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INVESTIGATION OF DYNAMIC RESPONSE OF ROCKING WALL STRUCTURES USING INELASTIC MODAL DECOMPOSITION TECHNIQUE

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Abstract. *Precast post-tensioned concrete rocking wall structures have been developed in the recent past as innovative damage avoidance structural systems. Recent experimental tests have identified the presence of large-amplitude high-frequency acceleration spikes in the dynamic response of rocking wall structures. The current study is focused on the investigation of the effects of these acceleration spikes on the dynamic performance of rocking wall structures. For this purpose, a 4 story rocking wall structure is designed and the response history analysis is performed. The results show a significant contribution of higher modes in the acceleration response which can increase the shear demand considerably. To further investigate these higher mode accelerations, inelastic modal decomposition of dynamic response is performed. Results show that the inelastic modal decomposition technique can be effectively used to understand and quantify the effects of these acceleration spikes and it can also be employed to check the effectiveness of different mitigation strategies for acceleration spikes.*

1 INTRODUCTION

Typically, seismic codes allow the structures to undergo large non-linear deformations against the rare seismic events while avoiding a total collapse of the structure. Although it ensures the life safety of occupants, excessive damage to the structures leads to a costly retrofitting and becomes a hindrance to the immediate serviceability of the structure after a major seismic event. In the recent past, extensive research work has been carried out to develop structural systems that are capable of resisting large seismic actions without inducing any significant damage to the structural members. Precast Seismic Structural Systems (PRESSS) technology [1] represents such a solution that relies on a nonlinear elastic behavior of gap opening and closing at the connection to accommodate the lateral demand without incurring any damage to the precast members. Such a connection is called as a rocking connection, developed essentially for beam column joints, but later on extended to the precast shear walls [1, 2]. A shear wall with a rocking connection is called as a rocking wall. A rocking wall consists of vertically stacked precast concrete panels clamped by unbonded Post-Tensioning (PT) from the top of the wall to the foundation. Initial post-tensioning in the PT tendons along with the self-weight resist the lateral force demand and once the lateral actions overcome the vertical forces, the wall starts to rock about its toes, commonly referred as Gap opening. The gap opening and closing phenomenon in a dynamic environment cause a high velocity contact simply called as impact between the wall base and the foundation. Recent experimental work on the dynamic performance of rocking walls have shown that the impact of wall base with the foundation can cause short-duration large-amplitude acceleration spikes in lateral direction called as Horizontal Acceleration Spikes (HAS). HAS phenomenon is not unique to the dynamic response of rocking wall structures only, but are also found in the systems whose dynamic response shows a sudden change in force values or direction such as self-centering and friction-damped base isolation systems [3, 4]. Wiebe and Christopoulos [5] analyzed the source of HAS in rocking wall structures and identified that a sharp change in the rate of change of elastic force during gap opening and closure at rocking wall-foundation interface creates an imbalance of forces in the system. To maintain the dynamic equilibrium, seismic masses accelerate and thus create an acceleration spike in lateral direction. In a low rise rocking wall where the gap would open in the first mode response, acceleration spikes can be described as an excitation of higher modes due to a change of force in first mode implying an interaction of different modes. HAS cannot only increase the inertial forces at different floor levels, but can also cause damage to the acceleration-sensitive non-structural elements like fire suppression systems, suspended ceilings, emergency power generators and computer systems [6]. Hence, there is need to identify, quantify and reduce these acceleration spikes. This work aims to quantify the effects of lateral acceleration spikes on the seismic demands of the rocking walls. For this purpose, a 4 story rocking wall is selected to investigate and quantify the effects of acceleration spikes on the dynamic performance of rocking wall structures.

2 CASE STUDY STRUCTURE

Previous experimental works have shown that the lateral acceleration spikes are more visible in low-rise rocking walls. Also, the presence of a dedicated Energy Dissipation (ED) mechanism for rocking walls is found to decrease the intensity of these effects. To investigate the phenomenon of HAS, a low-rise 4 story rocking wall with no external energy dissipation mechanism is selected. This will ensure that the acceleration spikes are more visible in the system response, making it easier to study in detail. Figure 1 and Table 1 shows the dimensions and different properties of the case study rocking wall. The case study rocking wall is part of a complete structural system and is assumed to take all the lateral load of the structure. The connection between floor and the rocking wall is assumed to be similar to the one being used in Diaphragm Seismic Design Methodology (DSDM) project [7]. This vertically sleeved connection allows the lateral forces to be transmitted from the floor to the wall while prohibiting any vertical action to be transmitted from the rocking wall to flooring units. A constant story height of 3m is used for all floor levels. The lumped mass in lateral direction represents the floor load while the vertical seismic mass is the mass of the rocking wall only.

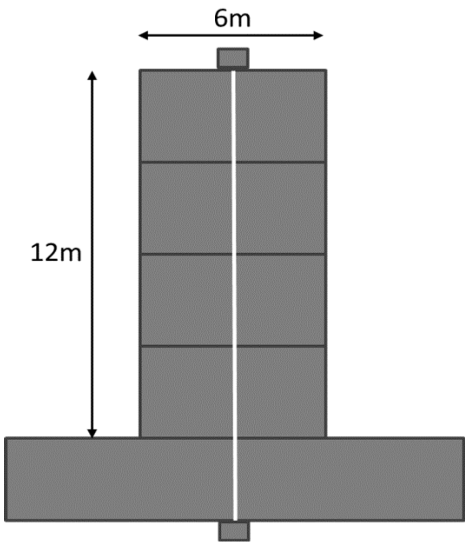

Figure 1: Case study rocking wall.

Table 1: Properties of case study rocking wall.

Thickness of wall (mm)		300
Comp. strength of wall concrete (MPa)		45
Ultimate strength of PT steel (MPa)		1860
Initial post-tensioning (MPa)		651
Area of Post-tensioning (mm^2)	No ED	15300
	With ED	7595
Total area of ED bars (mm^2)		11187
Lateral seismic mass for each floor (Ton)		350
Modal periods (sec)	1st Mode	0.395
	2nd Mode	0.073
	3rd Mode	0.035

The case study building is assumed to be located at a California site (Postal code 91495) with a soil type S_D. A suite of 5 pairs of 10 ground motions is selected such that the scaled spectra of the selected ground motions resemble the target spectrum. Furthermore, RSPmatch2005b software [8] is used for the time domain spectral matching of the ground motions spectrums with the target spectrum. Figure 2 shows the design spectra for the site along with the matched spectrums of different ground motions. Design forces for the case study rocking wall are calculated by using force based design procedure in accordance with the earlier works on the design of rocking wall structures [9].

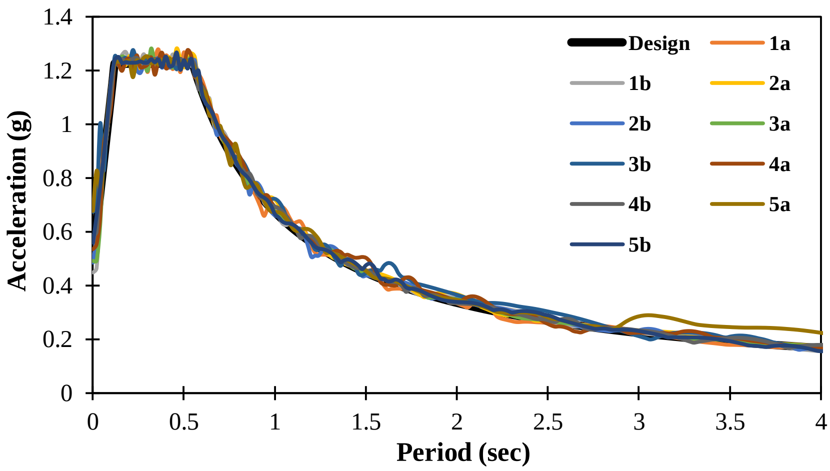

Figure 2: Matched ground motions spectra for 5% damping.

3 DYNAMIC MODELING OF ROCKING WALL

Different modeling techniques like lumped plasticity model, multi-spring fiber model and finite element models have been used in the past for the modeling of rocking wall structures. Lumped plasticity model is the simplest one but the input behavior of the bilinear elastic / flag shaped hysteresis for rocking/hybrid walls with sharp edged corners at gap opening and closure are found to increase the HAS unjustifiably [10]. 2D Multi-spring fiber model has been found efficient in predicting the neutral axis migration with significant accuracy and also gives relatively round corners at the gap opening and closure, provided that enough base springs are used. 3D finite element models are quite efficient in predicting the overall dynamic response as well as the local behavior due to impact at the wall-foundation connection. However, finite element models are computationally expensive, especially for the dynamic analyses. For the current study, multi-spring fiber model is selected as it is robust and computationally economical.

3.1 Multi-spring Fiber Model

Ruaumoko 2D [11] is used for the multi-spring fiber modeling of rocking wall. The model is shown in Figure 3. Modeling of different materials and members have been done based on the recommendations proposed by Smith and Kurama [12]. Elastic beam-column element is used to represent the rocking wall. PT steel is modeled by using a truss member with a pre stressing force and an elasto-plastic hysteresis behavior. Gap opening at the wall base is modeled by using a number of compression only contact springs connecting the wall base with the foundation through a rigid member to simulate plane section remain plane behavior. Contact spring stiffness basically comes from the axial stiffness of rocking wall and foundation and can be idealized as two axial springs acting in series. Assuming that the foundation is near rigid, rocking wall axial stiffness can be used as contact stiffness by using the relationship for axial stiffness as E_cA_c/H_c, where E_c, A_c and H_c are the modulus of elasticity of wall concrete, area of concrete that a particular spring is representing and the height of rocking wall contributing to the axial stiffness of rocking wall. Previous studies [13, 14] have found that a value of H_c equal to $0.5L_w$ gives reasonably accurate results and this value is used in the current study. 25 contact springs are used in this study to accurately predict the migration of neutral axis. Wilson-Penzien damping model which allows the users to provide constant modal damping for different modes have been used in this study. A constant 3% damping for all the modes have been used which is consistent with the findings of the previous experimental works [15, 16]. Nonlinear Response History Analysis (NLRHA) is conducted for the case study rocking wall for the selected ground motions. The selected ground motions are scaled up 1.5 times to represent a Maximum Considered Earthquake (MCE) level hazard.

4 NLRHA RESULTS

Figure 4 shows the envelope results for the time history analysis along with the design values. Lateral drift values for the rocking wall is found to be higher than the design value because of the lack of energy dissipation mechanism for rocking wall. The envelope results of lateral acceleration show the average value to be equal to 2g at the top floor. Furthermore, a bulge in the acceleration envelope at around first story shows the presence of higher mode effects in the acceleration response. Similar trend can be seen in the shear envelope where the average base shear value is 3.25 times the design value which is quite high for a 4 story structure.

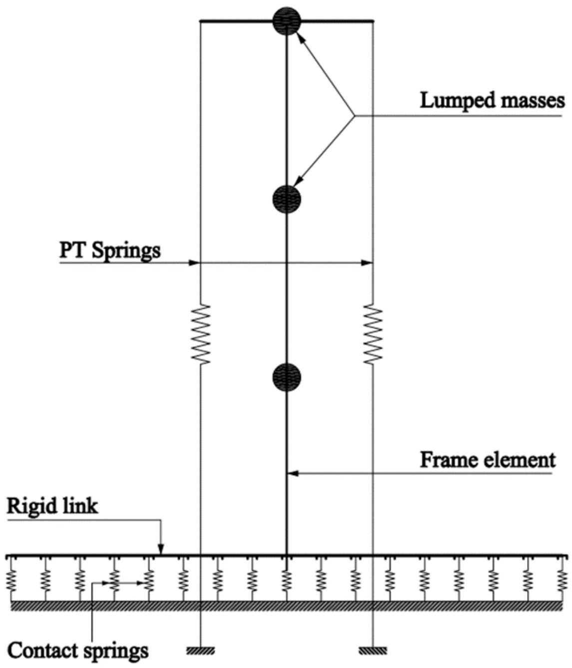

Figure 3: 2D Multi-spring fiber model.

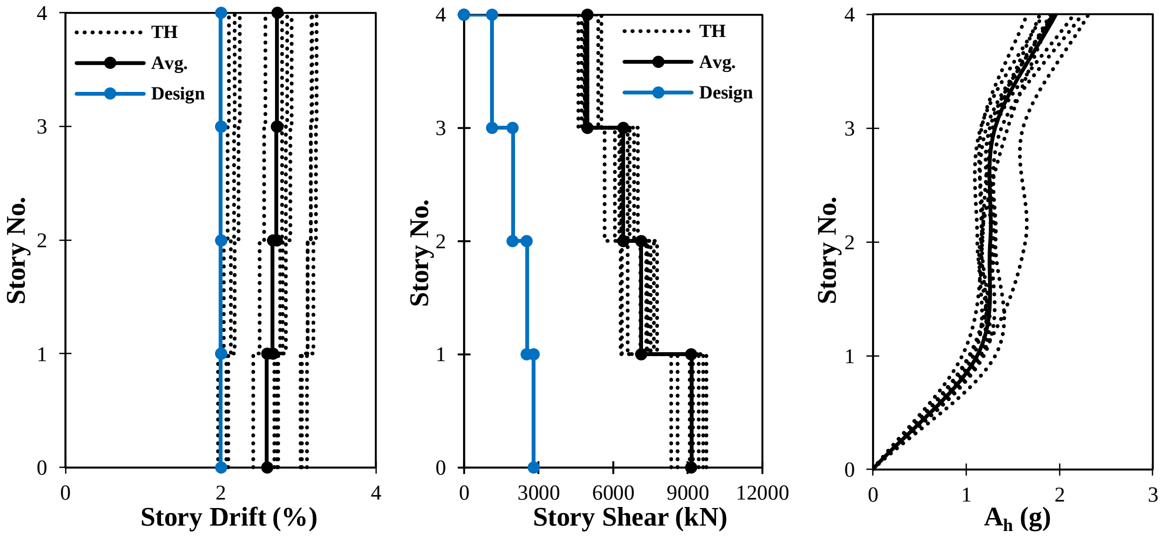

Figure 4: Comparison of envelope results.

To further study these higher mode effects, Figure 5 shows the time history of lateral accelerations at different floor levels for a particular ground motion. Lateral acceleration spikes have been recognized in the past experimental works as a peak in the acceleration values right after the gap opening and closure. Similar trend is obvious in the lateral acceleration histories where the acceleration values peaks during the start of a vibration cycle. Figure 6 shows the histories of story shear forces. The top story shear is found to be affected most by these acceleration spikes.

As explained earlier, the HAS phenomenon is a manifestation of the excitation of higher modes due to a sudden change of force in the first mode response implying a strong coupling of the modes. To further understand this phenomenon, different modal responses are separated to see their contribution to the total response. Modal decomposition techniques have been developed in the past to separate the total responses into modal responses [17-19]. Uncoupled

Modal Response History Analysis (UMRHA) procedure is one such technique that is based on the decomposition of total responses into uncoupled modal responses.

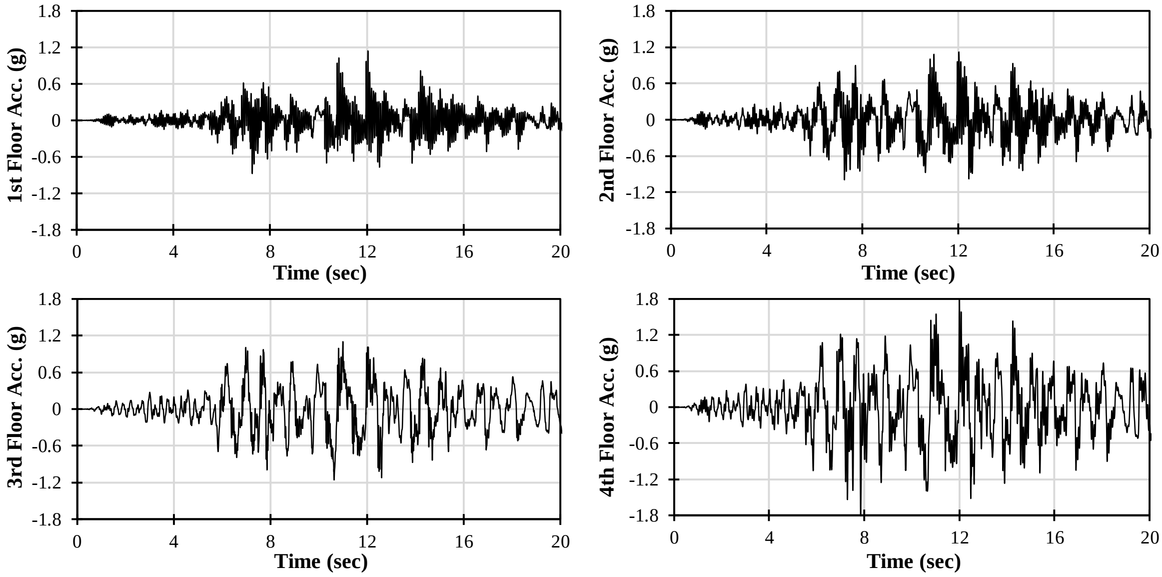

Figure 5: Time history of floor accelerations.

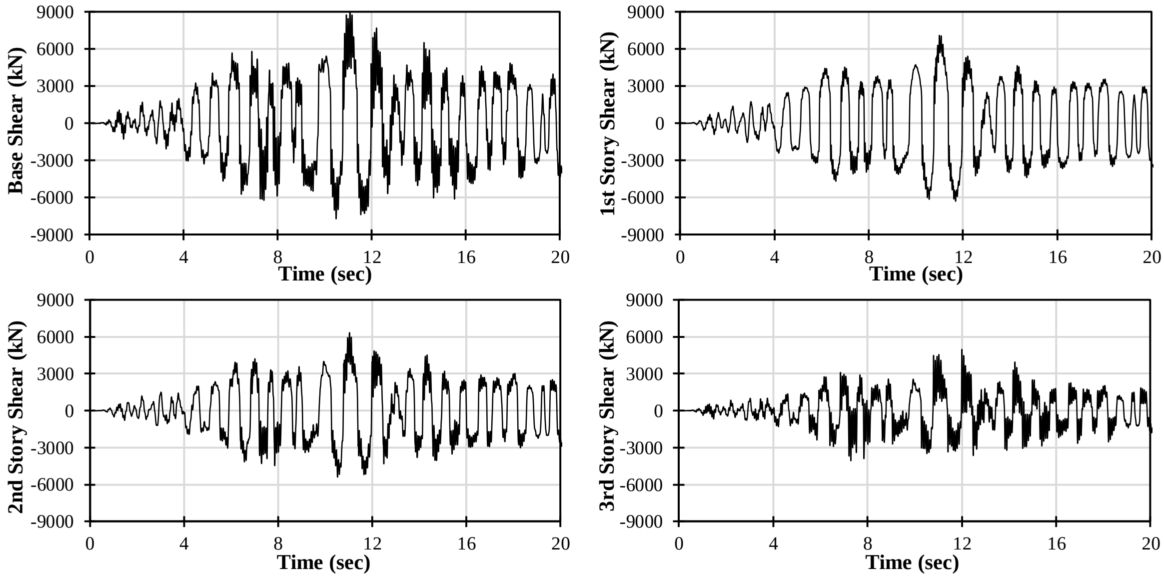

Figure 6: Time history of story shears.

5 UMRHA PROCEDURE

The UMRHA procedure [17] can be viewed as an extended version of the classical modal analysis procedure. In the latter, the complex dynamic responses of a linear Multi-Degree-of-Freedom (MDOF) structure are considered as a sum of many independent vibration modes. The response behavior of each mode is essentially similar to that of a Single-Degree-of-Freedom (SDOF) system which is governed by a few modal properties, making it easier to understand. Furthermore, only a few vibration modes can accurately describe the complex structural responses in most practical cases. This classical modal analysis procedure is applicable to any linear elastic structure. However, when the responses exceed the elastic limits, the governing equations of motion become nonlinear and, consequentially, the theoretical basis for modal analysis becomes invalid. Despite this, the UMRHA procedure assumes that, even for inelastic responses, the vibration modes still exist, and the complex inelastic responses can be approximately expressed as a sum of these modal responses. By following this presumption, governing equation of motion of SDOF to a horizontal ground motion $\ddot{x}_g(t)$ can be written as:

$$\ddot{D}_i + 2\xi_i\omega_i\dot{D}_i + F_{si}(D_i, \dot{D}_i)/L_i = -\ddot{x}_g(t) \tag{1}$$

Where $L_i = M_i\Gamma_i$ and $M_i = \boldsymbol{\phi}_i^T\mathbf{M}\boldsymbol{\phi}_i$, ω_i and ξ_i are the natural vibration frequency and the damping ratio of the ith mode, respectively. Equation (1) is a standard governing equation of motion for inelastic SDOF systems. To compute the response time history of $D_i(t)$ from this equation, one needs to know the nonlinear function $F_{si}(D_i, \dot{D}_i)$. A cyclic pushover analysis is performed for each important mode to identify the nonlinear function $F_{si}(D_i, \dot{D}_i)$. The cyclic pushover analysis for the ith mode can be carried out by applying a force vector with the ith modal inertia force pattern $\boldsymbol{s}_i^* = \mathbf{M}\boldsymbol{\phi}_i$ (where M is the mass matrix of the building and $\boldsymbol{\phi}_i$ is the ith natural vibration mode of the building in its linear range. The relationship between roof displacement, obtained from the cyclic pushover (denoted by x_i^r), and Di is approximately given by

$$D_i = x_i^r/(\Gamma_i\phi_i^r) \tag{2}$$

where ϕ_i^r is the value of $\boldsymbol{\phi}_i$ at the roof level. The relationship between the base shear V_{bi} and F_{si} under this modal inertia force distribution pattern is given by

$$F_{\mathrm{s}i}/L_i = V_{bi}/\Gamma_i L_i \tag{3}$$

By this way, the results from the cyclic pushover analysis are first computed in the form of cyclic base shear (V_{bi})—roof displacement (x_i^r) relationship, and then transformed into the required F_{si}— D_i relationship. At this stage, a suitable nonlinear hysteretic model can be selected, and its parameters can be tuned to match with this F_{si}— D_i relationship. The response time history of $D_i(t)$ as well as $F_{si}(t)$ can then be calculated from the nonlinear governing Equation (1). The response of each mode belongs to the ith vibration mode and can be generally represented by $r_i(t)$. By summing the responses of all significant modes, the total response $r(t)$ is obtained:

$$r(t) = \textstyle\sum_{i=1}^{m} r_i(t) \tag{4}$$

The force-related story response can be obtained by using their relationship with F_{si}, obtained from the modal pushover analysis in the linear response range. Similarly the deformation-related story responses can be obtained by using their relationship with D_i, obtained from the modal pushover analysis in the linear response range.

6 COMPARISON OF NLRHA AND UMRHA RESULTS

Different assumptions of UMRHA procedure include the use of an approximate hysteresis behavior for SDOF system, the use of elastic inertial force pattern to obtain the nonlinear modal responses, the linear summation of modal responses and the absence of interaction between different vibration modes. In the current study, it is made sure that the behavior of the selected hysteretic model, used for each SDOF system, matches perfectly with the hysteretic behavior of the structure under each modal inertial force pattern. Figure 7 shows the comparison of hysteresis behavior for the first mode. The cyclic modal pushover analysis is carried out for each vibration mode by applying the force vector with the ith mode inertial force pattern $\boldsymbol{s}_i^*$. The magnitude of the inertial force is reversed and increased gradually to push the structure into the nonlinear range (i.e., gap opening). The results in the nonlinear range show that the pattern of story forces of the rocking wall is almost similar to the story forces pattern in the linear elastic range. Therefore, the assumption of using elastic inertial force distribution pattern for the estimation of nonlinear story force distribution is justifiable. Another assumption used in the formulation of the UMRHA procedure is the linear summation of modal responses, which is strictly speaking only valid for linear elastic responses. Since the shear response is modeled as linear elastic in this study, the linear summation of modal shear responses can be used to get the total shear response. The only assumption left in the UMRHA procedure is that there is no interaction among different modal responses. However, this limitation of the UMRHA procedure can be beneficial for the current study where the difference between the responses obtained from the NLRHA and the UMRHA procedure would represent the effect of acceleration spikes, a manifestation of modal interaction.

Figure 8 shows the comparison of envelope results of story forces, story shears and floor accelerations. Results show a significant difference in the results of NLRHA and UMRHA showing a presence of coupling of modes. Figure 9 and 10 show the histories of lateral floor accelerations and story shears. Comparison of histories shows that the UMRHA procedure is not able to predict the high frequency acceleration spikes observed in NLRHA during the start

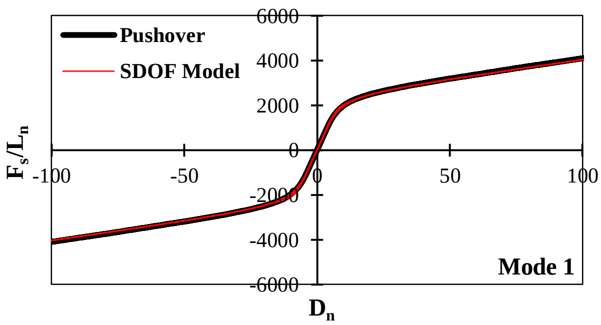

Figure 7: Comparison of hysteresis behavior for first mode

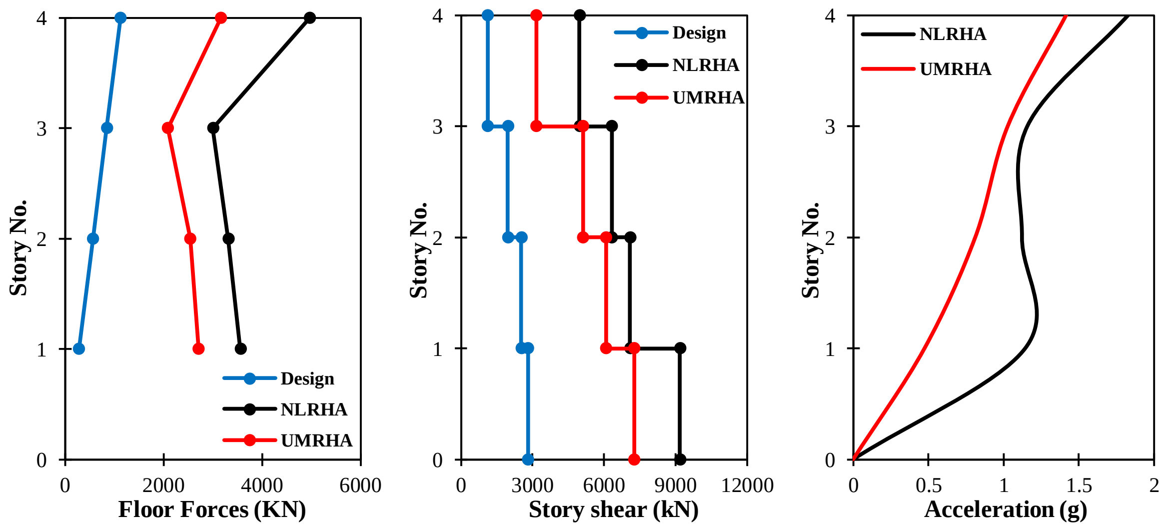

Figure 8: Comparison of envelope results.

of a vibration cycle. It is quite clear from the results that without the modal coupling the higher mode effects are quite low which is normally expected for a low-rise structure. Also, the bulge in the acceleration envelope as observed in the NLRHA results is not present in the UMRHA acceleration envelope. This implies that if the acceleration spikes are removed from the acceleration response then the UMRHA procedure, with the removal of different assumptions, should be able to predict the acceleration response accurately. To check this, the rocking wall structure is redesigned with a dedicated energy dissipation mechanism and a reduced contact stiffness to keep the acceleration spikes to a minimum value. Energy dissipation bars at the wall-foundation joint are designed to take 45% of the moment demand with the other 55% being resisted by a combination of post-tensioning force and the self-weight of the rocking wall. Also,

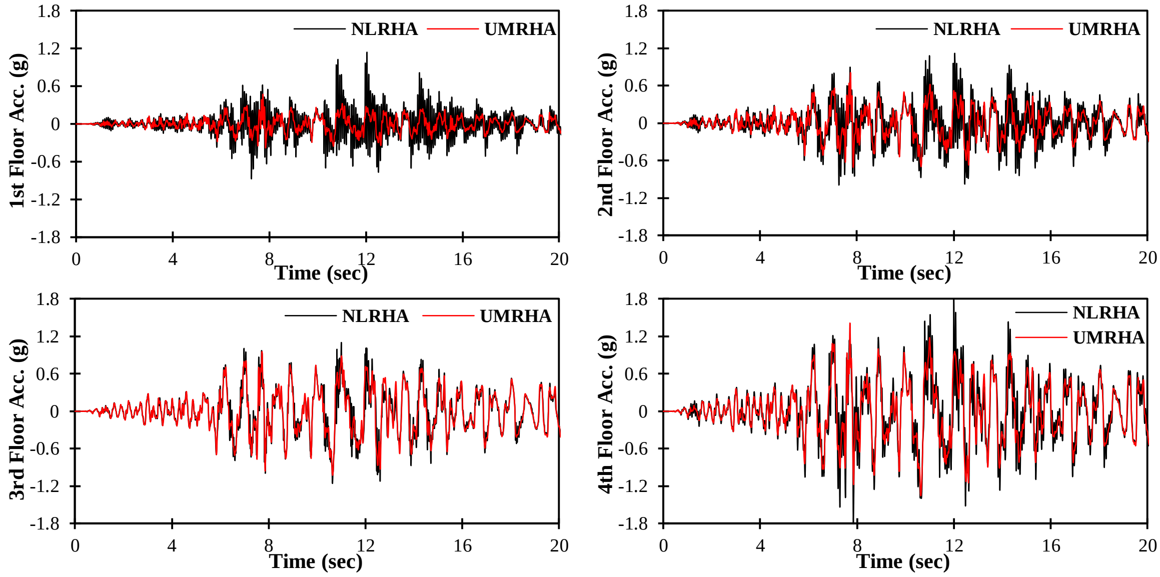

Figure 9: Comparison of floor acceleration histories.

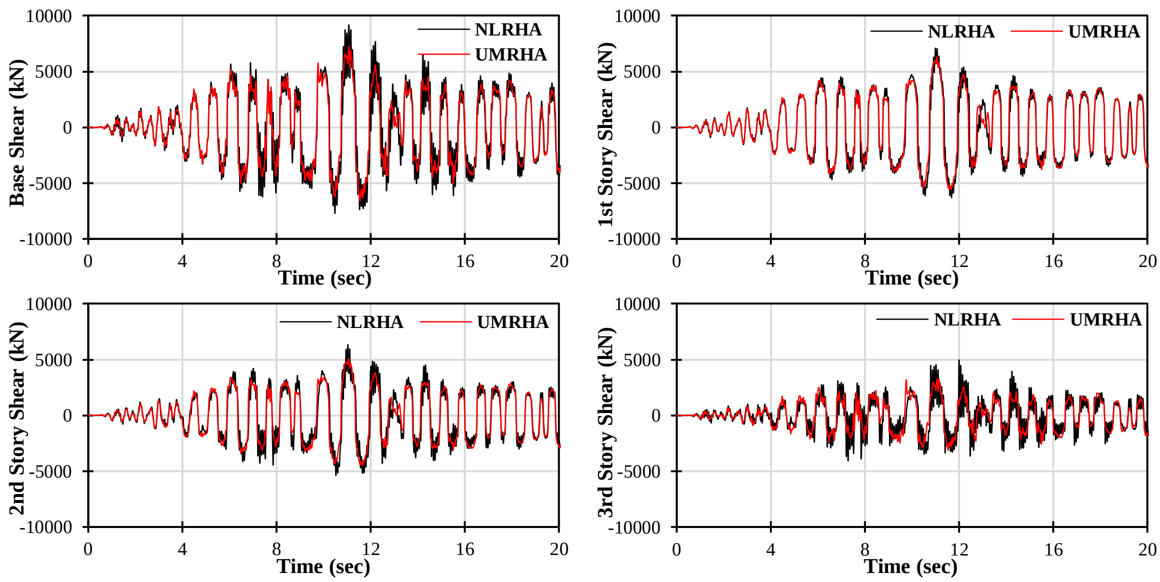

Figure 10: Comparison of story shear histories.

the contact stiffness is reduced by a factor of 4 to reduce the impact related acceleration spikes. The NLRHA and UMRHA procedures are employed for the same ground motion and the results are compared. Figure 11 shows the comparison of envelope results which are matching quite well. Histories of floor accelerations (Figure 12) further show that the UMRHA procedure is able to predict the response with much accuracy in the absence of acceleration spikes. The comparison of results obtained from the UMRHA procedure and the NLRHA procedure for two different cases, without dedicated ED mechanism and with ED mechanism, shows that the UMRHA procedure works reasonably accurate when the acceleration spikes phenomenon is not present while for the cases where dynamic response is significantly affected by the acceleration spikes the UMRHA procedure failed to predict dynamic response accurately. Furthermore, the energy dissipation mechanism and the reduction in contact stiffness is found to reduce the acceleration response significantly, especially for the first floor level. Also, the shear demand for the rocking wall is also reduced significantly highlighting the effectiveness of the proposed strategies to reduce the acceleration spikes. This implies that the UMRHA procedure can be used effectively as a tool to differentiate the dynamic response that arises from the normal higher mode effects and the one that stems from the complex impact problem in the form of acceleration spikes. Furthermore, the effectiveness of different mitigation techniques for acceleration spikes can also be validated by using this method.

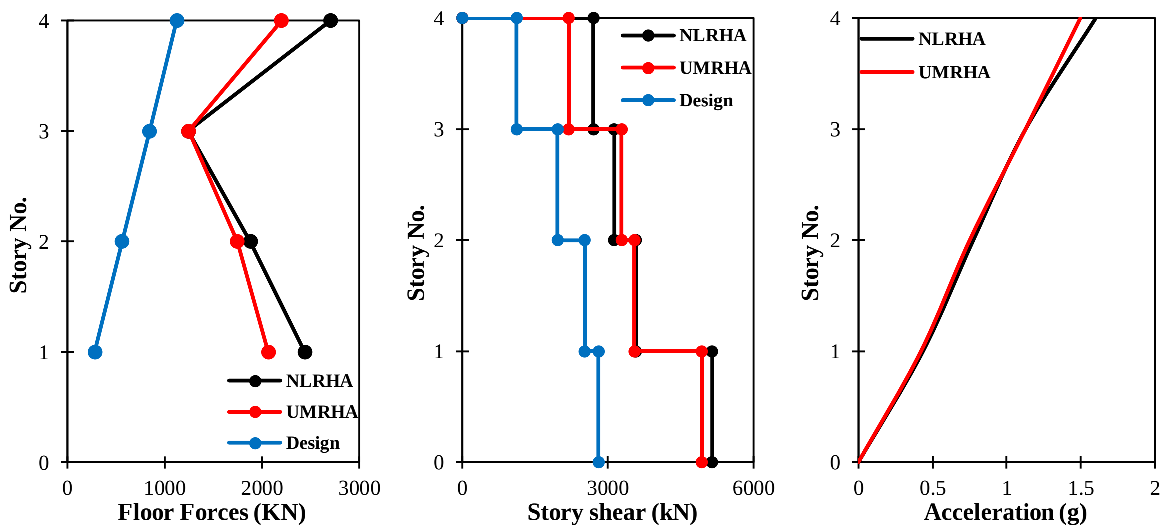

Figure 11: Comparison of envelope results with modified design.

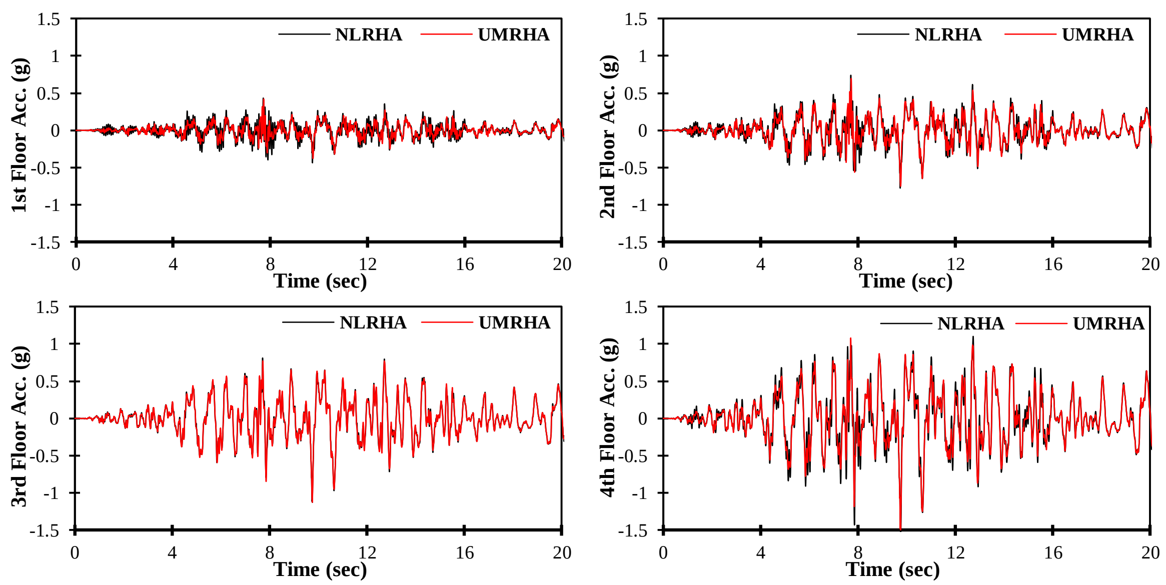

Figure 12: Comparison of floor acceleration histories with modified design.

7 CONCLUSIONS

The current study focuses on the phenomenon of lateral acceleration spikes in the dynamic response of rocking wall structures. A rocking wall structure is designed and a suite of ground motions is applied. Furthermore, inelastic modal decomposition of dynamic response is performed to investigate the higher mode effects. Based on the results, following conclusions are drawn.

- NLRHA results for 4 story rocking wall structure show a significant presence of high frequency accelerations in the dynamic response. Floor acceleration histories show that these high frequency accelerations are similar to the impact related acceleration spikes observed in the past experimental works and are also found to be responsible for a high shear amplification of 3.25 for the case study rocking wall.
- An innovative modal decomposition technique is used to investigate the higher mode effects. Different assumptions of the UMRHA procedure are removed except that the UMRHA procedure considers the modal responses as uncoupled. This modified UMRHA procedure allows to separate the impact related acceleration spikes from the total acceleration response and to quantify their effects on the shear response.
- The presence of energy dissipation mechanism and the reduction of contact stiffness at wall-foundation interface are found to be effective strategies for the reduction of these acceleration spikes. Furthermore, both of these strategies are found to reduce the acceleration and shear demand considerably ensuring a safety factor against any shear slip failure.

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