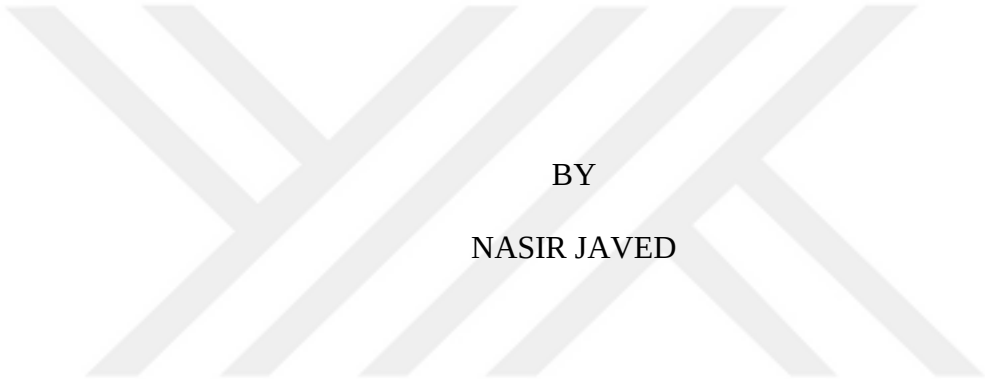


DEVELOPMENT OF ROCKING-COLUMN (ROC) SEISMIC BASE
ISOLATION SYSTEM FOR BUILDINGS

A THESIS SUBMITTED TO
THE GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES
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NASIR JAVED

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Approval of the thesis:

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ISOLATION SYSTEM FOR BUILDINGS**

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ABSTRACT

DEVELOPMENT OF ROCKING-COLUMN (ROC) SEISMIC BASE ISOLATION SYSTEM FOR BUILDINGS

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Earthquake mitigation of buildings has been a long-standing matter of research. Researchers have proposed many systems that can be categorized as passive control, active control, and hybrid systems. Seismic base isolation, usually linked to the passive control category, is a well-known technique to isolate a structure from harmful effects of earthquakes. In this research, a new seismic isolation system, different from the conventional seismic base isolation systems, has been proposed and named as Rocking-Column (ROC) seismic isolation system. The working principle of ROC is to isolate a building by the use of story-high rocking columns. The study is inspired from earthquake response of ancient rocking columns used in Greek Civilization as well as Pagoda structures constructed mostly in Japan, where the first floor or all stories are flexible and behave like isolators. The research comprises following stages: 1) comprehensive review of the literature regarding rocking of rigid blocks, 2) development of a nonlinear numerical model in commercially available software (SAP2000), which simulates the rocking column behavior, 3) experimental validation of the numerical model via testing of small scale prototypes, 4) analyses of different ROC seismically isolated building models (2D, 3D, mass and stiffness vertical irregularities), and 5) development of preliminary

design guidelines for ROC seismic isolation systems installed in medium rise residential buildings. Analyses results have proven that the width and height (slenderness) of rocking rectangular columns are the most important parameters which outline the performance of the ROC seismic isolation systems. Other parameters that influence the effectiveness of the ROC seismic isolators include the nature and magnitude of earthquakes, total number of floors, and column bay size of buildings. The simulation results have demonstrated that ROC systems indubitably isolates buildings even for largest earthquakes. Additionally, the suggested “restraining mechanism” in the ROC isolation system prevents any total collapse of buildings. The proposed system may be applied to existing buildings, which may also provide an additional parking space or an extra floor. The outcome of the analyses showed that the ROC system generally performs better than that of the conventional isolation systems of laminated rubber bearing (LRB) and friction pendulum system (FPS). The proposed ROC seismic isolation system may provide an alternative to the existing seismic isolation systems for medium rise buildings. The major design parameters of a) ROC’s column height and base width (slenderness), b) number of stories, c) bay size (5m or 7.5m), d) different earthquake records are changed and responses for i) overturning, ii) floor accelerations, iii) interstory drifts, and iv) first floor total shear were obtained in graphical format, which then helped to produce preliminary design tables of the ROC isolator system for medium rise residential buildings.

Keywords: Rocking Column, Seismic Base Isolation, Rigid Block Dynamics

ÖZ

BİNALAR İÇİN SALLANAN-KOLON (SAK) SİSMİK İSOLASYON SİSTEMİ GELİŞTİRİLMESİ

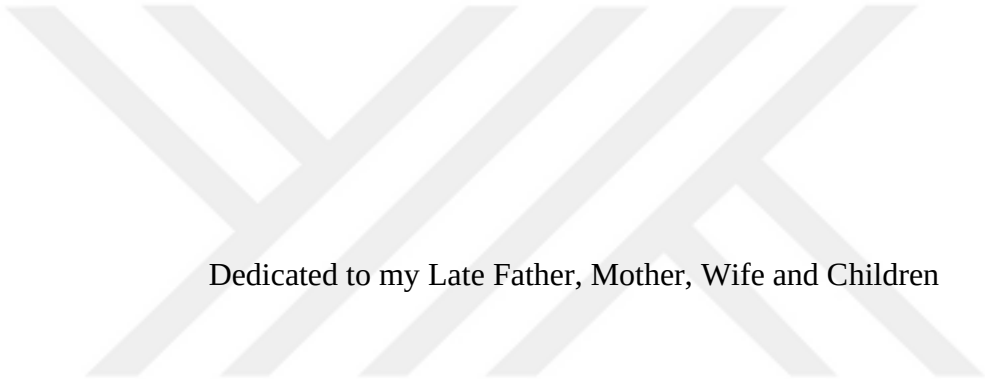
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Depremler tüm dünyada binalara ve yapılara ciddi zararlar vermektedir. Araştırmacılar yıllar boyunca depremlerden kaynaklanan zararları hafifletmek için pasif kontrol, aktif kontrol ve hibrit sistem olarak kategorize edilebilecek birçok teknik önermişlerdir. Genellikle pasif kontrol kategorisi altında tanımlı olan sismik zemin izolasyonu, bir yapıda depremlerin zararlı etkilerini izole etmek için iyi bilinen bir tekniktir. Bu çalışmada, geleneksel sismik taban izolasyon sistemlerinden farklı yeni bir sismik izolasyon sistemi önerilmiş ve Sallanan-Kolon (SAK) sismik izolasyon sistemi olarak adlandırılmıştır. SAK'ın çalışma prensibi, bir binayı kat yüksekliğinde sallanan kolonlar üzerine yerleştirerek yerden sismik olarak izole etmektir. Çalışma, Yunan Uygarlığında kullanılan antik sallanan sütunların depreme olumlu tepkisinden ve çoğunlukla Japonya'da inşa edilen, birinci katın veya tüm katların esnek olduğu ve izolatörler gibi davrandığı Pagoda yapılarından esinlenmiştir. Araştırma aşağıdaki aşamalardan oluşmaktadır: 1) rijit blokların sallanmasına ilişkin literatürün kapsamlı bir şekilde gözden geçirilmesi, 2) sallanan kolon davranışını simüle eden ticari olarak mevcut yazılımda (SAP2000) doğrusal olmayan sayısal modellerin geliştirilmesi, 3) sayısal modelin, küçük ölçekli

prototiplerin test edilmesiyle deneysel olarak doğrulaması, 4) farklı SAK ile izole edilmiş bina modellerinin analizleri (2B, 3B, kütle ve rijitlik düşey düzensizlikleri) ve 5) orta yükseklikte 5-7 katlı konutlarda kurulu SAK sismik izolasyon sistemleri için ön tasarım tablolarının geliştirilmesi. Analiz sonuçları, sallanan dikdörtgen kolonların genişliğinin ve yüksekliğinin (narinlik), SAK sismik izolasyon sistemlerinin performansını özetleyen en önemli parametreler olduğunu kanıtlamıştır. SAK sismik izolatörlerinin etkinliğini etkileyen diğer parametreler arasında depremlerin niteliği ve büyüklüğü, toplam kat sayısı ve binaların kolon açıklığı boyutu bulunur. Simülasyon sonuçları, SAK sistemlerinin en büyük depremler için bile binaları etkin şekilde izole ettiğini göstermiştir. Ek olarak, SAK izolasyon sisteminde önerilen "sınırlama mekanizması", stabilitenin bozulması durumunda dahi binaların çökmesini önler. Önerilen sistem, ek bir araba park alanı veya ekstra bir kat da sağlayabilir ve mevcut binalara da uygulanabilir. Analizlerin sonucunda, SAK sistemi genel olarak lamine kauçuk yastık (LRB) ve sürtünme sarkaç sistemi (FPS) gibi geleneksel izolasyon sistemlerinden daha iyi performans göstermiştir. Önerilen SAK sismik izolasyon sistemi, orta yükseklikte binalar için mevcut sismik izolasyon sistemlerine bir alternatif sağlayabilir. Yapılan çalışmada, a) SAK'ın kolon yüksekliği ve taban genişliği (narinlik), b) kat sayısı, c) kolon açıklık boyutu (5m veya 7.5m), d) farklı deprem kayıtları değiştirilmiş ve i) devrilme, ii) döşeme kat ivmeleri, iii) katlar arası göreceli yatay deformasyonlar ve iv) birinci kat toplam kesme kuvvetleri grafik formatında elde edilmiştir. Bu veriler kullanılarak orta yükseklikte (4-7 kat) konutlar için SAK izolatör sisteminin ön tasarım tabloları hazırlanmıştır.

Anahtar Kelimeler: Sallanan Kolon, Sismik Zemin İzolasyonu, Rijit Blok Dinamiği



Dedicated to my Late Father, Mother, Wife and Children

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CHAPTER 1

INTRODUCTION

Earthquakes may cause loss of lives and extreme damages to infrastructure. Researchers and scientists have proposed many techniques for buildings to mitigate damages due to earthquakes; some are passive control systems, active control systems, hybrid systems, tuned mass dampers, braced dampers, and base isolation systems. Seismic base isolation also known as base isolation is a type of passive control system used to isolate the structure from harmful effects of earthquakes. This research is focused on development of a base isolation system suitable for buildings which is named as Rocking-Column (ROC) seismic base isolation system.

It is essential to understand the concept of rocking before the explanation of ROC seismic isolation system. Rocking is the back-and-forth motion of an object. Rocking of a rigid rectangular column may be defined as the to-and-fro motion of the column in which column rotates about one edge having no contact of the second edge with the ground or vice versa. Figure 1.1 shows a simple rectangular column having some weight at its top. During rocking motion, say the rocking towards right direction, the column rotates about the pivot of rocking uplifting the other edge. As a result, a restoring force develops in the system, which tries to bring the column back to its original position. However, if the rocking displacement exceeds the width of the rocking column, it may cause overturning of the rocking column.

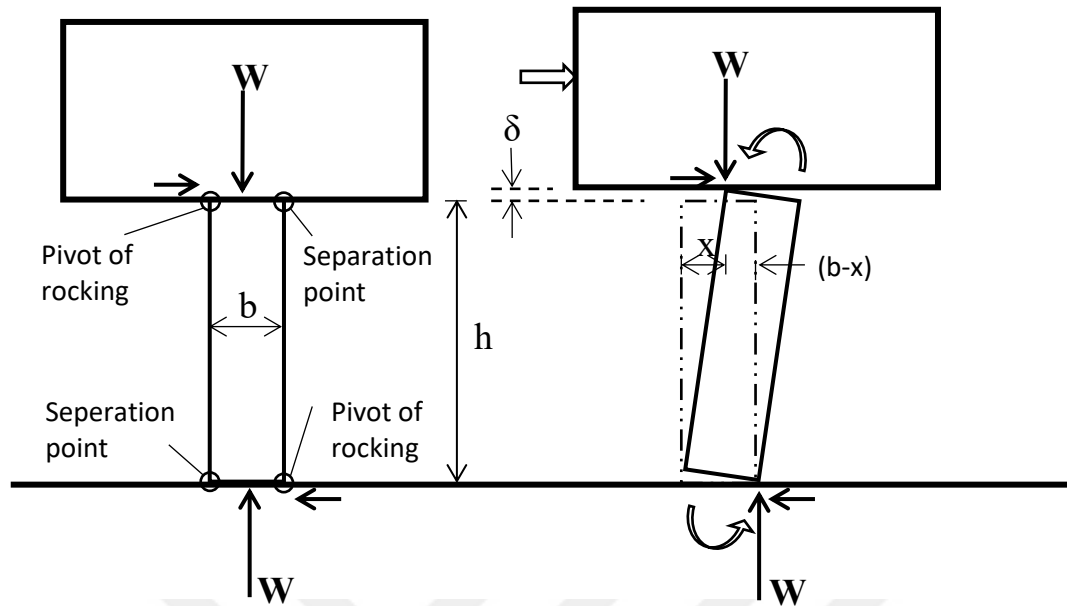


Figure 1.1 Basic concept of a rocking column

The basic concept of the rocking column may be extended to the whole story to isolate the upper structure. The application of the concept of rocking column to use as a story high isolator may be envisaged in Figure 1.2 in which bottom story columns are allowed to rock.

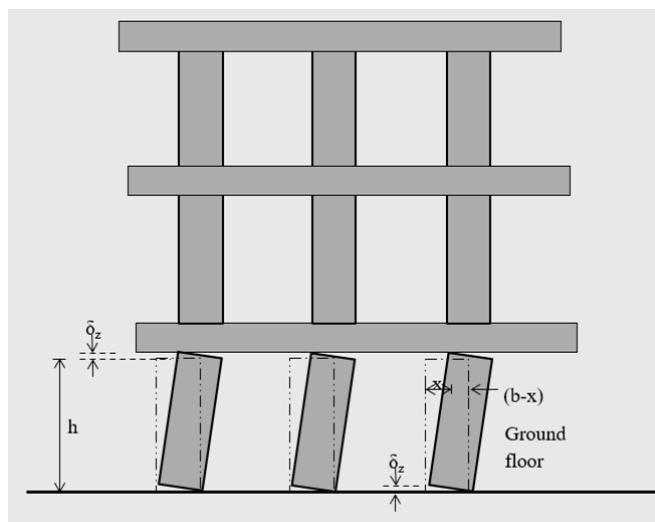


Figure 1.2 Concept of rocking column extended to use as a story high seismic isolation.

A brief description of ROC seismic base isolation system is explained in Figure 1.3. In ROC seismic base isolation, super structure is isolated by rocking of columns during earthquakes and rocking is restrained via walls which are constructed to control the degree of rotation of the columns and to increase stability of rocking system. Metallic plates and shear locks are provided at the top and bottom of columns to prevent damage at the rocking surfaces. It is advantageous to install Viscous Dampers (VD's) between the top of walls and upper slab to provide an additional damping mechanism.

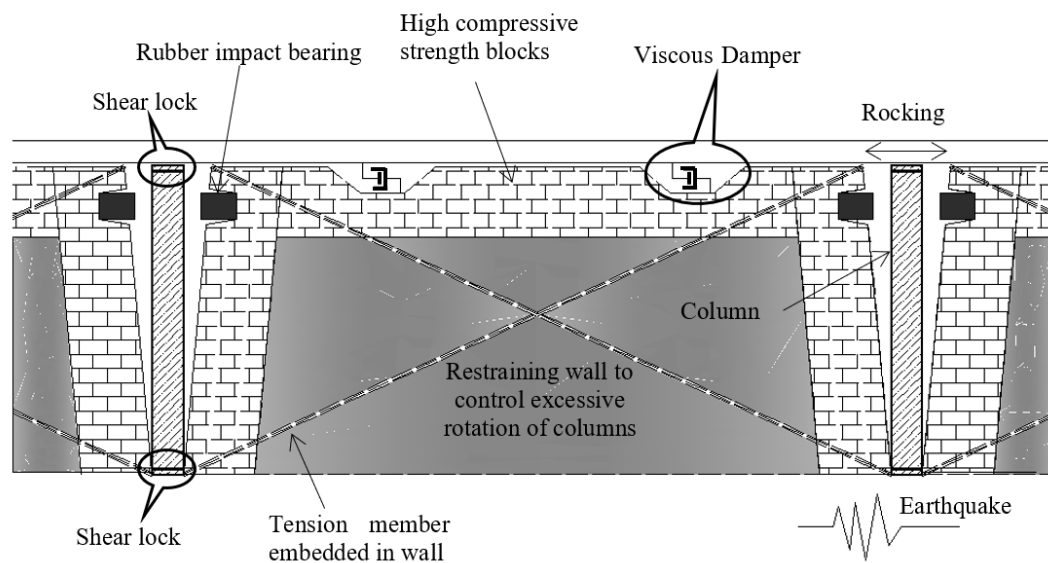


Figure 1.3 Schematic diagram of Rocking-Column (ROC) seismic base isolation system

The walls are reinforced with diagonal tension members embedded within them, and high compressive strength blocks are provided at the both ends and top of each restraining walls to provide additional strength. Rubber impact bearings are embedded within the compression masonry columns to reduce the impact of columns during rocking if large displacement demand occurs. The locations where the construction of restraining wall is not possible, like the interior grid of the structure, an alternative mechanism is proposed as shown in Figure 1.4. This alternative restraining mechanism consists of the extended wings at the upper end of the column. In case of rocking displacement approaching overturning displacement, these wings

will strike the slab and stop the motion. There is always a possibility that the column will come back to its original position, without overturning, after collision of the wing with the roof slab. Rubber impact dampers are installed at the upper sides of the collision wings to reduce the damages.

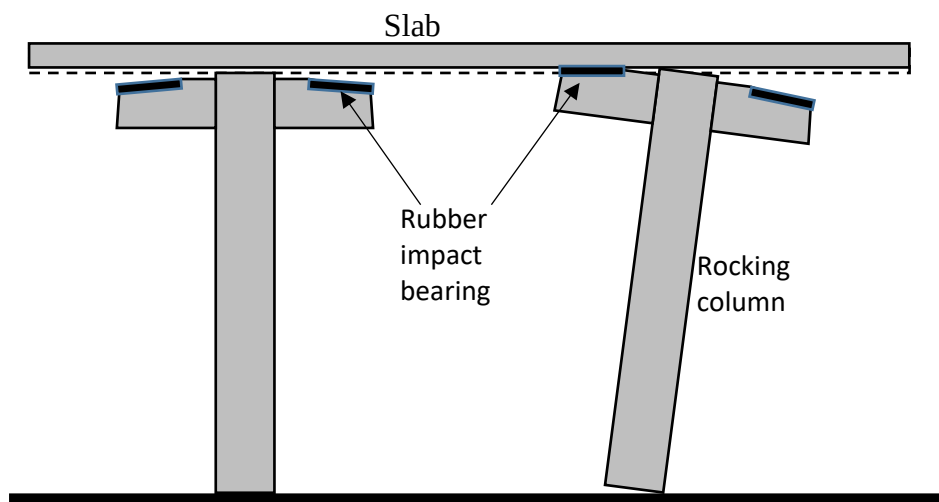


Figure 1.4 Restraining mechanism at the upper end of the rocking column to restrain overturning

The proposed ROC system is capable of isolating the structure via rocking of columns, and it may or may not damp the vibration with the help of VD's. The study is inspired from earthquake response of ancient rocking columns used in Greek Civilization as well as Pagoda structures constructed in Japan, where the first floor or all stories are flexible and behave like isolators. The weight of building and geometry of the columns will provide self-centering and restoring forces. The concept is applied to modern buildings with newly introduced details such as excessive deformation restraints, energy dissipating members, and shear keys. The proposed system may also apply to existing buildings as the first floor is cut and raised to an additional floor. The additional floor may be a solution for parking space problem of up to six floors high and more than twenty years old buildings, commonly faced for neighborhoods. The research is performed in five stages; (i) literature survey, (ii) numerical modeling of rocking column in SAP2000, (iii) experimental validation of the numerical model, (iv) 2D and 3D analyses of the proposed system,

(v) development of guide lines for preliminary design of ROC seismic isolation system. The advantages of the proposed system over conventional base isolation systems are (i) it does not require utility pipes to have deformation compensation parts, (ii) energy dissipating lead core in LRB pads must be replaced after an earthquake but ROC does not require replacement, (iii) may provide additional parking space or one extra floor, and (iv) changing isolated frequency avoiding resonant behavior for earthquakes of large period contents.

1.1 Aim and Objectives

This research aims to develop a new seismic isolator, named as Rocking-Column (ROC) seismic base isolation system for buildings. The development of ROC seismic isolation system has the following primary objectives:

- Isolation of building via rocking of columns
- Applicable to new and existing houses
- Remove necessity of modification for mechanical, electrical, and plumbing (may require flexible joints), in short MEP utility services to be changed after an earthquake.
- Development of the guide for preliminary design of the proposed ROC seismic isolation system.

The application of ROC is possible to existing houses through lifting-up of the structure (see Appendix-B).

1.2 Scope

Scope of the proposed research is as follows:

- Development of a rocking column model in available structural analysis (SAP2000) software.

- Experimental validation of the rocking column model proposed in SAP2000 software
- Analysis of the proposed ROC seismic isolation systems subjected to a group of selected ground motions.
- Preparation of performance graphs for overturning, floor accelerations, and total base shear force for four to seven story high buildings with varying parameters of width and height of the rocking columns, and building inter-column bay distance.

1.3 Research highlights

Concept of rocking column is new and many aspects of the rocking mechanism are not studied yet. The major research highlights of this research are as follows:

1. Comprehensive review of the existing literature regarding rocking of rigid blocks
2. Development of numerical model of the rocking column, which is applicable in SAP2000 software and other finite element software used for building analyses and design.
3. Experimental validation of the rocking model developed for SAP2000 framework.
4. Development of overturning performance graphs, acceleration performance graphs, and base shear force performance graphs, which are applicable for the preliminary design of ROC seismic isolation systems.

CHAPTER 2

LITERATURE REVIEW

Earthquakes may cause loss of lives and severe damages to infrastructure. Due to the factor that many of the earthquake-prone regions are becoming more populated, there is a need to advance the earthquake mitigation techniques for structures, and to provide innovative and promising solutions for this problem. In the past, researchers focused on the development of earthquake-resistant buildings and proposed different strategies. So far, the construction industry has adopted some of these strategies and implemented them on buildings as structural control systems. The author of the thesis has listed these systems into different categories, as presented in Table 1. Although a wide range of research is available about the structural control systems applied to buildings, Table 1 only enlists the eminent ones. Major categories of structural control systems include passive energy dissipation systems, semi-active and active control systems, hybrid systems, and seismic isolation systems¹⁻³.

2.1 Passive energy dissipation systems

In a passive energy dissipation system, an energy dissipation device installed on the structure works without any external energy input. The passive energy dissipation system is the most widely implemented technique in the construction industry, including applications of metallic dampers, friction dampers, viscoelastic dampers (VE), viscous fluid dampers, tuned mass dampers, tuned liquid dampers etc.¹ A significant part of the existing literature consists of articles which focus on exploring additional damping elements and on increasing overall damping in the buildings.

Morales-Beltran and Paul⁴ presented a review article explaining about the implementation of energy dissipation systems to control large scale earthquakes.

Table 1 Structural protective systems against earthquakes¹⁻³

Passive energy dissipation	Semi-active and active control	Hybrid system
Metallic dampers ^{1,2}	Active bracing systems ¹	Hybrid mass damper ^{1,2}
Friction dampers ^{1,2}	Active mass dampers ¹	Hybrid base – isolation system ²
Viscoelastic dampers ^{1,2}	Variable stiffness or damping systems ¹⁻³	Hybrid damper – actuator bracing control ²
Viscous fluid dampers ¹	Variable friction device ^{2,3}	Active base isolation with semi-active MR dampers ³
Tuned mass dampers ¹⁻³	Variable orifice damper ²	
Tuned liquid dampers ^{1,2}	Controllable tuned liquid damper ²	
Tuned liquid column damper ³	Semi-active impact damper ²	
Tuned liquid column ball damper ³	Controllable tuned mass damper ^{2,3}	
Circular tuned liquid column damper ³	Smart materials ¹	
Pendulum tuned liquid column damper ³	Magnetorheological (MR) dampers ³	
Pressurized fluid damper ²	Negative stiffness devices ³	
Preloaded spring-friction damper ²	Magneto-rheological tuned mass damper ³	
	Active bracing system ²	

2.1.1 Tuned mass damper

Tuned mass dampers (TMD's) are the supplementary masses, tuned at specific frequencies and are attached to the main structure. A schematic diagram of single TMD installed on a SDOF structure is shown in Figure 2.2.

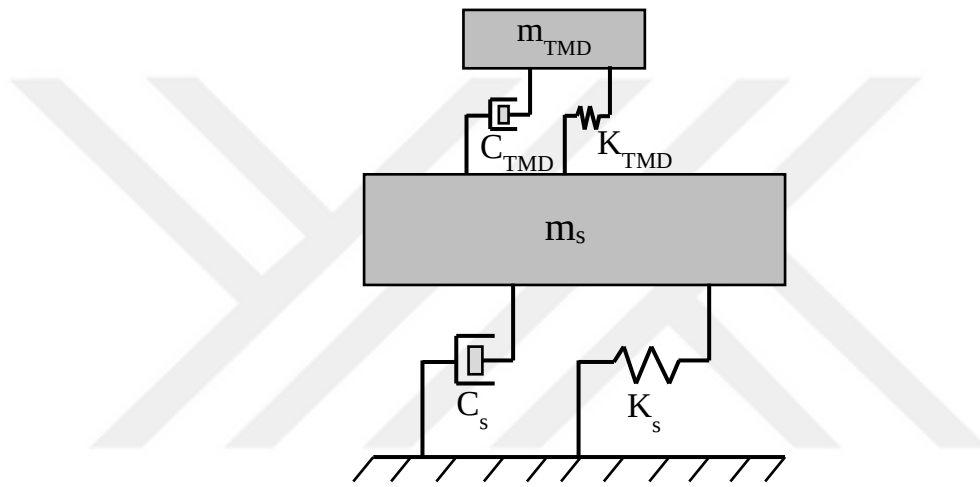


Figure 2.1 Analytical model for tuned mass damper

Tuned mass damper (TMS) is one of the well-investigated topics in earthquake engineering. The TMD's have applications on buildings as well as on bridges. Soto and Adeli⁵ have presented a brief review of TMD and its further modifications like pendulum TMD (PTMD), bidirectional TMD (BTMD), and tuned liquid column damper (TLCD). They have also presented the use of TMD in chronological order from 1976 to 2012 in a tabular form. The primary purpose of designing a TMD in a high-rise building is to control structural vibrations due to wind loading, and it can also perform well in equivalent earthquake excitation. TMD's are usually designed for specifically tuned frequency and perform well against wind loading and moderate earthquakes for high rise buildings. Still, the performance of TMD's is questionable under wide frequency range especially when it is subjected to strong ground motions

since other modal frequencies are highly excited. To overcome this problem and to increase the robustness of the TMD's, researchers⁶⁻⁸ have performed the studies on particle tuned mass dampers, tuned liquid-column dampers, multi-unit particle dampers, and tuned liquid-particle dampers. Additionally, the shift of natural frequencies caused by damage during large earthquakes would leave TMD's out of tune. The out of tune problem can be overcome in Rocking-Column (ROC) seismic base isolation system because ROC is not tuned at a specific frequency and can perform well under a wide range of frequencies.

2.1.1.1 Advanced versions of Tuned Mass Dampers (TMD's)

Tuned mass dampers perform well at specifically tuned frequency, but the performance of TMD's is questionable under a wide frequency range of ground excitation. The use of two TMD's tuned against two control modes and installed perpendicular to each other may perform well during biaxial earthquake excitation, and it also reduces the risk of the detuning effect.⁹ Lu et al.⁶ have proposed an improved version of TMD in which TMD is combined with particle damper and named as particle tuned mass damper (PTMD). Experimental results have shown that PTMD's have performed well under different earthquake excitations showing the robustness of the system. Carmona et al.¹⁰ have designed the TMD with a friction mechanism to dissipate energy. Zhu et al.⁷ have studied the effectiveness of tuned liquid column dampers (TLCD's) installed on a nine-story steel building under three different earthquake excitations. The performance of multiple TLCD's is better than that of single TLCD to control multi-order modal responses. Tuned liquid particle damper (TLPD), which is the integration of tuned liquid damper and particle damper, may increase the effectiveness and robustness of the system.⁸

2.1.2 Particle damper or impact damper

A particle damper is a passive energy dissipation device used for vibration control of buildings in which particles dissipate energy as a result of an impact. The particle damper is also known as the impact damper. A particle damper consists of a container attached with the primary structure and filled with a particle or particles free to move under vibration. The particle damper dissipates energy through the collision of particles with each other and with the walls of the container. In a single-unit particle damper, the impact of a single massive particle is enormous, and it may cause deterioration of the material and may produce noise. Therefore, a multi-unit particle damper may be proposed instead of a single-unit particle damper.¹¹ These dampers perform well for lightly damped primary structures at the cost of a small increase in weight. As the behavior of particle damper is highly nonlinear and complex physics involved in the particle collision, the researchers¹¹ have proposed simplified models for analyses of multi-unit particle dampers.

2.1.3 Hysteretic dampers

A hysteretic metallic damper is a type of passive energy dissipation devices which provides additional stiffness, additional damping due to yielding behavior in case of strong earthquakes, and controls deformations. Hence in this way, these devices minimize energy absorption and reduce the cost of the main structure.¹² Skinner et al.¹² introduced the concept that the energy dissipation function of supplementary energy dissipation devices could be considered separately from the load function. They studied the plastic deformation of a short beam element under bending as an energy dissipation mechanism. They provided a platform for the coming researchers to develop various types of hysteretic metallic dampers. Skinner et al.¹² have proposed a tapered round shape hysteretic steel damper for use in base isolation along with rubber pads. Desirable features of metallic yield dampers are; a) stable hysteretic behavior, b) improved low-cycle fatigue property, c) long term reliability,

d) relative insensitivity to environmental temperature, e) and bending moments produce a uniform flexural stress distribution over the height of the damper.^{1,13,14} The disadvantages associated with these dampers are; a) the nonlinear behavior that may require nonlinear analysis, b) low deformation capacity, and c) devices that may need a replacement after a major earthquake.^{14,15} It has been more than five decades; many researchers have developed and used hysteretic type metallic dampers for seismic retrofitting of existing buildings and installation in new buildings. From these researches, it is concluded that the hysteretic metallic damper, which yields over the whole length, bears better energy dissipation capacity and it can be achieved by optimizing the shape of these dampers similar to an hourglass shape.

A preliminary and comprehensive description of hysteresis damper used for earthquake-resilient structures was presented by Skinner et al.¹² They introduced four different types of hysteretic dampers; a) U-strips damper, b) torsional beam damper, c) flexural beam damper, and d) single-axis damper, and recommended the use of hysteresis dampers for energy dissipation in structures.

Friction damper is another hysteretic damper used to dissipate energy due to friction between two sliding surfaces. The amount of friction produced between two sliding surfaces follows the Coulomb's law of friction, which states that frictional force is directly proportional to the force applied normal to the sliding surfaces. In a friction damper, the clamping mechanism is used to control the normal force and the amount of energy dissipated. The magnitude of normal force along with the coefficient of friction dictates the state of stick or slip of the system under an earthquake which is an essential parameter of friction damper. The larger the clamping force, stronger earthquake force is required for surfaces to slip and activate the damper, and vice versa.

2.2 Semi-active and active control

Semi-active and active control systems are the passive control systems with partial or full control mechanisms. The effectiveness of the passive control system may be enhanced by controlling its properties intelligently. The active structural control system consists of three parts: a) sensors installed on buildings/structures and on the free field; b) devices to collect information from these sensors (datalogger), to process the data and define the required force to be applied on the structure to reduce vibration (an extensive algorithm); c) actuators which apply force to structures. The semi-active control system does not supply direct mechanical energy to the controlled structural part, but it provides indirect energy which is smaller in magnitude. Hence, it requires less power and can be operated using batteries.¹⁶ Most of the active and semi-active control systems are used for moderate scale earthquakes and strong wind loading conditions. Semi-active dampers require less space as these can be hidden within the walls where more space is available.⁴

Researchers have developed different active and semi-active control mechanisms installed with the passive control systems. In most of the cases, these are the developments of passive control systems. For example, the problem associated with passive friction damper is that a threshold value of earthquake force is necessary to trigger the sliding of the damper; otherwise, the damper will not activate. To overcome this problem, Lu¹⁷ has suggested a semi-active control of friction damper such that the damper activates in moderate and large earthquakes and dissipates energy. Chu et al.¹⁸ have developed leverage type stiffness controllable tuned mass damper (a semi-active tuned mass damper) having lever mechanism. The pivoted lever arm is attached with tuned mass damper at one end and with the main structure at the other end having a changeable position of pivot.

2.3 Hybrid system

A hybrid control system is the combination of passive and active control or semi-active control systems to use less input energy as compared with the active or semi-active control system and shift to the basic passive system in case of power shutdown. Some of the hybrid control systems are reported in the literature; e.g., hybrid mass damper, hybrid-base isolation system, controllable dampers, and hybrid damper – actuator bracing control system.^{1–3,16}

2.4 Conventional seismic base isolation

Seismic base isolation is a widely implemented passive structural control technique. The basic concept in seismic base isolation is to isolate the structure from harmful waves of earthquakes, and it is achieved by shifting the natural vibration period of the structures with high mass participation ratios, from high response (resonance) region to low response region as shown in a typical response spectra curve in Figure 2.2.

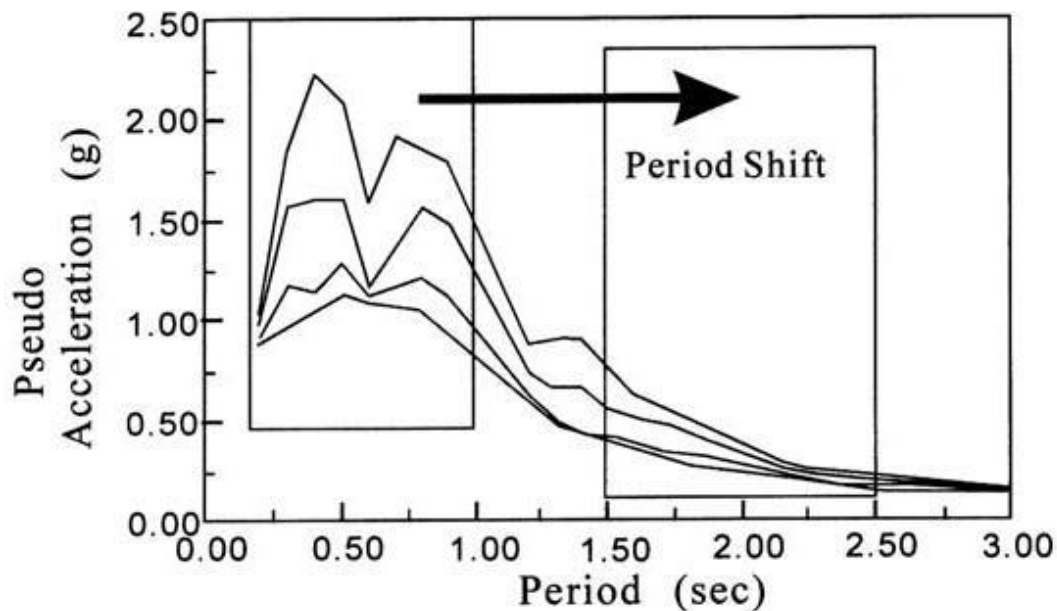


Figure 2.2 Typical response spectra curve¹⁹

Some of the base isolation techniques reported in the literature are listed in Table 2.

Table 2 List of seismic base isolation techniques

Authors	Soong and Spencer ¹ (2002)	Saeed et al. ² (2015)	Harvey and Kelly ²⁰ (2016)	Warn and Ryan ²¹ (2012)	Turer and Özden ²² (2008)	El-Khoury and Adeli ³ (2013)
Seismic isolation						
Elastomeric bearing	x	x				
Laminated rubber bearing				x		
Lead rubber bearing	x	x		x		
Sliding friction pendulum	x	x		x		
Combined elastomeric and sliding bearing		x				
Rolling type seismic isolator			x			
Scrap automobile rubber pad bearing					x	
Active base isolation		x				
Shape memory alloy (SMA) rubber bearing isolation system						x

One of the earliest passive structural control technologies for building isolation is the use of rollers, and some researchers in the past recommend it; but, the disadvantage associated with the use of rollers as a seismic isolator is the excessive displacement which is challenging to control. Harvey and Kelly²⁰ have reviewed the use of

cylindrical rollers and spherical rollers as a seismic isolator for buildings, bridges and equipment, with or without supplementary damping. For rolling type seismic base isolation, it is crucial to increase the seismic displacement capacity of rollers. Elastomeric bearing is the natural or synthetic rubber bearing, whose vertical stiffness is higher than its horizontal stiffness. This feature makes the elastomeric bearing flexible in the horizontal direction and hence increases the period of the structure when used as seismic base isolation. Laminated rubber bearing is a type of bearing in which steel layers are embedded in between the rubber layers to increase the vertical stiffness while keeping horizontal stiffness low. Lead rubber bearing is a rubber bearing having a lead plug (99.9% pure lead) core at the center and the core can deform in shear during an earthquake. Lead rubber bearings exhibit a larger enclosed area of hysteresis loop as compared with that of rubber bearings, indicating larger energy dissipation capacity. As lead rubber bearings are less affected by the stroke frequency; therefore, these may perform well under strong earthquakes.²³

Figure 2.3 shows the deformed configuration of laminated rubber bearing (left) and a typical cross-sectional view of lead rubber bearing (right). Sliding friction pendulum is a base isolation system that slides in case of vibrations and has the advantage of re-centering itself. The sliding friction pendulum system consists of an articulated slider, spherical concave surface, sliding interface, and base plate, as shown in Figure 2.4. A comprehensive review of the seismic base isolation systems with primary focus on low-damping rubber bearing, high damping rubber bearing, lead rubber bearing, and sliding friction pendulum is presented by Warn and Ryan.²¹ They have also reviewed small-scale and full-scale laboratory experiments of different types of bearings.²¹

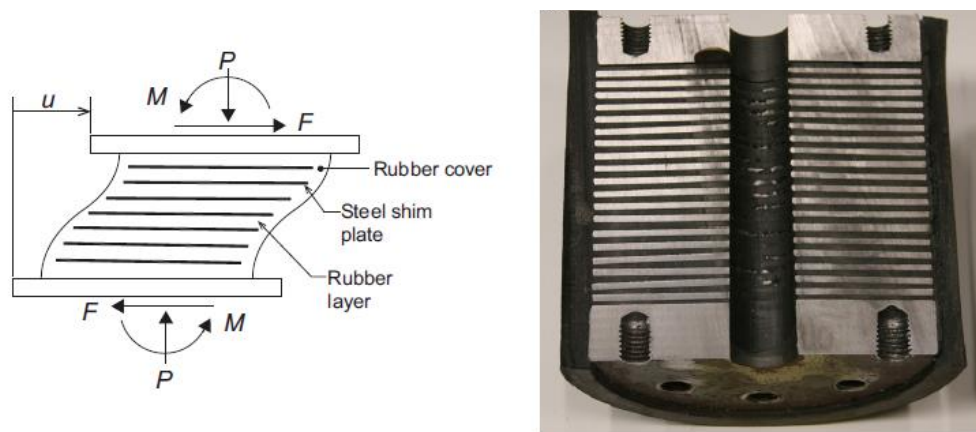


Figure 2.3 Laminated rubber bearing in horizontally deformed configuration (on the left) and lead rubber bearing (on the right), adapted from Wam and Ryan²¹

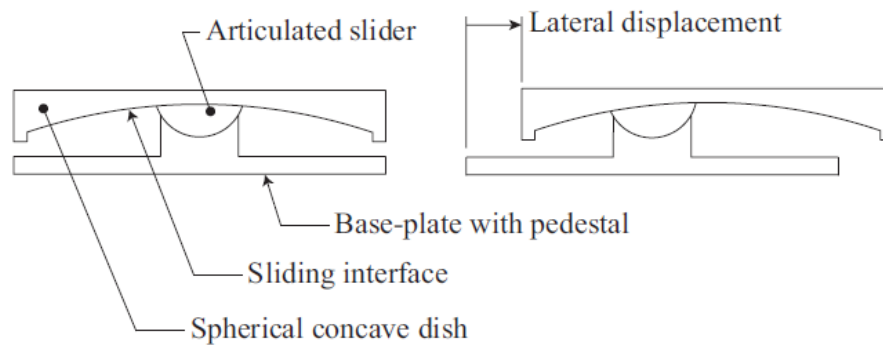


Figure 2.4 Schematic diagram of sliding friction pendulum bearing, adapted from Wam and Ryan²¹

The isolators are made flexible in a horizontal direction to increase the effectiveness of seismic base isolators; however, it may make seismically isolated structures vulnerable to pounding. The effect of pounding is more pronounced, especially at the level of the isolated floor, and it may be controlled by providing a seismic gap. However, the provision of the seismic gap does not ensure that superstructure will not pound with adjacent buildings during earthquakes.^{24,25} Therefore, the effect of pounding in seismically isolated buildings should be addressed at the design stage. In Japan, the flexible stiff model is becoming an appealing concept for seismic base isolation systems, having stiff parts as energy dissipaters (dampers) and flexible parts as isolators. A flexible-stiff model is presented by Ene et al.²⁶ (see Figure 2.5) in

which a U-shaped metallic damper is used along with rubber bearing to prepare a flexible-stiff seismic base isolation system.

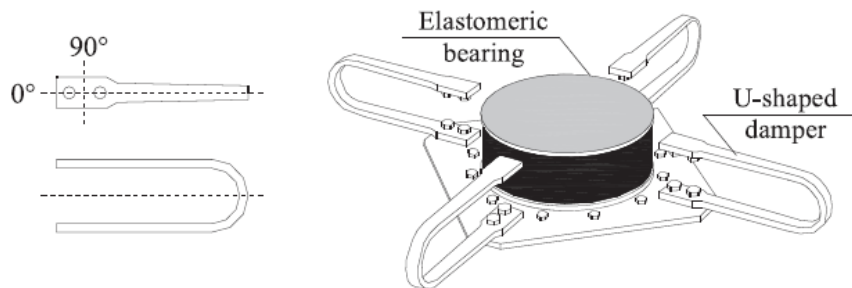


Figure 2.5 Flexural-stiff model for seismic base isolation²⁶

Another passive control base isolation technique is introduced by Turer and Özden²², in which the scrap automobile rubber pads are used to isolate the structures. Waste automobile rubber tires may be used as low-technology and no-cost seismic base isolation systems for small housing units. Waste automobile rubber tire base isolation is made up of tread of waste rubber tires cut into strips having a length in multiples of its width and placed on top of each other in different patterns. Compression test, reversed cyclic test, and inclined lateral test are performed in the laboratory and results have encouraged the use of waste automobile rubber tires as a seismic base isolator.

The torsional response of base isolated structure may cause additional stresses if the building is irregular in shape. The torsional response of base isolated system can be neglected if the center of stiffness (CS) and plasticity centroid (CP) of the base isolation device is directly under the center of mass (CM) of the superstructure.¹⁶ New technologies for torsion control using base isolation and energy dissipation devices are also reviewed by Anagnostopoulos et al.¹⁶ Some researchers have reported the use of smart materials like shape memory alloys (SMA's) for seismic base isolation.^{3,19}

2.5 Rocking type base isolation

Ancient Greeks had implemented the idea of rocking columns in their buildings to survive during many earthquakes. The concept of the rocking column has attracted a few researchers' interests, and they have tried to implement this concept for earthquake mitigation of buildings. Rocking is a mechanism (shown in Figure 2.6) in which columns start rocking in case of earthquakes and isolate the superstructure from damaging waves of earthquakes.

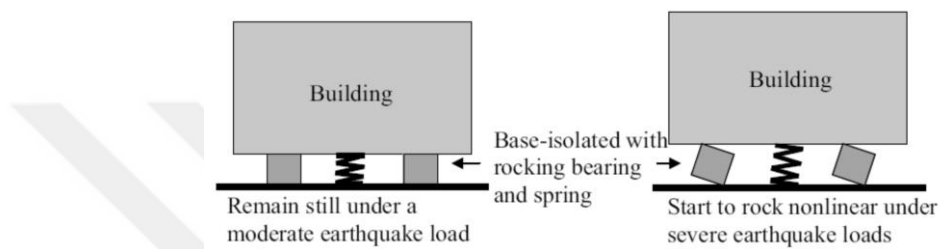


Figure 2.6 The basic function of rocking base-isolated structure, adapted from Cheng and Chao²⁷

Freddi et al.²⁸ have designed the rocking steel column base anchored at the bottom with post-tensioned steel bars and friction dampers, as shown in Figure 2.7.

“Rocking type” base isolator gives the advantage to avoid the resonance like response when subject to strong ground motion having long periods. The disadvantage of rocking type base isolators is the danger of overturning in case of strong ground excitation and excessive slip, and these may be prevented by providing proper clamping and shear keys, respectively. The primary function of any base isolation system is to isolate the superstructure and to provide sufficient damping in case of an earthquake. Rocking type base isolators not only isolate the superstructure but also dissipate some energy whenever its surfaces hit the base at the footing. The energy lost in this system is due to the rocking impact only which is insufficient in some cases and may require additional energy damping mechanisms. Energy dissipation during each collision may be calculated analytically assuming superstructure as a rigid body before and after rocking and applying the conservation of momentum. In another research, the concept of the rocking base is applied in self-

centering moment resisting frames (SC-MRF) and it is assumed that the proposed rocking base of steel columns would control the damage (yield) in case of earthquakes. A diagram of the rocking column base isolator is shown in Figure 2.7. Ad hoc post-tensioned steel bars and friction plates are integrated with the rocking base to enhance its performance.²⁸

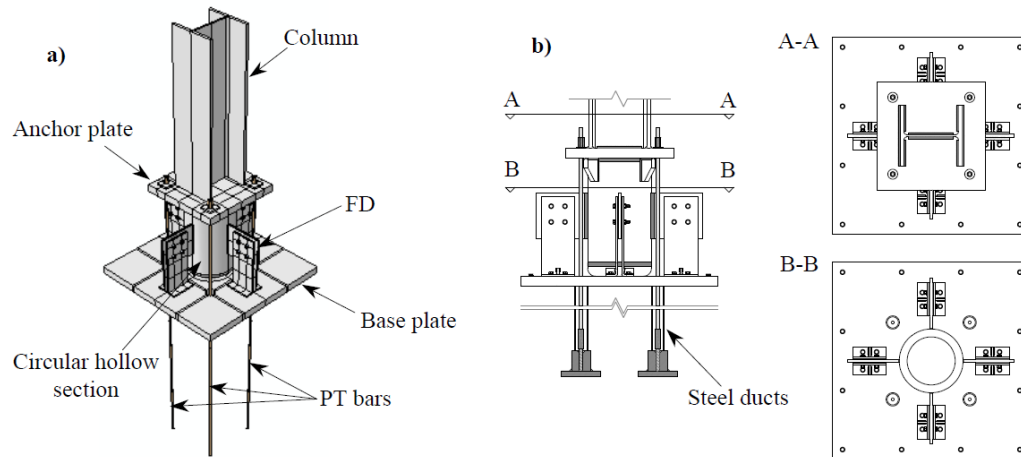


Figure 2.7 Rocking column base isolator, adapted from Freddi et al.²⁸

A parametric study investigating the role of the rocking surface geometry on the response of the structure is performed using the plane, spherical and polynomial surfaces. Results have shown the better performance of the polynomial surface. The plane surface shows negative stiffness after the start of the rocking, but the spherical and polynomial rocking surfaces show an increase in the stiffness after the rocking is started.²⁷

2.6 Concept of rocking rigid blocks

The dynamic behavior of rigid-block structure resting on a rigid foundation and subjected to ground excitation has been a matter of investigation for many years.²⁹ Perhaps rocking is the most regularly conceived dynamic behavior of ancient structures like Greek and Roman columns, obelisks, tombstones, etc. Furthermore, concrete blocks used in nuclear reactors for radioactive shielding, heavy equipment

installed in factory or laboratory, and historic masonry walls subjected to earthquake can be better explained with the theory of rocking rigid blocks.^{30–33} Over the last two decades, researchers have paid more attention to the rocking of rigid blocks, its theoretical and experimental advancements, and its applications in civil engineering, as is evident from the increasing number of publications in Figure 2.8. Housner³⁴ proposed the theory of rocking blocks to investigate the survival (resistance against overturning) of tall and slender structures. In his seminal work, he explored the fact that many apparently unstable (slender and tall) water tanks sustained through the Chilean earthquake 1960. These water tanks were thought to be more vulnerable from a static point of view but remained undamaged during dynamic loading (earthquake). It leads him to the conclusion that simple static analysis for establishing overturning criteria under dynamic loading is not enough and requires a dynamic investigation. This observation motivated Housner to study rocking of inverted pendulum type rigid blocks from a dynamic point of view and to publish one of the pioneer works in 1963. When a rigid block is placed on the rigid ground and subjected to base excitation, six types of motion are possible: (1) rest, (2) slide, (3) rotate, (4) slide-rotate, (5) translation-jump, and (6) rotation-jump. The necessary conditions for the activation of each type of motion are reported in Ishiyama.³⁵ If sliding and rocking both are possible, then the rocking will occur when the coefficient of static friction (μ_s) is greater than the slenderness ratio (width to height ratio, b/h ; Figure 2a), otherwise sliding prevails. The block will slide if $a_g^x > \mu_s g (1 + a_g^y/g)$; where a_g^x and a_g^y are the horizontal and vertical components of ground acceleration as shown in Figure 2.9(a). Ignoring the vertical component of the ground acceleration a_g^y , the above rocking condition is simplified as $\mu_s < b/h$ or $h/b < 1/\mu_s$. The important factors for sliding, rocking, overturning or remaining standing of a rigid block under ground motion are (a) the static coefficient of friction (μ_s) between the slab/floor and rigid block, (b) the slenderness (ratio b/h), where $2b$ is the width and $2h$ is the height of the rigid rectangular block, (c) the size R , where $2R$ is the diagonal length of the rigid block, (d) the moment inertia of rigid block, and (e) the magnitude and form of earthquake excitation.^{30,34–39} Even if pure rocking conditions

are satisfied, Kounadis⁴⁰ argues that the sliding-rocking mode of motion is possible in a real scenario, in which sliding occurs for a very short interval of time just after the initiation of rocking. The rocking of rigid blocks is a different dynamic problem as compared to conventional systems due to change in its basic characteristics; hence, it cannot be treated as conventional nonlinear systems and more research is required to develop equivalence between rocking block system and conventional dynamic system. The behavior of rocking of a rigid block on a rigid surface is nonlinear due to the following two reasons: (1) it is nonlinear due to the trigonometric terms involved in the equations of motion (see Equation 1) which can be linearized for slender blocks (see Equation 2), and (2) it is nonlinear due to switching between alternative equations as a result of rocking about alternative edges of rocking blocks. The nonlinear equations of motion are approximated as piecewise linear equations of motions and piecewise linear models of nonlinear behavior of rocking rigid blocks are also developed.^{29,39,41}

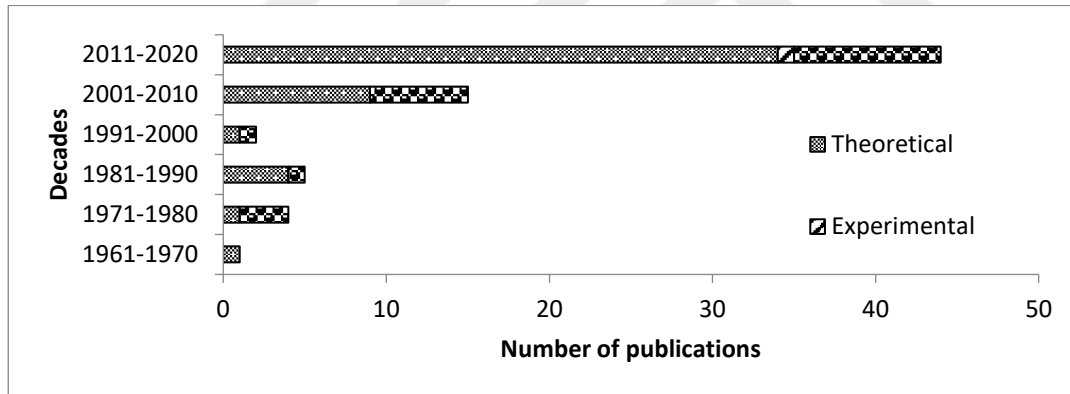


Figure 2.8 Number of publications regarding rocking of rigid blocks over the last six decades

2.7 Characteristics of Rocking Rigid Blocks

The rocking of a rigid rectangular block having width ' $2b$ ' and height ' $2h$ ' is shown in Figure 2.9(a). The block is rotated clockwise through an angle ' θ ' about the pivot point ' O '. The slenderness of the block is defined in terms of angle ' α ' in radians, which is the angle between the diagonal and side of the block. The length of the

diagonal '2R' represents the size of the block. The weight of the block is 'W', and 'cg' is the center of gravity of the block. And $\varphi_o = \theta_o/\alpha$ is the ratio of the initial rotational amplitude of vibration ' θ_o ' to slenderness.

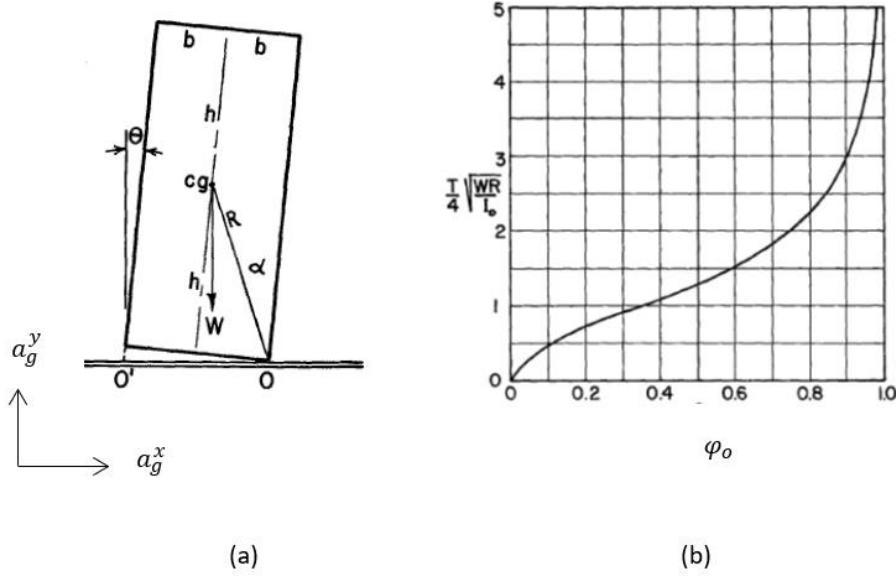


Figure 2.9 (a) Rocking of a rigid block and (b) variation of the rocking period 'T' with vibration amplitude ' φ_o ' (modified from Housner³⁴)

Housner³⁴ investigated the dynamics of the rocking block based upon the three assumptions: (1) the rocking block and the ground are rigid, (2) the coefficient of friction between the surfaces of the block and ground is so high that it does not allow any slip during motion, and (3) collision between the ground and block is inelastic so that it does not bounce during rocking. Based upon the above assumptions, $a_g^x > \frac{b}{h} g(1 + a_g^y/g)$ is the necessary condition to start rocking of the rigid block, under the horizontal (a_g^x) and vertical (a_g^y) components of ground excitation.^{30,39} If ' I_0 ' is the moment of inertia about pivots (either O or O'), the motion of free rocking rigid block about corners is defined by Equation 1.³⁹ The minus sign (-) is applicable for rocking about corner 'O' and the plus sign (+) is applicable for rocking about corner 'O'.

Equation 1

$$I_o \ddot{\theta} + W \left(1 + \frac{a_g^y(t)}{g} \right) R \sin(\alpha - \theta) = -\frac{W}{g} R \cos(\alpha \mp \theta) a_g^x(t)$$

There is a debate in the literature, about the inclusion of the vertical component of ground acceleration (a_g^y) into the equations of motion for rigid blocks. The probability of a block overturning may increase or decrease by including the vertical component of ground motion; however, it leads towards more exceptions to the general trends of rocking behavior.^{39,42} According to Aslam et al.³⁰, the vertical component of earthquake ground motion is essential because downward earthquake acceleration may reduce the effective weight of the rigid block hence reducing the frictional force. Effective weight (W_e) of the block is defined as $W_e = W(1 + a_g^y/g)$. On the other hand, Makris and Kambas³⁷ debated that the effect of inclusion of the vertical component of ground excitation is marginal due to the following three reasons: (a) vertical component of the ground acceleration is multiplied with the sine term (having value approaches to zero for high slenderness ratio) in the equation of motion, (b) frequency of the vertical component of ground acceleration is usually higher hence more contribution from the rotational moment of inertia is observed and more stable is the block, and (c) vertical component of the ground motion is usually smaller as compared to the horizontal component of ground motion. Due to the above mentioned reasons, majority of the researchers have not included the effects of the vertical component of ground acceleration, which is evident in Figure 2.10.

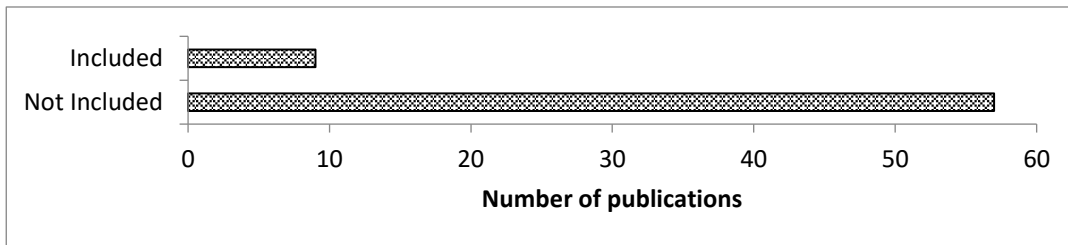


Figure 2.10 Number of publications included and not included the vertical component of the ground acceleration

In general, the rocking of rigid blocks is less affected by the vertical component of the ground acceleration a_g^y . Moreover, it may be ignored for the sake of simplicity in calculations, but the readers should keep in mind that this may cause minor variations in the results. For tall and slender blocks (usually, $\alpha < 20^\circ$, $\sin(\alpha-\theta) \approx \alpha-\theta$), and ignoring the vertical component of ground excitation, Equation 1 can be simplified into linearized form as described in Equation 2.³⁴ Equation 2 is the equation of motion of free rocking rigid block about corner ‘O’, and ‘ p ’ is the frequency parameter defined as $p^2 = \frac{WR}{I_o}$. For initial conditions; $\theta = \theta_o$ and $\dot{\theta} = 0$ at $t = 0$, the closed-form solution of Equation 2 is defined in Equation 3.³⁴

Equation 2

$$\ddot{\theta} - p^2\theta = -p^2\alpha$$

Equation 3

$$\theta = \alpha - (\alpha - \theta_o) \cosh pt$$

Equations 2 and 3 are valid for half vibration only, i.e., from disturbed position of rigid block (θ_o) to the mean rest (vertical) position. At the mean rest position, right after the impact, the pivot of rocking will shift from O to O', and the sign of angle ‘ θ ’ will change in the Equations 2 and 3. At the instant when the block will fall from $\theta = \theta_o$ to $\theta = 0$; Equation 3 will yield to Equation 4. In Equation 4, φ_o is defined as $\varphi_o = \theta_o/\alpha$.

Equation 4

$$T = \frac{4}{p} \cosh^{-1} \left(\frac{1}{1 - \frac{\theta_o}{\alpha}} \right) = \frac{4}{p} \cosh^{-1} \left(\frac{1}{1 - \varphi_o} \right)$$

It is evident from Equation 4 that period ‘T’ of the free rocking block is dependent upon the ratio of the initial amplitude of vibration to slenderness ratio (φ_o) and frequency parameter (p). The period of vibration of the rocking block changes due to the change in amplitude of vibration after each impact; therefore, discrete natural

period (or frequency) of a free rocking rigid block does not exist. The variation of the period 'T' with respect to the amplitude of vibration is shown in Figure 2.1(b), where 'W' weight, 'R' radius $\sqrt{h^2 + b^2}$, ' I_0 ' moment of inertia, ' α ' slenderness, and ' φ_0 ' ratio of initial rotation to slenderness ratio was previously defined. The only energy dissipation during rocking of rigid blocks is through the impact between the block and ground. If the impact is assumed to be inelastic, then the energy dissipation can be explained with the help of the coefficient of restitution 'r'. In the classical theory of rocking blocks as defined by Housner,³⁴ the coefficient of restitution is derived using conservation of momentum just before and after the impact. Figure 2.11(a) shows the relationship between the coefficient of restitution and the slenderness ratio. In reality, the coefficient of restitution is also dependent upon the types and materials of contact surfaces, and angular velocity before impact. However, in Housner's model of the rigid rocking block, the coefficient of restitution is independent of these factors and considered as constant.^{30,39} After each impact, the energy remained in the block will be equal to 'r' times energy in the block before impact. Using $\varphi = \theta/\alpha$ and $\varphi_o = \theta_o/\alpha$, the rotation angle amplitude of the block after the n^{th} impact is written in Equation 5 as defined by Housner.³⁴

Equation 5

$$\varphi_n = 1 - \sqrt{1 - r^n [1 - (1 - \varphi_o)^2]}$$

The behavior of the rocking block is similar to the rigid-plastic model to some extent, with a difference of second slope and hysteresis behavior. The moment-rotation characteristics of freely rocking block have a negative slope (-WR) with a large discontinuity at the zero-rotation position, as shown in Figure 2.11(b). In Figure 2.11(b), the definition of θ_c is the same as α in Figure 2.9(a). Beyond points θ_c or $-\theta_c$, the block is assumed to be overturned under static conditions, but under dynamic conditions, experimental results have shown that the block may rock slightly beyond the critical angle θ_c .³⁸ The rocking mechanism does not show any hysteresis curve (dissipation of energy), and only energy dissipated during rocking

of the rigid block is at the instant of impact.^{30,39} For out of plane rocking of masonry walls, three branch moment rotation relationship (instead of two branch moment-rotation relationship as shown in Figure 2.11b) has shown a good agreement with the experimental results and it can be defined into three zones: 1) pre-rocking; 2) rocking; and 3) overturning.^{32,43–45}

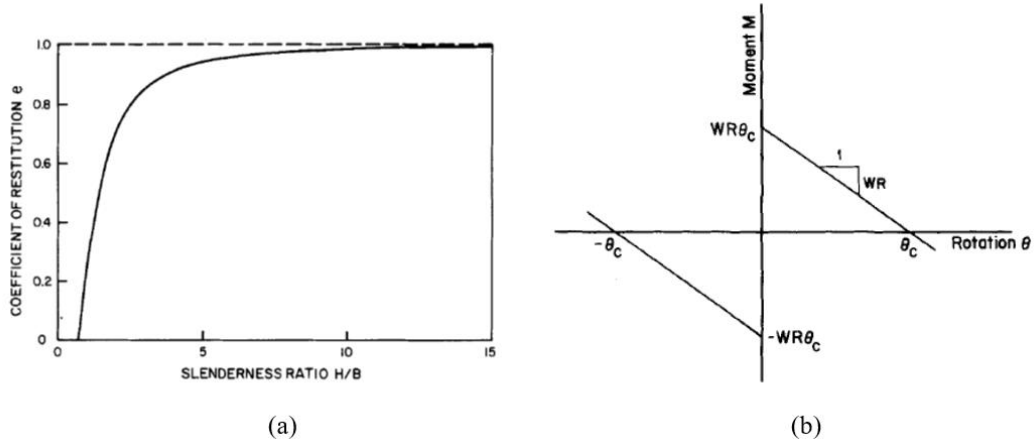


Figure 2.11 (a) Coefficient of restitution vs slenderness ratio, and (b) moment-rotation relation for slender block³⁹

2.8 Rocking of Rigid Blocks Subjected to Ground Excitations

2.8.1 Rocking of Rigid Blocks Subjected to Harmonic Excitations

The uplift or rocking behavior of a rigid-block under harmonic excitation is nonlinear. Spanos and Koh²⁹ have studied the rigid blocks resting on the rigid ground and subjected to harmonic ground excitations and concluded that rocking rigid block becomes more robust against toppling when subjected to high-frequency harmonic excitation as compared with low-frequency harmonic excitation. Ishiyama³⁵ has developed the overturning conditions keeping acceleration constant but varying excitation frequencies. In contrast, Spanos and Koh²⁹ were able to develop safe and unsafe regions on the excitation frequency vs excitation amplitude plane. Pena et al.³⁸ have performed the numerical as well as experimental studies on rocking rigid

blocks subjected to harmonic excitation. Steady-state response of rocking block under moderate harmonic excitations is possible, and all in-phase steady-state responses concerning harmonic base excitation are unstable and either leads towards out-of-phase response or toppling over of the rigid block. The only stable responses are from the out-of-phase steady-state responses with respect to harmonic base excitation.^{29,41} All in-phase vibrations will eventually become out-of-phase stable responses or will become unstable and overturn staying in-phase.

2.8.2 Rocking of Rigid Blocks Subjected to Pulse Excitations

The study of rocking rigid block, subjected to pulse excitations, has been a matter of interest for researchers due to the reason that pulse excitation possesses some earthquake-like characteristics.⁴⁶⁻⁴⁹ The rocking response of a rigid block rested on a ground subjected to a sudden rectangular ground acceleration of magnitude ' α ' for the small duration ' t_1 ' is defined with the help of Equation 6 and its solution for initial rest conditions is written in Equation 7.³⁴

Equation 6

$$I_o \ddot{\theta} - WR\theta = \frac{WRa}{g} - WR\alpha$$

Equation 7

$$\frac{\theta}{\alpha} = \left(\frac{a\alpha}{g} - 1 \right) (\cosh pt - 1)$$

The condition of overturning can be established by equating the work done by the inertial force Wa/g equal to the difference in potential energy from positions $\theta = \alpha$ to $\theta = 0$. Time duration ' t_1 ' required for the constant acceleration ' a ' to overturn the rigid block is estimated and presented in Table 1. Recorded data from accelerograms has shown that in some cases, earthquake ground excitation can be represented by one wave of sine pulse, $a \sin(\omega t + \psi)$, and the solution of rocking problem

subjected to this sine pulse with initial rest conditions is given in Equation 8 as defined by Housner.³⁴

Equation 8

$$\frac{\theta - \alpha}{\alpha} = \frac{1}{1 + \left(\frac{\omega}{p}\right)^2} \left[\frac{\omega \sinh pt}{p \tan \psi} - \left(\frac{\omega}{p}\right)^2 \cosh pt - \frac{\sin(\omega t + \psi)}{\sin \psi} \right]$$

In Equation 8, $\sin \psi = g\alpha/a$ and $\cos \psi = \sqrt{1 - (g\alpha/a)^2}$. Overturning condition for rigid block subjected to sine wave pulse excitation, $a \sin(\omega t + \psi)$, is that $\theta = 0$ at the instant when $t = (\pi - \psi)/\omega$. It is a necessary condition to ensure that the block reaches its maximum amplitude (about to overturn) just at the moment when ground acceleration completes its first half cycle.³⁴ When a rigid block is subjected to half sine-wave pulse ground excitation, and $\omega/p > 3$, the condition of overturning is defined in Table 3.³⁰ Where $\omega = 2\pi/T_s$ and $p = \sqrt{3g/4R}$. It is clear from the condition of overturning of the rigid block (as defined in Table 3) that the stability of rigid block for a half sine-wave pulse increases with an increase in size and decrease in slenderness ratio.

Table 3 Condition to overturn a rigid block under different types of pulse excitations^{30,34,39}

Type of pulse excitation	Condition to overturn
Constant acceleration pulse	$\cosh \sqrt{\frac{WR}{I_o}} t_1 = 1 + \frac{1}{2 \frac{a}{g\alpha} \left(\frac{a}{g\alpha} - 1 \right)}$
Half sine-wave pulse	$\frac{aT}{2} > 2\pi\alpha \sqrt{\frac{Rg}{3}}; \text{ if } \omega/p > 3$
Earthquake like series of pulses	$\alpha < \frac{S_v}{\sqrt{gR}}$

Dimitrakopoulos and DeJong⁵⁰ have proposed a method for directly developing the overturning plots without repeating simulations for different parameters. Like classical work on rocking blocks, Masi et al.⁵¹ proposed an analytical solution for rocking motion under blast loading. They developed the design charts to determine/calculate standoff distance against overturning. Bachmann et al.⁵² have studied the rolling and rocking phenomena of rigid blocks subjected to ricker pulse by widening its flat base using curved flanges.

2.8.3 Rocking of Rigid Blocks Subjected to Earthquake Excitations

A single-pulse solution of rocking rigid block is of little importance because actual ground motion consists of a series of pulses. Therefore, there is a possibility that during an earthquake, a rigid block overturns at a smaller magnitude as compared with a single pulse due to the accumulation of energy in the block from the preceding pulses.³⁰ The rocking response of rigid blocks may change significantly due to small perturbations and it is reported that rocking experiments are unrepeatable. However, the researchers have concluded that the rocking response of rigid blocks can be predicted statistically.^{34,39,48} Ishiyama³⁵ has used actual ground motion records to develop no rocking nor overturning, rocking, and overturning regions in the width '2b' vs height '2h' spectrum. Rocking behavior of discrete members of classical columns resting on top of each other and subjected to earthquake excitation are also studied theoretically and experimentally.⁵³ If the energy dissipation of the rocking block is neglected, then the stability of the rocking block subjected to earthquake-like series of pulses may be determined by comparing the energy required to overturn the block and energy input by the earthquake. From the velocity response spectrum S_v , the probabilistic safe slenderness ratio can be obtained as described in Table 3.³⁴ If the value of α is greater than the expression described in Table 3, the probability of overturning the block is less than 50%. Housner³⁴ concluded that it is the scale effect that makes the larger of two objects with the same slenderness ratio more stable during an earthquake.

2.9 Parametric Study for Rocking of Rigid Blocks

The prediction of the overturning of the rocking block, based on deterministic analysis, is not reliable and the researchers^{34,39} reached conclusive remarks after performing probabilistic analyses. The probability of overturning of the block increases due to an increase in slenderness ratio, decrease in size and increase in ground shaking intensity. Nevertheless, for the real earthquake of mixed waves, it is challenging to reach a general conclusion. Furthermore, the rocking response of the block is very sensitive to a little change in the parameters. Sorrentino et al.⁵⁴ unveiled the peak ground velocity (PGV) parameter of ground excitation as an assessment of seismic hazard for rocking objects. Although PGA is an important intensity measure parameter to start rocking, it does not play its role in overturning except for tiny blocks. Overturning probability of rigid blocks is associated with peak ground displacement (PGD) intensity measure for moderate slenderness and large slenderness; it is more controlled by peak ground velocity (PGV) intensity measure.⁵⁵ The dynamic behavior of free rocking rigid blocks is highly sensitive to the initial amplitude ratio, small changes in friction, and imperfections of the blocks. Behavior also changes with change in the interface materials. The energy dissipated during the free rocking motion is highest for rubber interface, followed by timber and concrete interfaces.⁵⁶ Due to an increase in the height of the same width block, both size and slenderness increase and these two parameters compete with each other. In the classical theory of rocking rigid blocks presented by Housner,³⁴ it is the size effect that eventually wins over the slenderness effect but it is not applicable for all frequency range of earthquake excitation. The stability of large and slender blocks is very sensitive to the nature of ground excitation. This is explained with the help of a single wavelet pulse of amplitude a_p and the period T_p . For a long duration pulse (change in acceleration is slow), hence contribution from rotational inertia (resistance to earthquake) is less and the block may become less stable. For short pulse (rapid change in acceleration) more contribution from rotational inertia is observed and more stable is the rocking block. Therefore, subject to long period

predominant intense part of an earthquake, the large and slender blocks will eventually overturn as slenderness (α) wins the oversize (R) effect (contrary to the classical theory of rocking rigid blocks). Eventually, it is the characteristic length scale $a_p T_p^2$ of the ground excitation that is more important in deciding about the overturning of the free-standing large and slender rocking column. The larger the length scale of pulse excitation, the lesser will be the stability. Sometimes, large and slender blocks may overturn without rocking, but a substantial seismic input is required for such conditions.^{37,57} Makris and Vassiliou⁴⁷ have proposed a closed-form solution for the design of the width of a large and slender block that can survive during pulse excitation.

2.10 Dynamic Models Proposed for Rocking Rigid Blocks

Rocking is a complex dynamic problem, and it is difficult to model it. Researchers have proposed different models and tried to solve these models using numerical analysis techniques, finite element method, discrete element method, and derived closed-form solutions as well. A brief review of these models and some other models is presented in the following sections.

2.10.1 Analytical Models

Perhaps, Housner³⁴ presented the first analytical model of rocking rigid blocks, conceiving it as an inverted pendulum problem. After this, many researchers developed analytical models and their solutions.^{30,37–39,46,55,57–60} Aslam et al.³⁰ and Psycharis and Jennings⁶⁰ have used the concept of equivalent viscous damping to model the impact mechanism during rocking and written their computer programs for numerical analyses. Peña et al.³⁸ and Prieto et al.⁶¹ have performed the analytical simulation using two different tools: (a) Complex Coupled Rocking Rotation (CCRR) method, and (b) Discrete Element Method (DEM). The CCRR method combines the two piece-wise nonlinear equations and coefficient of restitution,

defined by Housner,³⁴ into a single formulation. In this method, rocking angle θ is assumed to be a complex quantity and a single differential equation governs rocking motion. Prieto and Lourenço⁶² have used Lagrange's formulation to unify the two piecewise equations as defined by Housner³⁴ into a single formulation. They have also incorporated the damping mechanism into the same expression. Lipscombe and Pellegrino⁶³ have thoroughly studied the free rocking of rigid blocks using a simple rocking model (defined in their article) and compared it with other models. The rocking surface does not remain under elastic stresses due to the concentration of load on alternate edges during rocking. Therefore, Avgenakis and Psycharis⁶⁴ have proposed a macro element model to estimate the inelastic stresses developed on the edges and within the member during rocking. Truniger et al.⁴⁹ studied the flexible SDOF structure mounted on a rocking base and subjected to pulse and recorded earthquake ground excitations. In the case of out-of-plane rocking of a masonry wall, to account for the formation of secondary hinge near the corners due to crushing, a three-branch moment-rotation curve is proposed.^{32,33} Analytical results are closer to experimental results, using the proposed three-branch moment-rotation curve. As the slope of the moment-rotation curve of the rocking phase is negative; therefore, it is challenging to perform numerical analyses for that phase. Researchers have proposed a stepwise strength reduction method to simulate the negative stiffness behavior of the rocking phase.^{43,45} Winkler spring model and an equivalent two spring model (at the corners of the rocking block) are also used to study the rocking (after partial uplift) of buildings during earthquakes.^{60,65} Papantonopoulos et al.⁵³ have used a distinct (discrete) element method to study the rocking behavior of discrete members of the classical columns resting on top of each other. To make the solution of the rocking problem computationally efficient, Vassiliou et al.⁴⁶ have proposed a model, which consists of a nonlinear rotational spring having the same loading and unloading paths. Bachmann et al.⁶⁶ argued that the motion of rocking rigid blocks could be predicted more accurately using the statistical approach (using an ensemble/suite of ground excitations instead of a single ground excitation).

2.10.2 Finite Element Models

Most of the commercial and open-source programs available for structural analysis in civil engineering are based on the finite element method. Therefore, efforts are made by many researchers to develop finite element solutions of the rocking block problem. Vassiliou et al.⁴⁶ have proposed the rotational spring model for the rocking of both rigid and flexible blocks, which is more straightforward and compatible with readily available finite element software. Roh and Reinhorn⁴⁴ have developed a macro-level finite element model (using ABAQUS software) of the rocking column and validated it from experimental results. Ma et al.⁶⁷ have proposed a model for rocking structural elements which may be incorporated in finite element model packages. Vassiliou et al.⁶⁸ have introduced a finite element model that is compatible with the OpenSees framework. In this model, zero-length fibers are used at the rocking surface to produce rocking.

2.10.3 Constant Acceleration Models

In the classical theory of rocking rigid blocks proposed by Housner³⁴, rocking is defined in half-cycles between two consecutive impacts and the rocking block shows a little variation in acceleration between two impacts. A simplified rocking model is proposed, in which acceleration between two consecutive impacts is considered as constant.^{69,70} The proposed constant acceleration between two consecutive impacts is calculated by establishing conditions of equality between proposed and classical rocking theories.

2.10.4 Equivalent Nonlinear SDOF Model

Priestley et al.⁵⁸ proposed that the rocking system can be assumed with an equivalent SDOF elastic system having a constant damping ratio and period depending upon the amplitude of vibration. Hence based upon this assumption, displacement

response spectra can be used to estimate the maximum displacement during rocking of the system. They have also developed a curve to define the equivalent damping ratio of the rocking system based upon the coefficient of restitution and validated their theory performing laboratory experimentations. Lagomarsino⁵⁵ has also presented a nonlinear equivalent SDOF system to define the rocking of masonry structures and proposed a displacement-based capacity curve. For consideration of rocking behavior, FEMA 356 (2000)⁷¹ has also adopted a similar procedure using elastic response spectra and equivalent viscous damping. According to some other authors,^{36,37,59,72} the rocking of rigid block can't be treated as an equivalent SDOF elastic system due to the reason that these are two fundamentally different systems and behavior of rigid rocking block is nonlinear and highly sensitive to small perturbations. A conventional elastic model has positive stiffness and a set of natural frequencies, but the rocking of rigid block exhibits negative stiffness and does not possess discrete natural frequencies. As the rocking period of the rigid block is dependent upon the amplitude ratio (θ_o/α); an instantaneous frequency, instead of a discrete frequency, can be defined representing its highly nonlinear nature. The estimated natural period of the rocking block does not remain constant but it decreases as the free vibration proceeds as shown in Figure 2.9(b).

2.10.5 3D Model

The rocking of the rigid block is a 3D problem, while most of the articles discuss 2D rocking behavior which is a limiting case of it. The 2D rocking model does not exhibit the exact behavior in which the block may also twist on its corners instead of an edge. Chatzis and Smyth⁷³ have used Lagrange's equation of motion and proposed two 3D models for the rocking of rigid blocks: 1) the concentrated spring model; 2) the Winkler model. In the former, springs and dampers are considered at the corner only, while in the later, springs and dampers are distributed. Konstantinidis and Makris⁷⁴ have used the Lagrange equation of motion and rotation tensors to defined 3D rocking of rigid blocks. However, the authors did not incorporate energy

dissipation due to the impact of this formulation. The 3D rocking of rigid cylinders modeled on the Winkler foundation with spring and dashpot is studied by Koh and Hsiung.⁷⁵

Existing 2D rocking models may give a reasonable guess of the 3D rocking response without demonstrating the actual response of 3D behavior. Therefore, 2D rocking motion cannot be simply extended to define 3D rocking and wobbling motion of a rigid block or cylinder.⁷⁶ In 3D rocking and rigid wobbling block, the presiding mode of motion is wobbling, and rocking impact infrequently occurs.⁷⁶ Rajah et al.⁴⁸ have proposed a model to study the rocking and wobbling of four cylindrical columns having a rigid slab on top of it.

2.11 Resonance Free Behavior of Rocking Rigid Blocks

The rocking block does not vibrate at a specific frequency because its frequency is dependent upon the amplitude of the vibrations. Therefore, a rigid rocking block, instead of discrete natural frequency, exhibits instantaneous frequency which changes after each impact. Unlike elastic SDOF oscillatory systems, the rigid rocking block does not resonate at a specific frequency. Natural ground acceleration frequency matching changing frequencies of the rocking body and causing resonances is very unlikely. Therefore, to initiate resonance in rocking systems, it is necessary to apply a ground excitation that has identical or similar frequency changes following the frequency changes of the rocking body. This artificial excitation is known as “resonant input,” and it has paved the way towards the development of reliable resonance spectra for rigid rocking blocks.^{31,59,77}

2.12 Rocking Structures in Civil Engineering

The use of rocking members for earthquake damage mitigation of structures is one of the attractive ideas, which can be used for civil engineering structures.⁶⁴ Figure 2.12 gives an idea about the rocking structures used in Civil Engineering.

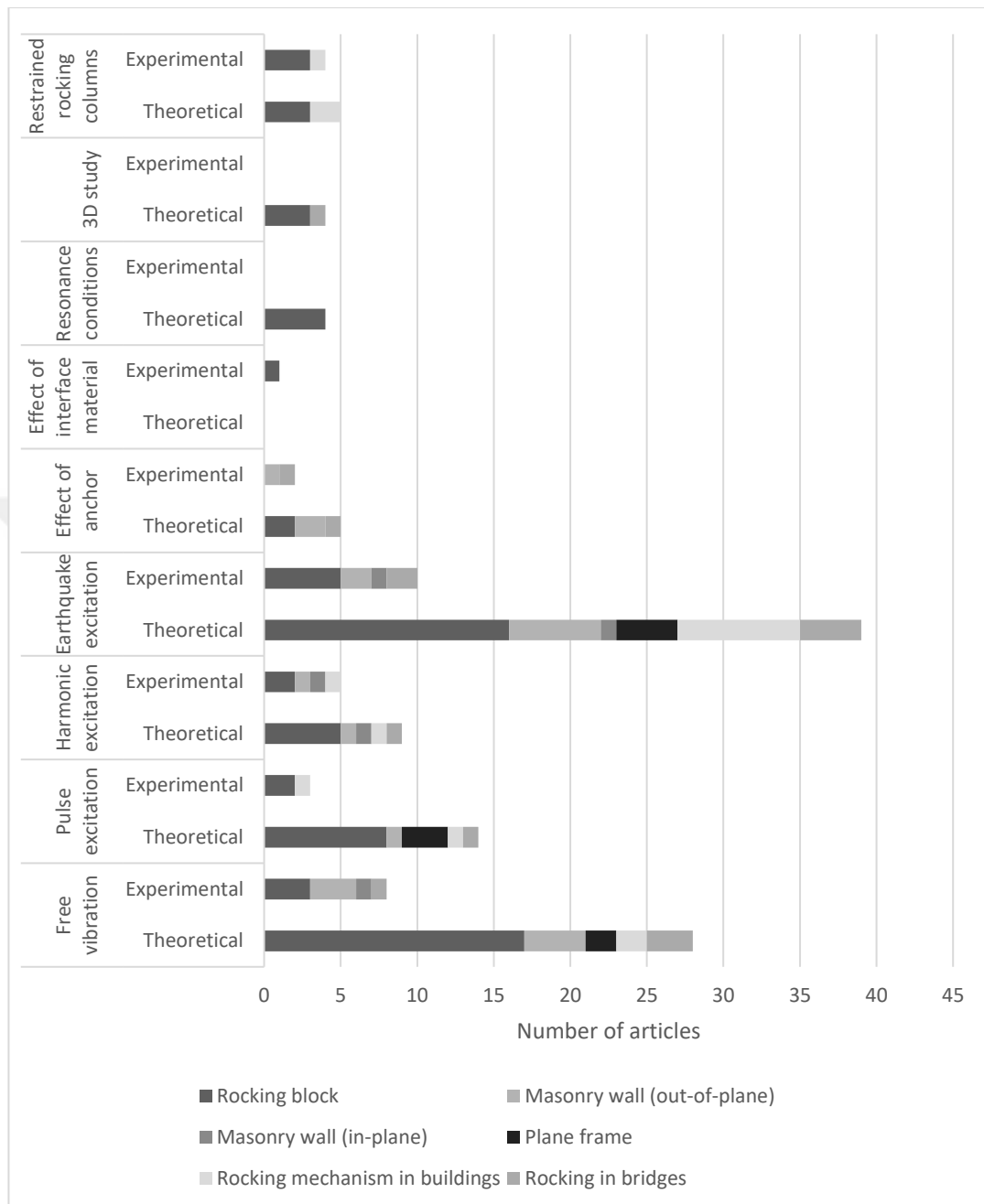


Figure 2.12 Spectrum of articles published in different research areas

2.12.1 Rocking in Bridges

The concept of rocking of piers was introduced as a source of isolation in bridges five decades ago.⁷⁸ Researchers^{79–81} have studied the rocking bridge piers as a source

of seismic isolation. Chen et al.⁶ have performed theoretical and experimental studies to evaluate the rocking of a viaduct pier in the lateral direction. Although the literature is available in support of rocking isolation of bridges and its structural stability, even then, designers are reluctant to design rocking pier systems mostly relying on conventional systems.³⁶ The rocking and wobbling pier isolation of a typical bridge may perform well even against strong ground excitations.⁴⁸ Han et al.⁸¹ introduced the yielding metallic damper at the base of rocking columns of a bridge pier in their small scale experimental study and FEM model. Expressing rocking motion through a displacement is more relevant as compared to the tilt angle. Therefore, the displacement-based design method may be applied for the designing of rocking columns of bridges and other structures.⁸² Sieber et al.⁸³ proposed a couple of simplified analysis models of rocking bridge pier rested on deformable soil. The proposed models can predict the maximum rotation of the rocking pier due to ground excitation but unable to predict the settlements and damping (due to soil-structure interaction) accurately.

2.12.2 Rocking of Masonry Walls

In the absence of box-type behavior of masonry wall structure, during an earthquake, masonry wall may rock and overturn in the out of plane direction.⁸⁴ Out of plane rocking of masonry walls can be defined with the help of a two-branch or three-branch moment-rotation curve.^{32,55} Casapulla et al.⁸⁴ have reviewed the recent advancements in the field of rocking behavior of masonry walls and explained how different researchers had extended the concept of rocking of rigid blocks to define the rocking behavior of more complex problems like the rocking of masonry walls. Sorrentino et al.³³ and Shawa et al.³² have studied the rocking behavior of unreinforced masonry facades restrained from one side. They have concluded that if the rocking is restrained on one side and the façade is allowed to rock on the other side (outward direction only) it may lead towards more destructive results due to rebound on impact with restraint. In one-sided rocking, there are two possible

impacts: (a) at the bottom of the wall due to rocking which may cause dissipation of energy and (b) the impact with the restraining transverse direction wall which may cause a rebound of the rocking façade. The latter may cause overturning or collapse of the rocking façade. Different researchers also developed equivalences between a rocking masonry wall and an SDOF. Priestley et al.⁵⁸ have performed the experimental study in which a masonry wall is subjected to free vibration, harmonic excitation and earthquake excitation for in-plane rocking motion, and developed an analysis procedure based upon elastic response spectrum. Shawa et al.³² have performed the experimental studies for the out-of-plane rocking of the façade connected with two masonry walls forming a U-shaped masonry structure. Experimental results are compared with the results obtained from the distinct element method (DEM) and an SDOF model. Both models have proposed good agreement with the experimental results. Masonry walls subjected to earthquake excitation possess extra dynamic stability against overturning (collapse) even after cracking (at the bottom) and rocking.³² DeJong and Dimitrakopoulos⁸⁵ have tried to develop an equivalence between a single rocking block and out of plane rocking masonry wall.⁸⁵

Capelli et al.⁸⁶ have proposed a simplified model for the deformable rocking block to study the out of plane rocking of a masonry wall and compared the results with the Housner's model. Thiers-Moggia and Málaga-Chuquitaype⁸⁷ studied that the performance of anchored rocking walls increases if supplemental rotational inertia is introduced into the system. Tomassetti et al.⁸⁸ proposed an equivalent viscous damping model based upon the experimental and analytical coefficient of restitution for out of plane rocking of the unreinforced masonry wall. Como et al.⁸⁹ have studied the rocking response of cracked masonry walls in case of ground excitations. Based on the Housner's rocking rigid block model, Casapulla et al.⁹⁰ proposed an equivalent 2D model to study the typical corner failure of a masonry building due to the development of a rocking mechanism during an earthquake.

2.12.3 Rocking of Plane Frames

Researchers have proposed an equivalent single free-standing rocking column to explain the behavior of the rocking plane frame and concluded that the rocking frame is more stable as compared with the single rocking column in case of an earthquake.^{36,48,79} DeJong and Dimitrakopoulos⁸⁵ have also developed an equivalence between a single rocking block and a rocking frame (as well as a rocking arch). Vassiliou et al.⁶⁸ have used the deformable rocking block model to study the simple rocking frame. Bachmann et al.⁴⁸ have studied the SDOF podium structures supported by a simple rocking plane (columns can exhibit in-plane rocking). They studied the behavior and investigated the accuracy of the rules of thumb defined by Polyakov.⁹¹

Furthermore, the equivalent static lateral force procedure does not capture the actual rocking behavior and dynamic analyses are required.^{36,79,80} Bachmann et al.⁵² introduced the concept of rolling and rocking of the frame in which the end flat rocking surfaces of the columns (in a frame) are widened using curved flanges. By doing so, the stability of the frame is increased without increasing the uplift acceleration.

2.12.4 Rocking of Rigid Blocks Anchored to The Ground

When rigid rocking blocks are restrained vertically with elastic anchors/tendons, the resistance against the overturning of small blocks subjected to long period excitation increases. However, the effect of vertical restraint on the rocking behavior of rigid blocks and anchored walls diminishes due to an increase in the block size or increase in the frequency of excitation.^{80,87}

2.12.5 Rocking Mechanisms in Buildings

Researchers have introduced various rocking mechanisms in buildings for earthquake response modification. Blebo and Roke⁹² have introduced a rocking core in concentrically braced frames along with buckling restrained columns to reduce inter-story drift. Cheng and Chao²⁷ have performed an experimental study by introducing the concept of rockers for the isolation of the steel frame. The experimental study has shown that the performance of the proposed rocking base isolation system depends upon the aspect ratio of the rocker, shape of endplates, and the nature of ground excitation. Freddi et al.²⁸ have designed the rocking column base along with post-tensioned steel bars and friction dampers. This rocking column base can be used in self-centering moment-resisting frames and it protects the first story column from yielding. Due to the presence of friction dampers and post-tensioned steel bars, the moment rotation curve has no negative slope but shows flag-shaped hysteretic behavior. A similar study of the rocking base of steel columns is performed by Liu et al.⁹³ in which metallic slit dampers are used instead of friction dampers at the base of the column. The theory of rocking blocks is used to study partial uplift or complete separation of building from its foundation during an earthquake and a simplified two spring model is proposed by Psycharis and Jennings⁶⁰, and Psycharis⁶⁵. A self-centering rocking system for steel buildings is proposed by Azuhata et al.⁹⁴ in which structural elements can rock during severe earthquakes and hence protects the steel structures from yielding and permanent damages. The rocking isolated frame (used in buildings) is introduced by reducing the dimensions of footing and allowing for the formation of hinges in beams in case of strong earthquakes. The performance of rocking isolated frames is better than that of its conventional counterpart.⁹⁵ Ríos-García & Benavent-Climent⁹⁶ proposed rocking isolation of building, in which the bottom story columns can rock and metallic yielding dampers damp these columns at the lateral restraining edges of the columns. Columns are restrained by central steel bars which provide restoring force in case of limit lateral displacement. However, the authors did not pay attention to

control the slip of the rocking columns and did not pay attention to the 3D behavior of rocking columns which is quite different from the 2D rocking behavior. Hashemi and Quenneville⁹⁷ proposed a friction damping and restoring mechanism at the corners of the in-plane rocking walls of the wooden structures. They developed the numerical model using SAP2000 and performed time history analyses for different earthquake records. Bachmann et al.⁹⁸ proposed an analytical model and its experimental verification of an SDOF structure connected at the top of a rocking column single-story frame structure.

2.12.6 Rocking of Chimneys

Sharpe and Skinner⁹⁹ have modeled and built an industrial chimney to rock during large earthquakes. Vassiliou et al.⁴⁶ have theoretically studied the performance of that chimney with and without rocking. They have used the simplified rotational spring model (to model rocking behavior) at the base of the chimney and concluded that rocking had enhanced the performance of chimney under seismic actions.

2.13 Remarks

The following remarks are summarized based on a comprehensive review of rocking rigid blocks which are presented in this chapter:

- Simple static analysis for establishing overturning criteria under dynamic loading is not enough and requires a dynamic investigation.
- The vertical component of ground excitation may be ignored during analyses of the rocking of rigid blocks.
- The probability of single rocking block overturning increases due to i) increase in slenderness (decrease in slenderness ratio α), ii) decrease in rocking column size (R), and iii) increase in ground shaking acceleration amplitude.

- Peak ground velocity (PGV) is an essential parameter for seismic hazard assessment of rocking objects.
- Larger weight and rotational inertia make the rocking blocks more stable.
- The rocking of a rigid block is nonlinear and highly sensitive to small perturbations (chaotic).
- The rocking of a rigid block cannot be treated as an equivalent SDOF elastic system.
- The rocking frame is more stable as compared with the single rocking column.



CHAPTER 3

METHODOLOGY

This research aims to develop a story high Rocking-Column (ROC) seismic isolator for the residential buildings. Figure 3.1 presents a brief flow chart of the research road map, and the research commences with a survey of existing literature for the better understanding of the rocking behavior. The next step is to develop the numerical model of the rocking column. A well-known structural analysis and design framework (SAP2000 v21 or SAP2000 v22) is used for the development of the rocking column model.

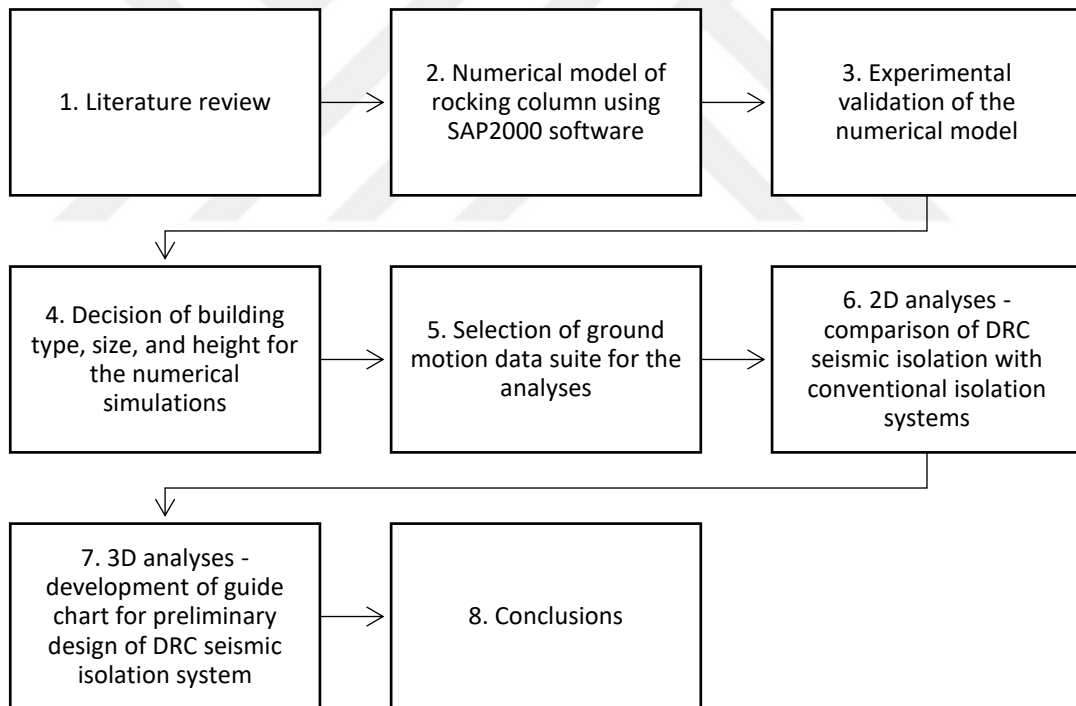


Figure 3.1 Flow chart explaining different steps involved in the research.

As the rocking model is sensitive to change in parameters (like ground excitation, damping ratio, slenderness and size of the rocking column), therefore, it is essential to validate the proposed numerical model with the help of laboratory experiments.

The laboratory experiments are performed in two phases. The first phase consists of experimental validation of the rocking of solitary wooden column. The second phase comprises of the experimental validation of a wooden podium structure which is made up of four rocking wooden columns and a stiff roofing board resting at top of these four column. The column-base and column-roof connections are made to rock the structure as a whole. After the accomplishment of the experimental validation of the numerical rocking model, the next step is to perform further numerical study using SAP2000 software. A data suite of eight different ground motion records is used to study the behavior of the rocking structure. The numerical study is categorized into 2D analyses and 3D analyses. The category of 2D analyses deals with the dynamic response of the ROC seismic isolated buildings and its comparison with the conventional seismic base isolation systems. This comparison gives an idea about the adequacy of the proposed ROC seismic isolation system in the buildings. The effect of different parameters like height of the building, width and height of the rocking column, bay size, and different earthquake records are studied in the category of 3D analyses. The results of the 3D analyses lead to the development of guide charts for the preliminary design of ROC seismic isolation for residential buildings.

3.1 Details of building plan, grid size, and height

A regular building plan is proposed for the 2D and 3D analyses using SAP2000. A uniform grid size of 5m is used for the 2D analyses, having four bays in both directions. Four and seven stories are the two different building heights for 2D analyses.

The building plan of 3D model is of two types, one with 5m bay size and the other with 7.5m bay size (as shown in Figure 3.2). The reason behind the selection of 5m and 7.5m bay sizes is that these are suitable for the parking facility. The former bay size is suitable for the parking of two cars in between two consecutive columns and the later bay size is suitable for the parking of three cars in between two consecutive

columns. As the ROC seismic isolation is proposed for the typical residential buildings; therefore, four to seven stories of the building are used for the analyses. Another variable parameter for the 3D analyses is the slenderness of the rocking columns and details about this parameter is available in the coming sections.

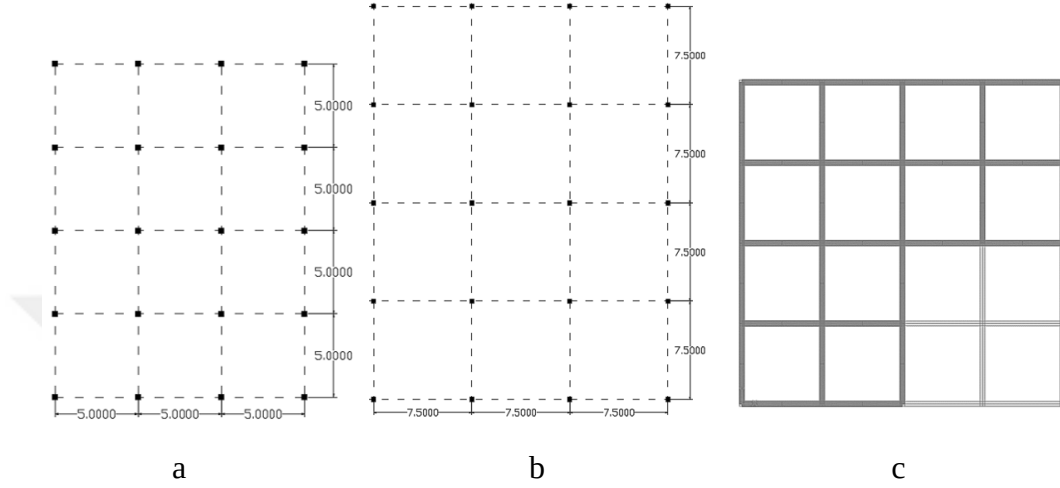


Figure 3.2 Grid plan of building for the 3D analyses; (a) 5m bay distance, (b) 7.5m bay distance, (c) L-shaped building with 5m bay distance

3.2 Selection of earthquake data suite

The behavior of the rocking structures is sensitive to the change in ground motion type and earthquake magnitude. The rocking structures are vulnerable to large changes in its response due to small changes in the ground motion record, and it leads us to use a range of ground motion data set. Therefore, a suite of eight different earthquake records (Table 4) is used for the numerical analyses of the rocking structures. The ground motion data is downloaded from the PEER. In Table 1, R_{rup} value is the shortest distance from the rupture surface to the station point where data is collected and V_{s30} is the average velocity of the shear wave at a depth of 30m from the surface. The selected ground motion data suite covers different earthquake magnitudes and types of fault mechanisms, e.g., normal, reverse, reverse oblique, and strike slip fault.

Table 4 Suite of eight different ground motion data (PEER)

Serial No	Earthquake Name	Year	Station Name	M _w	Mechanism	R _{rup} (km)	Vs ₃₀ (m/sec)
1	Imperial Valley-02_USA	1940	El Centro Array #9	6.95	strike slip	6.09	213.44
2	Kobe_Japan	1995	Nishi-Akashi	6.9	strike slip	7.08	609
3	Northridge-01	1994	Arleta - Nordhoff Fire Station	6.69	Reverse	8.66	297.71
4	Campano Lucano (aftershock 9)_Italy	1980	Sturno (STN)	4.9	Normal	28.68	382
5	Duzce_Turkey	1999	Duzce	7.14	strike slip	6.58	281.86
6	Erzincan_Turkey	1992	Erzincan	6.69	strike slip	4.38	352.05
7	Kalamata_Greece-01	1986	Kalamata (bsmt)	6.2	Normal	6.45	382.21
8	Loma Prieta_USA	1989	Corralitos	6.93	Reverse Oblique	3.85	462.24

In the ground motion data suite, each earthquake consists of two horizontal records. All of the records used for the analyses are unscaled and as recorded. Figure 3.3 to Figure 3.10 show the acceleration-time histories of two horizontal components and acceleration response spectra of all the selected earthquakes.

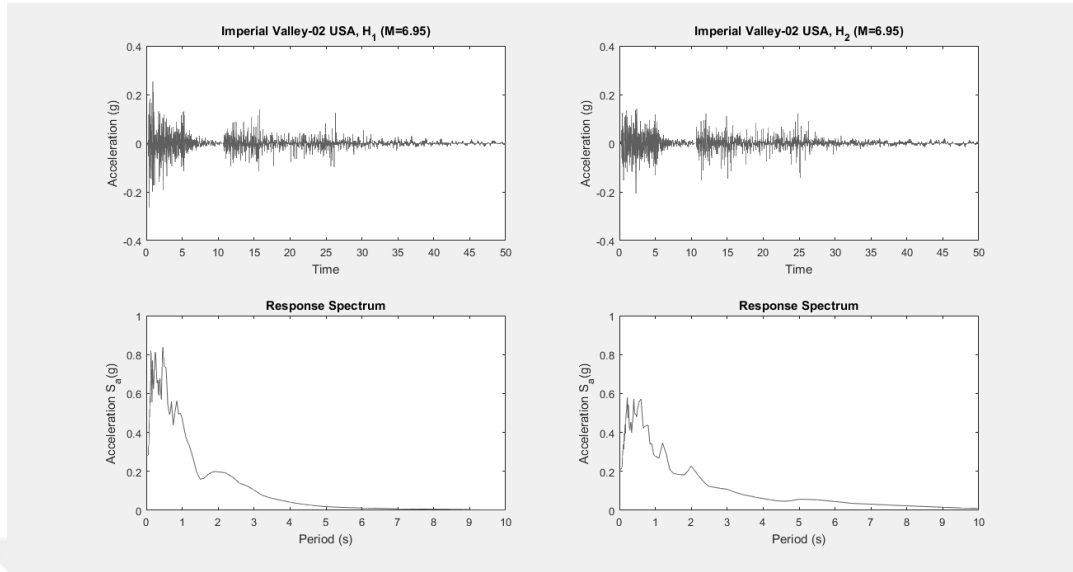


Figure 3.3 Imperial Valley-02 earthquake, 1940, USA, El Centro array # 9. Acceleration time history – component ELC180 (top left) and component ELC270 (top right). Acceleration response spectra (5% damping) – component ELC180 (bottom left) and component ELC270 (bottom right).

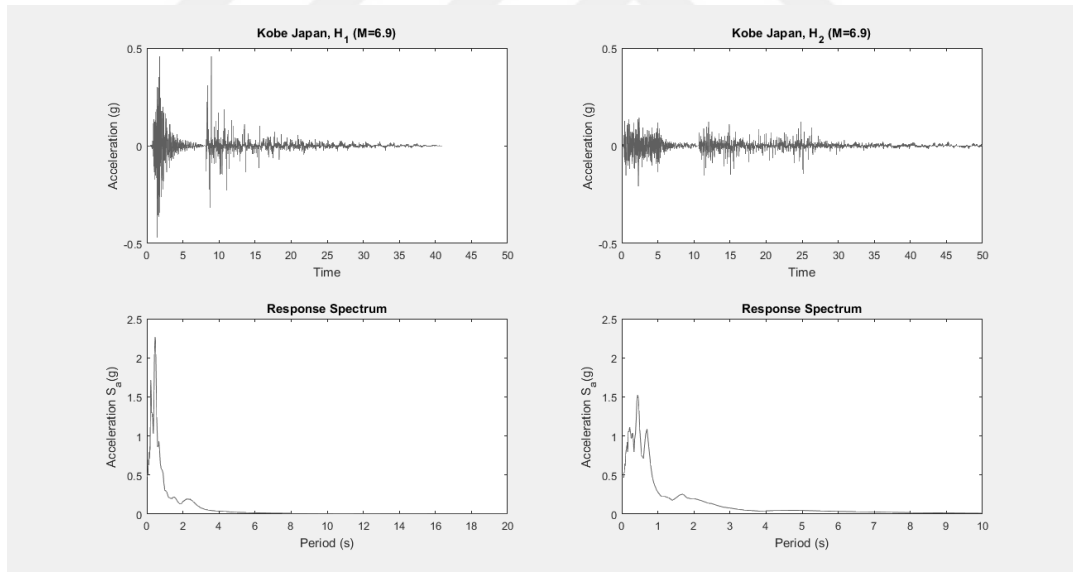


Figure 3.4 Kobe 1995, Japan, Nishi-Akashi. Acceleration time history – component NIS000 (top left), component NIS090 (top right). Acceleration response spectra (5% damping)– component NIS000 (bottom left), component NIS090 (bottom right).

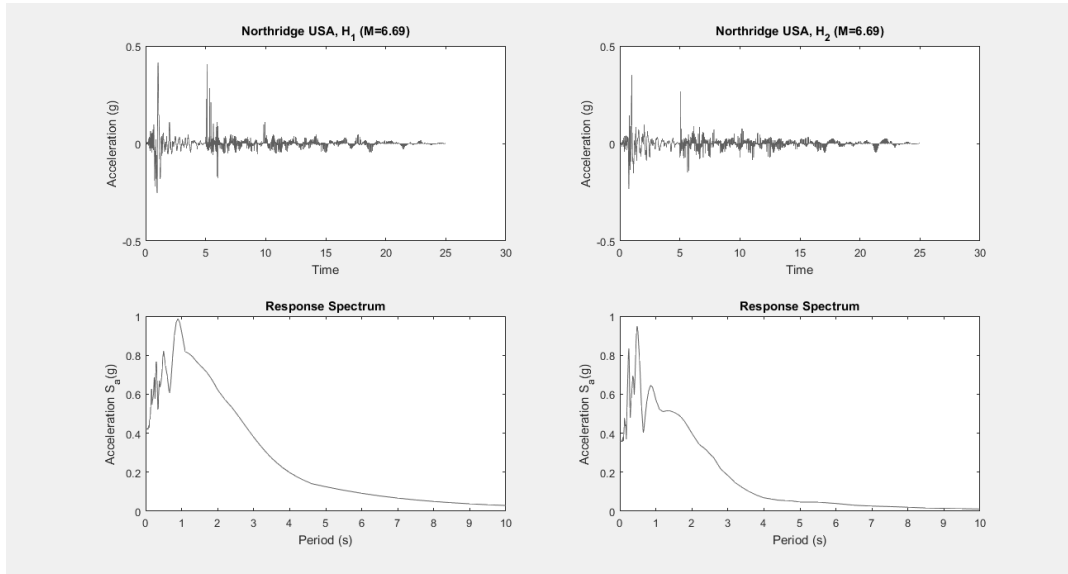


Figure 3.5 Northridge-01, USA, 1994, Arleta-Nordhoff Fire Station. Acceleration time history – component ARL090 (top left) and component ARL360 (top right). Acceleration response spectra (5% damping) – component ARL090 (bottom left) and component ARL360 (bottom right).

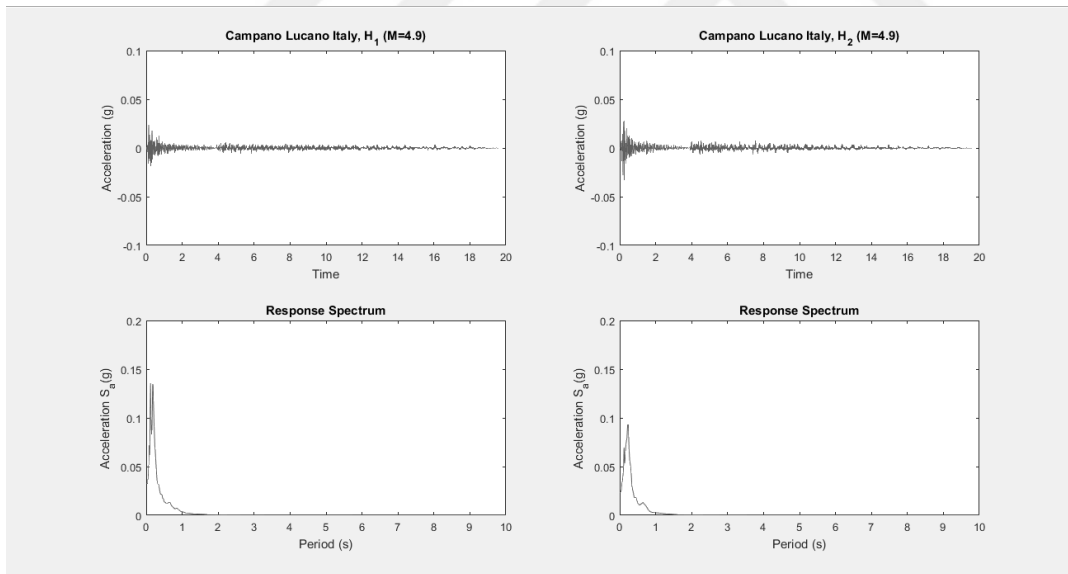


Figure 3.6 Campano Lucano (aftershock 9), Italy, 1980, Sturno. Acceleration time history – component STU000 (top left) and component STU0270 (top right). Acceleration response spectra (5% damping) – component STU000 (bottom left) and component STU (bottom right).

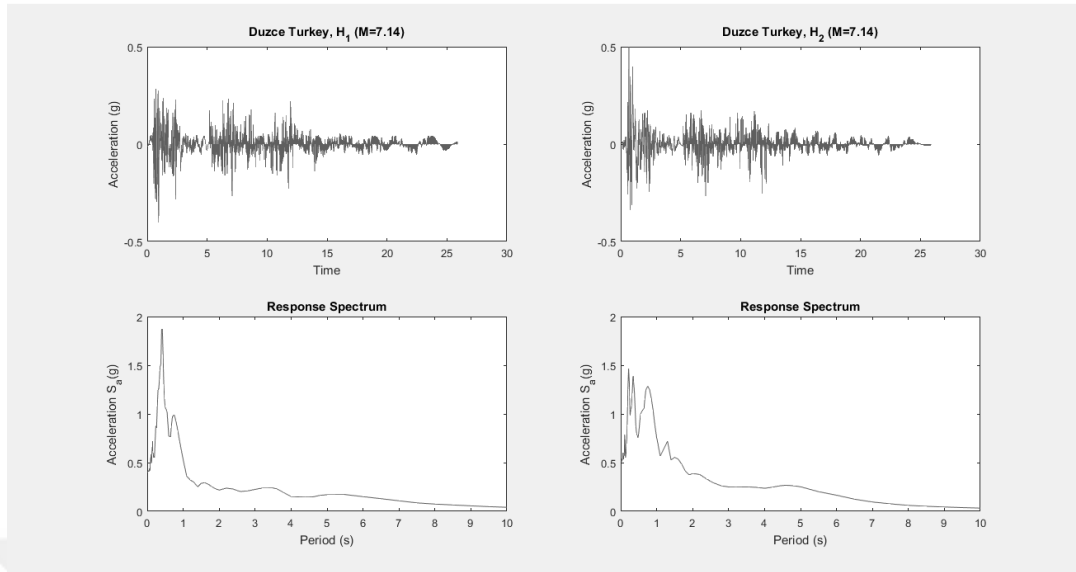


Figure 3.7 Duzce, Turkey, 1999. Acceleration time history – component DUZ180 (top left) and component DUZ270 (top right). Acceleration response spectra (5% damping) – component DUZ180 (bottom left) and component DUZ270 (bottom right).

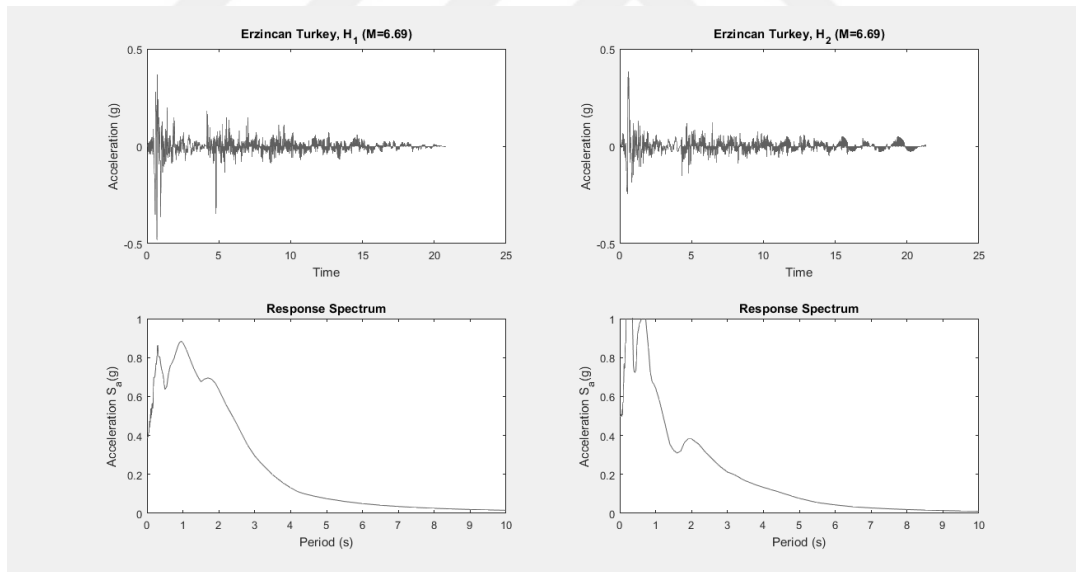


Figure 3.8 Erzincan, Turkey, 1992. Acceleration time history – component ERZ-EW (top left) and component ERZ-NS (top right). Acceleration response spectra (5% damping) – component ERZ-EW (bottom left) and ERZ-NS (bottom right).

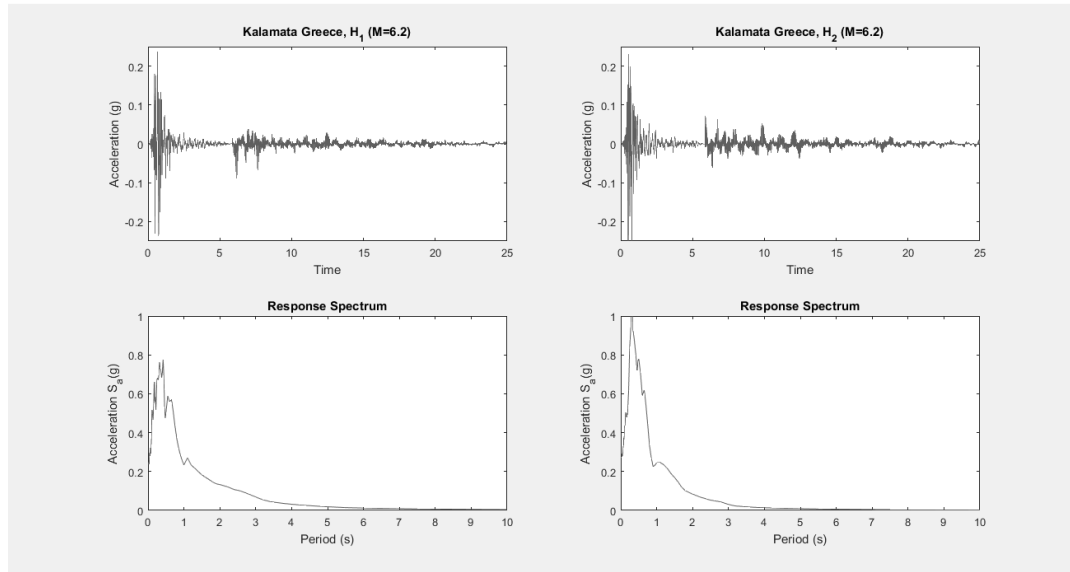


Figure 3.9 Kalamata-01, 1989, Greece. Acceleration time history – component KAL-NS (top left) and component KAL-WE (top right). Acceleration response spectra (5% damping) – component KAL-NS (bottom left) and component KAL-WE (bottom right).

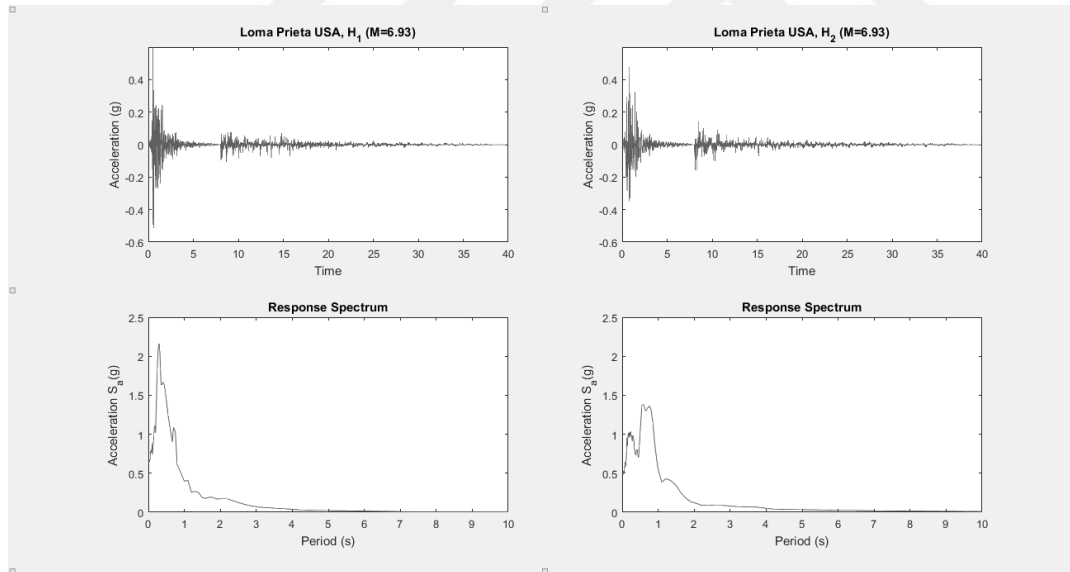


Figure 3.10 Loma Prieta, 1989, USA, Corralitos. Acceleration time history – component CLS000 (top left) and component CLS090 (top right). Acceleration response spectra (5% damping) – component CLS000 (bottom left) and component CLS090 (bottom right).

3.3 Anatomy of 2D analyses

In the first category of analyses, which are 2D analyses, two different building heights are proposed. The first building is a four story building and the second building is a seven story building. In the category of 2D analyses, ROC seismic isolation system is modelled at the bottom story and the analyses results are compared with that of conventional base isolation systems and regular building without isolators. Two conventional base isolation systems used for the comparison are the friction pendulum system (FPS) and laminated rubber bearing (LRB). Other variable parameters are the mass irregularity and stiffness irregularity along the height of the building. Mass irregularity is achieved by the means of introducing a massive slab at the first-floor level. In the mass irregular building models, the thickness of the first-floor slab is 0.4m as compared to the thickness of the regular remaining slabs being 0.15m. Whereas, stiffness irregularity is achieved by increasing the height of the ground floor from 3m to 4m (soft story effect at the ground floor level). The concept of mass irregularity (Mi) and stiffness irregularity (Si) is further explained in Figure 11.

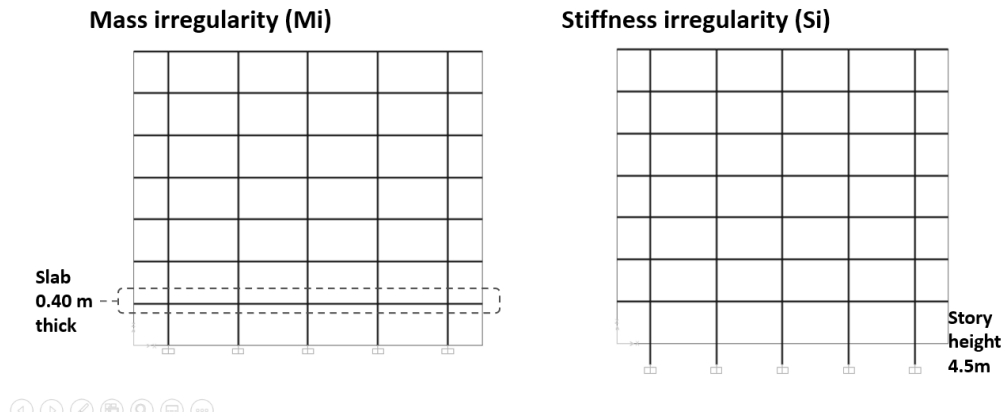


Figure 3.11 Introduction of mass irregularity by increasing the thickness to 0.4m (left) and introduction of stiffness irregularity by increasing the height of ground floor to 4.5m (right).

Height of the building, type of isolation, and type of irregularity introduced into the building are the variable parameter for the 2D analyses category. The combination

of all these parameters makes the total number of models equal to thirty. A nomenclature is defined to explain each model. An example of this nomenclature is explained in Figure 3.12. There are three portions of the nomenclature, the first stands for the number of stories, the second stands for the type of base isolation, and the third stands for the type of irregularity.

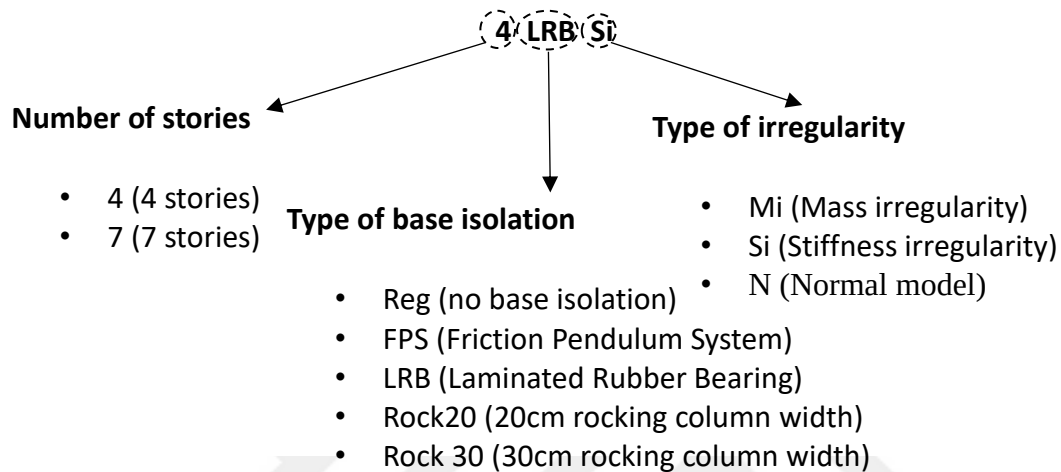


Figure 3.12 Nomenclature of the developed for 2D analyses

Typical building	Mass irregularity	Stiffness irregularity
• 4 Story • Regular column • FPS • LRB • Rock20 • Rock30 • 7 Story • Regular column • FPS • LRB • Rock20 • Rock30	• 4 Story • Regular column • FPS • LRB • Rock20 • Rock30 • 7 Story • Regular column • FPS • LRB • Rock20 • Rock30	• 4 Story • Regular column • FPS • LRB • Rock20 • Rock30 • 7 Story • Regular column • FPS • LRB • Rock20 • Rock30

Figure 3.13 List of the thirty models used for 2D analyses

3.4 Anatomy of 3D analyses

In the second category of analyses – the 3D analyses – the intention is to develop preliminary design graphs for the ROC seismic isolation system which applies to medium rise residential building. The variable parameters in this category of analyses are the width of rocking columns, height of the rocking column, height of the building, and bay size. The combination of all these parameters makes the total number of models equal to sixty four (64). A nomenclature is also defined to explain every model in this category and it is demonstrated in Figure 3.14. The nomenclature consists of four groups of letter, the first group stands for width of the rocking column, the second group stands for the height of the rocking column, the third group stands for the number of stories. The last group (if present) stands for the larger bay size (7.5m bay size). In case the last group is missing in the nomenclature, it represents the smaller bay size (5m bay size).

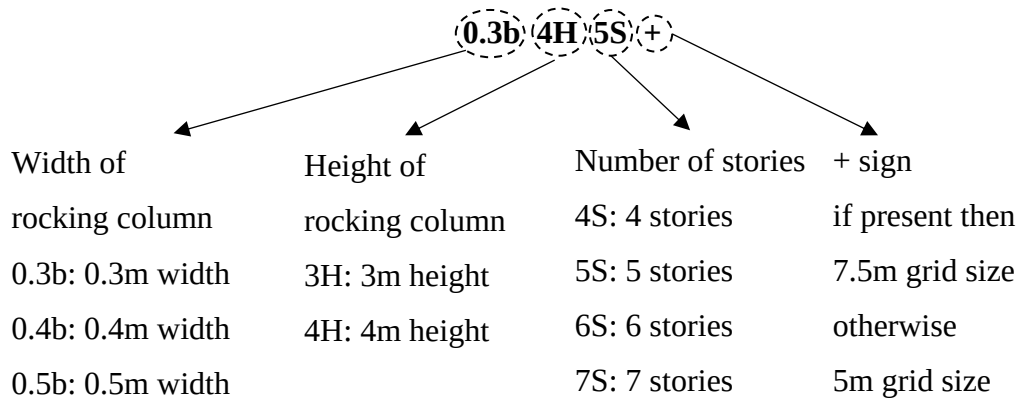


Figure 3.14 Nomenclature defined for 3D analyses.

CHAPTER 4

THEORY AND NUMERICAL MODEL OF THE ROCKING COLUMNS

The Rocking-Column (ROC) seismic isolation system is a system which isolates the super structure via rocking of columns. The complete knowledge of rocking behavior and its characteristics is important to understand. Let us consider an example of a solitary rectangular column for the better comprehension of the rocking behavior. Figure 4.1 shows a single rectangular column supporting the weight (**W**) of the super structure. The forces participating during rocking of column and the restoring moment to bring the column back to its original position are also shown in Figure 4.1. The lateral force due to earthquake is F_{EQ} , the width of the rocking column is b , and the height of the rocking column is h . The column starts rocking when the moment due to the lateral ground force F_{EQ} exceeds the restoring moment due the weight of superstructure **W** (if we ignore the self-weight of the column as compared with the weight of the superstructure). In other words, the rocking initiates when the acceleration due to ground motion (a_{EQ}) is greater than the ' b/h ' ratio multiplied by the gravitational acceleration ' g ' (i.e., $a_{EQ} > g \cdot b/h$). The mathematical derivation of this condition is presented as follows:

$$F_{EQ} \cdot h > W \cdot b \quad \text{Inequality 1}$$

$$F_{EQ} > W \cdot b/h \quad \text{Inequality 2}$$

As ' M ' is the mass of the super structure, therefore, $W = Mg$ and $F_{EQ} = a_{EQ}M$. The Inequality 2 becomes;

$$a_{EQ} \cdot M > M \cdot g \cdot b/h \quad \text{Inequality 3}$$

$$a_{EQ} > g \cdot b/h \quad \text{Inequality 4}$$

Hence, $a_{EQ} > g \cdot b/h$ is the necessary condition for the column to start rocking, having a slab at top of it. As the vertical component of the ground acceleration has marginal effect on the rocking response of rigid blocks; therefore, it does not appear in the above derivation.

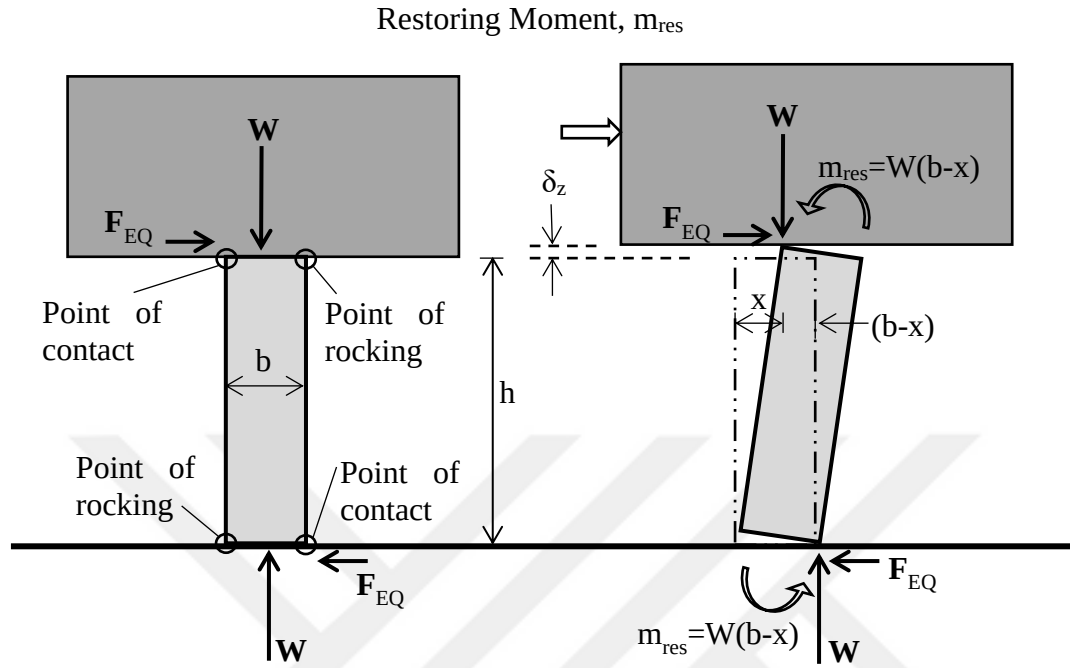


Figure 4.1 Diagram showing the forces required to trigger the rocking of column, and the restoring moment m_{res}

The restoring moment, $m_{res} = W \cdot (b-x)$, is present during rocking only, and it tries to bring the column back to its original position. For a single story high column ($h = 3.0\text{m}$) allowed, for example, to sway up to a distance of 30cm during rocking, the calculated width 'b' required for column not to overturn is larger than 30cm. Substituting $b = 30\text{ cm}$, $h = 300\text{ cm}$, and $g = 10\text{ m/s}^2$ in Inequality 4, the ground acceleration should be greater than 1 m/s^2 or $0.1g$ in order to trigger the rocking of columns. The proposed calculations show that columns initiate rocking at ground acceleration of $0.1g$ and can rock up to 30cm inter-story drift in one direction at the isolation level for the proposed dimensions of the column.

The literature review indicates that the slope of moment rotation curve is large and positive just before the initiation of rocking, but after the initiation of rocking, it becomes negative. A free body diagram and geometry of the rocking column is drawn in Figure 4.2 to derive the lateral force vs. lateral displacement relationship.

In Figure 4.2, $F(x)$ is the force required to produce a lateral displacement ' x ,' and ' W ' is the load from super structure. Rectangle $aecd$ is the original position of column before rocking and rectangle $a'ec'd'$ is the position of column after rocking. The width of column is b and the height is h . After the start of rocking motion, as the top edge of the column displaces at ' x ' amount of distance, the super structure experiences a vertical uplift equals to δ_z . It is essential to derive lateral displacement vs. lateral force relationship to compute the stiffness of the column during rocking motion. The stiffness of the rocking column is maximum at lateral displacement $x = 0$, and it decreases to zero at lateral displacement $x = b$. At $x > b$, system becomes unstable due to overturning; therefore, a stop mechanism is required to restrain the rocking column before ' x ' reaches ' b '. Otherwise, the rocking column will overturn and the structural system will become unstable. Equation 9 is derived from right angle triangle fed' in Figure 4.2 as follows:

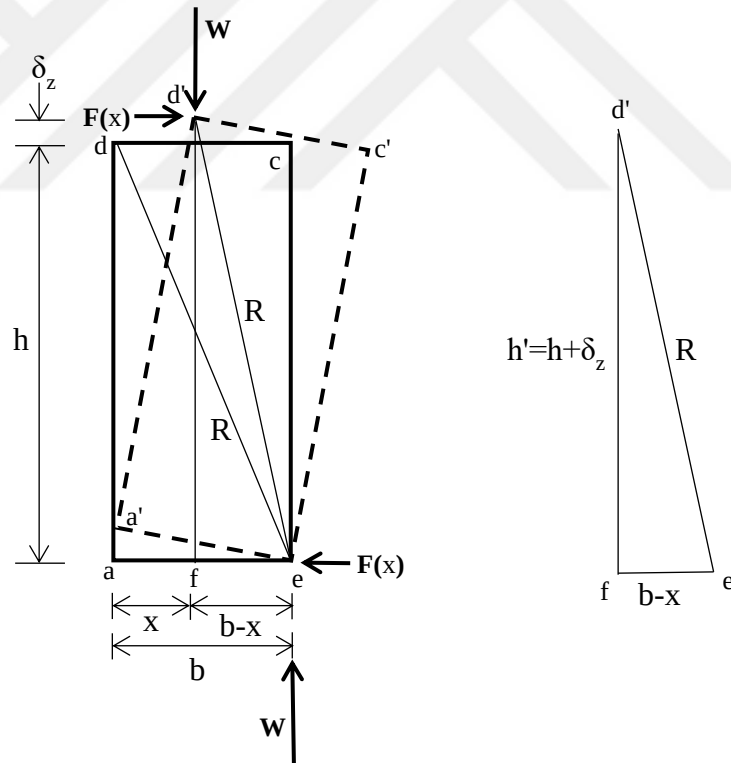


Figure 4.2 Free body diagram and geometry of rocking column

Equation 9

$$(b - x)^2 + (h + \delta_z)^2 = R^2$$

The vertical uplift δ_z may be expressed as a function of lateral displacement x and geometrical properties of the column, b , h , and R , as shown in Equation 10.

Equation 10

$$\delta_z = \sqrt{R^2 - (b - x)^2} - h$$

$\delta_z = 0$ @ $x = 0$; and $\delta_z = (R - h)$ @ $x = b$

Conservation of energy approach may be used to find the lateral force $F(x)$ as a function of lateral displacement. The work done to uplift the weight 'W' by a distance ' δ_z ' is equal to the energy stored in the system ($E = W \cdot \delta_z$). The lateral force may be obtained after taking the first derivative of the energy 'E' with respect to the 'x'. Equation 11 and Equation 12 show the expression for the lateral force, and Equation 13 shows the final form of the expression.

Equation 11

$$F(x) = \frac{dE}{dx}$$

Equation 12

$$F(x) = \frac{d \left[\left(\sqrt{R^2 - (b - x)^2} - h \right) W \right]}{dx}$$

Equation 13

$$F(x) = W \times \frac{b - x}{\sqrt{R^2 - (b - x)^2}}$$

Alternatively, the same expression may also be obtained from the equilibrium of the momentum. The moment equilibrium of the rocking column is shown in Equation 14.

Equation 14

$$W(b - x) = F(h + \delta_z)$$

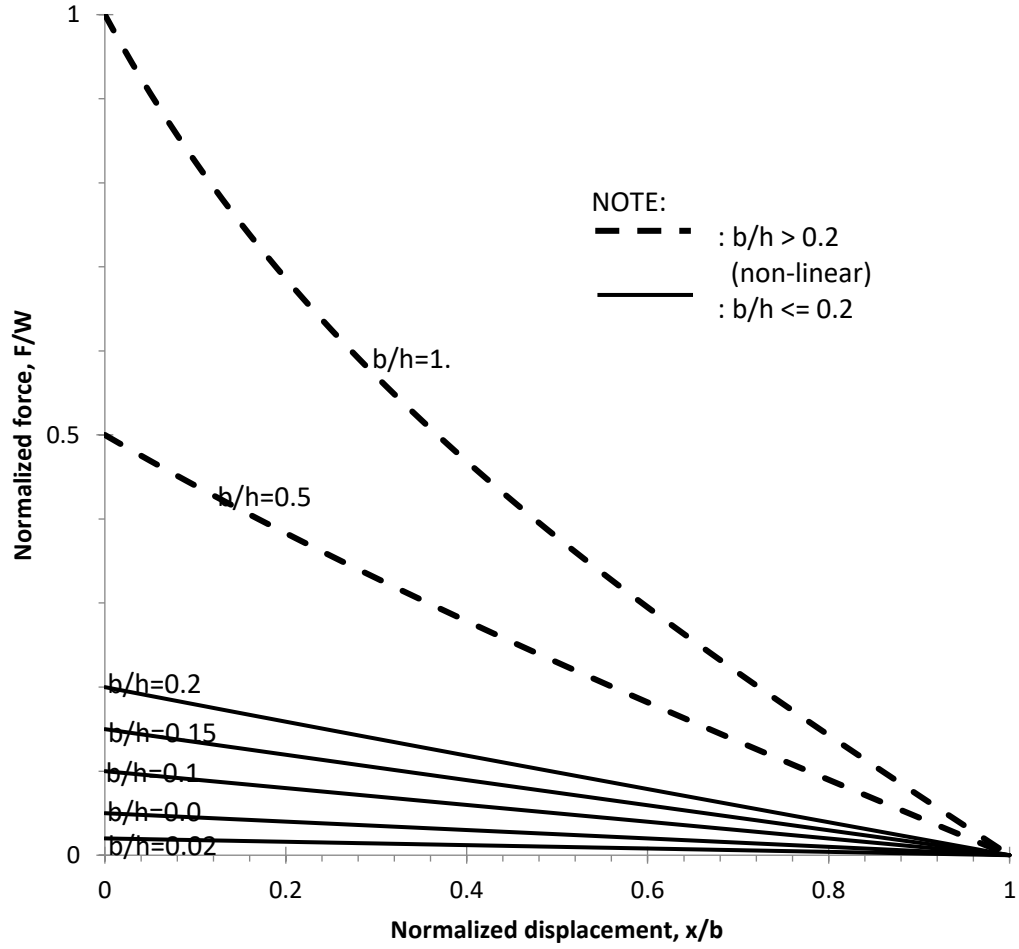


Figure 4.3 Normalized displacement vs. normalized force for different b/h ratios

Substituting the value of δ_z in Equation 14 yields the same expression as Equation 10. The normalized lateral displacement vs. normalized lateral force curve is shown in Figure 4.3. The behavior of $F(x)$ vs. x is nonlinear for high ratios of b/h but it becomes reasonably linear for b/h ratios less than 0.2. In most of the practical cases, b/h ratios of the rocking columns are usually less than 0.2; therefore, it is appropriate to simplify the above expression into a linear expression. For $b/h < 0.2$, let us assume that $h \approx \sqrt{R^2 - (b - x)^2} = h'$, the Equation 4 can be re-written in form of Equation 15 as below:

Equation 15

$$F(x) \cong W \times \frac{b-x}{h}$$

The Equation 6 applies to b/h ratios lesser than 0.2 as shown in Figure 4.4.

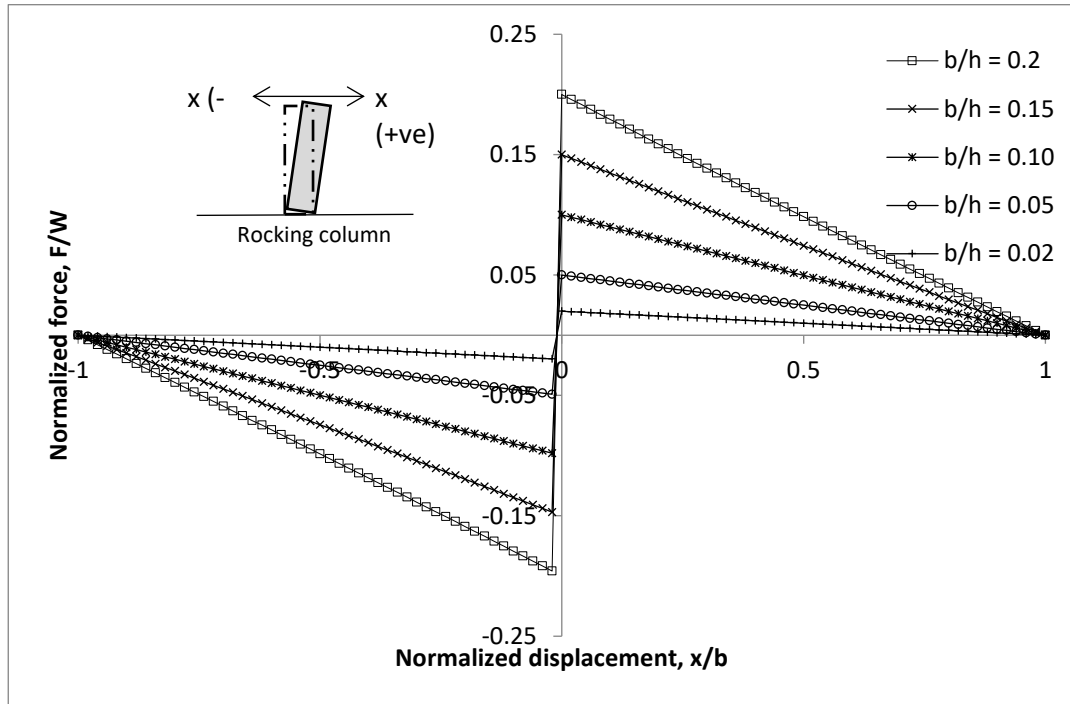


Figure 4.4 Linear approximation of F(x) vs. x relationship

4.1 Definition of the numerical model for SAP2000 software

As the rocking behavior is less understood among the community of civil engineers; therefore, the software developers put less efforts to incorporate the rocking behavior module in the commercially available finite element analyses frameworks. It is a tedious job to develop the numerical model of the rocking behavior which can also be utilized within the commercially available software. In this research, a simple numerical model is proposed which can be easily modelled in the already available

finite element software like SAP2000. Figure 4.5 shows the fundamental diagram of the free rocking column model.

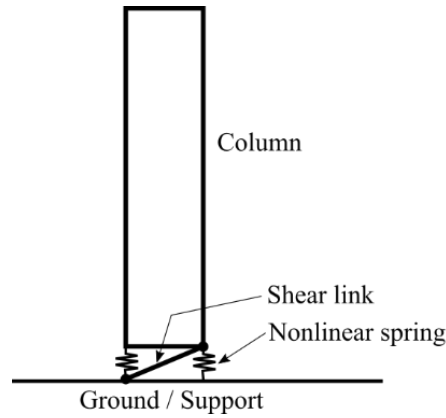


Figure 4.5 Numerical model of free-standing rocking column

In this model, the column is made up of concrete or steel. The bottom corners of the rectangular column (about which the column is assumed to rock alternatively) are attached with the ground with the help of nonlinear springs. The nonlinear spring is very stiff under compression and very flexible under tension. In this way, the spring allows the corner to uplift but does not allow the corner to sink. The rocking model is incomplete without the provision of a shear key or shear link, because in the absence of the shear link, the column may slide infinitely. The shear link is mandatory to prevent slip of the column and to transfer necessary shear forces. Any stiff prismatic truss element which does not deform axially during the earthquake loading may serve the purpose of the shear link. Hence the axial deformation of the shear link is negligible. In actual scenario, both ends of the column experience rocking motion; therefore, it is necessary to replicate the same rocking model at the top of the column as well. Figure 4.6 shows a column supporting a slab at top of it and experiencing rocking at both ends.

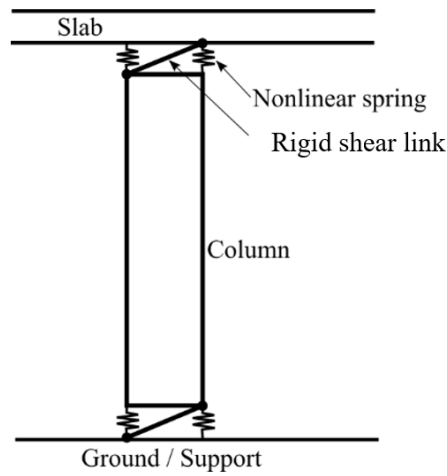


Figure 4.6 Numerical model of a rocking column supporting a slab on its top

The same rocking behavior may be obtained with four different combinations of the shear links. Two of them yield the similar stress (either tension or compression) in both shear links, and the remaining two combinations yield the opposite stress (tension in one end and compression in the other, or vice versa) in the shear links. The four possible combinations are drawn in Figure 4.7.

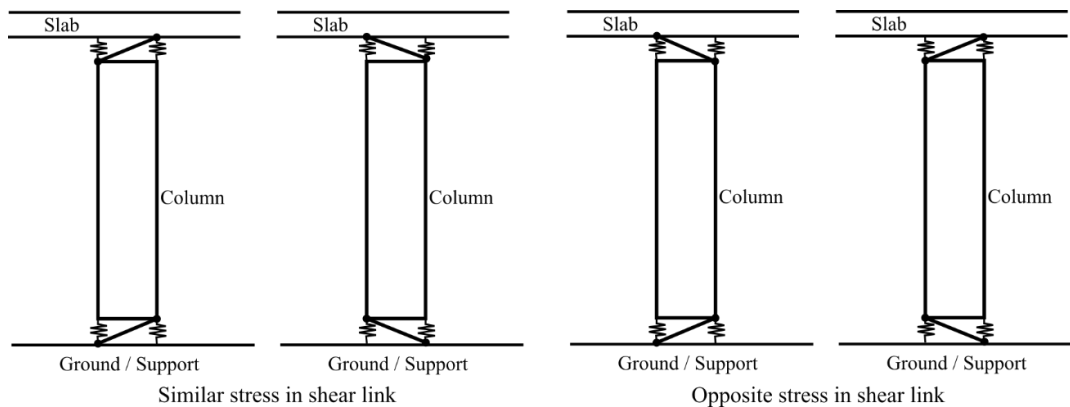


Figure 4.7 Different possible configurations of shear links in the numerical model of rocking column

Let us consider an example to explain further the direction of stress (either tension or compression) in the shear links. Consider the last combination shown in Figure

4.7 with opposite stress directions in the shear links. All possible six phases of rocking are drawn (in Figure 4.8) for these combinations.

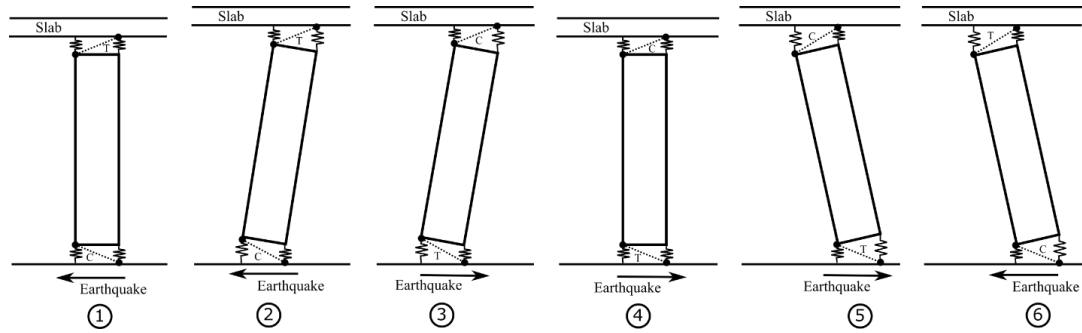


Figure 4.8 Different phases of rocking and the corresponding stresses (either tension or compression) in the shear links.

Phase 1 is the rest state of a rocking column (rest at both ends). When an earthquake hits the column (coming from right to left), the bottom shear link will experience the compression, and the top shear link will experience the tension. These shear links are shown with dotted lines and compression is indicated with 'C' and tension with 'T'. If the earthquake acceleration is so high that it fulfills the necessary condition to initiate the rocking, then the rocking column will uplift from one end (left) while rock about the other end (right). At phase 2, the bottom shear link is under compression and top shear link is under tension. Suppose that the direction of earthquake has changed, so the direction of stresses in shear links will also change as shown in Phase 3. Due to reversal of earthquake direction, the column will try to come back to its original position, as shown in Phase 4. At Phase 4, the bottom shear link is under tension and top shear link is under compression and same is the case with Phase 5. At Phase 5, right bottom end of the column is uplifted and column is rocking about left bottom end. If the earthquake changes its direction, then again, the direction of the stress in shear links will be the same as that in the phase 1. In this example, at any possible stage of an earthquake, the stress direction in the shear links (top and bottom) are opposite. There are only two possible shear link combinations with opposite stress directions. However, the reader can imagine the case with similar stress directions in the shear links as well.

4.2 Role of the gap length

The nonlinear spring element used to define the rocking column model introduces a gap between the ground and bottom of the column, and between the slab and top of the column. The length of the gap may cause little slide or slip of the column during rocking motion, as shown in Figure 4.9.

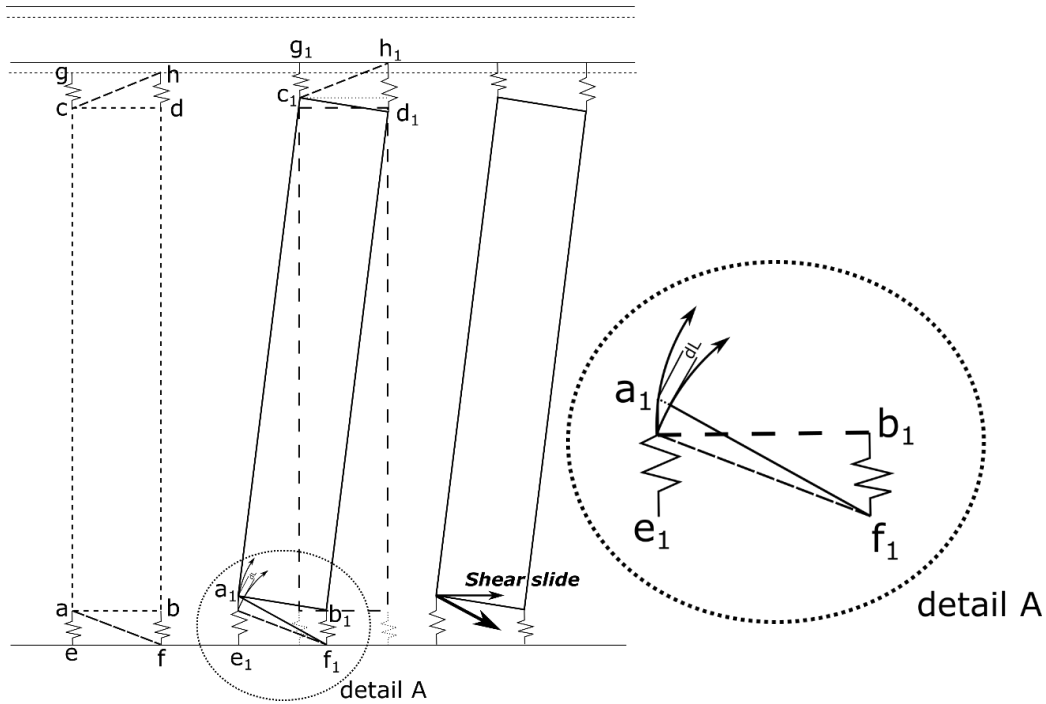


Figure 4.9 Shear slide at the ends of the rocking columns due to the high axial stiffness of the shear link can be ignored.

In the Figure 4.9, a rectangular column $abcd$ is attached with ground and upper slab with four nonlinear spring elements; ea , fb , cg , and dh . Two shear link elements, fa and dg , are attached in such a way that similar stresses will be developed during rocking. Let us suppose that the column has started rocking about point b and all the points have changed their positions as shown with subscripts 1. After uplift of corner a , the length of the shear link may increase from f_a to f_{1a1} , such that $f_{1a1} = f_a + dL$. As shear link is very rigid, this will not happen and shear link will apply a force on

the column to slide or slip. A similar force will also act at the top of the column due to the shear link. Smaller the length of the spring element, smaller will be the effect of additional slide or slip. Smaller length of nonlinear spring element will filter out the effect of additional shear slide or slip of the rocking column.

4.3 Extension of 2D rocking model concept to make 3D rocking model

The concept of 2D rocking model is extended to develop 3D model of the rocking column. The 3D model also employs the basic concept of using nonlinear spring elements and shear links. Finite element software like SAP2000 are capable of developing the proposed rocking model. The step by step formation of the 3D rocking model is shown in Figure 4.10. Figure 4.10(a) shows four nonlinear links which are rigid under compression and flexible under tension in along its axis. The stiffness of the spring in two other lateral directions is explained in Figure 4.11. For the 3D rocking model, four corner springs are required in contrast to the 2D model, where two springs are sufficient to define the model. The next stage of the development of the model is modeling two shear link elements in the xz as shown in Figure 4.10(b & c). Figure 4.10 (d) shows the modelling of frame elements so that the deformations of the springs remain in one plane. In Figure 4.10 (e,f,g, and h), the same model is replicated in the upper layer for the rocking in yz plane. This model is capable of producing rocking in the xz plane, yz plane, or in both planes. The model is also capable of rocking about the corner points. The stiffnesses of the nonlinear spring elements is explained in Figure 4.11. The springs at the bottom layers are fixed at the base against 6DOF.

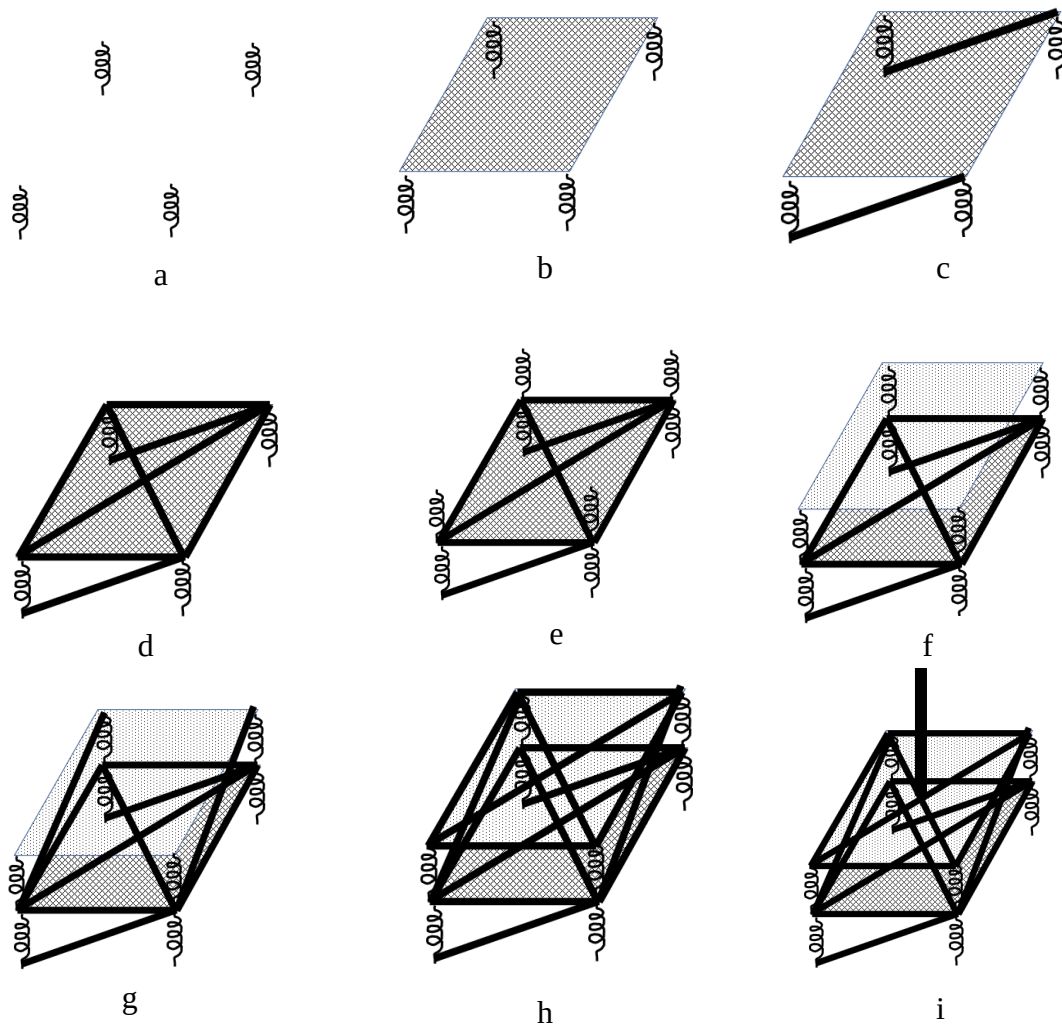


Figure 4.10 Step by step formation of 3D model of rocking column

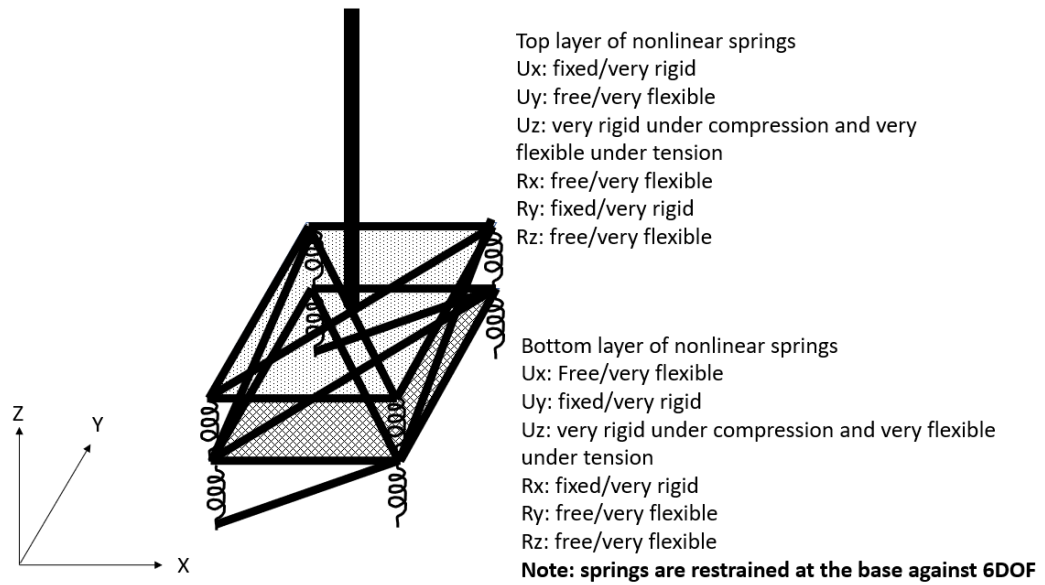


Figure 4.11 3D rocking model, DOF's are explained for the bottom layer of the springs and top layers of the springs.

CHAPTER 5

EXPERIMENTAL VALIDATION OF SAP2000 MODEL

The next step following the development of numerical model of the rocking column in SAP2000 is its experimental validation. The experiments performed for the validation of rocking column model are conducted in two phases. The following paragraph briefly overviews these two phases of the experiments.

5.1 Overview of the experiments

Two phases of the experiments are performed to validate the SAP2000 rocking column model. The first phase of experiments corresponds to the prototype of a free rocking wooden column. In the free rocking of a solitary wooden block, the free rocking tests are performed with and without additional weights at the top. The free vibration tests are repeated for different initial rocking conditions and the corresponding models are prepared in SAP2000. The comparison of the SAP2000 rocking model results and experimental observations have validated the proposed rocking model. Second phase of experiments conforms to the forced rocking of a simple podium like prototype. The podium prototype consists of a roofing board supported by four wooden columns at its four corners. The wooden columns are connected with the roofing board with the help of particular hinges which allow the structure to rock. Manual vibrations are induced at the base to rock the prototype. Accelerations at the base and the roof level are observed with the help of accelerometers. The experiments are repeated with different ground excitations (which are manually produced) and the corresponding models are developed in SAP2000.

5.2 Free rocking of a solitary wooden column

The free rocking of a solitary wooden column consists of a single wooden column prototype allowed to rock freely on glass and laminated floor surfaces. The column may or may not carry additional weight at the top. The rocking column is displaced at the top (not exceeding the overturning displacement) to initiate the rocking motion. An accelerometer is installed at the top of the column to record acceleration response history. The geometric properties and weight of the wooden column is presented in Table 5.

Table 5 Geometric properties and weight of the wooden column.

Width (b)	Depth (d)	Height (h)	Weight of column	Weight of accelerometer	Additional weight
7.8 cm	3.9 cm	67.5 cm	920 gm	40 gm	2716 gm

Figure 5.1 shows an image of the free rocking column prototype, indicating geometric properties (b, d, and h), additional weight, and the position of accelerometer. Four different experiments are performed under the phase of free rocking of solitary column. There are four sub categories of the free rocking column experiments. The set of first and second experiments correspond to the free rocking of column without additional weight. Whereas third and fourth experiments correspond to the rocking of column with additional weight at its top. The free rocking tests of the solitary columns (with and without additional weight) are repeated on the glass surface and the laminated floor surface.

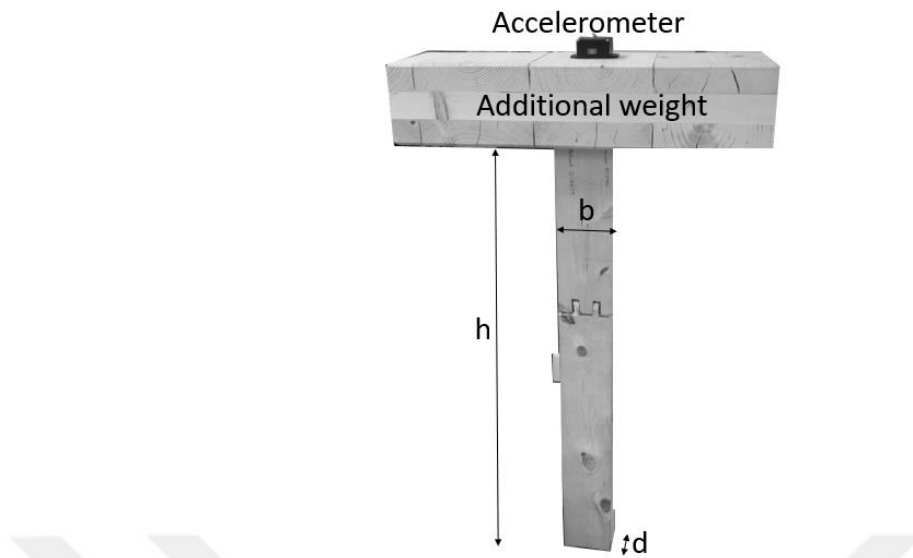


Figure 5.1 Free standing wooden column with additional weight at the top.

A typical acceleration response history of the free rocking column is shown in Figure 5.2. The rocking starts with the initial top displacement of the rocking column and releasing it to rock freely. The spikes at specific intervals in the acceleration response history represent the impact of the rocking column. The acceleration changes its direction after each impact. The interval between two consecutive impacts may be called as the “half-period” of the rocking motion. It is clear from Figure 5.2, that the half-period of the rocking column is not constant, but it decreases with time. The decrease in half period after each impact is because some energy is dissipated after each impact. Figure 5.2 also shows the zoomed in graph of the acceleration between two consecutive impacts. The typical shape of the variation of the acceleration between two consecutive impacts is like a parabola. As the magnitude of acceleration in between two consecutive intervals is large as compared to its variation; therefore, the acceleration between two consecutive impacts may be considered as constant. Hence, the magnitude of acceleration within a half-period is almost constant. The half-period acceleration magnitude of twelve different experiments is enlisted in the Table 6.

Table 6 Approximately constant half-period acceleration values observed from acceleration response histories of different experimental triggers.

Trigger number	1	2	3	4	5	6	7	8	9	10	11	12
Simplified constant acceleration (g)	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.65	0.65	0.65	0.65	0.65

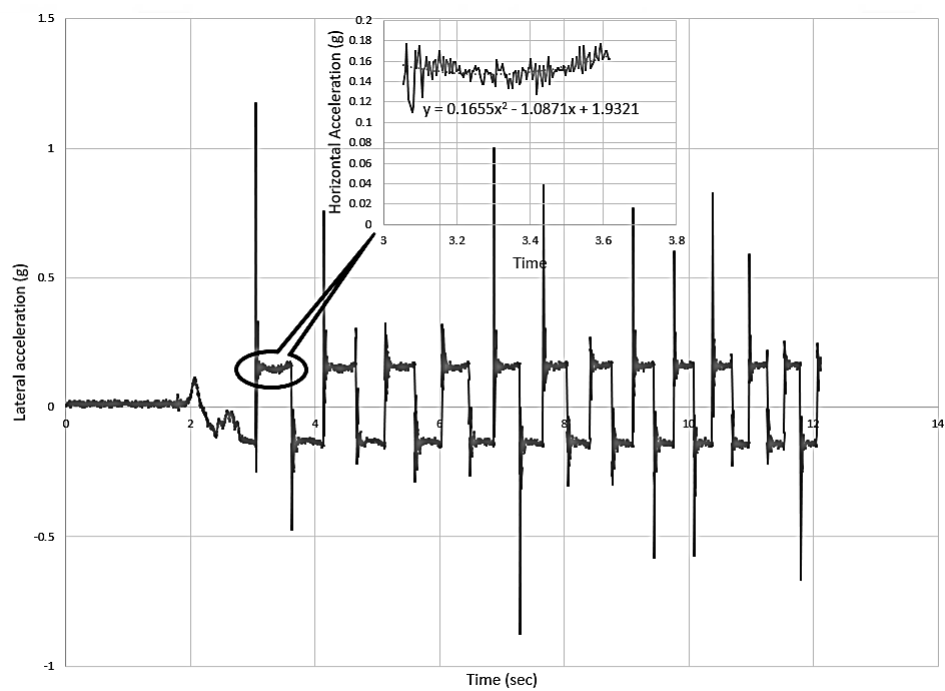


Figure 5.2 A typical acceleration response history of a free-standing rocking column observed from the accelerometer and zoomed-in image of a half-period.

The acceleration response histories of different experiments are shown in Figure 5.3. As discussed earlier, the magnitude of acceleration is almost constant within a half-period, and the rocking motion continues with the same magnitude of acceleration (alternating its direction after each impact). After maintaining almost the same acceleration magnitude (either positive or negative) throughout the response history,

the acceleration magnitude suddenly drops to zero at the end of the rocking motion. The first row of Figure 5.3(a and b) shows the rocking of wooden column prototype without additional mass whereas the second of row of Figure 5.3(c and d) shows the rocking of wooden column prototype with additional mass at the top. Part (a and c) represent the rocking on a glass base and part (b and d) represent the rocking on the laminated floor base. It is observed from the response histories that the average constant acceleration is decreasing due to increase in the additional weight at top of the rocking column. Furthermore, the nature of rocking surface (glass or laminated floor) also affects the rocking behavior of the wooden prototype.

Figure 5.4 shows the half-period accelerations (almost constant) obtained from the experimental observations and accelerations calculated from SAP2000 simulations. The value of half-period acceleration is about 0.15g and 0.065g for the rocking columns without and with additional weights at the top respectively. The same values are obtained from the SAP2000 analyses results. The same values of half-period accelerations obtained from the experimental observations and SAP2000 results validate the proposed numerical model for the rocking column.

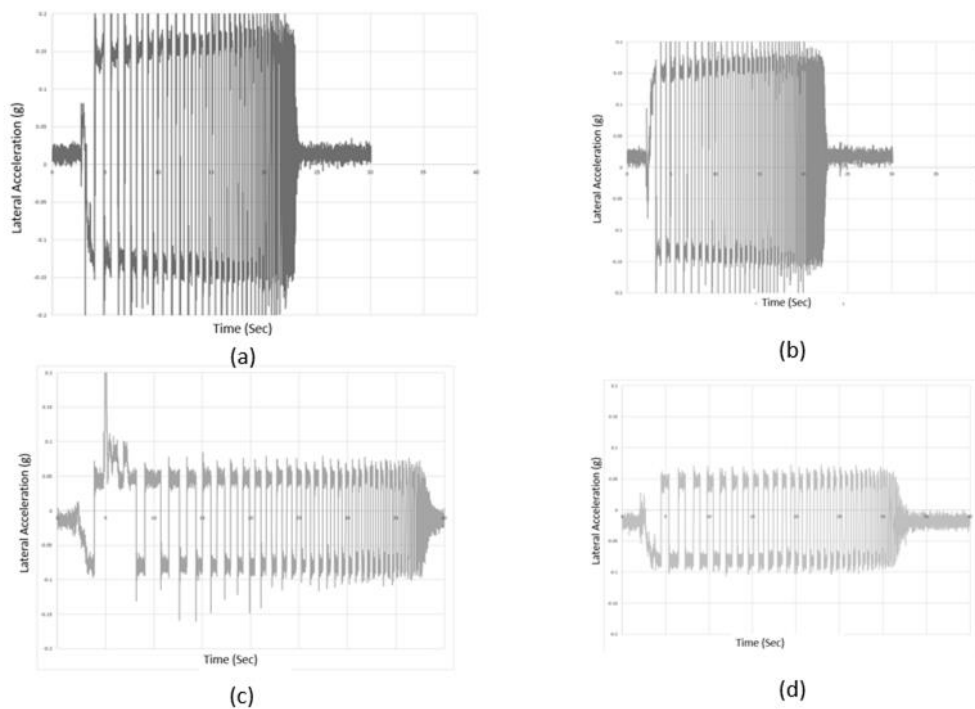


Figure 5.3 Acceleration response histories of the free rocking wooden prototype (a) without additional weight at top and glass base, (b) without additional weight at top and laminated floor base, (c) with additional mass and glass base, (d) without additional mass

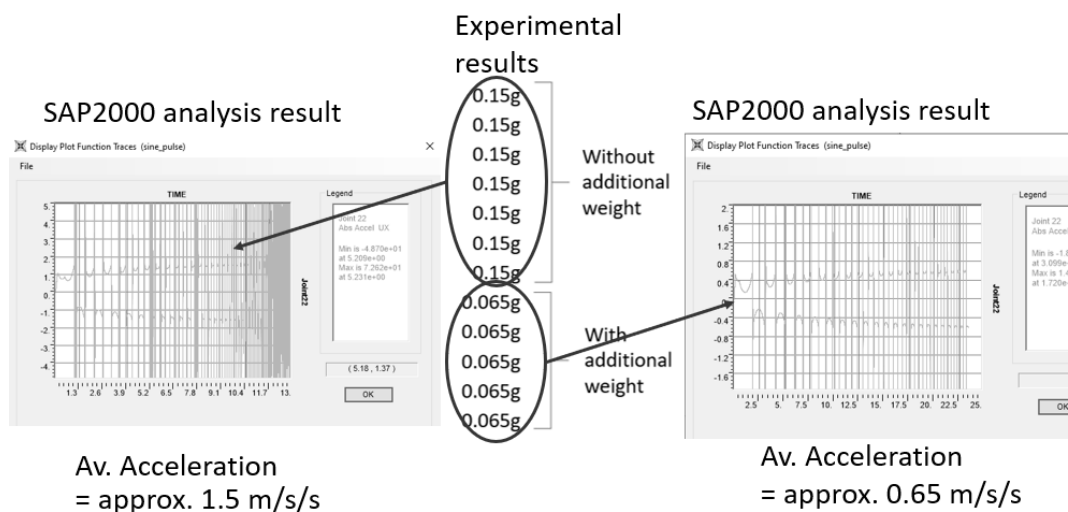


Figure 5.4 Match of experimental observations vs. SAP2000 results in terms of half-period acceleration

The experimental results and the SAP2000 analyses results are also compared in terms of variation in half-period, with time, to validate the proposed rocking model further. The amplitude and the half-period of the rocking vibration of a free rocking column decreases after each impact. The decrease in half-period after each impact is observed from the experiments and it is plotted for five different experimental triggers in Figure 5.5. Trigger 01 to trigger 05, shown in Figure 5.5, belong to the experiments of rocking columns without additional weight at the top and rocking on the glass base. All the results, irrespective of the initial condition of rocking (initial amplitude of rocking displacement), and total number of impacts, follow the same path on the number of impacts vs. half period graph. It shows that the experiments are repeatable and results are reliable to validate the rocking model. The numerical model of the same experimental triggers (01 to 05) are generated in SAP2000 and shown with the help of a red colored line in Figure 5.6.

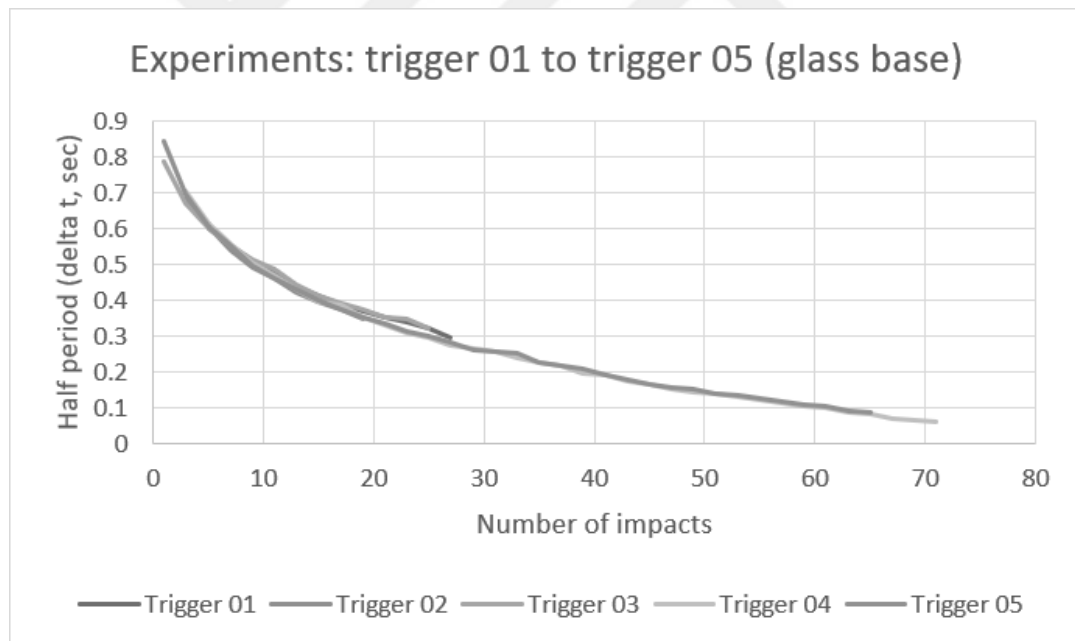


Figure 5.5 Experimental observations of decrease of half-periods with each impact. The results of the SAP2000 simulation results match quite well with that of the experiments (Figure 5.6). It is observed that the SAP2000 simulation results are affected by the integration time step and value of α defined in the Hilber-Hughes-Taylor- α (HHT- α) method (see Figure 5.7). In Figure 5.7, the value of α decreases

from left to right ($-1/12$, $-1/6$, and $-1/3$), and the value of integration time step is increases from top to bottom ($1/1000$ sec, $1/500$ sec, and $1/200$ sec). The best match with the experimental results is for integration time step equal to $1/200$ sec and α -value equal to $-1/3$. The SAP2000 simulations become unstable (for this particular case) for integration time step equal to $1/200$ and α -value equal to $-1/12$, as shown in Figure 5.7.

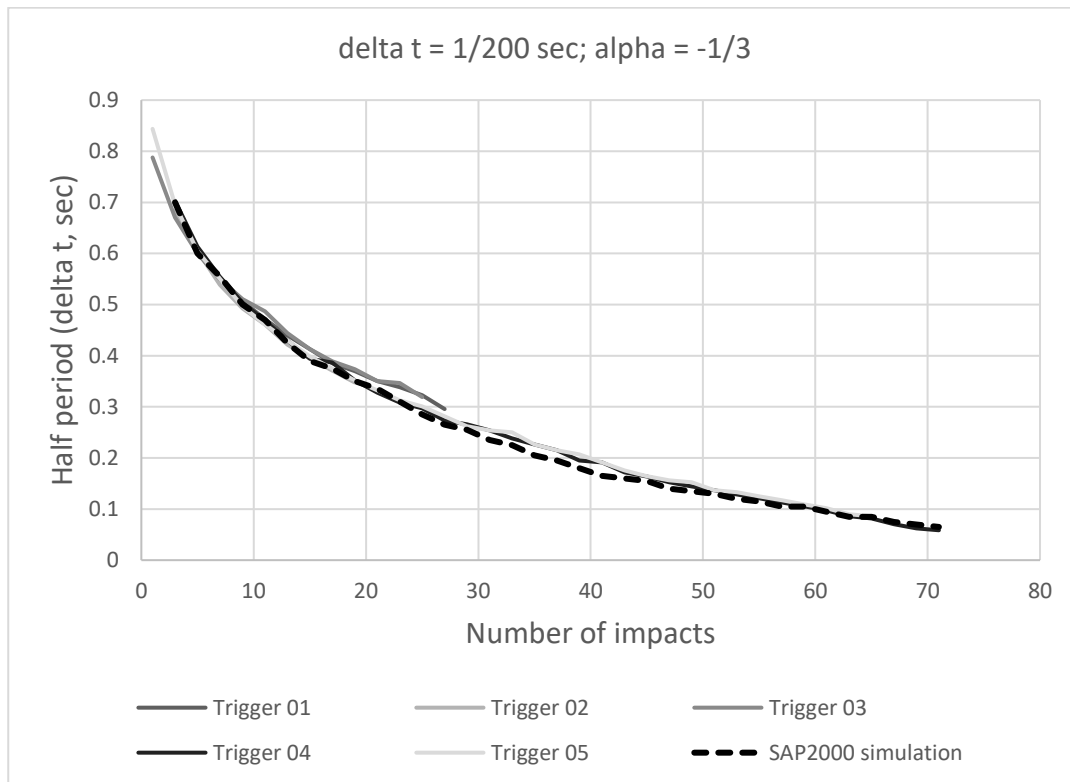


Figure 5.6 Free rocking of the wooden prototype without additional weight at the top; match of experimental results with that of SAP2000 simulations for the α -value of $-1/3$ and integration time step of $1/200$ sec.

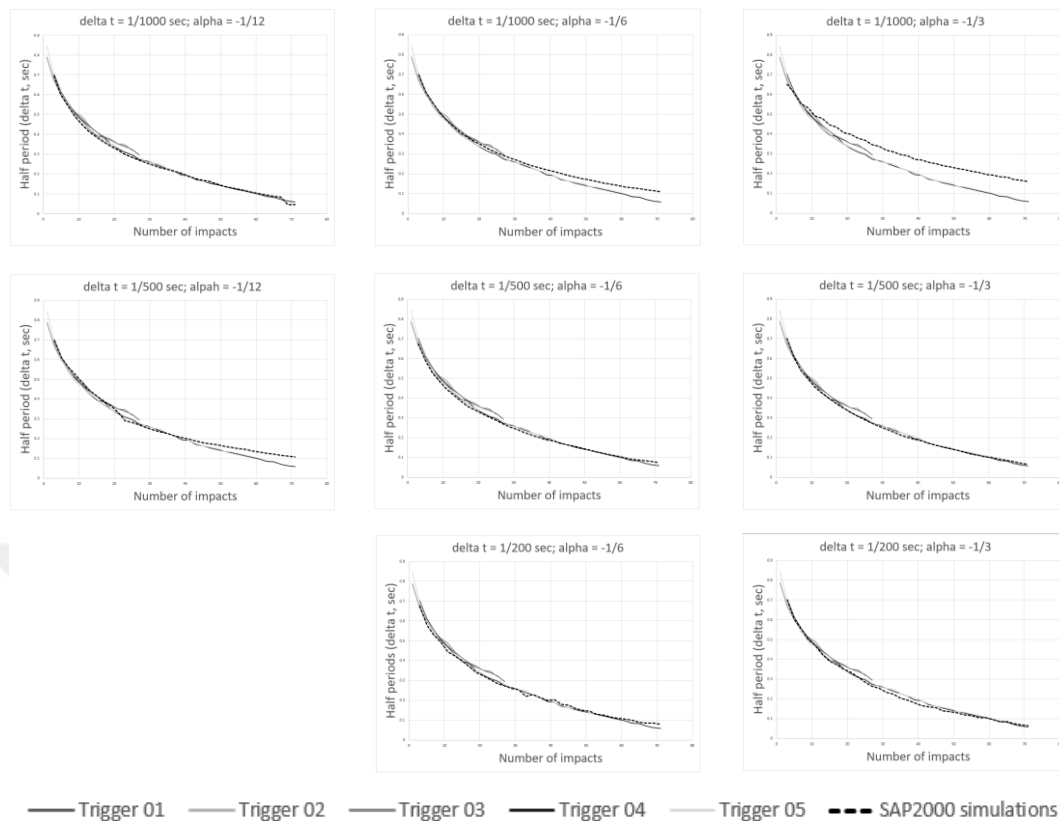


Figure 5.7 Free rocking of the wooden prototype without additional weight at the top; effect of integration time step and α -value on the SAP2000 simulation results. α -value is decreasing from left to right and integration time step is increasing from top to bottom

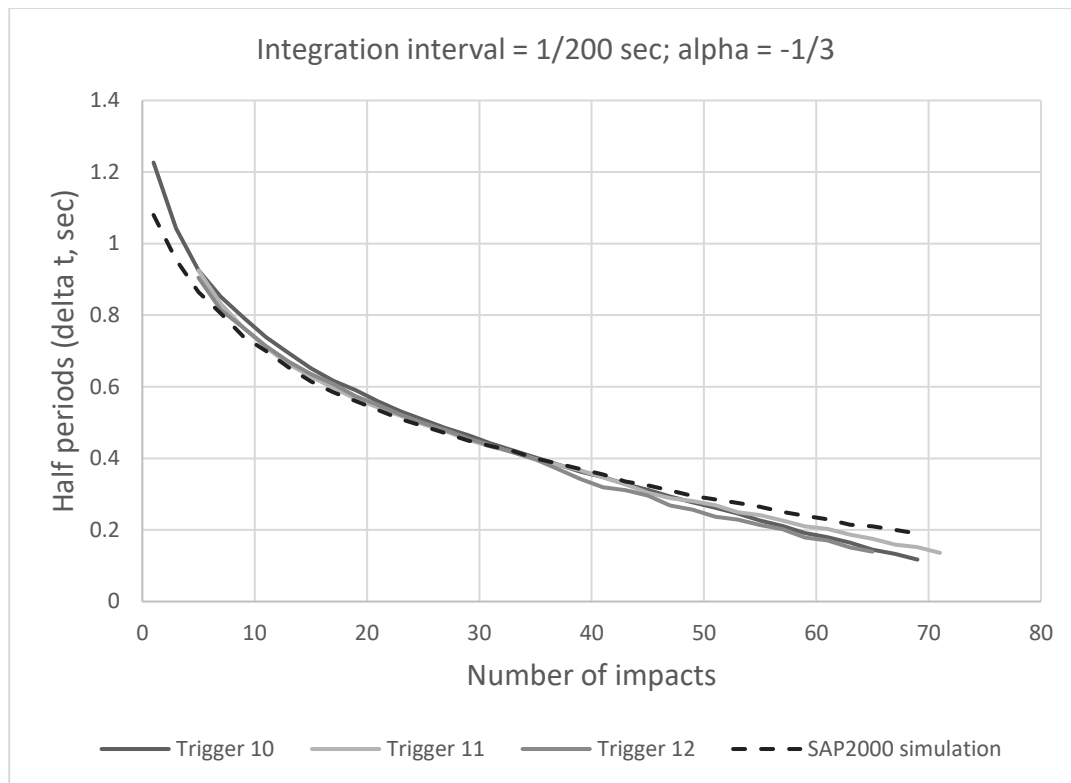


Figure 5.8 Free rocking of the wooden prototype with additional weight at the top; match of experimental results with that of SAP2000 simulations for the α -value of $-1/3$ and integration time step of 1/200 sec.

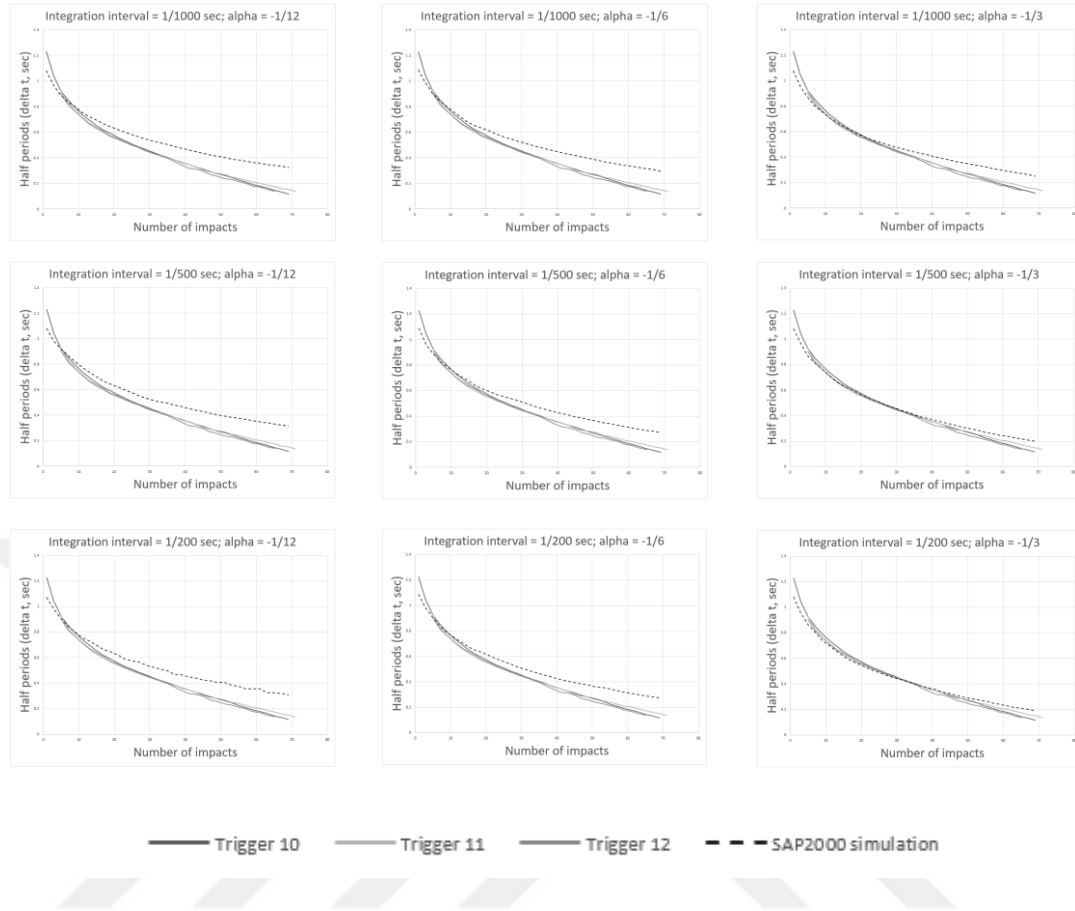


Figure 5.9 Free rocking of the wooden prototype with additional weight at the top; effect of integration time step and α -value on the SAP2000 simulation results. α - value is decreasing from left to right and integration time step is increasing from top to bottom.

As the α -value decreases ($-1/12$ to $-1/3$), the results become numerically stable, and the curve also fits well with the experimental results for small values of integration time intervals. Hence, for the future experiments, the α -value of $-1/3$ is used, and very small intervals for the time integration are avoided to minimize accumulated errors. Figure 5.8 shows the match of the wooden prototype experimental results with that of the SAP2000 simulations, and Figure 5.9 shows the effect of α -value and integration time step on the results of SAP2000 simulations.

5.3 Rocking of a single-story podium prototype

The second phase of the experimental work consists of the rocking of a single-story podium prototype. It is a simple structure in which four columns at its corners support roofing board. The dimensions of the roofing board is 48.1 x 60.2 cm². The rocking columns are made up of wood having a rectangular cross-section of 5.0 cm x 5.0 cm. The connections between the roofing boards and the columns are made so that the columns may exhibit the rocking behavior. Particular hinges are made in the laboratory of the Civil Engineering Department, METU, to allow the rocking of the structure. The detailed drawing of the experimental setup used for the second phase of experiments is shown in Figure 5.10.

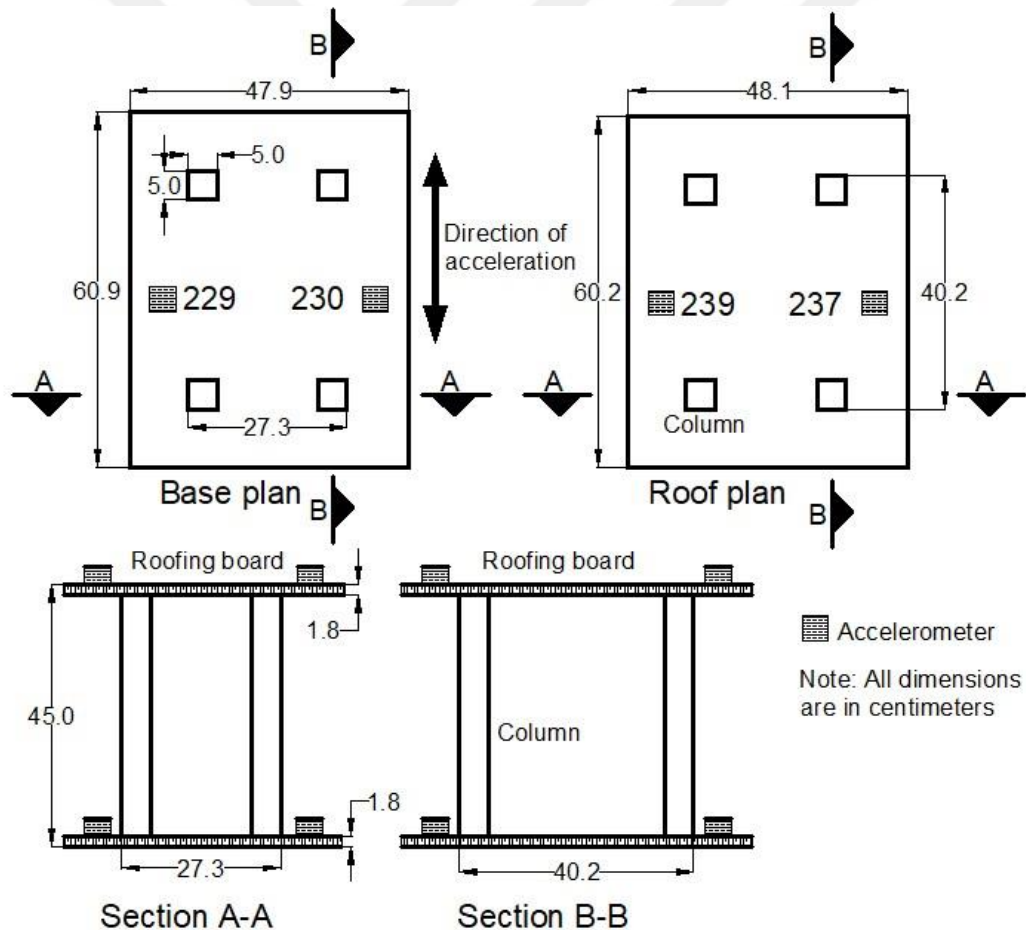


Figure 5.10 Experimental setup for the experiments of rocking podium structure.

A manual ground shaking is applied at the base board of the prototype. The accelerations at the base and the roof are observed with the help of the accelerometers for each manual ground shaking. Figure 5.11 to Figure 5.15 show the manual ground excitations induced to the prototype and the accelerations observed at the roof level. The significant difference of the magnitude of ground acceleration at the base and the roof level confirms the proposed idea of ROC seismic isolation system. Instead of whole base acceleration history, a segment of the base acceleration is selected for the numerical analyses in Sap2000. For example, in trigger 02 (Figure 5.11), instead of whole acceleration history of 40 sec, a segment of time history from 8 sec to 29 sec is selected for the validation of SAP2000 model. Similarly, time history segments from 7 to 14 sec, 13 to 20.5 sec, 17 to 31 sec, and 5 to 29 sec are selected for trigger 03 (Figure 5.12), trigger 04 (Figure 5.13), trigger 05 (Figure 5.14), and trigger 06 (Figure 5.15) respectively.

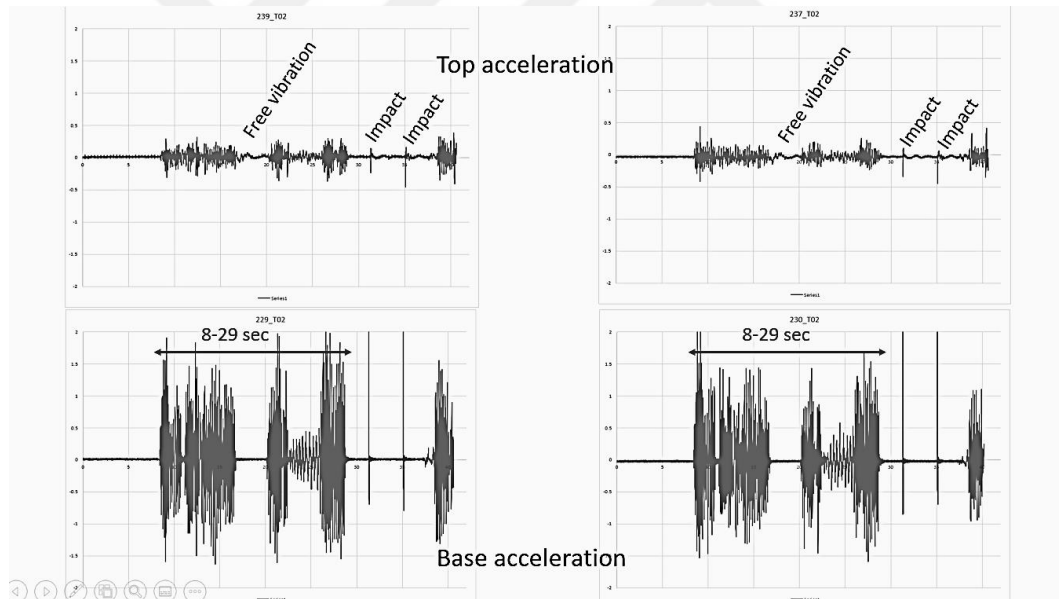


Figure 5.11 Base and roof accelerations of the rocking podium structure; Trigger 02

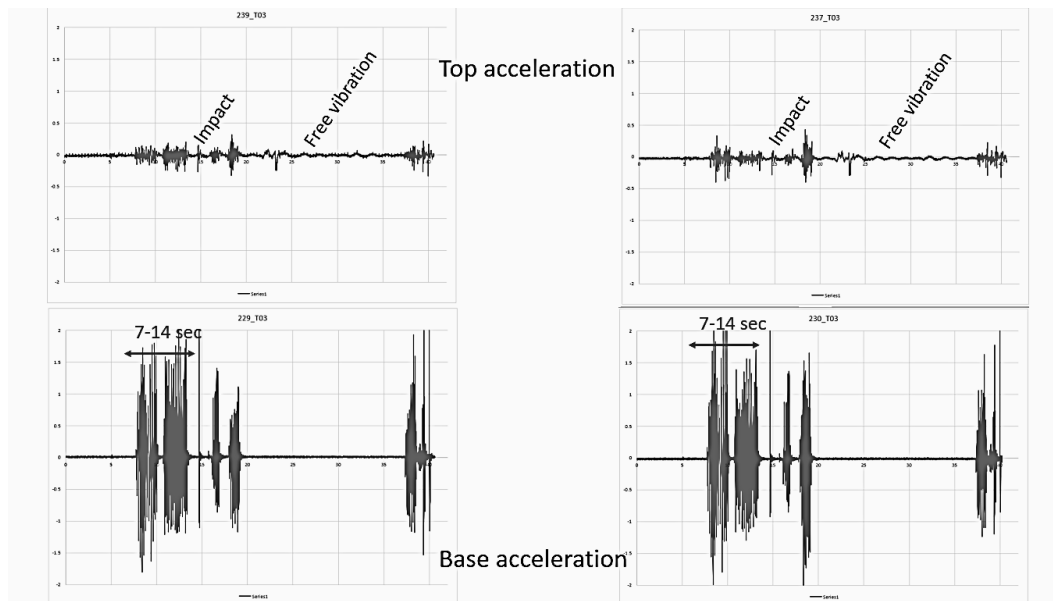


Figure 5.12 Base and roof acceleration of rocking podium structure with additional mass; Trigger 03

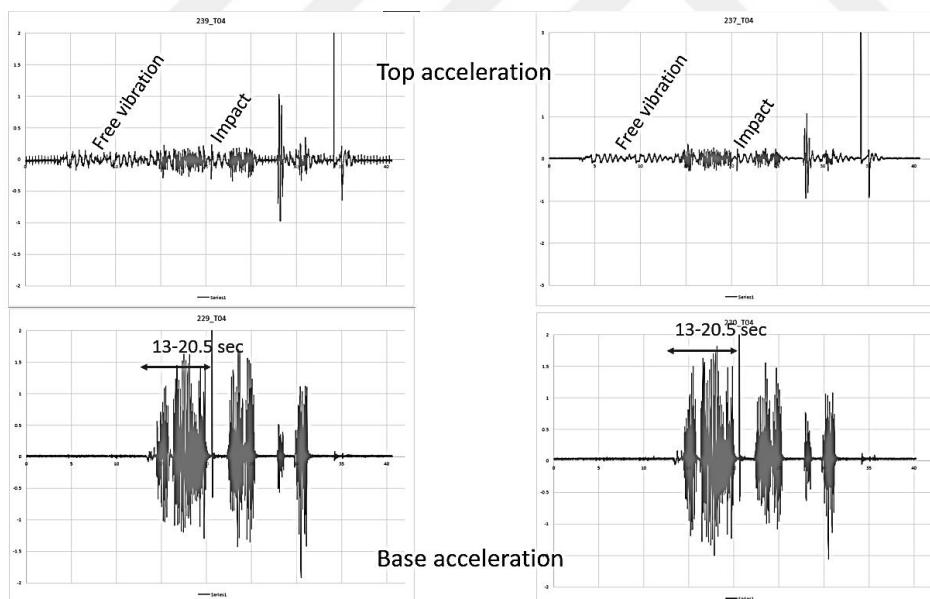


Figure 5.13 Base and roof acceleration of rocking podium structure with metallic rocking surface; Trigger 04

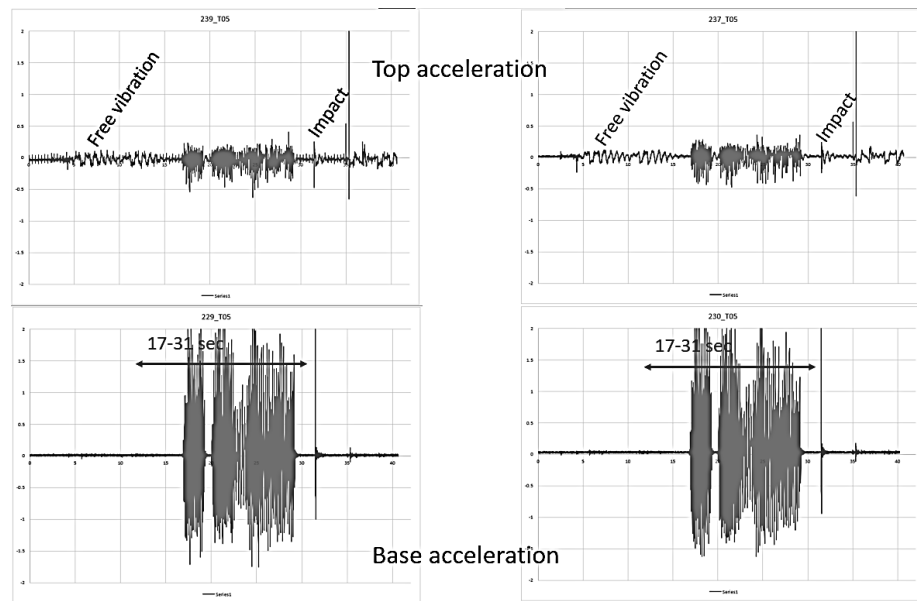


Figure 5.14 Base and roof acceleration of rocking podium structure with metallic rocking surfaces and additional mass at the top; Trigger 05

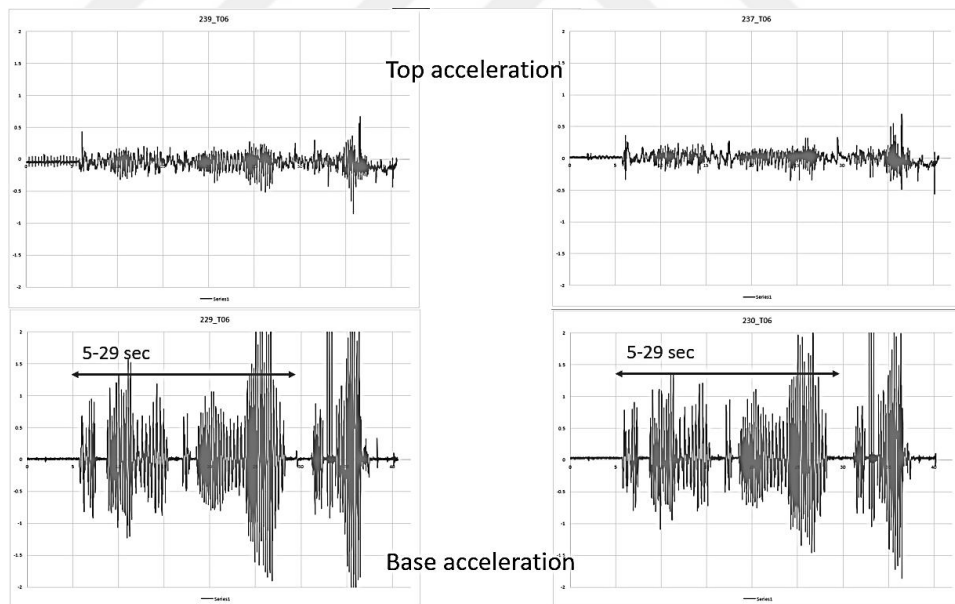


Figure 5.15 Base and roof acceleration of rocking podium structure with metallic rocking surfaces and additional mass at the top; Trigger 06

The Fast Fourier Transform (FFT) analyses are executed on the selected segments of acceleration time histories. The results of the FFT are shown in Figure 5.16. The

segments selected for the input ground acceleration of SAP2000 are analyzed to get an idea about the frequency range of the input ground acceleration.

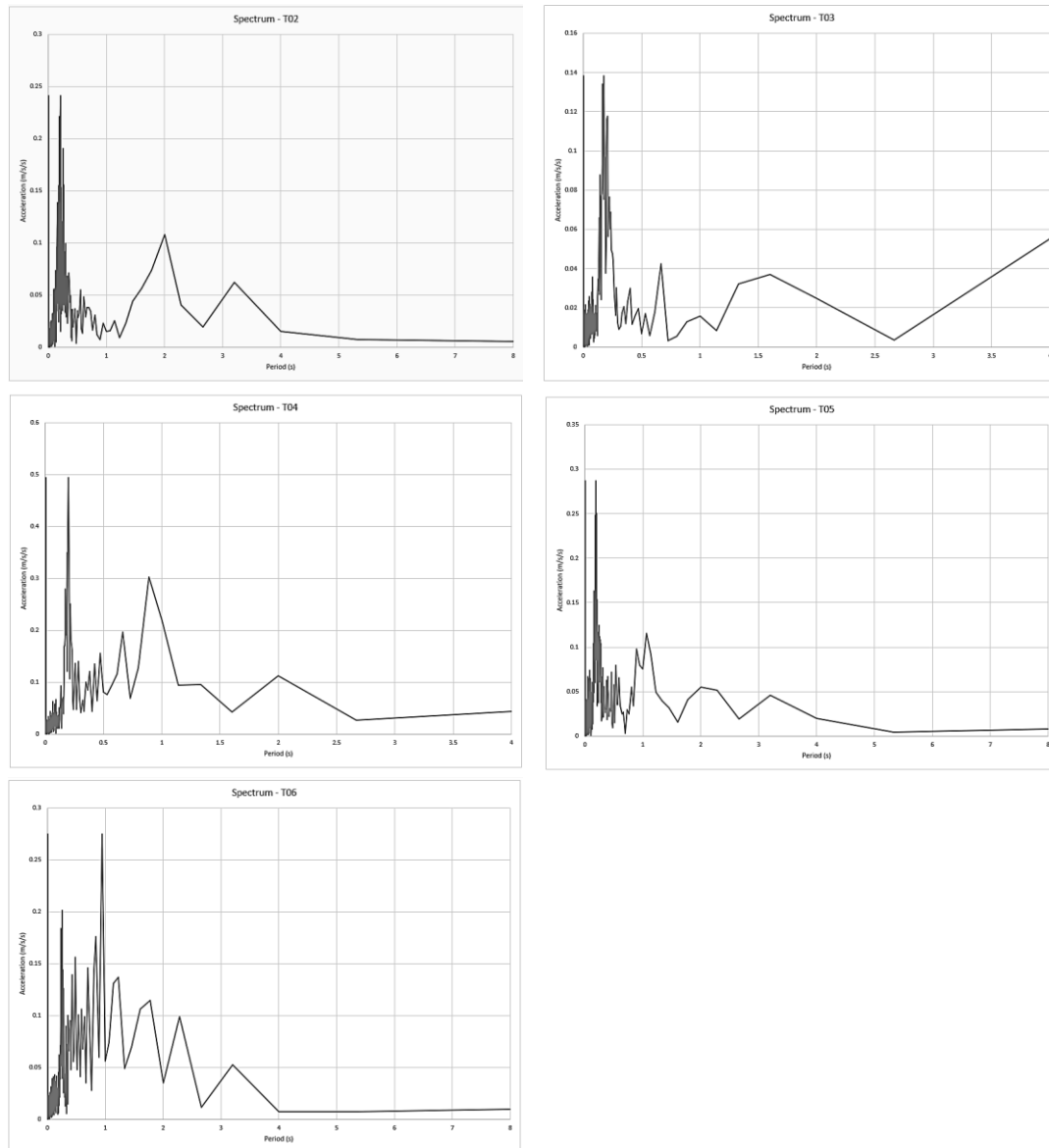


Figure 5.16 Fast Fourier Transform analyses results of the input ground accelerations.

For the experiments, trigger 04, trigger 05, and trigger 06, metallic pieces are glued at the rocking surfaces to achieve improved rocking behavior. All the experimental results are generated using the proposed rocking model in SAP2000 software. The

SAP2000 results are well matched with the experimental results, as shown from Figure 5.16 to Figure 5.20.

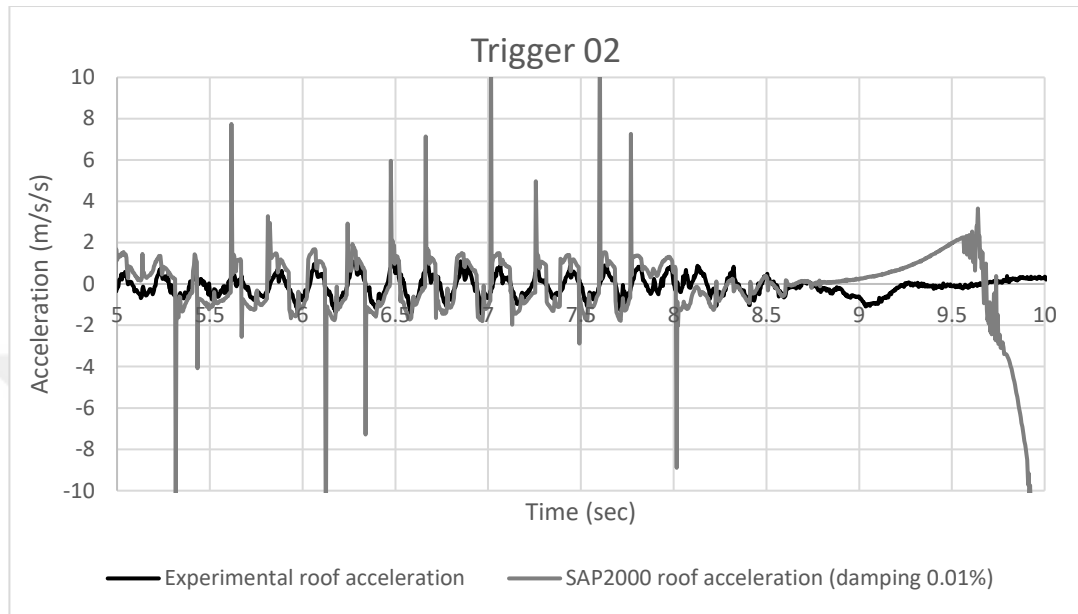


Figure 5.17 Comparison of roof accelerations; experimental vs. SAP2000 results; trigger 02

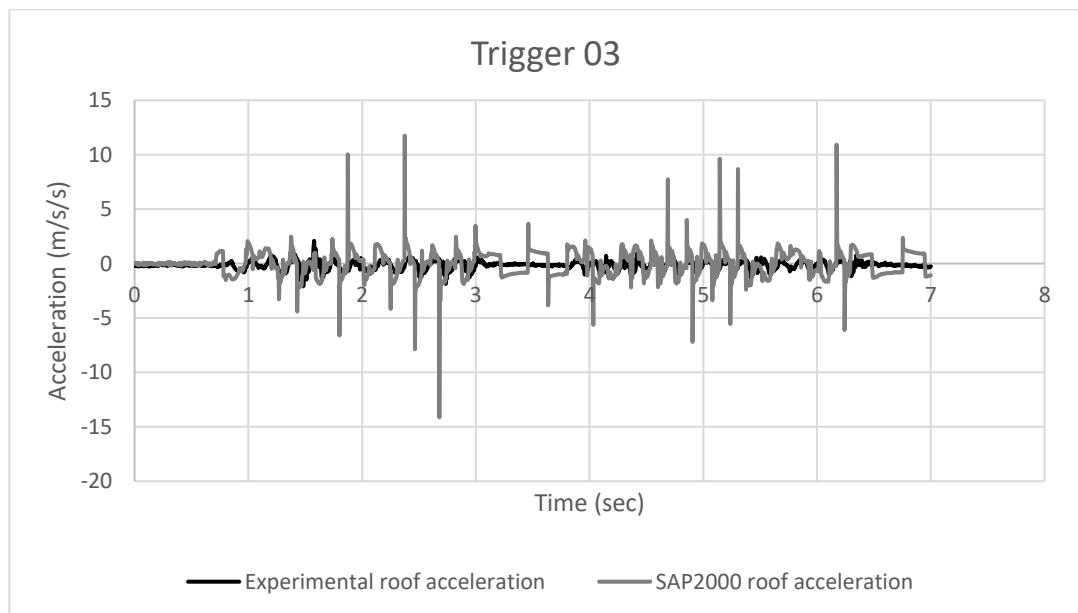


Figure 5.18 Comparison of roof accelerations; experimental vs. SAP2000 results; trigger 03

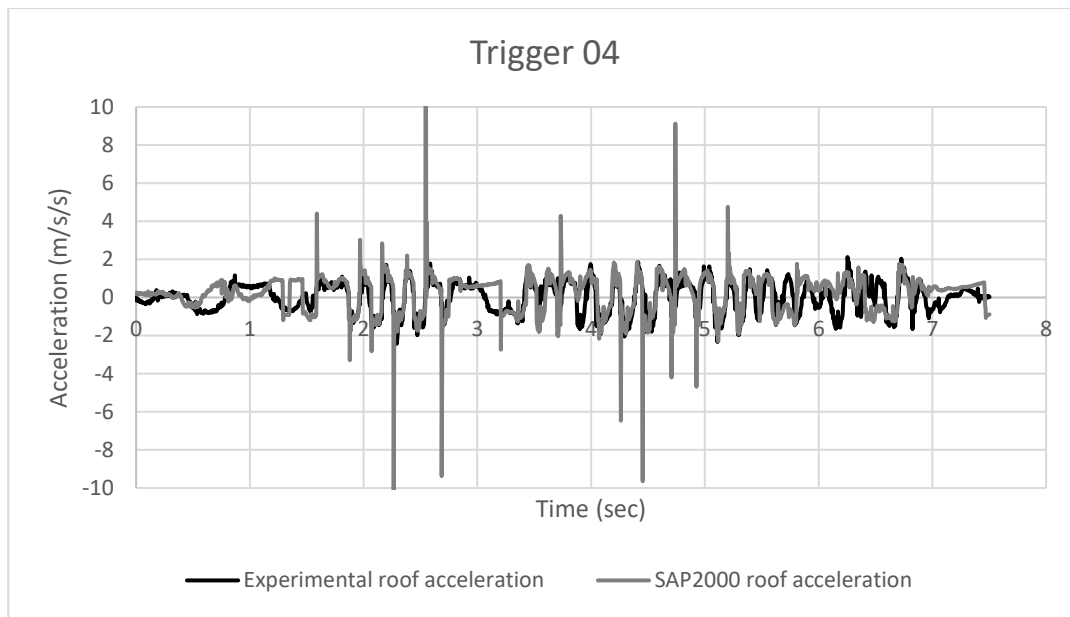


Figure 5.19 Comparison of roof accelerations; experimental vs. SAP2000 results; trigger 04

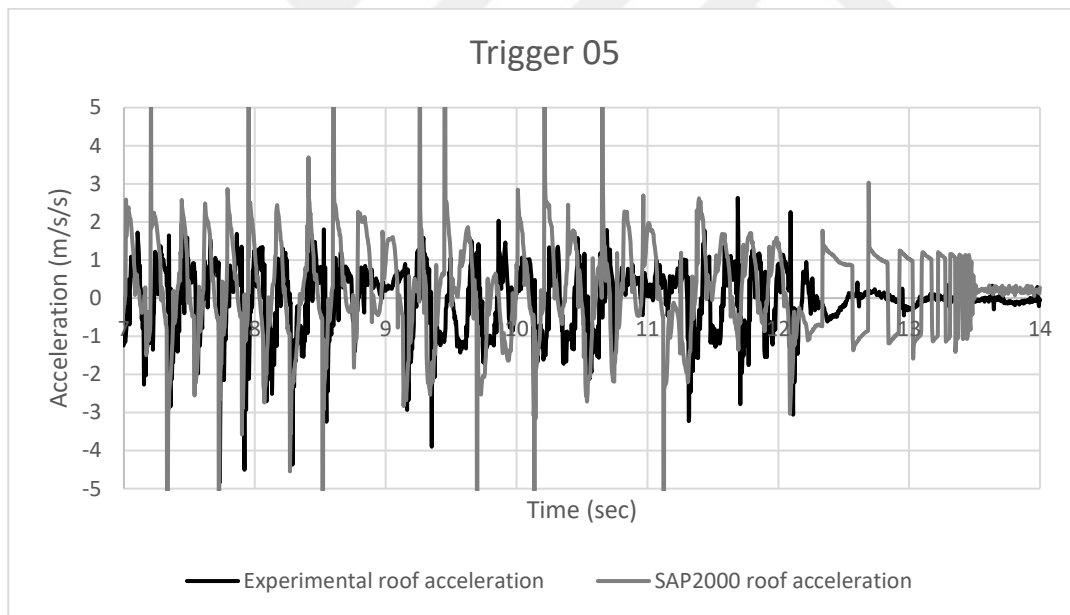


Figure 5.20 Comparison of roof accelerations; experimental vs. SAP2000 results; trigger 05

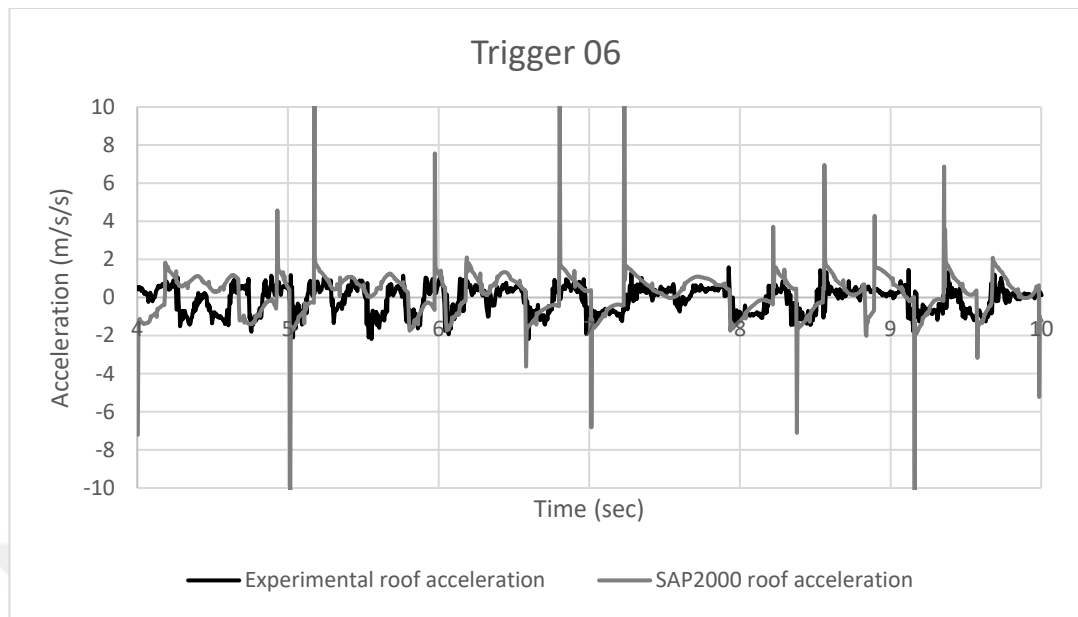


Figure 5.21 Comparison of roof accelerations; experimental vs. SAP2000 results; trigger 06

CHAPTER 6

FEM ANALYSES FOR DEVELOPING RESPONSE SPECTRA FOR ROC AND DISCUSSION

The primary objective of this part of the research is to perform the required analyses (both 2D and 3D) to have a better understanding of the ROC seismic isolation systems for medium rise residential buildings. The successful validation of the rocking model, defined in SAP2000, leads us to use this model for further investigation of the proposed system. The analyses are performed in two parts; 2D analyses and 3D analyses.

6.1 2D analyses

The purpose of 2D analyses is to compare the effectiveness of ROC seismic isolation systems (having 0.20m and 0.30m widths of the rocking columns) with conventional seismic base isolation systems and regular buildings without isolation systems. The conventional base isolation systems, considered for the comparison, are friction pendulum system (FPS) and laminated rubber bearing (LRB). Fifteen different numerical models are prepared to perform the 2D analyses in SAP2000.

Table 7 List of the numerical models used for 2D analyses in SAP2000

Normal	Mass irregularity	Stiffness irregularity
4Reg	4RegMi	4RegSi
4FPS	4FPSMi	4FPSSi
4LRB	4LRBMi	4LRBSi
4Rock20	4Rock20Mi	4Rock20Si
4Rock30	4Rock30Mi	4Rock30Si

These models are enlisted in Table 7 according to their nomenclature as defined in the chapter of Methodology. The results from the analyses are presented in the graphical forms from Figure 6.1 to Figure 6.24. These figures are divided into two super categories; first belongs to four story buildings and second belongs to seven story buildings. Each super category is divided into three groups (four figures for each group); first group is for normal buildings, second group is for buildings with mass irregularity, and group part is for buildings with stiffness irregularity. For example, Figure 6.1 to Figure 6.4 represent the first group of figures. In each group; first figure shows the inter-story drift ratio (ratio of inter-story drift to story height), second figure also shows the inter-story drift ratio but the first story is missing, third figure shows the story acceleration, and fourth figure shows the normalized story shears (concerning the total dead load of the building). The inter-story drift ratio, floor accelerations, and floor shears results are compared among the normal buildings, friction pendulum system (FPS) buildings, laminated rubber bearing (LRB) buildings, damped rocking column (ROC) system with rocking width of 0.20m and damped rocking column (ROC) system with rocking width of 0.30m.

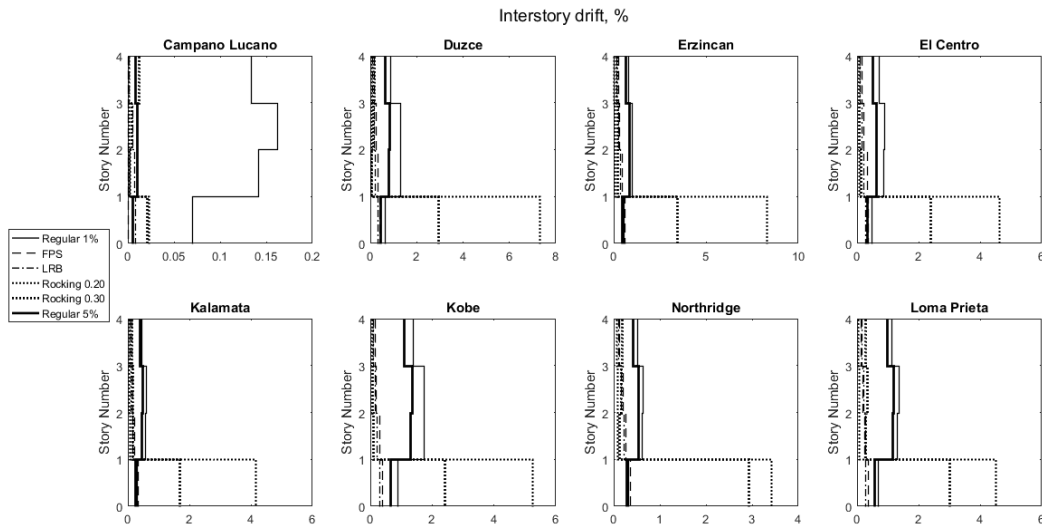


Figure 6.1 Comparison of interstory drift ratio of four-story buildings for regular, FPS, LRB, Rocking 0.20m, and Rocking 0.30m systems.

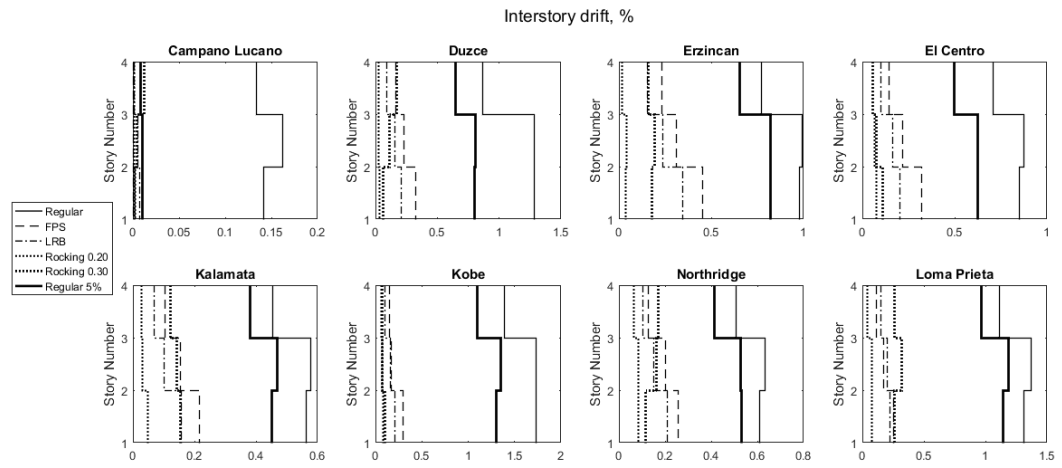


Figure 6.2 Comparison of interstory drift ratio of four-story buildings (except bottom story) for regular, FPS, LRB, rocking 0.20m, and rocking 0.30m systems.

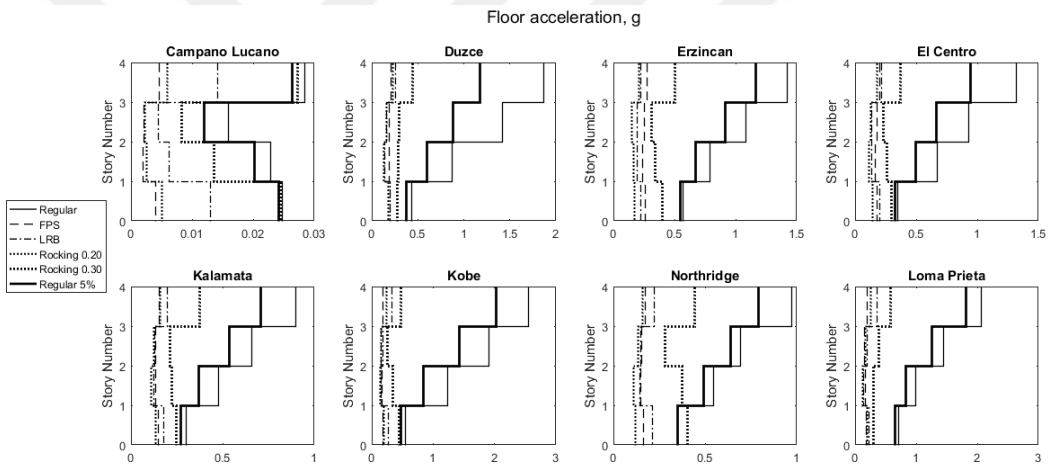


Figure 6.3 Comparison of story accelerations of four-story buildings for regular, FPS, LRB, Rocking 0.20m, and Rocking 0.30m systems.

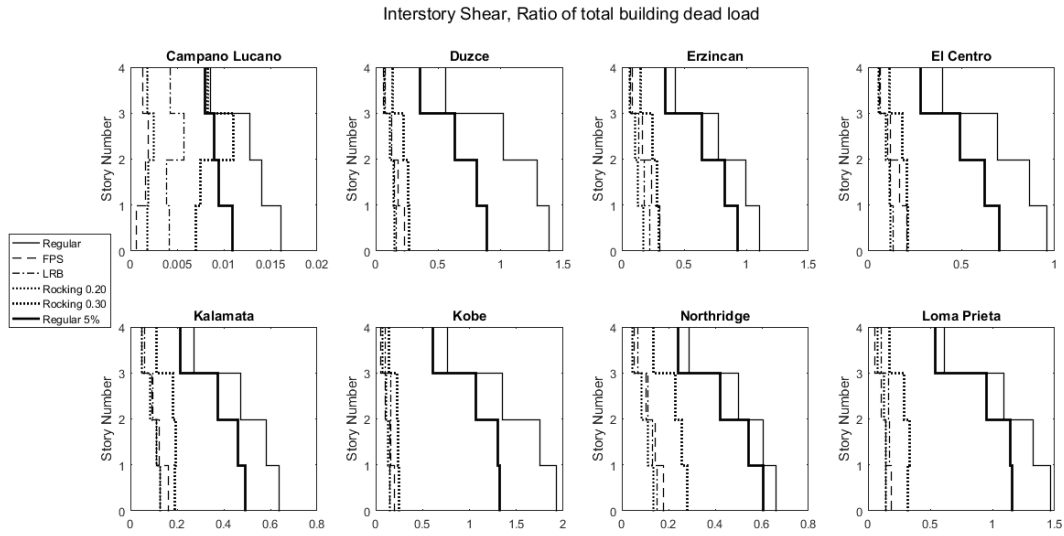


Figure 6.4 Comparison of story shears of four story buildings for regular, FPS, LRB, Rocking 0.20m, and Rocking 0.30m systems.

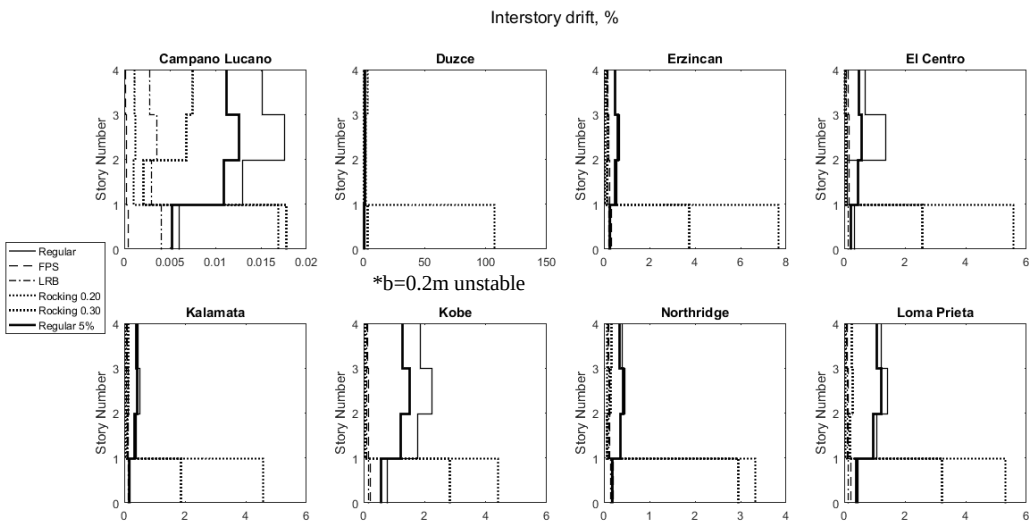


Figure 6.5 Comparison of interstory drift ratio of four story buildings with mass irregularity for regular, FPS, LRB, Rocking 0.20m, and Rocking 0.30m systems.

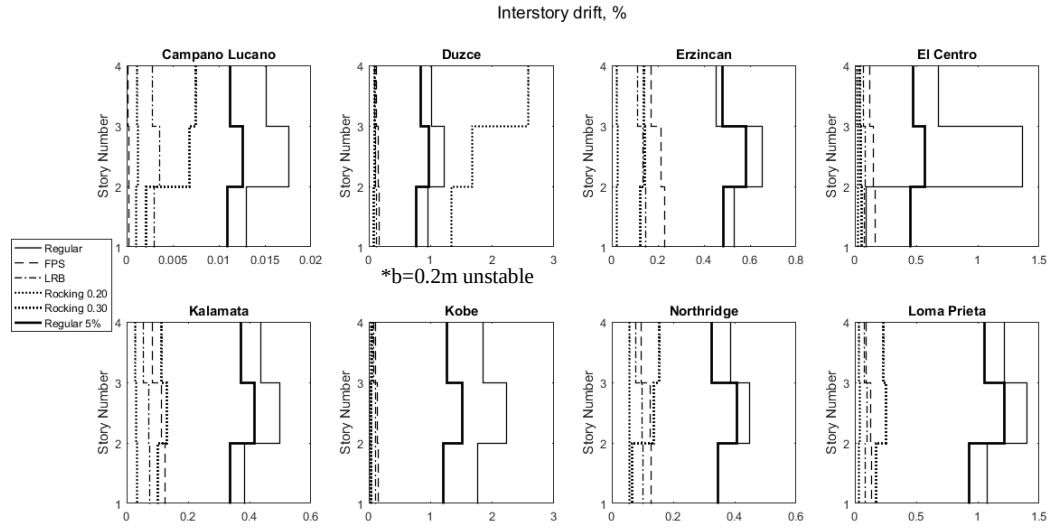


Figure 6.6 Comparison of interstory drift ratio of four story buildings (except bottom story) with mass irregularity for regular, FPS, LRB, Rocking 0.20m, and Rocking 0.30m systems.

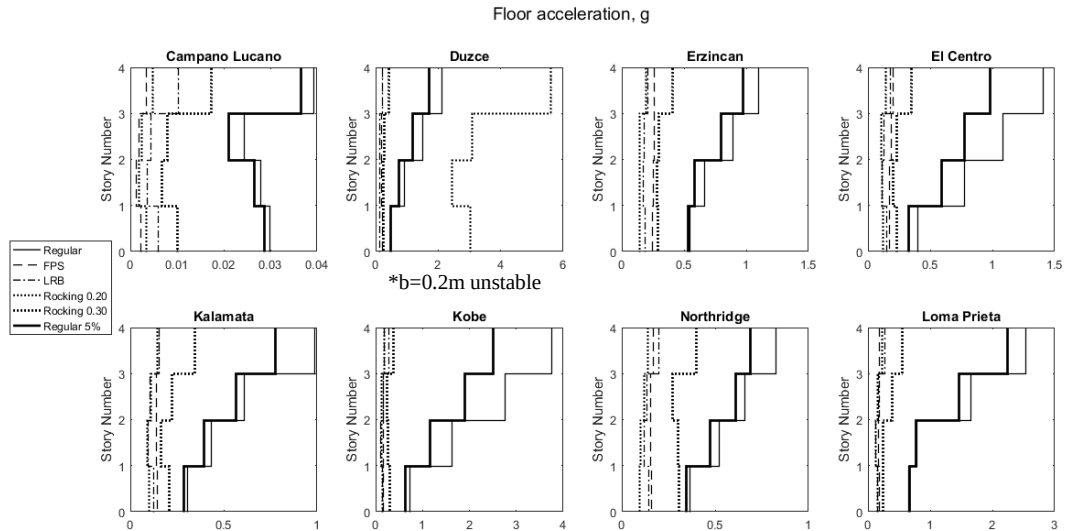


Figure 6.7 Comparison of floor accelerations of four story buildings with mass irregularity for regular, FPS, LRB, Rocking 0.20m, and Rocking 0.30m systems.

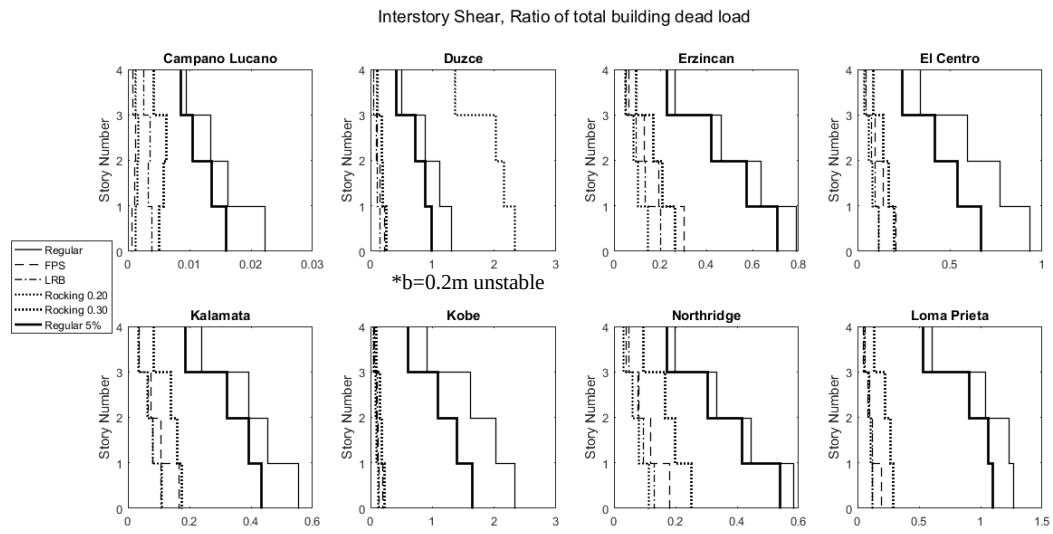


Figure 6.8 Comparison of floor shears of four story buildings with mass irregularity for regular, FPS, LRB, Rocking 0.20m, and Rocking 0.30m systems.

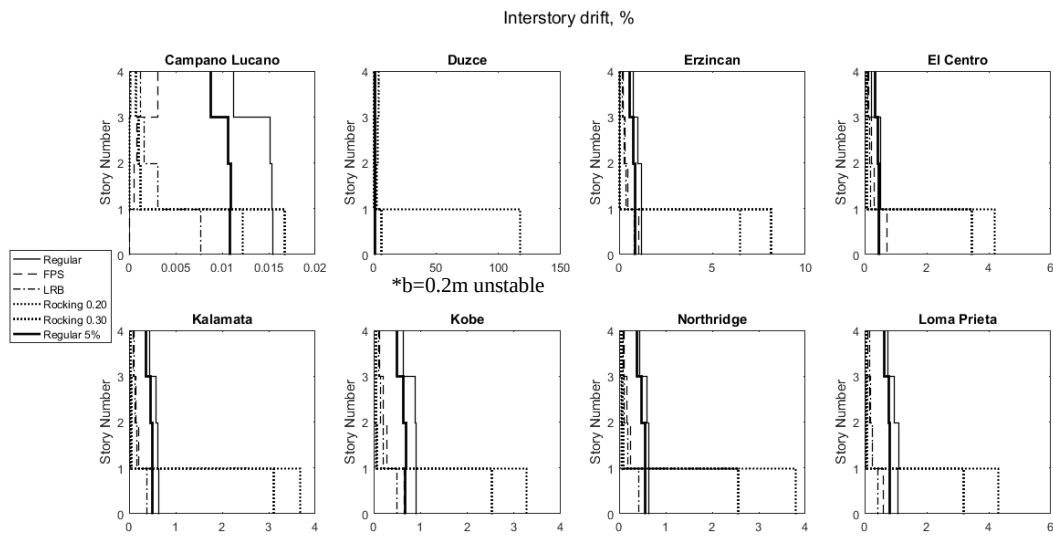


Figure 6.9 Comparison of interstory drift ratio of four story buildings with stiffness irregularity for regular, FPS, LRB, Rocking 0.20m, and Rocking 0.30m systems.

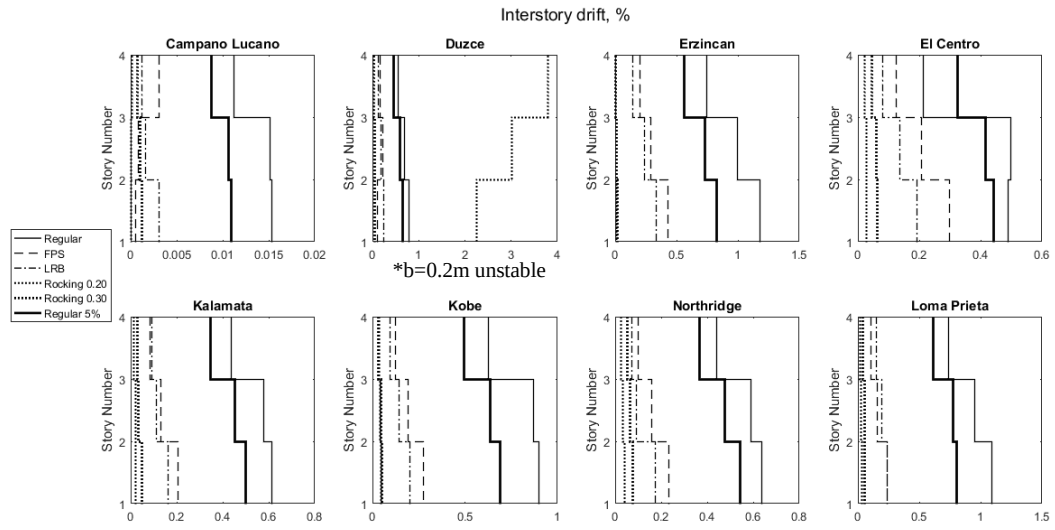


Figure 6.10 Comparison of interstory drift ratio of four story buildings (except bottom story) with stiffness irregularity for regular, FPS, LRB, Rocking 0.20m, and Rocking 0.30m systems.

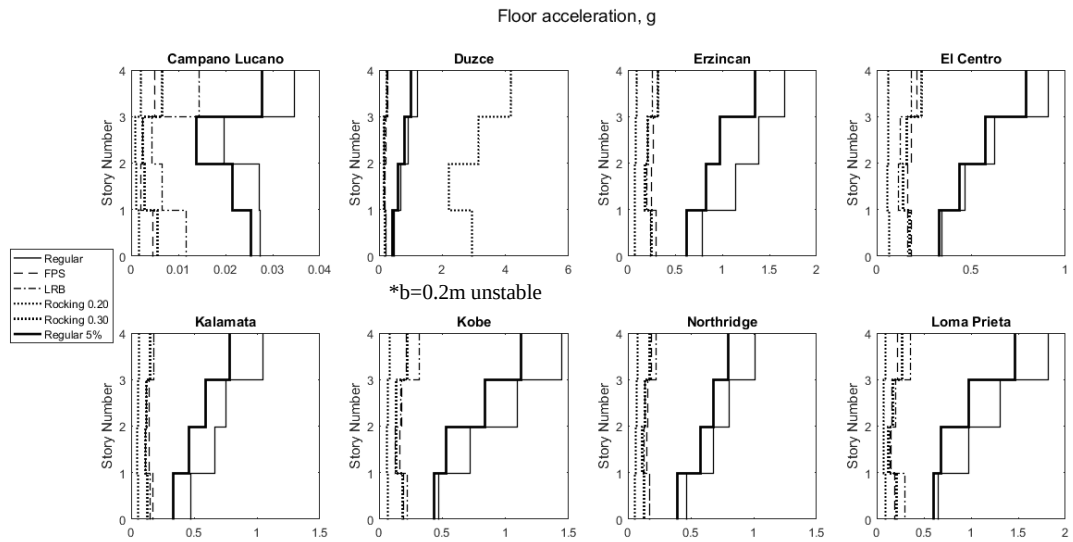


Figure 6.11 Comparison of accelerations of four story buildings with stiffness irregularity for regular, FPS, LRB, Rocking 0.20m, and Rocking 0.30m systems.

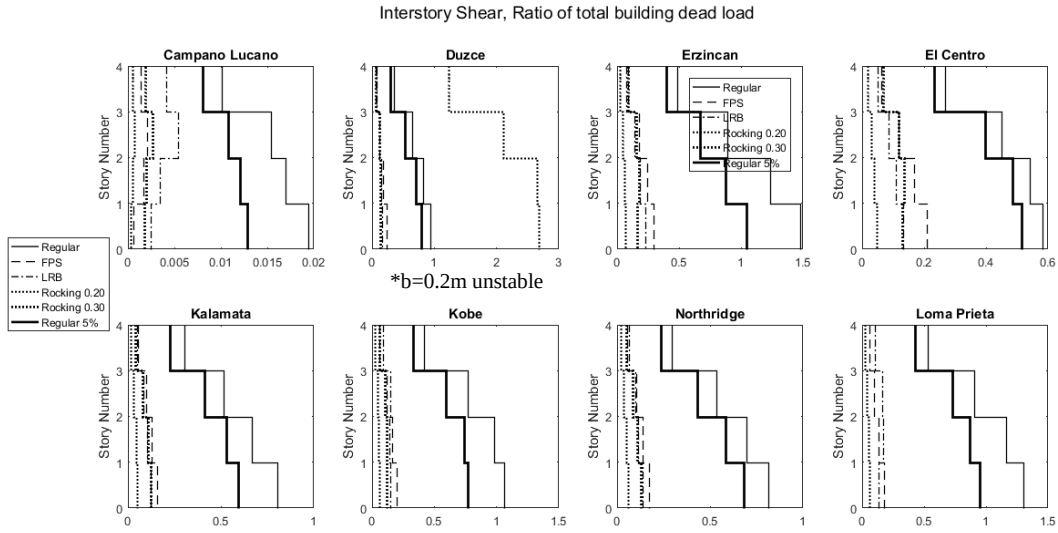


Figure 6.12 Comparison of story shears of four story buildings with stiffness irregularity for regular, FPS, LRB, Rocking 0.20m, and Rocking 0.30m systems.

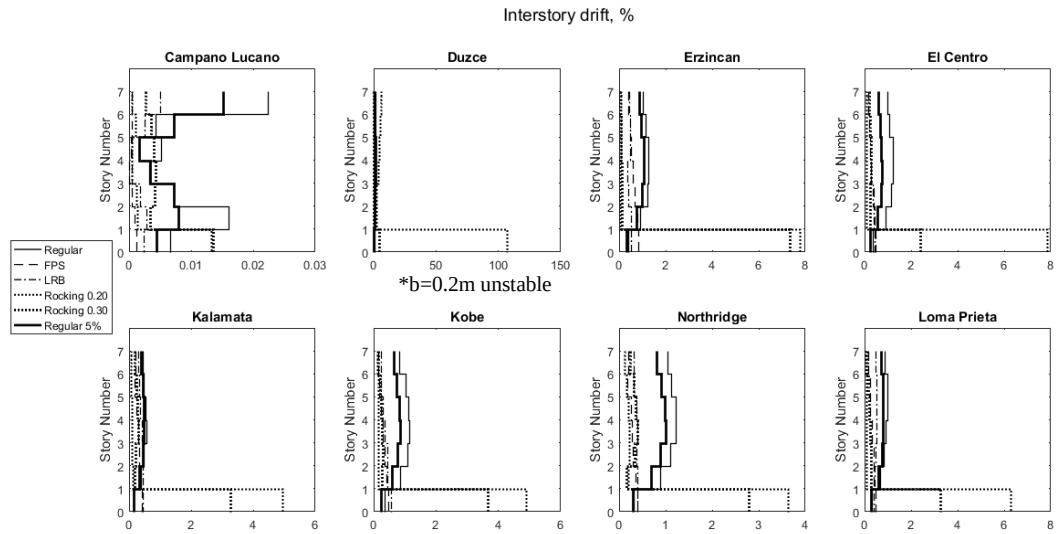


Figure 6.13 Comparison of interstory drift ratio of seven story buildings for regular, FPS, LRB, Rocking 0.20m, and Rocking 0.30m systems.

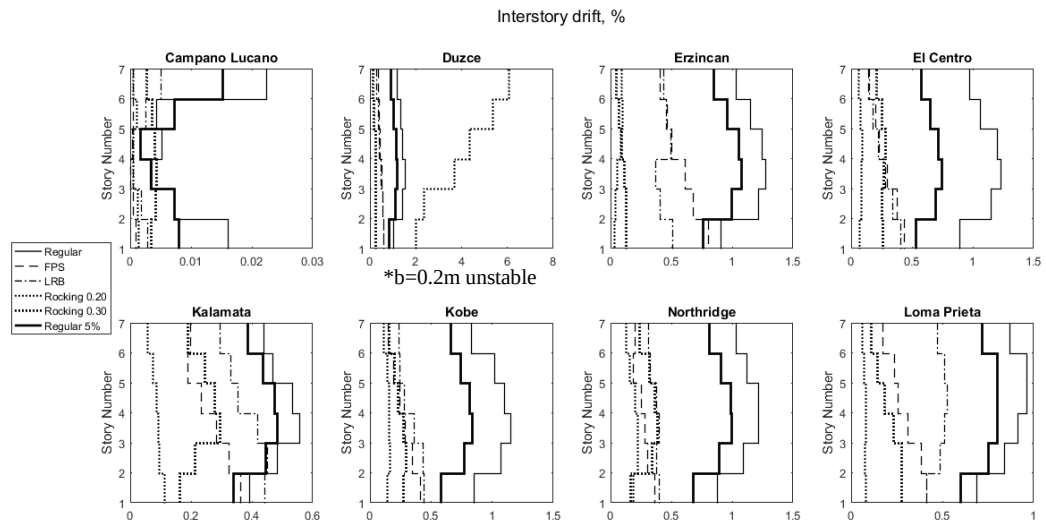


Figure 6.14 Comparison of interstory drift ratio of seven story buildings (except bottom story) for regular, FPS, LRB, Rocking 0.20m, and Rocking 0.30m systems.

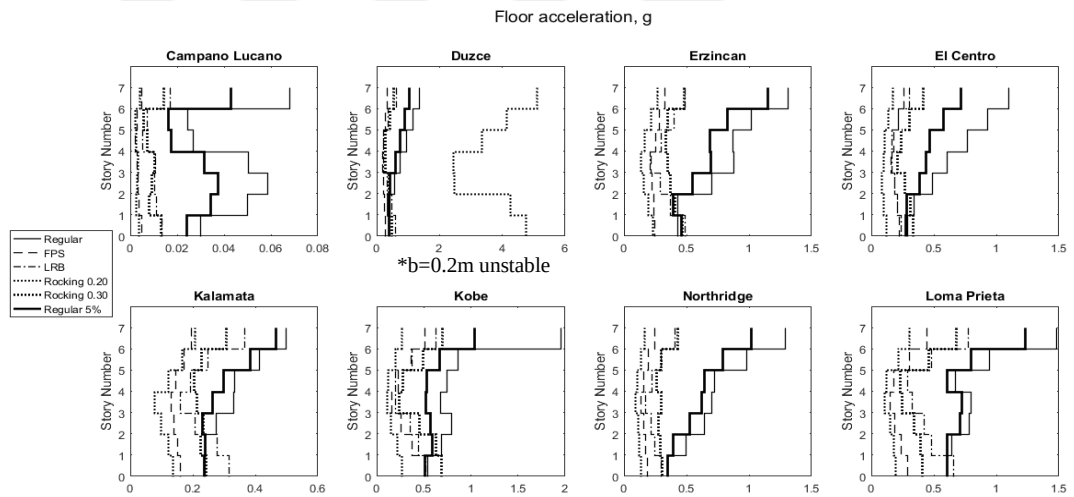


Figure 6.15 Comparison of floor accelerations of seven story buildings for regular, FPS, LRB, Rocking 0.20m, and Rocking 0.30m systems.

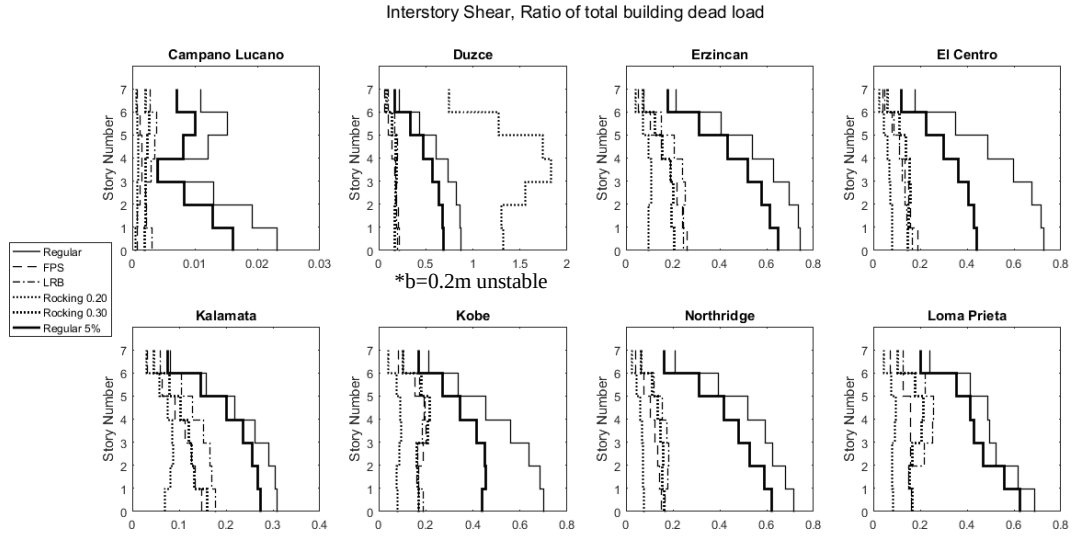


Figure 6.16 Comparison of story shears of seven story buildings for regular, FPS, LRB, Rocking 0.20m, and Rocking 0.30m systems.

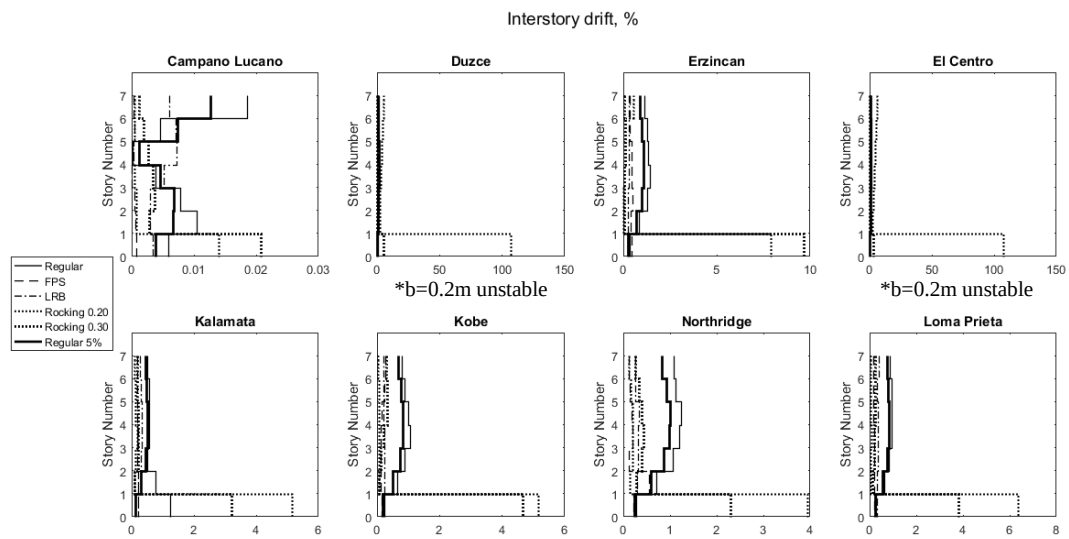


Figure 6.17 Comparison of interstory drift ratio of seven story buildings with mass irregularity for regular, FPS, LRB, Rocking 0.20m, and Rocking 0.30m systems.

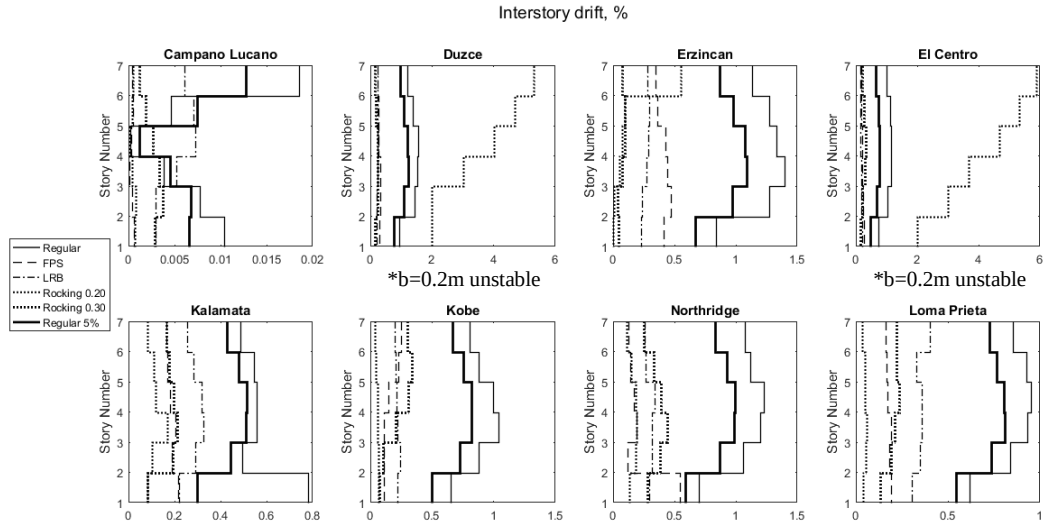


Figure 6.18 Comparison of interstory drift ratio of seven story buildings (except bottom story) with mass irregularity for regular, FPS, LRB, Rocking 0.20m, and Rocking 0.30m systems.

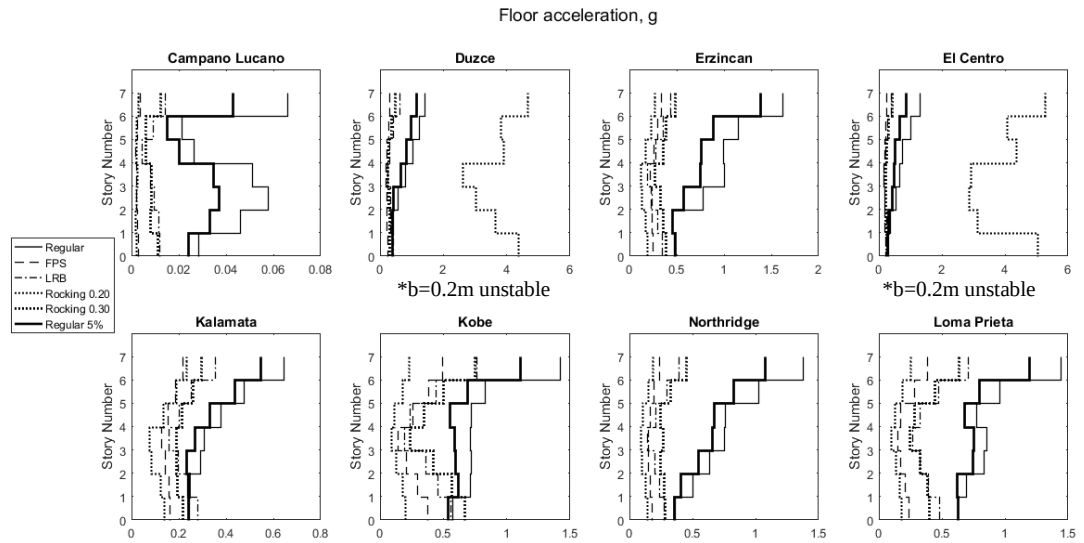


Figure 6.19 Comparison of floor accelerations of seven story buildings with mass irregularity for regular, FPS, LRB, Rocking 0.20m, and Rocking 0.30m systems.

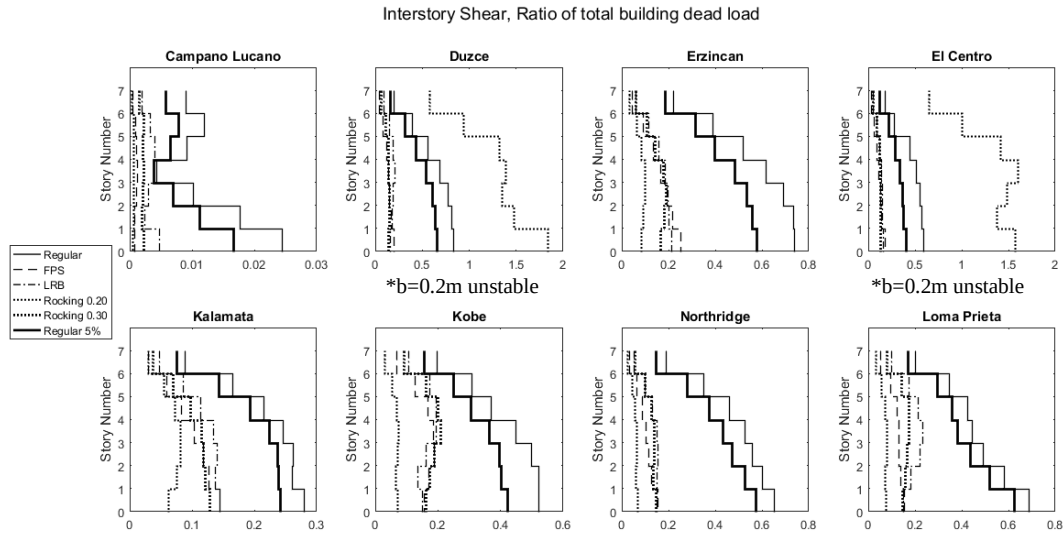


Figure 6.20 Comparison of story shears of seven story buildings with mass irregularity for regular, FPS, LRB, Rocking 0.20m, and Rocking 0.30m systems.

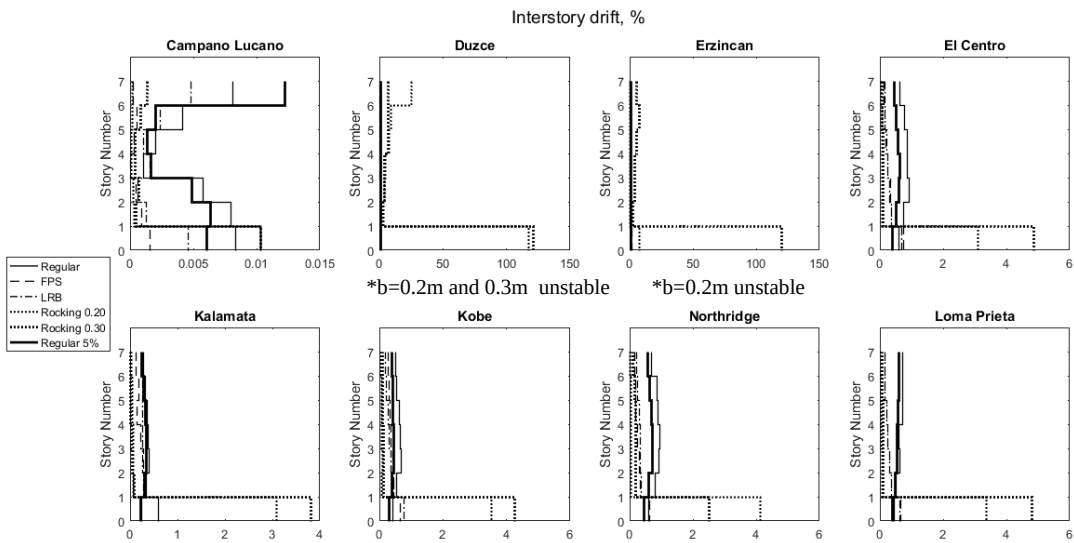


Figure 6.21 Comparison of interstory drift ratio of seven story buildings with stiffness irregularity for regular, FPS, LRB, Rocking 0.20m, and Rocking 0.30m systems.

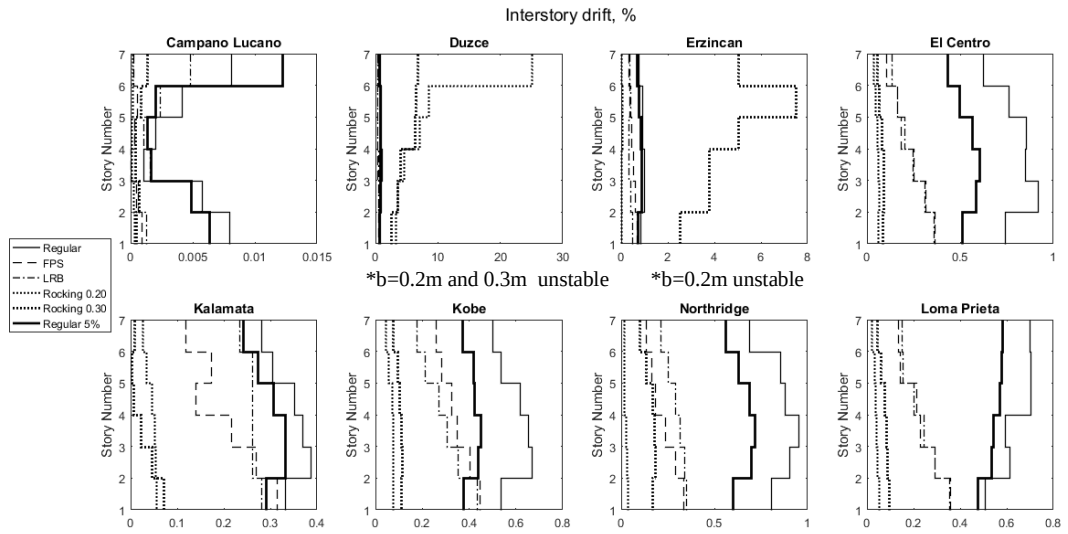


Figure 6.22 Comparison of interstory drift ratio of seven story buildings (except bottom story) with stiffness irregularity for regular, FPS, LRB, Rocking 0.20m, and Rocking 0.30m systems.

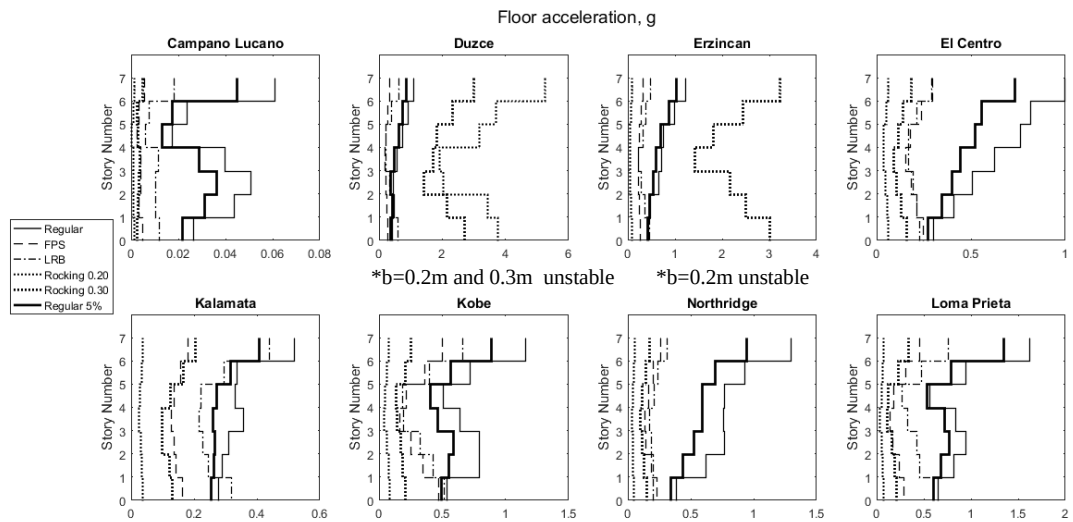


Figure 6.23 Comparison of floor accelerations of seven story buildings with stiffness irregularity for regular, FPS, LRB, Rocking 0.20m, and Rocking 0.30m systems.

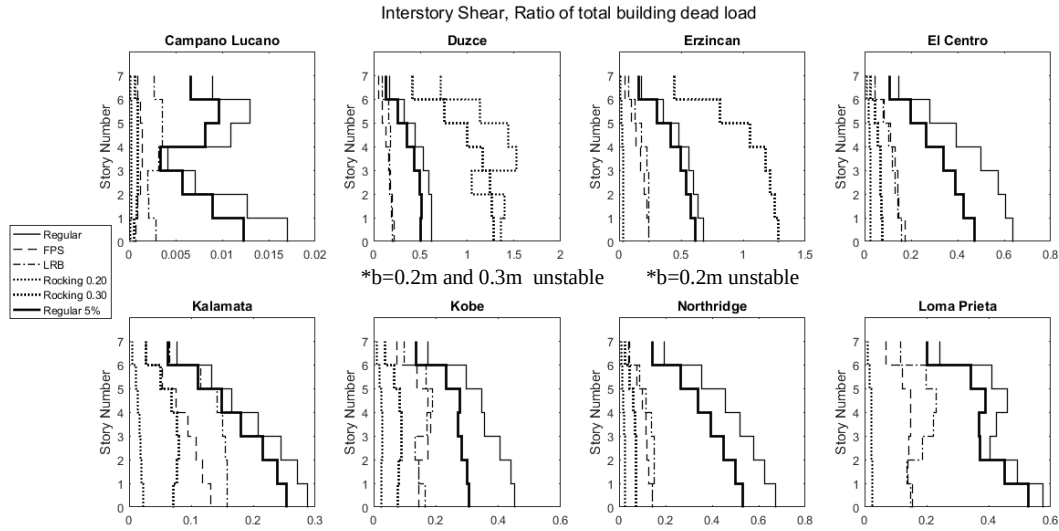


Figure 6.24 Comparison of story shears of seven story buildings with stiffness irregularity for regular, FPS, LRB, Rocking 0.20m, and Rocking 0.30m systems.

6.2 3D analyses

The purpose of 3D analyses is to develop guide charts for the preliminary design of ROC seismic isolation systems. The guide charts are developed in terms of overturning performance graphs, acceleration performance graphs at the first floor level, and shear performance graphs at the bottom story level. These graphs are presented in Figure 6.26 to Figure 6.31. A total of sixty four models prepared for the graphs covering different heights of the buildings, different widths and height of the rocking columns, and two different bay sizes. Each numerical model is analysed for eight different ground motions (considering both horizontal components) in SAP2000 software. A typical overturning performance graph is presented in Figure 6.25. The abscissa of the overturning performance graph is the width of the rocking column (b) and ordinate is the number of stories. The height of the rocking column and ground motion data record are shown in the title. The color bar shows the magnitude of the rocking displacement. Red color zone means the rocking displacement is equal to or greater than the width of the rocking column (overturning displacement). The black colored dots in the graphs represents the results obtained

from the SAP2000 analyses, whereas the results in between two black dots are linearly interpolated. The same color scheme is adopted in all the spectra.

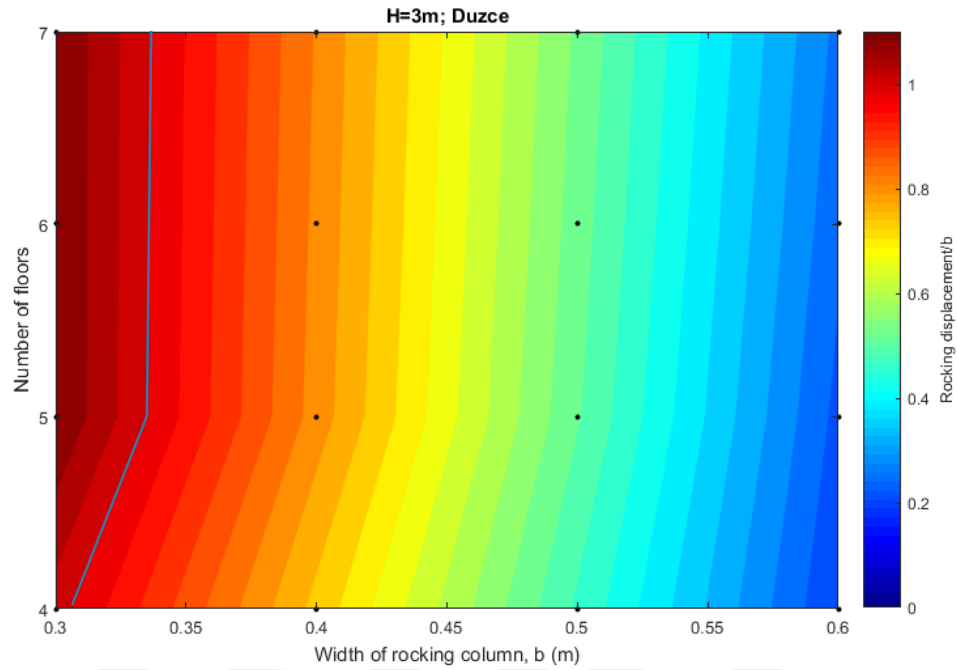


Figure 6.25 A typical overturning performance graph.

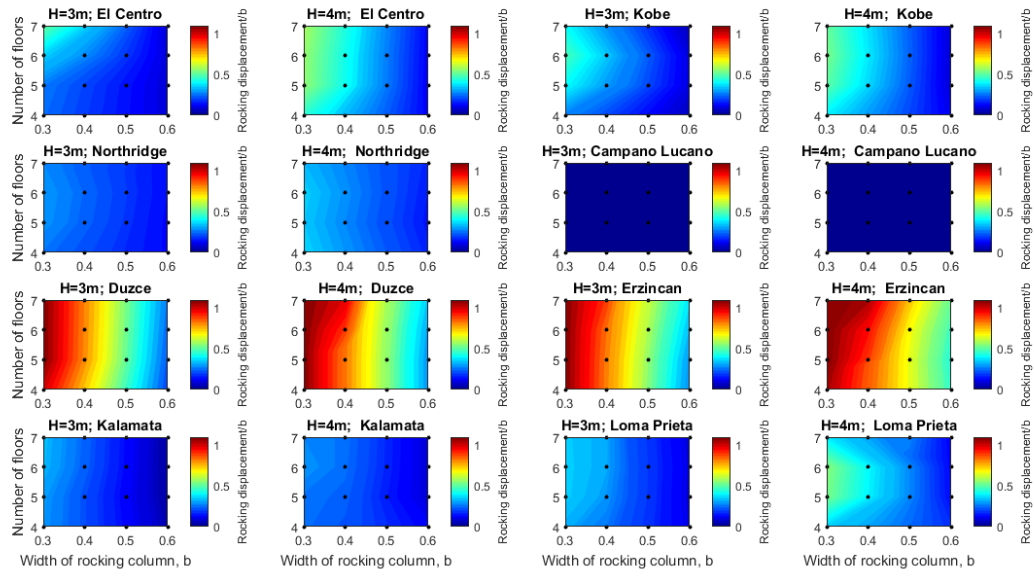


Figure 6.26 Overturning performance graph for building grid size of 5m

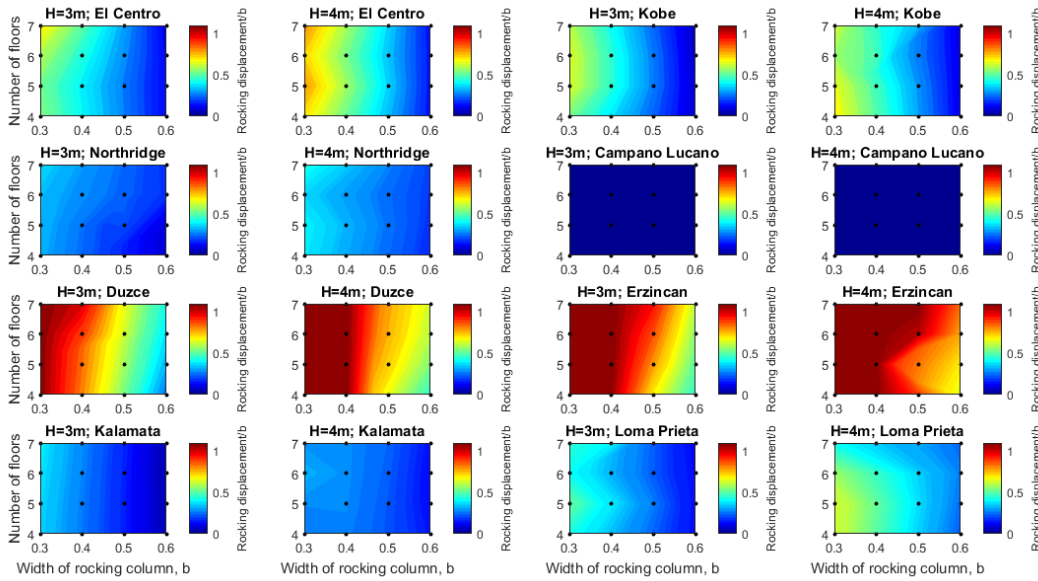


Figure 6.27 Overturning response spectra for building grid size of 7.5m

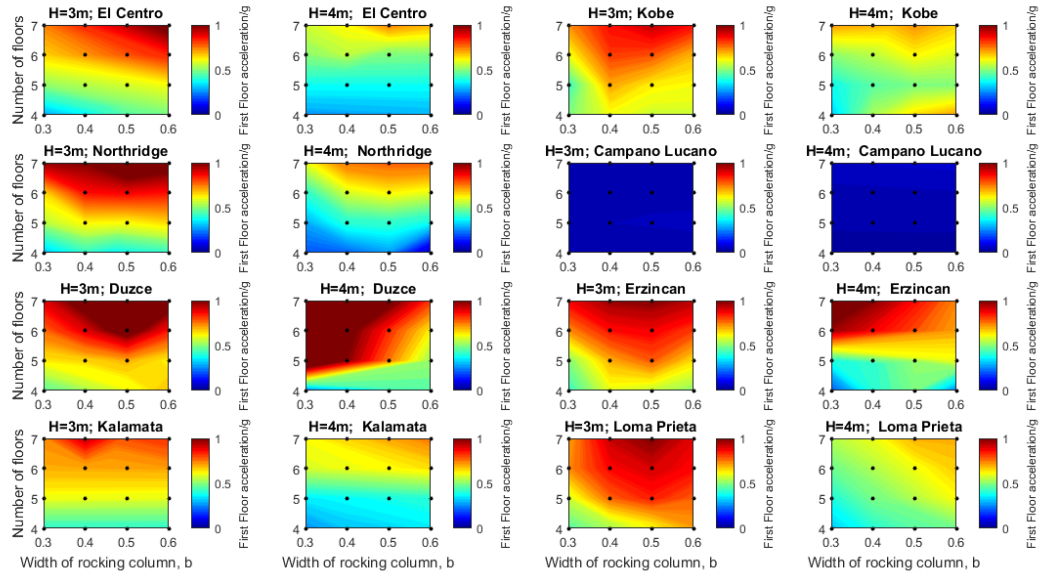


Figure 6.28 Acceleration performance graph for building grid size of 5m

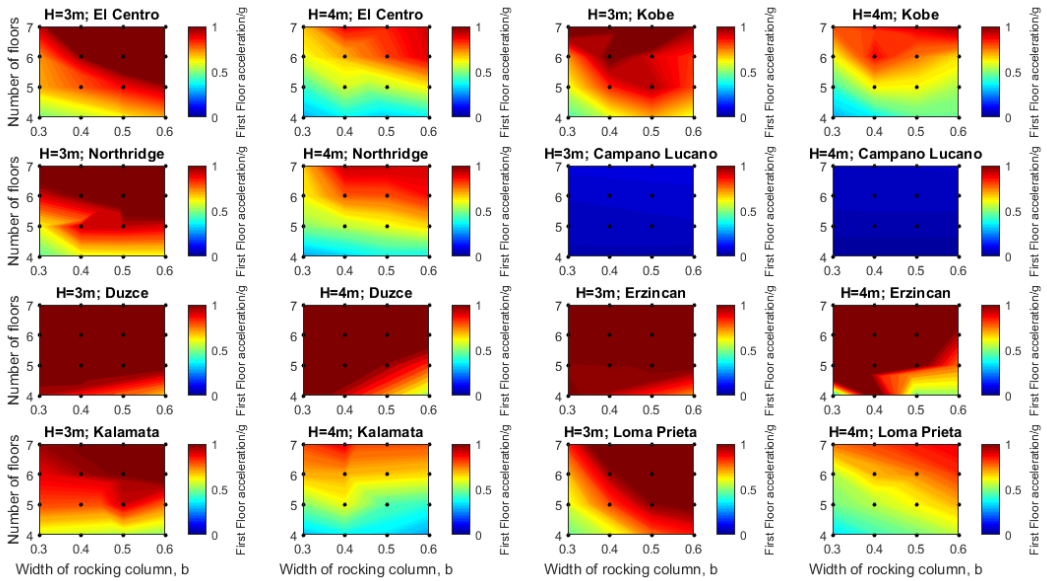


Figure 6.29 Acceleration performance graph for building grid size of 7.5m

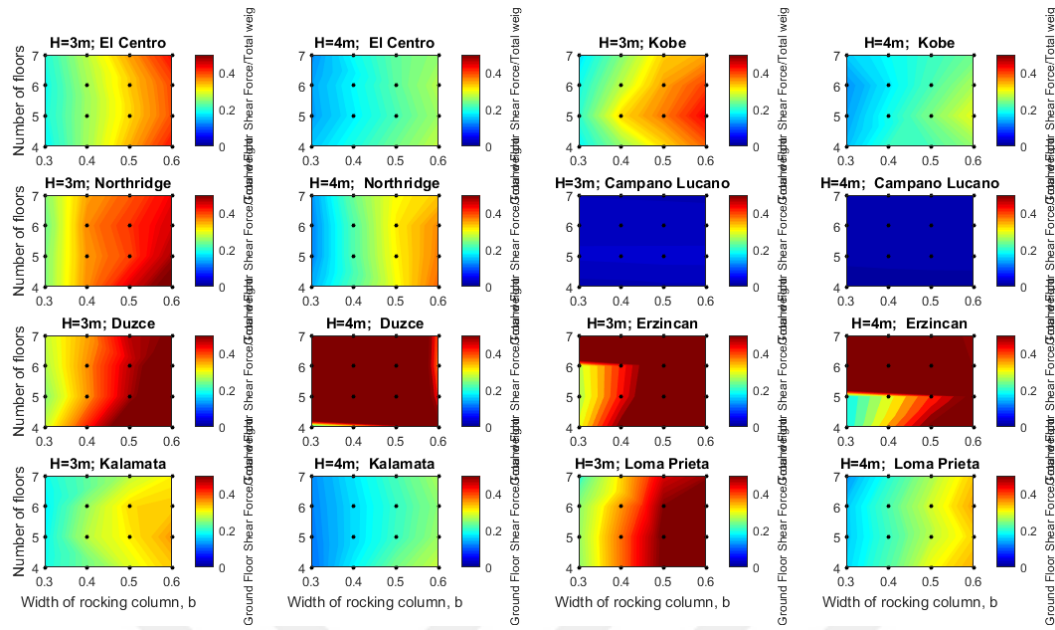


Figure 6.30 Shear performance graph for building grid size of 5m

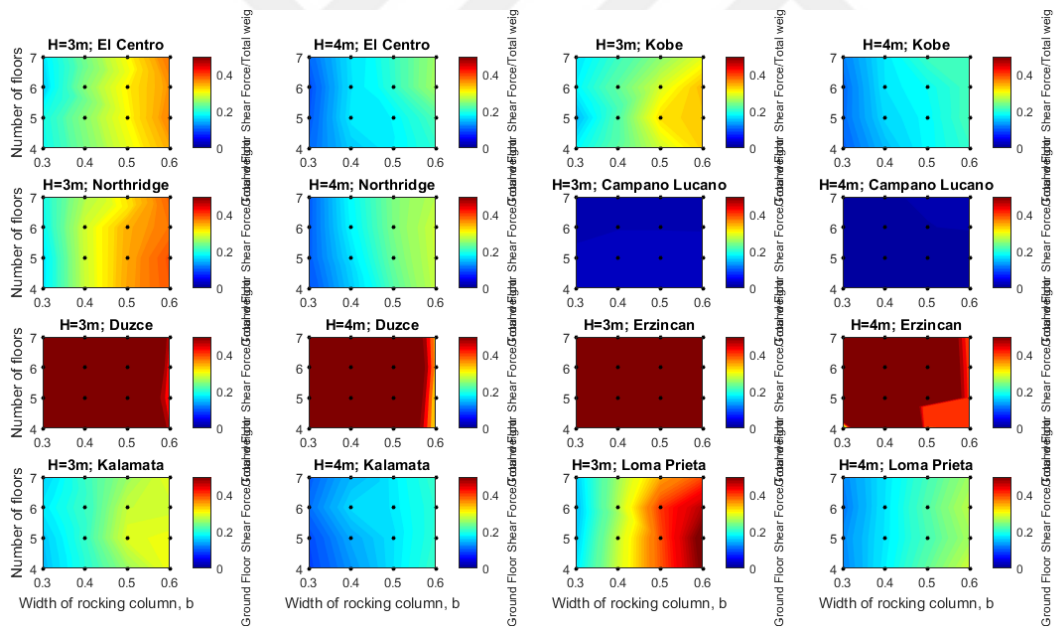


Figure 6.31 Shear performance graph for building grid size of 7.5m

6.3 Proposed procedure for the preliminary design of ROC seismic isolation system

The procedure proposed for the “preliminary design” of ROC seismic isolation system is explained in the following steps. Designer should conduct a series of detailed time history analyses to make sure the structure is safe and well designed. No factor of safety is used in preliminary design tables as displacement controlling precautions are mandatory. Preliminary design steps are as follows:

1. Select the number of stories and bay size of the building. The number of stories ranges from four to seven stories, and bay size of the building is either 5m or 7.5m. (other distances may be interpolated or larger size may be selected for larger factor of safety).
2. Select the ground motion record (for which the system is supposed to be designed).
3. Decide the width and height of the rocking column from the overturning response spectra given in Figure 6.26 and Figure 6.27.
4. Obtain the corresponding first floor acceleration and shear from the acceleration performance graph and shear performance graph respectively, as given in Figure 6.28 to Figure 6.31.
5. If the acceleration and shear values are within the acceptable range, proceed to step 6; otherwise go back to step 1.
6. Perform wind load analyses to ensure that the building is not overturning for the proposed ROC seismic isolation system.

Appendix-A exhibits the tables which may be used for the preliminary design of ROC seismic isolation system with different number of stories vs. earthquake records.

CHAPTER 7

CONCLUSIONS AND REMARKS

The concept of rocking column may be applied for the earthquake response modification of civil engineering structures. In this research, rocking columns are used as a story high seismic isolator (ROC) for the earthquake mitigation of medium rise residential buildings. The conclusions are divided into three section; first section belongs to the remarks drawn from the literature review, second section belongs to the experimental part of the research, and third section belongs to the results obtained from the finite element analyses and design.

The following remarks are summarized based on a comprehensive review of rocking rigid blocks which are presented in this paper:

- The rocking of a solitary rigid column cannot be treated as an equivalent SDOF elastic system.
- The vertical component of ground excitation may be ignored during analyses of the rocking of a solitary column.
- The probability of overturning of rocking column increases due to a decrease in slenderness ratio (α), a decrease in size (R), and an increase in ground shaking amplitude.
- Larger weight and rotational inertia make the single rocking columns more stable.
- The rocking of a rigid column is nonlinear and highly sensitive to small perturbations (chaotic).
- The rocking frame is more stable as compared with the single rocking column.

The experiments are performed in two stages; 1) experiment of a single rocking column prototype and 2) experiment of small-scale single-story podium structure

with rocking columns. After the experiments, the respective models are developed in a finite element software (SAP2000). The comparison of the numerical model defined in the SAP2000 and the experimental observation is concluded as follows:

- The numerical model defined in SAP2000 software is sensitive to the change in the integration time step, α -value, and damping ratio. The experimental validation of the numerical model has shown that optimum values of integration time step, α (α), and damping ratio, for the numerical convergence of the model and comparison with the experimental values are 1/200 sec, -1/3, and 0.1%, respectively. The same conclusion holds for all the experiments.
- The observed acceleration spikes at the impacts, in the SAP2000 results, are fake as these spikes are not present in the experimental results. The false spikes may be dispersed to some extent by decreasing the α -value up to -1/3 and these false acceleration spikes appeared in the SAP2000 results may be ignored since they were not observed in the experiments.
- The acceleration between two consecutive impacts of a free rocking rigid block is observed as a fluctuating parabola (instead a constant value as assumed in some of the literature). The acceleration changes its sign after each impact. However, for forced vibration testing (shaking table), the acceleration between two impacts is not constant or parabolic, but it apparently changes as a function of the excitation.
- The half-period of the free rocking column prototype decreases after each impact without any major decrease in the peak acceleration during the rocking motion. The acceleration and half-period values approaches to zero at the end of the free rocking vibration.
- For the rocking of the single story podium prototype, the comparison of acceleration at the top and the base shows the absolute effectiveness of the ROC (Figure 5.11 to Figure 5.15).
- Validation of the FEM simulation capabilities are performed using small scale shaking tests. As the results from simulation closely matches the experiment, full size analytical models are utilized to simulate earthquake response for mid-rise residential buildings. Preliminary design tables are prepared based on extensive

number of simulations using number of stories, rocking column width and height, column to column bay distance, and for different earthquakes.

Finite element analyses are performed to see the effectiveness of the ROC system and to develop the performance graphs of the proposed isolator for four to seven story residential buildings. In the 2D analyses, the proposed ROC system is compared with conventional base isolators (LRB and FPS) and a typical building without isolators. The results are compared in terms of inter-story drift ratio, floor acceleration, and normalized story shear values with respect to the total weight.

- Smallest inter-story drift ratios are obtained by using 0.2m width rocking columns for 4-story buildings, which resulted in almost a linear elastic behavior of the above stories by keeping drift ratios below 0.5% of the story height. The inter-story drift ratios for the 0.3m rocking width of the column is also acceptably good and in a comparable range of those obtained from conventional base isolators. As we increase the number of stories from four to seven, the rocking displacement increases for the same earthquakes, shifting the structure towards more instability. In general, smaller rocking width of the column better isolates a building but at an increased risk of overturning especially for taller structures.
- The value of floor accelerations and normalized story shear values are similar to those of the conventional isolation systems. Slightly better results are observed with 0.2m width of the rocking columns.
- The increase of mass at the first floor level are tested for 4 story and 7 story buildings with ROC and seems to have a little effect on the behavior of the rocking structures in terms of inter-story drift ratio for the upper floors, floor accelerations, and normalized story total shear forces.
- The inter-story drift ratio, floor accelerations, and normalized total story shear forces improve due to increase in the height of the rocking columns from 3m to 4m. However, it decreases the stability of the system.

In the 3D category of the finite element analyses, the focus is to develop the overturning response graphs, acceleration response graphs, and normalized shear force response graphs for the first floor level of the ROC system installed in medium rise residential buildings.

- Stability of ROC system increases due to increase in the width of the rocking column and decreases due to increase in height of the rocking column from 3m to 4m and due to increase in the size of the building inter-column bay distance from 5m to 7.5m. In general, the increase in number of stories from 4 to 7 does not seem to have a notable effect on the overturning performance of the ROC system.
- Acceleration of ROC system at the first floor level increases due to increase in width of the rocking column, increase in number of stories (height of the building), and increase in the grid size. However, it decreases as the height of the rocking column decreases.
- Normalized shear value of the ROC system at the first floor level increases due to increase in width of the rocking column and decreases due to increase in the bay size of the building (from 5m to 7.5) and increase in the rocking column height (from 3m to 4m).

These observations are summarized below in a tabulated format:

	Stability	Acceleration	Shear
Width of column	↑	↑	↑
Height of column	↓	↓	↓
Inter-column bay distance	-	↑	↑
Number of floors	-	↑	↑

As a general rule, the stability of the ROC against overturning increases due to increase in the width of the rocking column but at a cost of increase in floor acceleration and story shear value. The increase in floor acceleration and normalized shear value (due to increase in the width of the rocking column) may cause the partial damage to the building or total collapse in some cases. Likewise, the increase in

rocking column height improves the performance but at a risk of rocking column overturning.

As a summary, this thesis proposes an alternative seismic base isolation system using rocking columns with numerous advantages that other well-known systems don't have; such as absence of resonant frequency, no movement at small forces, extra floor space, smaller modifications for utility pipes, etc. Analytical models of ROC system are verified using small scale experiments and further preliminary design guides are prepared. This will also allow a better understanding of the rocking behavior among researchers. The designers might be reluctant to apply the concept of rocking in structures for earthquake damage mitigation by base isolation due to the lack of an exclusive, full-size experimentally proven robust approach and absence of proper guidance in the design codes. Therefore, more investigation and code preparation is necessary for practitioners to fully grasp the concept of rocking and be able to apply it for civil engineering structures. The 3D motion of rocking rigid blocks having rectangular and circular cross-sections (e.g., Greek and Roman orders having circular cross-sections) is less studied but survived millenniums in earthquake prone regions and more experimental research is required in this area. The deformability of rocking edges (due to concentrated load on one end only during rocking), and crushing of reinforced concrete edges must be strengthened using steel tip mechanisms. Damping inherited to rocking columns due to impact needs to be further investigated.

REFERENCES

1. Soong TT, Spencer BF. Supplemental energy dissipation: State-of-the-art and state-of-the-practice. *Eng Struct.* 2002;24(3):243-259. doi:10.1016/S0141-0296(01)00092-X
2. Saaed TE, Nikolakopoulos G, Jonasson JE, Hedlund H. A state-of-the-art review of structural control systems. *JVC/Journal Vib Control.* 2015;21(5):919-937. doi:10.1177/1077546313478294
3. El-Khoury O, Adeli H. Recent Advances on Vibration Control of Structures Under Dynamic Loading. *Arch Comput Methods Eng.* 2013;20(4):353-360. doi:10.1007/s11831-013-9088-2
4. Morales-Beltran M, Paul J. Technical Note: Active and Semi-Active Strategies to Control Building Structures under Large Earthquake Motion. *J Earthq Eng.* 2015;19(7):1086-1111. doi:10.1080/13632469.2015.1036326
5. Gutierrez Soto M, Adeli H. Tuned Mass Dampers. *Arch Comput Methods Eng.* 2013;20(4):419-431. doi:10.1007/s11831-013-9091-7
6. Lu Z, Chen X, Zhang D, Dai K. Experimental and analytical study on the performance of particle tuned mass dampers under seismic excitation. *Earthq Eng Struct Dyn.* 2017;46(5):697-714. doi:10.1002/eqe.2826
7. Zhu F, Wang JT, Jin F, Lu LQ. Real-time hybrid simulation of full-scale tuned liquid column dampers to control multi-order modal responses of structures. *Eng Struct.* 2017;138:74-90. doi:10.1016/j.engstruct.2017.02.004
8. Dai K, Wang J, Mao R, Lu Z, Chen SE. Experimental investigation on dynamic characterization and seismic control performance of a TLPD system. *Struct Des Tall Spec Build.* 2017;26(7):1-13. doi:10.1002/tal.1350
9. Ueng JM, Lin CC, Wang JF. Practical design issues of tuned mass dampers for torsionally coupled buildings under earthquake loadings. *Struct Des Tall Spec Build.* 2008;17(1):133-165. doi:10.1002/tal.336
10. Carmona JEC, Avila SM, Doz G. Proposal of a tuned mass damper with friction damping to control excessive floor vibrations. *Eng Struct.* 2017;148:81-100. doi:10.1016/j.engstruct.2017.06.022
11. Papalou A, Masri SF. Response of Impact Dampers With Granular Materials. *Earthq Engg Struct Dyn.* 1996;25(October 1995):253-267.
12. Skinner RI, Kelly JM, Heine AJ. Hysteretic dampers for earthquake-resistant structures. *Earthq Eng Struct Dyn.* 1974;3(3):287-296.

doi:10.1002/eqe.4290030307

13. Housner GW, Bergman LA, Caughey TK, et al. Structural control: Past, present, and future. *J Eng Mech.* 1997;123(9):897-971. doi:10.1061/(ASCE)0733-9399(1997)123:9(897)
14. Symans, MD, Charney FA, Whittaker AS, Constantinou MC, Kircher CA, Johnson MW and McNamara RJ. Energy Dissipation Systems for Seismic Applications: Current Practice and Recent Developments. *J Struct Eng.* 2008;134(1):3-21. doi:10.1061/(ASCE)0733-9445(2008)134
15. Lee CH, Woo SK, Ju YK, Lee DW, Kim SD. Modified Fatigue Model for Hourglass-Shaped Steel Strip Damper Subjected to Cyclic Loadings. *J Struct Eng (United States).* 2015;141(8):1-12. doi:10.1061/(ASCE)ST.1943-541X.0001183
16. Anagnostopoulos SA, Kyrkos MT, Stathopoulos KG. Earthquake induced torsion in buildings: Critical review and state of the art. *Earthq Struct.* 2015;8(2):305-377. doi:10.12989/eas.2015.8.2.305
17. Lu LY. Predictive control of seismic structures with semi-active friction dampers. *Earthq Eng Struct Dyn.* 2004;33(5):647-668. doi:10.1002/eqe.371
18. Chu SY, Yeh SW, Lu LY, Peng CH. A leverage-type stiffness controllable mass damper for vibration mitigation of structures. *Struct Control Heal Monit.* 2017;24(4):1-22. doi:10.1002/stc.1896
19. Alvandi S, Ghassemieh M. Application of Shape Memory Alloys in Seismic Isolation: A Review. *Civ Eng Infrastructures J.* 2014;47(2):153-171.
20. Harvey PS, Kelly KC. A review of rolling-type seismic isolation: Historical development and future directions. *Eng Struct.* 2016;125:521-531. doi:10.1016/j.engstruct.2016.07.031
21. Warn GP, Ryan KL. A review of seismic isolation for buildings: Historical development and research needs. *Buildings.* 2012;2(3):300-325. doi:10.3390/buildings2030300
22. Turer A, Özden B. Seismic base isolation using low-cost Scrap Tire Pads (STP). *Mater Struct Constr.* 2008;41(5):891-908. doi:10.1617/s11527-007-9292-3
23. Robinson WH. Lead-rubber hysteretic bearings suitable for protecting structures during earthquakes. *Earthq Engg Struct Dyn.* 1982;10:593-604. doi:10.2140/siaps.2011.2.5
24. Komodromos P, Polycarpou PC, Papaloizou L and Phocas MC. Response of seismically isolated buildings considering poundings. *Earthq Eng Struct Dyn.* 2007;36:1605-1622. doi:10.1002/eqe.692

25. Polycarpou PC, Komodromos P. Earthquake-induced poundings of a seismically isolated building with adjacent structures. *Eng Struct.* 2010;32(7):1937-1951. doi:10.1016/j.engstruct.2010.03.011
26. Ene D, Yamada S, Jiao Y, Kishiki S, Konishi Y. Reliability of U-shaped steel dampers used in base-isolated structures subjected to biaxial excitation. *Earthq Engg Struct Dyn.* 2017;(46):621-639. doi:10.1002/eqe
27. Cheng CT, Chao CH. Seismic behavior of rocking base-isolated structures. *Eng Struct.* 2017;139:46-58. doi:10.1016/j.engstruct.2017.02.027
28. Freddi F, Dimopoulos CA, Karavasilis TL. Rocking damage-free steel column base with friction devices: design procedure and numerical evaluation. *Earthq Eng Struct Dyn.* 2017;46(14):2281-2300. doi:10.1002/eqe.2904
29. Spanos PD, Koh AS. Rocking of rigid blocks due to harmonic shaking. *J Eng Mech.* 1984;110(11):1627-1642.
30. Aslam M, Fodden WG, Scalise DT. Earthquake rocking response of rigid bodies. *J Struct Div ASCE.* 1980;106(ST2, Proc. Paper, 15182):377-392.
31. Casapulla C. Rocking resonance of a rigid free standing block. In: *Brick and Block Masonry: Trends, Innovations and Challenges - Proceedings of the 16th International Brick and Block Masonry Conference, IBMAC 2016.* ; 2016:137-144. doi:10.1201/b21889-15
32. Shawa OA, de Felice G, Mauro A, Sorrentino L. Out-of-plane seismic behaviour of rocking masonry walls. *Earthq Engg Struct Dyn.* 2012;41:949-968. doi:10.1002/eqe.1168
33. Sorrentino L, Kunnath S, Monti G, Scalora G. Seismically induced one-sided rocking response of unreinforced masonry façades. *Eng Struct.* 2008;30(8):2140-2153. doi:10.1016/j.engstruct.2007.02.021
34. Housner GW. The behavior of inverted pendulum structures during earthquakes. *Bull Seismol Soc Am.* 1963;53(2):403-417.
35. Ishiyama Y. Motions of rigid bodies and criteria for overturning by earthquake excitations. *Earthq Eng Struct Dyn.* 1982;10:635-650.
36. Makris N. A half-century of rocking isolation. *Earthq Struct.* 2014;7(6):1187-1221. doi:10.12989/eas.2014.7.6.1187
37. Makris N, Kampas G. Size versus slenderness: Two competing parameters in the seismic stability of free-standing rocking columns. *Bull Seismol Soc Am.* 2016;106(1):104-122. doi:10.1785/0120150138
38. Pena F, Prieto F, Lourenco PB, Campos Costa A, Lemos JV. On the dynamics

- of rocking motion of single rigid-block structures. *Pacific Conf Earthq Eng.* 2007;36:2383-2399. doi:10.1002/eqe.739
39. Yim CS, Chopra AK, Penzien J. Rocking response of rigid blocks to earthquakes. *Earthq Eng Struct Dyn.* 1980;8(6):565-587. doi:10.1002/eqe.4290080606
 40. Kounadis AN. On the rocking-sliding instability of rigid blocks under ground excitation: SOME new findings. *Soil Dyn Earthq Eng.* 2015;75:246-258. doi:10.1016/j.soildyn.2015.03.026
 41. Wong CM, Tso WK. Steady state rocking response of rigid blocks part 2: Experiment. *Earthq Eng Struct Dyn.* 1989;18(1):107-120. doi:10.1002/eqe.4290180110
 42. Taniguchi T. Non-linear response analyses of rectangular rigid bodies subjected to horizontal and vertical ground motion. *Earthq Eng Struct Dyn.* 2002;31(8):1481-1500. doi:10.1002/eqe.170
 43. Roh H, Reinhorn AM. Modeling and seismic response of structures with concrete rocking columns and viscous dampers. *Eng Struct.* 2010;32(8):2096-2107. doi:10.1016/j.engstruct.2010.03.013
 44. Roh H, Reinhorn AM. Analytical modeling of rocking elements. *Eng Struct.* 2009;31(5):1179-1189. doi:10.1016/j.engstruct.2009.01.014
 45. Roh H, Reinhorn AM. Nonlinear static analysis of structures with rocking columns. *J Struct Eng.* 2010;136(5):532-542. doi:10.1061/(ASCE)ST.1943-541X.0000154
 46. Vassiliou MF, Mackie KR, Stojadinovic B. Dynamic response analysis of solitary flexible rocking bodies: modeling and behavior under pulse-like ground excitation. *Earthq Engg Struct Dyn.* 2014;43:1463-1481. doi:10.1002/eqe.2406
 47. Makris N, Vassiliou MF. Sizing the slenderness of free-standing rocking columns to withstand earthquake shaking. *Arch Appl Mech.* 2012;82(10-11):1497-1511. doi:10.1007/s00419-012-0681-x
 48. Rajah S, Bachmann JA, Vassiliou MF, Stojadinovic B. Dynamic response of a rigid slab supported by four rigid cylindrical rocking and wobbling columns. *COMPdyn 2017 - Proc 6th Int Conf Comput Methods Struct Dyn Earthq Eng.* 2017;2(June):3045-3061. doi:10.7712/120117.5625.18529
 49. Truniger R, Vassiliou MF and Stojadinovic B. An analytical model of a deformable cantilever structure rocking on a rigid surface: experimental validation. *Earthq Eng Struct Dyn.* 2015;44:2795-2815. doi:10.1002/eqe.2609

50. Dimitrakopoulos EG, DeJong MJ. Revisiting the rocking block: Closed-form solutions and similarity laws. *Proc R Soc A Math Phys Eng Sci.* 2012;468(2144):2294-2318. doi:10.1098/rspa.2012.0026
51. Masi F, Stefanou I, Vannucci P, Maffi-Berthier V. Rocking response of inverted pendulum structures under blast loading. *Int J Mech Sci.* 2019;157-158(May):833-848. doi:10.1016/j.ijmecsci.2019.05.024
52. Bachmann JA, Vassiliou MF, Stojadinovic B. Rolling and rocking of rigid uplifting structures. *Earthq Eng Struct Dyn.* 2019;48(14):1556-1574. doi:10.1002/eqe.3213
53. Papantonopoulos C, Psycharis IN, Papastamatiou DY, Lemos J V., Mouzakis HP. Numerical prediction of the earthquake response of classical columns using the distinct element method. *Earthq Eng Struct Dyn.* 2002;31(9):1699-1717. doi:10.1002/eqe.185
54. Sorrentino L, Masiani R, Decanini LD. Overturning of rocking rigid bodies under transient ground motions. *Struct Eng Mech.* 2006;22(3):293-310. doi:10.12989/sem.2006.22.3.293
55. Lagomarsino S. Seismic assessment of rocking masonry structures. *Bull Earthq Eng.* 2015;13(1):97-128. doi:10.1007/s10518-014-9609-x
56. ElGawady MA, Ma Q, Butterworth JW and Ingham J. Effects of interface material on the performance of free rocking blocks. *Earthq Eng Struct Dyn.* 2010;40:375-392. doi:10.1002/eqe.1025
57. Makris N. The role of the rotational inertia on the seismic resistance of free-standing rocking columns and articulated frames. *Bull Seismol Soc Am.* 2014;104(5):2226-2239. doi:10.1785/0120130064
58. Priestley MJN, Evison RJ, Carr AJ. Seismic Response of Structures Free To Rock on Their Foundations. *Bull New Zeal Natl Soc Earthq Eng.* 1978;11(3):141-150.
59. Casapulla C. On the resonance conditions of rigid rocking blocks. *Int J Eng Technol.* 2015;7(2):760-771.
60. Psycharis IN, Jennings PC. Rocking of slender rigid bodies allowed to uplift. *Earthq Eng Struct Dyn.* 1983;11(1):57-76. doi:10.1002/eqe.4290110106
61. Prieto F, Peña F, Lourenço PB, Lemos J V. Numerical simulation of rigid blocks subjected to rocking motion. *Anal Hist Constr.* 2006;(Table I). <http://repositorium.sdum.uminho.pt/handle/1822/6551>.
62. Prieto F, Lourenço PB. On the rocking behavior of rigid objects. *Meccanica.* 2005;40(2):121-133. doi:10.1007/s11012-005-5875-7

63. Lipscombe PR, Pellegrino S. Free rocking of prismatic blocks. *J Eng Mech.* 1993;119(7):1387-1410.
64. Avgenakis E, Psycharis IN. Modeling of Inelastic Rocking Bodies under Cyclic Loading. *J Eng Mech.* 2020;146(4):1-19. doi:10.1061/(ASCE)EM.1943-7889.0001751
65. Psycharis IN. Dynamics of flexible systems with partial lift-off. *Earthq Eng Struct Dyn.* 1983;11(4):501-521. doi:10.1002/eqe.4290110405
66. Bachmann JA, Strand M, Vassiliou MF, Broccardo M, Stojadinović B. Is rocking motion predictable? *Earthq Eng Struct Dyn.* 2018;47(2):535-552. doi:10.1002/eqe.2978
67. Ma QT, Butterworth JW, Davidson BJ. Modelling rocking structures using standard finite elements. *2005 NZSEE Conf.* 2005;(20):1-8.
68. Vassiliou MF, Mackie KR, Stojadinović B. A finite element model for seismic response analysis of deformable rocking frames. *Earthq Eng Struct Dyn.* 2017;46(3):447-466. doi:10.1002/eqe.2799
69. Casapulla C, Maione A. A simplified equation of motion for free rocking rigid blocks. *Insights Innov Struct Eng Mech Comput - Proc 6th Int Conf Struct Eng Mech Comput SEMC 2016.* 2016;(October 2016):120-126. doi:10.1201/9781315641645-21
70. Casapulla C, Maione A. Free damped vibrations of rocking rigid blocks as uniformly accelerated motions. *Int J Struct Stab Dyn.* 2017;17(6):1-19. doi:10.1142/S0219455417500584
71. FEMA. Prestandard and Commentary for the Seismic Rehabilitation of Buildings. *Rehabil Requir.* 2000;(1):1-518.
72. Makris N, Konstantinidis D. The rocking spectrum and the limitations of practical design methodologies. *Earthq Eng Struct Dyn.* 2003;32(2):265-289. doi:10.1002/eqe.223
73. Chatzis MN, Smyth AW. Modeling of the 3D rocking problem. *Int J Non Linear Mech.* 2012;47(4):85-98. doi:10.1016/j.ijnonlinmec.2012.02.004
74. Konstantinidis D, Makris N. The dynamics of a rocking block in three dimensions. *8th HSTAM Int Congr Mech.* 2007;(September 2015):12-14.
75. Koh AS, Hsiung CM. Base isolation benefits of 3-D rocking and uplift. I: Theory. *J Eng Mech.* 1991;117(1):1-18.
76. Vassiliou MF. Seismic response of a wobbling 3D frame. *Earthq Eng Struct Dyn.* 2018;47(5):1212-1228. doi:10.1002/eqe.3013

77. Casapulla C, Maione A. Rocking resonance conditions of large and slender rigid blocks under the intense phase of an earthquake. *COMPdyn 2017 - Proc 6th Int Conf Comput Methods Struct Dyn Earthq Eng*. 2017;2(June 2017):2867-2878. doi:10.7712/120117.5612.17190
78. Beck JL, Skinner RI. The seismic response of a reinforced concrete bridge pier designed to step. *Earthq Eng Struct Dyn*. 1973;2(4):343-358. doi:10.1002/eqe.4290020405
79. Makris N, Vassiliou MF. Planar rocking response and stability analysis of an array of free-standing columns capped with a freely supported rigid beam. *Pacific Conf Earthq Eng*. 2013;42:431-449. doi:10.1002/eqe.2222
80. Vassiliou MF, Makris N. Dynamics of the vertically restrained rocking column. *J Eng Mech*. 2015;141(12):1-10. doi:10.1061/(ASCE)EM.1943-7889.0000953
81. Han Q, Jia Z, Xu K, Zhou Y, Du X. Hysteretic behavior investigation of self-centering double-column rocking piers for seismic resilience. *Eng Struct*. 2019;188(September 2018):218-232. doi:10.1016/j.engstruct.2019.03.024
82. Reggiani Manzo N, Vassiliou MF. Displacement-based analysis and design of rocking structures. *Earthq Eng Struct Dyn*. 2019;48(14):1613-1629. doi:10.1002/eqe.3217
83. Sieber M, Klar S, Vassiliou MF, Anastasopoulos I. Robustness of simplified analysis methods for rocking structures on compliant soil. *Earthq Eng Struct Dyn*. 2020;(May):1-18. doi:10.1002/eqe.3294
84. Casapulla C, Giresini L, Lourenço PB. Rocking and kinematic approaches for rigid block analysis of masonry walls: State of the art and recent developments. *Buildings*. 2017;7(3). doi:10.3390/buildings7030069
85. DeJong MJ, Dimitrakopoulos EG. Dynamically equivalent rocking structures. *Earthq Eng Struct Dyn*. 2014;43:1543-1563. doi:10.1002/eqe.2410
86. Cappelli E, Di Egidio A, Vestroni F. Analytical and Experimental Investigation of the Behavior of a Rocking Masonry Tuff Wall. *J Eng Mech*. 2020;146(6):1-13. doi:10.1061/(ASCE)EM.1943-7889.0001775
87. Thiers-Moggia R, Málaga-Chuquitaype C. Dynamic response of post-tensioned rocking structures with inerters. *Int J Mech Sci*. 2020;187(June):105927. doi:10.1016/j.ijmecsci.2020.105927
88. Tomassetti U, Graziotti F, Sorrentino L, Penna A. Modelling rocking response via equivalent viscous damping. *Earthq Eng Struct Dyn*. 2019;48(11):1277-1296. doi:10.1002/eqe.3182
89. Como M, Di Carlo F, Coccia S. Dynamic response of rocking cracked masonry

- walls. *Meccanica*. 2019;54(3):381-398. doi:10.1007/s11012-019-00949-w
90. Casapulla C, Giresini L, Argiento LU, Maione A. Nonlinear Static and Dynamic Analysis of Rocking Masonry Corners Using Rigid Macro-Block Modeling. *Int J Struct Stab Dyn*. 2019;19(11):1-32. doi:10.1142/S0219455419501372
 91. Polyakov S V. *Design of Earthquake Resistant Structures*. 2nd ed. Moscow: Mir Publishers; 1985.
 92. Blebo FC, Roke DA. Seismic-resistant self-centering rocking core system with buckling restrained columns. *Eng Struct*. 2018;173(July):372-382. doi:10.1016/j.engstruct.2018.06.117
 93. Liu Y, Guo Z, Liu X, Chicchi R, Shahrooz B. An innovative resilient rocking column with replaceable steel slit dampers: Experimental program on seismic performance. *Eng Struct*. 2019;183(May 2018):830-840. doi:10.1016/j.engstruct.2019.01.059
 94. Azuhata T, Midorikawa M, Ishihara T. Earthquake damage reduction of buildings by self-centering systems using rocking mechanism. *14th World Conf Earthq Eng*. 2008.
 95. Gelagoti F, Kourkoulis R, Anastasopoulos I, Gazetas G. Rocking isolation of low-rise frame structures founded on isolated footings. *Earthq Eng Struct Dyn*. 2012;41:1177-1197. doi:10.1002/eqe.1182
 96. Ríos-García G, Benavent-Climent A. New rocking column with control of negative stiffness displacement range and its application to RC frames. *Eng Struct*. 2020;206(May 2019):110133. doi:10.1016/j.engstruct.2019.110133
 97. Hashemi A, Quenneville P. Seismic performance of timber structures using rocking walls with low damage hold-down connectors. *Structures*. 2020;27(May):274-284. doi:10.1016/j.istruc.2020.05.050
 98. Bachmann JA, Jost C, Studemann Q, Vassiliou MF, Stojadinović B. An analytical model for the dynamic response of an elastic SDOF system fixed on top of a rocking singlestory frame structure: Experimental validation. *ECCOMAS Congr 2016 - Proc 7th Eur Congr Comput Methods Appl Sci Eng*. 2016;3(October):5023-5062. doi:10.7712/100016.2164.10112
 99. Sharpe RD, Skinner RI. Seismic Design of an Industrial Chimney With Rocking Base. *Bull New Zeal Natl Soc Earthq Eng*. 1983;16(2):98-106.

APPENDICES

A. Preliminary Design Table for ROC seismic isolation system

Number of stories vs earthquake tables. Rocking column dimensions width (b) and height (h) parameters are given below:

5m bay distance															
Soil type / earthquake	4-story building					5-story building					6-story building				
	b (m)	h (m)	V _{max} /W (%)	Displ. Demand (m)	Acceleration (g)	b (m)	h (m)	V _{max} /W (%)	Displ. Demand (m)	Acceleration (g)	b (m)	h (m)	V _{max} /W (%)	Displ. Demand (m)	Acceleration (g)
Soft soil (El Centro 1940)	0.3	3	0.18	0.07	0.29	0.3	3	0.20	0.08	0.52					
	0.3	4	0.16	0.14	0.27	0.3	4	0.14	0.17	0.38					
	0.4	4	0.19	0.09	0.26	0.4	4	0.18	0.09	0.35					
	0.5	4	0.23	0.08	0.28	0.5	4	0.22	0.08	0.33					
Rock soil (Kobe 1995)	0.3	4	0.15	0.12	0.33	0.3	4	0.14	0.15	0.39					
						0.4	4	0.18	0.10	0.46					
Northridge	0.3	4	0.15	0.11	0.21	0.3	4	0.04	0.35	0.29	0.3	4	0.13	0.11	0.42
	0.4	4	0.21	0.09	0.22	0.4	4	0.09	0.21	0.41					
Erzincan	0.4	4	0.37	0.21	0.42	0.4	4	0.29	0.36	0.43					
Loma Prieta	0.3	4	0.17	0.11	0.31	0.3	4	0.16	0.16	0.40					
	0.4	4	0.22	0.10	0.41										
Kalamata	0.3	3	0.19	0.09	0.41	0.3	4	0.12	0.26	0.36					
	0.4	3	0.23	0.07	0.36	0.4	4	0.16	0.24	0.37					
	0.5	3	0.30	0.04	0.38	0.5	4	0.21	0.16	0.37					
	0.6	3	0.34	0.03	0.41	0.6	4	0.26	0.09	0.41					
	0.3	4	0.11	0.07	0.27										
	0.4	4	0.18	0.09	0.32										
	0.5	4	0.24	0.09	0.33										
	0.6	4	0.28	0.08	0.36										

Note:

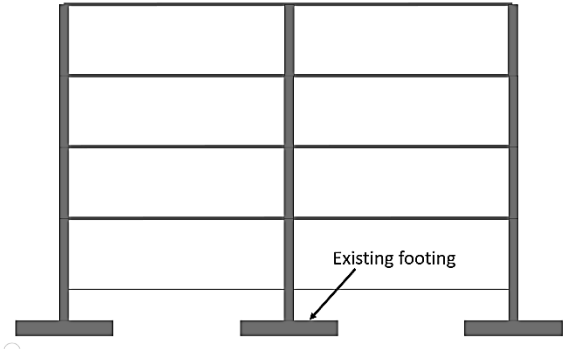
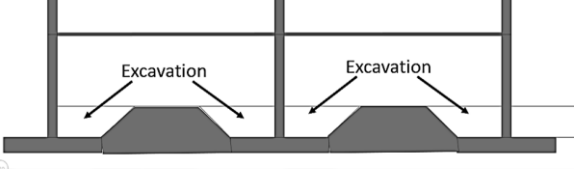
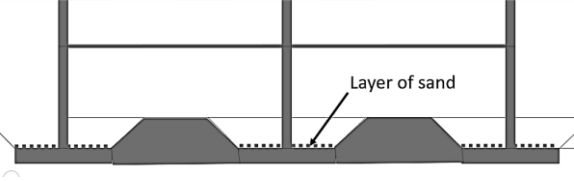
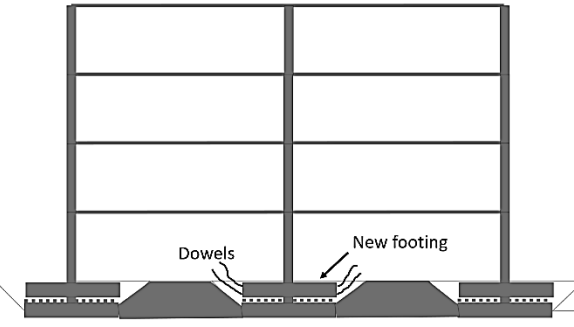
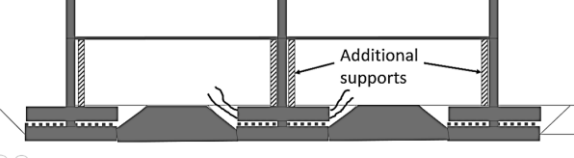
- 1) No suitable results were possible for Duzce earthquake
- 2) No suitable results were possible for 7 story buildings

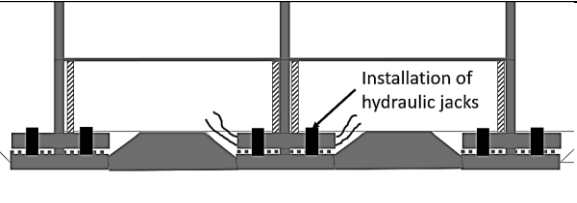
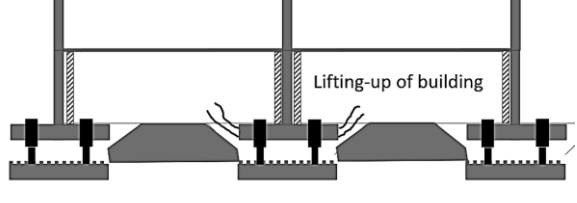
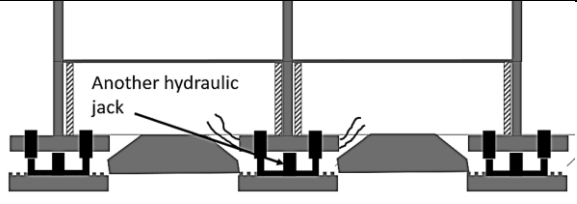
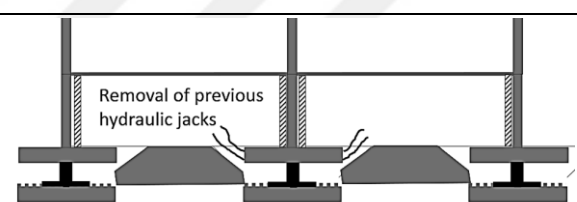
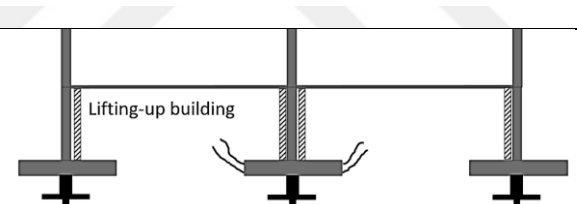
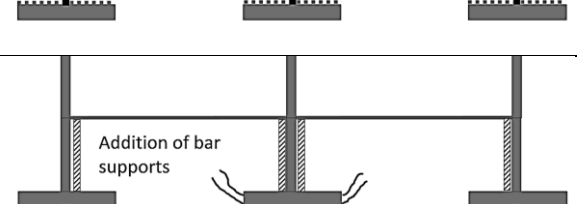
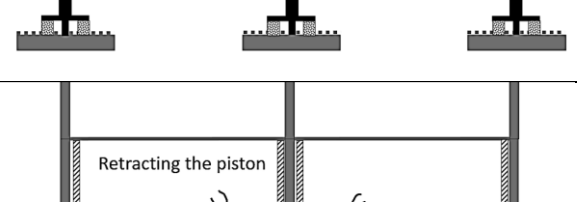
7.5m bay distance											
Soil type / earthquake	4-story building						5-story building				
	b (m)	h (m)	Vmax/W (%)	Displ. Demand (m)	Acceleration (g)	b (m)	h (m)	Vmax/W (%)	Displ. Demand (m)	Acceleration (g)	
Soft soil (El Centro 1940)	0.3	4	0.11	0.22	0.29	0.3	4	0.10	0.24	0.43	
	0.4	4	0.16	0.18	0.37						
	0.5	4	0.18	0.10	0.33						
	0.6	4	0.21	0.09	0.37						
Rock (Kobe 1995)	0.3	4	0.11	0.21	0.29	0.3	4	0.11	0.20	0.42	
	0.4	4	0.15	0.13	0.37						
Northridge	0.3	4	0.10	0.12	0.27						
	0.4	4	0.15	0.10	0.33						
	0.5	4	0.23	0.10	0.39						
	0.6	4	0.28	0.11	0.38						
Kalamata	0.3	4	0.09	0.08	0.31						
	0.4	4	0.13	0.11	0.38						
	0.5	4	0.17	0.11	0.35						
	0.6	4	0.21	0.09	0.28						
Loma Prieta	0.3	4	0.13	0.19	0.34						
	0.4	4	0.17	0.12	0.43						

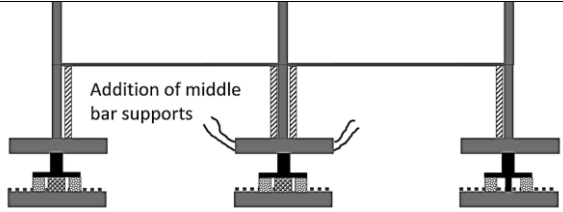
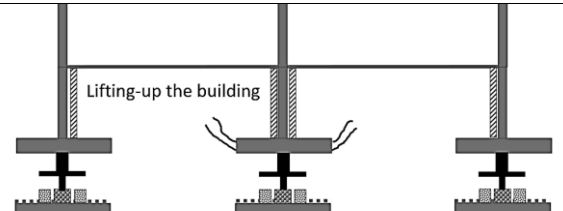
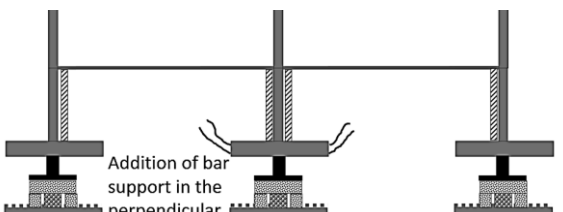
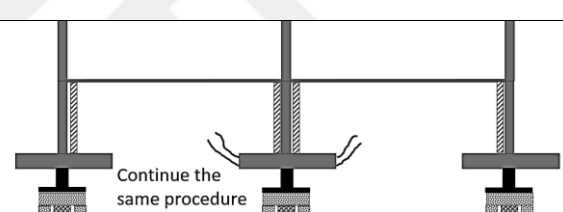
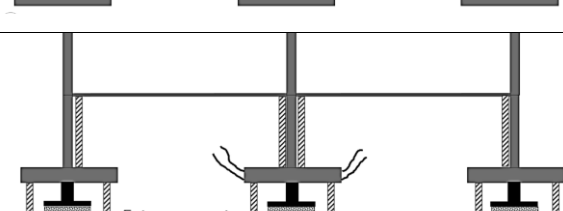
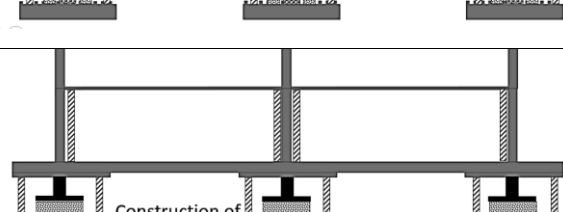
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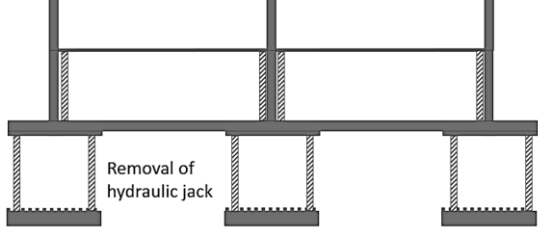
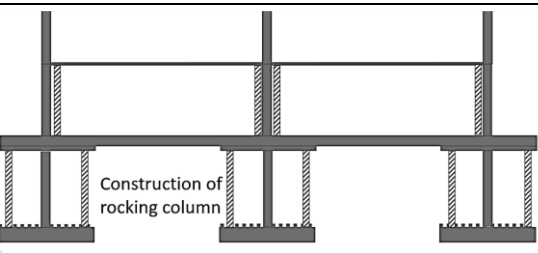
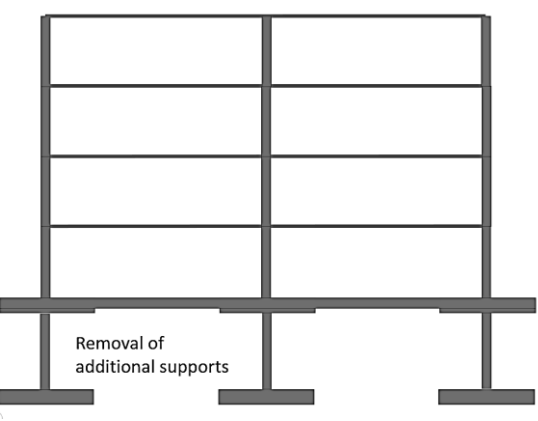
- 1) no suitable results were possible for Duzce and Erzincan earthquakes
- 2) no suitable results were possible for 3m height of rocking column
- 3) no suitable results were possible for 6 and 7 story buildings

B. Lift-up of buildings

Lift-up of an existing building for the construction of a Rocking-Column (ROC) seismic isolation system may consists of the following steps.	 Existing footing
Excavate around the columns	 Excavation
Pour a layer of sand on the existing footing and maintain its thickness according to requirement	 Layer of sand
Construct new footings about the sand layer having proper anchorages with the existing columns, dowel steel bars for construction of slab, and proper grooves for installation of hydraulic jacks	 Dowels New footing
Provide supports to the existing columns and cut the columns just below the new footings	 Additional supports

Install hydraulic jack within the grove assembly of new footings to lift-up the structure	 Installation of hydraulic jacks
Lift-up the building with the help of hydraulic jacks (up to its stroke length limit)	 Lifting-up of building
Install another hydraulic jack in between the two footings (already existing and newly constructed)	 Another hydraulic jack
Remove the previous hydraulic jacks from the building. Now the building is supported by new hydraulic jacks only	 Removal of previous hydraulic jacks
Lift-up the structure with the help of new hydraulic jacks	 Lifting-up building
Add bar supports at two ends. These bar supports are strong enough to carry the load of whole building	 Addition of bar supports
Retract the piston of hydraulic jack into its neutral position such that whole structure is resting on the bar supports	 Retracting the piston

Add a middle bar support just below the piston position of hydraulic jack	
Lift-up the building again. This time piston is applying the load on bar support instead of existing footing	
Add couple of bar supports in perpendicular direction, retract the piston of hydraulic jack and add middle bar support just below the piston position	
Repeat the same procedure to achieve required story height	
Provide extra supports for safety of new story and scaffolding for construction of slab	
Construct a slab after providing proper connections to the dowels and anchorages	

Remove the assembly of hydraulic jacks	 Removal of hydraulic jack
Construct the rocking columns. Provide steel ends to avoid the crushing of concrete and restraining mechanism to prevent collapse	 Construction of rocking column
Remove the additional supports. Hence the ROC seismic isolation system may be applied to the existing buildings.	 Removal of additional supports
References: Jack-up and moving of buildings and structures. (2020, October 4). Retrieved from http://www.solestech.it/articolo.asp?articolo=506&lng=EN. Climbing Jacks. (2020, October 9). Retrieved from https://hyquip.co.uk/specialty-jacks-and-lifting-products/4855-enerpac-bls-series-climbing-jacks.html Mammoet Hydraulic Climbing Jacks (2020, October 9). Retrieved from https://www.youtube.com/watch?v=bPkVmOd16xw Jack-up System Enerpac Heavy Lifting Technology (2020, October 9). Retrieved from https://www.youtube.com/watch?v=YLYf9kIux8	

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