DYNAMIC RESOPNSE OF WEAKENED STRUCTURES USING ROCKING COLUMNS

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ABSTRACT :

Acceleration response of simple yielding structures is proportional to its structural strength. Therefore, in order to reduce accelerations, to protect structural elements and acceleration-sensitive nonstructural components in inelastic structures, the overall strength of the structure should be decreased. The use of "rocking columns", a type of double hinged gravity column, is suggested to reduce the strength of part of the lateral load resisting system (weaken the structure) for controlling the story accelerations. A simplified model of the rocking column, verified through experimental tests, was implemented into a nonlinear dynamic analysis platform IDARC2D. A numerical study is conducted to demonstrate that story acceleration responses can be reduced compared to conventional structures. An analytical model of a 1/3 scaled model structure which was tested at University at Buffalo in 1992 is used to evaluate the behavior of several weakening alternatives. The analytical results show that rocking columns are an effective means of weakening the structure to reduce story accelerations. However, such columns have both non-negligible stiffness and strength prior to rocking, which in turn affects both the local and global behavior of the structure.

KEYWORDS: Rocking columns, Quasi-static cyclic test, Simplified model, Nonlinear elastic-cyclic model, Controlled-weakened structures, Nonlinear dynamic analysis

1. INTRODUCTION

Depending on the critical components of a structure, the acceleration, velocity, and/or displacement must be controlled to maintain their functionality during earthquakes. Current retrofit methods, such as those employing supplemental bracing, lead to an increase in the global strength of the structure. In such cases, although the displacements and the ductility demands decrease, there is an increase in the floor accelerations. Reducing the strength of a structure primarily reduces its structural accelerations and associated forces when subjected to seismic excitation. Recent work (Viti et al, 2006) has conceptually shown that it would be more beneficial to reduce the strength of structures defined as "weakening" in order to reduce accelerations. In this study weakening of structures is achieved by using "rocking columns" as a practical technique to reduce structural strength. The rocking column is a type of double hinged column or cracked base and top column, which resists vertical loads with minimum or no lateral strength. In order to develop an analytical model for rocking column for use in dynamic analysis, this paper investigates the cyclic behavior of such rocking columns using an experimental study. A simplified relationship for moment versus curvature is developed from principles of mechanics and a nonlinear dynamic analysis is conducted to investigate the response of weakened structures using rocking columns.

2. USE OF ROCKING COMPONENTS IN STRCTURAL SYSTEMS

Scarce information is available for use of rocking components, and even less for use in weakening of structures. Un-reinforced masonry buildings constructed with rocking masonry walls were investigated experimentally by Costley and Abrams (1996) in shaking table tests. The rocking walls at the first-story created a stable soft-story mechanism which isolated the floors above from the dynamic base motion. Moreover, another effect of this mechanism observed in the experimental study was a relative reduction of second-story drifts after cracking and rocking of the wall. For seismic retrofit of bridge piers, Astaneh-Asl and Shen (1993) suggested a semi-rigid connection between the column base and the foundation which permits limited rocking of the piers. The target structure for the retrofit was the San Francisco-Oakland Bay Bridge. An analytical study was performed with DRAIN2D and SAP90 and showed that the seismic forces obtained were significantly smaller and the displacement demands were slightly larger compared to the strengthening and stiffening solution. Recently, Viti et al (2006) carried out studies of weakened and damped structures to demonstrate conceptually the reduction of accelerations and displacements, without presenting practical methods for weakening a structure. Similar effects are investigated and addressed in this paper.

3. QUASI-STATIC CYCLIC TEST

As indicated above, a rocking column is a type of double hinged or cracked base and top column, which resists vertical loads with minimum or no lateral strength. Quasi-static cyclic tests of rocking columns were conducted at University at Buffalo to investigate their cyclic behavior. Reinforced concrete specimens of 7x7x48 in of depth, width, and height, respectively were tested. The reinforcing bars Grade 40 had a yield stress of 40ksi. In the quasi-static tests, two column specimens named CY-5 and CY-10 are loaded with external axial loads of 5% and 10% of the nominal strength (256.66kips), respectively. The nominal strength of the rocking column specimen is defined as $N_0 = f_c^{'} A_g$, where A_g is the gross section area and $f_c^{'}$ is the compressive strength. When the external axial load was increased to 20% of the nominal strength, its edges suddenly crushed during first rocking phase (Roh, 2007). The test was conducted in displacement control in order to observe the "apparent negative stiffness" behavior. Photographs of the two specimens during testing are shown in Fig. 1.

Figure 1 Photographs of specimens during cyclic test

The experimental results of the three cycle test are shown in Fig. 2. The strength and displacement in negative and positive sides of the axes in each specimen differ because the external axial load was not constant. These axial load fluctuations summarized in top of Fig. 2 were considered as a one of parameters to compare of the test results with analytical model described in next section.

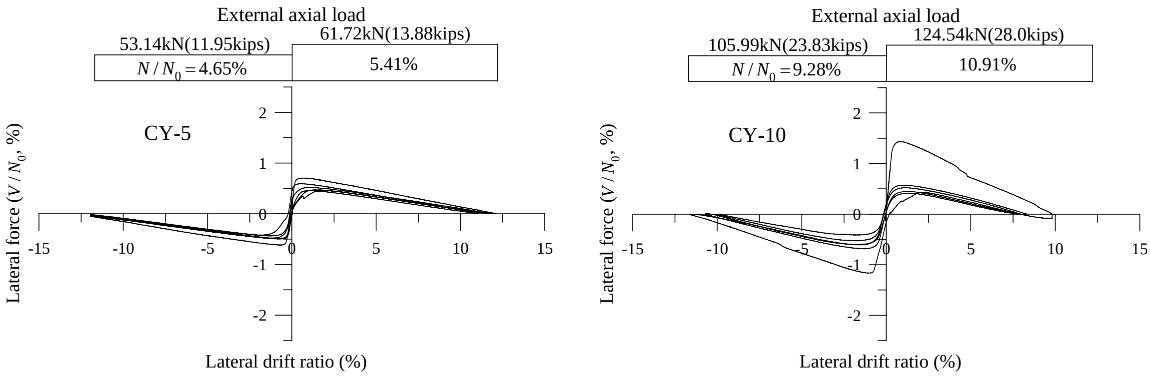

Figure 2 Results of quasi-static cyclic test of rocking columns

From the test results, it was found that the rocking columns have a non-negligible stiffness prior to rocking. After reaching the maximum lateral drift in the first cycle, the rocking column returns to a curve that remains constant in subsequent cycles, without degradation. This defined as the “steady low bond” in this study. This behavior is a result of gradual crushing and “rounding” the edges, as shown in Fig. 3. The “rounded” shapes release the stress concentration and prevent further breaking at the edges of column. Furthermore, when small external axial load is applied to the column, the curves obtained in the first and last cycles are very close.

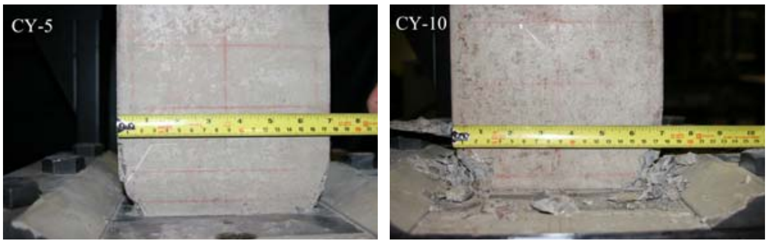

Figure 3 Crushing of rocking column edges after testing

Based on these investigations, the “steady low bound capacity” curve can be directly achieved in columns with the bottom section properly designed with rounded edges or those loaded with very low external axial load. In these cases the cyclic behavior of rocking columns can be simulated with *a nonlinear elastic-cyclic model*.

4. MOMENT - CURVATURE RELATIONSHIPS

From the test results, a rocking column is characterized by two capacity curves as shown in Fig. 4: (a) an upper bound curve when the column has rectangular edges without initial damage at the boundaries and (b) a steady low bound curve when there is a local damage, spalling, or crushing at the edges of the column.

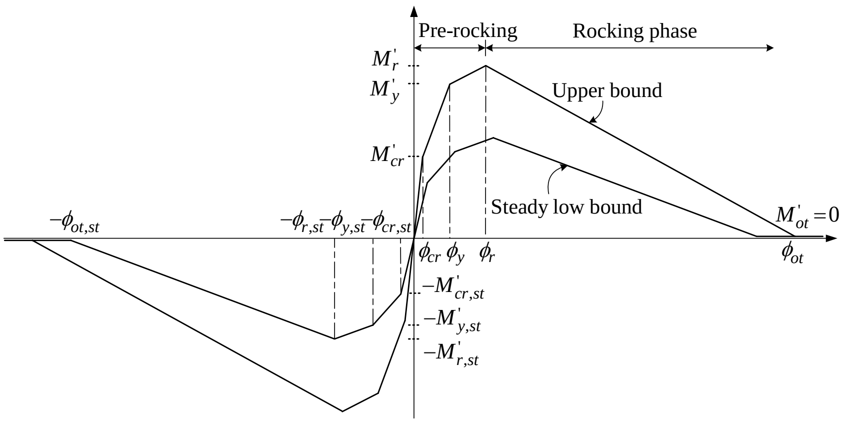

Figure 4 Simplified model of rocking columns for upper and steady low bounds

The upper bound and the steady low bound curves consist of several parts (i) cracking (subscript “cr”) state, (ii) yielding (subscript “y”) state, (iii) rocking point (subscript “r”), and (iv) overturning point (subscript “ot”). In this paper, only the *steady low bound* curve is addressed due to the paper length limitation. The base moments for the steady low bound curve are defined as the follows:

$$M'_{cr,st} = N\left(\frac{d'}{6}\right); \quad M'_{y,st} = N\left(\frac{d'}{2} - \frac{d_{c,y}}{3}\right); \quad M'_{r,st} = N\left(\frac{d' - d_{c,r}}{2}\right); \quad M'_{ot,st} = 0 \qquad (4.1)$$

Where $M'_{cr,st}$, $M'_{y,st}$, and $M'_{r,st}$ are the base moments of the steady low bound at each state. At overturning point, rocking column does not provide any lateral resistance. The parameters N and d are an external axial load and a column depth, respectively. Also, the parameters $d_{c,y}$ and $d_{c,r}$ are contact depth at yielding and rocking states, respectively, and defined as $d_{c,y} = \left(2N / f_c^{'} A_g\right) d$ and $d_{c,r} = d_{c,y}/2$. The parameter d' is a reduced column depth at the ends after edge damage and defined as $d' = d - 5d_{c,r}$. The detail derivations are presented in Roh (2007). Curvatures of the steady low bound curve are defined as the followings.

$$\phi_{cr,st} = \phi_{cr}\left(2 - \frac{d'}{d}\right); \quad \phi_{y,st} = \phi_{y}\left(2 - \frac{d'}{d}\right); \quad \phi_{r,st} = \phi_{r}\left(2 - \frac{d'}{d}\right) \tag{4.2}$$

Where $\phi_{cr,st}$, $\phi_{y,st}$, and $\phi_{r,st}$ are the base curvatures of the steady low bound at cracking state, yielding state, and rocking point, respectively. Also, The parameters, ϕ_{cr}, ϕ_{y}, and ϕ_{r}, are the curvatures of the upper bound at each state and evaluated by considering strain distribution at the contact surface. In Eqn. 4.2, it is assumed that the curvatures increase proportionally to the reduced end depth. If no damage is developed at the edges ($d' = d$), the curvatures of steady low bound equal to the curvatures of its upper bound. The curvature at overturning point is computed by using the relationship between moment versus curvature and lateral force versus displacement responses of a rocking column as follows.

$$\phi_{ot,st} = \phi_{y,st} + \frac{K_3\left(\delta_{ot,st} - \delta_{y,st}\right)}{EI_3}\frac{L'}{2} \tag{4.3}$$

Where $\delta_{ot,st}$ is the maximum lateral displacement at the overturning point of steady low bound, which is estimated from a force balance and a geometric configuration, represented with $\delta_{ot,st} = d - 4d_{c,r}$. The parameter EI_3 is a tangential slope between yielding state to rocking point of moment-curvature curve shown in Fig. 4 and the parameter L' is a column length. The parameter K_3 is a tangential stiffness in lateral force-displacement response at the rocking point. The parameter $\delta_{y,st}$ is a lateral displacement at a yielding state of steady low bound. Both parameters, K_3 and $\delta_{y,st}$, are defined during analysis for a rocking column.

5. MODEL FOR ROCKING PHASE SIMULATION

The particular characteristic of rocking columns in a cyclic behavior is that a “nonlinear stress zone” develops and recovers when the rocking column is unloaded to the original state. In this model, the development of the nonlinear stress zone starts as the base moment is higher than a cracking moment. The analytical model is flexibility matrix based on variable flexibilities. The “nonlinear stress zone” can be viewed in the flexibility formulation having same effects as the “yield penetration zone” in inelastic analysis of beam columns (Valles et al, 1996).

For the simulation of the rocking phase, “stepwise strength reduction scheme” (Roh, 2007) is used, which is an alternative technique to keep always a positive stiffness in analysis as shown in Fig. 5. This allows the use of the regular solution algorithms in every analysis step. Nonlinear dynamic analysis is carried out using a combination of the Newmark-Beta integration method and the pseudo-force method. The solution is obtained in incremental form, which assumes that the properties of the structure do not change during the time step of analysis to reduce the cost of performing iterations in the nonlinear analysis. Since the stiffness or strength of rocking column elements is likely to change during the time step of analysis as shown in Fig. 5, the new configuration may not satisfy equilibrium. A compensation procedure is adopted in order to minimize the error, ΔF_{cerr}, by applying a “one step unbalanced force correction” (Valles et al, 1996). To minimize the unbalanced forces, a sufficiently small time increment must be selected for analysis.

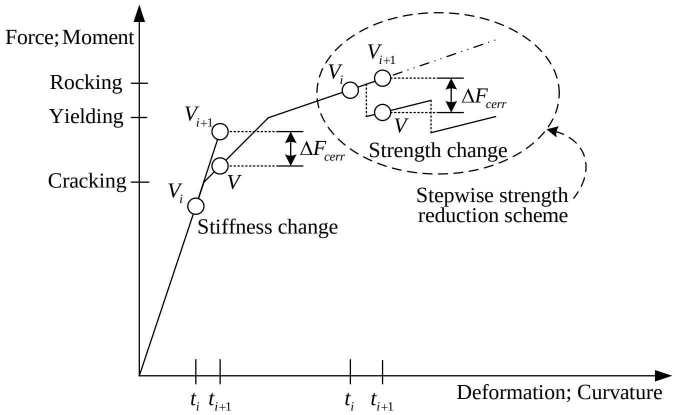

Figure 5 Stepwise strength reduction and one step unbalance force correction

6. NONLINEAR ELASTIC-CYCLIC MODEL

Nonlinear elastic-cyclic model (NECM) mentioned in the previous section assumes that the behavior of the element follows the same path for both loading and unloading without loss of energy, as shown in Fig. 6.

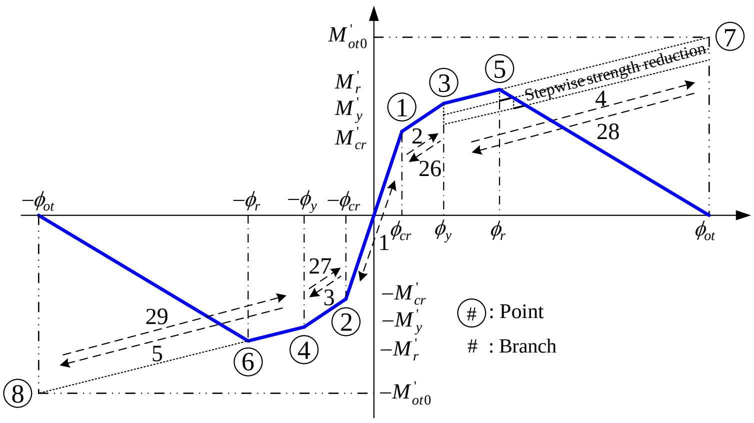

Figure 6 Points and branches of nonlinear elastic-cyclic model of rocking column

This model is governed by *branches* that occur during the response and *rules* that dictate the transitions between various stages. The nonlinear elastic-cyclic model described in detail by Roh (2007), is added to an existing polygonal hysteretic model in IDARC2D (Valles et al., 1996) in its Version 7.0 (Roh et al, 2008). Considering the variation of axial loads from the test, the analysis results using the nonlinear elastic-cyclic model for specimens CY-5 and CY-10 are compared with the test results in Fig. 7, including the upper bound.

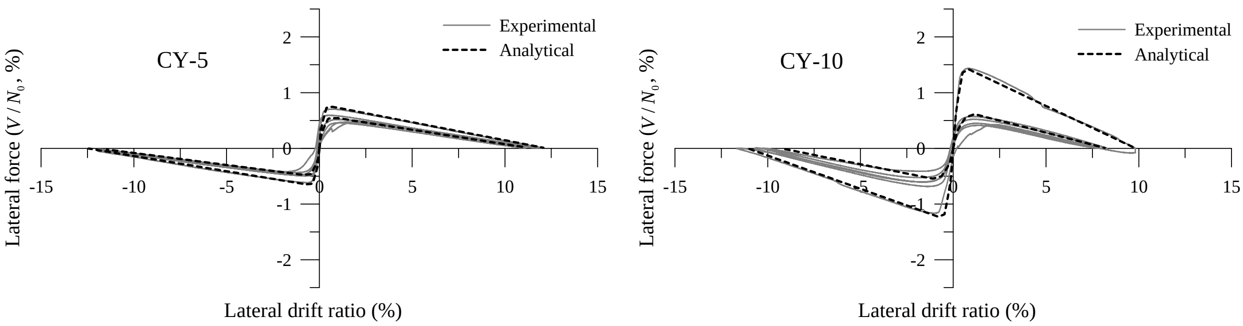

Figure 7 Comparison of analytical model with cyclic tests for upper and steady low bounds

When a high axial load is applied to the column, such as in CY-10 specimen, the damage at the bottom edges in the first half cycle is large. The stiffness, strength, and peak lateral drift of the simplified model are in good agreements.

7. WEAKENED STRUCTURES USING ROCKING COLUMNS

Alternative weakening methods using rocking columns are applied to a previously studied structure designed only for gravity loads (Bracci et al., 1995) and are shown in Fig. 8.

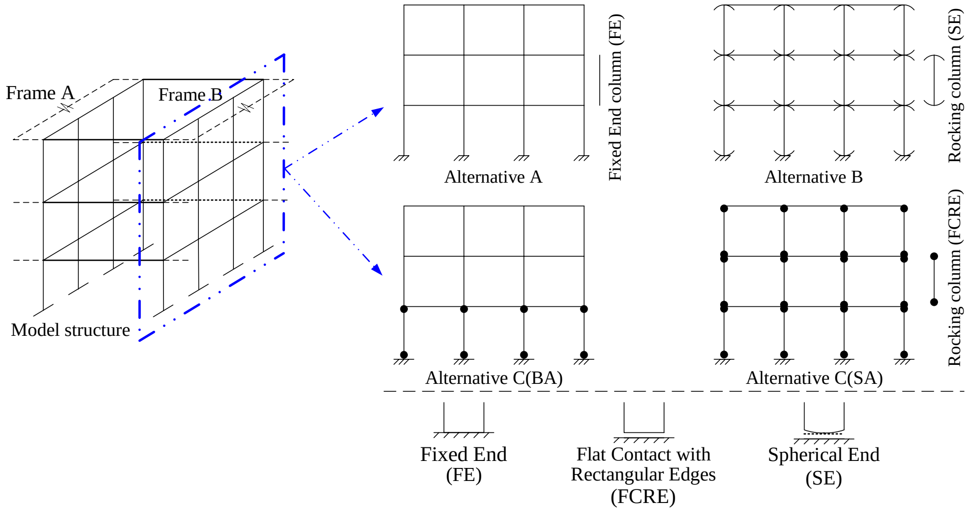

Figure 8 Model structure and alternatives for weakening

Strategies of weakening are applied to *frame B only*. Three types of structural columns are considered depending on the end shape of the rocking columns. (i) If all rocking columns have fixed ends (FE), the frame B provides full lateral resistance, which is defined as alternative A as an original structure. (ii) If the rocking columns of frame B have spherical or round ends (SE), the frame B does not contribute to any lateral resistance while still supporting gravity loads, which is defined as alternative B. It is an ideal case of weakening the frame B fully. (iii) If the columns are allowed to rock, having a flat contact with rectangular edges (FCRE) at the ends, then the lateral resistance of frame B is defined as alternative C. In Alternative C, two cases are considered depending on the rocking column locations. Alternative C(BA) allows all columns in frame B at first floor to rock and at all stories in alternative C(SA). Alternative C(BA) investigates the structural response with a weak-first-story and alternative C(SA) investigates the effect of proportional weakening. In alternative C, nodal weights are considered to model the moment-curvature capacity of rocking column. The cyclic behavior of rocking columns is simulated using a nonlinear elastic-cyclic model. For the computation, it is assumed that the edges of the base section are cut into the shape which would have developed after the first cycle of loading.

8. NONLINEAR DYNAMIC ANALYSIS OF WEAKENNED STRUCTURES

The case studies presented in the previous section are analyzed using nonlinear dynamic procedure. A white noise base excitation is used to investigate the inelastic behavior of the structure. The key feature of the white noise is that the power spectral density function is constant at all frequencies from elastic to inelastic responses. Although such excitation is extremely severe for a structure, this motion was selected in order to observe the effect of weakening, which changes drastically the natural frequencies of the structures. The Peak Ground Acceleration (PGA) is selected to be 0.3g for a total duration 30 seconds. The response histories of the original and weakened structures are shown in Fig. 9. Energy balance was checked. The hysteretic energy, the frame B in alternative C(SA) provides very small hysteretic energy dissipation due to the use of rocking columns.

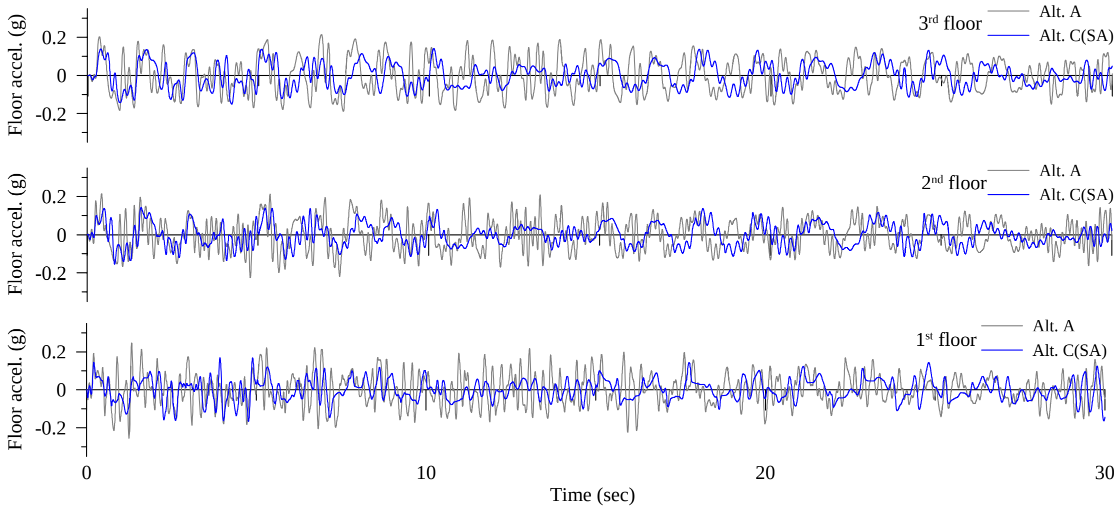

Figure 9 Response histories of floor acceleration in conventional and weakened structures

Analysis results obtained for the above case studies are presented in Fig. 10. The upper stories of alternative C(BA) remain elastic because yielding is confined to the first story. Alternatives C(SA) and B reach an inelastic behavior in first and second stories.

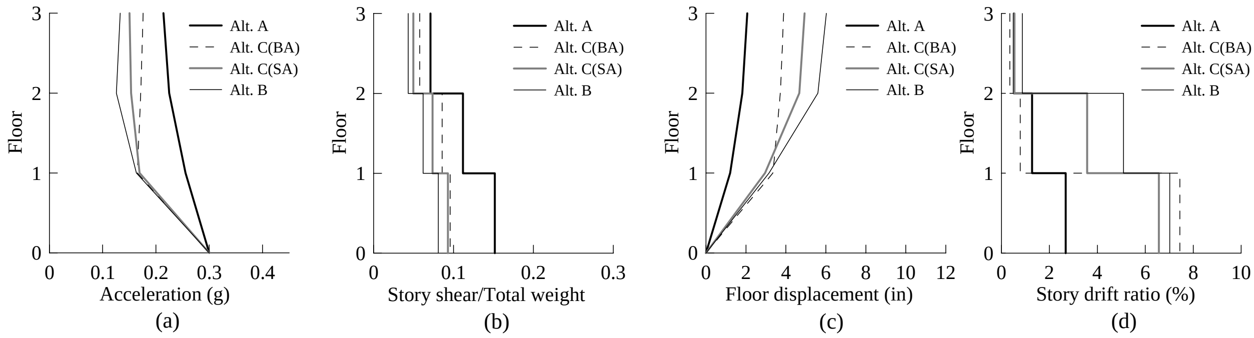

Figure 10 Peak responses: (a) story acceleration, (b) story shear, (c) floor displacement, and (d) story drift

The peak acceleration and peak shear force decrease in all stories throughout weakening the conventional structure. In alternative C(SA) the peak accelerations, compared to the original structure, reduce by 33.78%, 31.81%, and 29.85% at first, second, and third stories, respectively. Note that the yield strength reduction of alternative C(SA), compared to the original structure, is 38.87% and 35.94% at first and second stories where the story shear reached its maximum story strength level during the time history analysis. Therefore, the peak story acceleration is decreased, roughly proportional to the degree of weakening of the conventional structure. When first story only is weakened such as alternative C(BA), the apparent decrease in acceleration response appears in the first story, and the acceleration is very close to the extreme weakened case, alternative B. However, the upper story peak accelerations are close to the first story response because the upper stories are affected by the first story response in this system. The peak floor displacements for alternative C(SA) remain between the responses of alternatives A and B. Also, the displacement profiles are almost proportional to the responses of the original structure. In peak story drift response, when the weakening is concentrated at first story such as alternative C(BA), a story drift on first floor are substantially increased while the drifts at the other stories are decreased. This is also a typical well known behavior of "first-weak-story" structure.

9. REMARKS AND CONCLUSIONS

Motivated by the need to control passively story accelerations, the dynamic responses of weakened structures using rocking columns were investigated in this study. The cyclic behavior of rocking columns was investigated first by conducting an experimental study, which shows that the rocking column has a steady low bound whose curve provides no strength and stiffness degradations due to the ”rounding effect” at section edges. This low bound is able to be modeled with a nonlinear elastic-cyclic behavior. When a small axial load is applied to the column, less “crushing” is developed and the column is also governed by the nonlinear elastic-cyclic behavior without noticeable hysteresis. Simplified model of the rocking column was introduced and implemented into IDARC2D and compared with experimental results. The effects of weakening considering several alternative strategies were investigated using a nonlinear dynamic analysis. The numerical analysis results show that the use of rocking columns is an appropriate technique to achieve weakening. However, weakening alone is not sufficient if the displacement response exceeds a desirable limit. The addition of damping, however, corrects this problem.

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