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Effect of column-foundation connection stiffness on seismic performance of rocking steel braced frame

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Abstract

Residual drifts after severe earthquakes interrupt serviceability of buildings. Retrofitting of such buildings is in many cases very difficult and consumes lots of time and money. Recently, there are some attempts to develop the seismic design procedures to not only satisfy life safety criteria but also lead to more economical buildings. One of these modern methods of improving seismic performance of the steel structures is using systems with ability of rocking. The main features of these new systems are to concentrate the damages in specific easily repairable locations of structures, to dissipate more energy and to reduce and limit the residual deformations. In this paper the effects of the column-foundation connection stiffness on the seismic performance of rocking steel braced frame are studied. Nonlinear dynamic time history analyses are applied, using seven far-field ground motion records in two intensity levels. The response parameters are mean of the maximum vertical accelerations of rocking columns, drifts, performance levels, in addition of the positive and negative vertical displacements. The results of this study indicate the positive effect of increase in the tensile stiffness and decrease of compressive stiffness in the column base connection on vertical accelerations, uplift and performance levels in the models with viscous dampers.

Keywords: rocking steel braced frame, viscous dampers, column-foundation connection, compressive stiffness and tensile stiffness.

1. Introduction

Higher building performance can be achieved by minimizing inelastic deformation and damage in primary structural elements and reducing residual drifts. Generally it is neither practical nor economical to strengthen conventional seismic systems to achieve higher performance. However, controlled rocking steel braced frames are seismic force resisting systems that almost completely eliminate residual drifts and concentrate the damages in replaceable ductile fuses. In recent years extensive research has been done concerning low damage self-centering systems. The first practical and engineering application of controlled rocking self-centering structures is probably the design and construction of Rangitikei Railway bridge in New Zealand[1]. Recently there are much attention to these systems, in 2004 Palermo et al studied application of controlled rocking in the seismic design of bridges[2]. Weibe proposed the inclusion of multiple rocking sections along walls height to reduce the demands caused by higher mode force effects in a multi-story building[3]. Tremblay et al. have investigated a similar rocking braced frame concept for seismic resistance of building structures but have investigated implementation of nonlinear fluid viscous dampers as the energy dissipation device at the base of the rocking frame column[4]. In 2009 and 2010 Eatherton et al investigated seismic design and behavior of 2D steel frames with controlled rocking motion[5]. Pollino et al proposed a similar rehabilitation technique for sub-standard steel framing utilizing large pin-supported steel columns or trusses[6]. In 2015 Pollino discussed the behavior and seismic design approach for rocking steel braced frame buildings with both steel yielding and viscous damping devices and a simplified

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approach is proposed to quantify peak dynamic deformation and force responses[7].

The main aim of this paper is to evaluate the seismic performance of rocking steel braced frame with the tensile and compressive stiffness in the column base. The effect of the response parameters are investigated in nine steel braced frame with the rocking motion modeled by SAP2000 v18.1.1[8] subjected to seven far-field ground motion records in two intensity levels (peak ground acceleration equal to 0.35g and 0.7g) in nonlinear dynamic time history analyses. The responses such as mean of maximum vertical accelerations of rocking columns, drifts, performance levels, positive and negative vertical displacements are investigated. In the next part of this paper the 9 model are introduced, then the ground motion records are identified and finally the results obtained from nonlinear dynamic time history analyses are studied.

2. Analytical modeling

The base model studied in this research is a three story building with rocking motion based on a prototype building of the SAC project configuration[9]. Figure 1 shows a 3D view of the model. The damping of viscous dampers that are used in the column base connection is 8500 N.s/mm. The vertical post tensioning strands provide self-centering forces. The strands are initially stressed to 1128 KN. Plans, the loading and other information are provided elsewhere [10].

To model the possibility of the uplift at column base in SAP2000, gap elements that contains of opening and spring are used that are stiff in the compression, however, have zero stiffness in tension (Fig. 2). In the fact, the gap elements in this study do not have opening and as such the value of opening is considered zero. To evaluate the effect of the tensile stiffness on the rocking columns, this parameter is modeled using the hook element in SAP2000. Tables (1 and 2) display sections, gap, hook and fuses properties.

3. Earthquake records in nonlinear dynamic time history analyses

This study used seven far field records from the appendix A of FEMA p695[11] in two intensity level for nonlinear dynamic time history analyses. To evaluate the responses of the system, the horizontal components of the earthquake ground motions with the maximum acceleration are used. Table 2 displays selected records and their properties.

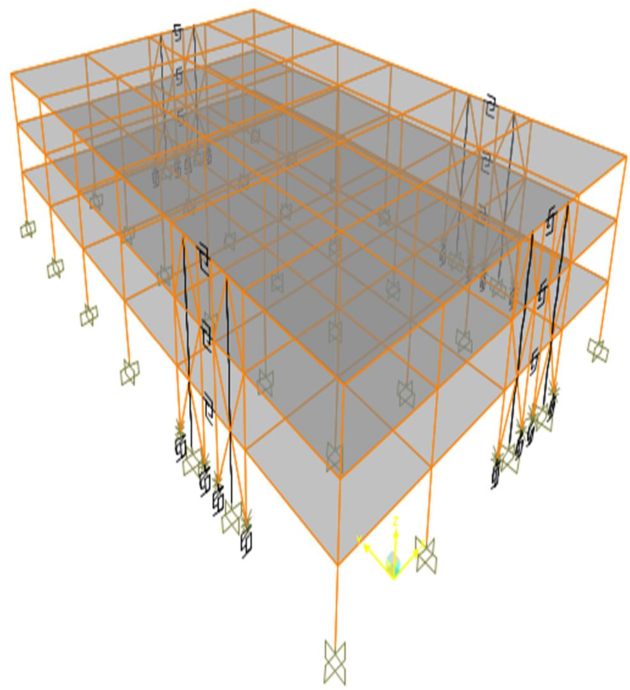


Fig1. Three dimensional view of studied model

Table1. Section properties

Non braced frames		Braced frames	
First story columns	W12×170	Columns	W12×305
Second and third story columns	W12×58	Beams	W10×68

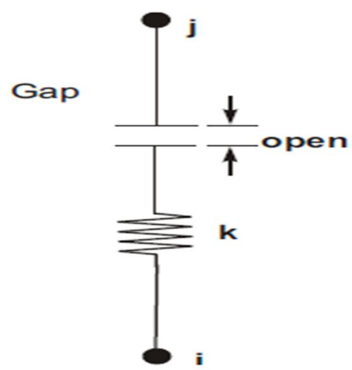


Fig 2. Gap element in SAP2000 (In this study open=0)

Table2.Models properties

	Viscous damper	Yielding fuse		Hook properties		Gap properties	
model	**C (N.s/mm)**	**K (kN/mm)**	**Fp (kN)**	**Open (cm)**	**K (ton/cm)**	**Open (cm)**	**K (ton/cm)**
1	8500	100	100	0	0	٠	500
2	8500	100	100	0	62.5	٠	500
3	8500	100	100	0	250	٠	500
4	8500	100	100	0	438	٠	500
5	8500	100	100	0	0	٠	1000
6	8500	100	100	0	125	٠	1000
7	8500	100	100	0	500	٠	1000
8	8500	100	100	0	650	٠	1000
9	8500	100	100	0	876	٠	1000

Table 3.Selected records properties used in nonlinear dynamic time history analyses

Record No.	Earthquake	station	Year	PGA(g)	M
1	Northridge	Beverly Hills-Mulhol	1994	0.516	6.7
2	Duzce, Turkey	Bolu	1999	0.822	7.1
3	Imperial Valley	El centro Array#11	1979	0.38	6.5
4	Kobe, Japan	Akashi-Nishi	1995	0.509	6.9
5	Kocaeli,Turkey	Duzce	1999	0.358	7.5
6	Loma Prieta	Gilroy array#3	1989	0.555	6.9
7	Manjil, Iran	Abbar	1990	0.515	7.4

4. Nonlinear dynamic time history analyses results

4.1. Comparison of vertical acceleration

In models with the viscous damper and the compressive stiffness equals to 500 ton/cm, increase in tensile stiffness at the column base connection, leads to decrease in the vertical positive acceleration. However, as seen in a recent study [8], the models with the smaller compressive stiffness (500 ton/cm) have the smaller vertical positive acceleration than the ones with the larger compressive stiffness (1000 ton/cm). In this case increase in tensile stiffness leads to decrease in the vertical acceleration up to 59%. Also, increase in tensile stiffness changes building seismic performance and transfers the peak acceleration from first story to second or third story and decreases the differences between stories vertical acceleration. As Fig. 3a shows vertical positive acceleration in the model with 250 ton/cm stiffness (model 3) is approximately equal to one with 438 ton/cm tensile stiffness (model4). As a result, increase in the tensile stiffness up to 250 ton/cm seems to be sufficient for decrease in the vertical positive acceleration in the models with 500 ton/cm compressive stiffness. In all models, by increasing intensity level, the vertical positive acceleration is also increasing (Fig. 3b) and shows that it is sensitive to level of intensity. The vertical negative acceleration doesn't significantly change when the tensile stiffness increases. However, the models with maximum tensile stiffness have the smaller vertical negative acceleration and seismic performances in all models are similar. In the models the with viscous damper that have compressive stiffness equals to 1000 ton/cm (Fig. 4), any increase in tensile stiffness leads to decrease in the vertical positive acceleration. For example, increasing the tensile stiffness up to 876 ton/cm, leads to decrease in the vertical positive acceleration up to 80% and 70% respectively in 0.35g and 0.7g intensity levels. As a result, existence of the tensile stiffness in the models with the higher compressive stiffness is more useful. In the models with the tensile stiffness more than 500 ton/cm, the building seismic performance is changed and caused transfer of the peak acceleration to the upper stories and decrease in the acceleration difference between stories. In the models with the tensile stiffness equals to 650 ton/cm and 876 ton/cm, as intensity level increases, the vertical acceleration have a negligible change and the two cases practically perform like each other. In two intensity levels, increasing the tensile stiffness doesn't have any significant effect on the vertical negative acceleration.

4.2. Comparison of drift

In the models with viscous damper, as the tensile or compressive stiffness increases, drift doesn't significantly change (Figs. 5-6). With increase in intensity level, again drift values remain almost unchanged.

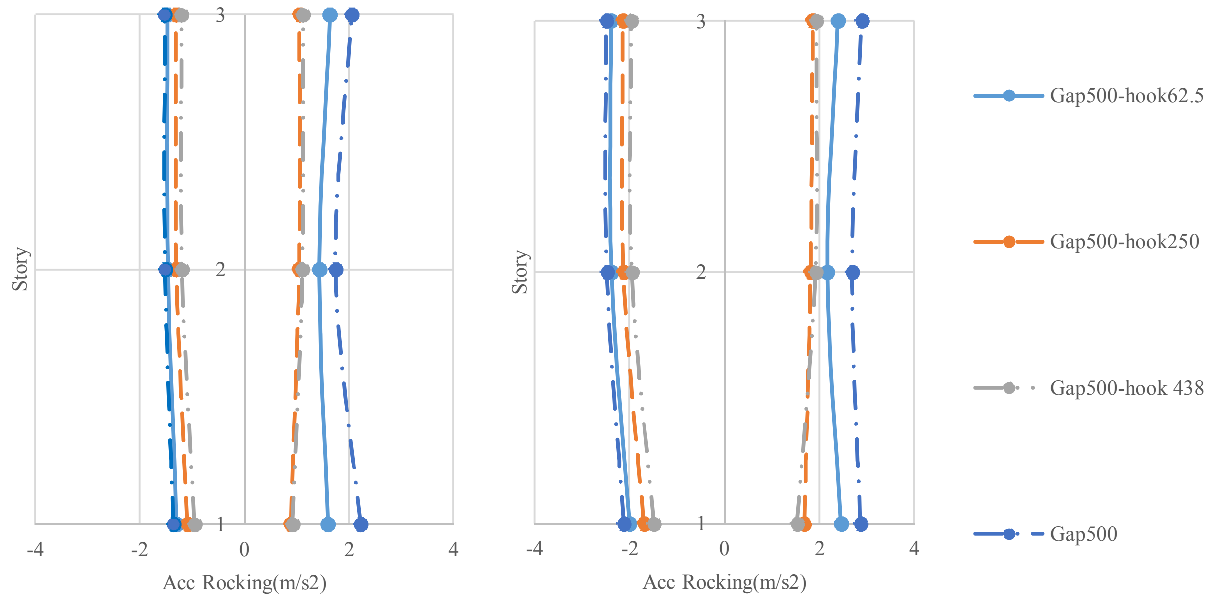
4.3. Comparison of rocking columns vertical displacement

Figures 7 and 8 indicate that increase in the tensile stiffness decreases the positive vertical displacement of rocking columns. However, this fact increases their negative vertical displacement. For example, increase in the tensile stiffness up to 438 ton/cm in models with 500 ton/cm compressive stiffness decreases the uplift up to 56% and 45% for 0.35g and 0.7g intensity level respectively. Negative vertical displacements also increase up to 48% and 80%. In all models, by increasing intensity level, vertical positive acceleration is also increasing (Figs. 7-8) that indicates that sensitivity to intensity level. The sum of the absolute positive and negative vertical displacements, especially in higher intensity levels was almost identical. As a result, existence of the tensile stiffness decreases the positive vertical displacement and increases negative vertical displacement, however, the total structural movement is constant.

In models with the viscous damper, increase in compressive stiffness from 500 ton/cm to 1000 ton/cm, leads to decrease in the positive vertical displacements and increase in the negative vertical displacements. For example, increasing the tensile stiffness up to 876ton/cm in 0.35g and 0.7g intensity levels leads to decrease in the positive vertical displacement up to 64% and 55% and increase in negative vertical displacement to 2 and 2.5 times.

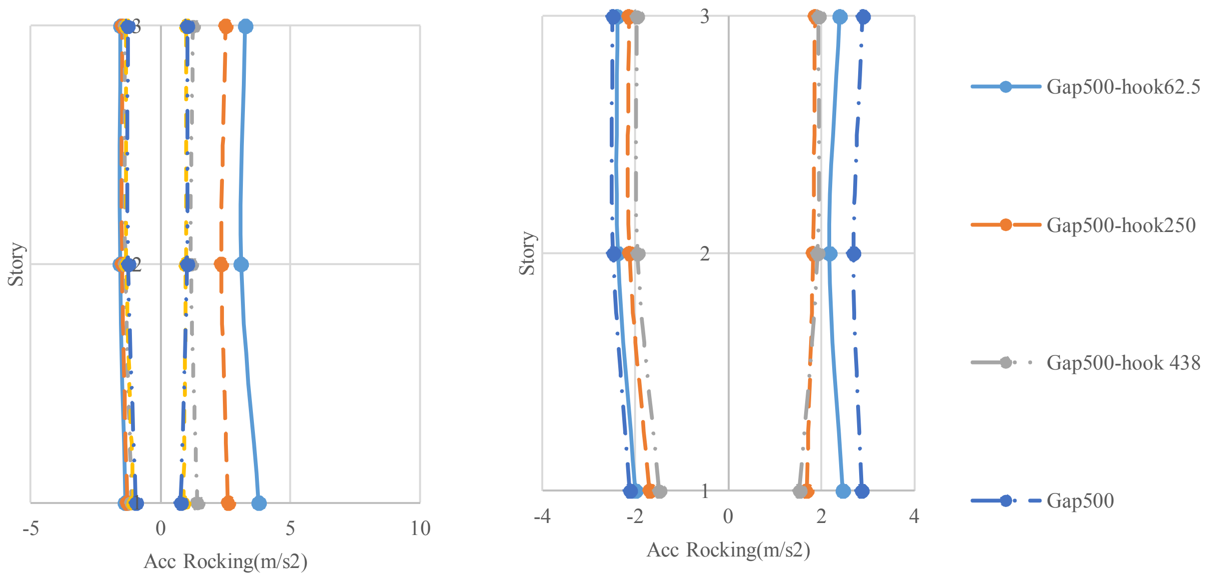
4.4. Comparison of models performance level

All models have linear performance in 0.35g intensity level. The models with low tensile stiffness have the best performance (model 2 and 6). Increasing tensile stiffness in models with compressive stiffness equal to 500 ton/cm, leads to increase in the number of plastic hinges. The number of plastic hinges that formed in the models with 1000 ton/cm compressive stiffness is more than the models with 500 ton/cm compressive stiffness (Fig9).



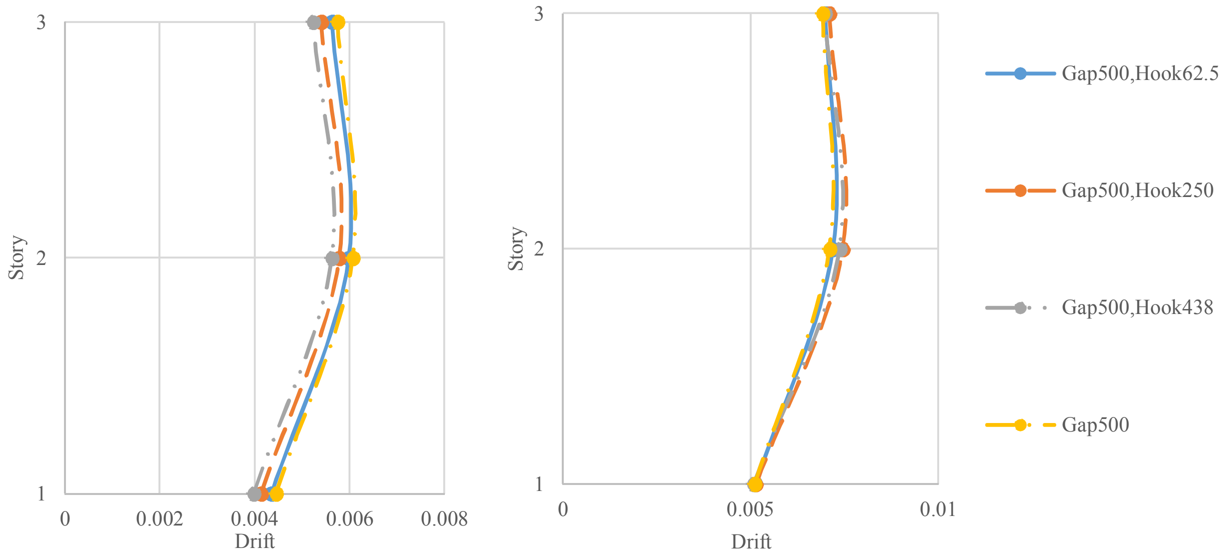
a) PGA equal to 0.35g b) PGA equal to 0.7g

Fig. 3 Vertical acceleration in models with 500 ton/cm compressive stiffness



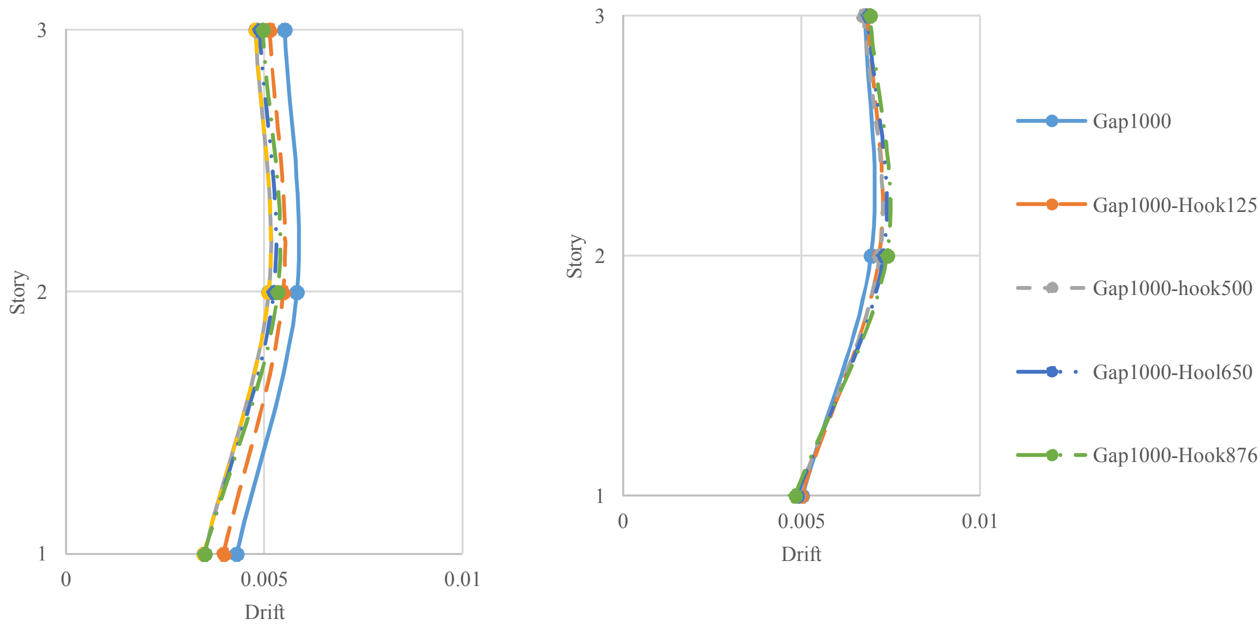
a) PGA equal to 0.35g b) PGA equal to 0.7g

Fig. 4 Vertical acceleration in models with 1000ton/cm compressive stiffness



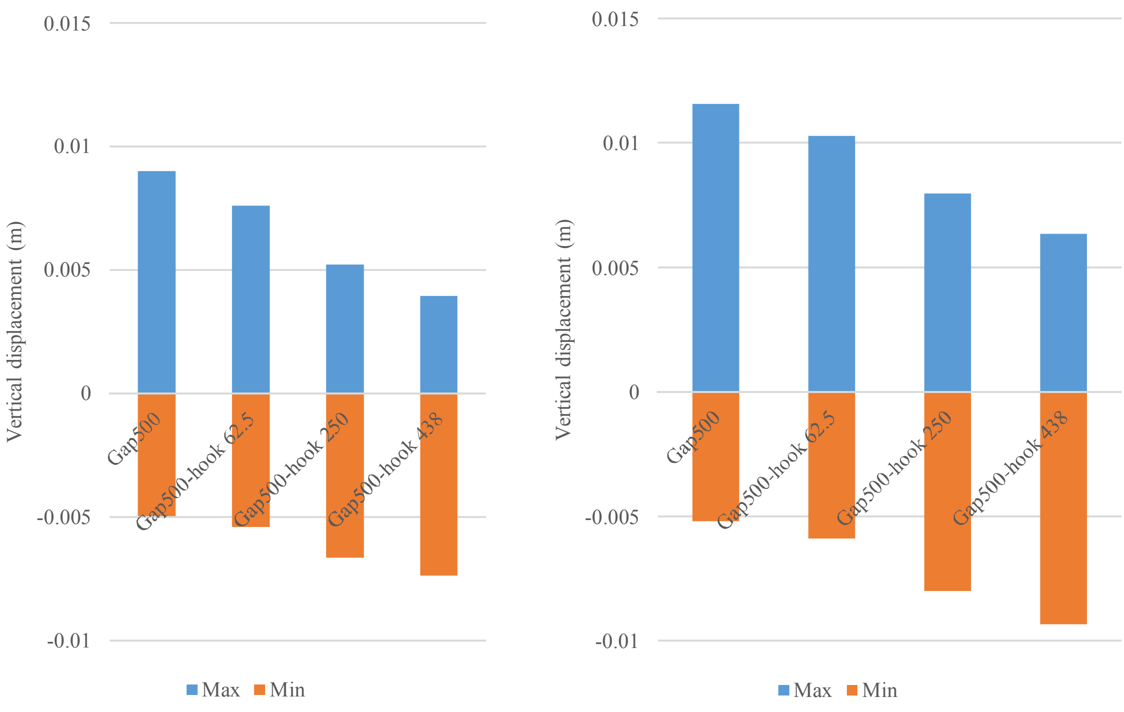
a) PGA equal to 0.35g b) PGA equal to 0.7g

Fig 5) drift in models with 500 ton/cm compressive stiffness



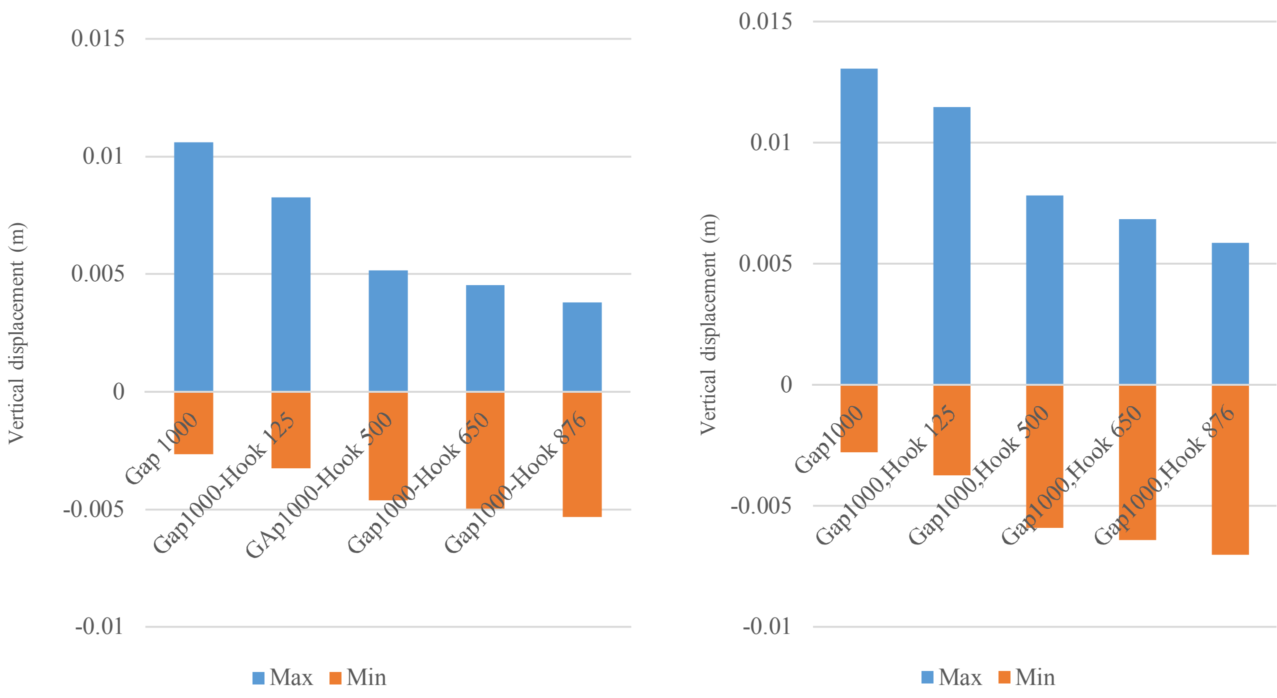
a) PGA equal to 0.35g b) PGA equal to 0.7g

Fig. 6 drift in models with 1000 ton/cm compressive stiffness



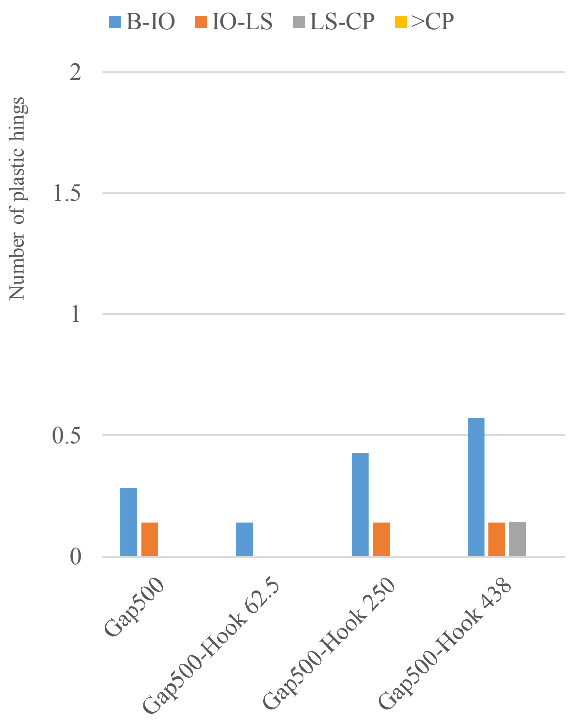
a) PGA equal to 0.35g b) PGA equal to 0.7g

Fig. 7 Positive and negative vertical displacement in models with 500 ton/cm compressive stiffness (m)

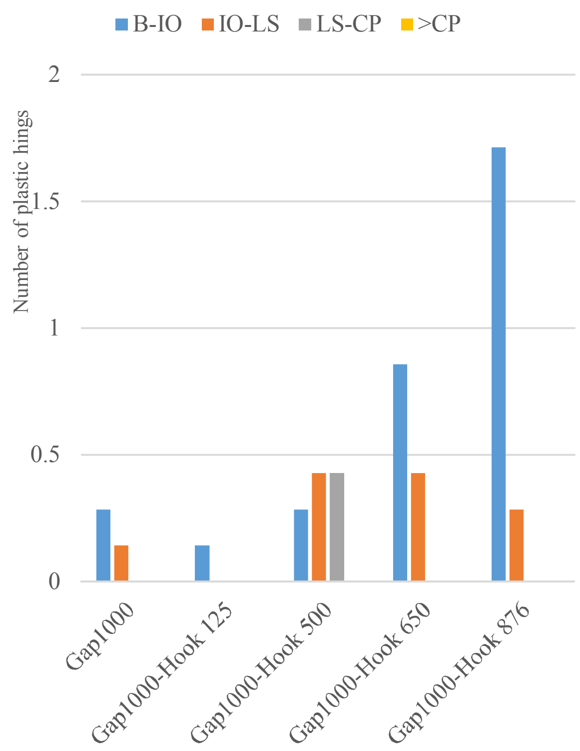


a) PGA equal to 0.35g b) PGA equal to 0.7g

Fig. 8 Positive and negative vertical displacement in models with 1000 ton/cm compressive stiffness (m)



a) Models with 500ton/cm compressive stiffness



b) Models with 1000ton/cm compressive stiffness

Fig. 9 Comparison of models performance level in 0.7g intensity level

5. Conclusions

Based on the investigated models in this study the following outcomes can be expressed

1- Increase in the tensile stiffness leads to the decrease in the vertical acceleration. Also, increase in the tensile stiffness changes building seismic performance and transfers the peak acceleration from the first story to the second or third story and decreases the differences between stories vertical acceleration. The vertical negative acceleration doesn't significantly change when tensile stiffness increases.
2- As the tensile or compressive stiffness and intensity level increases, drift doesn't change significantly.
3- Although Increase in the tensile stiffness decreases the positive vertical displacement of rocking columns, it increases the negative vertical displacement. The sum of positive and negative vertical displacements, especially in higher intensity levels was almost identical.
4- The models with low tensile stiffness have the best performance level. Increasing compressive stiffness leads to increase in the number of plastic hinges.

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