SEISMIC EVALUATION OF STEEL MOMENT FRAME-ROCKING WALLS WITH A VIEW TO COLLAPSE PREVENTION, SELF-ALIGNMENT AND REPARABILITY

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ABSTRACT

Rocking system are one of the recent developments devised to improve the seismic behavior of structures. This paper introduces a special type of Rocking Wall-Moment Frame (RWMF) combination that consists of a Grade Beam Restrained Moment Frame, (GBRMF) attached to a co-planar, post tensioned (PT) pin supported Rigid Rocking Core (RRC) by means of Gap Opening link beams (GOLBs) and supplementary devices. It compares a 4-storey and an 8-storey steel moment frame with Fixed base Shear Walls (FSW) with the proposed alternative. Simulation and numerical analysis are carried out using SAP2000 software. Incremental dynamic analysis (IDA) of the subject systems is conducted using 7 earthquake records on soil types C defined by USGS. Seismic responses of the RWMFs are presented using fragility curves. The results illustrate that RWMFs can be treated as repairable structures because plastic hinges are well distributed and prevent sever damage to columns and footings. Furthermore, RWMFs prevent soft storey failure by imposing uniform drift along the height of the structure. RWMFs lend themselves well to self- alignment and post-earthquake repairs by using post tensioned cables and known supplementary devices.

Keywords: Fragility curves, Rocking wall-moment frames, Reparability, Self- alignment

1. INTRODUCTION

While fixed base moment frame–rigid core combinations are the most popular earthquake resisting systems worldwide, they are not free from technical flaws and economic drawbacks. Conventional fixed-base moment frame–shear wall or braced frame combinations rely on uncontrolled inelastic responses of their members to absorb seismic energy. These systems have served their functions rather well in the past. However, they are practically un-repairable and are prone to catastrophic collapse due to major seismic events. Here, a relatively new, dual earthquake resisting system consisting of ductile moment frames (MFs) with rotationally controllable column supports in combination with post-tensioned RRCs is introduced, see Fig.1. The proposed configuration is capable of damage control, collapse prevention and self centering due to strong ground motion. The idea that rocking motions may reduce damage to MFs during earthquakes in not new, and was originally recognized by MacRae et al.[1, 2] Ajrab et al. [3], Panian et al. [4], Ji, et al. [5] and Wada et al.[ 6]. The pioneering effort in developing design concepts for controlled rocking of self-centering cores are due to Christopoulos et al. [7,8], Deierlein et al.[9], Eatherton et al.[10,11] . Takeuchi et al. [12], Janhunen et al. [13] and Grigorian et al. [14]. The focus of the present paper is on the global response of the RWMF rather than member design. GOLBs are used to connect the MF to the RRC; [15, 16]. Further research has shown that while these connections display excellent realignment abilities in laboratory testing, they tend to damage the adjoining columns and diaphragms in assembled systems; [17, 18]. [10 and 19] have both

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proposed innovative detailing methods to prevent damage due to rocking. The interested reader in rocking frame innovations is referred to two well documented bibliographies [20 and 21]. The full introduction of RWMF system and its formulas have been described by Grigorian and Moghadasi [22]. ASCE 41[23] defines specific performance levels for immediate occupancy, life safety, and collapse prevention, where collapse prevention, is defined as “the post-earthquake damage state in which the building is on the verge of partial or total collapse”. A well-designed RWMF can be expected to provide reliable support for the gravity system, facilitate the rescue/evacuation effort and improve the reparability of the frame after a major seismic event [22]. The present article focuses on collapse prevention employing RWMF technologies for new structures. This leads to the notion that if collapse prevention is feasible through reliable technologies, then life safety and immediate occupancy can also be achieved through similar strategies. To ensure that near perfect self-centering is achieved, design strategies need to focus on a basic issue: selection of a structural system that can dissipate the stipulated minimum seismic energy and retain little to no residual drifts.

This research studies and evaluates the proposed RWMF properties and compares its performance with conventional systems. For this purpose two types of structural system are used. The first type is a regular moment frame with Fixed base Shear Wall (FSW) and the second is a Rocking Wall Moment Frame (RWMF) consisting of a RWMF with and without prestressing cables. Both systems are modeled for two levels of height, 4 and 8-storey in SAP2000 v19. Both parametric and nonlinear analysis consisting of time history and Incremental Dynamic Analysis (IDA) were conducted. The accuracy of the analytical case has been verified by the simulated model.

2. VALIDATION OF A ROCKING WALL MODEL UNDER MONOTONIC LOADS

A full-scale pre-stressed rocking wall with unbounded tendons and unbounded steel dowels was tested [24]. Figure 2 shows the full-scale experimental study. The wall cross-section is 2800 × 300 mm², and the wall height is 10 m. The lateral load is applied to the wall at a height of 8.8 m (Figure 2(a)). Unbounded steel dowels were installed at the base cross-section in order to create adequate shear over-strength, against friction lose due to local dynamic effects or damage. As shown in Figure 2(b), only horizontal stirrups were used in the wall in order to prevent concrete shear failure and crushing. Eight unbounded tendons that include three 15-mm strands were replaced with reinforcing longitudinal bars. The vertical loads are provided by post-tensioned tendons, which are 2500 kN in total. The confined concrete compressive strength is 55 MPa. Figure 4(b) displays a schematic view of the methodology used for modeling of the experimental model in SAP2000. Unbounded steel dowels and unbounded tendons are modeled by cable element in the software. The wall is modeled to rock on its toes (Figure 4(a)). This mechanism is done by applying compression only gap on rocking toes. The foundation is assumed rigid, so in order to create this condition, the modulus of elasticity of gaps is given as 10e20 kN/m. Steel dowels are linked to the wall by the rigid beam. As mentioned before, steel dowels are not bonded to the wall and are just wrapped in sheaths. So, it is assumed that steel dowels are displaced in the x direction of the wall and not linked to it in y and Mz directions. The loading history applied to the rocking(stepping) wall and the hysteresis response of rocking wall is shown in Figure 3. The concept of modeling in the software, schematic figure of the modeling and structural elements is shown in figure 4. It should be mentioned that the mechanism of the model is mimicked by using an equivalent column instead of a shear wall, compression only gap on stepping toes, the rigid foundation
and rigid beam on top of the wall and post-tensioned unbounded cables. In this paper push over curve of the analytical model has been compared with the same experimental model in figure 5.

Figure 2. (a) Full-scale experimental rocking wall. (b) details of the wall [24].

Figure 3. (a) protocol of loading. (b) hysteresis response of rocking wall [24].

Figure 4. (a) concept of modeling (b) schematic view of the methodology used for modeling

Figure 5. comparison of push over curve of analytical model with the experimental model

As shown in figure 5. in linear analysis, the analytical model is fully compatible with the experimental
study. Although, in the nonlinear analysis the result is not entirely matched, it is overestimated in estimating the capacity of the structure.

3. ANALYTICAL MODEL

In this paper, 4-storey and 8-storey 2D models are used in three types which contain FSW, RWMF without cable and RWMF with post-tensioned cable. The modeling and analysis of frames were done using SAP2000 shown in figure 6. Steel moment frame, selected in a regular plan and shown in figure 6(b), has BOX and IPE profiles for columns and beams sections respectively designed based on AISC360[25].

![Figure 6. (a) view and sections of FSW. (b) Plan of main structure. (c) View of FSW (model 1). (d) View of RWMF without cable (model 2). (e) View of RWMF with cable (model 3)](image)

The compressive strength of confined concrete is assumed to be 25 MPa. The stress–strain relationship of confined and unconfined concrete is calculated following Paulay and Priestley (1992). The yield stress and ultimate strain of reinforcing steel are assumed to be 400 MPa and 0.12, respectively. The type of steel frame material is ST37. The column and beam elements are modeled using plastic hinge elements, in SAP2000. In these elements, the plasticity is concentrated at the ends of the element defined based on FEMA 356[26].

Applied vertical loads include dead load and live load are 11.03 kN/m and 4.9 kN/m for stories respectively and 12.6 kN/m and 3.68 kN/m for the roof. To consider P-Delta effect of the other frames, leaning columns are used based on AISC by applying point loads on columns. Shear wall cross section is 3000x200mm$^2$ and 3000x300mm$^2$ for 4-storey and 8-storey models respectively and its reinforcement bars are designed based on ACI318.

In modeling post-tensioned tendons, elastic cables are used in the software. The post-tensioning process of the tendons is done by applying loads at the end of the cables. Tensile strength and modulus of elasticity of steel cables are 1.6E+08 kgf/m2 and 1.60E+10 kgf/m2 respectively. Since the RRC acts as an upright simply supported beam or braced frame, the corresponding distribution of bending moments due to $F_i$ and $\bar{F}_i = P\phi = \bar{F}$ at any elevation $i \times h$ can be expressed as:

$$M_{core,i} = \frac{F_i h}{6m} \left[ (m^2 - 1) - (i^2 - 1) \right] + \frac{\bar{F}_i h}{2} \left[ (m - 1) - (i - 1) \right]$$

$$M_C / d' = T_C$$  \hspace{1cm} (1)
All terms are defined in figure 6(e) On the basis of the static analysis, required tendon force for 4-storey and 8-storey models are 93 ton and 350 ton respectively.

The total tendon force composed of initial tendon force $T_{0c}$ and that due to additional extensions can be shown to be equal to:

$$T_c = T_{0c} + 2d \frac{(E_w A_w / H)(E_c A_c / L_c)}{(E_w A_w / H) + (E_c A_c / L_c)} \phi$$  \hspace{1cm} (2)

With respect to cable forces, allowable drift, structural properties of RRC and cable, the area of cables are 45cm$^2$ and 60 cm$^2$ for 4-storey and 8-storey models are calculated respectively. In this calculation tendon is assumed without initial force ($T_{0c} = 0$).

To show the effect of initial force on the performance of RRC in the 4 storey model, two models are considered and analyzed under cyclic load. In one of them $A_c=10$ cm$^2$ and $T_{0c}=72$ ton (case 1) and the other one $A_c=45$ cm$^2$ and $T_{0c}=0$ (case 2). Figure 7 shows hysteresis behavior of two models and the strong compatibility between them. However, the cable with initial force has a higher strength about 100 kN.

![Hysteresis behavior of two models](image)

Figure 7. Hysteresis behavior of two models (case 1 and case 2)

### 4. TIME HISTORY ANALYSIS

As noted earlier, eleven earthquake records on soil type C are used in this research shown in table 1. These records are far field selected from FEMA-P695 [27]. Peak Ground Acceleration (PGA) is considered as a seismic parameter and scaled to 0.3g, 0.7g and 1.4g.

<table>
<thead>
<tr>
<th>Name, Station</th>
<th>Vs (m/s)</th>
<th>Fault</th>
<th>R (km)</th>
<th>M</th>
<th>PGA (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Northridge, Beverly Hills - Mulhol USC</td>
<td>356</td>
<td>Thrust</td>
<td>13.3</td>
<td>6.7</td>
<td>0.42</td>
</tr>
<tr>
<td>3 Duzce, Turkey Bolu ERD</td>
<td>326</td>
<td>Strike-slip</td>
<td>41.3</td>
<td>7.1</td>
<td>0.73</td>
</tr>
<tr>
<td>6 Imperial, Valley El Centro Array #11 US</td>
<td>196</td>
<td>Strike-slip</td>
<td>33.7</td>
<td>6.5</td>
<td>0.36</td>
</tr>
<tr>
<td>7 Kobe, Japan Shin-Osaka CUE</td>
<td>256</td>
<td>Strike-slip</td>
<td>46</td>
<td>6.9</td>
<td>0.24</td>
</tr>
<tr>
<td>9 Kocaeli, Turkey Duzce ERD</td>
<td>276</td>
<td>Strike-slip</td>
<td>98.2</td>
<td>7.5</td>
<td>0.31</td>
</tr>
<tr>
<td>14 Loma Prieta, Gilroy Array #3 CDMG</td>
<td>350</td>
<td>Strike-slip</td>
<td>31.4</td>
<td>6.9</td>
<td>0.56</td>
</tr>
<tr>
<td>15 Manjil-Iran UNAMUCSD</td>
<td>724</td>
<td>Strike-slip</td>
<td>40.4</td>
<td>7.4</td>
<td>0.5</td>
</tr>
</tbody>
</table>
5. PARAMETRIC STUDY

In this section, seismic performance of structures is evaluated with different cable initial forces under 7 earthquake records in three levels (0.35g, 0.7g, and 1.4g). With respect to the effect of cable initial forces in self centering, residual displacement and maximum inter storey drift are assessed. By considering static analysis mentioned before, the area of cable section is 45 cm$^2$ and 60 cm$^2$ for 4-storey and 8-storey respectively. Then 6 different initial forces for the 4-storey frame which include zero, 75, 150, 250, 350 and 500 ton and for the 8-storey frame which include 350, 450, 550 and 650 ton, are assumed. Also, a model without cable is compared to the others to show the influence of cables in the behavior of frames. Comparison of residual displacement and maximum inter storey drift for both frames are shown in figure 9 and 10.

Figure 8. Response spectra of 7 earthquake records

Figure 9: residual displacement of (a) 4 storey frame and (b) 8 storey frame
Figure 10: maximum inter storey drift of (a) 4-storey frame and (b) 8-storey frame

As seen, the effect of cable initial forces is noticeable in residual displacement so that RWMF without cable has maximum value and cable with 150 ton initial force has minimum value nearly in the 4-storey frame. Also, maximum and minimum values in the 8-storey frame are RWMF without cable and cable with 450 ton initial force respectively. The sensitivity of structural behavior to cable forces is due to non-rigidity of rocking wall. Furthermore, diversity of initial forces in cables does not affect maximum inter storey drift considerably. However, the best value can be selected by considering both results. Due to relaxing between cable and shear wall, the optimum value of initial forces should be calculated. In this case, 150 ton and 350 ton are the optimum initial force among these options for 4-storey and 8-storey respectively.

6. RESULT AND DISCUSSION

Time history analysis were performed for three models in two levels of height (4 and 8-storey) under 7 earthquake records with PGA= 0.7g. Global plastic failure patterns generally result in smaller maximum drift ratios and more economical solutions [22]. The use of rocking walls in conjunction with reduced beam sections leads to preferred plastic collapse patterns, such as those shown in Figure 11. In this figure, distribution of plastic hinges of three models is represented under Imperial Valley earthquake record with PGA=0.7g. The rocking wall can prevent progressive collapse [22], due to soft story failure and impose a uniform rotation on the MF but cannot increase the ultimate carrying capacity of the system.

<p>| Table 2: First period of (a) 4-storey models and (b) 8-storey models |
|---------------------------------|----------------|---------------------------------|</p>
<table>
<thead>
<tr>
<th>First period (s)</th>
<th>4-storey frames</th>
<th>First period (s)</th>
<th>8-storey frames</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.68</td>
<td>Model 1</td>
<td>1.2</td>
<td>Model 1</td>
</tr>
<tr>
<td>1.6</td>
<td>Model 2</td>
<td>4</td>
<td>Model 2</td>
</tr>
<tr>
<td>1.34</td>
<td>Model 3</td>
<td>3.4</td>
<td>Model 3</td>
</tr>
</tbody>
</table>

Figure 11: performance level and plastic hinges distribution of (a): model 1 (b): model 2 (c): model 3 under Imperial Valley earthquake record
As seen, the proposed approach is capable of treating collapse prevention, self-alignment, and uniform drift as inherent properties of the proposed system. In FSW, the performance level of structure is CP since the shear wall collapsed. However, in RWMF, seismic performance of the structure is improved from CP to LS. The MF provides lateral restraint and makes the core act as a statically determinate element. This forces the reference line of displacements to pass through both the pin and the free end of the rocking wall. In addition, the RRC tends to redistribute seismic moments rather evenly between groups of similar members such as beams and columns of equal spans and heights respectively. The physical behavior of the RWMF can best be visualized by the MF and the RRC resisting the lateral forces together until the MF becomes a mechanism [22]. To show drift distribution of 4-storey and 8-storey frames under 7 earthquake loads, average of inter story drift is drawn in figure 12.

Figure 12: Average of maximum inter storey drift of a): 4storey frames b): 8storey frames

Figure 12 (a) shows that maximum inter story drift concentration reduced in 4-storey RWMF considerably. However, in 8-storey RWMF, uniform drift is not imposed completely because the rigidity of rocking wall is no sufficient. The total value of drift increases due to rocking motion and it should be controlled using supplementary devices to be in allowable range. Elastic unbounded cables help to realign the system after loading is removed so that residual displacement reduces enormesly. Average of residual displacement for each model under 7 earthquake records is calculated after free vibration and shown in figure 13.

Figure 13: residual displacement of 4-storey and 8-storey frame

In this figure, model 3 (RWMF) has the lowest residual displacement because of existing pretensioned cables whereas the other models have higher values. Rocking wall causes increasing the frist period of the frame so that the input energy and seismic loads and consequently the absolute acceleration of storeis decrease. The average of the absolute acceleration of storeis under 7 earthquake records and input energy under lomaperita record is compared for each model in figures 14 and 15.
Figure 14: Average of storey absolute acceleration of a) 4-storey frames b) 8-storey frames

Figure 15: Comparison of input energy of 3 models under Duzce earthquake

According to above figures RWMF without cable has the lowest period among three models. Thus, it has minimum acceleration and input energy whereas FSW has the maximum value.

7. INCREMENTAL DYNAMIC ANALYSIS (IDA)

Incremental dynamic analysis (IDA) is a parametric analysis method that has recently emerged in several different forms to stimulate more thoroughly structural performance under seismic loads. It involves subjecting a structural model to one (or more) ground motion record(s), each scaled to multiple levels of intensity, thus producing one (or more) curve(s) of response parameterized versus intensity level [28]. Although there are several seismic parameters to show the effect of earthquake records, in this paper Peak Ground Acceleration (PGA) is selected as an intensity measure in IDA curves. Each record is applied to the models with the step of 0.2g and maximum intensity of 3g. On the basis of FEMA-356 [26], the selected structural parameter is maximum inter story drift. To assess the collapse level of structures, one of the criteria must be met according to FEMA-P695[27] as follows:

1. If inter story drift is more than 10%
2. If slope of IDA curve is reduced less than 20%

IDA curves of three models are obtained in 4-storey and 8-storey under 7 earthquake records and shown in figures 16 and 17 ((a), (b) and (c)). Also median values of IDA curves are compared in figures 16 and 17(d).
In 4-storey FSW model, the number of plastic hinges is increased enormously in PGA=2g and the structure tends to collapse so that it is collapsed in PGA= 2.4g finally. Seismic performance of RWMF
with and without cables (model 2 and 3) are nearly the same in seismic levels less than 2.4g. However, RWMF with cable has better seismic performance in a higher level of 2.4g. In 8-storey model, SWMF (model 1) and RWMF without cable (model 2) are collapsed in PGA= 2.4g and 2.2g respectively. In addition, the number of plastic hinges of both models is increased enormously in PGA=1.6g. In Seismic levels lower than 1.6g, FSW has the best seismic performance. However, after this level, RWMF with cable (model 3) is the best one.

8. CONCLUSIONS

- The proposed RWMF, with or without supplementary devices, tends to deform with zero to negligible drift concentration along its height.
- The proposed RWMF tends to prevent soft story failure and the formation of column base plastic hinges [38].
- The proposed RRCs reduce residual stresses and deformations due to the strong ground motion.
- The drift is not sensitive to minor changes in the wall and supplementary device stiffnesses.
- In GBRMFs no moments are transmitted to the footings. The grade beams prevent the formation of plastic hinges at column supports.
- For equal mass, RWMFs have longer natural periods of vibration and attract significantly smaller seismic forces than their conventional counterparts.
- RRCs suppress contributions of higher modes of vibrations. In other words the dominant mode shape remains unchanged during all phases of loading.
- Rocking wall can reduce the storey acceleration.
- Performance of mid rise structures with rocking walls is sensitive to the existence of cables and their initial forces.
- RWMF has better performance level than conventional counterparts.

9. REFERENCES


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