Study on Seismic Performance of Hinged Frame-Rocking Wall System

ZHU Chunyang[1*], MEI Siyang[2]

School of Architecture & Construction, University of Shenyang Jianzhu, Shenyang 110168, China
[1*]Cyzhu1087@163.com, [2]1302908846@qq.com

Abstract. The hinged frame-rocking wall system is proposed to optimize the internal force of the rocking wall, and the column foot is hinged with the foundation to reduce the bending moment demand of the rocking wall at the corresponding part. Six 10-story 4-span single-span frame models were established by Sap2000 software. The dynamic elastic-plastic time history analysis were carried out to explore the influence of relaxing the bottom constraint on the overall stability and collapse resistance of the structure. The results show that the reasonable setting of the column base hinged frame can reduce the lateral stiffness of the first floor, move the peak bending moment from the middle to the middle, and realize the optimal design of the rocking wall.

Keywords: Frame-rocking wall; Hinged structure; Time history analysis; Earthquake resistant behavior; Mechanical model

1 Introduction

Wada[1] reinforced the G3 building of the Tokyo Institute of Technology using prestressed concrete rocking walls and steel dampers. The reinforcement of the rocking walls and dampers enhanced the seismic performance and energy dissipation capacity of the original structure, achieving a more reliable seismic design. Pollino[2] used nonlinear time history analysis of near and far-field seismic motions and found that the frame-rocking wall structural system can effectively avoid the occurrence of weak structural layers. Najam[3] employed a displacement correction method during the nonlinear static analysis process and proposed a simplified analytical approach to easily evaluate the nonlinear seismic demands of high-rise rocking wall structures. For the traditional structure with additional frame-rocking wall[4][5][6][7], the structural deformation mode is changed, the local yield is transformed into the overall yield, and the inter-story deformation of the structure is more uniform, which effectively improves the seismic performance of the frame structure.

The base columns of frame structures are typically rigidly connected to the foundation, resulting in a significantly higher lateral stiffness at the bottom level compared to other levels. Consequently, peak bending moments are more likely to occur at the first level of the rocking wall.

A. S. B. A Rashid et al. (eds.), *Proceedings of the 2024 3rd International Conference on Applied Mechanics and Engineering Structures (AMES 2024)*, Advances in Engineering Research 34,
https://doi.org/10.2991/978-94-6463-473-0_17

In order to achieve optimized design of rocking walls, this study first establishes a mechanical model of the frame-rocking wall structural system, analyzes the force mechanism of the rocking wall, and aims to reduce the demand for rocking wall bending moments. By analyzing the column-base hinged frame-rocking wall structural system, seismic response is studied, and the impact of hinged column bases on the overall structure's seismic response is discussed, providing a reference for the optimized design of frame-rocking wall structures.

2 Internal Force Analysis of Rocking Wall

This study simplifies N-story frame-rocking wall structures into a beam-shear mass model, utilizing the displacement of the top story as the degree of freedom, and establishes an equivalent single-degree-of-freedom system based on the first mode. Damping effects are temporarily disregarded. Based on the equilibrium of bending moments at the base hinge supports of the rocking wall, the following expression is derived:

$$M_I + M_J + \sum_{n=1}^{N} F_{lin}^{n} H_n = 0 \tag{1}$$

The symbolsM_I、 M_J represent the bending moments at the base hinge supports of the rocking wall due to inertial forces and rotational inertia, respectively. For rocking walls with uniformly distributed mass, these moments are calculated according to Equations 2 and 3, respectively.

$$M_I = \frac{1}{4} m_w \ddot{u}_t H \tag{2}$$

$$M_J = \frac{m_w \ddot{u}_t}{12H}(B^2 + 4H^2) \tag{3}$$

m_w represents the mass of the rocking wall, u_t and $\ddot{u}_t$ denote the displacement and acceleration of the top of the rocking wall, B、H stand for the width and height of the rocking wall. In Equation 1,F_{lin}^{n}denotes the axial force between the rocking wall and the frame, H_n represents the mass height of the nth layer. By considering the mass of the nth layer as an isolated system, an equilibrium equation is established as follows Equation 4 :

$$F_{lin}^{n} = m_n \ddot{u}_n - \mathrm{SF}_n \qquad n \leq N \tag{4}$$

From Equation 4, it can be observed that the internal forces within the rocking wall are primarily induced by the axial force F_{lin}^{n}, mainly composed of the overall seismic action $m_n \ddot{u}_n$ and the distributed restoring forces SF_n along the floors. Based on the above equation, the force decomposition of the rocking wall is illustrated in Figure 1.

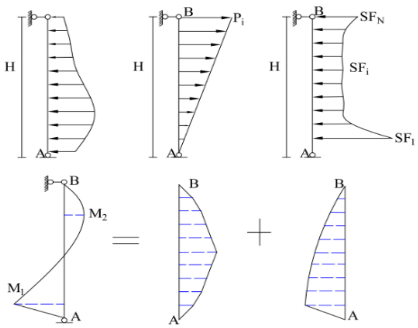

Fig. 1. Schematic view of rocking wall loading

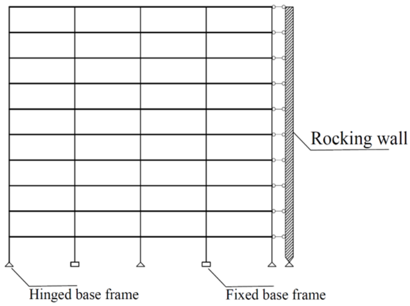

Fig. 2. Plan view of frame

The distribution of floor restoring forces SF_n varies due to the different lateral stiffness at each level. There is an increase in SF_n at locations where there are abrupt changes in floor stiffness. The base columns of frame structures are typically rigidly connected to the foundation, resulting in a significantly higher lateral stiffness at the bottom level compared to other levels. This leads to an elevated restoring force SF_1 at the ground level. Consequently, peak bending moments are more likely to occur at the first level of the rocking wall.

3 Basic Conditions

The structure is a 10-story 4-span frame structure with a height of 3 m, a span of 8 m, and a seismic fortification level of C. The fortification intensity is 7 degrees, seismic design group is the first group, and the site category is Class II. The floor is subjected to constant and live loads of 5kN/m^2 and 2kN/m^2, respectively. The design is completed according to the specifications, as shown in Table 1. Finite element models of the structure under various conditions are established using the SAP2000 platform.

In order to analyze the seismic response of the frame-rocking wall structure to hinged column bases, the following conditions are designed: all fixed base frames (FF), all hinged base frames (FH), all fixed base frame-rocking wall (RFF), all hinged base frame-rocking wall (RFH) and combined fixed/hinged base frame-rocking wall (RFC-i/j/k…), where i/j/k… represent the axis numbers of the hinged columns, starting from the column closest to the rocking wall as number 1 and increasing towards the column furthest from the rocking wall as number 5, as shown in Fig.2.

Table 1. Section size and reinforcement

floor	sectional dimension		beam(C30 concrete)		columns		rocking wall
	columns	beam	Upper reinforcement	Lower reinforcement	reinforcement	concrete	
1-3	800×800	300×600			4B22+12B18	C40	sectional dimension：3000×250
4-6	700×700	250×600	4B18+2B16	4B12	4B20+12B16	C35	reinforcemen ：B12@200
7-9	650×650	250×600			20B14	C30	concrete：C40
10	650×650	250×600	2B18+2B16	2B16			

4 Time history analysis

Conducting time-history analysis on the aforementioned structures to study their responses under rare seismic events. Selecting three natural seismic excitations and two artificial seismic excitations based on the standard response spectrum, with specific information provided in Table 2.

Table 2. Ground motion record selection

Name	seismic magnitude	PGA (g)	Time (s)
"Southern Calif"	6.00	0.1468	40.02
"Borrego Mtn"	6.63	0.1687	45.22
"San Fernando"	6.61	0.1629	41.74
Artificial ground shaking	-	0.1636	40.02
Artificial ground shaking	-	0.1762	40.02

From Fig.3, the maximum inter-story displacement angle response of the structure shows that the distribution pattern of the maximum displacement angle in the hinged column base frame with rocking wall is similar to that of the hinged column base frame. The rocking wall reduces the maximum displacement angle response in the middle and lower part, and slightly increases the maximum displacement angle response in the upper part. Compared with the frame with rocking wall, the maximum displacement angle response at the bottom of the frame with rocking wall hinged column base increases, but the maximum displacement angle response at the middle and upper parts of the structure decreases significantly.

From Fig.4, in the RFF structures, the maximum moment response of the rocking wall appears at the bottom floor, mainly due to excessive lateral stiffness at the bottom causing it. In the RFH structure, the maximum moment response of the rocking wall shifts to the middle of the structure, and the maximum moment response is less than that of the column base fixed frame condition. For combined fixed/hinged base frame-rocking wall structure, the maximum moment response of the rocking wall still peaks at the first floor, and as the number of hinged column bases increases, the amplitude of the maximum moment response of the rocking wall decreases at the corresponding position. The maximum displacement angle responses of RFH-135 and RFH-234 show little difference. But, when the rotational restraint of the adjacent column of the rocking wall is relaxed, the maximum moment response of the rocking wall decreases significantly.

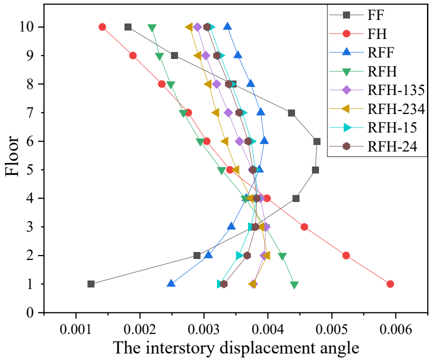

Fig. 3. Story drift response

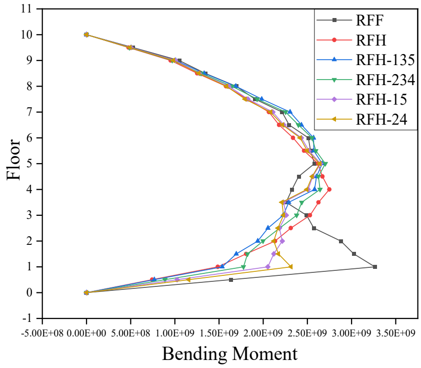

Fig. 4. Bending moment response

Through time history analysis, it was found that the structural deformation mode is controlled by the swing wall structure, making the deformation of the structure more uniform. With the introduction of hinged column base frames, the displacement at the bottom of the structure increases, while the displacement at the top decreases, and the bending moment value at the first-floor position decreases.

5 Conclusion

This paper proposes a column base hinged frame-shear wall structural system, the impact of column base hinges on the seismic response of the structure is investigated.

Through finite element analysis, the influence of the hinged column foot on the overall seismic response of the structure is discussed. The conclusions are as follows:

Relaxing the constraints at the bottom will result in increased deformation at the lower part of the structure and reduced deformation at the upper part. The peak moment will transfer from the bottom to the middle of the structure, leading to a decrease in the peak moment of the rocking wall, achieving a reduction in the shear wall's internal force demand.

References

1. Wada A, Qu Z, Motoyui S, et al. Seismic retrofit of existing SRC frames using rocking walls and steel dampers[J]. Frontiers of Architecture and Civil Engineering in China, 2011, 5: 259-266.
2. Pollino, M., et al. "Research needs for seismic rehabilitation of sub-standard buildings using stiff rocking cores." Structures Congress 2013: Bridging Your Passion with Your Profession. 2013.
3. Najam, Fawad Ahmed, et al. "Prediction of nonlinear seismic demands of high-rise rocking wall structures using a simplified modal pushover analysis procedure." The Structural Design of Tall and Special Buildings 27.15 (2018): e1506.
4. Zhang, Fuwen, et al. "Experimental Study of the Seismic Performance of a Prefabricated Frame Rocking Wall Structure." Buildings 12.10 (2022): 1714.
5. Jiang, Shaofei, et al. "Assessment of Soft-First-Floor Structures Reinforced by Rocking Frame Based on Seismic Resilience." *Buildings* 14.1 (2024): 197.
6. Piri, Mehrdad, and Ali Massumi. "Seismic performance of steel moment and hinged frames with rocking shear walls." *Journal of Building Engineering* 50 (2022): 104121.
7. Aghagholizadeh, Mehrdad, and Nicos Makris. "Response Modification of Moment Resisting Frames using Rocking Wall with Supplemental Dampers."