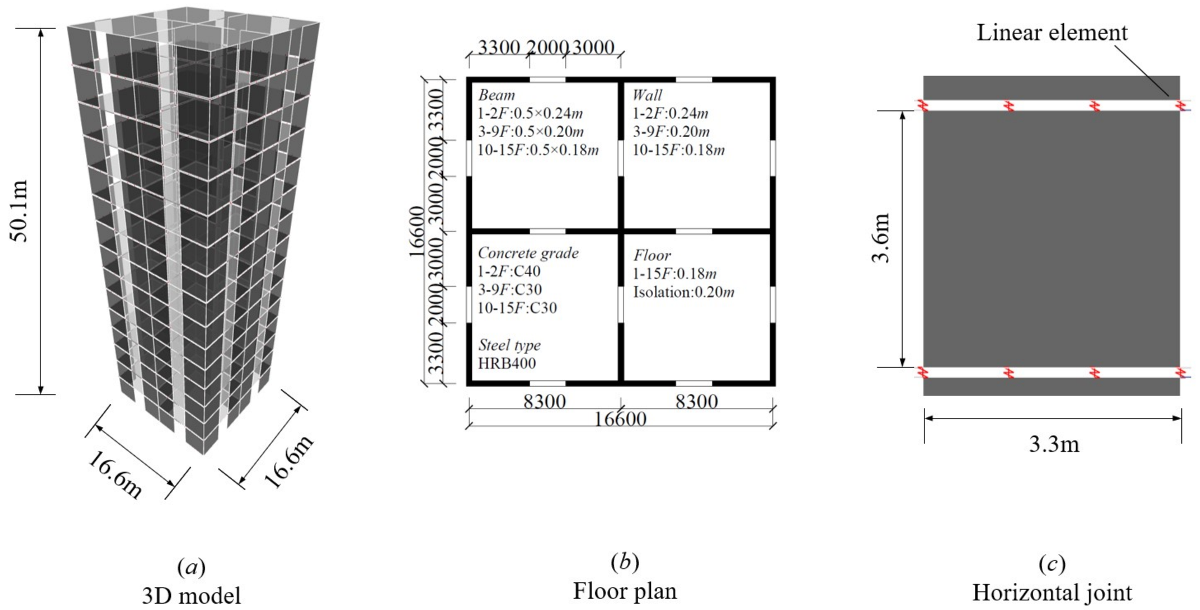


Fig. 1 – Information of the non- isolated PCSW structure

The fabricated connection members of the horizontal joints of the PCSW structure should be able to withstand axial forces, shear forces and bending moments to ensure strength and continuity of deformation. With reference to[9-12], this paper considered that in the existing equivalent cast-in-place connections and fabricated connections, the force transmission path of the flange-type steel connection is clear, and it has good bending and shear resistance and better deformation performance, which can realize the good deformation capacity of shear wall connected area, so the horizontal joints will be connected by flange-type steel. Each horizontal joint will be connected by 4 flange-type steels with Q345 steel, and the flange-type steel will be connected by 10.9-grade M22 high-strength bolts with 0.022 m diameter. The size of the flange-type steel is 0.8×0.3×0.06 m, the number of bolt holes on the flange-type steel is 8, and the diameter of the bolt holes are reserved for 0.025 m. The connection form and overall stress mechanism of flange-type steel are shown in Fig.2. In the figure, V is the horizontal shear force of the horizontal joint of this story, N is the axial force of the prefabricated wall, and M is the bending moment of the prefabricated wall. In the finite element model, the relationship between the horizontal force and displacement of the horizontal joints will be simulated by the springs, and in ETABS 2016, the setting of the springs will be achieved by a linear element and the spring stiffness will be equivalently obtained by the relative shear displacement of the flange-type steel.

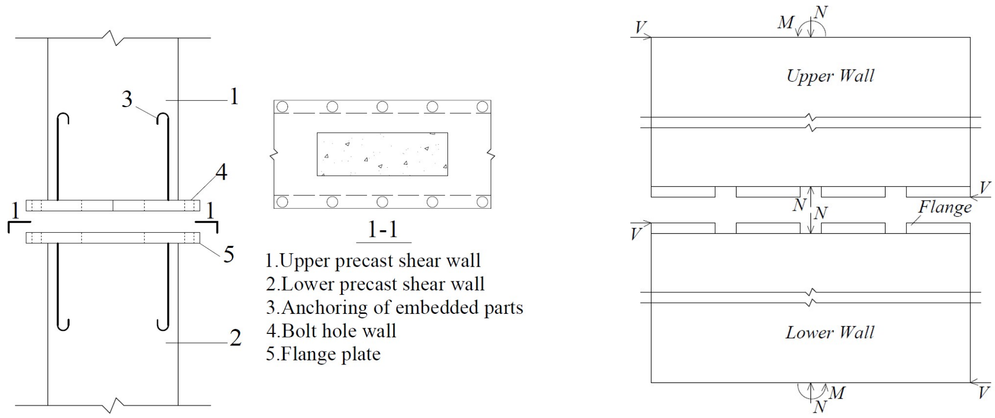
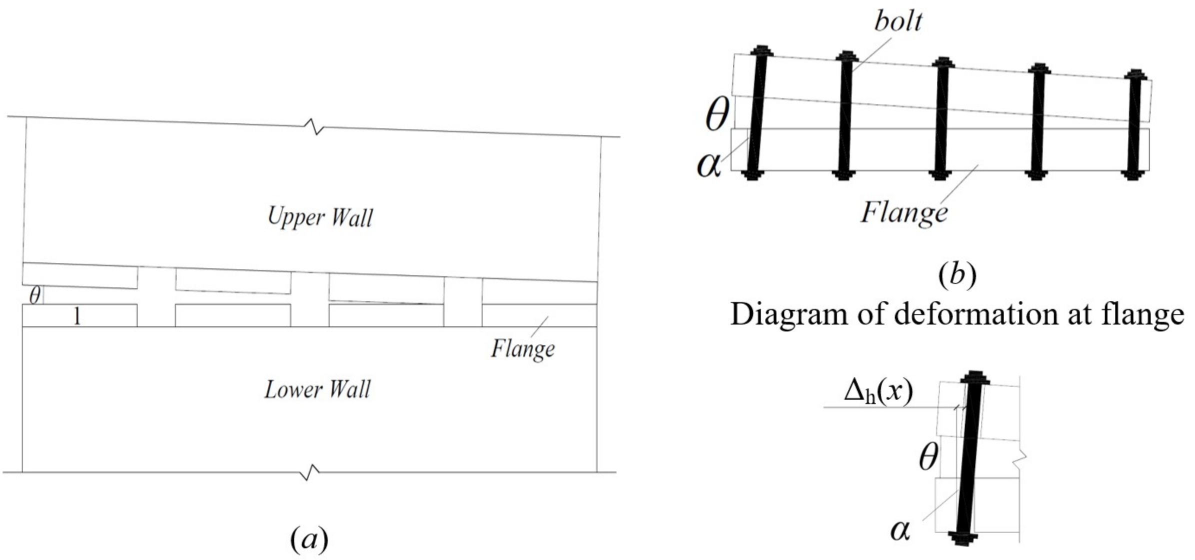


Fig. 2 – Flange-type steel and force diagram

2.2 Equivalent bending stiffness

In this paper, it is assumed that the bolt hole will be made and positioned accurately, and the high-strength bolt will be located at the center of the bolt hole. Because there is a difference between the bolt diameter and the bolt hole diameter, the upper and lower prefabricated wall panels will slip δ under the horizontal load. Then, the upper and lower prefabricated walls will be staggered and the bolts will be squeezed tightly with the reserved bolt holes of the flange-type steel, which will generate an angle θ of the joint on the tension side and an angle α on the outermost bolt of the flange-type steel. As shown in Fig.3, $\alpha=\theta$ can be obtained.



(*a*)
Diagram of deformation at joints

(*b*)
Diagram of deformation at flange

(*c*)
Diagram of deformation at bolt

Fig. 3 – Flange-type steel connection and force diagram

Assuming that the horizontal load of a shear wall with a total height of H is an inverted triangular distributed load, the horizontal shear force on a horizontal section at any x height is $F(x)$, as shown in Eq. (1).

$$F(x)=\int_x^H q(\lambda)d\lambda=\int_x^H \frac{q_{\max}}{H}\lambda d\lambda=\frac{q_{\max}H}{2}\left(1-\frac{x^2}{H^2}\right) \tag{1}$$

The top displacement of the prefabricated wall caused by the angle θ of the joint can be obtained by the relative lateral displacement $\Delta_h(x)$ of the horizontal joints. The relative lateral displacement of the horizontal joints is composed of the relative lateral displacement of the flange-type steel and the sliding amount δ of the prefabricated upper and lower wall panels. Ignoring the structural defects of the flange-type steel, the relative lateral displacement of the horizontal joints is composed of three parts: ① the bending deformation of bolt δ_1; ② the shear deformation of bolt δ_2; ③ the compression deformation of steel hole δ_3. The relative lateral displacement $\Delta_h(x)$ of horizontal joints shown in Fig.3 (c) will be calculated by the following equation.

$$\delta_1=\frac{2F_{bc}(x)l_{bc}^3}{3E_sI_{bc}}=\frac{2F(x)l_{bc}^3}{3mnE_sI_{bc}} \tag{2a}$$

$$\delta_2 = \frac{2\mu F_{bc}(x) l_{bc}}{G_s I_{bc}} = \frac{5\mu F(x) l_{bc}}{mnE_s A_{bc}} \tag{2b}$$

$$\delta_3 = \frac{2F_f(x) l_f}{md_{bc} h_f E_s} = \frac{2F(x) l_f}{mnE_s A_f} \tag{2c}$$

$$\Delta_h(x) = \frac{\delta_1 + \delta_2 + \delta_3}{2} + \delta \tag{3}$$

Where, n is the number of flange-type steel; h_f is the thickness of flange-type steel; b_f is the width of flange-type steel; l_f is the length of flange-type steel; A_f is the compression area of plate hole wall; $F_f(x)$ is the shear force of flange-type steel; m is the number of bolts; d_{bc} is the diameter of bolt; $2l_{bc}$ is the length of bolt; A_{bc} is the area of bolt; μ is the non-uniformity coefficient of shear stress at the interface; $F_{bc}(x)$ is the shear force on the bolt; E_s is the elastic modulus of steel; G_s is the shear modulus of steel, respectively.

After obtaining the relative lateral displacement $\Delta_h(x)$ of the horizontal joints from Eq. (2) and Eq. (3), the geometric relationship between $\Delta_h(x)$ and α can be obtained from Figure 3, and then α can be calculated by Eq. (4). Finally, according to Eq. (5), the top displacement $\Delta_{\theta i}$ of the prefabricated wall caused by the angle θ can be obtained. Where h is the height of the upper prefabricated wall.

$$\tan\alpha = \frac{\Delta_h(x)}{l_{bc}} \tag{4}$$

$$\Delta_{\theta i} = h \cdot \sin\theta \tag{5}$$

We can add the $\Delta_{\theta i}$ obtained by Eq. (5) to the top displacement of the shear wall to obtain the top displacement of the prefabricated shear wall with semi-rigid horizontal joints under an inverted triangular horizontal load, as shown in Eq. (6). Then the top displacement Δ is completely expressed in the form of bending deformation, as shown in Eq. (7), and the equivalent bending stiffness EI_{eq} is obtained by Δ_{wh}=Δ. Where EI_w is the bending stiffness of the prefabricated shear wall and GA_w is the shear stiffness of the prefabricated shear wall, respectively.

$$\Delta_{wh} = \frac{11q_{max}H^4}{120EI_w} + \frac{\mu q_{max}H^2}{3GA_w} + \sum\Delta_{\theta i} \tag{6}$$

$$\Delta = \frac{11q_{max}H^4}{120EI_{eq}} \tag{7}$$

After obtaining the equivalent bending stiffness EI_{eq} from Eq. (6) and Eq. (7), the total lateral stiffness EI_x of the shear wall in the x-direction of the non-isolated PCSW structure will be obtained by continuous link method. Then, the basic natural vibration period T_x in the x-direction will be obtained from Eq. (8), where, Δ_x is the imaginary horizontal top displacement of the non-isolated PCSW structure, ψ_T is the reduction coefficient of basic natural vibration period. Finally, the shear force of each story in the x-direction will be obtained by the bottom shear method. The results above will be compared with the finite element results to verify the feasibility of the design method of non-isolated PCSW structure with semi-rigid horizontal joints. The basic natural vibration period of the structure calculated by this method is 1.50s, and the basic natural vibration period of the finite element calculation is 1.63s. The comparison of the shear force is shown in Fig.4. From the comparison of the basic natural vibration period and the shear force of the structure, it can be seen that the proposed design method of the non-isolated PCSW structure with semi-rigid horizontal joints has ideal accuracy.

$$T_x = 1.7\psi_T\sqrt{\Delta_x} \tag{8}$$

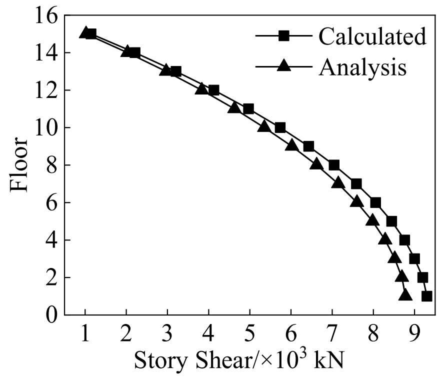


Fig. 4 – Story shear

2.3 Isolated PCSW structure with semi-rigid horizontal joints

The base isolation will be added to the above 15-story non-isolated PCSW structure, and the height of the isolation story is 1.6 m. Under the representative value of gravity load, the vertical compressive stress of the isolation bearing of the Class B building does not exceed 12Mpa. Based on this, the diameter of the isolation bearing will be calculated and 33 LRB600 isolation bearings will be selected and arranged between the superstructure and foundation. The structural 3D model and the floor plan of the isolated bearings and the location of the isolation story are shown in Fig.5. Under the fortified earthquake, the horizontal equivalent stiffness and equivalent damping ratio of the isolated bearing will be taken as the values at 100% of the shear deformation of the bearing, which are K_j=1.83kN/mm and ζ_j=0.23, respectively. The equivalent stiffness and equivalent damping ratio of the isolation story will be determined iteratively based on the response spectrum through direct design analysis[13]. And the specific iteration is as follows:

(1) Assuming the initial isolation story displacement, the equivalent stiffness and equivalent damping ratio of the isolated bearing will be calculated from the initial stiffness and yield displacement according to Eq. (9*a*) and Eq. (9*b*).

$$K_{eq} = K_y + \frac{Q_d - K_y D_y}{D} \tag{9a}$$

$$\zeta_{eq} = \frac{4Q_d\left(D - D_y\right)}{2\pi K_{eq} D^2} \tag{9b}$$

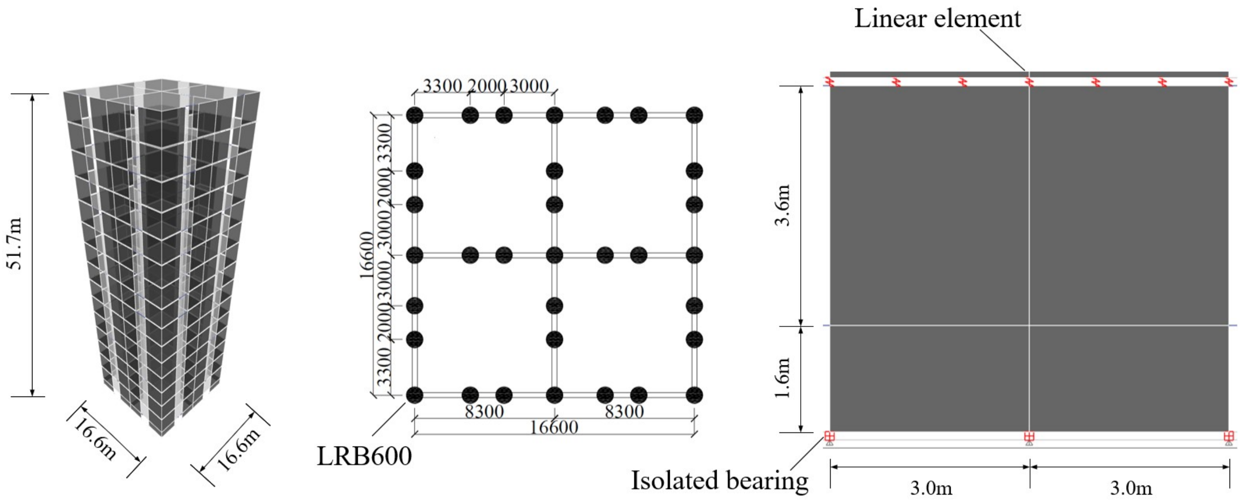
Where, K_{eq} is the horizontal equivalent stiffness of the isolation bearing, ζ_{eq} is the equivalent damping ratio of isolation bearing, D_d is the yield force of the isolation bearing, D is the displacement of the isolation story, D_y is the yield displacement of the isolation bearing, K_y is the post-yield stiffness of the isolation bearing, K_0 is the stiffness of the bearing before yielding, respectively.

(2) The equivalent stiffness and equivalent damping ratios in (1) will be substituted into the two-mass points design method[4] to calculate the period, mode shape and mode damping ratio of the structure, and then we will obtain the base shear force V by the mode analysis method. Finally, the displacement D of the isolation story will be calculated by Eq. (10).

$$D = V_b / K_{eq} \tag{10}$$

(3) Repeating the above steps until the displacement difference of isolation story between the i^{th} and the i-1^{th} is less than the allowable error $|D|$.

Through the above iterative convergence, we will obtain the equivalent stiffness and equivalent damping ratio of the isolation story, the base shear force and the superstructure story shear force by the two-mass points.



(*a*)
3D model

(*b*)
Plan layout of isolated bearings

(*c*)
Location of isolation floor

Fig. 5 – Information of the prefabricated shear wall isolated structure

The 16-story isolated PCSW structure will be designed by the above method, and the calculated results will be compared with the finite element results to verify the feasibility of the direct two-mass points analysis method. The period in the *x*-direction of the structure and the comparison of the shear are shown in Table 1 and Fig.6, respectively. From the comparison of the structural period and the story shear, it can be seen that the design method has better accuracy.

Table 1 – The modal analysis

Modal	**Calculated**	**Analysis**
1	1.83s	1.99s
2	0.54s	0.68s

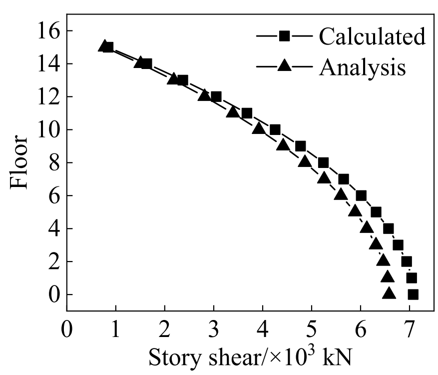


Fig. 6 – Story shear

Based on the above design of the non-isolated PCSW structure with semi-rigid horizontal joints and the isolated PCSW structure with semi-rigid horizontal joints, we will obtain the preliminary design outline of the two structures as shown in Fig.7.

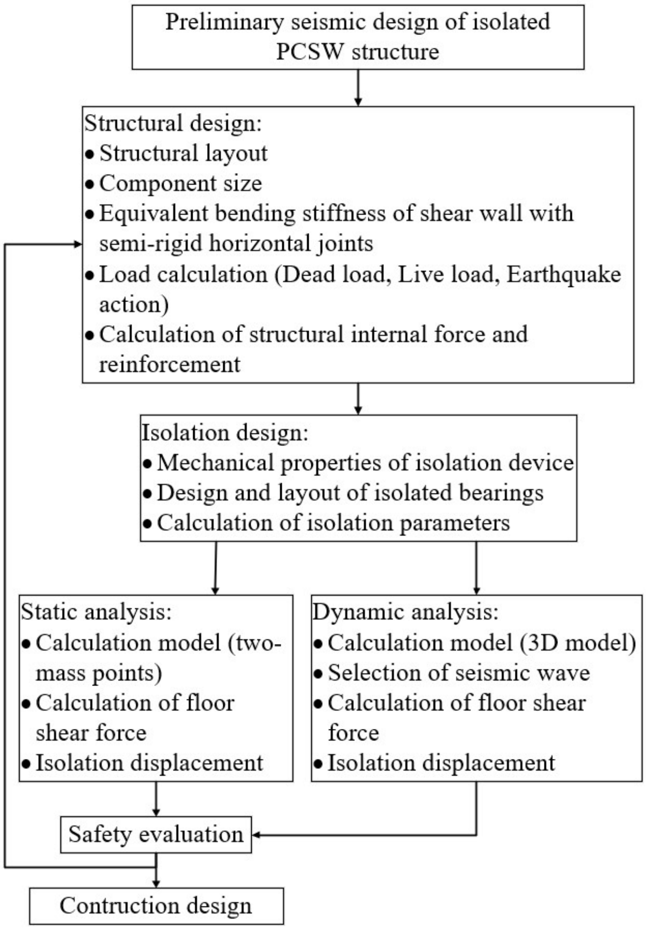


Fig. 7 – Preliminary design outline

3. Comparative analysis between non-isolated PCSW structure and isolated PCSW structure under multi-level earthquakes

Adopting the same design method of isolated structure in Section 2.3 to design an non-isolated PCSW structure whose structure information is shown in Fig.2 (a) and Fig.2 (b). Then compare and analyze the story acceleration and story shear force under multi-level earthquakes with the isolated PCSW structure. Referring to the code of design for seismic isolated buildings[14], multi-level earthquakes can be divided into the fortified earthquake, the rare earthquake and the extremely rare earthquake, with corresponding peak accelerations of 300gal, 510gal, and 840gal, respectively. Under the rare earthquake and extremely rare earthquake, the horizontal equivalent stiffness and equivalent damping ratio of the isolated bearing will be taken as the values at 250% of the shear deformation of the bearing, which are K_j=1.45kN/mm and ζ_j=0.20, respectively. In the time-history analysis, two natural seismic waves and one artificial seismic wave will be selected, and the amplitude modulation of seismic waves under three seismic levels will be carried out. The maximum acceleration and story shear from the time-history analysis of the two isolated structures under multi-level earthquakes are shown in Fig.8. In the figure, P represents the isolated PCSW structure, C represents the non-isolated PCSW structure, AW represents the artificial seismic waves, and HE and EUR represents the two natural seismic waves.

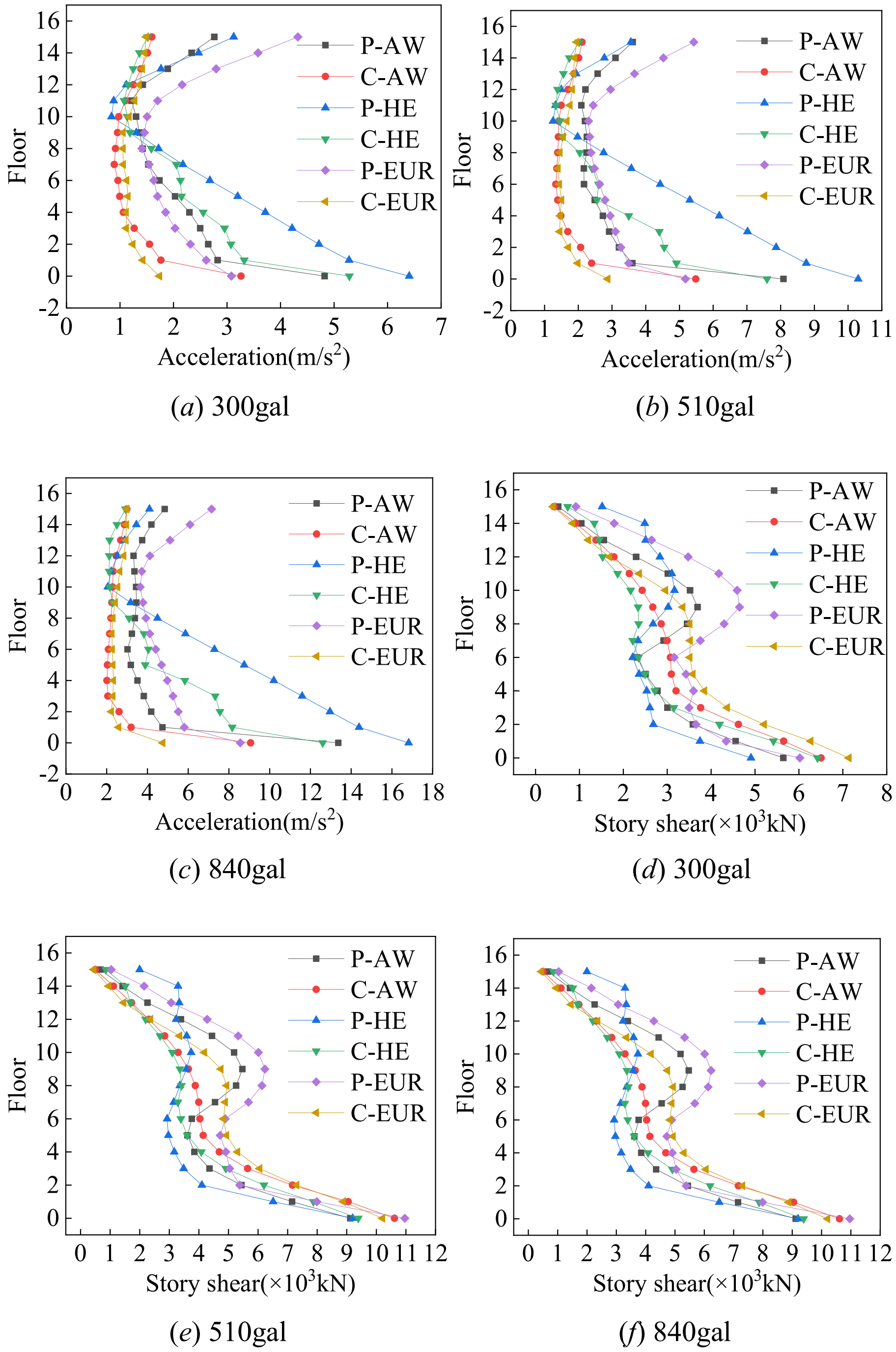


Fig. 8 – The maximum acceleration and story shear force under multi-level earthquakes

According to figure 8, it can be concluded that: (1) As the input acceleration of three seismic waves increases, the increase in the maximum acceleration response of the superstructure of the two isolated structure models is less than the increase rate of the input acceleration, that is, the greater the input acceleration of the earthquake, the more effective the isolation effect will be. (2) The shape of the maximum acceleration response distribution of the non-isolated PCSW structure is a smooth curve, while the maximum acceleration response distribution shape of isolated PCSW structure is changed to a fold line with a large acceleration response in the isolation story and the top story and a small acceleration response in the middle because of the existence of semi-rigid horizontal joints. (3) Under the different earthquake levels, with the increase of the story height, the shear of the non-isolated PCSW structure gradually decreases, while the story shear of the isolated PCSW structure shows an inverted *S*-shape that decreases first, then increases and then decreases.

4. Conclusion

(1) Considering the semi-rigid design method of PCSW structure with horizontal joints and the direct two-mass points design analysis method, the obtained structural period and story shear are consistent with the calculation results of the ETABS 2016 3D model. It has ideal accuracy, which is convenient for structural designers to perform seismic response analysis and preliminary design, however, the influence of the horizontal joint interface is not taken into account, resulting in the deviation between shear displacement and the actual situation, and the error between the preliminary design results and the finite element analysis results.

(2) Under the different earthquake levels, compared to the non-isolated PCSW structure, the presence of semi-rigid horizontal joints changes the maximum acceleration response distribution and story shear distribution of the isolated PCSW structure. The distribution of maximum acceleration response is a fold line with a large acceleration response in the isolation story and the top story and a small acceleration response in the middle, and the story shear shows an inverted *S*-shape that decreases first, then increases and then decreases.

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Appendix: Nomenclature

Symbol	Definition	Unit
α	Angle on the outermost bolt of flange-type steel	rad
δ	Slip displacement of the upper and lower prefabricated wall	m
δ_1	Bending deformation of bolt	m
δ_2	Shear deformation of bolt	m
δ_3	Compression deformation of plate hole wall	m
ζ_j	Equivalent damping ratio of the isolated bearing	
ζ_{eq}	Equivalent damping ratio of the isolated bearing After the iteration	
θ	Angle of the joint on the tension side	rad
μ	Non-uniformity coefficient of shear stress at the interface	
ψ_T	Reduction coefficient of basic natural vibration period	
V	Horizontal story shear force of horizontal joints	kN
N	Axial force of the prefabricated wall	kN

M	Bending moment of the prefabricated wall	kN·m
H	Total height of shear wall structure	m
$F(x)$	Horizontal shear force on a horizontal section at any x height	kN
n	Number of flange-type steel	
m	Number of bolts	
D	Displacement of the isolation story	m
q_{max}	Maximum value of the inverted triangular distributed load	kN/m
h_f	Thickness of flange-type steel	m
b_f	Width of flange-type steel	m
l_f	Length of flange-type steel	m
A_f	Compression area of plate hole wall	m^2
$F_f(x)$	Shear force of flange-type steel	kN
d_{bc}	Diameter of bolt	m
l_{bc}	Half length of bolt	m
A_{bc}	Area of bolt	m^2
$F_{bc}(x)$	Shear force of bolts	kN
E_s	Elasticity modulus of steel	MPa
G_s	Shear modulus of steel	MPa
$\Delta_h(x)$	Relative shear displacement of horizontal joints at x height	m
$\Delta_{\theta i}$	Top displacement of the prefabricated wall caused by the angle θ	m
EI_w	Bending stiffness of prefabricated shear wall	kN·m^2
GA_w	Shear stiffness of prefabricated shear wall	kN
EI_{eq}	Equivalent bending stiffness of prefabricated shear wall	N·m^2/rad
EI_x	Total lateral stiffness of the non-isolated structure in the x-direction	kN·m^2
Δ_{wh}	Top displacement of the prefabricated shear wall with horizontal joints	m
Δ	Top displacement of the cast-in-place shear wall	m
Δ_x	Imaginary horizontal top displacement	m
T_x	Basic natural vibration period in the x-direction	s

K_j	Equivalent stiffness of the isolated bearing	kN/m
K_{eq}	Equivalent stiffness of the isolated bearing after the iteration	kN/m
Q_d	Yield force of the isolation bearing	kN/m
D_y	Yield displacement of the isolation bearing	m
K_y	Post-yield stiffness of the isolation bearing	kN/m
K_0	Stiffness of the bearing before yielding	kN/m
V_b	Base shear force	kN

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