

T.C.
İSTANBUL KÜLTÜR UNIVERSITY
INSTITUTE OF GRADUATE STUDIES

**PARAMETRIC INVESTIGATION OF THE SEISMIC RESPONSE OF
STRUCTURES WITH TRIPLE FRICTION PENDULUM ISOLATORS**

Master of Science Thesis

Abdulaziz MAZED

1800004570

Department: Civil Engineering

Programme: Structural Engineering

Supervisor: Assist. Prof Dr. Gökhan YAZICI

August 2020

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Supervisor and Chairperson:	Asst. Prof. Dr. Gökhan YAZICI
Members of Examining Committee:	Asst. Prof. Dr. Erdal COŞKUN
	Asst. Prof. Dr. Cenk ÜSTÜNDAĞ

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Preamble:

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Abbreviations:

SFP: Single Friction Pendulum

DFP: Double Friction Pendulum

TFP: Triple Friction Pendulum

TFPB: Triple Friction Pendulum Bearing

FNA: Fast Non-linear Analysis

TFA: Triple Friction Pendulum using Type “A” Material

TFB: Triple Friction Pendulum using Type “B” Material

TFC: Triple Friction Pendulum using Type “C” Material

TFD: Triple Friction Pendulum using Type “D” Material

TFE: Triple Friction Pendulum using Type “E” Material

TFF: Triple Friction Pendulum using Type “F” Material

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List of Symbols:

R_1 = Radius of The Outer Bottom Concave Surface of the Triple Friction Pendulum

R_2 = Radius of The Inner Bottom Concave Surface of the Triple Friction Pendulum

R_3 = Radius of The Inner Top Concave Surface of the Triple Friction Pendulum

R_4 = Radius of The Outer Top Concave Surface of the Triple Friction Pendulum

d_1 = Displacement Capacity of The Outer Bottom Surface of The Triple Friction Pendulum

d_2 = Displacement Capacity of The Inner Bottom Surface of The Triple Friction Pendulum

d_3 = Displacement Capacity of The Inner Top Surface of The Triple Friction Pendulum

d_4 = Displacement Capacity of The Outer Top Surface of The Triple Friction Pendulum

h_1 = Height of The Outer Bottom Surface of The Triple Friction Pendulum

h_2 = Height of The Inner Bottom Surface of The Triple Friction Pendulum

h_3 = Height of The Inner Top Surface of The Triple Friction Pendulum

h_4 = Height of The Outer Top Surface of The Triple Friction Pendulum

μ_1 = Friction Coefficient of The Outer Bottom Surface of The Triple Friction Pendulum

μ_2 = Friction Coefficient of The Inner Bottom Surface of The Triple Friction Pendulum

μ_3 = Friction Coefficient of The Inner Top Surface of The Triple Friction Pendulum

μ_4 = Friction Coefficient of The Outer Top Surface of The Triple Friction Pendulum

W = Weight Applied on The Triple Friction Pendulum Bearing

ΣW = Total Weight Applied on All Bearings

Üniversite:
Enstitü:
Anabilim Dalı:
Programı:
Tez Danışmanı:
Tez Türü ve Tarihi:

İstanbul Kültür Üniversitesi
Lisansüstü Eğitim Enstitüsü
İnşaat Mühendisliği
Yapı (İngilizce)
Dr. Öğretim Üyesi Gökhan YAZICI
Yüksek Lisans – Ağustos 2020

KISA ÖZET

ÜÇLÜ SÜRTÜNMELİ SARKAÇ SİSTEMLİ YAPILARIN DEPREM YÜKLERİ ALTINDA DAVRANIŞININ PARAMETRİK OLARAK İNCELENMESİ

Abdulaziz MAZED

Bu çalışmada, üçlü sürtünmeli sarkaç sistemli beş katlı bir çelik yapının deprem yükleri altında davranışı parametrik olarak incelenmiştir. Çalışmada kullanılan yapı modelinden elde edilen sonuçlar, literatürde verilen bir çalışma ile doğrulanmıştır. Parametrik çalışma kapsamında, ilk aşamada ön tasarım ile ortak izolatör boyutları seçilmiş olup, farklı sürtünme katsayıları ve deprem kayıtları altında yapının davranışı incelenmiştir. Analizler için yakın fay ve uzak fay kayıtlarından oluşan 10 farklı deprem kullanılmış ve bu deprem kayıtları servis (SLE), tasarım (DBE) ve maksimum olası deprem (MCE) seviyelerine göre ölçeklenmiştir. Zaman tanım alanında gerçekleştirilen analizlerde üst yapının doğrusal davranış gösterdiği, sismik yalıtım sisteminin ise doğrusal olmayan davranış gösterdiği kabul edilmiştir. Analiz sonuçlarının değerlendirilmesinde, izolatör yer değiştirmeleri, mutlak ivme, bağıl kat öteleme oranı, taban kesme kuvvet/ağırlık oranı ve histeretik döngüler göz önüne alınmıştır. Sonuç olarak, $(\mu_1-\mu_2)$ değerinin düşük olduğu vakalarda daha iyi performans elde edildiği ve uzak alan kayıtlarının kullanıldığı vakalardan elde edilen sonuçların yakın fay depremlerinden elde edilen sonuçlara göre daha tahmin edilebilir olduğu görülmüştür.

Anahtar Kelimeler: Depreme Dayanıklı Tasarım, Deprem Mühendisliği, Sismik izolasyon, Üçlü Sürtünmeli Sarkaç Sistemi, Sürtünmeli Sarkaç Sistemi

University : **İstanbul Kültür University**
Institute : **Institute of Graduate Studies**
Department : **Civil Engineering**
Programme : **Structural Engineering**
Supervisor : **Assist. Prof. Dr. Gökhan YAZICI**
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ABSTRACT

PARAMETRIC INVESTIGATION OF THE SEISMIC RESPONSE OF STRUCTURES WITH TRIPLE FRICTION PENDULUM ISOLATORS

Abdulaziz MAZED

In this study, the response of a five-storey steel structure with a triple friction pendulum system under earthquake loads was investigated parametrically. Within the scope of the parametric study, in the first stage, common isolator dimensions were selected with preliminary design, and the response of the structure was examined under different friction coefficients and earthquake records. For analysis, 10 different earthquakes consisting of near fault and far field records were selected and scaled to service (SLE), design (DBE) and maximum possible earthquake (MCE) levels. In the analysis performed in the time domain, it was assumed that the superstructure shows linear behavior and the seismic insulation system shows nonlinear behavior. In evaluating the analysis results, isolator displacements, absolute acceleration, inter-storey drift, base shear force/weight ratio and hysteretic loops were taken into consideration. As a result, it has been observed that better performance is obtained in cases where (μ_1 - μ_2) value is low and the results obtained from cases where remote field records are used are more predictable than the results obtained from near fault earthquakes.

Key Words: Earthquake Resistant Design, Earthquake Engineering, Seismic isolation, Triple Friction Pendulum System, Friction Pendulum System

Science Code: 624.03.01

1. Introduction:

1.1. General Introduction:

Resisting earthquake hazard is one of major challenges structural buildings face. Earthquake engineering focus on making structures withstand the lateral forces and deformations resulted out of seismic motion. There are several techniques developed to do this function and could be classified into the following: [1]

- Passive Control Devices, do not require an energy source or real time operation. These devices are widely used in base isolation systems. [2]
- Active Control Devices, have real time computation and operating tools connected to sensors. Their operation involves processing of response within the structure from the feedback coming out of sensors, and actively controlling the forces are developed to the structure.[3]
- Semi-Active Control Devices, are a combination of passive and active control systems. These devices act mostly as passive control systems and the mechanical properties of the system may be adjusted with the active component. Semi-Active devices demand little measurement operations of external forces. [4]

When the seismic wave penetrates up to the structure, there are several approaches to control its damage and energy coming out of it and improve the superstructure response, for example:

- By Dissipating the wave energy inside the structure.
- By Dissipating the wave energy of wider range of frequencies part.
- By Dissipating and absorb the wave energy of the whole wave frequencies.

Among all different earthquake resistant design techniques, base isolation systems were proven to be efficient during the last four decades. The idea of base isolation is to separate the superstructure from the ground with base isolation bearings. Base

isolation bearings increase the vibration period of the structure and dissipate the energy coming from ground motion instead of letting the building fully resist the seismic load. Base isolation systems can be categorized into two main groups, Elastomeric and Sliding Systems. Examples for elastomeric bearings include laminated rubber bearings, lead rubber bearings and high damping rubber bearings. On the other side, the category of Sliding System Isolators itself has many types under it, starting from Single Friction Pendulum & Double Friction Pendulum and reaching the recent friction isolator developed which is Triple Friction Pendulum Bearing (TFPB) that allows movement on different concave surfaces with different frictions coefficients. The problem with Single Friction Pendulum, containing only one friction coefficient which means small lateral forces when it occurs the system won't move and building will absorb the entire load. It dissipates the wave energy of wider range of frequencies part only. Therefore, the need for TFPB came in place with its different concave surfaces with different friction coefficients that allow later displacements on different ranges of earthquake ground motions.

1.2. Motivation:

Investigating Triple Friction Pendulum Bearing's parameters and understanding the effect of changing these parameters is very significant for design engineers and researchers as well. This type of researches that focus on the practical side by testing common dimensions produced by manufactures will not only help manufactures themselves noticing the effects of changing these parameters but might also lead to develop formulas in the future to choose optimum or limit isolator dimensions depends on expected earthquake level designed for.

1.3. History upon Seismic Isolation Systems:

Back in 1876, where modern form of seismic isolation system was initiated, John Milne professor in mining department at Tokyo Imperial University, he made the first seismic isolated structure which was standing upon spherical balls inside cast-iron plates with Plate-like edges over the pile heads. Later in 1885, the structure experimented under earthquake tremor and was reported by British Association for the

Advancement of Science [5]. In 1992, National Institute of Standards and Technology (NIST) published about these two volume reports [6],[7]. At the beginning of 1980s, many Japanese construction companies established experiments on isolation systems using natural rubber bearing. The aim of the design model was to allow some displacement using rubber material, keeping in mind the vertical stiffness. Therefore, steel plates were added between the rubber layers as is it shown in Figure 1.1. [8]

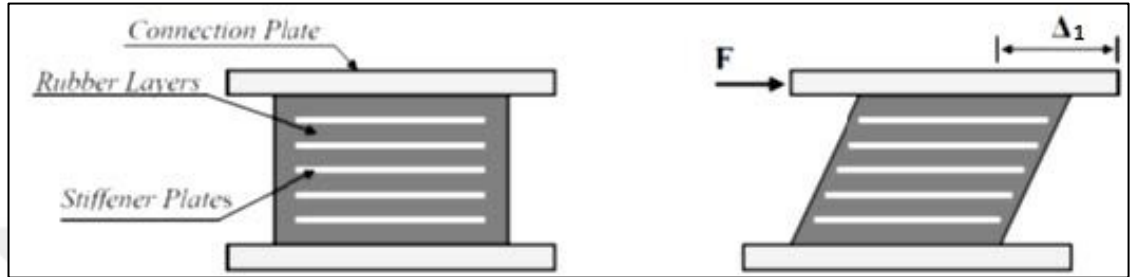


Fig. 1.1: Steel Reinforced Elastomeric Isolator Section (SREI) [8]

Nowadays, Steel Reinforced elastomeric isolators (SREI) has been used for different types of structures even for private buildings for some cases [8]. SREI was developed by adding lead-plug embedded inside spherical gap in middle of it in order to increase isolator damping (see Figure 1.2).

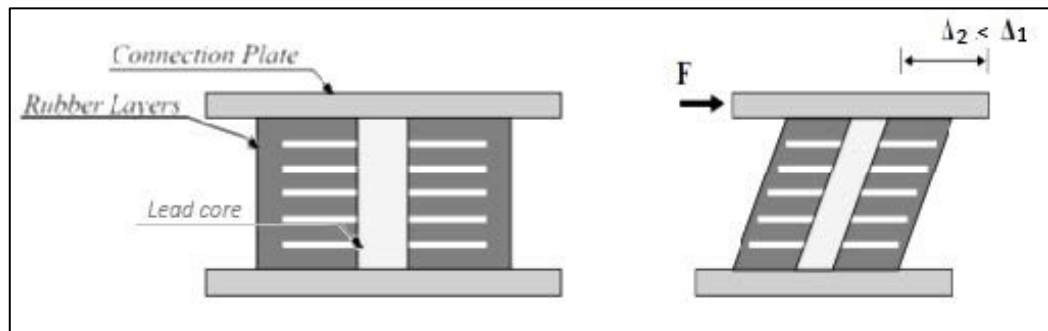


Fig. 1.2: Lead Plug Rubber Bearing Section [8]

First propositions on sliding base isolation systems were not efficient to re-locate the structure to its original position. Zayas et al [9]. tested a primitive pendulum system in attempt to reply the sliding roller that has lack of re-focusing. The suggested system was made of two curved surfaces with a slider positioned in the middle of buried

surfaces. Friction Pendulum is designed to re-position the building to its original position. System is illustrated in the Figure 1.3.

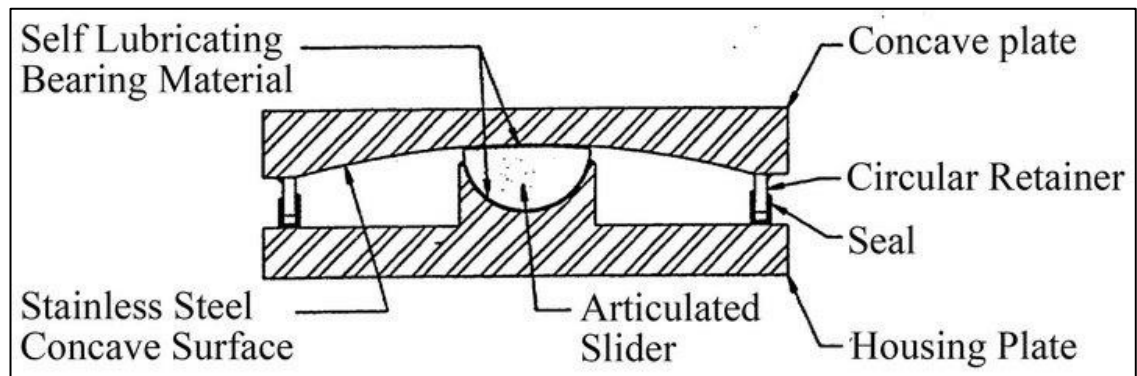


Fig. 1.3: Primitive Friction Pendulum Bearing Section [9]

2. Literature Review:

To understand how changing parameters of the TFB effect on the non-linear behavior we need to understand how TFB works in first place. Fig 2.1 illustrates the geometry of TFB.

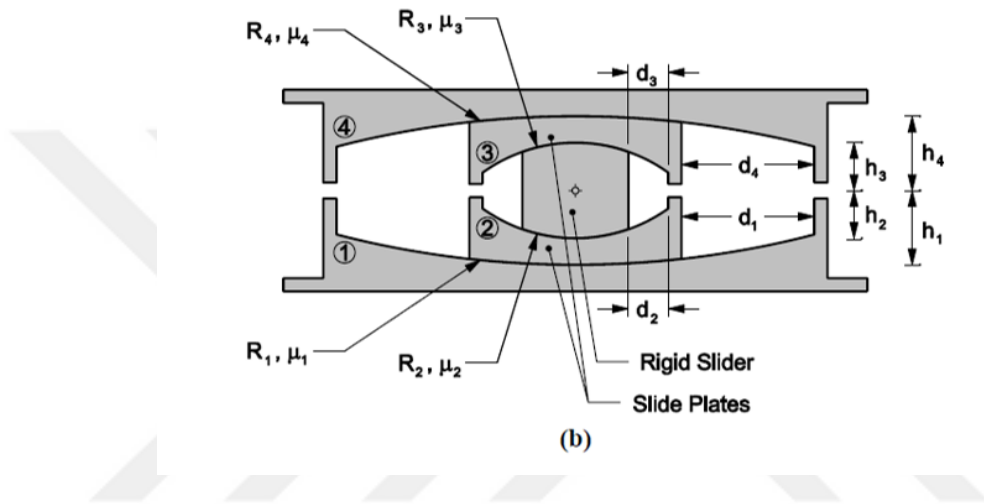


Fig. 2.1: Triple Friction Pendulum Bearing's Parameters [10]

The Triple Friction pendulum has four main concave surfaces, it is usually numbered from down to top. each surface has its own Radius (R), Height (h), Friction Coefficient (μ) and Displacement Capacity (d). The middle surfaces (2 and 3) normally have same properties ($R_2, h_2, \mu_2 = R_3, h_3, \mu_3$). For the lower and upper concave surfaces, they are also symmetrical in some cases and in other cases their Friction Coefficients are not the same values where $\mu_1 \neq \mu_4$.

The Force-Displacement Relation of the TFP isolator is showed in Figure 5. It illustrated the Five loops, each one of these loops stands for one of five different regimes of displacement. Geometry, the gravity load W carried by the isolator and Friction coefficients values of the bearing controls the loop parameters. as illustrated by Fenz and Constantinou (2008a & 2008b) [10,11]. Triple Friction Pendulum isolators are typically designed to operate in regimes I to IV, whereas regime V is reserved to act as a displacement restrainer. Where the isolator has consumed its

displacement capacities from d_1 to d_4 and only sliding on surfaces 2 & 3 left. (see Fig 2.2).

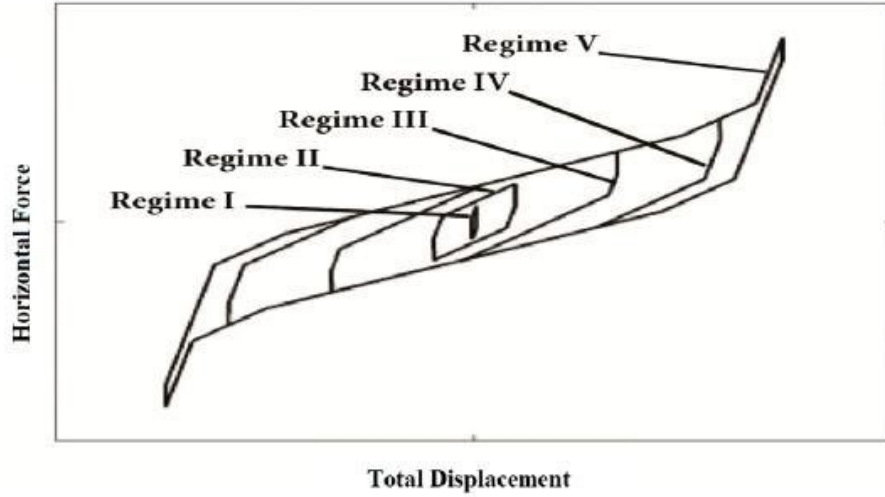


Fig. 2.2: Force-Displacement Loops of Triple Friction Pendulum Bearing [10]

A simplified Tri-linear hysteric behavior by Fenz and Constantinou 2008a [10] illustrated the Force-Displacement Relationship as it showed in Figure 2.3. Note it is only for special configuration where $R_{eff1} = R_{eff4}$, $R_{eff2} = R_{eff3}$, $d_1^* = d_4^*$, $d_2^* = d_3^*$, $\mu_1 = \mu_4$ and $\mu_2 = \mu_3$. In addition, it should be considered that the bearing won't reach regime V. The elastic behavior begins when the isolator starts moving on the inner plates at point $(\mu_2 \times W)$ where any lateral force less than this the pendulum won't move. The pendulum starts moving on outer plates after reaching $(\mu_1 \times W)$ and causing displacement equals to u^* .

Force-Displacement Relationship for this special configuration equation given by [10]:

$$F = \frac{W}{2R_{eff1}} u + \left[\mu_1 - (\mu_1 - \mu_2) \frac{R_{eff2}}{R_{eff1}} \right] W \quad (1)$$

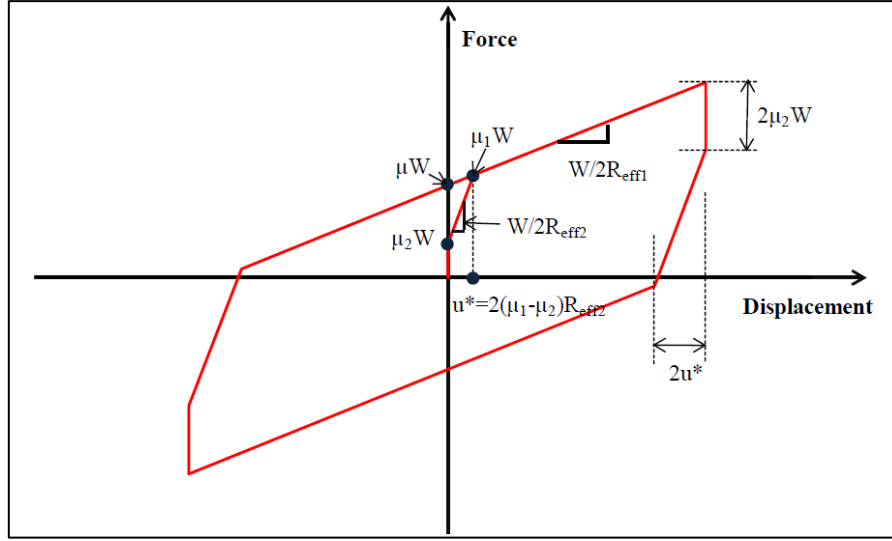


Fig. 2.3: Lateral Force Displacement Loop of Triple FP Bearing [12]

The given equation (1) is valid until the lateral force and displacement reach the values given by the following equations [10]:

$$F = \frac{W}{R_{eff1}} d_1^* + F_{f1} \quad (2)$$

$$u = u^* + 2 d_1^* = 2(\mu_1 - \mu_2)R_{eff2} + 2d_1^* \quad (3)$$

$$u^* = u^{**} = 2(\mu_1 - \mu_2)R_{eff2} \quad (4)$$

The Force at Zero Displacement is given by [10]:

$$\mu W = \left[\mu_1 - (\mu_1 - \mu_2) \frac{R_{eff2}}{R_{eff1}} \right] W \quad (5)$$

At u^* displacement is the end of elastic behavior. After that point, most of the energy will be dissipated throw displacement made by the isolator, and start doing its expected role.

At the upper corner of the loop point where maximum capacity is and the curve about to drop downward, we could find the actual maximum isolator displacement for the cycle. However, Design Displacement (Dd) could be possibly taken as half of maximum capacity (depends on what earthquake level designed for).

The simple Tri-linear hysteric behavior is practical for the special configuration. However, can't be used for other configurations. According to Fenz and Costantino (2008a, 2008b) [10,11], TFPB and its Force-Displacement hysteresis loop for the Five regimes summed up in Table 2.1.

Table 2.1: Summery of Force-Displacement Behavior of TFP (Fenz & Constantinou 2008a) [10,11].

Regime	Description	Force-Displacement Relationship	
I	Sliding on surfaces 2 and 3 only	$F = \frac{W}{R_{eff2} + R_{eff3}} u + \frac{F_{f2}R_{eff2} + F_{f3}R_{eff3}}{R_{eff2} + R_{eff3}}$ Valid until: $F = F_{f1}, \quad u = u^* = (\mu_1 - \mu_2)R_{eff2} + (\mu_1 - \mu_3)R_{eff3}$	
II	Motion stops on surface 2; Sliding on surfaces 1 and 3	$F = \frac{W}{R_{eff1} + R_{eff3}} u + \frac{F_{f1}(R_{eff1} - R_{eff2}) + F_{f2}R_{eff2} + F_{f3}R_{eff3}}{R_{eff1} + R_{eff3}}$ Valid until: $F = F_{f4}, \quad u = u^{**} = u^* + (\mu_4 - \mu_1)(R_{eff1} + R_{eff3})$	
III	Motion is stopped on surfaces 2 and 3; Sliding on surfaces 1 and 4	$F = \frac{W}{R_{eff1} + R_{eff4}} u + \frac{F_{f1}(R_{eff1} - R_{eff2}) + F_{f2}R_{eff2} + F_{f3}R_{eff3} + F_{f4}(R_{eff4} - R_{eff3})}{R_{eff1} + R_{eff4}}$ Valid until: $F = F_{d1} = \frac{W}{R_{eff1}} d_1 + F_{f1}, \quad u = u_{d1} = u^{**} + d_1 \left(1 + \frac{R_{eff4}}{R_{eff1}} \right) - (\mu_4 - \mu_1)(R_{eff1} + R_{eff4})$	
IV	Slider contacts restrainer on surface 1; Motion remains stopped on surface 3; Sliding on surface 2 and 4	$F = \frac{W}{R_{eff2} + R_{eff4}} (u - u_{d1}) + \frac{W}{R_{eff1}} d_1 + F_{f1}$ Valid until: $F = F_{d4} = \frac{W}{R_{eff4}} d_4 + F_{f4}, \quad u = u_{d4} = u_{d1} + \left[\left(\frac{d_4}{R_{eff4}} + \mu_4 \right) - \left(\frac{d_1}{R_{eff1}} + \mu_1 \right) \right] (R_{eff2} + R_{eff4})$	
V	Slider bears on restrainer of surface 1 and 4; Sliding on surfaces 2 and 3	$F = \frac{W}{R_{eff2} + R_{eff3}} (u - u_{d4}) + \frac{W}{R_{eff4}} d_4 + F_{f4}$	
Assumptions: (1) $R_{eff1} = R_{eff4} \gg R_{eff2} = R_{eff3}$, (2) $\mu_2 = \mu_3 < \mu_1 < \mu_4$, (3) $d_1 > (\mu_4 - \mu_1)R_{eff1}$, (4), $d_2 > (\mu_1 - \mu_2)R_{eff2}$ (5) $d_3 > (\mu_4 - \mu_3)R_{eff3}$			

For detailed representation and different types of configurations cases Fenz and Constantinou suggested some formulas to represent Force-Displacement of Triple FP bearing (Figure 2.4) other than the original theory (Figure 2.5), some were special for certain cases, others were more general for general configurations such as $R1 \neq R4$ case, for more details check MCEER-13-0010 technical report [12].

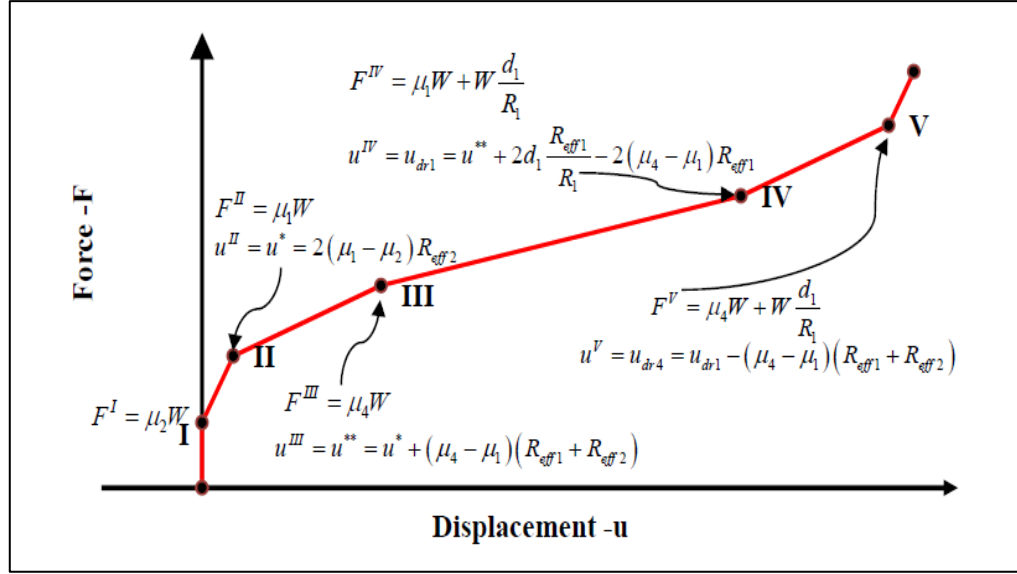


Fig. 2.4: Force Displacement of Triple FP Bearing of Configuration A with $R_1=R_4$, $\mu_2=\mu_3$, $R_2=R_3$, $h_1=h_4$ and Based on The Theory of Fenz and Constantinou [12]

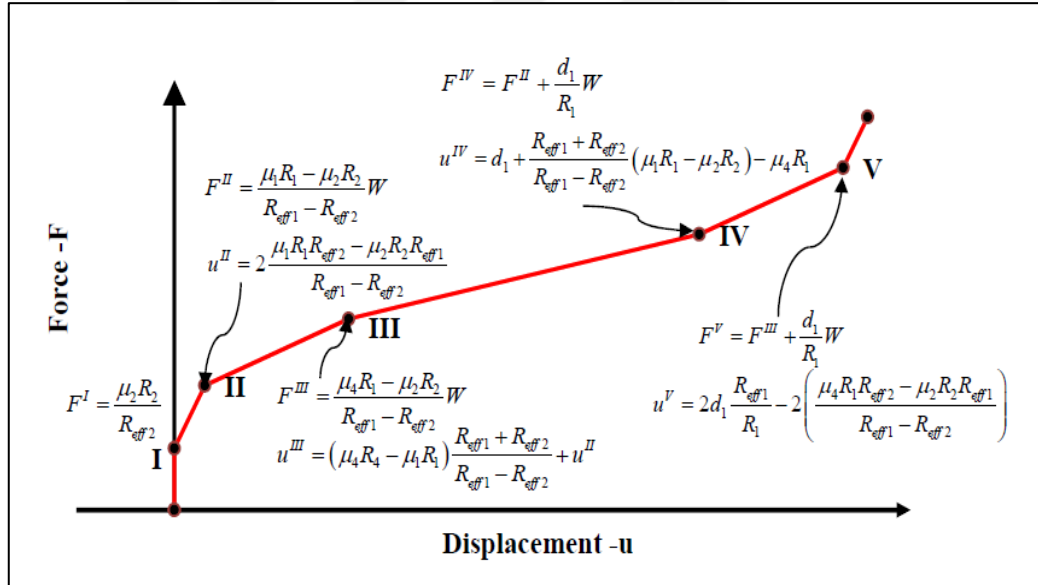


Fig. 2.5: Force Displacement of Triple FP Bearing of Configuration A with $R_1=R_4$, $\mu_2=\mu_3$, $R_2=R_3$, $h_1=h_4$ and Based on The Current Theory [12]

Implementing three single friction pendulums connected in series with two gap elements were done by Fenz and Constantinou to replace Triple Friction Pendulum Behaviour because before 2016, there wasn't any feature available for it in SAP2000 or ETABS. Fortunately, since 2016 the CSI implemented the TFPB to the program and allowed us to define it as one link element. If we are going to deal with modeling Triple Friction Pendulum bearings in ETABS or SAP2000 we must mention Kitayama and

Constantinou et al. evaluated the new TFPB feature in SAP2000 [13] which simplifies the series modelling with three single friction pendulums and two gap elements. However, as the data of the three single friction pendulums in series were compared in the previous paper, it will be used to compare the Time History analysis approach to make sure substitutions were done correctly and giving realistic results.



3. Verification Model of Triple Friction Bearing:

Kitayama and Constantinou worked on one-dimensional model from Sarlis and Constantinou's paper (see Figure 3.1), Modes were checked first to make sure the building model defined right (see Table 3.1). Then, the New Triple FB element were entered with its' properties as it is mentioned in the paper (see Table 3.2). In this study, we conducted same Time History analysis to test new Triple FP element only as it is our concern in this report. Results in this study were obtained from ETABS for new TFPB feature, which Constantinou et al. conducted in SAP2000 for both new TFPB feature and series model [14].

Using TCU-065-E component from the unscaled 1999 Chi-Chi, Taiwan earthquake record, each isolator assumed to carry a load of 900kips (4003 kN), total weight on the isolators about 5400kips (24020 kN). The mass is added at the joints as shown in Figure 9 with all joints have same mass distribution. Periods of vibration for the first four modes of the fixed base superstructure is presented in Table 3.1.

Table 3.1: Periods of Vibration of Superstructure when Fixed [14]

Mode	Period (Seconds)
1	0.776
2	0.253
3	0.138
4	0.093

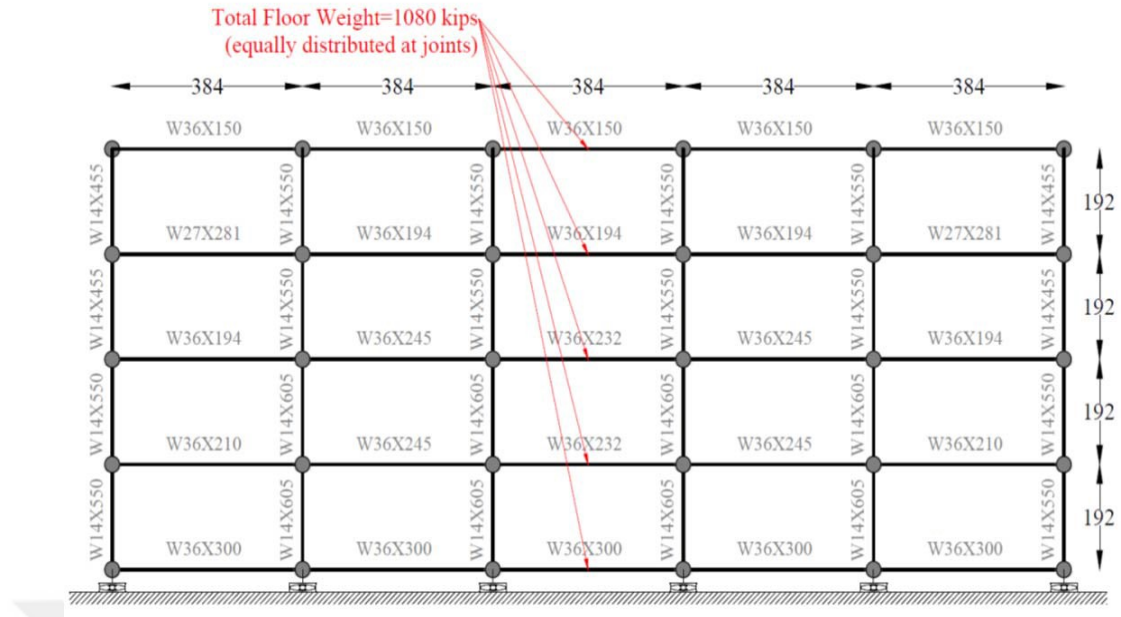


Fig. 3.1: Graphic of the Four-Storey Structure (dimensions in inch) [14]

Structural critical damping of 2% were taken for all modes and assumed (constant damping model). FNA Analysis was conducted using 200 Ritz vector modes. Table 3.2, presents the properties of Triple FP element used by Kitayama & Constantinou [14].

Table 3.2: Properties of the Triple Pendulum Isolator Element Evaluated by Sarlis & Constantinou [14]

Element Mass (kip-s ² /in)	0.003			
Effective Stiffness (kip/in)	9.675			
Distance from End-J - U2, U3 (in)	8			
Height for Inner Surface (in)	0			
Height for Outer Surface (in)	0			
Vertical Stiffness (kip/in)	71176.7			
Rotational/Torsional Stiffness (R1,R2,R3)	R1=0, R2=Fixed, R3=Fixed			
Rotational Moment of Inertia (kip-in-sec ²)	1.0			
	Outer Top	Outer Bottom	Inner Top	Inner Bottom
Stiffness (kip/in)	3000	3000	3000	3000
Coefficient of friction-slow	0.065	0.035	0.015	0.015
Coefficient of friction-fast	0.130	0.070	0.030	0.030
Rate parameter	2.54	2.54	2.54	2.54
Radius (in)	82.5	82.5	7.5	7.5
Stop Distance (in)	18.0	18.0	2.0	2.0

The Base Displacement values were close with same curve shape.

This Report: -26.3 in (66.8 cm) (Figure 3.3)

Kitayama & Constantinou's: -26.5 in (67.31 cm) (Figure 3.2)

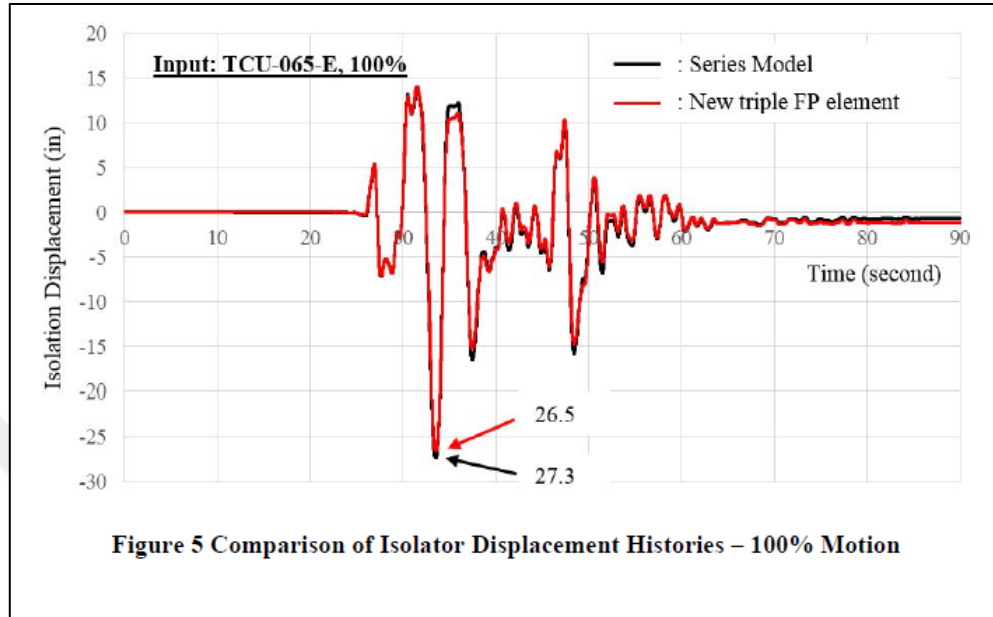


Fig. 3.2: Base Displacement (from Kitayama & Constantinou's) [14]

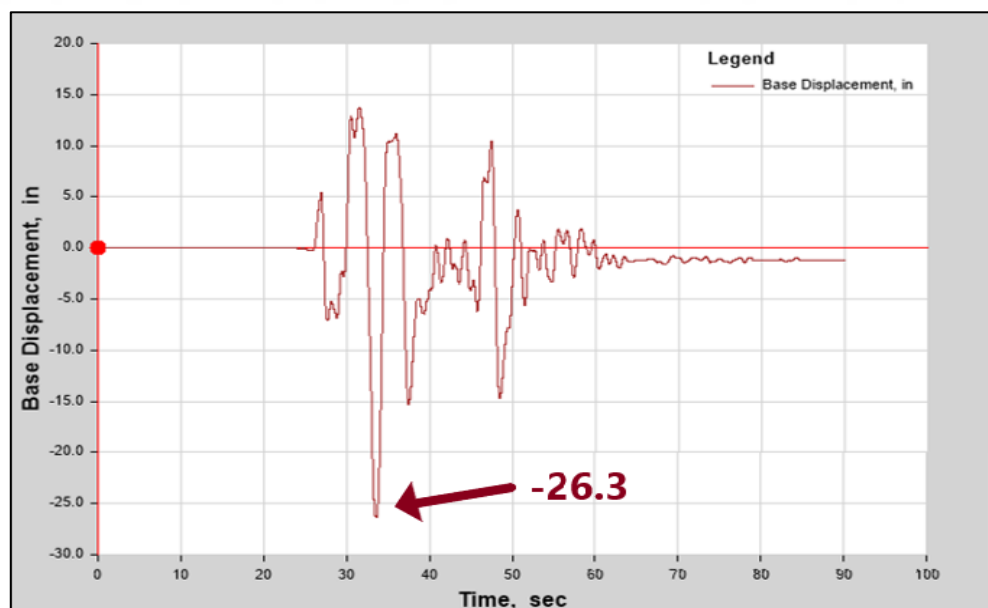


Fig. 3.3: Base Displacement (This Report)

Maximum Roof Acceleration result was also close with same curve shape.

This Report: 0.315 g (Figure 3.5)

Kitayama & Constantinou's: 0.316 g (Figure 3.4)

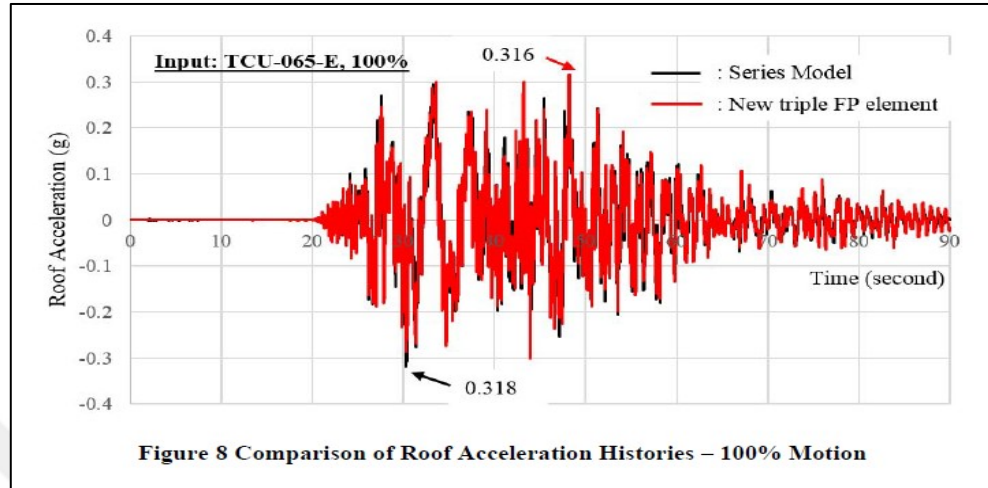


Fig. 3.4: Roof Acceleration (from Kitayama & Constantinou's) [14]

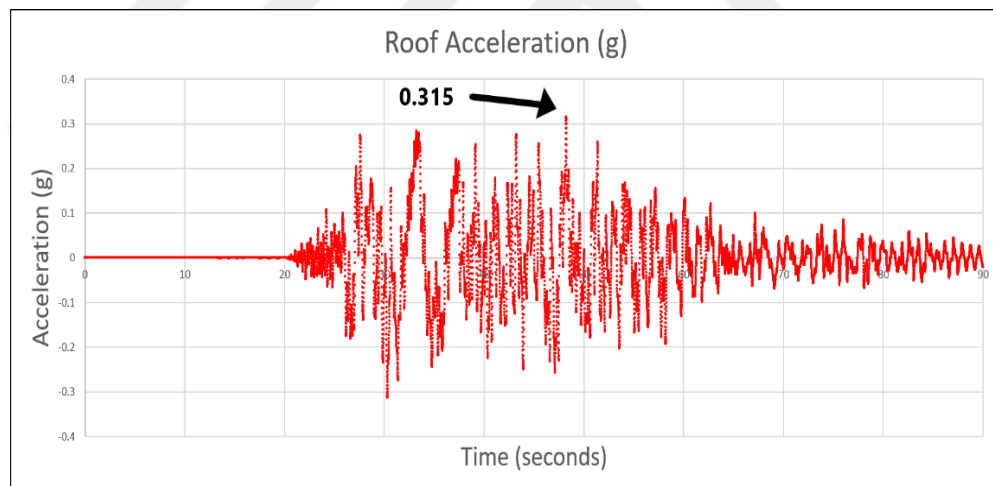


Fig. 3.5: Roof Acceleration (This Report)

The Hysteresis loop shape was similar with slightly higher Base Shear/Weight Values (check Figure 3.6 & 3.7).

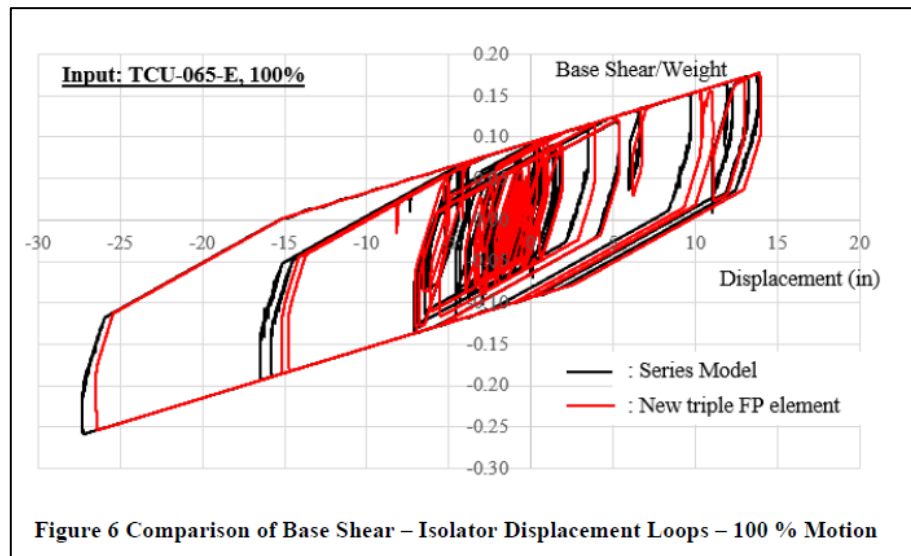


Fig. 3.6: Hysteresis Loop (from Kitayama & Constantinou's) [14]

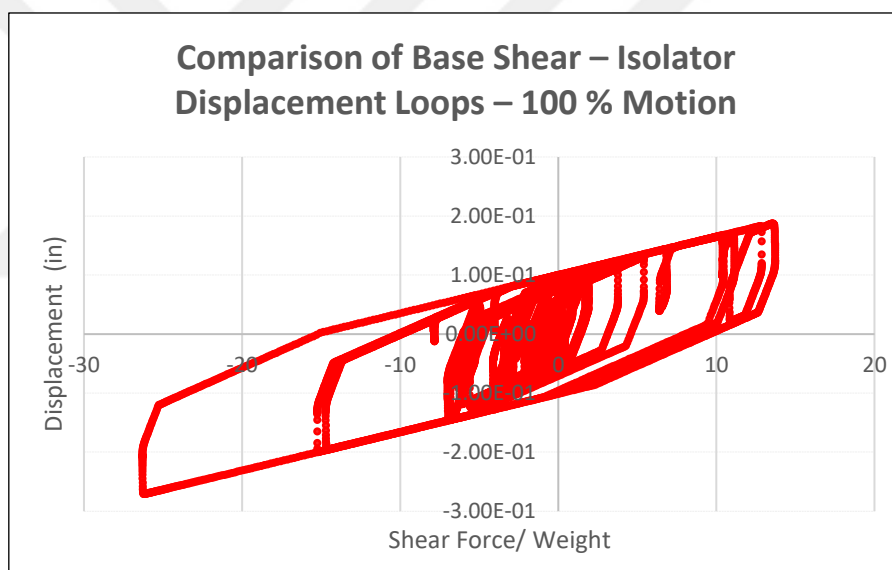


Fig. 3.7: Shear/Weight Vs Displacement, Hysteresis Loop (This Report)

4. Details of The Analyzed frame:

The parametric study was conducted on a moment resisting steel frame. A 5-storey frame with storey height and bay width lengths is equal to 3 m and 5 m, respectively (see figure 4.1). For the first three storeys column HE240B section was used with IPE360 beam section. In the 4 and 5 storeys column HE240B section was used with IPE330 beam section (check table 4.1).

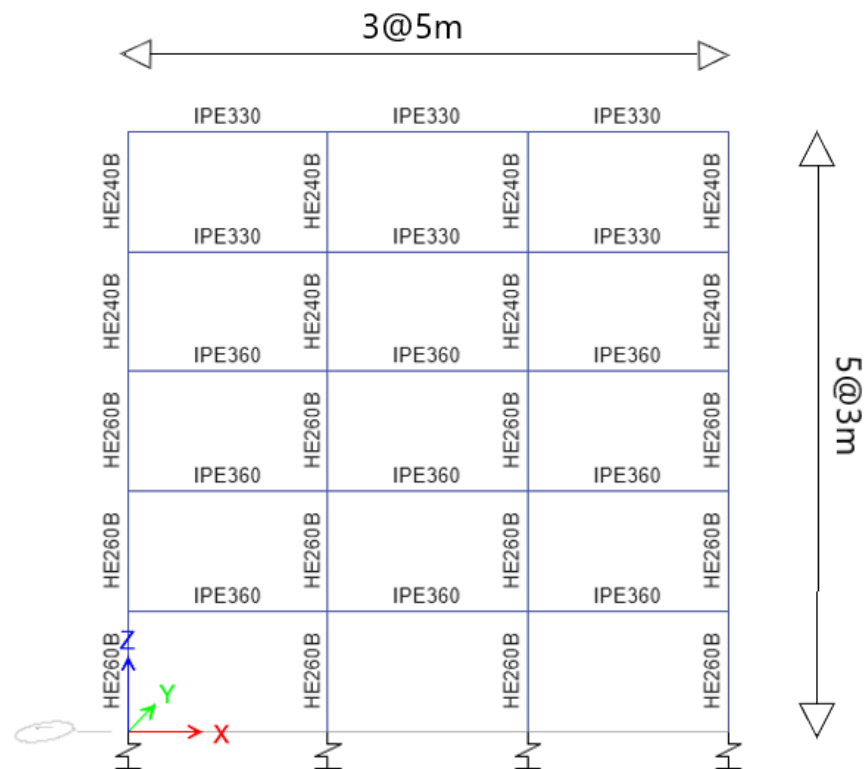


Fig. 4.1: Elevation View of Moment Resisting Frame Under Investigation

Table 4.1: Sections Details

Floor	Beam Section Used	Column Section Used
1, 2, 3	IPE360	HE260B
4, 5	IPE330	HE240B

Gravity load was assumed to be 27.5 kN/m, which was distributed in a realistic way where mid-joints carried more load. Therefore, joint loads with load equal to 137.5 kN was added at middle joints and load equal to 68.75 kN was added at corner joints as it appears in Figure 4.2. Default Material A992Fy50 was selected with minimum yield stress of 344.7 MPa. Note that in order to consider Fast non-linearity (FNA) all joints and diaphragms were set to be fully rigid.

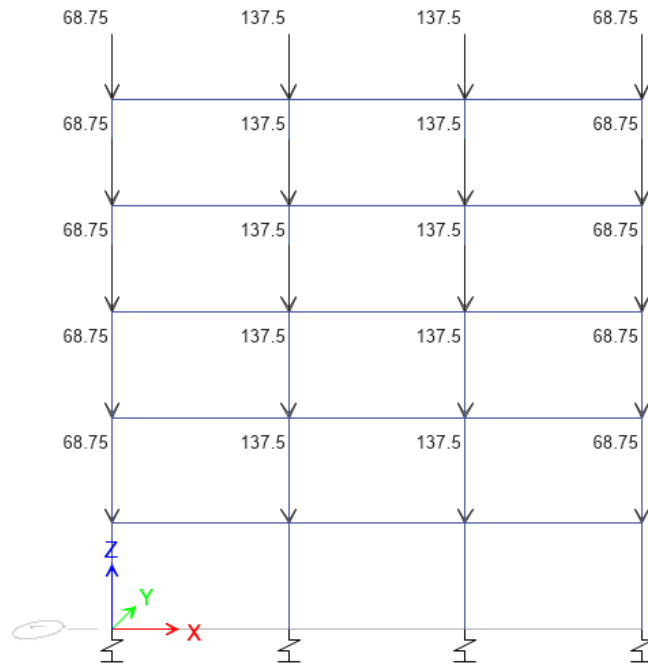


Fig. 4.2: Loads Distribution on Joints

5. Pre-Selection Design Check:

It is important choose optimum radius size for the selected frame. In order to have rational range to sustain the load coming from the chosen set of records. As well as considering not having over designed isolation systems. Therefore, pre-selection design was made beside trial and error method for different earthquake records to choose optimum radius size and earthquake records for the sake of comparison. Otherwise structure response won't be in optimum range for comparison of different friction coefficients. Among the following common radii were given by (EPS Inc.) manufacturer [15]:

Response spectrum for an arbitrary location in Istanbul Turkey was used, different frame structures of 5-Storey was tested using time history analysis and modeled in ETABS. Arbitrary location was picked for pre-selection design with following details (check Table 5.1):

Table 5.1: User Entries of the Location Chosen:

Report Address:	Bakırköy
Earthquake Ground Motion Level:	DD-2
Local Ground Class	ZB
Latitude:	40.9848°
Longitude:	28.7960°

Using the inputs from Table 5.1 into the website for Earthquake Hazard Map of Turkey [17], we managed to get the following results regarding response spectrum properties for DBE seismic event level that has a uniform 10% probability of occurring in 50 years and equals a return period of 475 years:

$$\begin{aligned} SS &= 1.216, & S1 &= 0.331, \\ SDS &= 1.094, & SD1 &= 0.265 \\ PGA &= 0.499 & PGV &= 30.465 \end{aligned} \quad \text{Where,}$$

SS: Short period map spectral acceleration coefficient [dimensionless].

S1: Map spectral acceleration coefficient [dimensionless] for 1.0 second period.

SDS: Spectral design acceleration short period coefficient [dimensionless].

SD1: Spectral acceleration design coefficient [dimensionless] for 1.0 second period.

PGA: Peak ground acceleration [g].

PGV: Peak ground velocity [cm/sec].

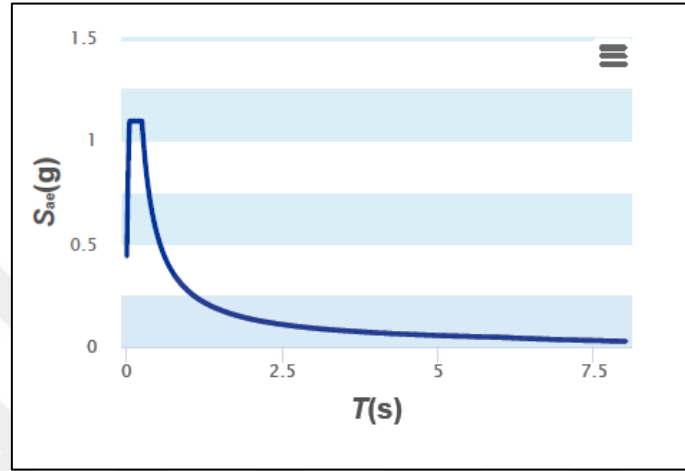


Fig. 5.1: Horizontal Elastic Design Spectrum [17]

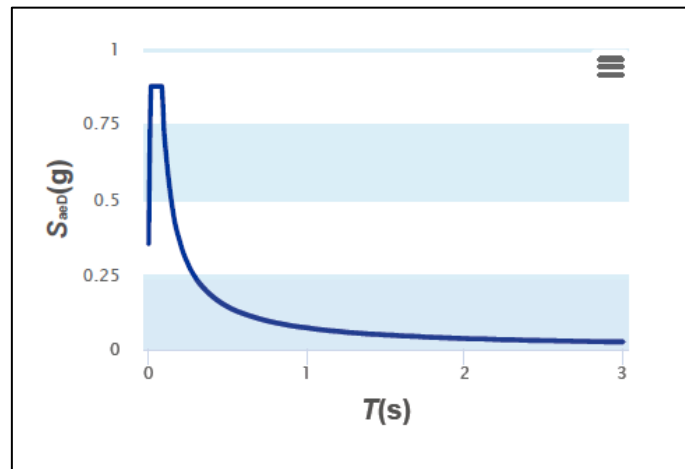


Fig. 5.2: Vertical Elastic Design Spectrum [17]

[21 39 61 88 156 238]

Radius of $R = 39$ inch ≈ 1 m has been eventually selected. In this study, Isolation systems were checked according to ASCE 7-10 standards [16] (check Appendix A).

6. Selection of Different Classes of TPB:

Reducing the acceleration and lateral force acted on the structure is the ultimate goal of design phase by using base isolation system to increase the effective period of the structure. In order to reach that goal, the isolation system should be designed to be flexible enough in the lateral direction. However, this flexibility is attained by allowing some sliding movement of pendulum on the concave plates, which have several friction coefficients. The mechanical properties of the Isolation system design can be divided into two types:

- A) Upper Bound Properties: It describes aged bearings behavior or contaminated for high speed cycle of seismic ground motion and it usually result in high-force demand or/and high temperature.
- B) Lower Bound Properties: It describes the fresh bearings behavior at normal temperature and the initial cycle of high-speed motion. Lower Bound Properties mostly result in the large displacement demand acting on the base isolators.

In this study, Upper bound Properties were taken in the calculations during the pre-selection design check.

The dynamic behavior of TPB is highly dependent on the geometric parameters and friction coefficients. These parameters include: spherical radii of four pendulums (R_1 , R_2 , R_3 , R_4), slider heights (h_1 , h_2 , h_3 , h_4). Friction coefficients for each surface (μ_1 , μ_2 , μ_3 , μ_4) vary from fast and slow (maximum and minimum). In this research, we chose different set of friction coefficients, we picked common values keeping in mind the manufactures' standard sizes and practical limitations.

It was considered while choosing parameters the isolator design period to give at average minimum 2 second period between corner isolator and mid-Isolator (see Figure 6.1). Damping ratio of these selected isolators are also presented in Figure 6.2. One set of spherical radii and slider heights were chosen based on commonly used sizes by the manufacturer (EPS Inc.) [15]. and they are $R=1$ (39 inch) for outer radii and 0.2 for inner radii:

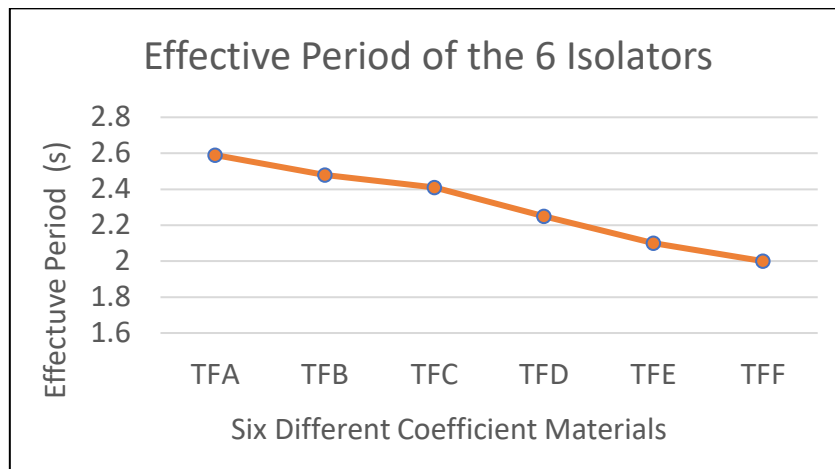


Fig. 6.1: Average Effective Period of 6 Isolator

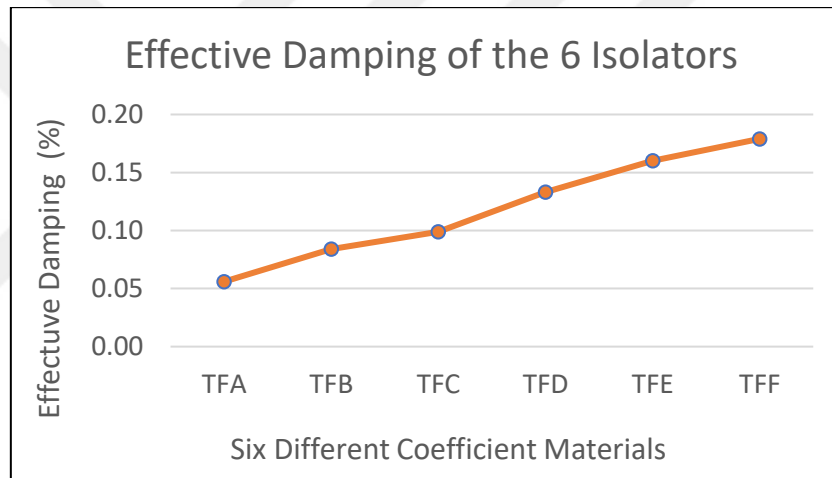


Fig. 6.2: Effective Damping of the 6 Selected Isolators

$$\text{Set R (m)} = [1 \quad 0.2 \quad 0.2 \quad 1]$$

$$h \text{ (m)} = [0.04 \quad 0.02 \quad 0.02 \quad 0.04]$$

Effective Radius will be calculated using following formula ($R_{\text{eff}} = R_i - h_i$):

$$R_{\text{eff}} \text{ (m)} = [0.96 \quad 0.18 \quad 0.18 \quad 0.96]$$

Slider heights were fixed as well as allowed displacement for simplicity stayed fixed on reasonable value of 0.6 m to notice the effect of changing our prime parameter which is the friction coefficient. Table 6.1 shows the six commonly used sets by the manufacturer (EPS Inc.) [15] were tested with following friction coefficients:

Table 6.1: Six Materials Used and its Own Friction Coefficients

Material/ Surface	Surface 1	Surface 2	Surface 3	Surface 4
Material Set A	$\mu_1 \text{ min} = 0.025$	$\mu_2 \text{ min} = 0.01$	$\mu_3 \text{ min} = 0.01$	$\mu_4 \text{ min} = 0.025$
	$\mu_1 \text{ max} = 0.04$	$\mu_2 \text{ max} = 0.02$	$\mu_3 \text{ max} = 0.02$	$\mu_4 \text{ max} = 0.04$
Material Set B	$\mu_1 \text{ min} = 0.035$	$\mu_2 \text{ min} = 0.01$	$\mu_3 \text{ min} = 0.01$	$\mu_4 \text{ min} = 0.035$
	$\mu_1 \text{ max} = 0.05$	$\mu_2 \text{ max} = 0.02$	$\mu_3 \text{ max} = 0.02$	$\mu_4 \text{ max} = 0.05$
Material Set C	$\mu_1 \text{ min} = 0.04$	$\mu_2 \text{ min} = 0.01$	$\mu_3 \text{ min} = 0.01$	$\mu_4 \text{ min} = 0.04$
	$\mu_1 \text{ max} = 0.06$	$\mu_2 \text{ max} = 0.02$	$\mu_3 \text{ max} = 0.02$	$\mu_4 \text{ max} = 0.06$
Material Set D	$\mu_1 \text{ min} = 0.05$	$\mu_2 \text{ min} = 0.01$	$\mu_3 \text{ min} = 0.01$	$\mu_4 \text{ min} = 0.05$
	$\mu_1 \text{ max} = 0.07$	$\mu_2 \text{ max} = 0.02$	$\mu_3 \text{ max} = 0.02$	$\mu_4 \text{ max} = 0.07$
Material Set E	$\mu_1 \text{ min} = 0.06$	$\mu_2 \text{ min} = 0.01$	$\mu_3 \text{ min} = 0.01$	$\mu_4 \text{ min} = 0.06$
	$\mu_1 \text{ max} = 0.08$	$\mu_2 \text{ max} = 0.02$	$\mu_3 \text{ max} = 0.02$	$\mu_4 \text{ max} = 0.08$
Material Set F	$\mu_1 \text{ min} = 0.07$	$\mu_2 \text{ min} = 0.01$	$\mu_3 \text{ min} = 0.01$	$\mu_4 \text{ min} = 0.07$
	$\mu_1 \text{ max} = 0.09$	$\mu_2 \text{ max} = 0.02$	$\mu_3 \text{ max} = 0.02$	$\mu_4 \text{ max} = 0.09$

With six different sets of friction coefficients, we will have 6 different isolators of common coefficients values will be tested in research summed up in Table 6.2:

Table 6.2: Isolators Properties Analyzed

Isolator Name	Outer Radii (m)	Material Set for Friction Coefficients
TFA	R= 1	Material Set A
TFB	R= 1	Material Set B
TFC	R= 1	Material Set C
TFD	R= 1	Material Set D
TFE	R= 1	Material Set A
TFF	R= 1	Material Set B

7. FNA Time History Analysis Modeling in ETABS:

7.1. Time History Analysis:

Time History analysis it is known as one of the important techniques for seismic analyses, Structure analyzed by Time History requires a representative earthquake time history. It is time dependent where load varies with each step to give dynamic response of the analyzed structure. Therefore, to find the seismic response of a structure under dynamic load of the representative earthquake Time History analysis is used. (Wilkinson and Hiley, 2006) [18].

Non-linear Time History Analysis could be done by two solution types, Fast Nonlinear Analysis (FNA) method and Direct Integration method:

7.1.1. FNA Method:

Nonlinear Modal Time History analysis, also called as Fast Nonlinear Analysis (FNA), is commonly more precise and efficient compared to Direct Integration Time History Analysis. The accuracy of FNA depends upon choosing adequate mode shapes. Therefore, the number of Ritz modes should be chosen carefully. similar to how direct integration needs small enough time steps to accurately describe dynamic behavior. In term of damping, it is handled in a different way between these the two analysis methods. FNA limits proportional damping when the frequency exceeds to 0.99995 that of critical. On the other hand, direct integration uses mass and stiffness proportional damping in which damping at very low and very high frequencies case might exceed critical. [19]

Fast Nonlinear Analysis (FNA) is one of the modal analysis methods. It could be useful for the static or dynamic evaluation of nonlinear studies. Fast Nonlinear analysis is compatible with time-history analysis due to its efficient formulation in computation. Through nonlinear FNA analysis, models should:

- Be primarily linear-elastic.
- Number of predefined nonlinear members shouldn't be large.
- Lump non-linear behavior within link objects.

7.1.2. FND Advantages:

- In this modal solution there is no stiffness reduction
- Has closed form solution that makes it very fast
- In this method the time step independent
- Captures high frequency response
- Ritz vector calculated once
- Multiple time histories are fast.

7.1.3. FNA Key Point:

The Ritz modes generated by the nonlinear deformation loads are used to modify the basic structural modes whenever the nonlinear elements go nonlinear. [19]

Fast Nonlinear Analysis equation efficiency is high because of the idea of keeping the nonlinear-object force vector $R_{NL}(t)$ apart from the elastic stiffness matrix and the damped equations of motion, as it appears in the exact fundamental equilibrium equation of FNA, given in the following equation [20]:

$$M\ddot{u}(t) + C\dot{u}(t) + Ku(t) + R_{NL}(t) = R(t) \quad (7)$$

Where M , C and K are Mass, proportional Damping and Stiffness matrices, respectively. The size of mentioned square matrices are equal to the total of number of unknown node point displacements N_d . The Elastic Stiffness matrix is separated from stiffness of nonlinear elements. The vectors which they are time dependent $\ddot{u}(t)$, $\dot{u}(t)$, $u(t)$ and $R(t)$ are node point acceleration, velocity, displacement and external applied load, respectively. And $R_{NL}(t)$ is the global node force vector from the sum of all forces of the nonlinear elements and is it calculated by iteration at each time point [20]. Stiffness and mass-orthogonal Load-Dependent Ritz Vectors represent the equilibrium relationships within the elastic structural system at each time rise, the uncoupled modal equations are solved exactly, while forces within the predefined nonlinear DOF which has the index of $R_{NL}(t)$ are solved by an iterative process which converges to equalize the formula. FNA is an efficient and accurate dynamic nonlinear

approach that satisfies equilibrium, compatibility relationships, and force-deformation.

7.1.4. Direct Integration Method:

The Degree of Freedom (DOF) in this method is taken as a variable parameter. Equilibrium Equation of the total DOF could be integrated progressively with relation of time in order to get the results. Outputs are measured for each time step without any sort of change occurs to equilibrium eq. and several integration methods could be taken. The analysis time in Direct Integration Method is more compared to Modal Method, one of expected reason is because this method conducts analysis for the time steps and Number of time steps is proportional to the analysis time. Therefore, Number of time steps should be selected appropriately.

7.1.5. Time Step:

In the Time History analysis, the time step is not same for the direct Integration method and modal method. In the direct Integration method, the defined time step to conduct time integration. However, the accuracy is highly depending on the size of time step. In General, adequate and accurate results could be calculated when using a time step which is less than 10% of the minimum period selected. Taking large time steps creates errors in the time integration and taking too small-time steps creates more computation cost.

The Modal method conducts the time integration in an analytical way. Hence, the accuracy of the results is not affected by the time step. The time step in Modal method is used to set the time to see the results period range of the time history.

7.1.6. Comparison between the Modal method and Direct Integration method:

In the analysis time the Modal takes less time than the Direct Integration method. Hence, if the model is big and requires many time steps, it is more effective to use the Modal method (see Table 7.1). If there are many active frequencies calculated it is suggested to use the direct integration method to get more precise response. [21]

Table 7.1: Comparison between the Modal FNA and Direct Integration

ELEMENT	DIRECT INTEGRATION	MODAL (FNA)
COMPUTATION TIME	Long analysis time	Relatively Short analysis time
DETAILS	Time step selecting is important	Number of modes selecting is important
SUGGESTED CONVERGENCE TOLERANCE	1×10^{-4}	1×10^{-3}
ANALYSIS ACCURACY	Long analysis time, but highly accurate results	Generally, accurate and efficient

7.2. Defining Load Case of Time History Analysis:

Time history analysis method could be used for linear and non-linear studies. Load could be applied using Modal or Direct integration method as it was explained in Section 7.1. In this research Modal FNA was used. In order to define Time History analysis properly in ETABS two main things should be specified correctly Time History Function (See Section 7.6) and Load case.

During load case specifying if nonlinear option is selected, initial conditions of nonlinear modal will appear, it has two different initial conditions to select from. First option, zero initial conditions this basically means the structure will start with zero displacement and all elements are not subjected to any stress. On the other hand, the second option continues from the end of an existing nonlinear case this essentially says that the displacements and nonlinear deformation from the end of a previous load case is taken before applying the time history acceleration to the structure.

In this study, dead load case was started with zero initial condition but the FNA load case that has the acceleration of earthquake record was started after the dead load case ends using the second option which allow us this. Next, Acceleration should be defined with right direction of the study. In the function option, the time history function which was defined in section 7.6 was selected.

ET Load Case Data

General

Load Case Name: FNA [Design...]

Load Case Type/Subtype: Time History Nonlinear Modal (FNA) [Notes...]

Exclude Objects in this Group: Not Applicable

Mass Source: Previous (MsSrc1)

Initial Conditions

☐ Zero Initial Conditions - Start from Unstressed State

☒ Continue from State at End of Nonlinear Case (Loads at End of Case ARE Included)

Nonlinear Case: Dead

Loads Applied

Load Type	Load Name	Function	Scale Factor
Acceleration	U1	RSN160_IMPVAL...	9.81

[Add] [Delete] [Advanced]

Other Parameters

Modal Load Case: Modal_Ritz

Number of Output Time Steps: 8000

Output Time Step Size: 0.005 sec

Modal Damping: Constant at 0.02 [Modify/Show...]

Nonlinear Parameters: Default [Modify/Show...]

[OK] [Cancel]

Fig. 7.1: Example of Load Case Defining of Non-linear Time History Analysis

In the scale factor for acceleration load case, as the acceleration used was normalized record and it was metric units, factor was multiplied by gravity acceleration. In this case, 9.81 m/s^2 . The output time step which the program calculates and shows in the end of analysis suggested to be defined same is input steps defined in Time History function. Smaller output time step could be taken, very big output time step will reduce the calculation accuracy. However, output time step size multiplied by number output time steps should give at least same time duration in the ground motion record defined in Time History function.

7.3. General Background About Modeling TFP and Its limitations:

Before adding the new feature of TFP as one link element into SAP2000 and ETABS, Fenz and Constantinou (2008d, e) [22,23] developed two models to represent TFP bearing in the SAP2000, the two models consisted of three single FP bearings

connected in parallel for first case and in series for the second one. The parallel model is a simple way to represent the behavior for special configuration of TFP bearing where $R_{eff1}=R_{eff4}$, $R_{eff2}=R_{eff3}$, $d_1^*=d_4^*$, $d_2^*=d_3^*$, $\mu_1=\mu_4$, $\mu_2=\mu_3$ (for more details check Sarlis et al (2009)). In addition, it has many limitations such as this type of modeling cannot show the behavior of the isolator at regime V and the limit of displacement where parallel model cannot capture the stiffening behavior of the TFP bearing and based on that should not be used when the calculated response displacement resulted from history analysis exceeds the limit of u_{dr4} (See Table 1.1). However, the program follows now the same concept of connecting three single FP bearings in series for the new feature. Moreover, the series model uses large number of degrees of freedom per isolator with artificial masses placed at the connecting joints which has several effects as it stated in the following [14]:

- 1) The computational effort will increase.
- 2) Choosing secondary parameters become more sensitive such as link element, elastic stiffness and effective stiffness.
- 3) Increase in required No. of Ritz Vector Modes. During the Fast-Non-Linear Analysis.

7.3.1. Damping Specification in Fast Nonlinear Analysis (FNA)

Structural Damping determining in Non-linear analysis may appear to be secondary importance. However, as we stated previously when we are modeling the TFP bearing using the new feature or the series model this parameter becomes sensitive and has big effect.

The Stiffness matrix stays the same all during the Fast-Nonlinear Analysis (FNA) based on the effective stiffness given to the isolator. Non-linear forces coming from the isolators are treated as loads in the right-hand of equation motion, this controls the damping leakage into the isolators. In order to do that, small effective stiffness of isolators is specified. And the following Ritz vectors modes could be discriminated [14]:

- 1- Purely Isolated: In this type the structure behaves as one rigid part, when diaphragm constrain is applied for all storey-levels and loads added as joint masses, modes number for two-dimensional study is

reduced to one, for three-dimensional study, it is reduced to three (see Figure 7.2).

- 2- Mixed Modes: These modes are categorized by the mixed response of the structure and the isolator. Again, when diaphragm constrain is applied for all storey-levels and loads added as joint masses, modes number for two-dimensional study is reduced to be equal number of floors, for three-dimensional study, it is reduced to be three times of number of floors (see Figure 7.3).

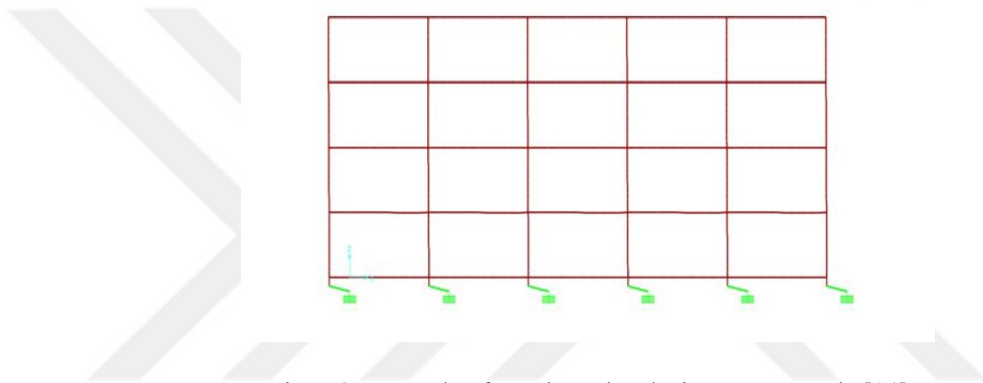


Fig. 7.2: Example of Purely Isolated Ritz Vector Mode [14]

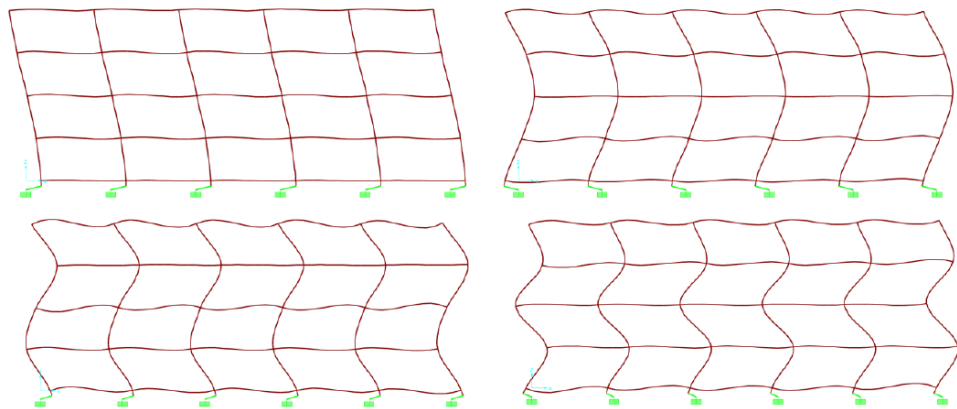


Fig. 7.3: Examples of Mixed Ritz Vector Modes [14]

- 3- Local or vertical Modes: These modes usually come with high frequency modes. They create negatively affect after the processing is done and computation difficulty. Nevertheless, they play significant role in calculation parameters such as internal forces of the member, accelerations and isolator uplift (see Figure 7.4).

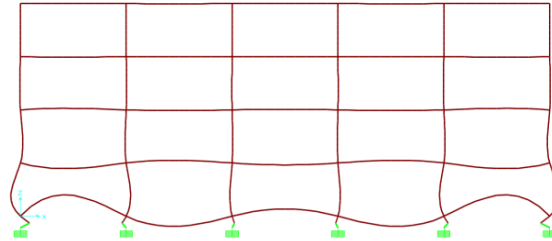


Fig. 7.4: Example of Local Ritz Vector Mode [14]

Damping could be categorized in ETABS (similarly SAP2000) as following:

- Constant Damping for All Modes:

Reducing damping leakage could be done by this simple method where constant damping is taken for all modes with considering small effective stiffness. This method keeps damping ratio the same for each mode. Figures 7.5 illustrates ETABS window for specifying Constant Damping for all modes. Same procedure was made in this study. The suggested Constant modal damping for non-isolated structures is 5% or less and for Isolated Structures 2% or less is suggested. In this study, 2% Constant Damping was taken.

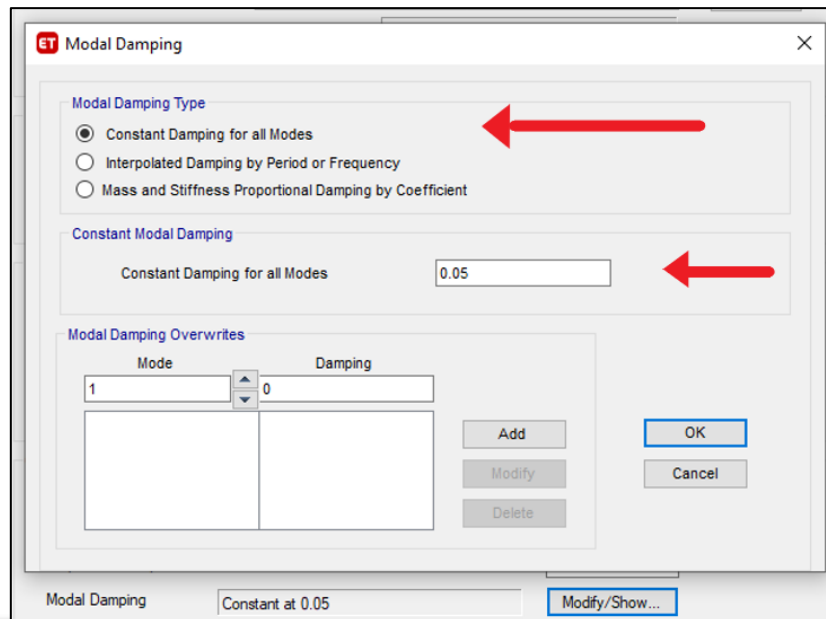


Fig. 7.5: illustration of How to Assign Constant Damping inside Model Damping window

- Constant Damping with Override

This method has faster computational speed. The concept of this method is same as previous one where constant damping is taken except for specific modes like purely isolated modes specified with zero damping override. The practical way to use this method when small number of isolators existed. This method could be used as well for higher modes to give it override with different damping ratio. Figures 7.6 illustrates ETABS window for specifying Constant Damping with override.

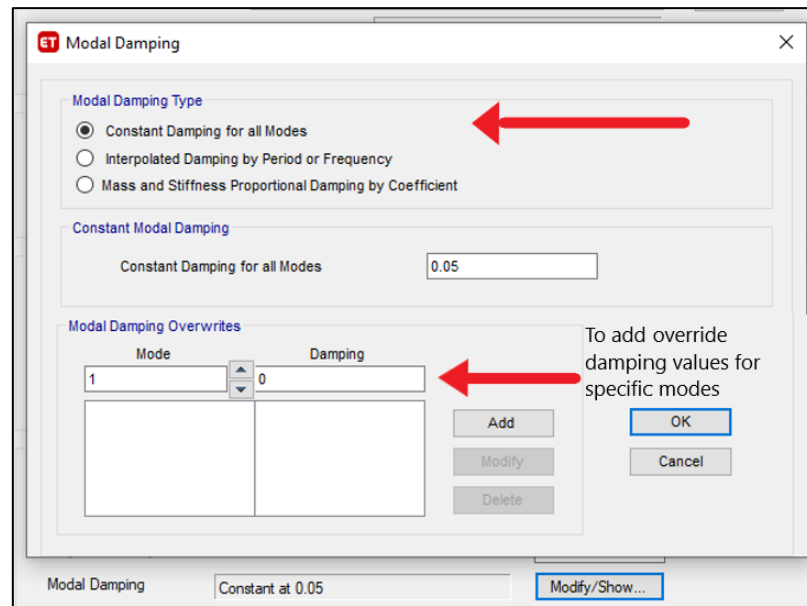


Fig. 7.6: illustration of How to Assign Constant Damping with override inside Model Damping window

- Rayleigh Damping:

Damping ratio values in this method is specified based on two frequencies points. An Easy way to select these two frequencies is when they bound the frequencies of mixed modes and the purely isolated (see Figure 7.7) [14].

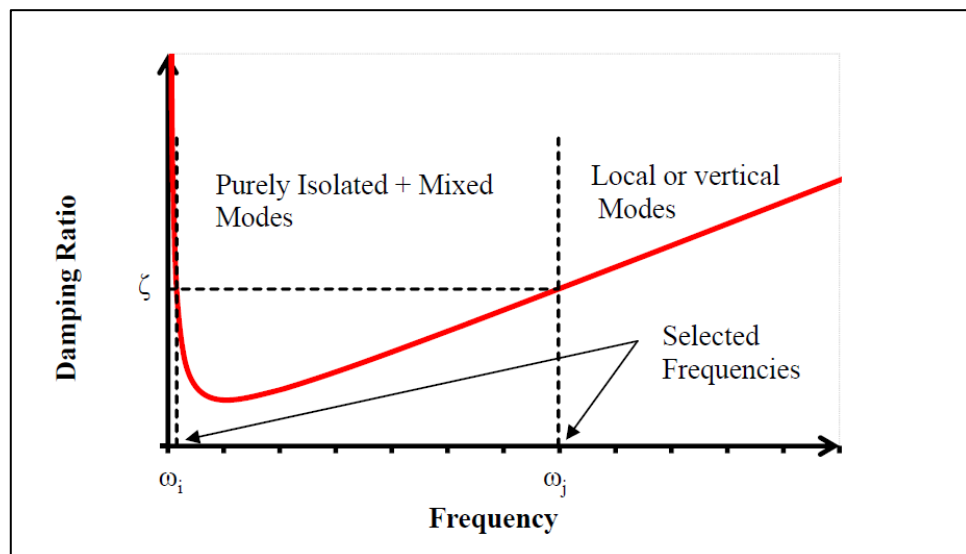


Fig. 7.7: Frequencies Selection for Rayleigh Damping Assignment [14]

- Rayleigh Damping with Override:

This method is one of the best effective way to determine the Damping for large number of isolators and analysis speed does matter. Rayleigh Damping with Override is similar with Rayleigh Damping with more control in hand. However, this method demand defining the purely isolated modes, local modes and mixed modes. When low values of effective stiffness are taken for the isolator element, this could be relatively simple.

using the lowest and highest frequency of the mixed modes the Rayleigh Coefficients are calculated. Frequency modes with lower than the lowest mixed mode (assumed purely isolated modes) should be taken as zero damping (or low damping) by using the damping override feature in ETABS or SAP2000. Damping ratio based on this approach is showed in Figure 7.8.

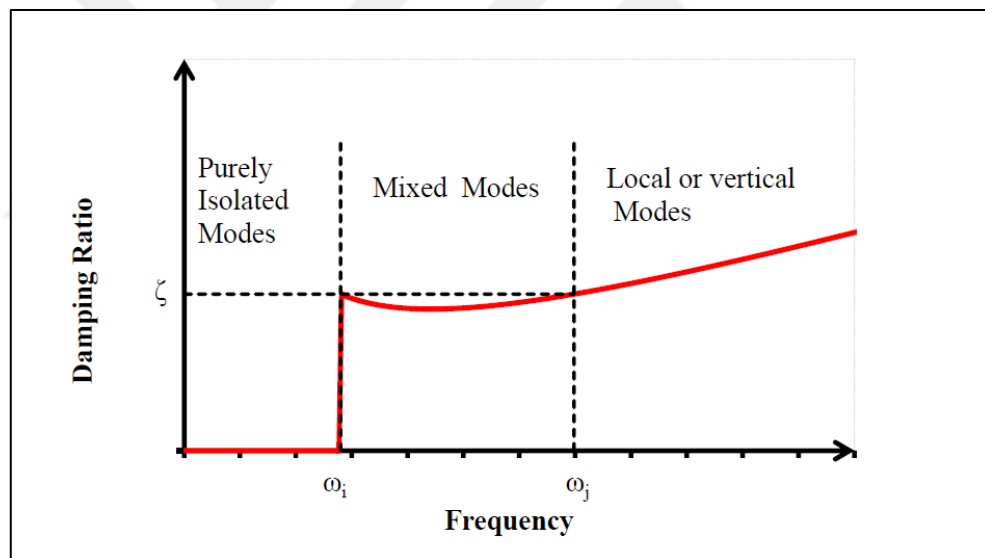


Fig. 7.8: Rayleigh Damping with override over Frequencies Selected [14]

- Interpolated Damping:

In this Approach, damping ratio values varies depends on frequency range. for the frequency range of the purely isolated modes user could assign zero damping or very small value. for the mixed modes certain damping ratio could be assigned. Finally. for local modes, high constant damping ratio suggested or a linearly changing damping ratio may be assigned up to a maximum value of damping ratio (see Figure 7.9). This

damping method is efficient in terms of accuracy and minimizing damping leakage of superstructure results, it obtains results similar to the constant damping with override and Rayleigh damping with override but with higher analysis computation speed [14]. Figures 7.10 illustrates ETABS Model damping window for specifying Interpolated Damping.

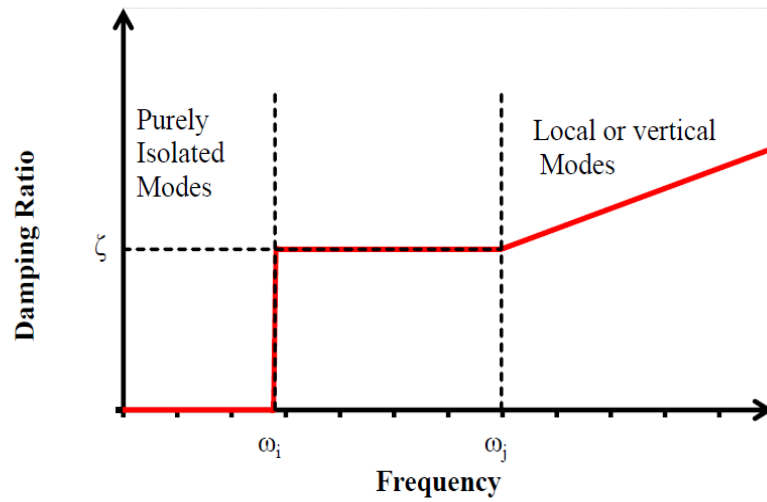


Fig. 7.9: Damping Ratio using Interpolated Damping [14]

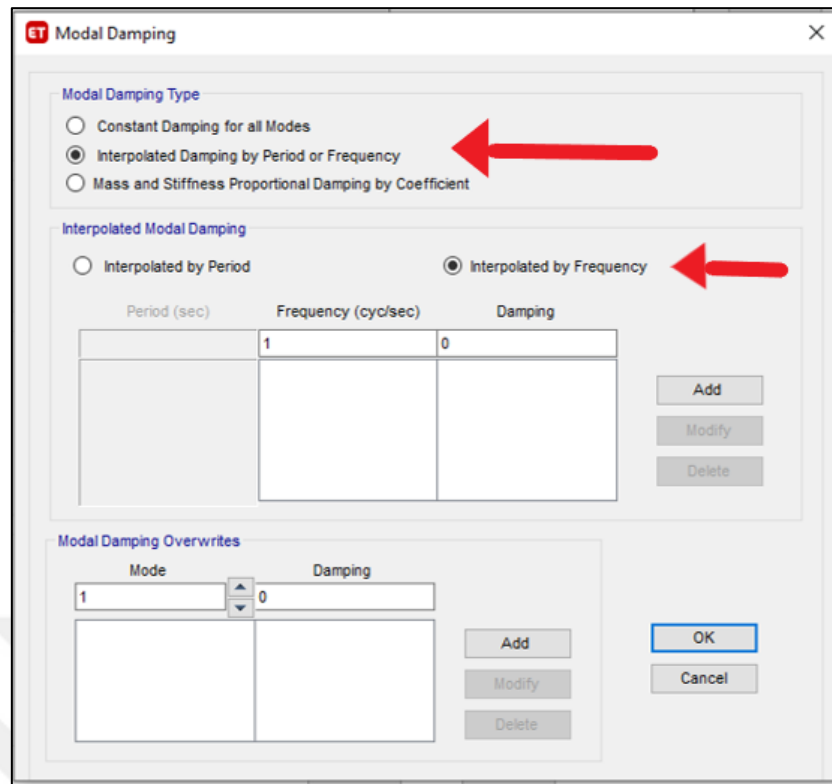


Fig. 7.10: illustration of How to Assign Interpolated Damping inside Model Damping window

7.3.2. Ritz Vector Modes

The number of Ritz Vector modes in non-linear analysis should be kept in mind whereas choosing a number less than certain limit may lead to wrong representing of dead load applied on the isolators (significant with sliding isolators). Based on this, member forces and gravity load will be under-estimated and will affect the results of studied model.

Example from (Constantinou et al, 2007) was done on bridge showed total axial forces on the supports verse number of Ritz as function of time. Weight of the bride is almost 6000 kip and needed minimum 33 Ritz vector modes to sufficiently represent the load (Figure 7.11). More than that number won't affect the results significantly but will slow down the computation process. For more details check [14] Constantinou.

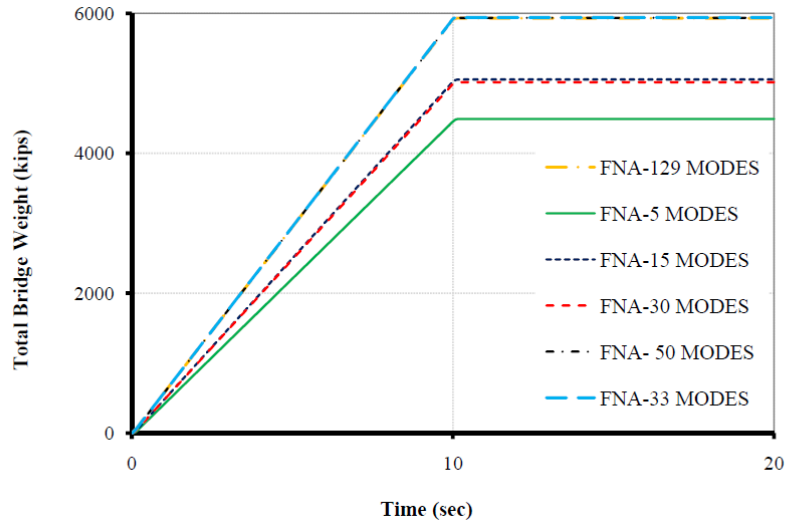


Fig. 7.11: FNA Modes Vs Bridge Weight (Constantinou et al, 2007) [14]

A simple suggested formula by Computers and Structures, Inc. (CSI) [24]. to choose Ritz Vector given by:

{No. of Frames + No. of Dampers + No. of Isolators + Dead load}, total multiplied by x2 or x3 times just to be in safe zone. In this Study, the required ritz vectors needed of 200 modes were taken.

7.4. Defining TFP Bearing as Link Element in ETABS:

ETABS is a structural software program developed by Computers & Structures, Inc. [24] for buildings analysis and design. In this part, illustration is made on TFP bearing properties and inputs inside this program (See Figure 7.12).

Fig. 7.12: User Interface of Triple Pendulum Isolator Properties Determination

TFP bearing was assigned under each column. First, link element was defined as Triple friction pendulum isolators with special selected properties for this study. The TFP Isolator has several degrees of freedom (DOF) stated as following:

- a) U1 window is designated for axial degree of freedom This DOF is set to be non-linear as default and vertical stiffness should be determined. It could be estimated as it we are vertical stiffness of cylinder assumed to have same bearing height with inner radius with modulus of elastic equals half of steel modulus of elasticity. However, if vertical stiffness is not available realistic high value is suggested.
- b) U2 and U3 are designated for shear degree of freedom in two orthogonal dimensions. Shear DOF in the two orthogonal directions, effective stiffnesses for U2 and U3 should be determined. In the nonlinear analysis, more properties should be determined such as isolators inner and outer radii, heights, friction coefficients, rate parameters and stop distance.
- c) R1 is designated for Torsional Degree of freedom, for TFP isolator R1 set to be zero or a very small value in our case.

- d) R2 and R3 are designated for Rotational stiffness. In this study, they were assumed to be fixed.

The four concave sliding surfaces of TFP isolator are named as following:

- Outer Top
- Outer Bottom
- Inner Top
- Inner Bottom

The four sliding surfaces each of them has the following properties that need to be specified (See Figure 7.13):

1. Stiffness: is the shear stiffness at stop just before sliding begins.
2. Friction Coefficient, Slow: Friction Coefficient when the velocity at zero.
3. Friction Coefficient, Fast: Friction Coefficient when the velocity at fast rate.
4. Rate Parameter: inverse of the characteristic velocity for friction.
5. Radius of the surface: Actual radius of the concave surface.
6. Stop Distance: transversal displacement allowed before reaching a stiff stop

Link/Support Directional Properties

Identification

Property Name: TFP Corner Type: Triple Pendulum Isolator
Direction: U2; U3 NonLinear: Yes

Linear Properties

Effective Stiffness - U2: 165 kN/m Effective Stiffness - U3: 0 kN/m
Effective Damping - U2: 0.06 kN-s/m Effective Damping - U3: 0 kN-s/m

Shear Deformation Location

Distance from End-J - U2: 0 m Distance from End-J - U3: 0 m

Height and Symmetry of Sliding Surfaces

Height for Outer Surfaces: 0.04 m ☒ Outer Bottom Surface is Symmetric to Outer Top Surface
Height for Inner Surfaces: 0.02 m ☒ Inner Bottom Surface is Symmetric to Inner Top Surface

Nonlinear Properties for Directions U2 and U3

	Outer Top	Outer Bottom	Inner Top	Inner Bottom	
Stiffness	3452	3452	1381	1381	kN/m
Friction Coefficient, Slow	0.025	0.025	0.01	0.01	
Friction Coefficient, Fast	0.04	0.04	0.02	0.02	
Rate Parameter	0.1	0.1	0.1	0.1	sec/mm
Radius of Sliding Surface	1	0.2	0.2	0.2	m
Stop Distance	603	603	3	3	mm

OK Cancel

Fig. 7.13: Example of U2, U3 Orthogonal DOF User Interface Window

7.5. Ramp Function:

Ramp Function is a Time History Function that allow us to ramp up the load with function of time. Figure 7.14 illustrates example of the window inputs RAMP Function. For TFP bearing analyzed in Fast Non-linear Analysis (FNA) method, dead load should be applied slowly using ramp function. The base Isolation used are Friction Pendulums type which their performance consequently controlled by friction forces that means the isolators should be loaded with vertical load before the earthquake analysis initiates. The ramp function will gradually add the load. Later, high damping is specified in Dead Load case to simulate the static load [24].

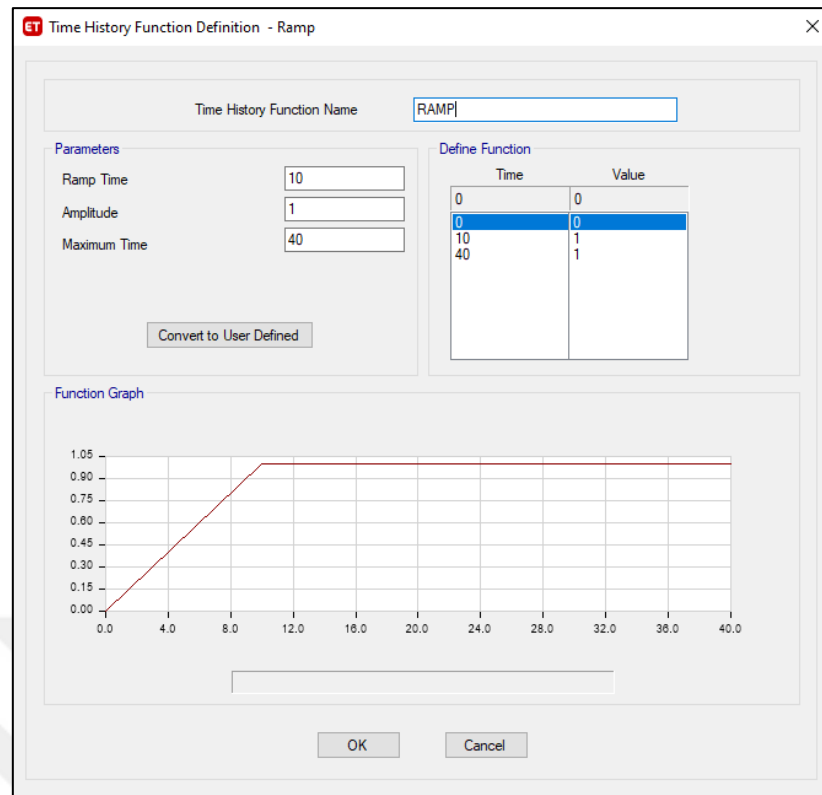


Fig. 7.14: Definition Window for Ramp Function

7.6. Defining Earthquake Records in ETABS for Time History Analysis:

Two Different Groups of seismic records were chosen Near Fault and Far Field (Tables 7.2 & 7.3), Near Fault were no pulse records due to the fact a big section will be needed if we used them and as we are testing fixed isolators properties, isolators would be oversized under other records and the comparison between them will be inefficient. The Near Fault group are Imperial Valley, Loma Prieta, Cape Mendocino, Gazli and Northridge. Far Field Group are Landers, Kobe, Kocaeli, Duzce and Manjil. Original earthquake records scaled to gravity acceleration will be considered as Design Base Earthquake (DBE). Analyzed models were conducted again with DBE records times 1.5 and were considered as Maximum Credible Earthquake (MCE). Finally, same models were reanalyzed with half of DBL and were considered Service Level Earthquake SLE according to the Turkish code. Adding more variation gave better overview to compare to six different friction coefficients. These ground motion records were uploaded from the Pacific Earthquake Engineering Research Center (PEER) [26] check Tables 7.2 & 7.3.

Table 7.2: Characteristics Near Fault Ground Motion Records

Near Fault Records						
Earthquake Record	RSN	Year	Component	Lowest Freq. (Hz.)	PGA	PGV
Imperial Valley-06	160	1979	233	0.13	0.76	44.3
Loma Prieta	741	1989	000	0.13	0.64	55.9
Cape Mendocino	825	1992	000	0.07	1.43	119.5
Gazli	126	1976	000	0.06	0.71	71.2
Northridge-01	1004	1994	270	0.12	0.73	70.1

Table 7.3: Characteristics Far Field Ground Motion Records

Far Field Records						
Earthquake Record	RSN	Year	Component	Lowest Freq. (Hz.)	PGA	PGV
Landers	848	1992	CLW-LN	0.13	0.42	42
Kobe, Japan	1111	1995	000	0.13	0.51	37
Kocaeli, Turkey	1158	1999	180	0.24	0.36	59
Duzce, Turkey	1602	1999	000	0.06	0.82	62
Manjil, Iran	1633	1990	-L	0.13	0.51	54

In order to define the earthquake records for the Time History analysis in ETABS, A new Time History function need to be defined representing each record. Example of the window inputs for defining the motion record illustrated in Figure 7.15. Note that first file location should be selected. Then, number of lines the program should skipped to tell the program from which line it should start, usually first five lines are general information of the record. Prefix Characters (if available) before the data records numbers should clarified as well. Third line is to state how many data or number in each line inside the ground motion record file. Finally, in our case the file is made with values at equals intervals, this interval step or dot time DT varies from each station

and could be typically taken from the ground motion record file in first five lines that has general information of the record.

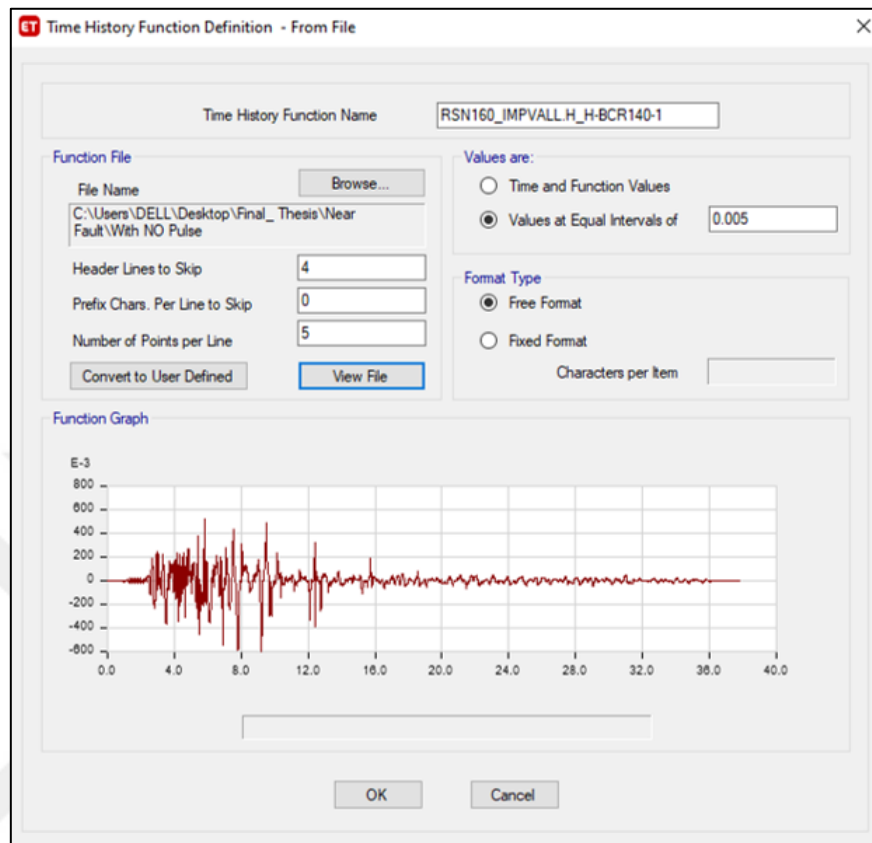


Fig. 7.15: Example of the window inputs of defining Time History Function for Earthquake Record

8. Discussion of The Results:

This study investigated the six different TFPB systems with different frictions coefficient. the performance of these Isolators was evaluated in highlighted in terms of several engineering demand parameters as Isolator Displacement, Inter-Storey Drift Ratio, Storey Drift, Maximum Absolute Acceleration, Maximum Base Shear and Hysteretic Loop.

8.1. Isolator Displacement:

Non-linear Time History analysis's done on the six different Isolator (TFA, TGB, TFC, TFD, TFE, TFF). In General, the variations were not remarkably big. However, different response were noticed from different seismic records. Having big displacement doesn't mean the isolator system not working efficiently just the demand to make later displacement is highest as long as Isolator Displacement is lower than the allowed displacement which is 60 cm in our study.

For DBE records, Maximum Isolator Displacements under Loma Prieta and Northridge records were illustrated in Figure 8.1, the super-structure model showed similar responses with small differences among the six isolators systems with average of 10.8 and 16.7 cm, respectively. TFC showed slightly more displacement of 12.1 cm under Loma Prieta. On the other side, under Cape Mendocino, a jump was noticed from TFA to TFB to reach a maximum displacement of 26.3 cm then a descended from TFB To TFF in term Maximum Isolator Displacement. Under Gazli earthquake, Maximum Isolator Displacement demand was the Highest in TFA and TFB isolators that has small friction coefficient differences between inner and outer surfaces. Then, Maximum Isolator Displacements increased from TFC to TFF with small slope. The near fault record left which is Imperial Valley and the five Far Field records Duzce, Kobe, Kocaeli, Landres and Manjil (Figure 8.2). They showed increasing pattern of Maximum Isolator Displacement from TFA to TFF. However, for Landres record this

increase was slight. Note that Landres record is the weakest in terms of Frequency, PGA, PGV and earthquake duration compared to other chosen records in this study.

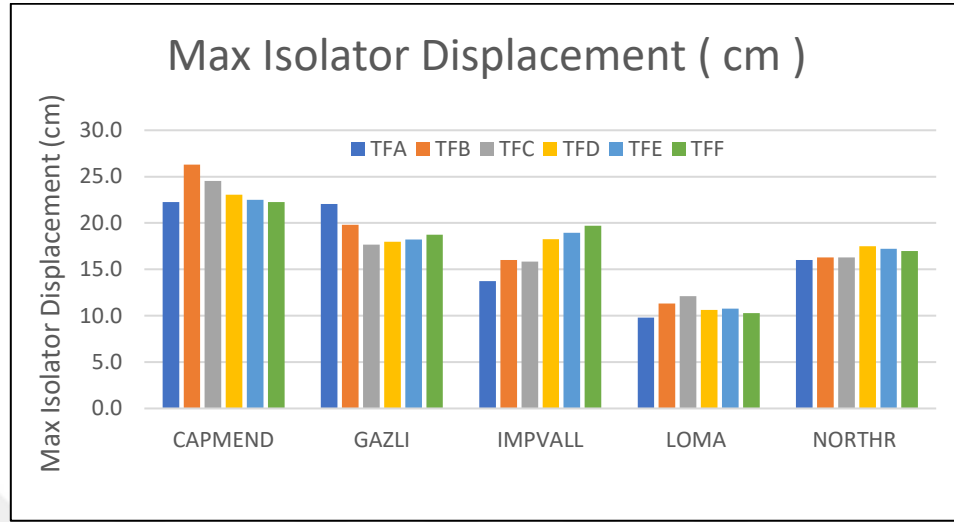


Fig. 8.1: Maximum Isolator Displacement of DBE Scaled Near Fault Records

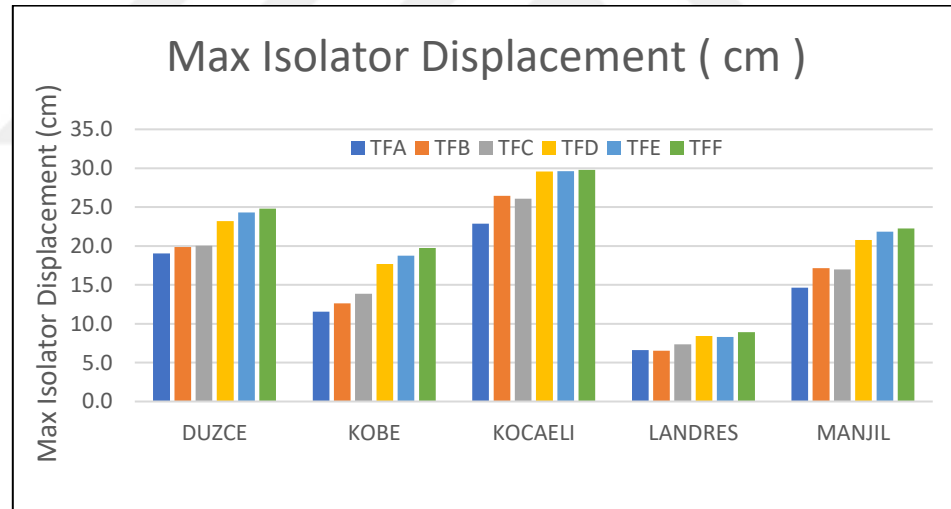


Fig. 8.2: Maximum Isolator Displacement of DBE Scaled Far Field Records

For MCE Scaled Records, starting with near fault earthquakes (Figure 8.3), under Imperial Valley, Loma Prieta and Northridge, the six Isolators types displaced similarly with average of 27 ,16.7 and 25.8 cm, respectively. However, type TFD was slightly more in both cases. Under Cape Mendocino and Gazli earthquakes, Isolators with upper surfaces of low friction coefficient values (TFA & TFB) displaced more than these with upper surfaces of high friction coefficient values isolators (TFE &

TFF). Far Field earthquake records (Figure 8.4), in general Maximum Isolator Displacements increased from TFA to TFF with small slope where differences among the six different isolators were not too big, except for Manjil record the slope was higher a little compared to other records.

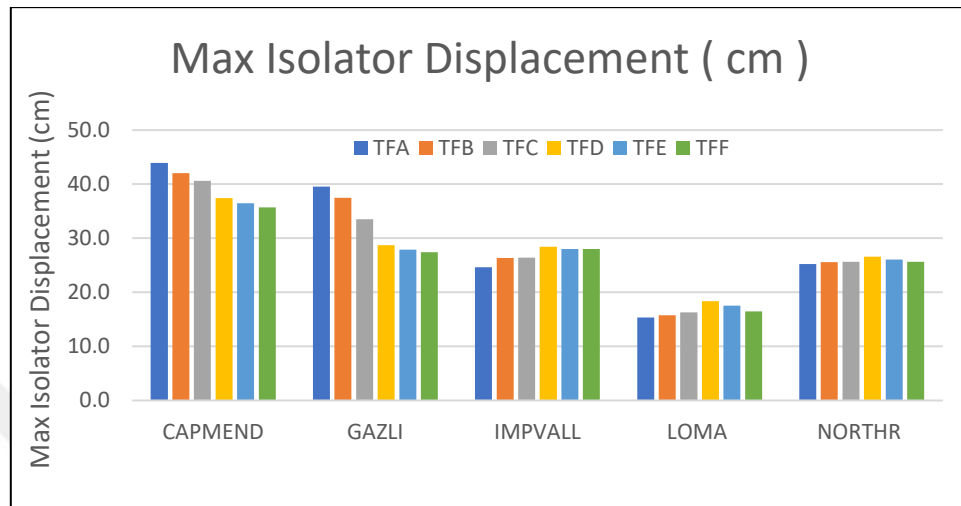


Fig. 8.3: Maximum Isolator Displacement of MCE Scaled Near Fault Records

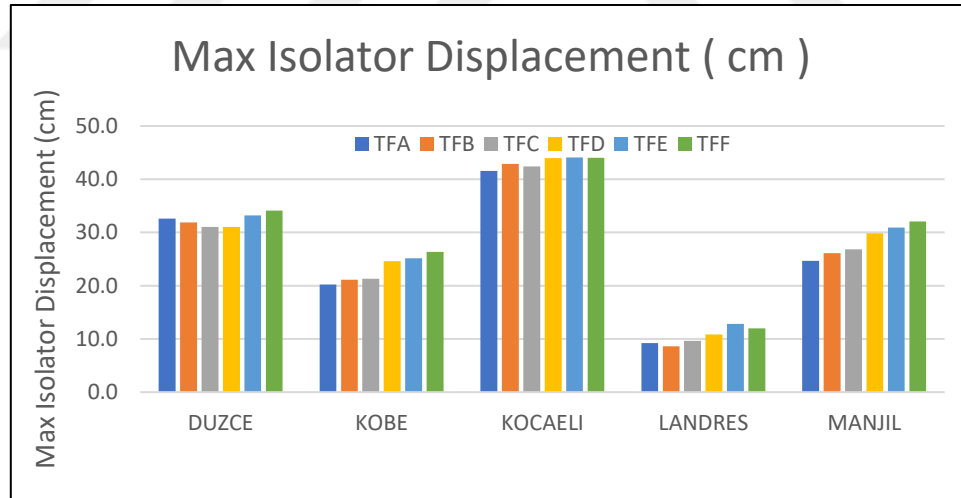


Fig. 8.4: Maximum Isolator Displacement of MCE Scaled Far Field Records

For SLE scaled Earthquake records, Cape Mendocino and Gazli showed high Maximum Isolator Displacements, using first two types (TFA, TFB), then it slightly increased from TFC to TFF (Figure 8.5). Loma Prieta Showed similar behavior where using TFA type made higher maximum Isolator Displacement compared to TFB. After that drop it steady increased noticed from TFB to TFF type. Under Northridge

earthquake, isolators showed same range of Maximum Isolator Displacements with average of 16.7 cm. Under Imperial Valley and the five other Far Field records (Figure 8.6), the It was observed an upward pattern from TFA to TFF type with a big step difference between TFC and TFD except for Landres record the increasing behavior was there but difference between TFC and TFD wasn't that big.

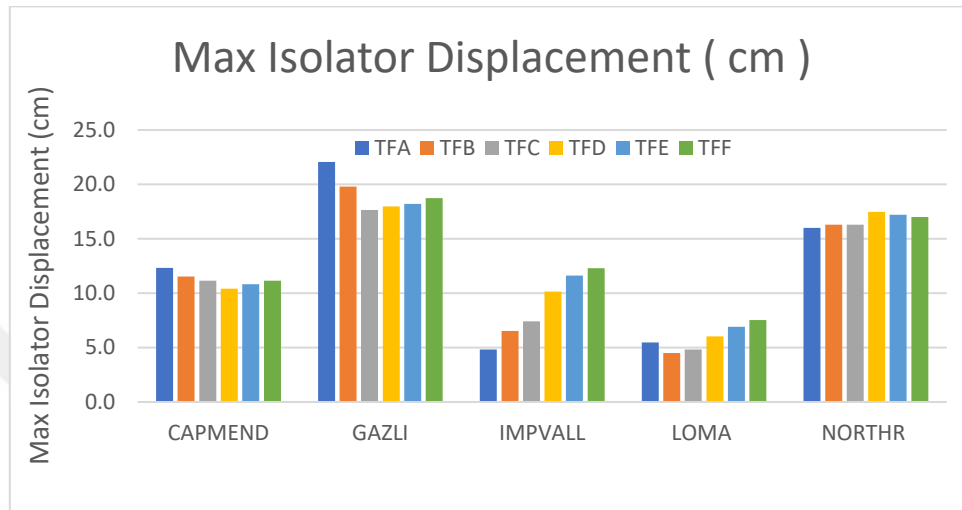


Fig. 8.5: Maximum Isolator Displacement of SLE Scaled Near Fault Records

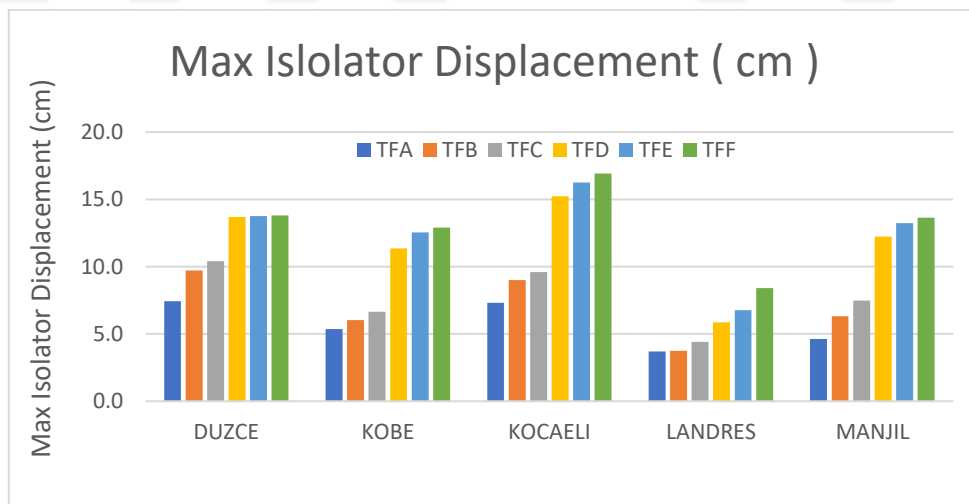


Fig. 8.6: Maximum Isolator Displacement of SLE Scaled Far Field Records

8.2. Inter-Storey Drift Ratio:

Inter-Storey Drift Ratio is one of the key parameters to observe the effect of earthquake damages on the structure. It is the relative displacements difference between two consecutive storeys normalized by dividing it by storey height. To make it easier to for

comparison highlighting it was showed in the following only five records out of ten scaled to three different levels DBE, MCE and SLE, but the ten records were considered while making finally conclusions. To see the other five records results, check appendix B.

Firstly, for near fault records DBE scaled, In Cape Mendocino earthquake (Figure 8.7), inter-storey drift ration range was between 0.26% to 0.63%. TFA had the lowest inter-storey drift ratio while TFF had the highest value. Trend was upward climbing from TFA to TFF. Under Gazli Earthquake (Figure 8.8), the Maximum Drift Ratios were observed in Types TFD, TFE and TFF while least Maximum Drift Ratios observed in types TFA, TFB and TFC. Among them all TFF isolator type had the Maximum value with 0.53% and TFC had the least Maximum with 0.39%.

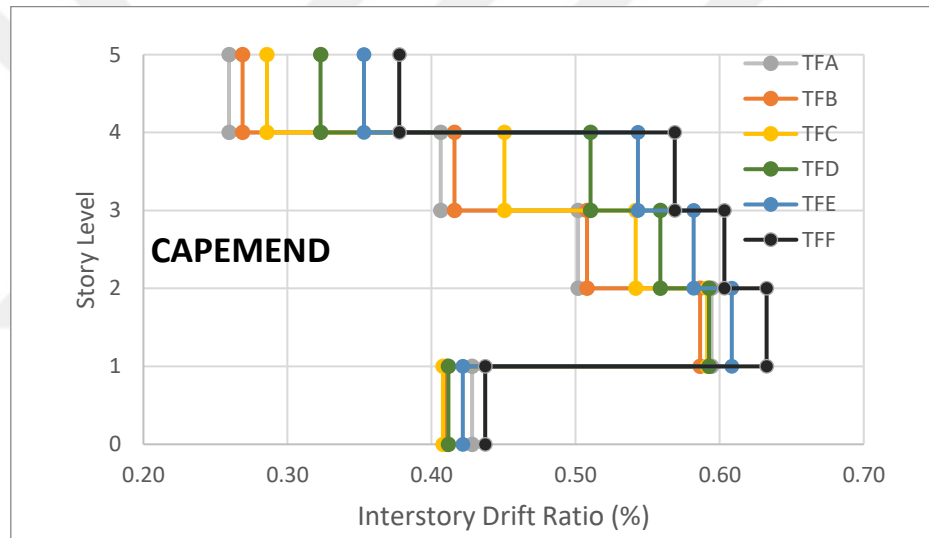


Fig. 8.7: Inter-Storey Drift Ratio Vs Storey Level Under Cape Mendocino Earthquake DBE Scaled

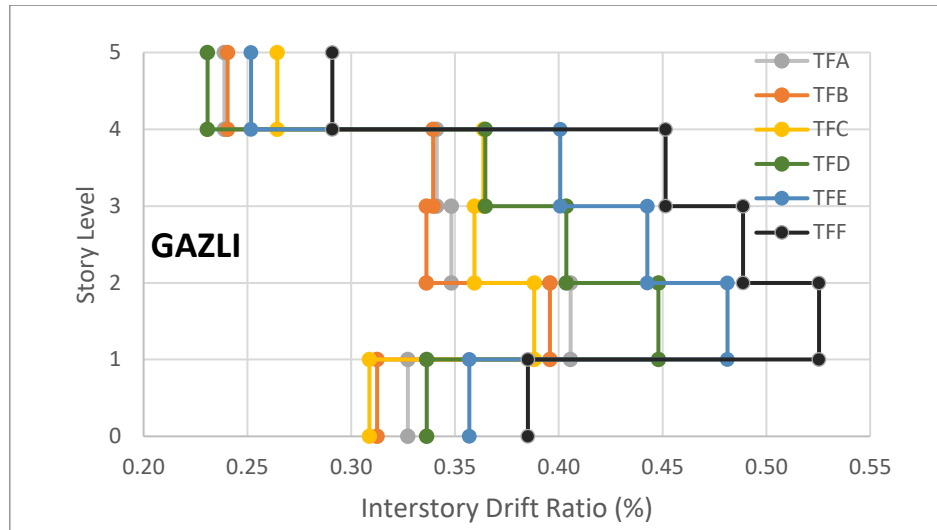


Fig. 8.8: Inter-Storey Drift Ratio Vs Storey Level Under Gazli Earthquake DBE Scaled

For Far field records DBE scaled, starting with Duzce (Figure 8.9), it showed same increase trend in Inter-Storey Drift Ratio from TFA to TFF while this time TFB had less Maximum Ratio value than TFA. Maximum Ratios obtained from TFB and TFF were 0.42 and 0.66%, respectively.

For Kocaeli record (Figure 8.10), structure responded in similar pattern with steady increase of Drift Ratio from TFA to TFF with maximum values equal to 0.46 and 0.66%, respectively.

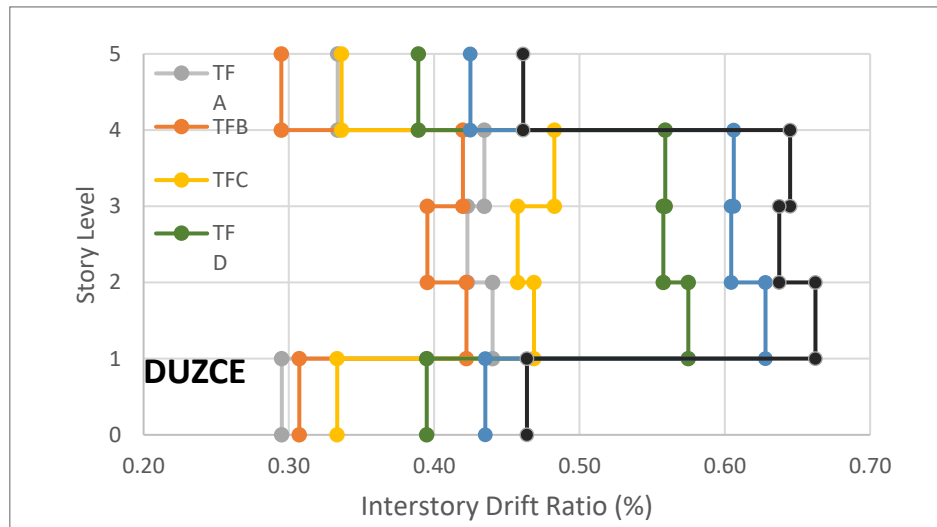


Fig. 8.9: Inter-Storey Drift Ratio Vs Storey Level Under Duzce Earthquake DBE Scaled

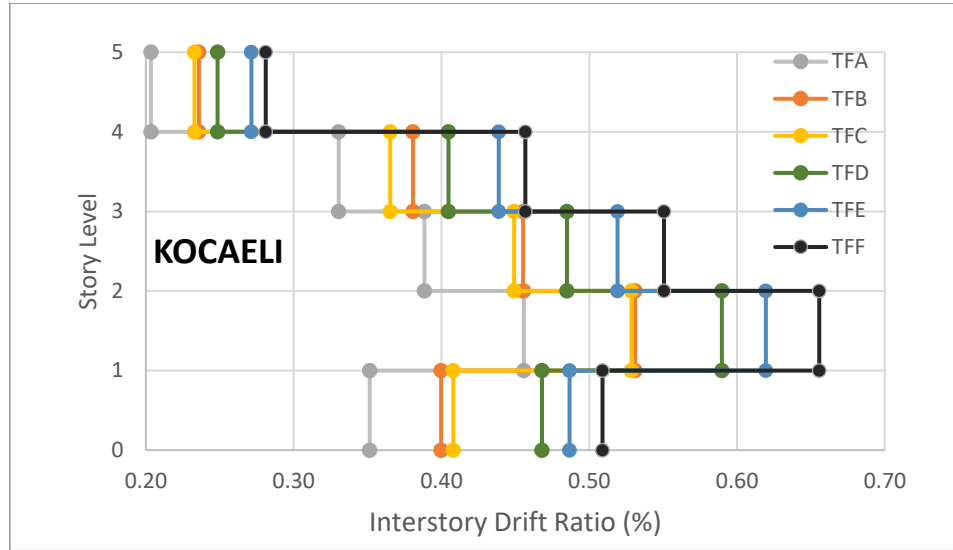


Fig. 8.10: Inter-Storey Drift Ratio Vs Storey Level Under Kocaeli Earthquake DBE Scaled

Finally, in Kobe (Figure 8.11), maximum Inter-Storey Drift Ratios obtained from TFA, TFB and TFF were equal to 0.57, 0.54 and 0.62%, respectively. For This five-storey model it was noticed that 8 records out of ten had the Maximum Storey Drift Ratio at Second Storey Except for Kobe and Loma Prieta they had it at Fourth Storey.

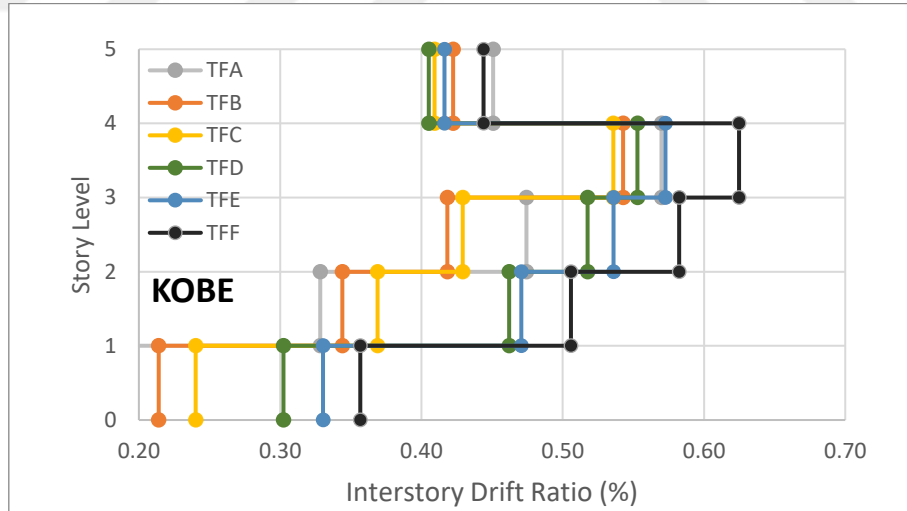


Fig. 8.11: Inter-Storey Drift Ratio Vs Storey Level Under Kobe Earthquake DBE Scaled

For MCE scaled Near Fault records and under CAPEMEND (Figure 8.12) earthquake, it showed same increase trend in Inter-Storey Drift Ratio from TFA to TFF. However, the differences were not big between the six different isolators types. Maximum Ratios obtained from TFA and TFF were 0.81 and 0.88%, respectively. under Gazli (Figure

8.13) earthquake, Maximum Drift Ratios were close, started with 0.69% using type TFA system and decreased to 0.60% with type TFC. Then, increased again to 0.65% using type TFF isolation system.

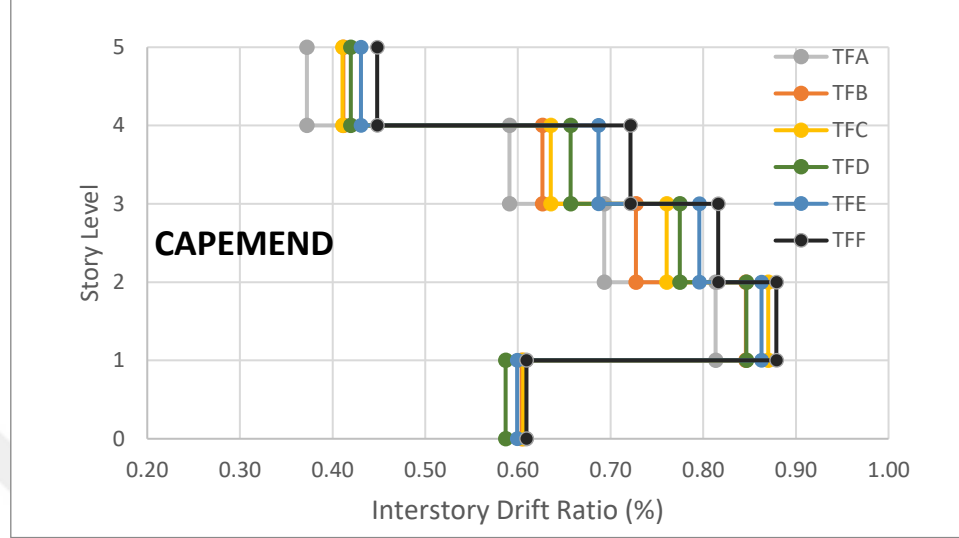


Fig. 8.12: Inter-Storey Drift Ratio Vs Storey Level Under Cape Mendocino Earthquake MCE Scaled

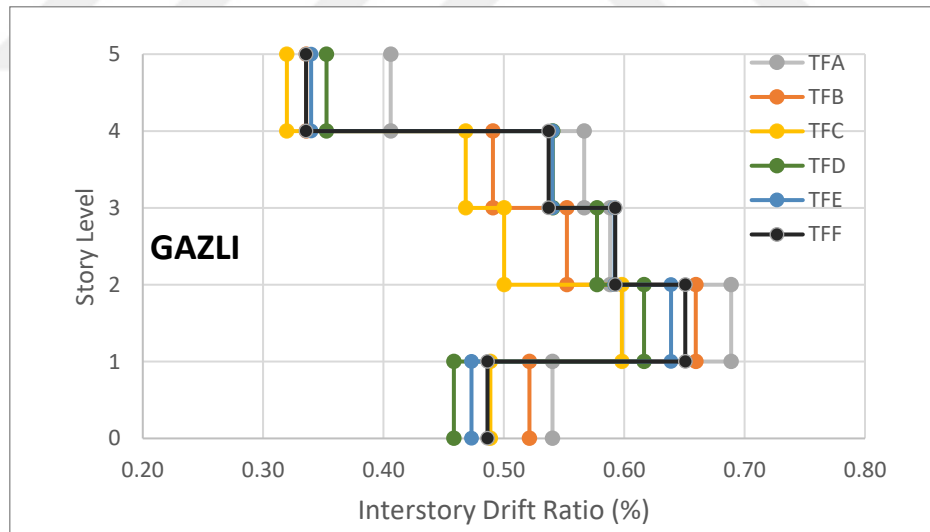


Fig. 8.13: Inter-Storey Drift Ratio Vs Storey Level Under Gazli Earthquake MCE Scaled

For MCE scaled Far Field records and under Duzce (Figure 8.14) earthquake, Maximum Drift Ratio Values from TFA to TFF were 0.66, 0.64, 0.61, 0.61, 0.75 and 0.81%, respectively. Under Kocaeli earthquake (Figure 8.15), The Six Isolators types responded in similar pattern. The least Maximum Drift Ratio occurred in TFA with value of 0.76% and Highest Maximum Drift Ratio occurred in TFF with Value of

0.85% at the second floor. Under Kobe earthquake (Figure 8.16), It showed increasing pattern from TFA to TFF with Maximum Drift Ratios of 0.74 to 0.84%, respectively occurred at the 4th floor.

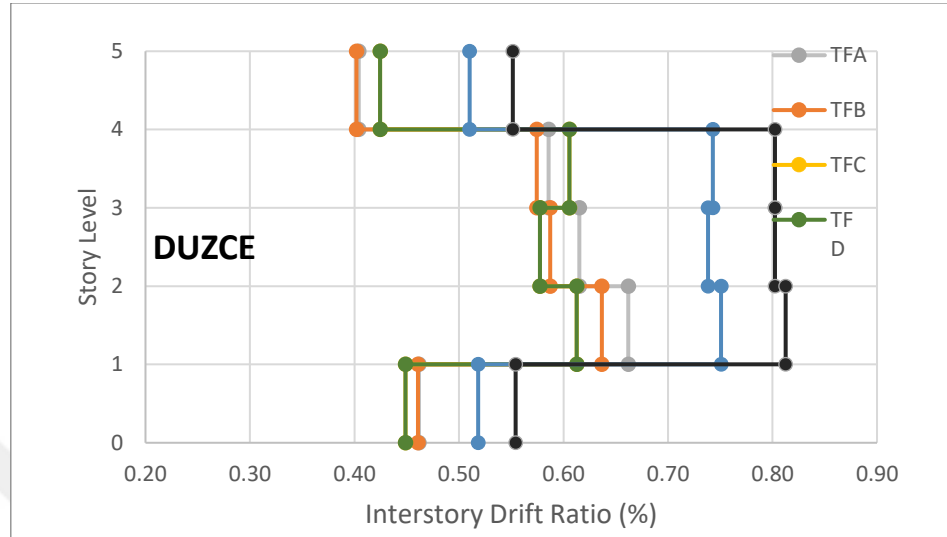


Fig. 8.14: Inter-Storey Drift Ratio Vs Storey Level Under Duzce Earthquake MCE Scaled

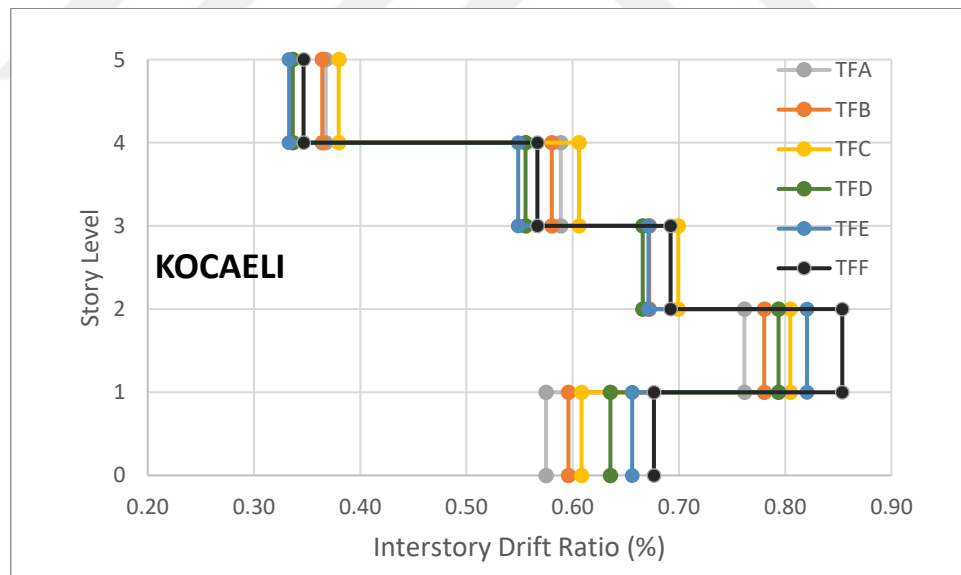


Fig. 8.15: Inter-Storey Drift Ratio Vs Storey Level Under Kocaeli Earthquake MCE Scaled

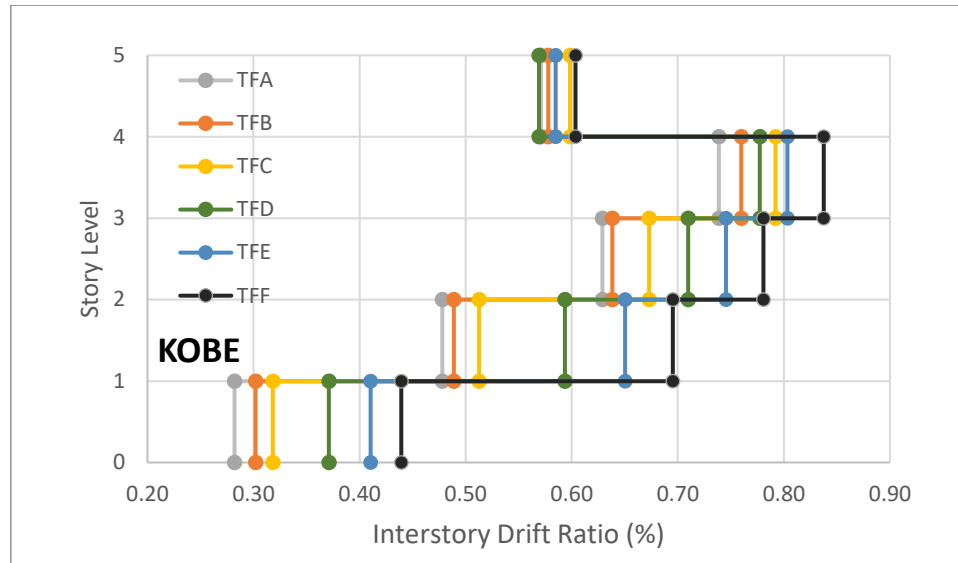


Fig. 8.16: Inter-Storey Drift Ratio Vs Storey Level Under Kobe Earthquake MCE Scaled

For SLE scaled Near Fault records and under Cape Mendocino (Figure 8.17) Earthquake, the Least Maximum Ratios increased from TFA to TFF with values of 0.18 to 0.24%, respectively. But the Highest maximum Drift Ratios using the six isolates were too close between 0.34% and 0.36%. Under Gazli earthquake (Figure 8.18), TFC showed least Maximum Drift Ratio with value of 0.39% and TFF isolator showed the Highest Maximum Ratio of 0.53%. However, TFA and TFB showed closed values with Maximum Drift Ratios of 0.41 and 0.40%, Respectively.

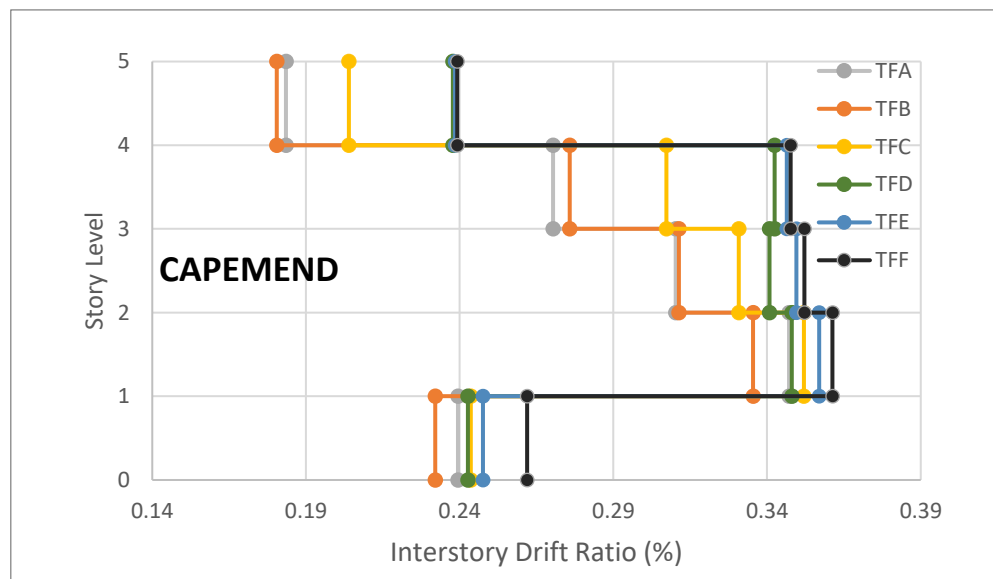


Fig. 8.17: Inter-Storey Drift Ratio Vs Storey Level Under Cape Mendocino Earthquake SLE Scaled

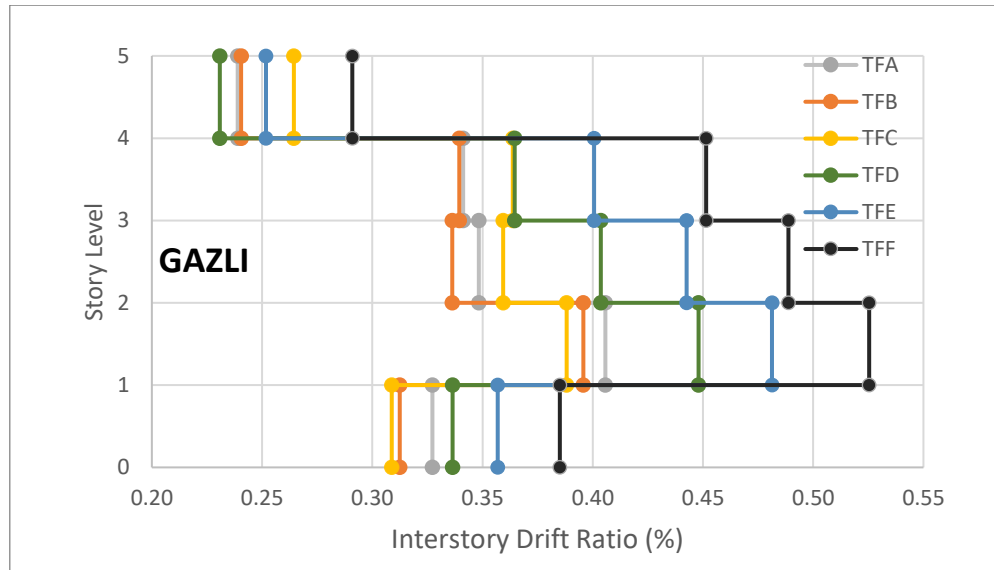


Fig. 8.18: Inter-Storey Drift Ratio Vs Storey Level Under Gazli Earthquake SLE Scaled

For SLE scaled Far Field records and under Duzce (Figure 8.19) earthquake, an increase pattern of maximum Drift Ratios from TFA to TFF was noticed with values of 0.29 and 0.46%, respectively. Under Kocaeli earthquake (Figure 8.20), using the six isolators types frame responded more in similar way compared to Duzce earthquake. Kocaeli showed increase values of maximum Drift Ratios from TFA to TFF type with values of 0.23 and 0.45% at second floor. Finally, under Kobe earthquake (Figure 8.21), frame behaved in similar shape while using six different isolators systems. maximum Drift Ratio using TFA was 0.35%, reduced to 0.32% using TFB. Then, increased gradually to reach 0.39% using TFF isolator type at fourth floor.

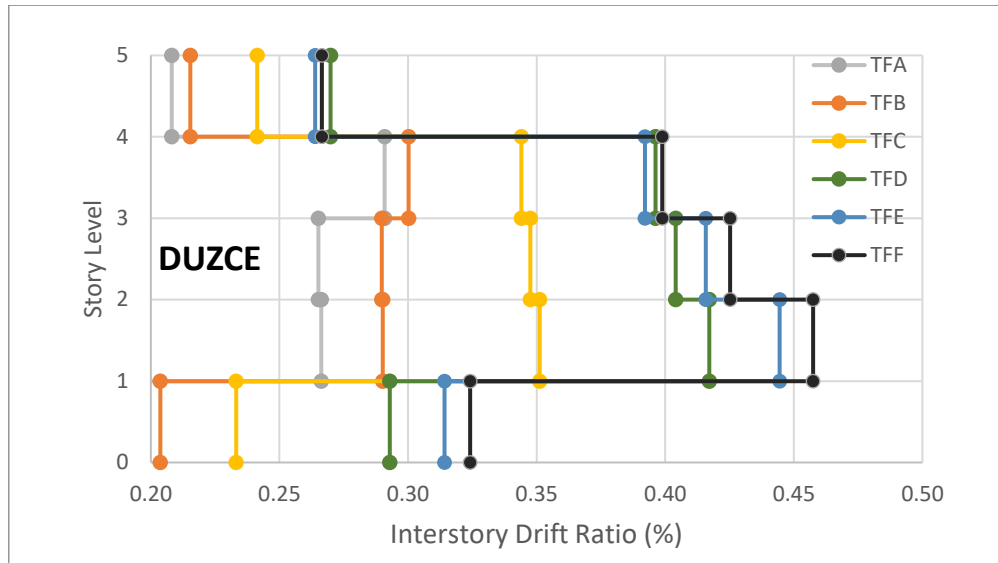


Fig. 8.19: Inter-Storey Drift Ratio Vs Storey Level Under Duzce Earthquake SLE Scaled

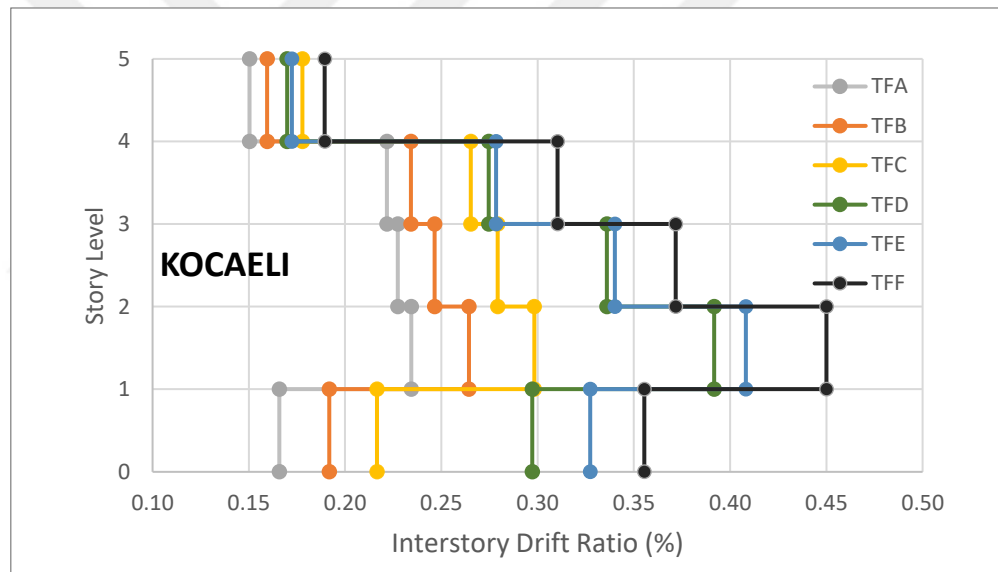


Fig. 8.20: Inter-Storey Drift Ratio Vs Storey Level Under Kocaeli Earthquake SLE Scaled

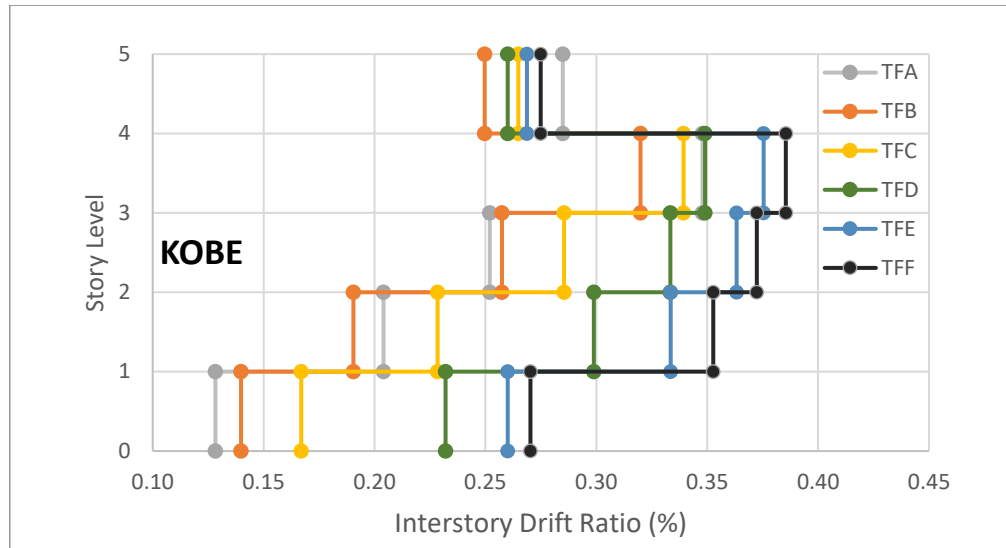


Fig. 8.21: Inter-Storey Drift Ratio Vs Storey Level Under Kobe Earthquake SLE Scaled

8.3. Base Shear:

Base Shear demand values were computed and illustrated in Figures 8.22 and 8.23. In general, all six different isolators tested showed effectiveness with reducing the base shear demand. To make effective comparison, base shear was divided by building own weight. Under near fault DBE scaled records (Figure 8.22) Gazli, Imperial Valley, Loma and Northridge, it was witnessed isolators with smaller outer surfaces friction coefficients responded better and gave less base shear demand. Where highest base shear values were obtained at TFF isolator system with values equal to 0.16, 0.13, 0.10 and 0.15%, respectively. Under Cape Mendocino earthquake, all six isolators gave close shear/weight value of average 0.13%.

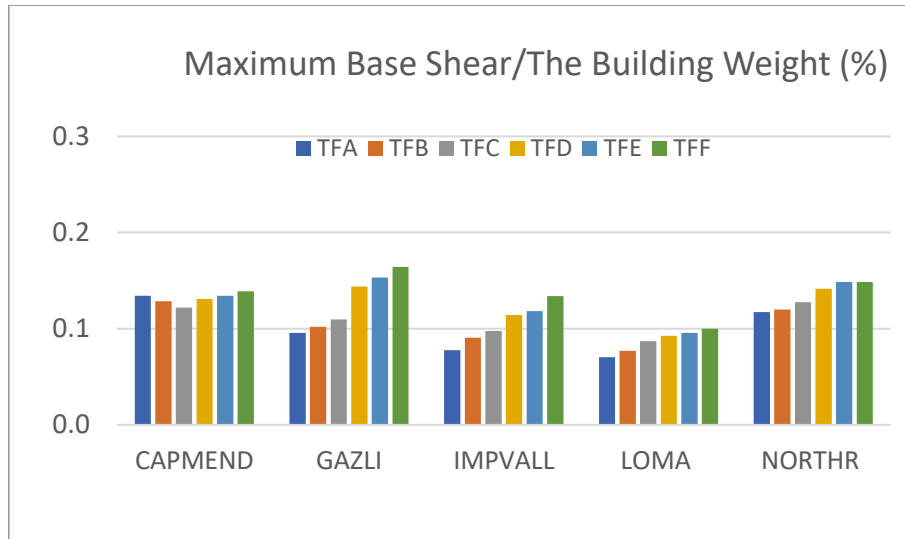


Fig. 8.22: Variation of The Maximum Base Shear with Six Isolators System under Near Fault Ground Motion Records DBE Scaled

Under Far Field DBE scaled records (Figure 8.23), all the five earthquakes showed similar increase in base shear from TFA to TFF isolation system. Maximum Shear/Weight values were obtained from DUZCE, KOBE, KOCAELI, LANDRES and MANJIL were 0.14, 0.16, 0.18, 0.09 and 0.16%, respectively.

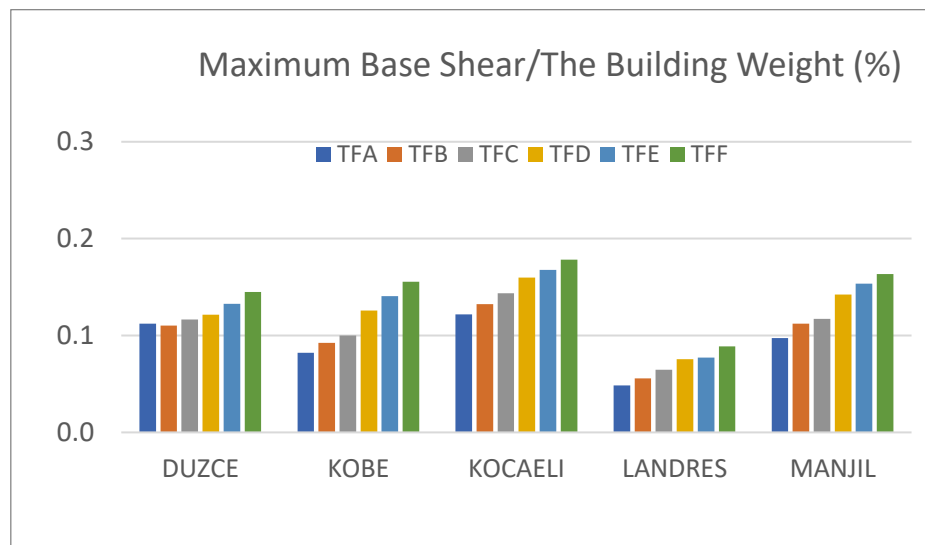


Fig. 8.23: Variation of The Maximum Base Shear with Six Isolators System under Far Field Ground Motion Records DBE Scaled

Under near fault MCE scaled earthquakes (Figure 8.24) Gazli, Imperial Valley, Loma and Northridge, an increase slope from TFA to TFF was noticed again. In General, High friction coefficient outer surfaces isolators types had highest ratios (TFE and

TFF), especially at TFF where the peak obtained with values equal to 0.21, 0.17, 0.12 and 0.20%, respectively. Under Cape Mendocino earthquake, different responses were noticed where the slope was decreasing from TFA to TFF but small percentage.

For Far Field MCE scaled records (Figure 8.25), under Kocaeli earthquake, both TFA and TFB had the ratio of 0.21% then the ratio decreased when TFC was used. Later, it increased again gradually from TFC to TFF to reach 0.24%. Under Duzce earthquake, first four types had close ratio of average 0.16%, increased a little to reach 0.16% using TFE and 0.17% with TFF. The rest records Kobe, Landres and Manjil, they had a clear increasing slope going up from TFA to TFF.

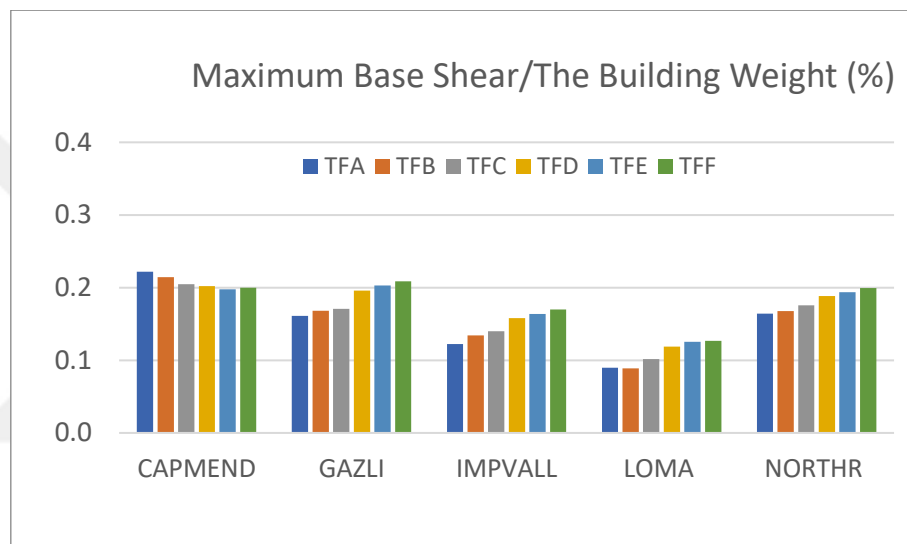


Fig. 8.24: Variation of The Maximum Base Shear with Six Isolators System under Near Fault Ground Motion Records MCE Scaled

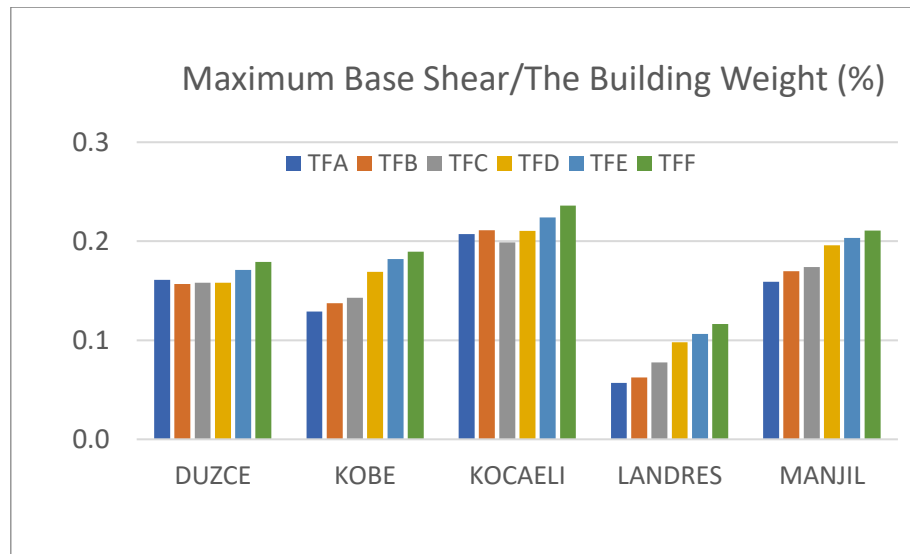


Fig. 8.25: Variation of The Maximum Base Shear with Six Isolators System under Far Field Ground Motion Records MCE Scaled

Under Near Fault SLE scaled earthquakes (Figure 8.26) Cape Mendocino, Gazli, Imperial Valley, Loma and Northridge, the increasing pattern from TFA to TFF was observed in all of them. High-Range friction coefficient isolators types had highest ratios (TFE and TFF). at TFF the ratios were equal to 0.10, 0.16, 0.11, 0.08 and 0.16%, respectively. Same behavior was noticed under Far Field SLE scaled earthquakes (Figure 8.27). Under Duzce, Kobe, Kocaeli, Landres and Manjil they had the lowest Maximum Shear/Weight ratios occurred at TFA. Highest ratios occurred at TFF with values equal to 0.12, 0.11, 0.13, 0.09 and 0.12%, respectively.

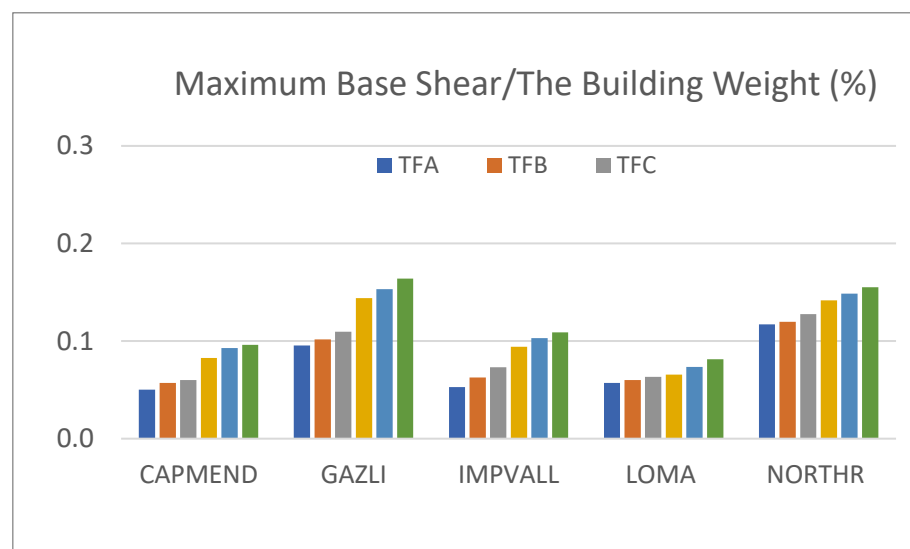


Fig. 8.26: Variation of The Maximum Base Shear with Six Isolators System under Near Fault Ground Motion Records SLE Scaled

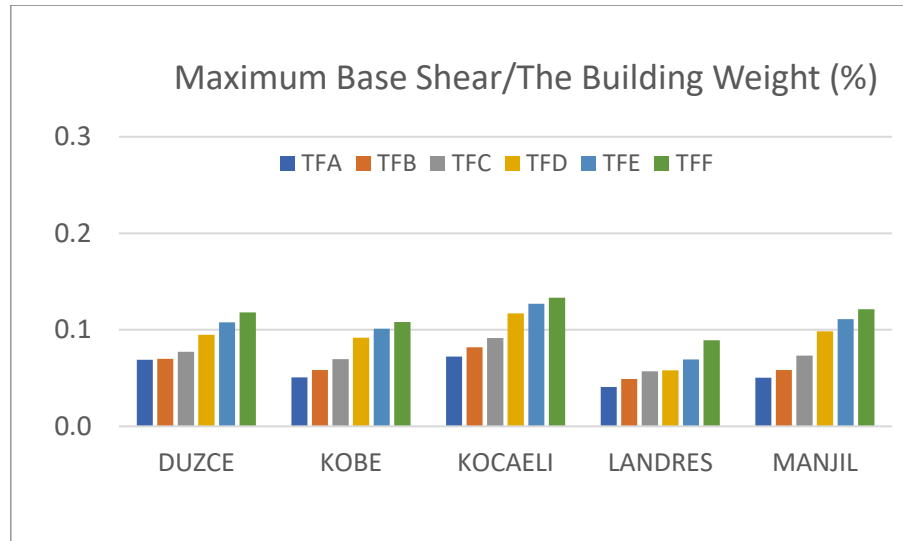


Fig. 8.27: Variation of The Maximum Base Shear with Six Isolators System under Far Field Ground Motion Records SLE Scaled

8.4. Roof Drift Ratio:

Roof Drift Ratio is another good indicator for the structure response under earthquake motions, it is calculated by finding relative displacement between the roof-storey and the base, normalized by dividing the building height to remove the shape effect.

Under Near Fault records DBE scaled (see Figure 8.28), Starting with Gazli, the Roof Drift Ratios were proportionally low values when isolators with small-range friction coefficients of outer surfaces isolators were used such as TFA, TFB and TFC. Later, it increased at higher-range friction coefficient isolators (TFE, TFF). Maximum Roof Drift Ratio was obtained at TFF with value equal to 4.3% and minimum value occurred at TFA with value equal to 3.2%. Under Cape Mendocino, Imperial Valley and Northridge, Roof Drift Ratios increased gradually from TFA to TFF. Highest Ratios observed at TFF with values equal to 5.2, 4.8 and 4.1%, respectively. Under Loma Prieta, TFA type showed lower Maximum Roof Drift Ratio compared to the other 5 types with value equal to 2.5%. The rest five type systems showed close range of maximum ratios with average of 3.5%.

Under Far Field records DBE scaled (see Figure 8.29), an increase in Maximum Roof Drift Ratios from TFA to TFF was noticed in Duzce, Kobe, Kocaeli and Manjil. However, the increase wasn't gradually steady for example under Duzce and Kobe, using TFA system gave slightly higher ratio than TFB. And under Kocaeli, using TFB system gave higher ratio compare to TFC. For the four records top maximum ratios

occurred at TFF of 5.7, 4.8, 4.9 and 4.1%, respectively. Under Landres earthquake, the ratios observed using the six systems were proportionally close to each other with average of 2.1%.

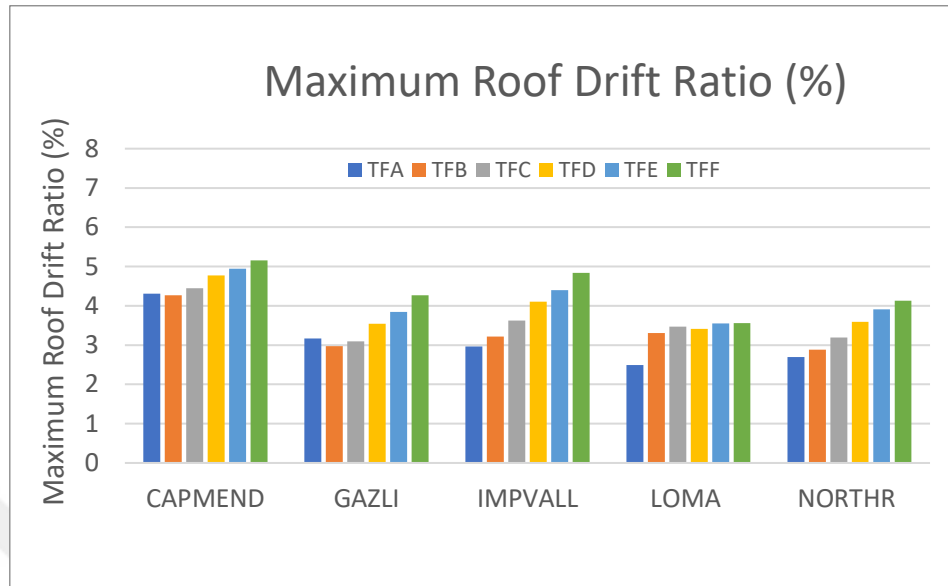


Fig. 8.28: Variation of The Maximum Roof Drift Ratio with Six Isolators System under Near Fault Field Ground Motion Records DBE Scaled

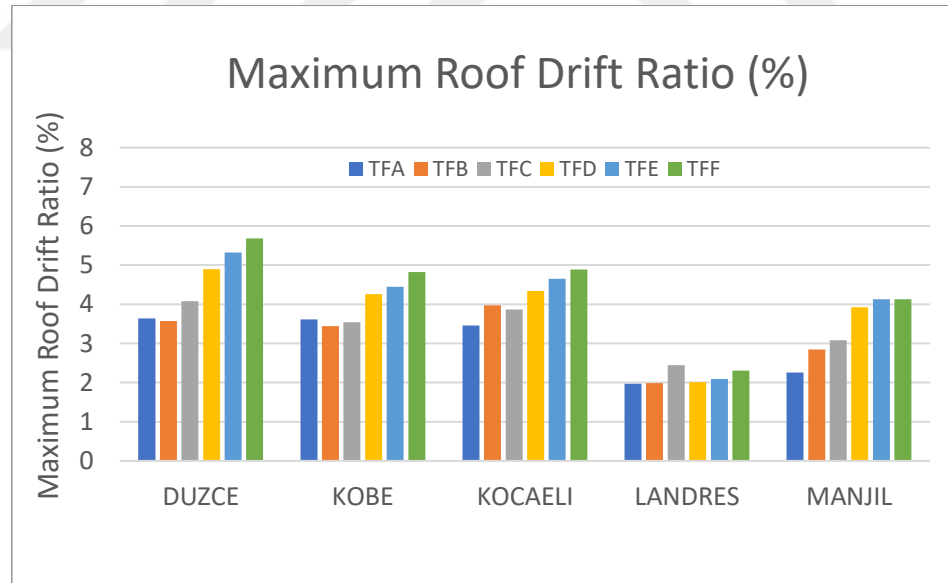


Fig. 8.29: Variation of The Maximum Roof Drift Ratio with Six Isolators System under Far Field Ground Motion Records DBE Scaled

Under Near Fault records MCE scaled (see Figure 8.30), Under Cape Mendocino, Imperial Valley, Loma Prieta and Northridge, Roof Drift Ratios increased gradually from TFA to TFF. Highest Ratios noticed using TFF type with values equal to 6.9, 5.7,

4.9 and 5.2%, respectively. Under Gazli, decrement was observed in Maximum Roof Drift Ratios from TFA to TFC, 5.2% to 4.4%, respectively. Then, there was a slight increase from TFC to TFF reaching the Maximum Roof Drift Ratio of 5.2%.

Under Far Field records MCE scaled (see Figure 8.31), a gradually increase in Maximum Roof Drift Ratios from TFA to TFF was noticed under Kobe and Manjil to reach highest values at TFF equal to 6.5% and 5.6%. Under Duzce earthquake record, a jump increase happened from TFD to TEE from 5.1 to 6.5%, respectively. Increasing pursued with TFF to reach highest ratio of 7.0%. Under Landres earthquake, increase pattern from TFA to TFE was noticed to reach value of 3.4%. Then, it went downward using TFF to 2.8%.

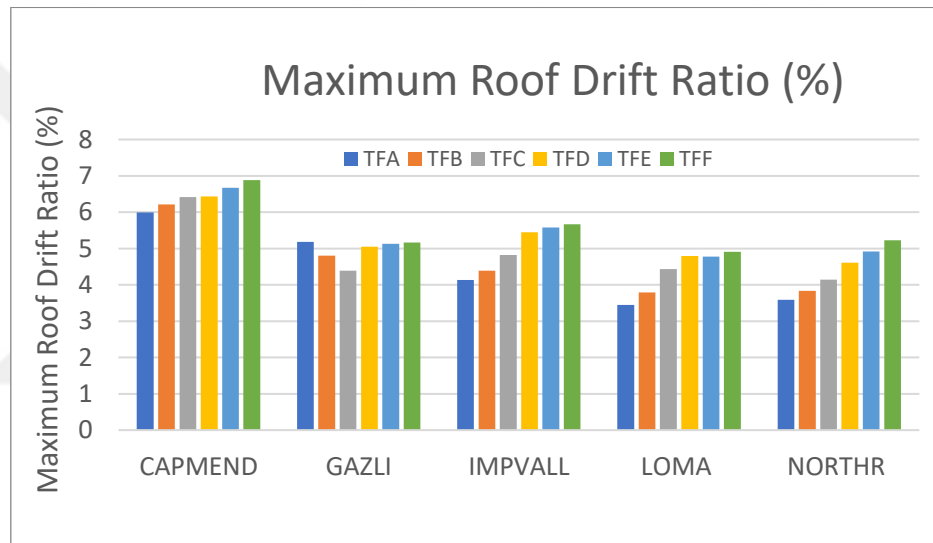


Fig. 8.30: Variation of The Maximum Roof Drift Ratio with Six Isolators System under Near Fault Field Ground Motion Records MCE Scaled

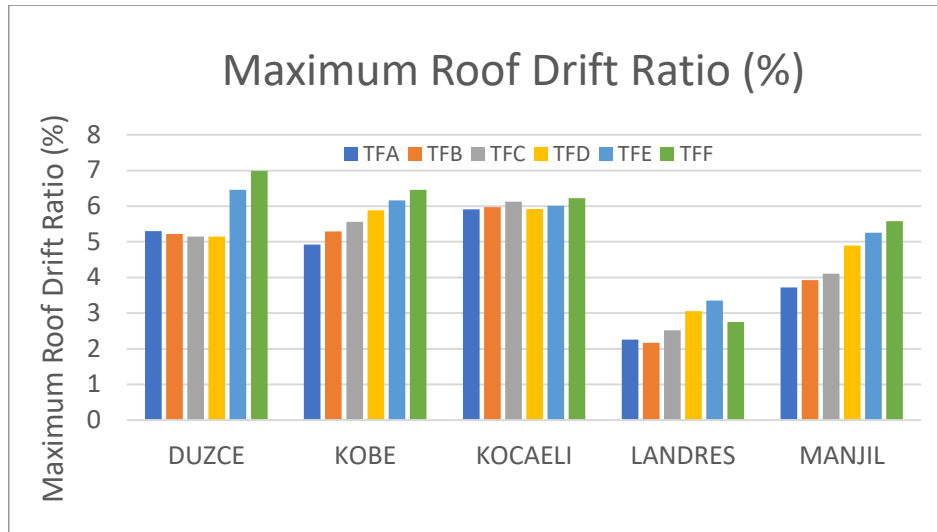


Fig. 8.31: Variation of The Maximum Roof Drift Ratio with Six Isolators System under Far Field Ground Motion Records MCE Scaled

Under Near Fault records SLE scaled (see Figure 8.32) records, Cape Mendocino, Imperial Valley and Northridge, Roof Drift Ratios increased gradually from TFA to TFF reaching highest ratios of 0.31, 0.31 and 0.41%, respectively. Under Gazli earthquake, using TFA gave higher ratio compared to TFB. However, from TFB to TFF increasing slope was observed. Under Loma Prieta, there was no noticeable difference in response came out from the six different types and the average of the values was 2.0%.

Under all five Far Field records SLE scaled (see Figure 8.33), Maximum Roof Drift Ratios went gradually upward from TFA to TFF, except for Landres record where TFA gave slightly higher ratio compared to TFB. However, for the five records (Duzce, Kobe, Kocaeli, Landres and Manjil) highest points were noticed when TFF was used with values of 3.7, 3.1, 3.3, 2.1 and 4.1%, respectively.

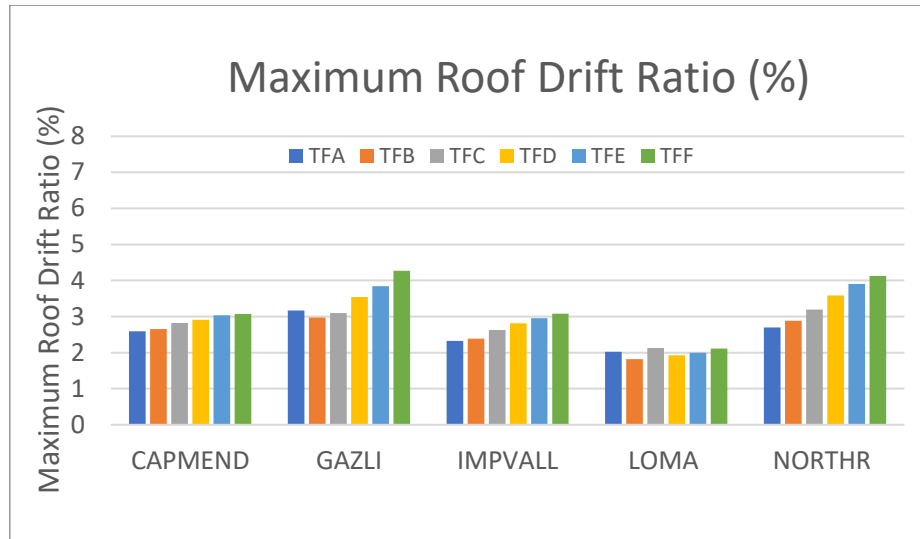


Fig. 8.32: Variation of The Maximum Roof Drift Ratio with Six Isolators System under Far Field Ground Motion Records SLE Scaled

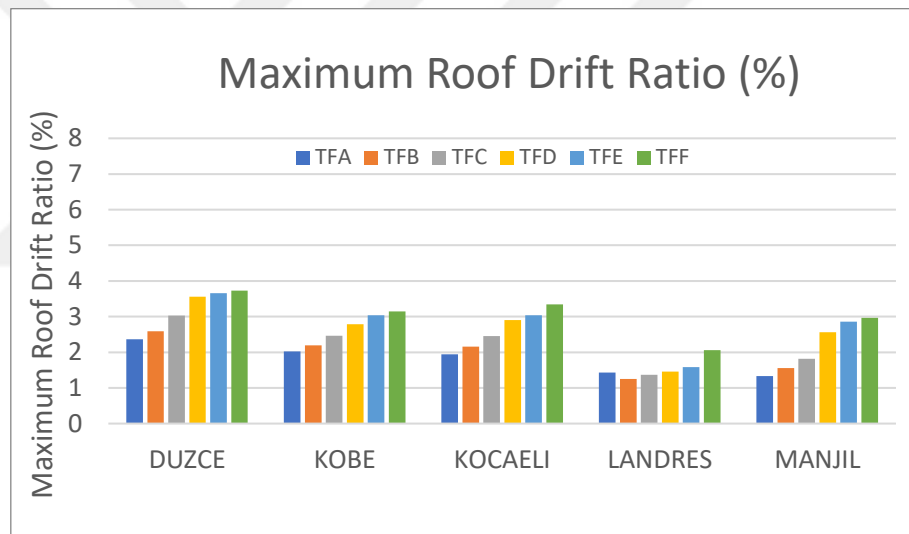


Fig. 8.33: Variation of The Maximum Roof Drift Ratio with Six Isolators System under Far Field Ground Motion Records SLE Scaled

8.5. Absolute Acceleration:

Maximum corresponding Absolute Acceleration of the steel frame tested and scaled for DBE illustrated for Near Fault and Far Field records in Figures 8.34 and 8.35, respectively. Different responses noticed under different earthquakes in term of Absolute Acceleration, no certain general pattern was found. However, under Duzce, Kocaeli, Manjil, Northridge and Cape Mendocino, Mid-Range Friction isolators (TFC and TFD) exhibited better performance and lower Absolute Accelerations compared

to High Friction isolators (TFE and TFF) and Low Friction isolators (TFA and TFB) but again Under Loma Prieta and Gazli, TFC and TFD showed higher values. Among the ten tested records and based on the values obtained, TFB isolator system showed lowest Absolute Acceleration performance.

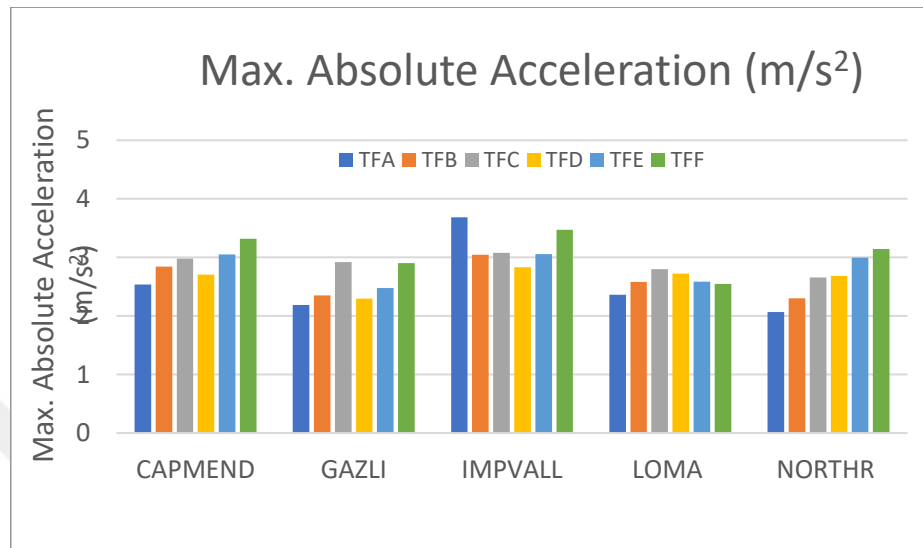


Fig. 8.34: Variation of The Absolute Acceleration with Six Isolators System under Near Fault Ground Motion Records DBE Scaled

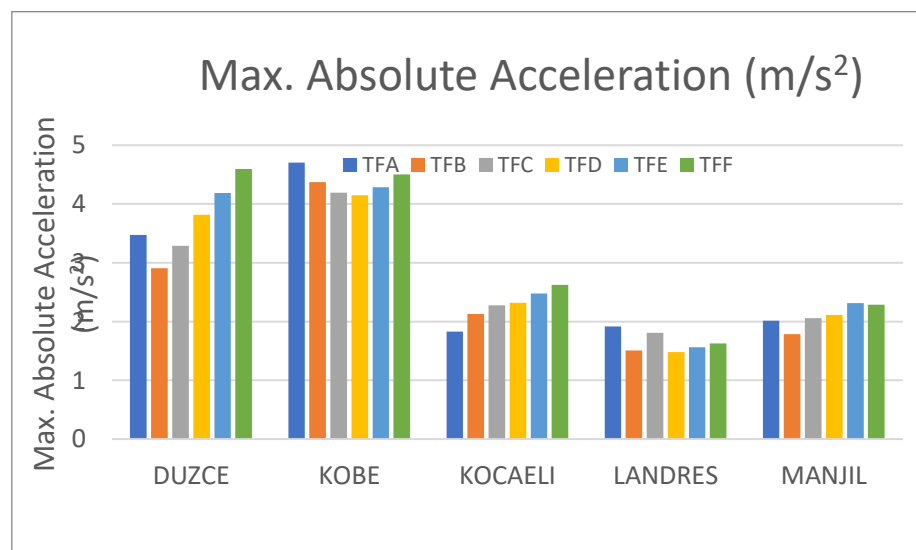


Fig. 8.35: Variation of The Absolute Acceleration with Six Isolators System under Far Field Ground Motion Records DBE Scaled

For MCE scaled records as well there were not certain response pattern to generate. However, TFB and TFC system showed good performance in most cases either Near Fault or Far Field records, see Figures 8.36 and 8.37.

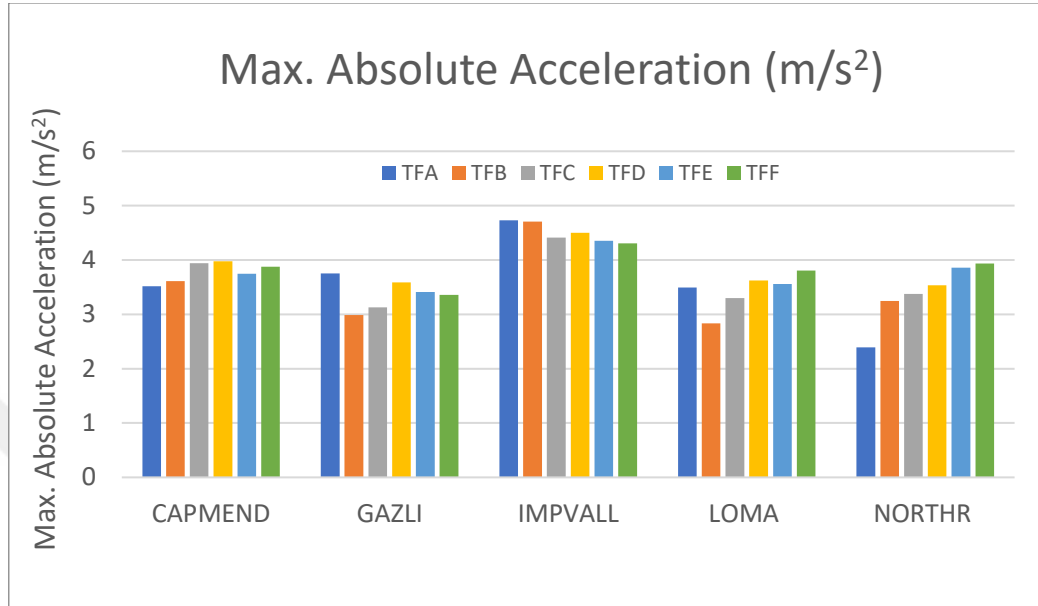


Fig. 8.36: Variation of The Absolute Acceleration with Six Isolators System under Near Fault Ground Motion Records MCE Scaled

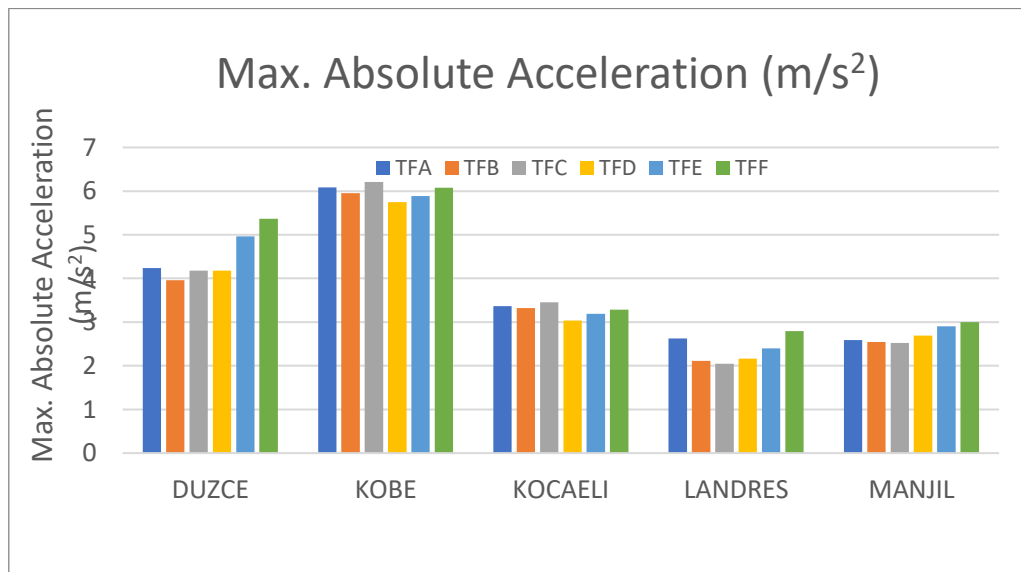


Fig. 8.37: Variation of The Absolute Acceleration with Six Isolators System under Far Field Ground Motion Records MCE Scaled

For SLE scaled records see Figures 8.38 and 8.39, Under Imperial Valley, Loma Prieta, Kobe and Kocaeli Maximum Absolute Accelerations were proportionally close using

the six different isolator systems. Under Northridge, Cape Mendocino, Duzce and Manjil earthquakes, an increasing value from TFA to TFF was noticed.

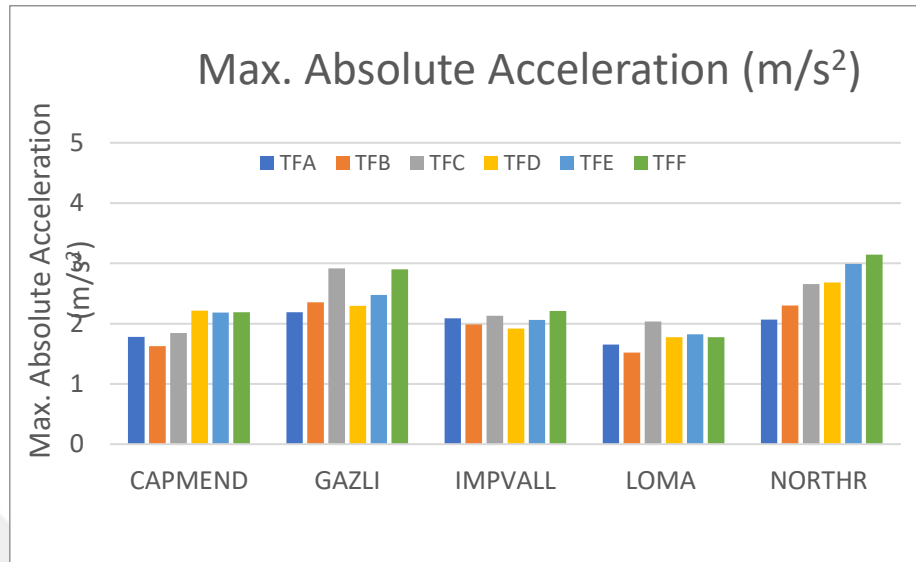


Fig. 8.38: Variation of The Absolute Acceleration with Six Isolators System under Near Fault Ground Motion Records SLE Scaled

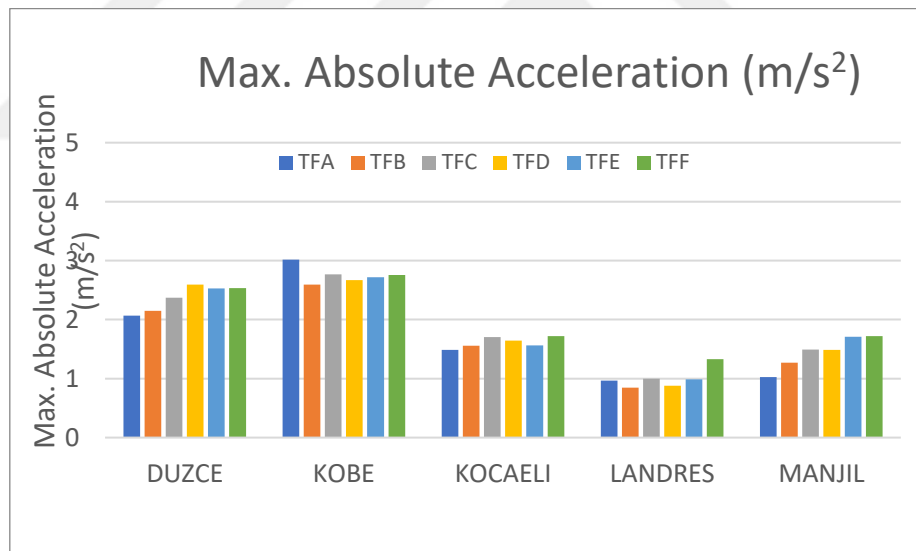


Fig. 8.39: Variation of The Absolute Acceleration with Six Isolators System under Far Field Ground Motion Records SLE Scaled

8.5.1. Maximum Absolute Acceleration Versus Storey:

Maximum Absolute Acceleration Versus Storey level Variation under Loma Prieta, Gazli, Kocaeli and Kobe earthquakes Scaled to DBE showed were shown in Figures (8.40, 8.41, 8.42 and 8.43), respectively. To look to other records' results, check

Appendix B. Under Loma Prieta earthquake, Maximum Absolute Acceleration occurred when TFC was used with value of 2.72 m/s^2 at 5th floor while least Maximum Absolute Acceleration happened when TFA was used with value of 2.36 m/s^2 . Under Gazli Maximum Absolute Acceleration occurred when model was isolated with TFC type with value of 2.92 m/s^2 and least Maximum Absolute Acceleration happened when TFA was used with acceleration equal to 2.19 m/s^2 . Under Kocaeli Earthquake (Figure 8.42), it was the only record among the ten tested records that showed clear pattern. The six isolators types systems showed similar response behavior with increase in Maximum Absolute Acceleration from TFA to TFF with values of 1.83 m/s^2 to 2.63 m/s^2 , respectively. Under Kobe earthquake (Figure 8.43), The Maximum Absolute Acceleration occurred with TFA system with value of 4.71 m/s^2 and least Maximum Absolute Acceleration occurred with TFD system with value of 4.15 m/s^2 at 5th floor.

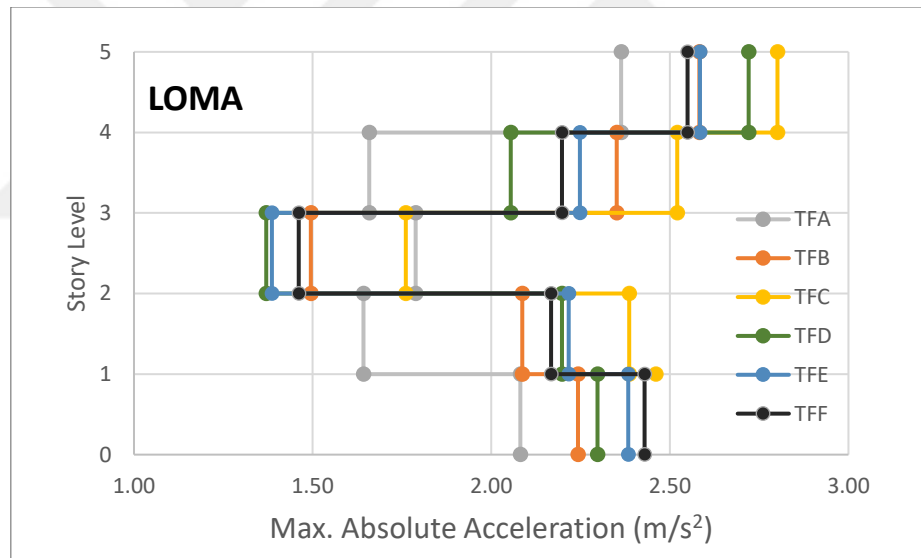


Fig. 8.40: Absolute Acceleration Against Storey Height Under Loma Prieta DBE Scaled

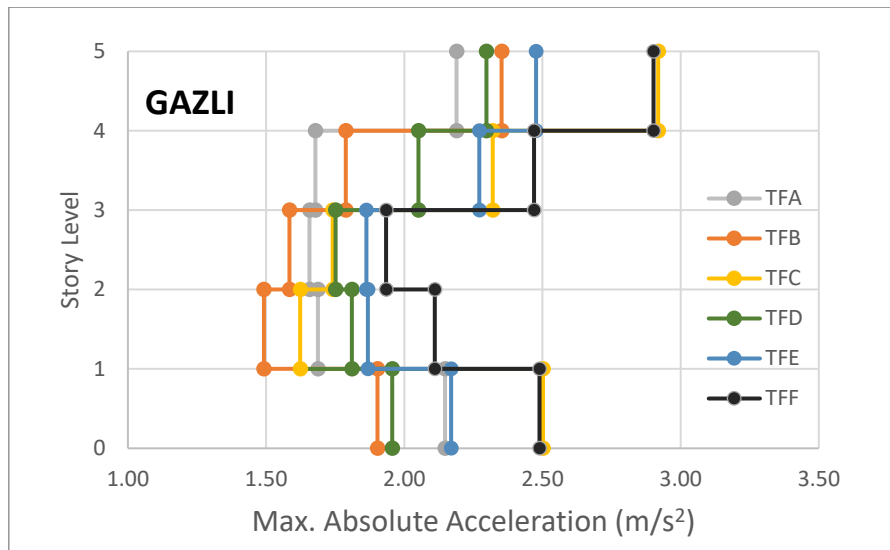


Fig. 8.41: Absolute Acceleration Against Storey Height Under Gazli DBE Scaled

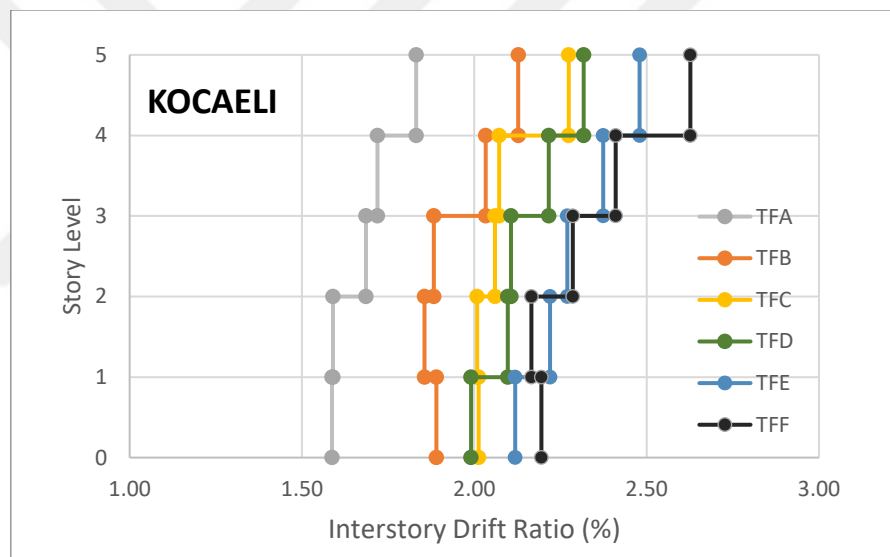


Fig. 8.42: Absolute Acceleration Against Storey Height Under Kocaeli DBE Scaled

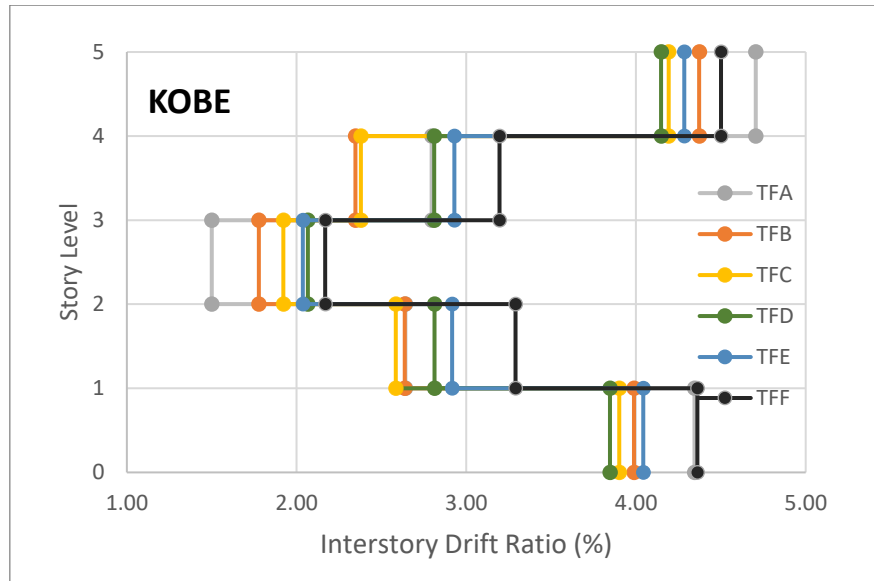


Fig. 8.43: Absolute Acceleration Against Storey Height Under Kobe DBE Scaled

For MCE scaled analyses, starting with Loma Prieta earthquake (Figure 8.44), Maximum Absolute Acceleration occurred when TFF was used with value of 3.81 m/s^2 while least Maximum Absolute Acceleration happened when TFB was used with value of 2.83 m/s^2 . Under Gazli earthquake (Figure 8.45), Highest Max. Acceleration value happened when the frame isolated by TFA system with Acceleration value equal to 3.75 m/s^2 and lowest Max. Acceleration value was equal 2.99 m/s^2 occurred when TFB system was used. Under Kocaeli earthquake (Figure 8.46), Maximum Absolute Acceleration occurred when TFC was used with value of 3.45 m/s^2 while least Maximum Absolute Acceleration happened when TFD was used with value of 3.02 m/s^2 . Under Kobe earthquake (Figure 8.47), there was some sort of similar response behavior of the six isolators systems. However, highest Maximum Acceleration value was obtained at TFC with value of 6.21 m/s^2 and lowest Maximum Acceleration obtained at TFD with value of 5.75 m/s^2 .

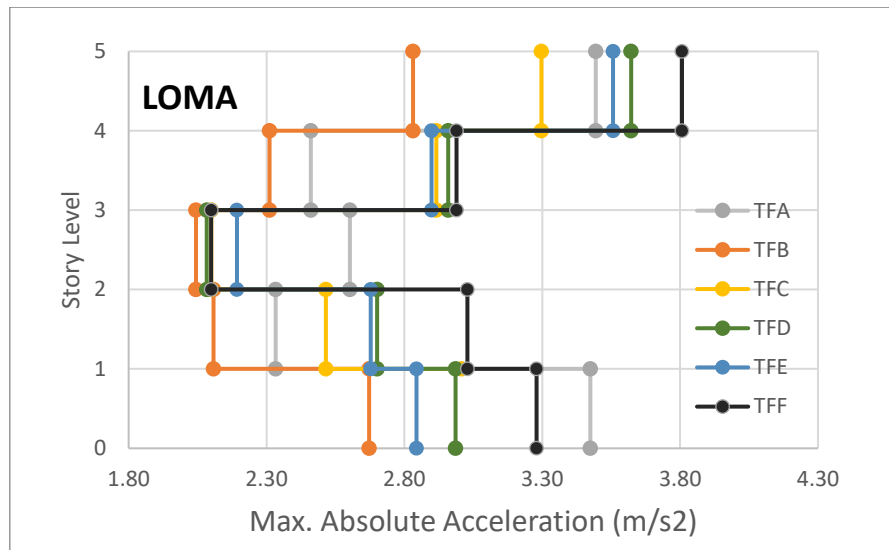


Fig. 8.44: Absolute Acceleration Against Storey Height Under Loma Prieta MCE Scaled

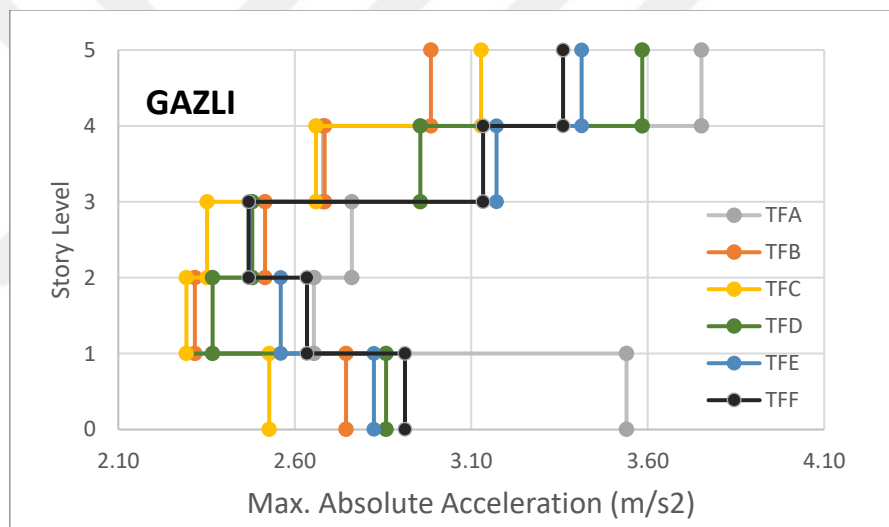


Fig. 8.45: Absolute Acceleration Against Storey Height Under Gazli MCE Scaled

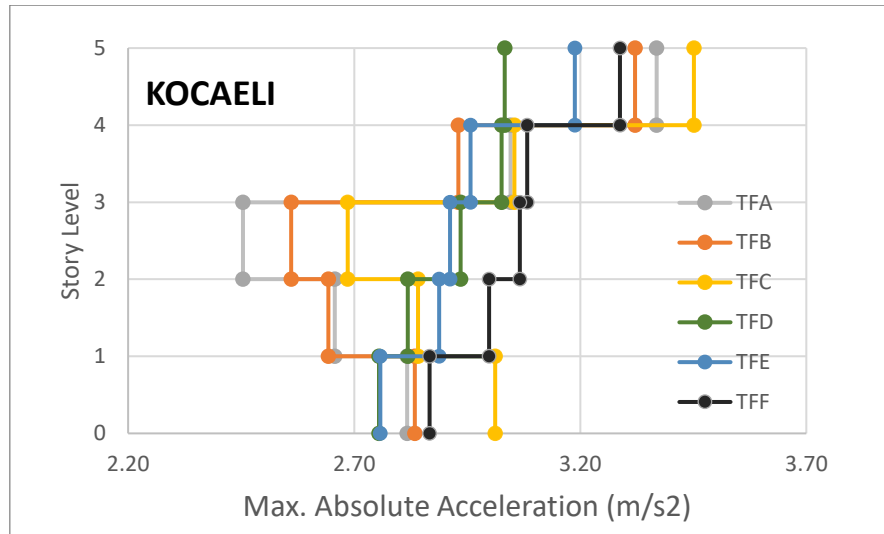


Fig. 8.46: Absolute Acceleration Against Storey Height Under Kocaeli MCE Scaled

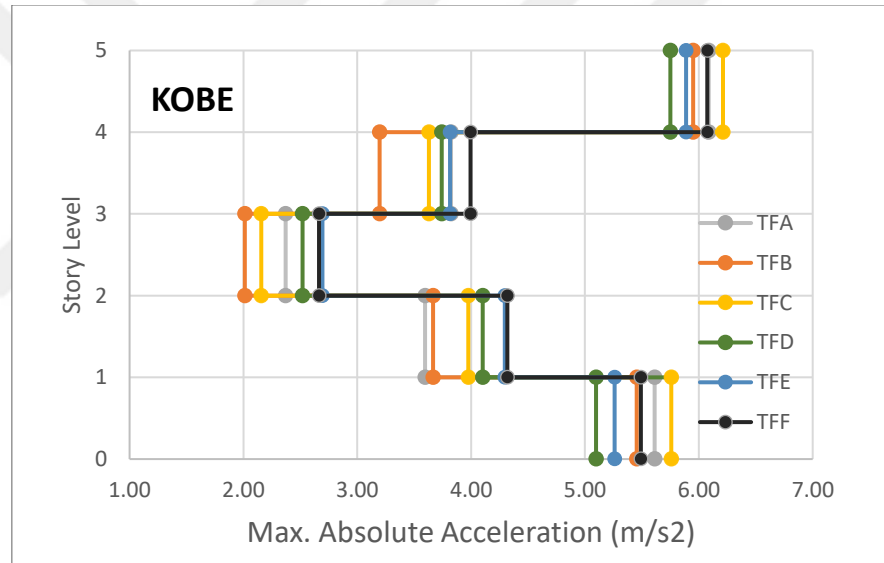


Fig. 8.47: Absolute Acceleration Against Storey Height Under Kobe MCE Scaled

For SLE scaled frame models, under Loma Prieta earthquake (Figure 8.48), Maximum Absolute Acceleration occurred when TFF was used with value of 3.81 m/s^2 while least Maximum Absolute Acceleration happened when TFB was used with value of 2.83 m/s^2 . Under Gazli earthquake (Figure 8.49), Again when TFC was used occurred the highest acceleration compared to other isolators types with value of 2.92 m/s^2 . Lowest Max. Acceleration Value obtained at TFA system with value of 2.19 m/s^2 . Under Kocaeli earthquake record (Figure 8.50), Maximum Absolute Acceleration values were proportionally close to each other, ranged from 1.48 m/s^2 and 1.72 m/s^2

for TFA and TFF respectively. under Kobe earthquake (Figure 8.51), Maximum Absolute Acceleration occurred when TFA was used with value of 3.01 m/s^2 while least Maximum Absolute Acceleration happened when TFB was used with value of 2.60 m/s^2 .

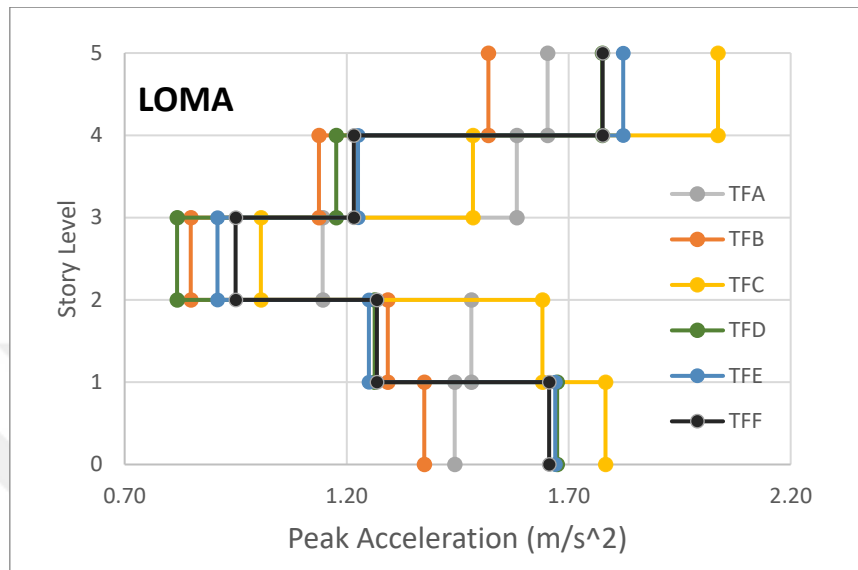


Fig. 8.48: Absolute Acceleration Against Storey Height Under Loma Prieta SLE Scaled

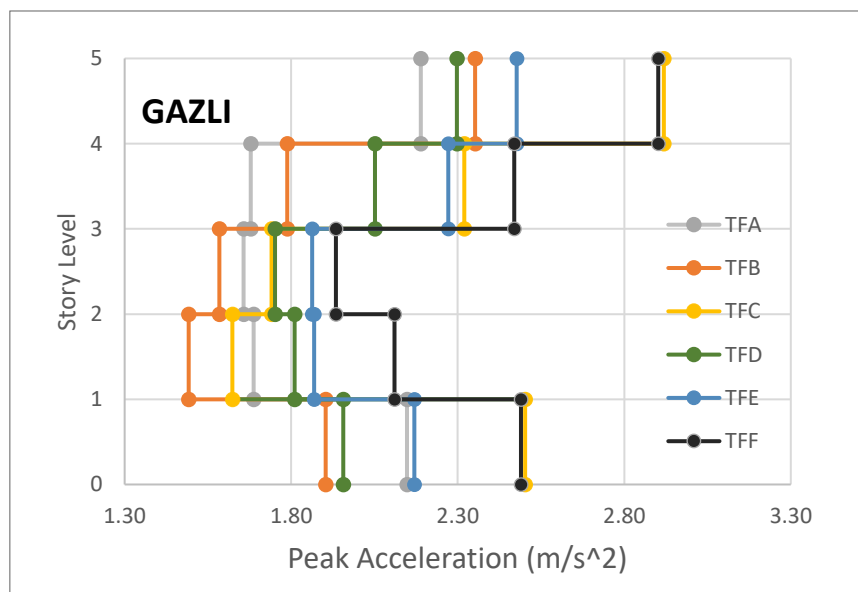


Fig. 8.49: Absolute Acceleration Against Storey Height Under Gazli SLE Scaled

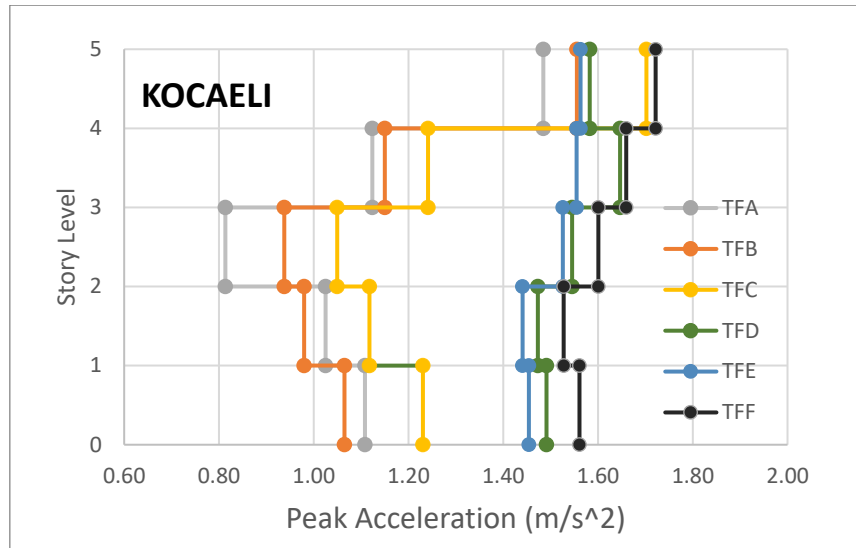


Fig. 8.50: Absolute Acceleration Against Storey Height Under Kocaeli SLE Scaled

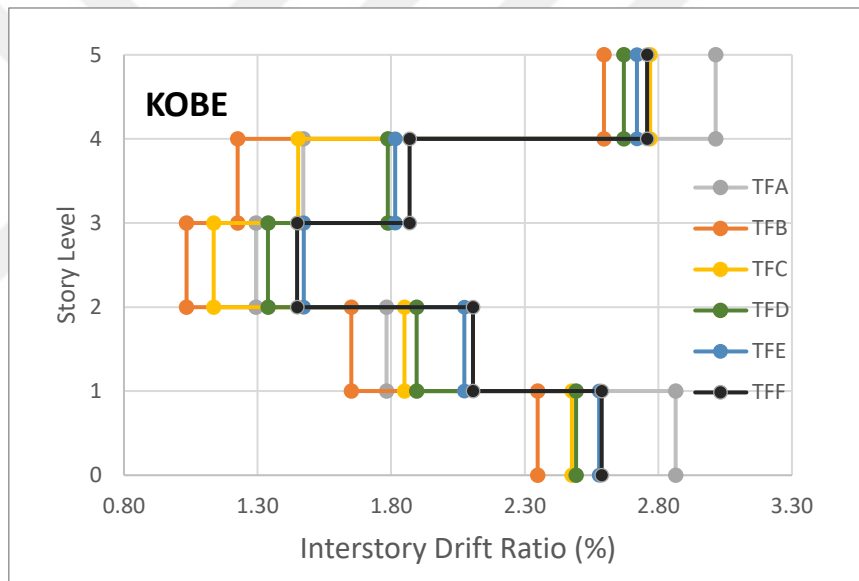


Fig. 8.51: Absolute Acceleration Against Storey Height Under Kobe SLE Scaled

8.6. Hysteric Loop:

The Hysteric Force-Deformation loop shown in Figures 8.52, 8.53, 8.54 and 8.55 under three earthquakes Cape Mendocino, Imperial Valley and Northridge, respectively. All agreed with Constantinou's loop curve in Figure 6. Firstly, under Cape Mendocino earthquake (Fig. 8.52, 8.53) DBE Scaled, average maximum shear obtained among the isolators was around to 393 kN for TFA, TFB, TFC, TFD, TFE and TFF. Low-range and Mid-range friction coefficients isolators showed smooth loop

movement. Using High-range friction coefficients isolators (TFE, TFF) systems, the loop lines became less straight and solid due to big shift between inner plates and outer plates friction coefficients in my assumption. For SLE scaled Cape Mendocino record (Figure 8.54), First three types (TFA, TFB, TFC) created the expected loop shape. However, TFD, TFE and TFD types made more linear-shape response due to the fact of selected isolator size became over designed for this reduced record when there is not enough force to make the upper surfaces with higher friction coefficients move. For MCE scaled Cape Mendocino record (Figure 8.55), Differences between the six isolators types became less noticeable. Although, isolators with lower friction coefficient differences TFA, TFB and TFC gave slightly more base isolation period.

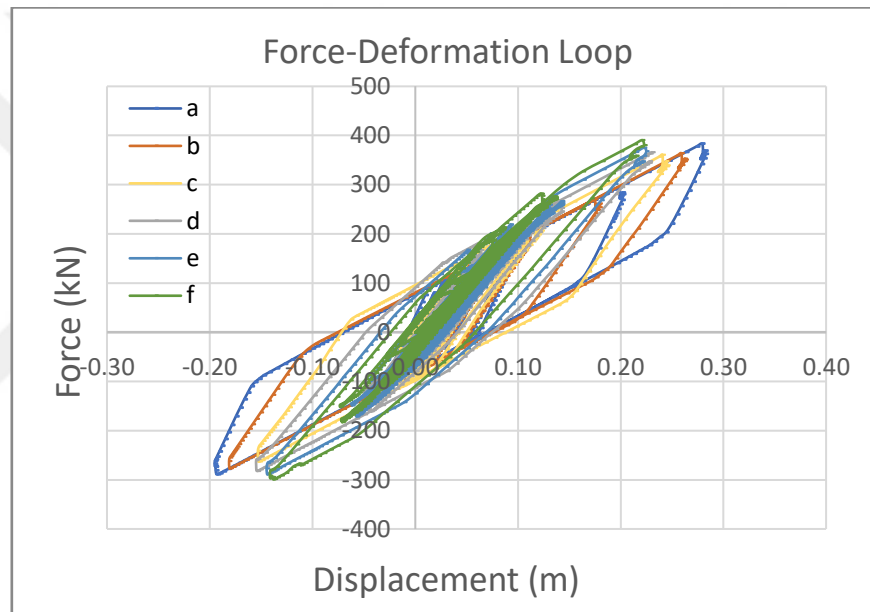


Fig. 8.52: Force-Deformation Loop of Six Different Isolation Systems under Cape Mendocino Earthquake DBE Scaled

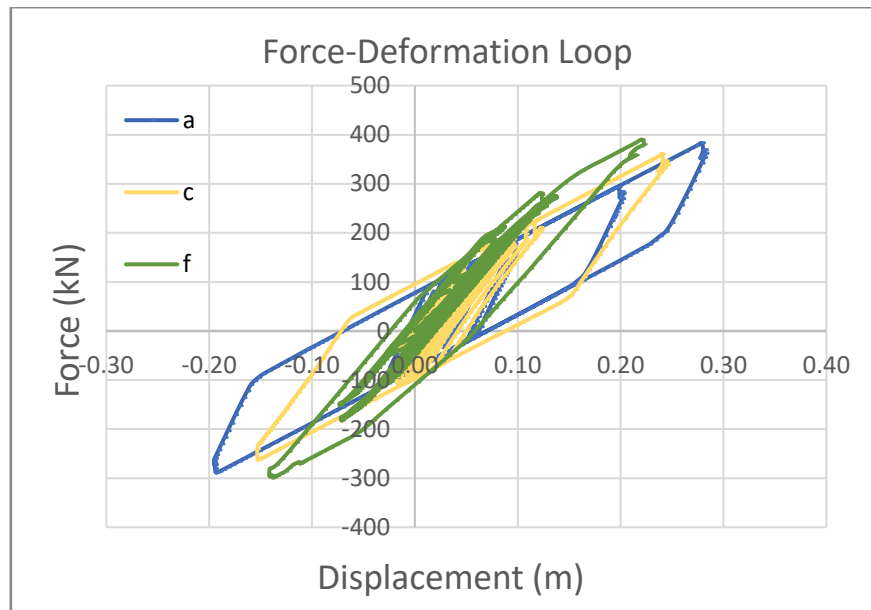


Fig. 8.53: Force-Deformation Loop of Three Isolation Systems only (Spotlight) Under Cape Mendocino Earthquake DBE Scaled

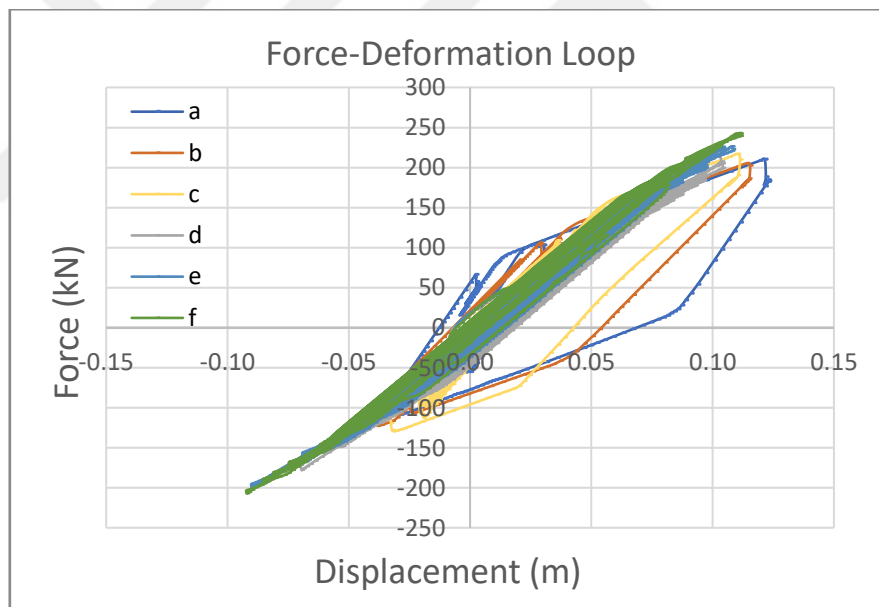


Fig. 8.54: Force-Deformation Loop of Six Different Isolation Systems under Cape Mendocino Earthquake SLE Scaled

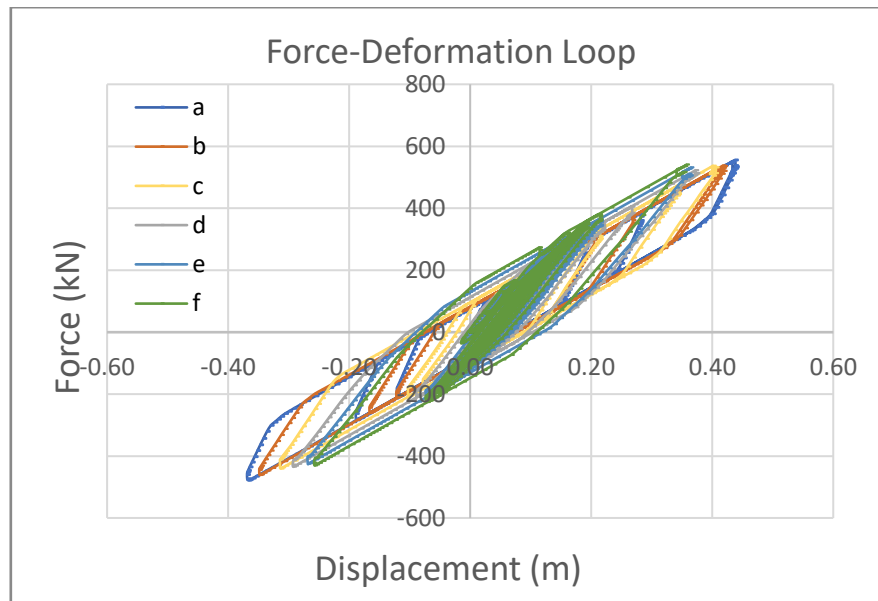


Fig. 8.55: Force-Deformation Loop of Six Different Isolation Systems under Cape Mendocino Earthquake MCE Scaled

Under Imperial Valley earthquake (Fig. 8.56, 8.57) DBE scaled, lowest isolator in friction coefficients TFA succeeded to obtain the least force shear of 226 kN, values increased from TFA to TFF type, accompanied this increase direct increase in isolator displacements and as it seen from the Figure 8.56 it made greater loop means better energy dissipating. However, still the shear increase should be considered. Imperial Valley earthquake (Fig. 8.58) SLE scaled, TFA, TFB and TFC gave loop curvature shape but starting from TFD to TFF the Force-Deformation became more linear indicates on the applied force less than force required to make the pendulum moves on outer surfaces with higher friction coefficient for these types of isolators. Under Imperial Valley earthquake (Fig. 8.59) MCE scaled, loop sizes' increased from TFA to TFF accompanied with shear increase from 347 kN to 455 kN. Although all six types showed efficient response under Imperial Valley MCE scaled.

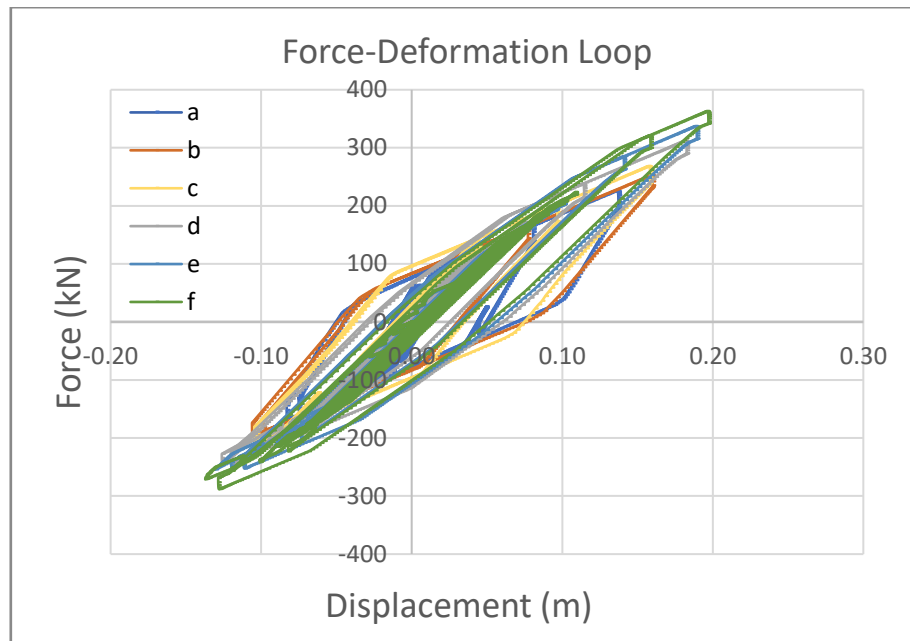


Fig. 8.56: Force-Deformation Loop of Six Different Isolation Systems under Imperial Valley Earthquake DBE Scaled

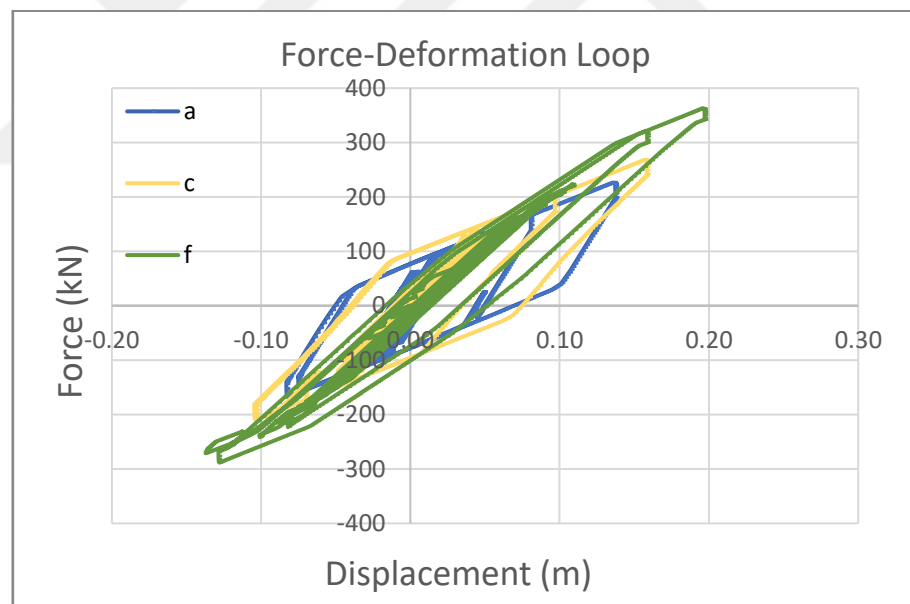


Fig. 8.57: Force-Deformation Loop of Three Isolation Systems only (Spotlight) Under Imperial Valley Earthquake DBE Scaled

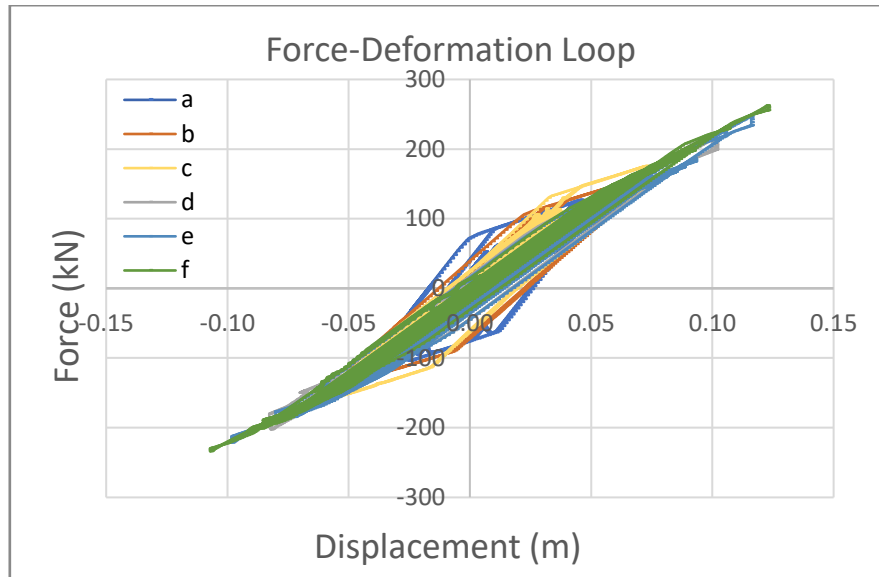


Fig. 8.58: Force-Deformation Loop of Six Different Isolation Systems under Imperial Valley Earthquake SLE Scaled

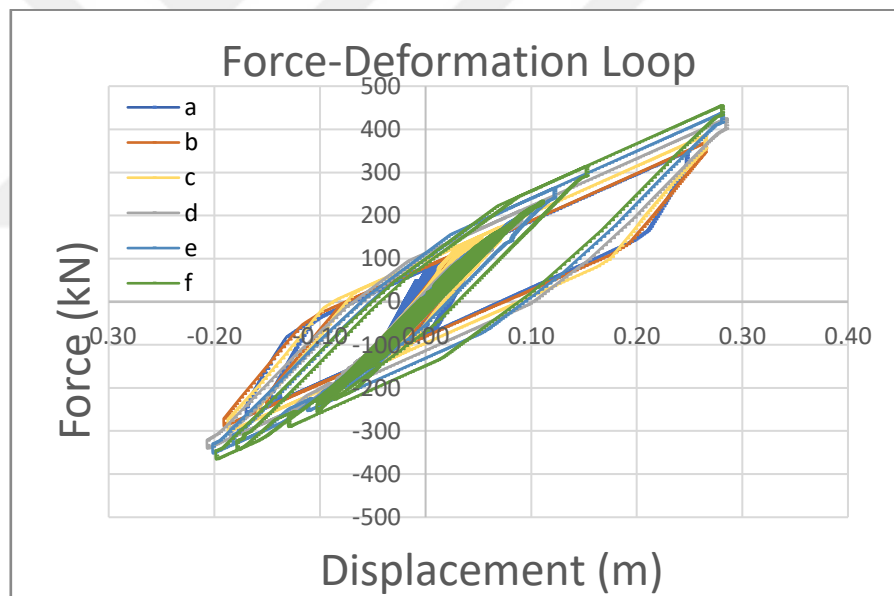


Fig. 8.59: Force-Deformation Loop of Six Different Isolation Systems under Imperial Valley Earthquake MCE Scaled

Under Northridge earthquake (Fig. 8.60, 8.61) DBE scaled. In terms of the loops size, the six isolators types gave almost the same loop size and expecting to dissipate close range of energy values and isolation periods. However, Types TFA, TFB and TFC managed to lower shear force compared to TFD, TFE and TFF. Under Northridge earthquake (Fig. 8.62) SLE scaled, similar responded to DBE were noticed but with proportionally smaller numbers. Under Northridge earthquake (Fig. 8.63) MCE scaled,

again similar responded to DBE were noticed but with proportionally higher numbers and the loops became closer to each other's.

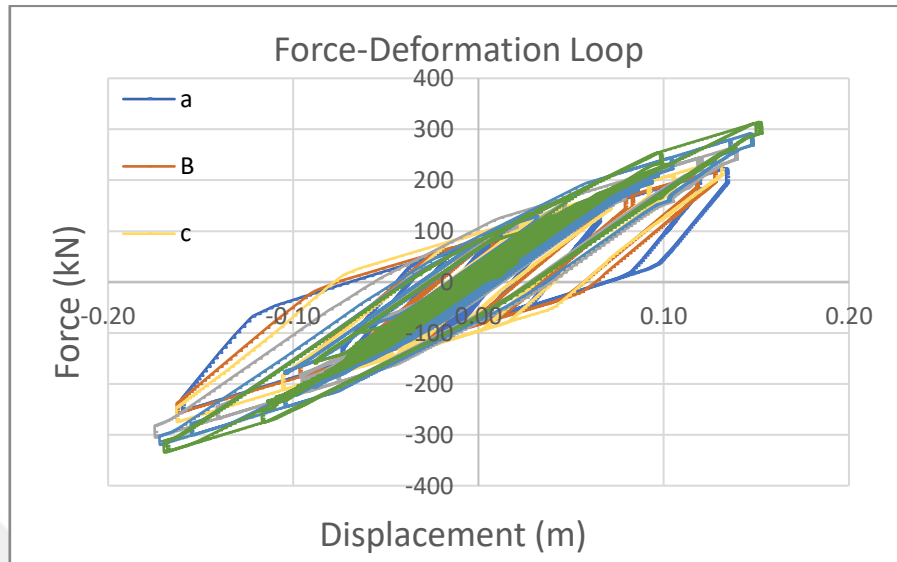


Fig. 8.60: Force-Deformation Loop of Six Different Isolation Systems under Northridge Earthquake DBE Scaled

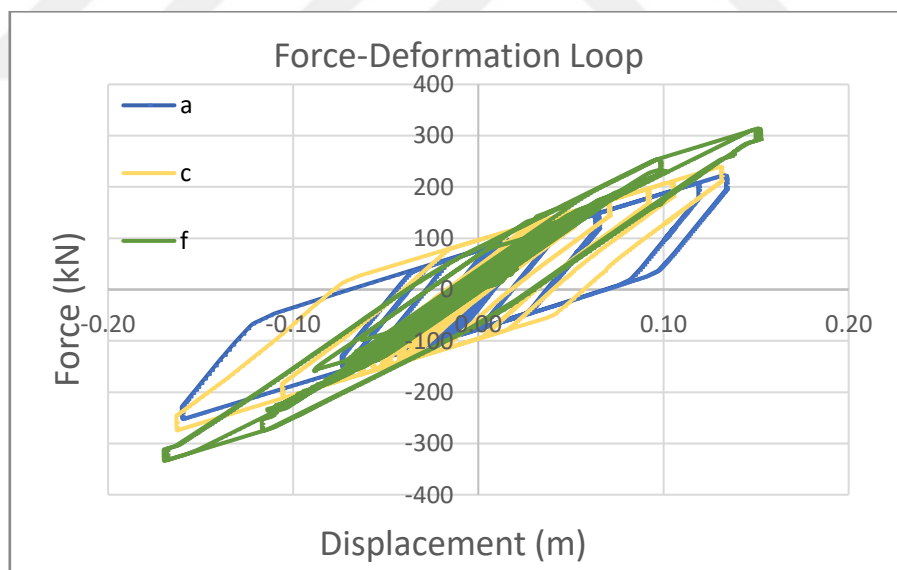


Fig. 8.61: Force-Deformation Loop of Three Isolation Systems only (Spotlight) Under Northridge Earthquake DBE Scaled

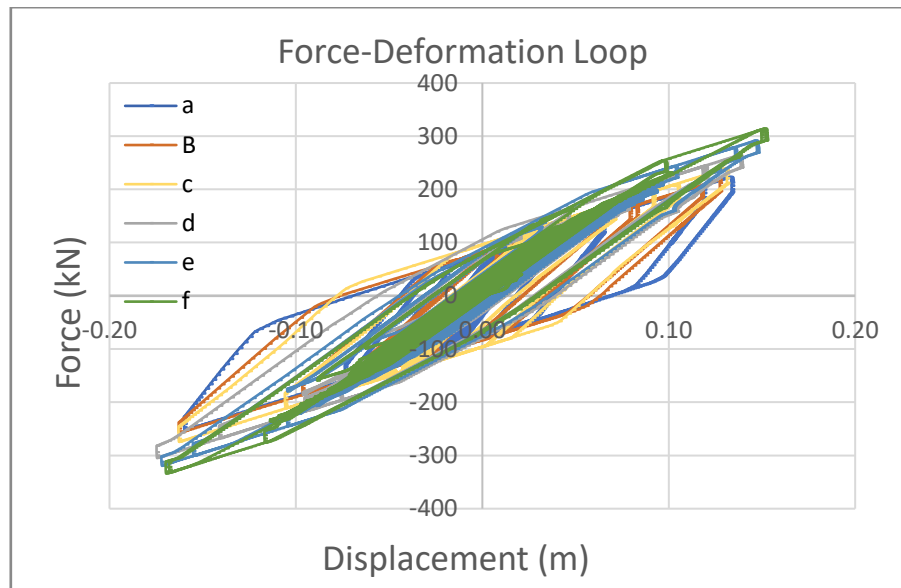


Fig. 8.62: Force-Deformation Loop of Six Different Isolation Systems under Imperial Valley Earthquake SLE Scaled

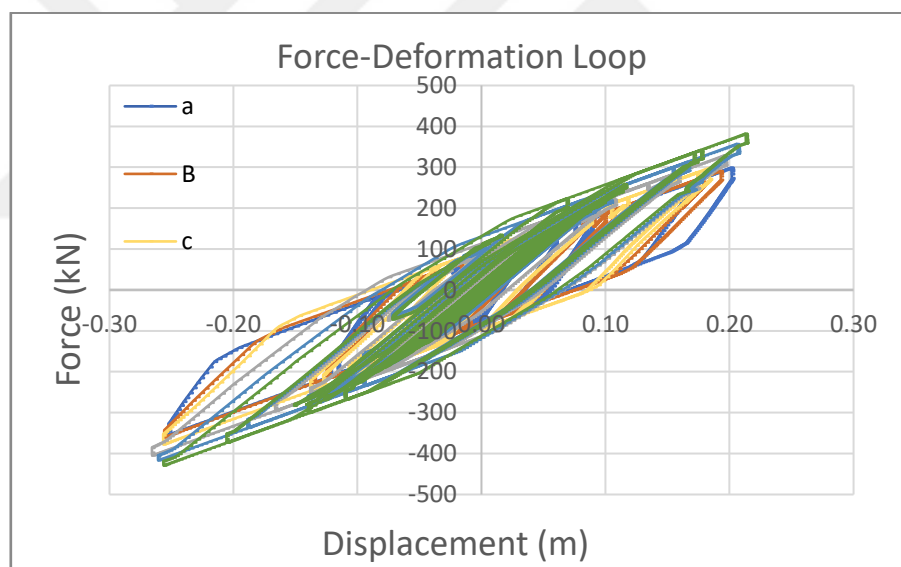


Fig. 8.63: Force-Deformation Loop of Six Different Isolation Systems under Imperial Valley Earthquake MCE Scaled

9. Conclusions:

In the following is the summary of conclusions inferred from this study:

- 1) For Isolator Displacement under near fault earthquake records, no certain or repeated response from the six isolator systems was noticed to be genericized to all of them. However, for far field earthquake records tested in this study, it was observed that isolators that have more difference between the upper surface's friction coefficients and inner surface's friction coefficients they showed more isolator displacements. This increase behavior in Isolator Displacement from TFA to TFF was clearer in SLE scaled records and became less obvious with increasing the scale of the earthquake records where at some MCE scaled records differences between the Isolator Displacements obtained from the six different systems became smaller relatively.
- 2) Maximum Inter-Storey Drift Ratio For both of near fault and all far Field records tested in this study, 90% of the these cases, it was observed that isolators with less difference between the outer surface's friction coefficients and inner surface's friction coefficients (TFA, TFB and TFC) showed less Inter-Storey Drift Ratios compared to (TFD, TFE and TFF)
- 3) For Maximum Base Shear for near fault and far field SLE scaled, the base shear obtained increased form TFA to TFF isolator systems. However, for DBE and MCE levels it was the same behavior for most cases except for some cases like Cape Mendocino (one of the near fault records) it showed higher values when (TFA, TFB and TFC) were used.
- 4) Maximum Roof Drift Ratio, under both near fault and far field records, 80% of tested earthquakes of different levels showed increase in Maximum Roof Drift Ratio from TFA to TFF. The rest showed either proportionally close values under the six different isolators or sudden increase like it happened under Duzce earthquake MCE scaled where Maximum Roof Drift Ratio increased over 1% from TFD to TFE.

- 5) Maximum Absolute Acceleration, it was difficult to find certain behavior to generalize it. Under near fault and far field as well scaled to different levels, different response for each analysis was observed.
- 6) Force Deformation Hysteric Loop, for both near fault and far field records scaled to different levels in this study, it was observed that the bigger difference between inner surface's friction coefficient and outer surface's friction coefficient $\mu_1-\mu_2$ (TFF as an example) Force Deformation Hysteric Loop became more edgy and pointy compared to these isolators with small $\mu_1-\mu_2$ like TFA. Theoretically this is explained due to the increase of $(\mu_1-\mu_2)$ leads to bigger u^* (Figure 2.3) that makes sequentially bigger shift when isolator about to repeat its stages of movement and the reason why it the loop became like a straight line as it happened under Imperial Valley SLE scaled record is due lack of enough force required for pendulum to slide on surfaces with high friction coefficient.
- 7) In general, results obtained under Far-Field Records were more predictable compared to results obtained from Near Fault records. For future research purposes, we suggest to focus on Far-Field Records and to select parameter other than Maximum Absolute Acceleration as it didn't show clear connection with changing Friction Coefficients. With more amount of data and analysis, optimum $\mu_1-\mu_2$, $\mu_4-\mu_3$ could be promising result to obtain. In addition, empirical equations to check the minimum and maximum limit of $\mu_1-\mu_2$ could be generated.

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11. Appendix A: Pre-Selection Design Check of TFP Bearings:

Table A. 1: Weight Ratios of Inner & Outer TFP Bearings

Triple Friction Bearings Weight and Positions				
Bearing	W (ton)	#	ΣW	Load share = $W / \Sigma W$
TFP Inner	71.7	2	143.4	0.327
TFP Outer	38	2	76	0.173
Σ		4	219.4 ton	

Geometric Properties:

$R_1=R_4=1000$ mm, $R_2=R_3=200$ mm

$h_1=h_4=40$ mm, $h_2=h_3=20$ mm $R_{1eff}=R_{eff4}=R_1-h_1=960$ mm

$R_{2eff}=R_{3eff}=R_2-h_2=180$ mm

Design Check for TFA Type (Inner):

Table A. 2: TFP Bearings Design Properties

μ	R_{eff1} (m)	R_{eff2} (m)	D_y	F_{d1}	W (ton)	NO. B.
0.030	0.96	0.18	0.0045	0.1919	71.1	4
			Σ	42.11	219.4	

1) Assume the displacement to be $D_D = 0.30$

2) Effective stiffness: $Q_d = \mu \cdot \Sigma W = 4.83$

$$K_D = \Sigma F_{D-Q} / D_D = 172.8$$

$$K_{eff} = K_D + Q_d / D_D = 188.84$$

For 1 bearing Share

1.57

56.45

61.71

3) Effective period: (Eq.17.5-2, ASCE 7-10)

$$T_{eff} = 2\pi \sqrt{\frac{\Sigma W}{(K_{eff})(g)}} = 2.16$$

4) Effective damping: (Eq.17.8-7, ASCE 7-10)

$$\beta_D = \frac{E}{2\pi K_{eff} D_D^2} = \frac{4\mu \Sigma W (D_D - D_y)}{2\pi K_{eff} D_D^2} = 0.054$$

5) Damping reduction factor:

$$\beta = \left(\frac{\beta_{eff}}{0.05} \right)^{0.3} = 1.022$$

6) Damping reduction factor:

$$D'_D = \frac{S_{D1} \cdot T_{eff}^2}{4\pi^2 \cdot \beta} g = 0.30$$

Design Check for TFB Type (Inner):

Table A. 3: TFP Bearings Design Properties

μ	R_{eff1} (m)	R_{eff2} (m)	D_y	F_{d1}	W (ton)	NO. B.
0.030	0.96	0.18	0.0045	0.1919	71.1	4
			Σ	42.11	219.4	

7) Assume the displacement to be $D_D = 0.23$

8) Effective stiffness: $Q_d = \mu \cdot \Sigma W = 6.58$

$$K_D = \Sigma F_{D-Q} / D_D = 183.1$$

$$K_{eff} = K_D + Q_d / D_D = 211.7$$

For 1 bearing Share

2.15

59.83

69.18

9) Effective period: (Eq.17.5-2, ASCE 7-10)

$$T_{eff} = 2\pi \sqrt{\frac{\Sigma W}{(K_{eff})(g)}} = 2.04$$

10) Effective damping: (Eq.17.8-7, ASCE 7-10)

$$\beta_D = \frac{E}{2\pi K_{eff} D_D^2} = \frac{4\mu \Sigma W (D_D - D_y)}{2\pi K_{eff} D_D^2} = 0.0844$$

11) Damping reduction factor:

$$\beta = \left(\frac{\beta_{eff}}{0.05}\right)^{0.3} = 1.170$$

12) Damping reduction factor:

$$D'_D = \frac{S_{D1} \cdot T_{eff}^2}{4\pi^2 \cdot \beta} g = 0.234$$

12. Appendix B: Extra Results and Figures Extracted from the Study:

Inter-Story Drift Ratio DBE Scaled:

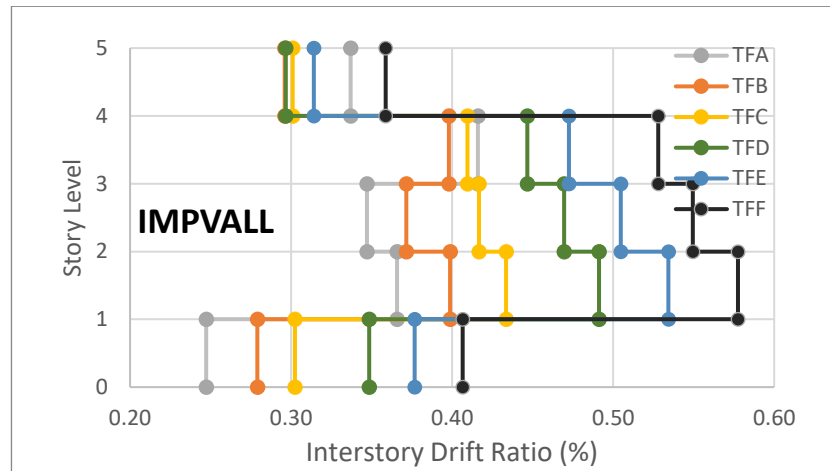


Fig. B. 1: Inter-Storey Drift Ratio Vs Storey Level Under Imperial Valley Earthquake DBE Scaled

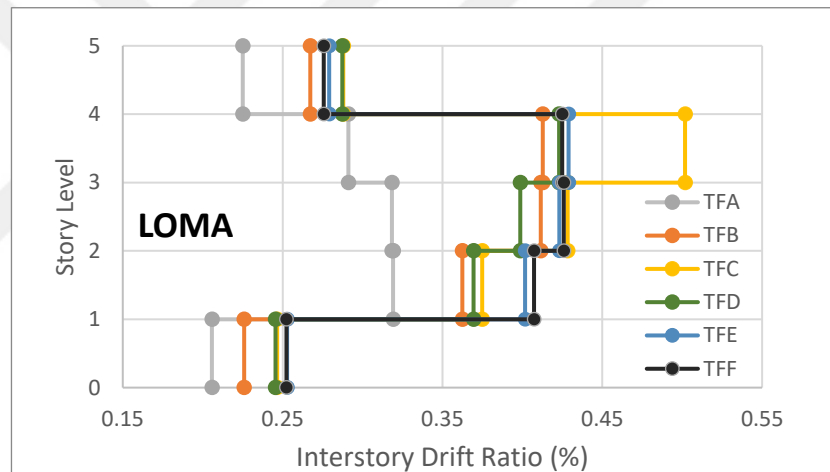


Fig. B. 2: Inter-Storey Drift Ratio Vs Storey Level Under Loma Prieta Earthquake DBE Scaled

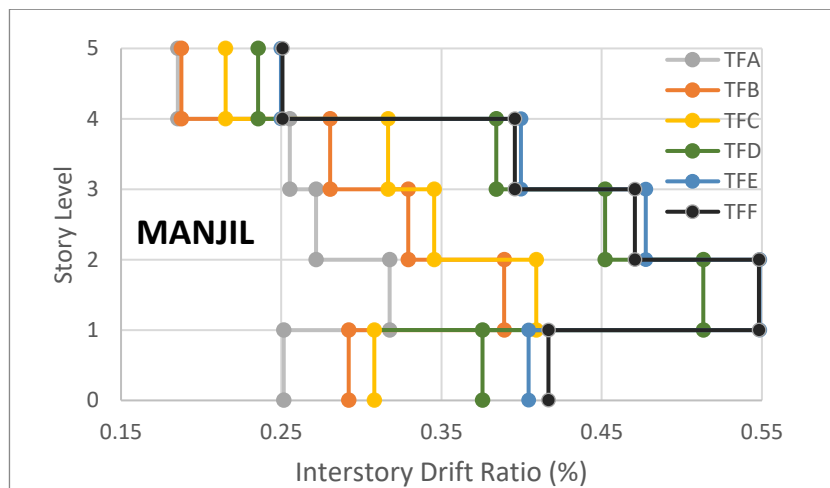


Fig. B. 3: Inter-Storey Drift Ratio Vs Storey Level Under Manjil Earthquake DBE Scaled

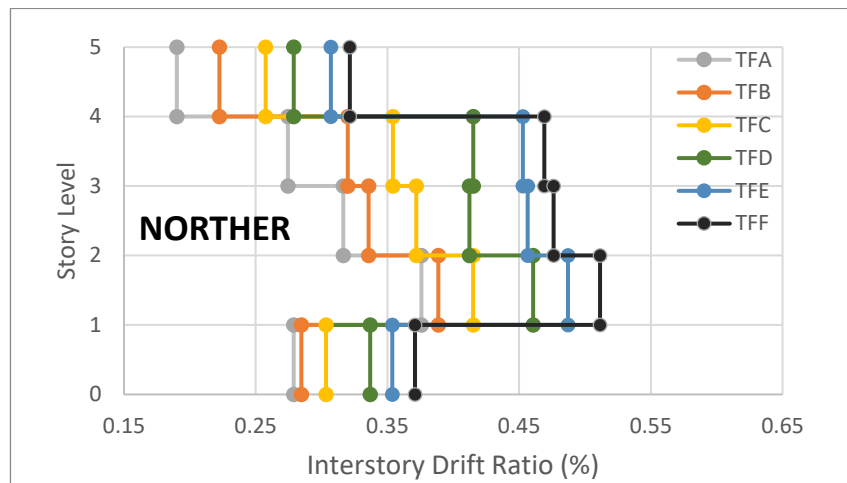


Fig. B. 4: Inter-Storey Drift Ratio Vs Storey Level Under Northridge Earthquake DBE Scaled

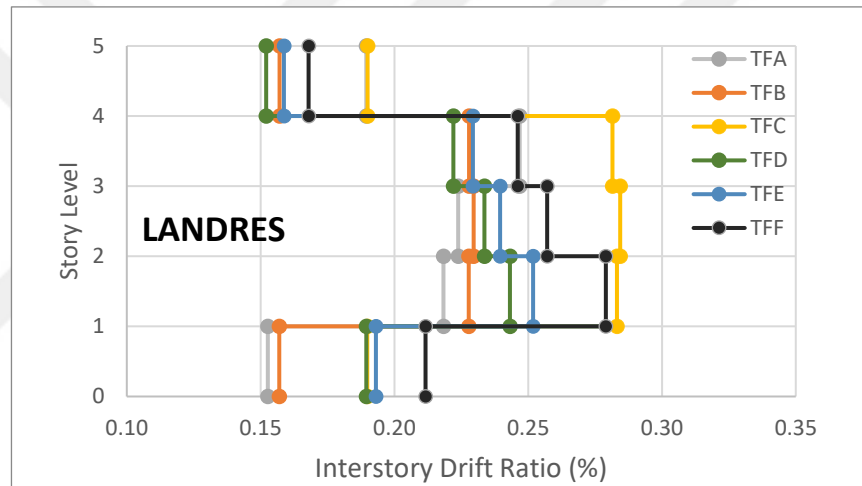


Fig. B. 5: Inter-Storey Drift Ratio Vs Storey Level Under Landres Earthquake DBE Scaled

Absolute Acceleration Vs Storey-Level DBE Scaled:

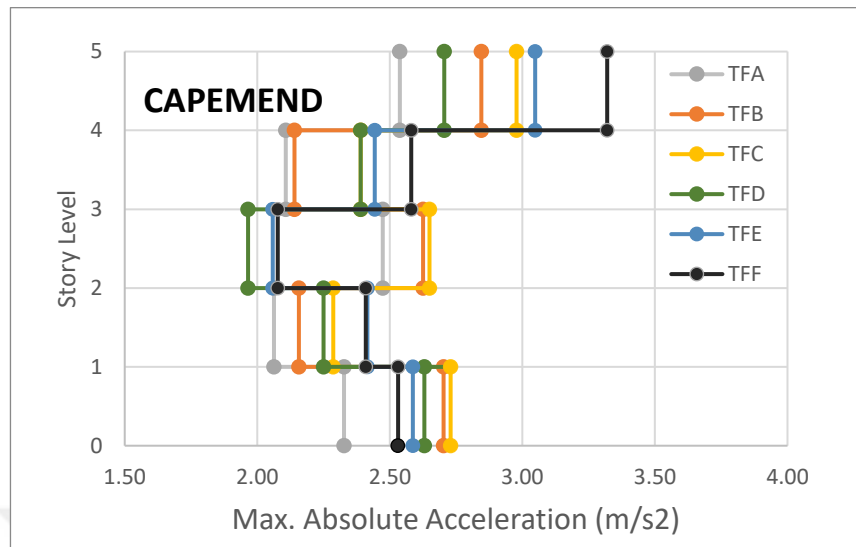


Fig. B. 6: Absolute Acceleration Against Storey Height Under Cape Mendocino DBE Scaled

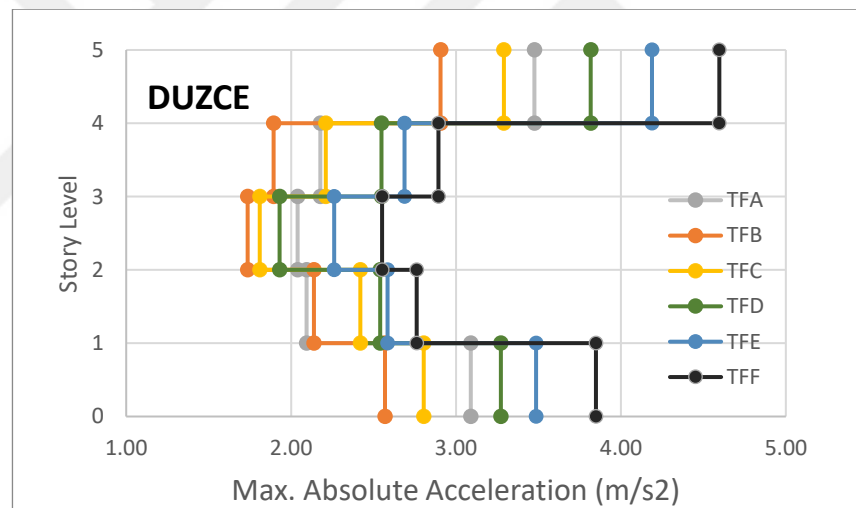


Fig. B. 7: Absolute Acceleration Against Storey Height Under Duzce DBE Scaled

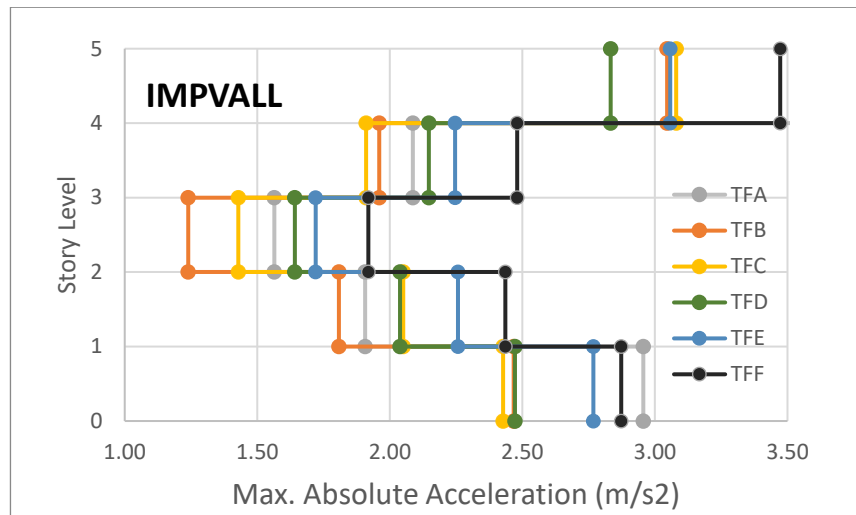


Fig. B. 8: Absolute Acceleration Against Storey Height Under Imperial Valley DBE Scaled

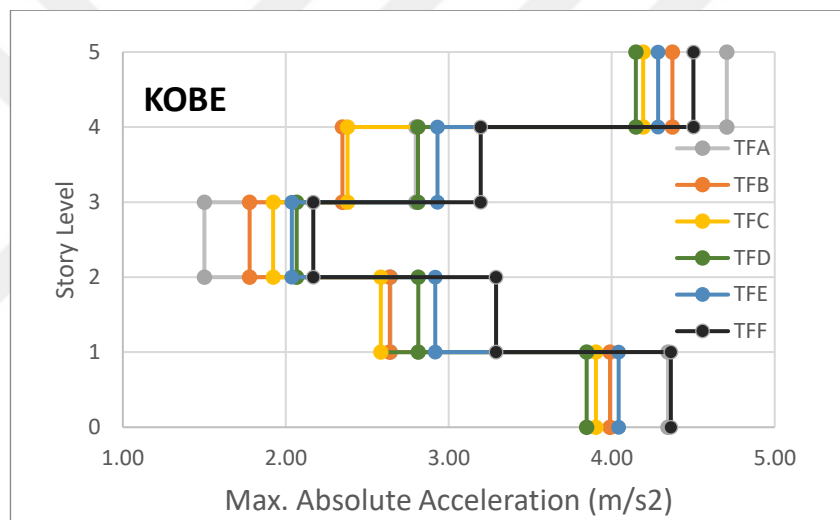


Fig. B. 9: Absolute Acceleration Against Storey Height Under Kobe DBE Scaled

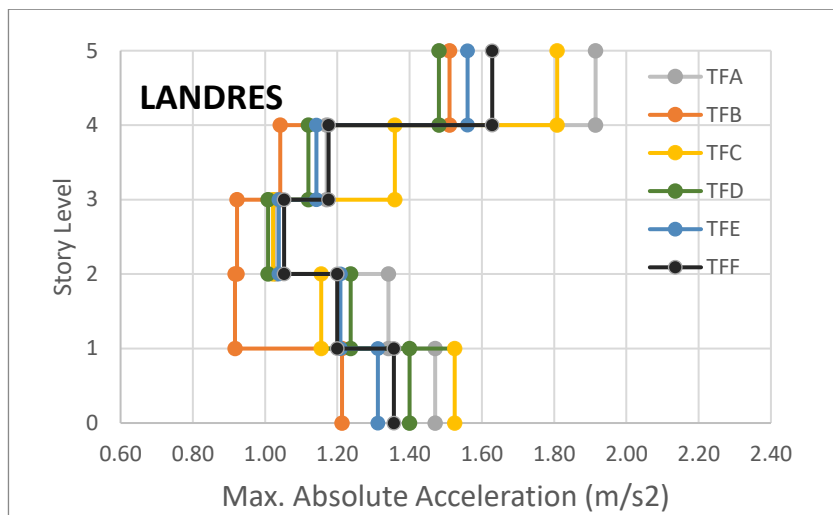


Fig. B. 10: Absolute Acceleration Against Storey Height Under Landres DBE Scaled

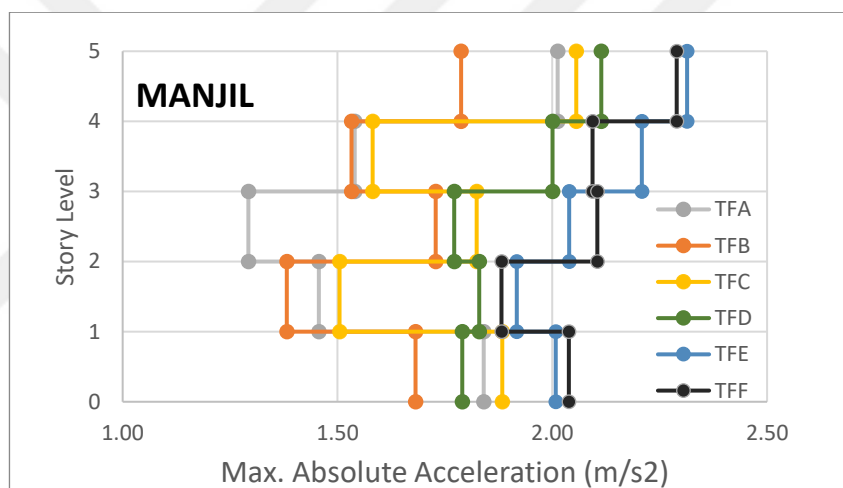


Fig. B. 11: Absolute Acceleration Against Storey Height Under Manjil DBE Scaled

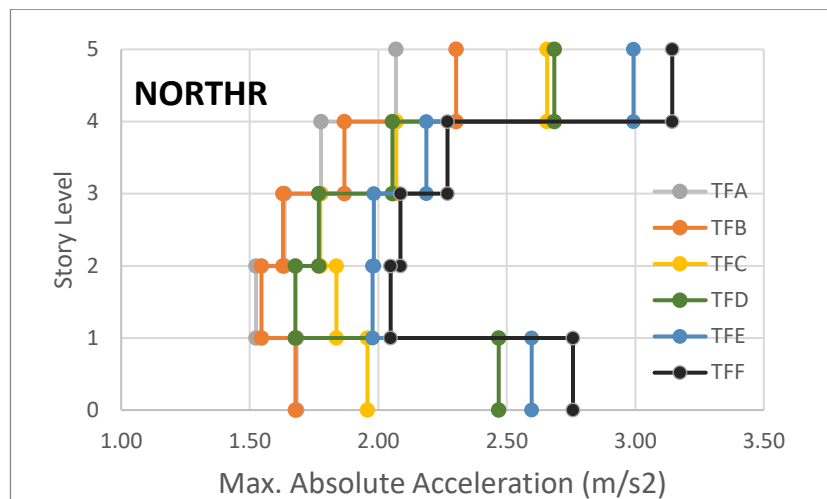


Fig. B. 12: Absolute Acceleration Against Storey Height Under Northridge DBE Scaled

Inter-Story Drift Ratio MCE Scaled:

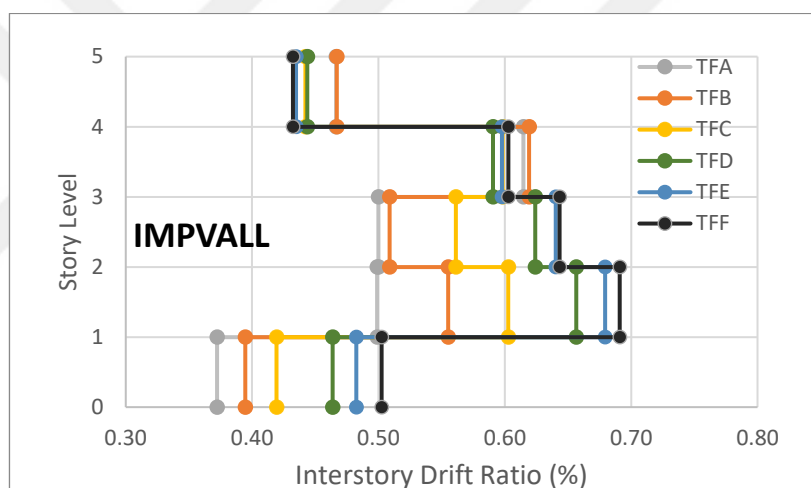


Fig. B. 13: Inter-Storey Drift Ratio Vs Storey Level Under Imperial Valley Earthquake MCE Scaled

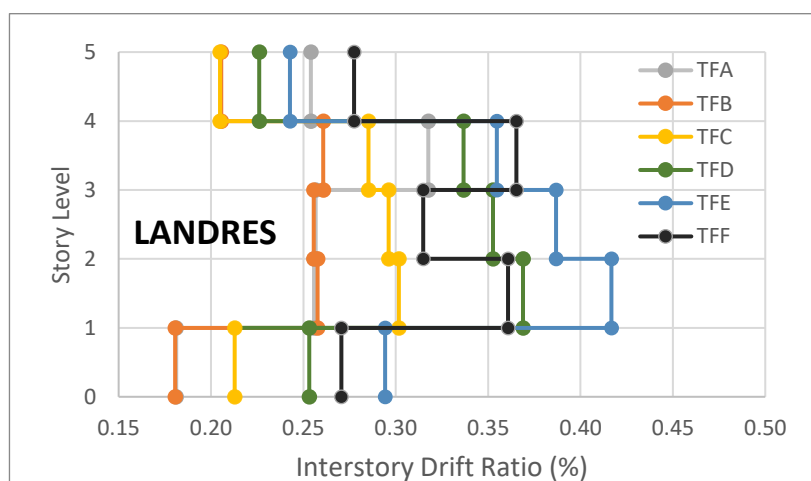


Fig. B. 14: Inter-Storey Drift Ratio Vs Storey Level Under Landres Earthquake MCE Scaled

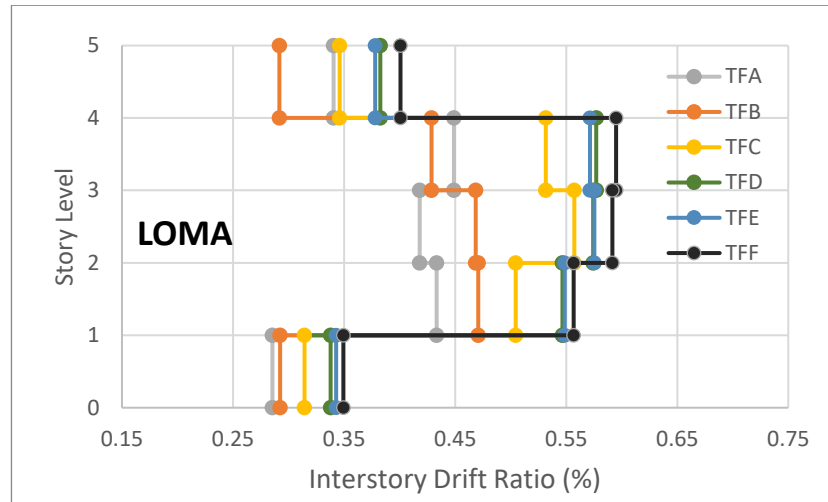


Fig. B. 15: Inter-Storey Drift Ratio Vs Storey Level Under Loma Prieta Earthquake MCE Scaled

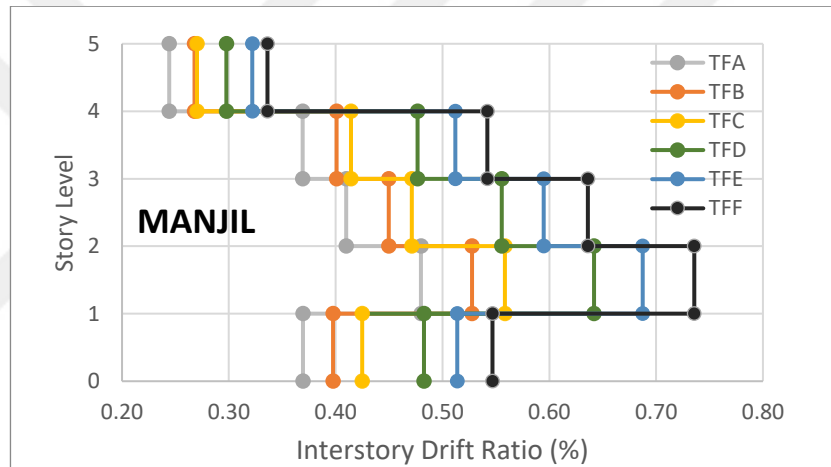


Fig. B. 16: Inter-Storey Drift Ratio Vs Storey Level Under Manjil Earthquake MCE Scaled

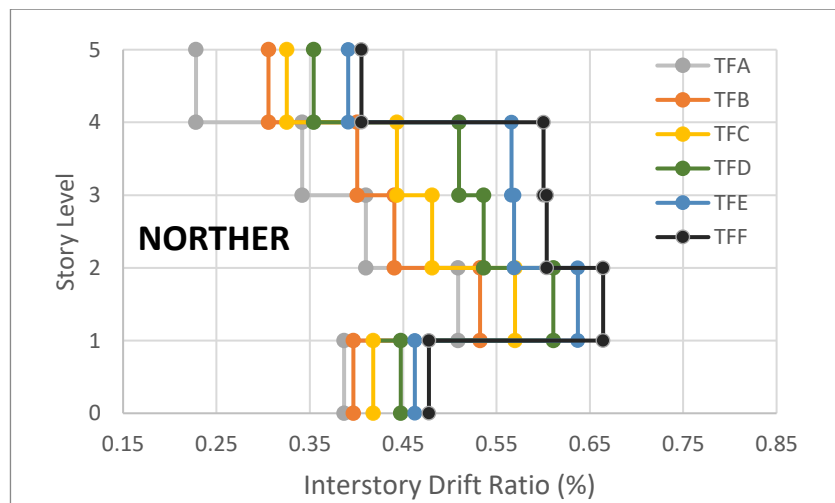


Fig. B. 17: Inter-Storey Drift Ratio Vs Storey Level Under Northridge Earthquake MCE Scaled

Absolute Acceleration Vs Storey-Level MCE Scaled:

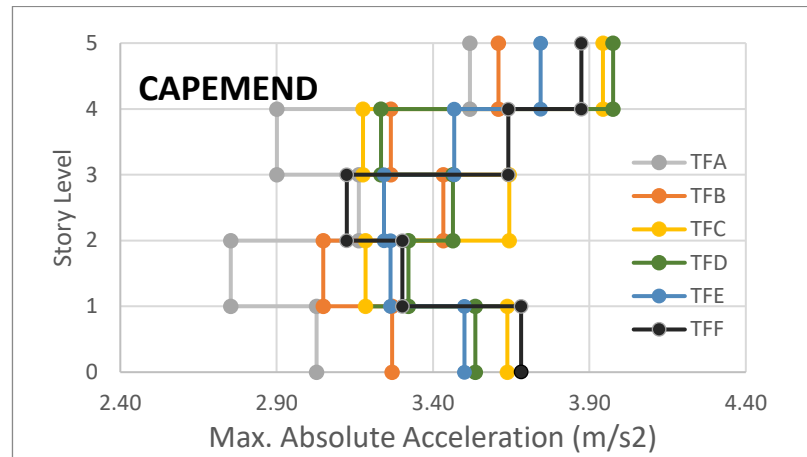


Fig. B. 18: Absolute Acceleration Against Storey Height Under Cape Mendocino MCE Scaled

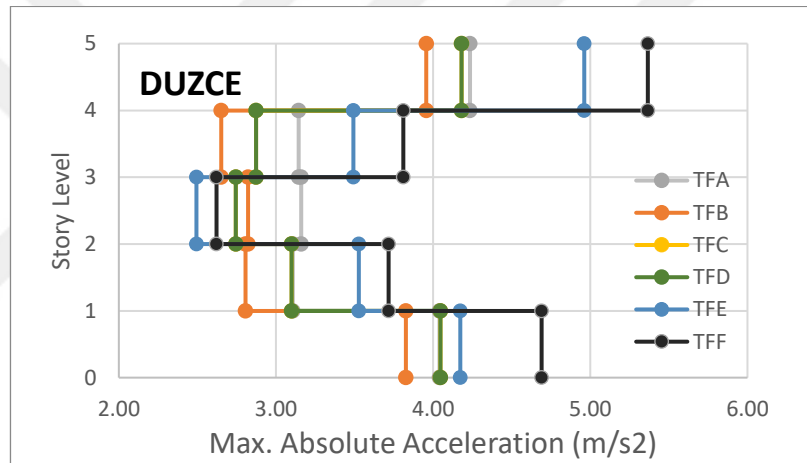


Fig. B. 19: Absolute Acceleration Against Storey Height Under Duzce MCE Scaled

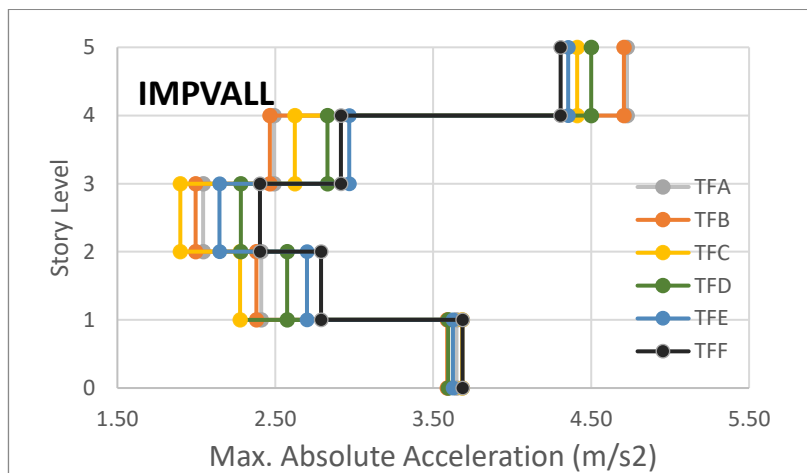


Fig. B. 20: Absolute Acceleration Against Storey Height Under Imperial Valley MCE Scaled

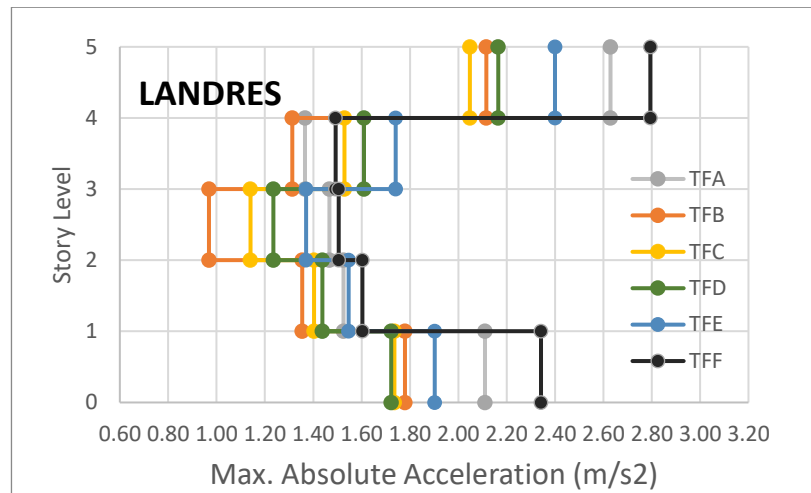


Fig. B. 21: Absolute Acceleration Against Storey Height Under Landres MCE Scaled

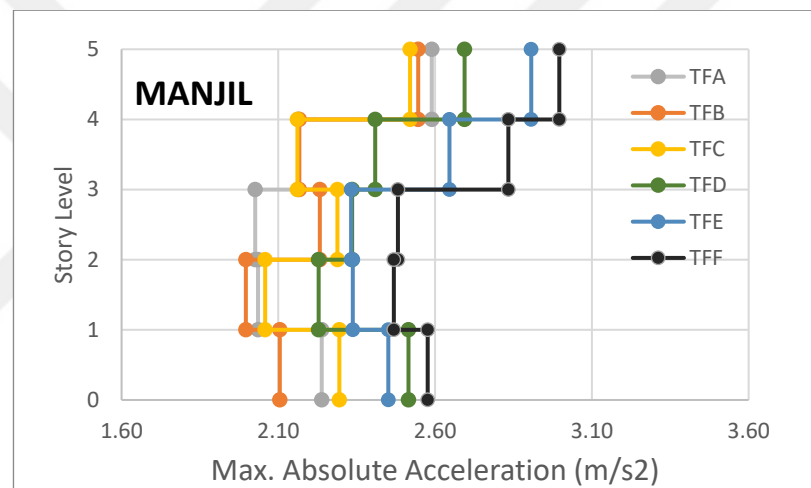


Fig. B. 22: Absolute Acceleration Against Storey Height Under Manjil MCE Scaled

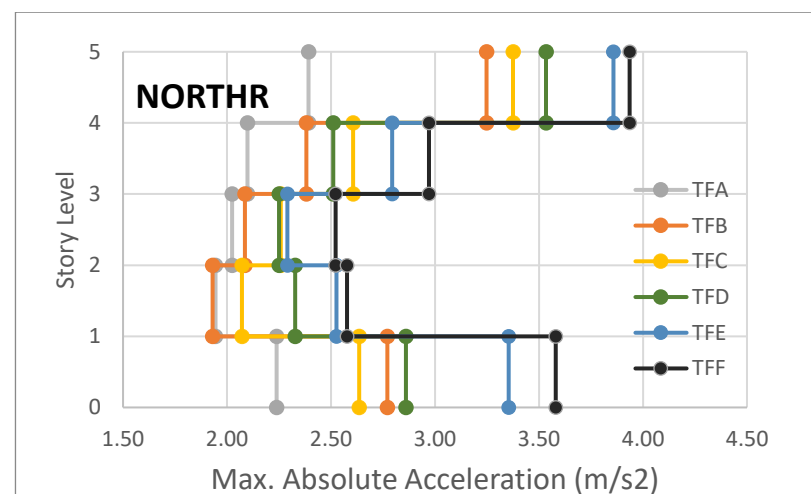


Fig. B. 23: Absolute Acceleration Against Storey Height Under Northridge MCE Scaled

Inter-Story Drift Ratio SLE Scaled:

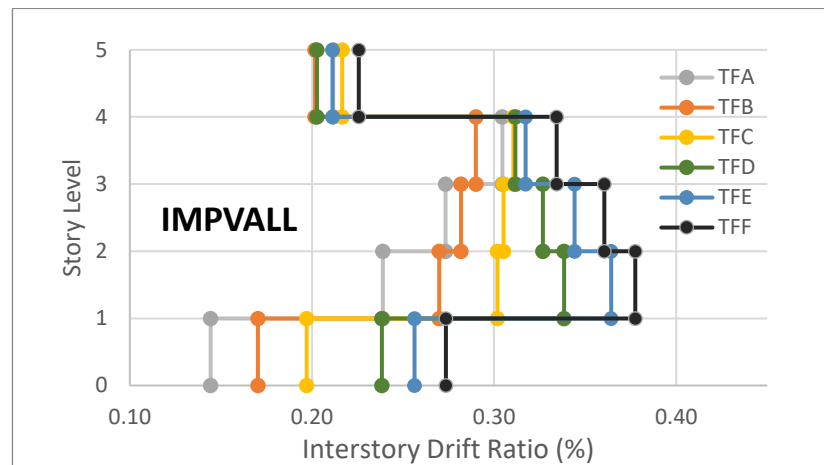


Fig. B. 24: Inter-Storey Drift Ratio Vs Storey Level Under Imperial Valley Earthquake SLE Scaled

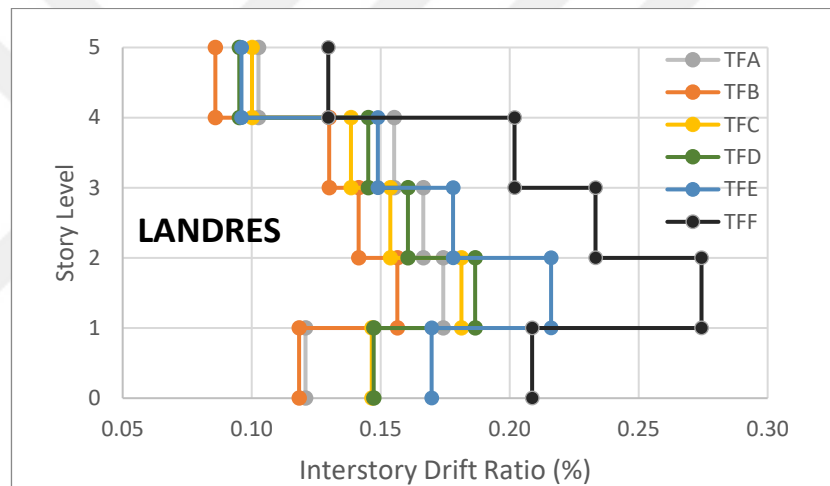


Fig. B. 25: Inter-Storey Drift Ratio Vs Storey Level Under Landres Earthquake SLE Scaled

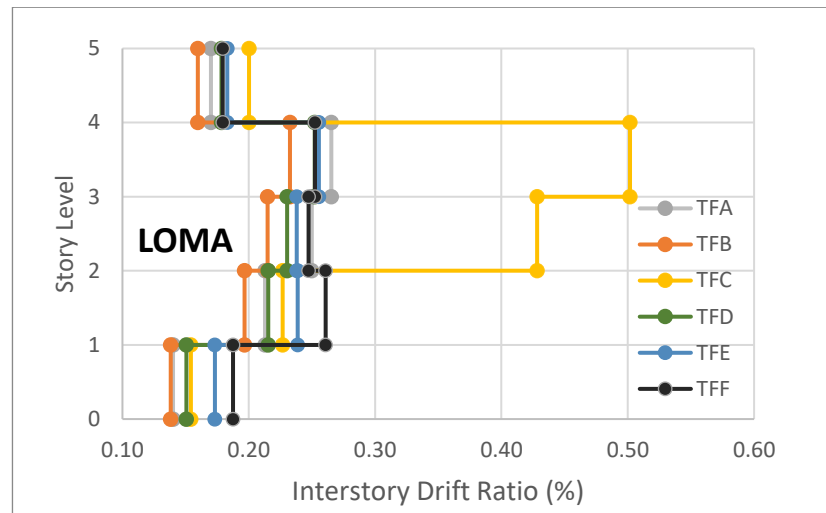


Fig. B. 26: Inter-Storey Drift Ratio Vs Storey Level Under Loma Prieta Earthquake SLE Scaled

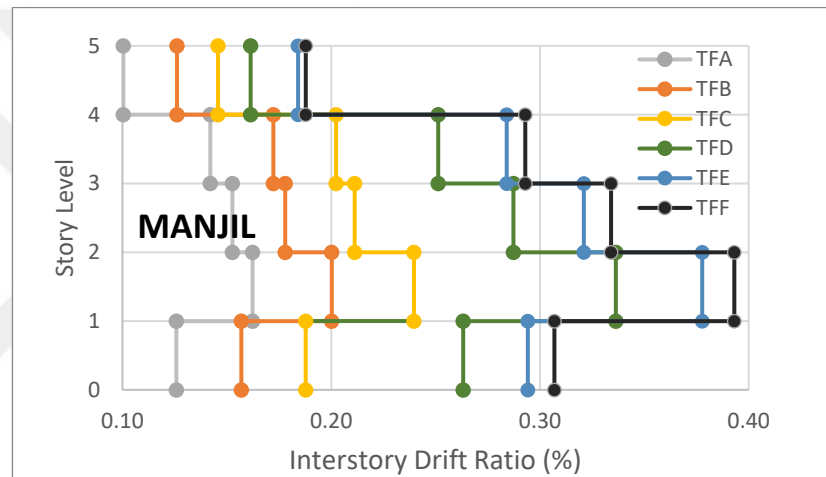


Fig. B. 27: Inter-Storey Drift Ratio Vs Storey Level Under Manjil Earthquake SLE Scaled

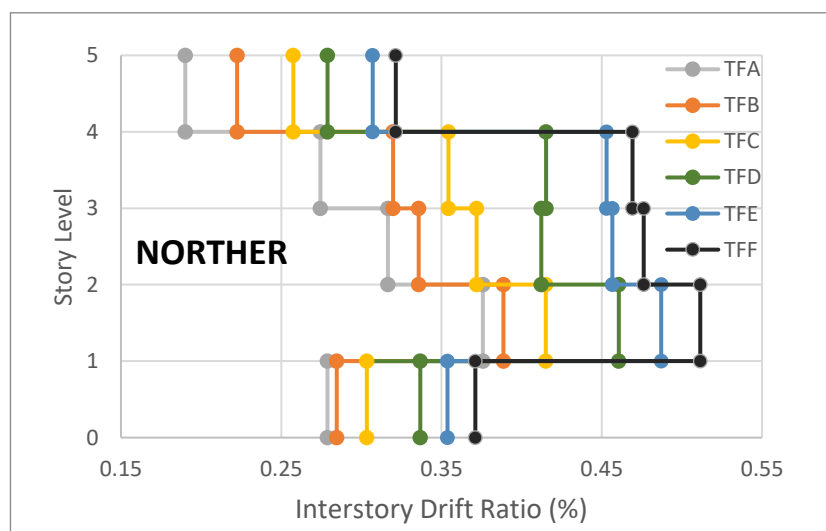


Fig. B. 28: Inter-Storey Drift Ratio Vs Storey Level Under Northridge Earthquake SLE Scaled

Absolute Acceleration Vs Storey-Level SLE Scaled:

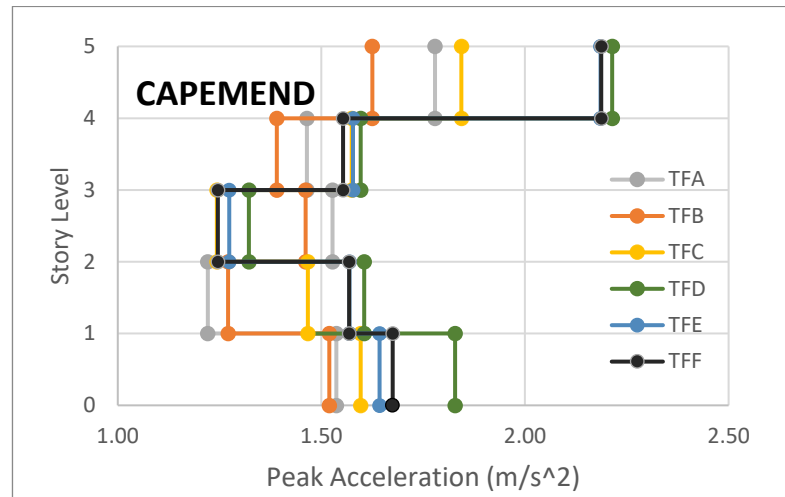


Fig. B. 29: Absolute Acceleration Against Storey Height Under Cape Mendocino SLE Scaled

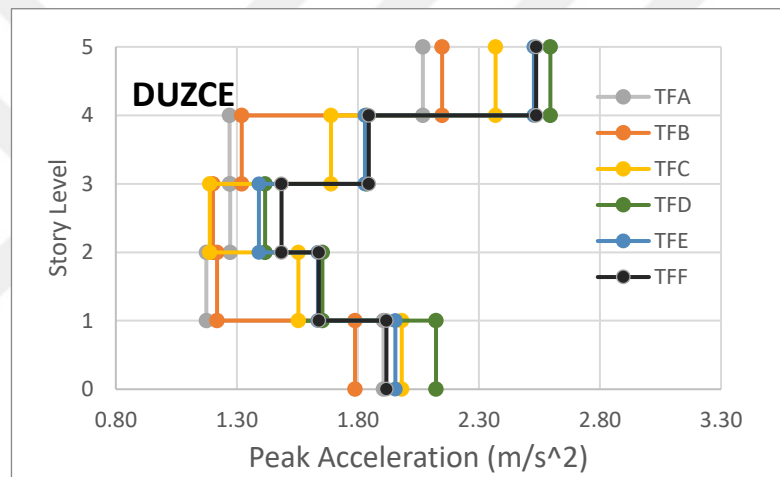


Fig. B. 30: Absolute Acceleration Against Storey Height Under Duzce SLE Scaled

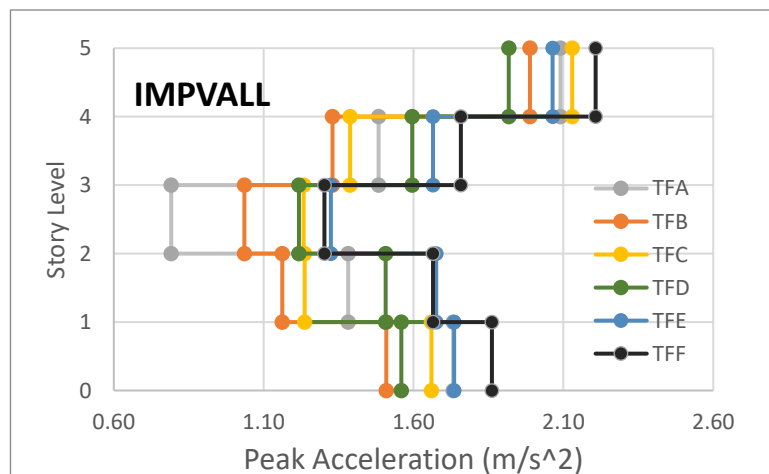


Fig. B. 31: Absolute Acceleration Against Storey Height Under Imperial Valley SLE Scaled

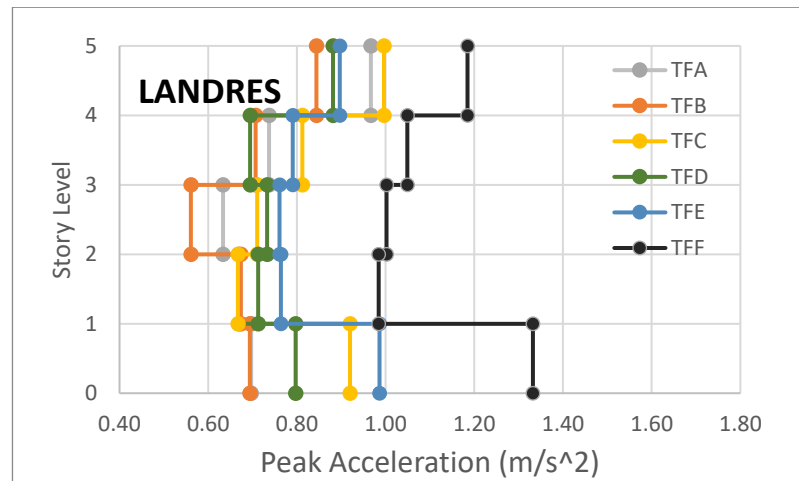


Fig. B. 32: Absolute Acceleration Against Storey Height Under Landres SLE Scaled

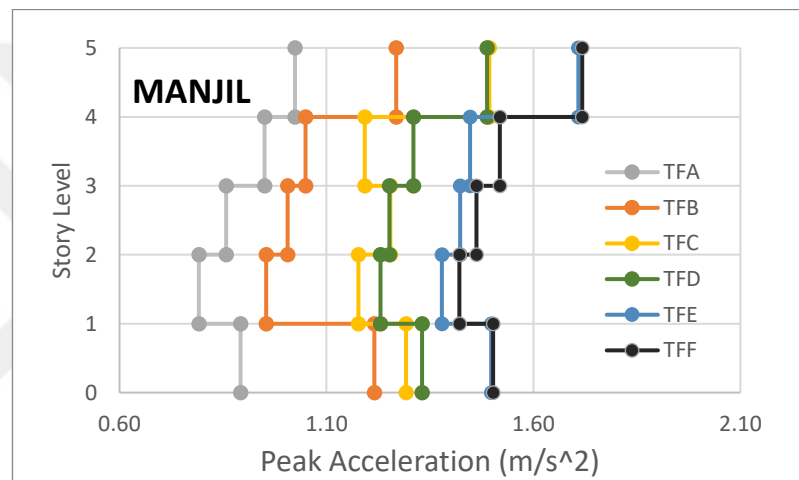


Fig. B. 33: Absolute Acceleration Against Storey Height Under Manjil SLE Scaled

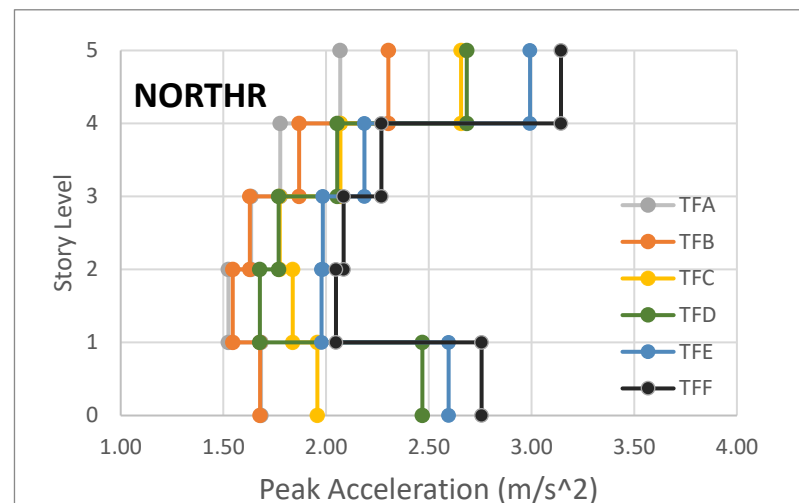


Fig. B. 34: Absolute Acceleration Against Storey Height Under Northridge SLE Scaled