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Seismic performance of vertical irregular hybrid spatial structures

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Abstract

Shanghai pudong international airport terminal building 2 has total floor area of 400 thousand square meters and constitutes a terminal building, a boarding hall and mass transit stations. Its total length and width are 414 and 150 meters, respectively. In addition, it is a hybrid spatial structure, in which the upper steel roof and lower RC frame are assembled with Y-shaped steel columns. It could also be classified as a vertically irregular structure due to irregular dirstribution of the transverse stiffness. Up to now, no Chinese Design Code can be applied efficiently to this type of structure.

The objective of this paper is to study the seismic performance of this hybrid spatial structure by means of nonlinear time history analyses when subjected to seismic excitations. Some important responses such as internal force, displacements are monitored and compared between different cases. The seismic performance of this structure is evaluated. The study attempts to investigate the weak point and accumulates the analytical evidence for establishing related design guidelines for such complex hybrid spatial structures in the future

Keywords: hybrid spatial structure, vertical regularity, nonlinear analysis, seismic performance.

1. Introduction

The target spatial structure considered in this study is terminal building 2 in Shanghai pudong international airport as shown in Figure 1. Pudong International Airport, Shanghai, China has total floor area of 400 thousand square meters and constitutes a terminal building, a boarding hall and mass transit stations (Figure 1). Terminal building 2 consist of 25 planar frames with 18m spacing. 3 hybrid columns (in 0/1A, A and G axis) and 1 pin-pin supported steel column (in K axis) serve as horizontal resistance together with beam string system. In addition, it is a hybrid structure, in which the Upper steel roof and lower RC frame are assembled with Y-shaped hybrid columns. Terminal building 2 could be classified as a vertically irregular structure due to SRC column, Y-shaped steel column and steel roof along the height. The unique design of its RC frame and Y-shaped steel column make it become an exceptional structure. Up to now, no Chinese Design Code can be applied efficiently to this type of structure.

Given these irregularities and complexity of the structure, the owner entrusted the State Key Laboratory for Disaster Reduction in Civil Engineering at Tongji University to perform a detailed study to verify the seismic safety and rationality of the design. Generally, the detailed study includes (i) a shaking table test of the scaled building model to study its overall seismic behavior [1], (ii) finite element analysis of the whole building to determine its dynamic response, and (iii) a static test on joints between SRC column and Y-shaped steel column. This paper presents mainly the results of the second part.

Figure 1: Panorama of Pudong international airport

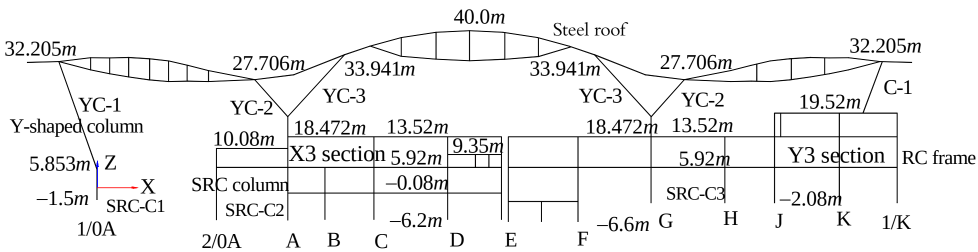


Figure 2: Elevation and major componet of terminal building 2

2. Customer developed material subroutine in ABAQUS

Many different models have been proposed so far for the stress-strain relationship of the material used in an RC member. In general, for each material the model for the monotonic response of the material serves as the envelope curve for the hysteretic behavior model.

Numerous tests have shown that the monotonic stress-strain curve for reinforcing steel can be described by three well-defined branches. This is generally the case for approximately all kinds of the reinforcing steel used in RC members. Moreover, Concrete is composed of two parts, confined concrete and cover concrete. Various models have been proposed to model the stress-strain relationship of confined and in turn unconfined concrete. The model employed in theoretical predictions plays a basic role in the compatibility of the data with the test results. Each model seems to have efficiency for a specific situation, while not for others. The models developed by the Asadollah Esmaeily-Gh [2] have been used in the present paper as shown in Figure 3. The typical hysteretic response of the cover and confined concrete when subjected to reverse loading is plotted in Figure 4.

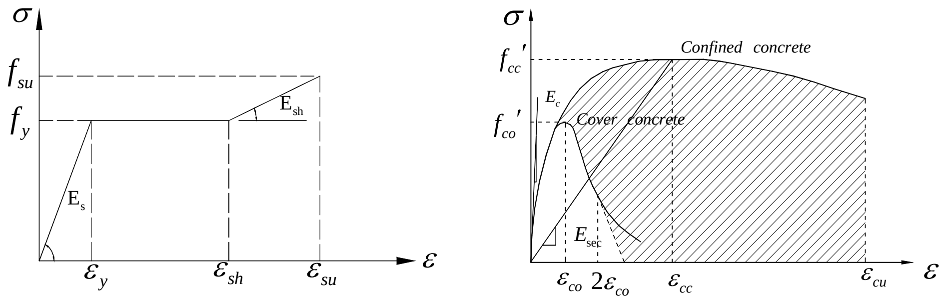


Figure 3: Model for monotonic response of the steel and concrete

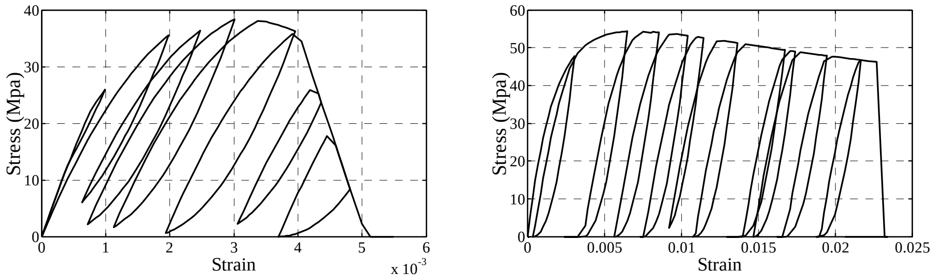


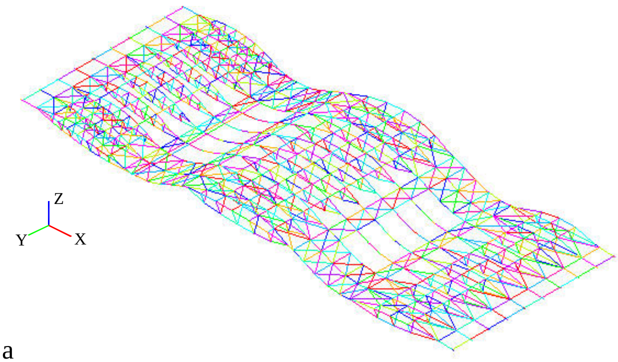
Figure 4: Typical hysteretic response of the cover and confined concrete

3. FE model of terminal 2 building

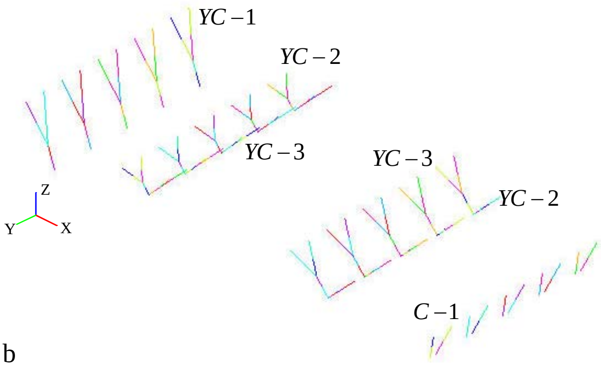
3D beam elements were selected for the beams and columns, and shell elements were used for the floor slab, respectively. Detailed information of the analytical model is listed in Table 1. Figure 5 shows the major three part of the analytical model, in which the longitudinal and lateral direction are defined as axis *X* and Y, respectively. The vertical direction as axis *Z*.

Table 1. Information of the finite element model

FE Model information	Number
Beam elements	8785
Link elements	764
Shell elements	4576
Nodes	11957

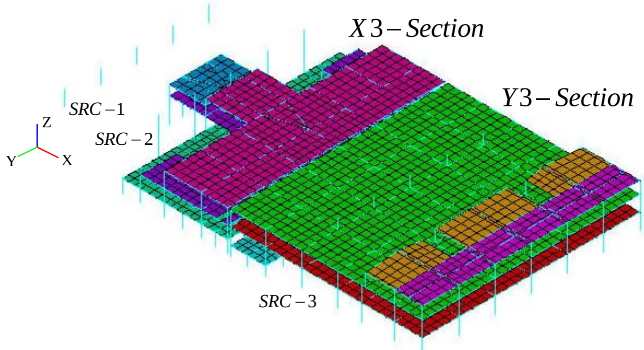


a



b

Figure 5: Three part of the terminal building 2 (a) steel roof; (b) Y-shaped column



c

Figure 5: Three part of the terminal building 2 (continue) (c) RC frame

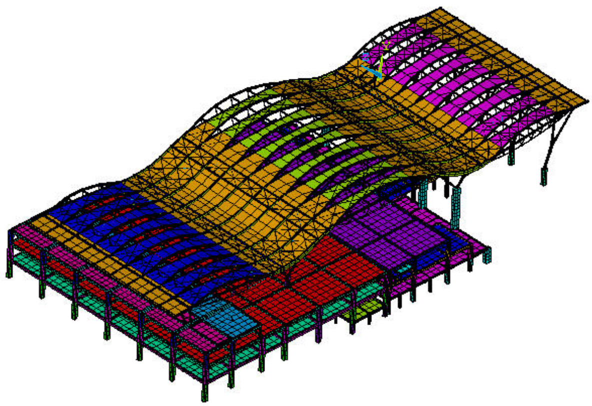


Figure 6: Full analytical FE model of terminal building 2

4. Results of the finite element analysis

4.1. Dynamic characteristics of the structure

Table 2 lists the analytical results of the first three frequencies and Figure 7 shows the shapes of vibration modes. In addition, the comparsion between the analytical results with those results extrapolated from the shaking table test are also presented. Due to the complexity of the structure, they are agree with each other in an acceptance level.

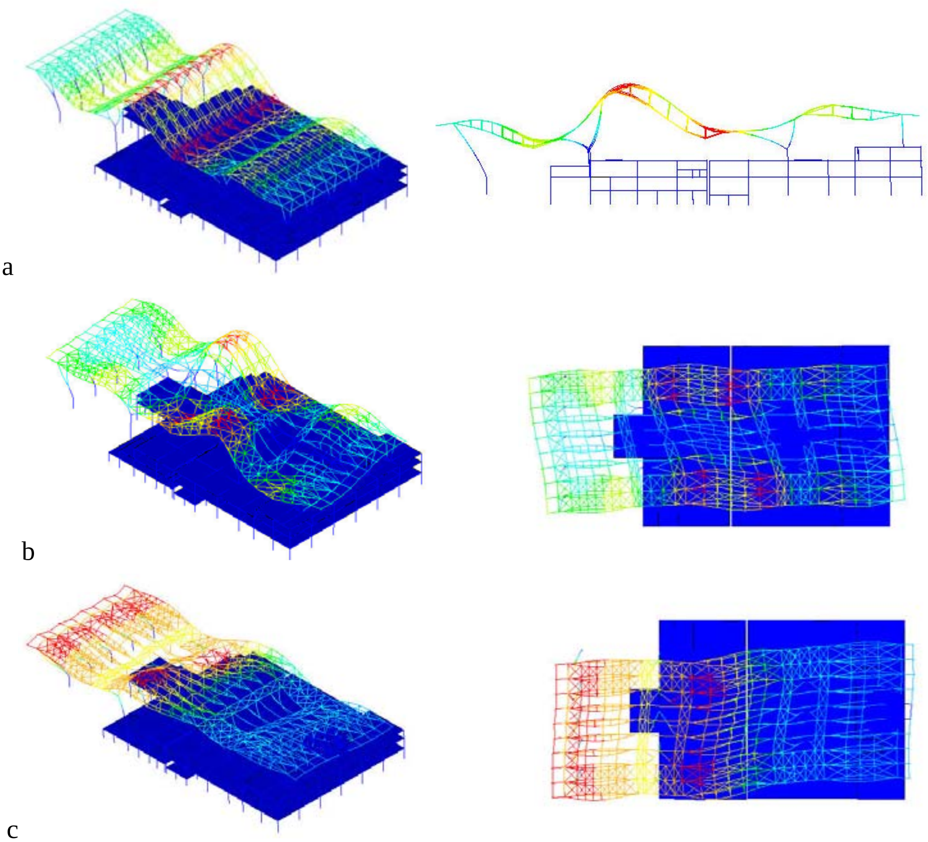


Figure 7: Fist three vibration mode. (a) First vibration mode (translation in X); (b) Second vibration mode (warp and torsion); (c) Third vibration mode (translation in Y)

Table 2. Experimental and analytical results of the prototype structure

Mode	Experiment (Hz)	Numerical analysis (Hz)	Error
1	0.594	0.574	3.37%
2	0.891	0.801	10.1%
3	1.018	0.984	3.34%

4.2. Verification of customer developed material subroutine

The constitutive model demonstrated above have been programed into a material subroutine and then complied into the well known finite element software ABAQUS [3]. Before it has been used in the analysis of the complex structure, a series of benchmark work has been done. As shown in fiure 8, the displacement history of the structure when subjected to SHW2 wave were compared to the results obtained from the shaking table test.

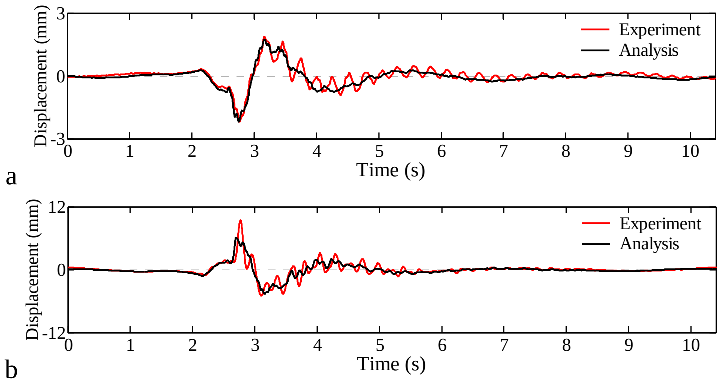


Figure 8: Comparsion of the displacemnt response at the top of the Yc-2 column in axis A. (a) Peak vaule of the acceleration is 0.035g; (b) Peak vaule of the acceleration is 0.40g

It is shown that the displacement agrees quite well when the peak value of the input acceleration is 0.035g, while not very good in the case of the 0.40g. The authors think the reason is the damge accumulation when the model subjected to a series of input acceleration during the whole shaking tabe test. While it is impossible taking it into account in the current nonlinear time history analysis. Due to the trends of displacemnt resopne agree with each other is well, the following nonlinear analysis is continued.

4.3. Analysis case

Condition of site soil is one of the important factors to determine the earthquake inputs for dynamic test. Considering the spectral density properties of Type-IV site soil, El Centro wave and SHW2 (Artificial wave simulated accoring to the site soli of Shanghai, China) wave are selected.

According to China code, frequent, basic and seldom occurrences represent three peak levels of ground motions with intensity less than, equal to and higher than the design intensity, respectively. Three different requirements related to the three levels, are set to evaluate the overall capacity of structure under corresponding intensity. Since the design intensity in Shanghai is specified as 7, the analysis is carried out in four phases representing frequent, basic and seldom occurrences of design intensity 7, and seldom occurrence of design intensity 8, respectively. The last phase is utilized for further investigation of dynamic responses of the targeted structure under extremely strong earthquakes. A summary of the inputs is listed in Table 3. The peak acceleration ratio of the principal direction to the other horizontal direction and vertical direction are designed to be 1 to 0·85 and 1 to 0.65, respectively.

In addition, in the analysis, the slab was assumed to be elastic and the damping ratio to be 0·035.

Table 3 Analysis case

Earthquake Occurrence Frequency	Input signal	Peak value of input acceleration (g)		
		X	Y	Z
Frequent 7	El Centro	0.035	0.03	---
		---	---	0.023
	Shw2	0.035		
			0.035	
				0.23
Basic 7	El Centro	0.10	0.085	
				0.065
	Shw2	0.10		
			0.10	
				0.065
Rare 7	El Centro	0.22	0.19	
				0.14
	Shw2	0.22		
			0.22	
				0.14
Rare 8	El Centro	0.40	0.34	0.26
	Shw2	0.40		
			0.40	
				0.26

4.4. Displacement response

For each recorded displacement time history, the maximum values can be found. However, these maximum values at different locations do not necessarily occur at the same point in the same time. In order to investigate the structural displacement responses under different earthquake levels, the maximum displacement obtained from two series of waves were compared to get the final maximum values. Figure 9 demonstrates the maximum displacement distributions along the height under four earthquake levels.

It is shown that, among the three SRC columns, the SRC columns located at the axis 1/0A and G may undergo the maximum and minimum displacement response, respectively. This phenomenon coincide with the obvious crack appeared at the top of SRC column in axis A and G compared to the axis 1/0A. For the Y-shaped steel column, the YC2 located in axis A may experience the largest displacement compared to the other steel columns. Moreover, compared to the SRC columns, the Y-shaped steel columns may exhibit the larger displacement response. The irregular distribution of the stiffness leads to the bigger difference in the displacement response along the height.

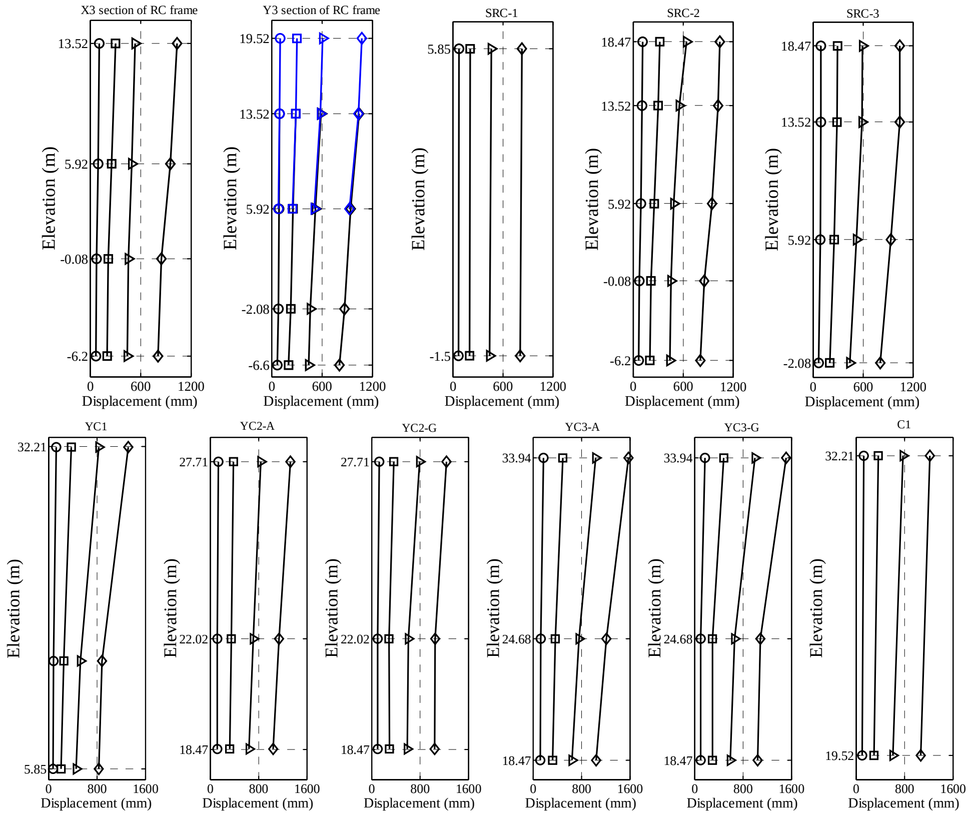


Figure 9: Envelop of displacement in major component of strucutre

4.5. Force response

Similar to the displacement response, the maximum forces in some key elements are obtained and shown in Figure10～12. The SRC column is dominated by flexure and compression. Due to the pin-pin connection of the Y-shaped column and the steel roof, the bending moment at the top of YC steel column is small. The axial force of the YC column is quite large and distribute evenly. Compared to the YC2 column, the C1 column is only dominated by the axial force.

Carefully observation in Figure 11, we can draw a conclusion that the curved beam in steel roof is dominated by the flexure, and the maximum bending moment appears at the axis A and G. It is verified that aggrandizement of the beam section in such position is advisable. The envelopes of stress in web member and lower chord demonstrate that these element exhibit elastic even in the case of rare 8.

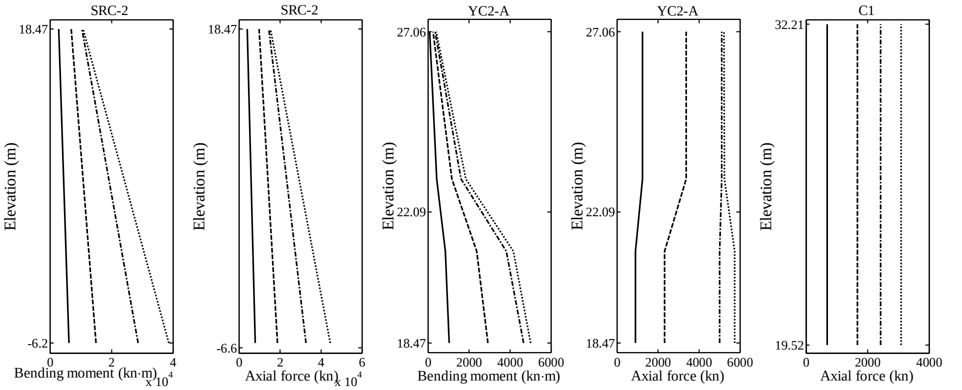


Figure 10: Envelop of the element force (SRC and Y-shaped steel column)

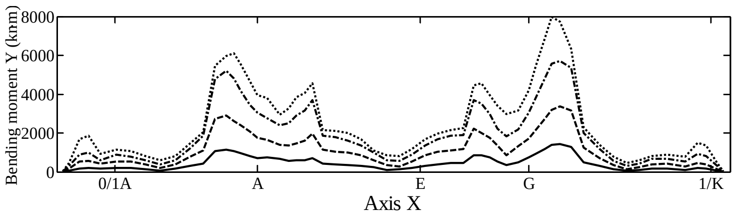


Figure 11: Envelop of bending moment envelop (curved beam in steel roof)

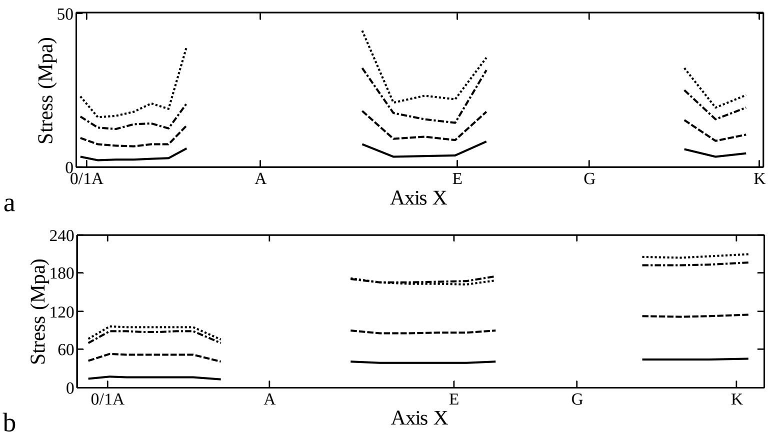


Figure 12: Envelop of stress (a) web memer; (b)lower chord in steel roof

4.6. Hysteretic response of key element

The hysteretic response of typical SRC and Y-shaped steel column is plotted in Figure 13～14. It is shown that in the case of frequent 7 (PGA=0.035g), all columns behave in the range of elasticity. While in the case of rare 8 (PGA=0.4g), many columns can enter into the range of plasticity. This phenomenon coincides with the observation in the shaking table test. The joint between the SRC column and Y-shaped steel column is the weak position for the whole structure.

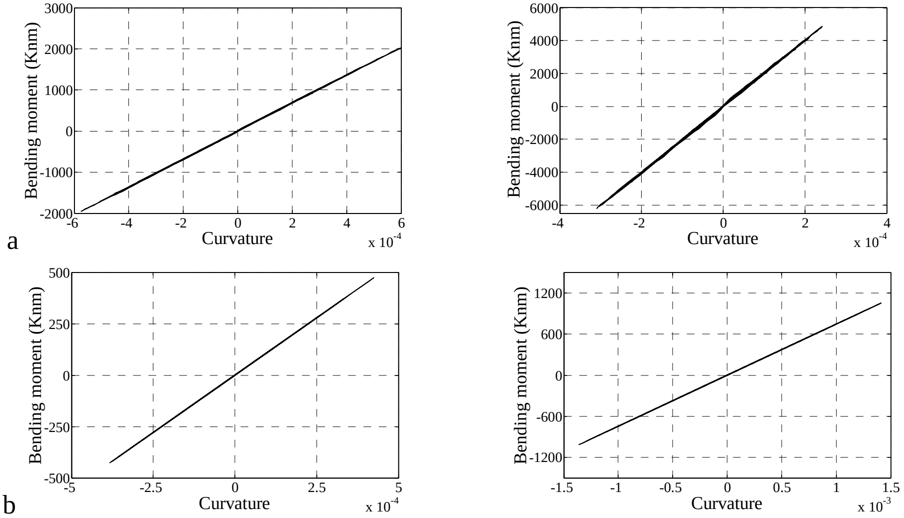


Figure 13: Hysteretic response of (a) SRC and (b)Y-shaped steel column (PGA=0.035g)

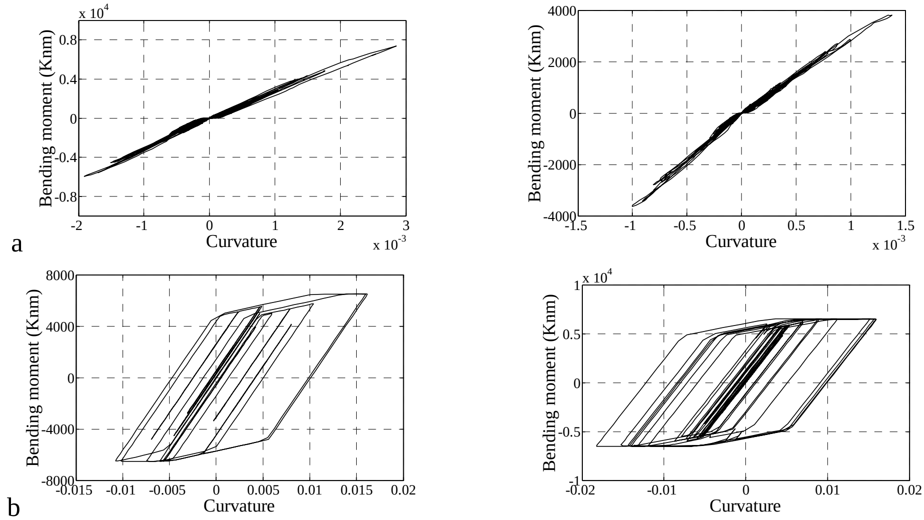


Figure 14: Hysteretic response of (a) SRC and (b)Y-shaped steel column (PGA=0.4g)

5. Conclusions and suggestions

As a typical vertical irregular hybrid spatial structure, the seismic behavior of terminal building 2 in Shanghai Pudong international airport has been analytically investigated using the customer developed software ABAQUS. The following conclusions can be drawn from the analytical results.

(1) The customer developed material subroutine is proposed and developed in the present paper, the benchmark work shwon that it has a good accuracy in the nonlinear time history analysis.

(2) The model test results indicate that the structure is able to withstand frequent occurred, basic intensity and rare-occurred earthquakes of intensity 7 without sever damage. The structural system in this building demonstrates good quality in resisting earthquakes.

(3) The big dynamic response difference exists between the upper steel roof and lower RC frames due to the irregular distribution of the stiffness.

(4) The web member and chord in steel roof exhibit elastic even in the case of rare 8 (PGA=0.4g), so these members is strong enough for this type of structure.

(5) The SRC and Y-shaped steel column are dominated by the flexure and axial compression, and the joint between these two key componts is the weak position for the whole strucutre. Design measures are suggested to increase the ductility to avoid extensive deformations. In detail:

(a) For SRC columns of the ground floor: Control the axial-compressive ratio; increase the reinforcement; configure the small-spacing stirrups along the column.

(b) For the joints of SRC column and Y-shaped steel column: prolong the encased steel column up to the SRC columns.

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