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ILEE ROBUST PROJECT: DEVELOPMENT OF A MULTIPLE ROCKING STEEL STRUCTURAL SYSTEM

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Abstract

In recent years, rocking isolation has become an effective approach to improve seismic performance of steel and reinforced concrete structures. These systems can mitigate structural damage through rigid body displacement and thus relatively low requirements for structural ductility, which can significantly improve seismic resilience of structures and reduce repairing costs after strong earthquakes. A number of base rocking structural systems with only a single rocking interface have been proposed. However, these systems can have significant high mode effect for high rise structures due to the single rocking interface. This RObust BUilding SysTem (ROBUST) project is a collaborative China-New Zealand project sponsored by the International Joint Research Laboratory of Earthquake Engineering (ILEE), Tongji University, and a number of agencies and universities within New Zealand including the BRANZ, Comflor, Earthquake Commission, HERA, QuakeCoRE, QuakeCentre, University of Auckland, and the University of Canterbury. A number of structural configurations will be tested [1, 2], and non-structural elements including ceilings, infilling walls, glazed curtain walls, precast concrete panels, piping system will also be tested in this project [3].

Within this study, a multiple rocking column steel structural system was proposed and investigated mainly by Tongji team with assistance of NZ members. The concept of rocking column system initiates from the structure of Chinese ancient wooden pagoda. In some of Chinese wooden pagodas, there are continuous core columns hanged only at the top of each pagoda, which is not connected to each stories. This core column can effectively avoid collapse of the whole structure under large storey drifts. Likewise, there are also central continuous columns in the newly proposed steel rocking column system, which can avoid weak story failure mechanism and make story drifts more uniform. In the proposed rocking column system, the structure can switch between an elastic rigidly connected moment resisting frame and a controlled rocking column system when subjected to strong ground motion excitations. The main seismic energy can be dissipated by asymmetric friction beam–column connections, thereby effectively reducing residual displacement of the structure under seismic loading without causing excessive damage to structural members. Re–centering of the structure is provided not only by gravity load carried by rocking columns, but also by mould coil springs.

To investigate dynamic properties of the proposed system under different levels of ground excitations, a full-scale three-story steel rocking column structural system with central continuous columns is to be tested using the International joint research Laboratory of Earthquake Engineering (ILEE) facilities, Shanghai, China and an analytical model is established. A finite element model is also developed using ABAQUS to simulate the structural dynamic responses. The rocking column system proposed in this paper is shown to produce resilient design with quick repair or replacement.

Keywords: Shake table, multiple rocking, friction connection, self–centering, seismic, robust.

1. Introduction

Earthquakes can induce significant economic losses due to retrofitting costs of buildings and infrastructures, and downtime of business can lead to additional losses. It always costs a great deal of time and money to repair damaged structures. Actually, To reduce damage and downtime of engineering structures after an earthquake, civil engineers have developed some high-performance damage-resistant seismic structural system including replaceable structural elements, controlled rocking systems as well as self-centering systems. As one of these seismic resisting methods, controlled rocking structural systems proposed in previous studies have been proved to obtain enough ability to reduce damage caused by earthquakes. However, most of controlled rocking structural systems only have a single rocking interface at base, which can have significant high mode effect for high rise structures.

Wiebe and Christopoulos (2009) found that an upper rocking joint could reduce bending moment and shear force of rocking concrete walls enveloped with little or no increase in peak displacement demand [4]. In this paper, a multiple rocking column system was developed, consisting of continuous beams and separated columns with asymmetric friction connections (AFC) at column bases. Rocking columns are designed to only rock for earthquakes beyond design basis earthquake (DBE) and maximum considered earthquake (MCE) levels. This indicates that it can switch between an elastic rigidly connected moment resisting frame and a controlled rocking column system when subjected to strong ground excitations. AFCs can effectively dissipate seismic energy and reduce maximum story drifts. For this system, residual displacement of the structure is small, which is owing to the fact that both gravity load and mould coil springs can provide self-centering forces for the structure.

2. Building information

Layout of the ROBUST specimen is as shown in Fig. 1, in which rocking columns (C4) are located at outer bays in the transverse direction. Since there are still several other low-damage structural systems to be tested first, corner columns (C1) will be separated during testing of the rocking column system. This is also a compromise in order to test all the structural systems within the ILEE project. In this study, the rocking column system will be tested only along the transverse direction due to the aforementioned compromise. Elevation view of a standard story in the proposed rocking column system is shown in Fig. 2.

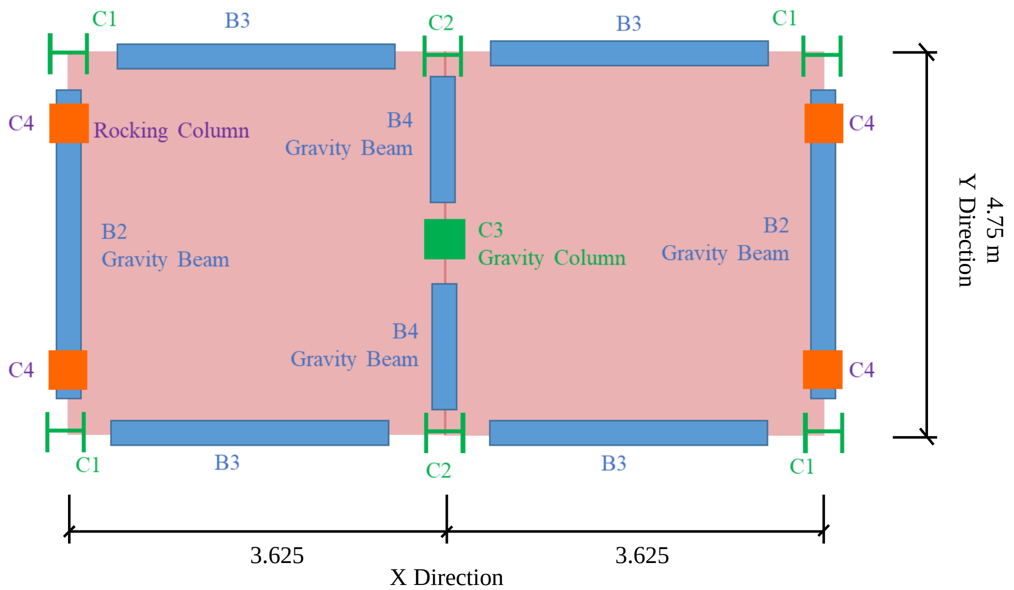


Fig. 1 – Layout of a standard level

The column end plates can rotate along their edges when subjected to a strong earthquake. Configuration of the column base joint is shown in Fig. 3. The column base joint consists of an AFC at the column web, mould coil springs and a column end plate as shown in the figure.

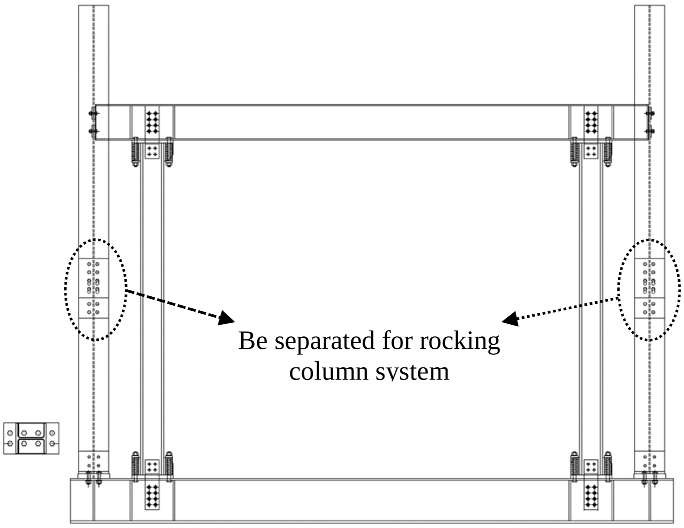
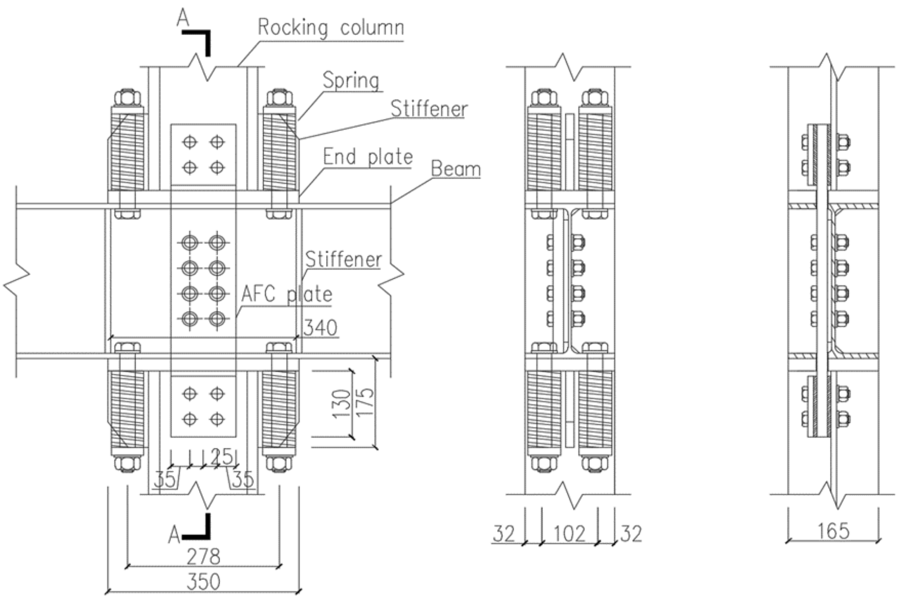


Fig. 2 – Rocking column system



(a) front view (b) lateral view (c) A–A section

Fig. 3 – Detailing of column base (standard level)

3. Design of Multiple Rocking Column Steel Structural System

3.1 Design of Rocking Column

Structure design of the rocking column system follows an elastic design principle under frequent earthquakes according to a Chinese seismic design code [5], in which members are required to be in elastic states. When subjected to DBE and MCE level earthquakes, the beam–column and column base joints are expected to rock to reduce structural internal forces. Beams and columns will be kept in elastic range under MCE level, while AFCs can dissipate energy and reduce maximum story drift. Inter–storey drifts should be limited to avoid excessive damage of non–structural elements. For the above reasons, stress ratios of the columns and beams under frequent earthquakes are relatively small compared with conventional design methods.

Section of the rocking column is chosen as HM200×165×10×20 mm (sections specified in a Chinese code). Sections of other structural members are shown in Table 1, where the beam and column sections are standard cross sections specified in a NZ code. The load information is shown in Table 2.

Table 1 – Sections of structural members

Member labels	Section information
C1/C2	254×254×89 UC
C3	200 ×10 SHS
B2/B3/B4	305×165×40 UB
Slab	Comflor 80 150 concrete fill

Table 2 – Load information

<table>
<tr><th>Level</th><th colspan="2">Dead load</th><th>Live load
(Additional mass)</th><th>Seismic load</th></tr>
<tr><td>1</td><td rowspan="3">B2: 5.87 kN/m
B4: 11.75 kN/m</td><td>0.12 kPa</td><td>3.53 kPa</td><td rowspan="3">According to the Chinese seismic design code: Beijing, with seismic fortification intensity of 8, with a PGA of 0.2g for DBE level of input ground motion and a PGA of 0.4g for MCE one.</td></tr>
<tr><td>2</td><td>3.20 kPa</td><td>3.53 kPa</td></tr>
<tr><td>3</td><td>3.26 kPa</td><td>4.71 kPa</td></tr>
</table>

The dead load includes self-weight of structural and non–structural elements. Weight of the composite floor is sustained directly by surrounding beams, in which the floor is deemed as a one–way slab with an effective thickness of 121 mm. Weight of beams and columns is calculated automatically by SAP2000. Since the live load is determined in the test, it is regarded as additional mass in the analysis model.

The beam–column connection will not rock under frequent earthquake level, and can be regarded as rigid during this stage. In this case, the structure becomes a moment–resisting frame, and the stress ratios of structural members can be obtained using a SAP model. Stress ratios of one bay of the frame is shown in Fig. 4. The maximum stress ratio of rocking columns under frequent earthquake is 0.20, and that of the beams 0.41.

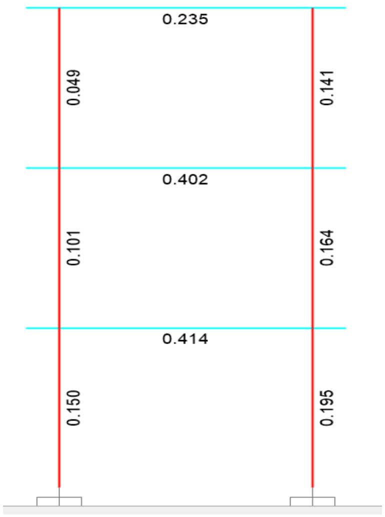


Fig. 4 – Stress ratios of members in rocking column system
(load combination: 1.0 ×dead load + 1.0 ×Y earthquake, 0.10g)

3.2 Design of Column Base

3.2.1 Design philosophy

The columns are not allowed to rock when subjected to frequent earthquakes, which requires that the earthquake–induced moments of the column base joints should be less than the corresponding resistant moments of the connections. Meanwhile, the columns will rock when the system is under an earthquake exceeding the frequent earthquake level.

In the case of DBE and MCE, storey drifts of the system are estimated by a ductility coefficient. The rotation angles of the column end plates can be calculated from these drifts, ensuring that all the members of the system be in elastic states. Under DBE, the drift angle can be estimated by multiplying the elastic drift by the ductility ratio. Under MCE, the drift angle can be obtained by multiplying the DBE one by a factor of 2.0 (The PGA of MCE is twice of DBE in China). A mechanical model of the column base is presented as shown in Fig. 5.

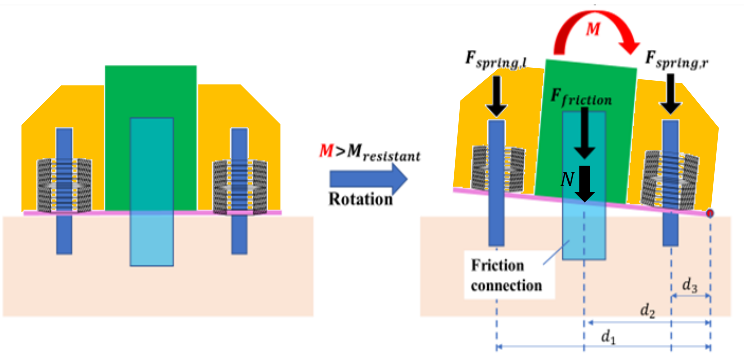


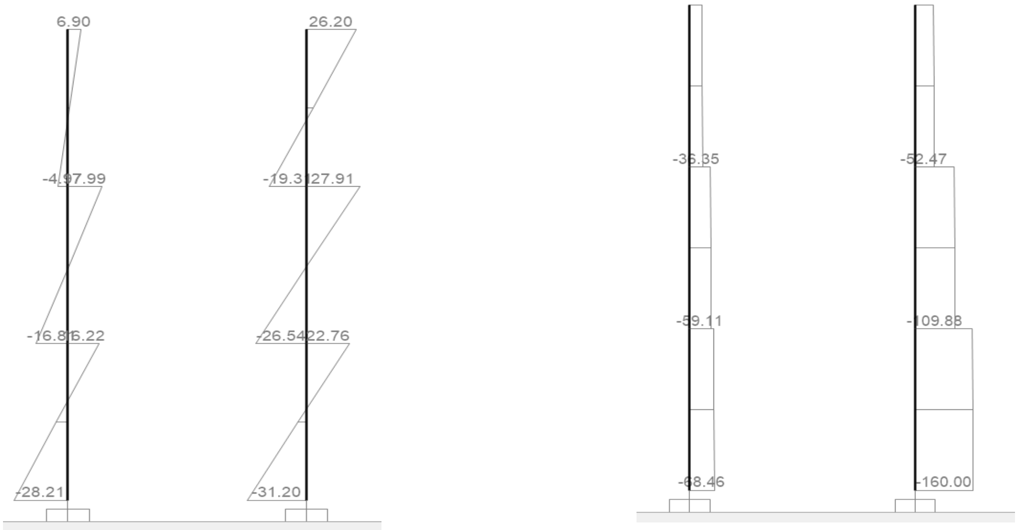
Fig. 5 – Mechanical model of rocking column base joint

According to the mechanical model shown in Fig. 5, resisting moment of the connection is contributed by AFC, mould coil springs and column axial force. However, the column axial force is unknown, without which the resistant moment of column base joint can be given as follows,

$$M_{resistant} = F_{spring,l} \times d_1 + F_{friction} \times d_2 + F_{spring,r} \times d_3 = M_{spring} + M_{friction} \quad (1)$$

3.2.2 Moment of column

Internal forces of the rocking columns (load combination: 1.0×dead+1.0×Y earthquake, 0.10g) is shown in Fig. 6.



(a) moment (kN·m) (b) axial force (kN)

Fig. 6 – Internal forces of structural members (0.10g).

Design of the connections is based on internal forces obtained through pushover analysis results under the MCE level, and the related maximum moment is used to check the capacities of the connections. This is to make sure that the column end plates are strong enough without yielding.

3.2.3 Asymmetric friction connection

Borzouie et al. [6] proposed a calculation formula for column base joints with AFC in their research. Referring to their conclusion, contribution of AFC to the resisting moment can be obtained as,

$$M_{\text{friction}} = n_{\text{Bolt}} \times F_s \times d \tag{2}$$

where n_{Bolt} is number of bolts in AFC, F_s is friction force of a single bolt and d is distance between bolt group center and rotation center.

Width of the base plate, l, is 350 mm, and the base plate is regarded as a rigid plate when it rotates, so the rotation center will locate at the edge of the base plate. d is distance from the center of column width to edge of the base plate,

$$d = l/2 = 175 \text{ mm} \tag{3}$$

For each AFC, four M16 high–strength friction bolts with a Grade of 10.9 are used. The friction force of a single bolt can be obtained,

$$F_{\text{friction}} = \Phi V_{\text{fss}} = 0.8 V_{\text{fss}} = 0.8 \times 2 \times 0.21 \times 100 = 33.60 \text{ kN} \tag{4}$$

$$M_{\text{friction}} = n_{\text{Bolt}} \times F_s \times d = 4 \times 33.6 \times 0.175 = 23.52 \text{ kN·m} \tag{5}$$

3.2.4 Mould coil spring

Standards mould coil springs are employed in this study, and their dimension information is as follows. SB60×30×150 mould coil spring is used and its limit deformation is 24% of its length, i.e., 36 mm, and its stiffness is 470.40 N/mm. In addition, no pre–compression is applied to the springs considering good self-centering properties of the proposed rocking column system.

3.2.5 Rotation behaviour of column base joint considering column axial force

It can be predicted that the left bottom column base joint in Fig. 6 is the first one to rotate according to the above design procedure. In the design method for column base joint, relationship between resisting moment (with column axial force considered) and earthquake–induced moments under an earthquake load of 0.10g can be obtained as,

$$M_{\text{resistant}} = 23.52 \text{ kN·m} > (M - Nd_1) = 28.21 - 68.46 \times 0.175 = 16.23 \text{ kN·m} \tag{6}$$

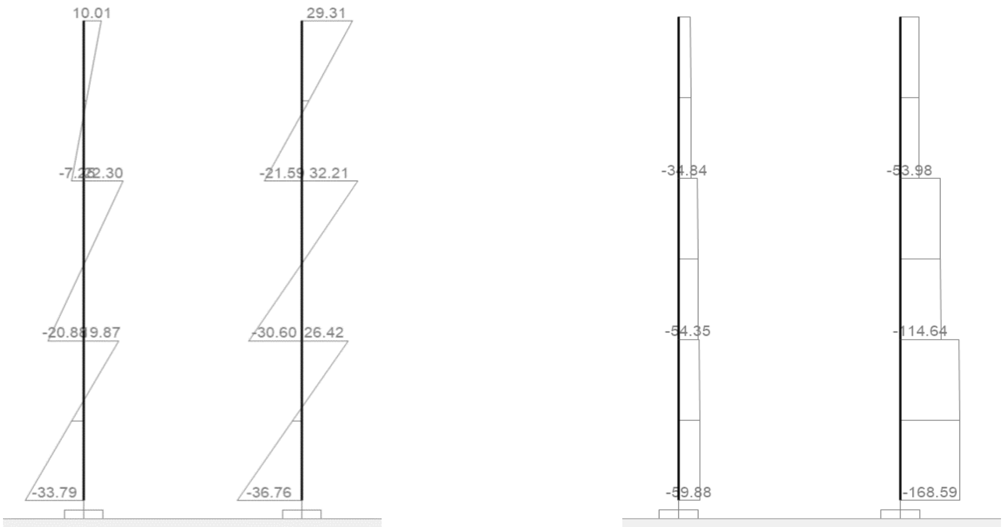
From the above result, the column base joint will not rock. The actual joint moment of the SAP model is 28.21 kN·m under an earthquake load of 0.10g, where the corresponding maximum seismic influence coefficient, $\alpha_{\max}$, is 0.16. Considering the column axial force, the left bottom column base joint will rock when $\alpha_{\max}$ equals 0.19. Internal forces of rocking columns under a new earthquake load of 0.12g (corresponding to $\alpha_{\max} = 0.19$) are shown in Fig. 7, and the drifts are given in Table 4. To check validity of the design method, condition of the left bottom column base under an earthquake load of 0.12g is checked,

$$M_{\text{resistant}} = 23.52 \text{ kN} \cdot \text{m} \approx (M - Nd_1) = 33.79 - 59.88 \times 0.175 = 23.31 \text{ kN·m} \tag{7}$$

To ensure all the three floors start to rock at the same time, resistant moments at each floor need to be modified. The resistant moments should be close to the corresponding earthquake–induced moments. Therefore, the newly designed resistant moments for the first to third floor are 23.31 kN · m, 10.73 kN · m and 19.86 kN · m, respectively.

Table 4 – Inter-story drift (0.12g)

Story	Displacement (mm)	Inter–story drift (mm)	Inter–story drift angle
3	20.60	5.60	1/534
2	15.00	8.40	1/357
1	6.60	6.60	1/455



(a) moment (kN·m) (b) axial force (kN)

Fig. 7 – Internal forces of the rocking columns (0.12g).

The moment–rotation curve of the column base joint is as shown in Fig. 8.

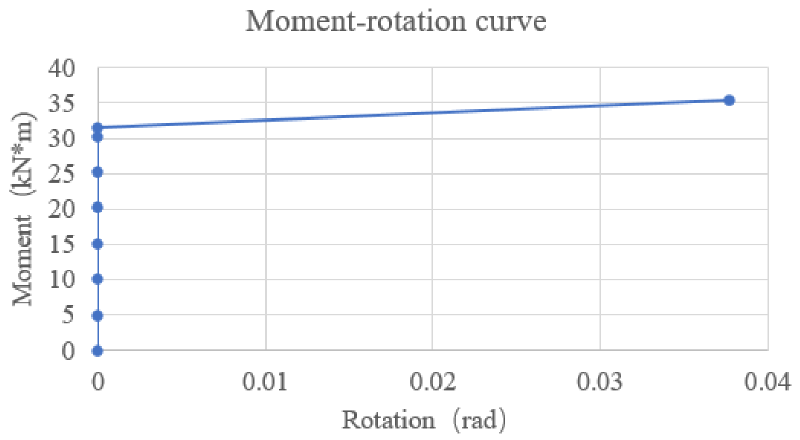


Fig. 8 – Moment-rotation curve of column base joint

According to Table 4, the average story drift can be obtained as Δ_e=6.87 mm per floor. Ductility coefficient, μ, is 3, so one can estimate the inter-story drift under DBE earthquake,

$$\Delta_{DBE} = \Delta_e \times \mu = 6.87 \times 3 = 20.60 \text{ mm} \tag{9}$$

The inter-story drift under MCE,

$$\Delta_{MBE} = \Delta_e \times \mu \times 2 = 6.87 \times 3 \times 2 = 41.20 \text{ mm} \tag{10}$$

3.2.6 Dimension of holes on AFC connection plate

Borzouie et al. [6] suggested to design the holes according to the limits of the maximum inter-storey drift angle,

$$D = 2 \times 1.25 \times (\theta_{\mathrm{rock}} \times W_1) + d_{\mathrm{sh}} + 2 \text{ mm} \tag{11}$$

where θ_{rock} is the limit rotation angle of the joint (3%); W_1 is the maximum distance of bolts and rotation center, $W_1 = 237$mm; d_{sh} is the diameter of the screw, d_{sh}=16mm; 1.25 is a safety factor. Then,

$$D = 12 \times 1.25 \times 0.03 \times 237 + 16 + 2 = 36 \text{ mm} \tag{12}$$

3.2.7 Consideration of the self-centering force

Assuming that the rocking column rotate as a rigid body. When the deformation is 3%, the vertical gravity acting on the upper part is beneficial to the self–centering of the joints. This section is to check whether the gravity effect should be considered or not when there is on spring preload.

When the deformation is 4%, the force arm of gravity can be roughly estimated as

$$l_{\mathrm{d}} = 350 - 3000 \times 3\% = 260 \text{ mm} \tag{13}$$

Take the bottom column for verification, and the axial force of the bottom column is 162.70 kN from Fig. 6(b).

The self-centering moment of the gravity effect of the bottom column can be calculated as,

$$M_G = N \cdot l_d = 160.0 \times 0.26 = 41.60 \text{ kN·m} > 23.52 \text{ kN·m} \tag{14}$$

where 23.52 kN is the resistant moment of friction.

It can be seen that the self-centering effect caused by gravity can completely overcome the frictional resistance and reach the self-centering of the joint.

4. Conclusions

This paper proposes a multiple rocking column steel structural system and corresponding design method. The testing will be conducted in August 2020 at ILEE facilities, Shanghai, China. This test is expected to lower high mode effect for high rise strctures because of the single rocking interface and demonstrate validity of the newly proposed mutiple rocking steel structural system in achieving a low-damage structural system.

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