INFLUENCE OF ROCKING ON THE SEISMIC RESPONSE OF HIGH RISE BUILDINGS RESTING ON SLIDING BEARINGS

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ABSTRACT

Increased seismic performance demands in high seismicity regions have led to the application of base isolation to the design of a continuously increasing number of high-rise office and residential buildings. Sliding base isolation systems that combine high displacement capacities and low friction coefficients can effectively change the dynamic characteristics of the building’s superstructure and result in the required shift of the fundamental natural period and subsequently the elimination of the influence of higher modes.

In this paper, the nonlinear dynamic analysis of an existing and representative, high-rise, base isolated building is used to demonstrate the effectiveness of sliding base isolation in the reduction of seismic response. The relation between earthquake intensity and the corresponding development of rocking response is quantified via an incremental dynamic analysis (IDA). Despite the magnitude of the accidental stiffness eccentricity developed in the case of FP isolation systems due to rocking response, the parasitic torsional effects appear to be of low significance. Analyses presented herein have been carried out using a specialized software, specially developed by the authors, for efficient nonlinear time-history analysis of base isolated structures.

Keywords: Base isolation; High-rise buildings; Sliding isolation; Rocking response; Incremental dynamic analysis;

1. INTRODUCTION

Base isolation has been applied to various types of structures (e.g. buildings, bridges, tanks, nuclear power plants etc.) over the past 50 years. Structures designed by this design technique behave linearly and elastically even for destructive earthquakes, as very low seismic forces are induced into them. These reduced seismic forces ensure the integrity of their structural and nonstructural elements and provide a higher level of protection for the occupants. The economic advantages of the implementation of base isolation are therefore evident and can be quantified according to the concept of performance based design.

The reduction of seismic forces through the adoption of base isolation is achieved by anti-seismic devices aiming at the elongation of the fundamental natural period of the structure in the range of low earthquake spectral accelerations and possibly at the energy dissipation by means of a reliable elastoplastic mechanism. The increase of the natural period of the order of 5 seconds provides the additional elimination of the influence of higher modes of vibration, usually laying in the range of high spectral accelerations, even in the case of flexible structures such as high-rise buildings.

The low effective stiffness of the isolation system, required to achieve such a level of increase of the fundamental natural period, implies the development of large relative displacements between the ground and the isolation disk. Friction-driven base isolation systems, not facing buckling problems, have high displacement capacities and are therefore highly effective when such high values of the fundamental natural periods are required.

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The magnitude of the developed overturning moments during seismic response is an important aspect of the seismic design of high-rise buildings. These moments can lead to local or global uplifts and to their possible global overturning. Extensive research has been directed to the investigation of rocking especially focusing on the effects it can have on the enhancement of seismic response (Meek, 1978). In the case of base-isolated high-rise buildings, seismic forces and the corresponding overturning moments at the base are drastically decreased, reducing the otherwise developed rocking response. In the current paper, the extent of this reduction is quantified considering various levels of earthquake intensities. Moreover, the variation of the rocking dependent stiffness of friction pendulum (FP) bearings is examined and discussed.

2. OBJECTIVES

The research presented herein concerns the behavior of high-rise buildings. Its main objectives are the investigation of:

- the effectiveness of friction driven base isolation systems regarding the magnitude of the developed base shear forces and overturning moments,
- the rocking behavior of buildings with dual structural system, consisting of moment resisting frames and a central core,
- the comparison of rocking response between fixed base and isolated building configuration,
- the relation between earthquake intensity and the corresponding development of rocking,
- the variability of the horizontal stiffness of FP bearings due to rocking and the quantification of the effect it has on the response of the structure.

3. ANALYSIS DETAILS AND METHOD OF APPROACH

For the needs of this study a series of non-linear time history analyses are performed with the use of a structural analysis program specially developed by the authors for the analysis and design of large scale seismically isolated real structures.

The structure examined is an existing and representative, 24-story building, seismically isolated with elastomeric isolators, which herein is considered to be resting on sliding isolators. The superstructure of the building is modeled with the use of beam elements with 16434 nodal unknowns. Each floor is considered to be a nondeformable in-plan disk with its mass considered lumped at the mass center, with two horizontal translational degrees of freedom and a rotational one around the vertical axis. The superstructure is assumed to remain elastic and the non-linear frictional behavior of isolators, flat or curved, is modeled with the use of the Bouc-Wen hysteresis model. Adaptive step fourth order Runge-Kutta method is used for the integration of the corresponding system of differential equations, in which, step size is a function of the velocity of sliding occurring in every frictional interface. For both flat and curved sliding isolators, the friction coefficient is assumed to be velocity dependent according to experimental results in Teflon-Steel sliding interfaces (Constantinou, et al., 1990).

The first objective is investigated by comparing the response between the base isolated version of the structure using flat sliding bearings, and the fixed base one. Realistic characteristics of the base isolation system have been chosen regarding both stiffness and frictional properties of the Teflon-steel sliding interfaces. The comparison is made by means of developed base shear forces and overturning moment as both these measures are critical to the seismic design of the superstructure.

Regarding the second objective, the variation of vertical support reactions due to rocking is computed in each time step, taking into account the stiffness of the three-dimensional structural model. The overturning moments at the level of the isolation interface undertaken by the central core and the moment resisting frames respectively, are examined as they are considered indicative of the rocking behavior of the structure.

Regarding the third objective, the relation between earthquake intensity and the corresponding development of rocking is identified by Incremental Dynamic Analysis (IDA) (Vamvatsikos &
Cornell, 2002), in which the recorded earthquake ground motions are gradually scaled, with peak ground acceleration (PGA) as the intensity measure of the record.

Concerning the fourth objective, the horizontal stiffness of FP isolators is proportional to the corresponding vertical reactions which obviously vary during the response to earthquake excitation. The effect of stiffness eccentricity of the isolation system due to the rocking response on the rotational seismic response is examined. The relative distance between the stiffness center of the isolation system and the center of mass of the structure in plan is computed for each time step of the dynamic analysis.

4. DESCRIPTION OF THE BUILDING

4.1. Superstructure characteristics

The high-rise building used for the time history analyses is an existing seismically isolated 24-story building with a total height of 87.3m measured from the level of the isolation interface. This innovative building designed by Dr. Eng. Aristarchos Ikonomou implements his A1 earthquake guarding system (Ikonomou, 1972). It was constructed in 1972 and is located in Piraeus Port, Greece. The structure fully rests on elastomeric isolators. Uplift restraint configurations are provided at each support location at the level of the isolation interface. The structural system of the building is made of reinforced and prestressed concrete and consists of moment resisting frames in the perimeter of the structure and shear walls for the central core. The total weight considered for the dynamic analyses is $W_{\text{tot}} = 277\,\text{MN}$ distributed in the various floors of the building. In Figure 1 two vertical sections along the principal directions of the building are presented. For the needs of this study, alternative sliding supports conditions are assumed.

![Figure 1. Vertical sections at the principal directions of the building](image)
4.2. Base isolation systems considered

Two different sliding base isolation systems are considered, both having stiffness $K = 55 \text{MN/m}$. The first system consists of 28 flat sliding isolators supporting the weight of the superstructure and 28 separate linear elastic springs with constant stiffness providing the required restoring forces.

The second system consists of 28 FP isolators with radius $R = 5 \text{m}$ supporting the superstructure and also providing the required restoring forces.

The frictional characteristics of the two systems are assumed to be the same. In particular the coefficient of friction $\mu_s$ is assumed to vary with respect to the sliding velocity $\dot{U}(t)$ at each sliding position between $f_{\text{min}} = 0.02$ and $f_{\text{max}} = 0.06$ according to relation (Constantinou, et al., 1990):

$$\mu_s = f_{\text{max}} - Df e^{-a|\dot{U}(t)|}$$  

Where $f_{\text{max}} = 0.06$ the maximum coefficient of friction, $Df = 0.04$ and $a = 25 \text{sec/m}$ a constant selected for the description of variation of the coefficient of friction.

Before the initiation of sliding increased value of breakaway friction is taken into account with breakaway coefficient $b = 2$ multiplying the minimum coefficient of friction $f_{\text{min}}$.

<table>
<thead>
<tr>
<th>Eigenmode</th>
<th>Fixed Base</th>
<th>Base Isolated</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>X Direction</td>
<td>Z Direction</td>
</tr>
<tr>
<td>Eigenmode</td>
<td>T (sec)</td>
<td>Modal Participation (%)</td>
</tr>
<tr>
<td>1</td>
<td>2.04</td>
<td>59.03</td>
</tr>
<tr>
<td>2</td>
<td>2.02</td>
<td>≈0</td>
</tr>
<tr>
<td>3</td>
<td>1.59</td>
<td>0.35</td>
</tr>
<tr>
<td>4</td>
<td>0.63</td>
<td>14.16</td>
</tr>
<tr>
<td>5</td>
<td>0.56</td>
<td>0.18</td>
</tr>
<tr>
<td>6</td>
<td>0.50</td>
<td>19.05</td>
</tr>
<tr>
<td>7</td>
<td>0.34</td>
<td>2.51</td>
</tr>
<tr>
<td>8</td>
<td>0.32</td>
<td>5.03</td>
</tr>
<tr>
<td>9</td>
<td>0.22</td>
<td>0.52</td>
</tr>
<tr>
<td>10</td>
<td>0.21</td>
<td>10.24</td>
</tr>
</tbody>
</table>

Sum 81.25 | 82.31 | Sum 99.95 | 99.94

5. EARTHQUAKE GROUND MOTIONS CONSIDERED

Six destructive seismic events with magnitudes greater than 6.5 have been chosen. For each seismic event, two records each with 3 components are used representing both near fault and mid-distance seismic excitations. Records have been obtained from the PEER Ground Motion Database, details about these seismic records are presented Table 2.

Scaling of the records with respect to the PGA of the two horizontal components is used in incremental dynamic analyses presented herein. Therefore far-field, low intensity records are avoided as when scaled extensively they represent unrealistic seismic events due to the filtering of the high frequency content by the soil.
Table 2. Ground motions considered for the time-history analyses.

<table>
<thead>
<tr>
<th>Earthquake Event</th>
<th>Year</th>
<th>Magnitude</th>
<th>GNA RSN</th>
<th>PGA (g)</th>
<th>Arias Intensity (m/s)</th>
<th>5-95% Duration (sec)</th>
<th>Closest Distance to Rupture Plane (km)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Imperial Valley</td>
<td>1979</td>
<td>6.53</td>
<td>179</td>
<td>0.53</td>
<td>1.40</td>
<td>10.30</td>
<td>7.05</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>160</td>
<td>0.79</td>
<td>6.10</td>
<td>9.70</td>
<td>2.66</td>
</tr>
<tr>
<td>Loma Prieta</td>
<td>1989</td>
<td>6.93</td>
<td>741</td>
<td>0.65</td>
<td>5.4</td>
<td>9.8</td>
<td>10.72</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>802</td>
<td>0.52</td>
<td>1.50</td>
<td>9.40</td>
<td>8.50</td>
</tr>
<tr>
<td>Northridge</td>
<td>1994</td>
<td>6.69</td>
<td>1012</td>
<td>0.41</td>
<td>1.10</td>
<td>8.00</td>
<td>19.07</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1044</td>
<td>0.75</td>
<td>5.70</td>
<td>5.90</td>
<td>5.92</td>
</tr>
<tr>
<td>Duzce</td>
<td>1999</td>
<td>7.14</td>
<td>1602</td>
<td>0.88</td>
<td>3.70</td>
<td>9.00</td>
<td>12.04</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1617</td>
<td>0.90</td>
<td>10.00</td>
<td>13.10</td>
<td>3.93</td>
</tr>
<tr>
<td>San Fernando</td>
<td>1971</td>
<td>6.61</td>
<td>77</td>
<td>1.51</td>
<td>0.90</td>
<td>12.00</td>
<td>19.30</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>71</td>
<td>0.39</td>
<td>8.90</td>
<td>7.30</td>
<td>1.81</td>
</tr>
<tr>
<td>Kobe</td>
<td>1995</td>
<td>6.90</td>
<td>1107</td>
<td>0.34</td>
<td>1.70</td>
<td>13.20</td>
<td>22.50</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1111</td>
<td>0.51</td>
<td>3.40</td>
<td>11.20</td>
<td>7.08</td>
</tr>
</tbody>
</table>

6. EFFECTIVENESS OF BASE ISOLATION

The effectiveness of sliding base isolation in high-rise buildings is quantified by comparing the response of the base isolated structure with flat sliding bearings to the response of the fixed base one. For this comparison, the resultant maximum base shear forces $V_b = \sqrt{V_x^2 + V_z^2}$ and overturning moments $M_b = \sqrt{M_x^2 + M_z^2}$ are chosen as indicative measures. The effectiveness ratio, summarized for the twelve earthquake records in Figure 2, is the ratio of the response of the fixed base building to the response of the base isolated one.

Figure 2. Resulting base shear and overturning moment effectiveness ratio for flat sliding isolation system

The floor shear force and overturning moment envelopes for Imperial Valley earthquake (RSN 160) shown in Figure 3. reflect the amount of reduction achieved along the height of the building. Excessive values of floor shear forces are responsible for the damage of non-structural components and systems, inducing unacceptable interstorey drift values. Overturning moments are also critical due to the variability of axial forces they induce to the columns of the perimeter or to the central core elements of high-rise buildings. The reduction of overturning moments achieved can also be important for the rational design of the foundation and the corresponding reduction of the total cost.
ROCKING RESPONSE OF THE STRUCTURE

7.1. General

A great amount of research has been directed to the rocking response of rigid bodies mainly due to the unwanted overturning of equipment, museum exhibits etc. In the case of buildings, the rocking response deviates a lot from the rigid block model as the inertia forces developed result from the linear or non-linear response of a multiple degree of freedom system. The vertical reaction developed at the sliding interface of each flat or curved sliding isolator is distinctively changing during the response to earthquake ground motions both due to vertical component of the seismic input and to the rocking response of the superstructure. The vertical response is assumed to be identical to the vertical ground motion, due to the high vertical rigidity of the building and therefore the change in the vertical reaction at the sliding interface of each isolator is proportional to the vertical acceleration of the structure. Additional vertical reactions due to rocking response are calculated as a function of the developed inertia forces at the direction of the dynamic degrees of freedom of the superstructure. The rocking behavior depends on the geometry of the structure as well as on the structural system of the building. The quotient between width and height of the building under consideration, also known as slenderness ratio, is 1:3.5.

7.2. Rocking characteristics of dual structural system

The design of high-rise buildings in seismic regions has been relying on the concept of ductile behavior, with dual structural systems consisting of moment resisting frames and core walls being widely used. Such core walls are designed to exhibit an elastoplastic behavior near the base of the building and therefore limit the seismically induced forces. Central core walls are also very effective for the control of seismic interstorey drifts without major restrictions on the architectural design. In the case of seismically isolated buildings, seismic forces are drastically reduced, and therefore the elastoplastic behavior of the central core is no longer required. Possible need for additional lateral stiffness can be covered by uniformly increasing the stiffness of the moment resisting frames. As shown in Figure 4, overturning moments developed at the base of the central core of high-rise buildings can be considerably high, compared to the ones developed at the base of the moment resisting frames, resulting in an increased tendency for uplift in these supports.
Due to their articulated arrangement, Teflon-steel sliding isolators provide rotational releases around the two horizontal axes and therefore overturning moments developed at the level of the base are transferred to the foundation, only by means of variation of the vertical reactions at the support locations.

In Figure 5 the ratio of overturning moments undertaken by the core to the overturning moments undertaken by the frames of the perimeter is shown. This ratio is higher for seismic forces in X direction due to differences in the stiffness ratio between the frames and the central core in the two directions. Special attention must be given to the protection of anti-seismic devices supporting such central core walls under uplift conditions.

7.3. Reduction of rocking response by sliding base isolation

The reduction of rocking response by implementation of sliding base isolation is investigated through the study of eccentricity parameter \( e \) which is defined as the ratio of the overturning moment caused by lateral seismic forces to the total vertical reaction considering also the vertical component of the ground motions. This eccentricity is computed at the level of the isolation interface.

In Figure 6 this reduction is presented by evaluating the reactions’ eccentricity effectiveness ratio i.e. the ratio of the maximum eccentricity developed for the fixed base structure, to that of the base isolated one. This effectiveness ratio ranges between 1.57 and 2.75.
In Figure 7 the aforementioned maximum eccentricity effectiveness ratio is evaluated for variable intensities of the twelve PGA-normalized earthquake records. It is observed that for the majority of the earthquake events the effectiveness of base isolation in the reduction of rocking response increases with respect to the peak ground acceleration of the earthquake excitation.

7.4. **Relation between rocking and peak ground acceleration**

The numerical values of eccentricity $e$ with respect to the intensity of the earthquake event are also an important factor for the structural design. In Figure 8 the maximum values of the two components $e_x$ and $e_z$ along the principal axes of the building are evaluated for the selected earthquake records gradually scaled for PGA up to 0.8g.

It is observed that for the majority of seismic inputs, the slope of the eccentricity curves decreases with the increase of peak ground acceleration. The value of eccentricity $e$ remains in the area of the central core, with dimensions $12.70 \times 9.35$.

Specific seismic records like *Imperial Valley 179*, *Loma Prieta 802* and *San Fernando 77* lead to high eccentricity values. All these records have pulse-type characteristics resulting in excessive values of
ground displacements even larger than 70cm and the corresponding development of high base shear forces induced to the structure by the recentering devices.

Large dispersion of the results imply that the rocking response is weakly related to PGA which is the intensity measure chosen, on the contrary the frequency content characteristics of the ground motion record seem to be of high significance.

7.5. **Stiffness variability of friction pendulum isolators**

The force displacement relationship for the idealization of FP bearings consists of a linear and a non-linear part:

\[
\begin{align*}
(F_x(t)) & = W(t) \left( \frac{u_x(t)}{R} \right) + \mu(t) W(t) \left( \frac{z_x(t)}{z_z(t)} \right) \\
(F_z(t)) & = \mu(t) W(t) \left( \frac{z_x(t)}{z_z(t)} \right)
\end{align*}
\]

(3)

Where \( F_x \) and \( F_z \) are the forces developed by the isolator, \( W(t) \) the vertical reaction at the sliding interface, \( R \) is the radius of the concave sliding surface, \( \mu(t) \) is the active friction coefficient generally accounting for variations due to velocity, pressure, temperature etc. and \( z_x \) and \( z_z \) are the dimensionless coefficients of Bouc-Wen hysteresis model.

The stiffness of a FP bearing in the linear part of the above equations is \( K(t) = W(t)/R \) and is a function of time. Adaptability of the stiffness with respect to vertical reactions is an important characteristic of this system especially in structures with varying live loads (Zayas, et al., 1987).

The variation of the vertical reactions \( W(t) \) due to rocking response results in a continuous change in stiffness of FP bearings, a continuously changing position of the center of stiffness of the isolation system and a corresponding stiffness eccentricity \( e_s \). Stiffness eccentricity is defined as the horizontal distance between the center of stiffness of the isolation system and the center of mass of the building. The trace of the center of stiffness as well as the projection of the center of mass of the building on the isolation disk, for the various seismic events, in the case of FP isolators, are shown in Figure 9.
Figure 9. Isolation disk and the trace of center of stiffness of isolation system for the various seismic events

The relation between maximum stiffness eccentricity \( e_s \) and PGA is presented in Figure 10. For all selected earthquake records, a continuous increase in stiffness eccentricity is clearly observed. Despite these significant stiffness eccentricities, no systematic changes in the torsional response of the building are observed. This low torsional sensitivity is justified by the high rotational inertia of the whole structure and the small stiffness of the recentering devices.
8. CONCLUSIONS

The conclusions of the present non-linear dynamic analysis of an existing high-rise isolated building, assumed to rest on sliding bearings for the purposes of the present work and subjected to a wide range of earthquake intensities using an incremental dynamic analysis are the following:

- The high displacement capacities of sliding isolation systems in combination with low friction coefficients can ensure high levels of seismic protection effectiveness even for flexible high-rise buildings.
- The rocking response of core walls in buildings with dual structural system results in a range of vertical reactions in these locations, therefore special consideration must be given to the selection of anti-seismic devices for these areas.
- Implementation of sliding base isolation results in the reduction of rocking response by factors even greater than four, for the seismic records chosen. The level of these reductions depends on special characteristics of earthquake records, other than PGA, like the frequency content of the record.
- For the majority of seismic records examined, the eccentricity of the vertical reactions is located inside the area of the central core, independently of the PGA scaling. For seismic records having pulse type velocity characteristics and notable horizontal ground displacements, rocking behavior is increased as high values of shear forces are transferred to the superstructure through the recentering devices.
- The stiffness eccentricities of FP isolation system due to rocking are not significantly affecting the torsional response of the structure.
REFERENCES


