SEPTEMBER 2021 M.Sc. in Civil Engineering SAMI BENNI

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OPTIMUM USAGE OF MIXED DAMPER SYSTEMS (RUBBER CONCERETE OR X DIAGONAL DAMPERS) ON MULTISTORY BUILDING.

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BY

SAMI BENNI

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Supervisor

Assoc. Prof. Dr. Nildem TAYŞİ

by

Sami BENNI

September 2021

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ABSTRACT

OPTIMUM USAGE OF MIXED DAMPER SYSTEMS (RUBBER CONCERETE OR X DIAGONAL DAMPERS) ON MULTISTORY BUILDING

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This research aims to compare usage of X plate chevron dampers (Wen Plasticity Dampers) in a concrete multi-story framed building which installed in different locations of the structure, in accompany of using rubberized concrete - treated crumped rubber added to concrete aggregates - using different rubber percentages for the building's members. Comparison of Nonlinear Push Over Analysis of models' results were done, such as transmitted base shear forces, roof displacement, pseudo acceleration, pseudo displacement, effective period, ductility ratio and most importantly effective damping ratio which plays major role in reducing demand curve of the design response spectrum. Two and three-dimensional multi story building models had analyzed by SAP2000 and ETABS, compared analysis results according to multiple study cases will be shown, trying to reach optimum usage of such damping systems.

Keywords: Damping Systems, Nonlinear Static Analysis (Pushover), Response Spectrum, Crumbed Rubber Concrete, Chevron Braces.

ÖZET

ÇOK KATLI YAPILARDA KARIŞIK SÖNÜMLEME SİSTİMLERİNİN OPTIMUM KULLANIMI (ESNEK BETON VEYA X DİYAGONAL SÖNÜMLEYİCİLER)

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Bu araştırma, çok katlı binalarda X plakalı diyagonal sönümleyicilerin (Wen Plastiklik Damperler) farklı yerlerde kullanımı ile yapı elemanlarında esnek beton kullanımını karşılaştırmayı amaçlamaktadır. Bu iki sistemin doğrusal olmayan itme analizi karşılaştırılması, iletilen taban kesme kuvvetleri, performans eğrisinin çatı yer değiştirmesi, sözde ivme, sözde yer değiştirme, efektif periyot, süneklik oranı ve en önemlisi tasarım tepki spektrumunun talep eğrisinin azaltılmasında ana rol oynayan efektif sönüm oranı ile yapılacaktır. İki ve üç boyutlu çok katlı bina modelleri, SAP 2000 ve ETABS programları ile analiz edilecek, analiz sonuçları çeşitli durumlara göre karşılaştırıp yukarıda belirtilen çıktılara göre bu tür sönümleme sistemlerinin optimum kullanımına ulaşılmaya çalışılacaktır.

Anahtar Kelimeler: Sönümleme Sistemleri, Doğrusal Olmayan İtme Analizi, Sismik Tepki Spektrumu, Esnek Beton, X Diyagonal Sönümleyiciler.

"First and foremost, thanks to God who helped me to complete this research, and a special dedication to my mother, father, brothers, family, and everyone that I love"

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I would like to thank my supervisor, Assoc. Prof. Dr. Nildem TAYŞİ for her guidance and support throughout the study and taking into account the tough conditions I have been through. I am thankful for her encouragement and motivation.

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LIST OF SYMBOLS

ϕ	Mode Shape
α_1	Modal Mass Coefficient for the First Natural Mode
π	Pi
K	Stiffness Matrix
C	Proportional Damping Matrix
M	Diagonal Mass Matrix
u	Relative Displacement
$\dot{u}$	Relative Velocity
$\ddot{u}$	Relative Acceleration
m_x	X Units Direction Acceleration Loads
m_y	Y Units Direction Acceleration Loads
m_z	Z Units Direction Acceleration Loads
$\ddot{u}_{gx}$	Uniform Ground Acceleration in X Axes
$\ddot{u}_{gy}$	Uniform Ground Acceleration in Y Axes
$\ddot{u}_{gz}$	Uniform Ground Acceleration in Z Axes
n	Number of Plates in Metallic Damper
K_d	Initial Elastic Stiffness of Metallic Damper
F_y	The Yield Load of Metallic Damper Material
q	The Yield Displacement of the XPD
E	Elastic Modulus
σ_y	Yield Stress of Damper Material
a	Hight of XPD
b	Width of XPD
t	Thickness of XPD
P	Plastic Force in XPD
d	Displacement caused by plastic force in XPD
H	Strain Hardening of XPD's Material

σ_u	Failure Stress of Damper Material
r	Material Modal Damping Ratio
z	Internal Hysteretic Variable
PF_1	First Natural Modal Participation Factor
w_i	Mass assigned to level i
ϕ_{i1}	Amplitude of Mode1 to level i
N	Uppermost Level in main portion of the structure
g	Gravity
V	Base Shear
W	Building Dead Weight
Δ_{roof}	Roof Displacement
S_a	Spectral Acceleration
S_d	Spectral Displacement
β_{eq}	Effective Damping Ratio
β_0	Hysteretic Damping
E_D	Energy Dissipated by Damping
E_{S_0}	Maximum Strain Energy
SR_A	Spectrum Acceleration Reduction Factor
SR_V	Spectrum Velocity Reduction Factor
S_A	Soil Profile Type A
S_B	Soil Profile Type B
S_C	Soil Profile Type C
S_D	Soil Profile Type D
S_E	Soil Profile Type E
S_F	Soil Profile Type F
N_A, N_v	Near Source Factors
C_A	Effective Peak Acceleration (EPA) Coefficient
C_V	Acceleration Response of Velocity Domain Coefficient
T_{eff}	Effective Period

LIST OF ABBREVIATIONS

ADAS	Added Damping and Stiffness
ADRS	Acceleration-Displacement Response Spectra
ASCE	American Society for Civil Engineering
ASTM	American Society for Testing and Materials
ATC	Applied Technology Council
CSM	Capacity Spectrum Method
DE	Design Earthquake
EPA	Effective Peak Acceleration
IO	Immediate Occupancy
LS	Life Safety
ME	Maximum Earthquake
NC	Not Considered
PGA	Peak Ground Acceleration
PGD	Peak Ground Displacement
PGV	Peak Ground Velocity
PI	Plasticity Index
PO	Performance Objective
psf	Pound Square Feet
RMS	Root-Mean-Square
SE	Serviceability Earthquake
SR	Slip Rate
SS	Structural Stability
TCR	Treated Crumb Rubber
UPV	Ultrasonic Pulse Velocity
WMC	Water Moisture Content
XPD	X Plates Dampers

CHAPTER I

INTRODUCTION

1.1 General introduction

When earthquake occurs ground vibrations can intensely cause damage to structures and equipment existed in them. The transmitted acceleration, velocity, and displacement into a structure, are amplified. This amplified motion usually make forces and displacements, which can exceed those the structure can sustain. Various factors affect ground movement and the amplification; therefore, an understanding of how these factors affect the structures' reaction and equipment is important for a secure and economical design.

Earthquake ground movement is typically measured by movement instruments, which measures the acceleration of the bottom. The recorded accelerograms, after correction for instrument errors and baseline, are necessary to get displacements and velocity time-histories. The outmost values of ground movement Peak Ground Acceleration (PGA), Peak Ground Velocity (PGV) and Peak Ground Displacement (PGD) are of particular interest in seismic analysis and style. [7]

1.2 Resistance and performance

There are two essential parts for seismic performance objective which are - A level of hazard and a damage state -. Life Safety (LS)and Immediate Occupancy (IO) give the descriptions of injury situation which doesn't comprise performance objectives until they are chosen to be at selected stage of seismic hazard. This hazard might be an (earthquake) or a possibility of intense ground motion (the chance of exceeding in 50 years is almost 10%)

Relatively new analysis procedures help to describe the nonelastic behave of a given structural elements of the building, which might assess the particular behave of the building when a ground motion is selected. The analysis procedure tells a way to

identify which a part of the building will fail first. Because the load and displacement increase, other elements begin to yield and deform inelastically. The resulting graphic "curve" is representation of the capacity of building. Several alternative techniques allow the demand from a selected earthquake or intensity of ground shaking to be correlated with capacity curve to get some extent on the curve were capacity and demand are equal which called Performance Point (PP) of the structure which is an estimation of the exact building's displacement for the desired ground movement.
Levels of performance describe a condition of limiting damage which can be considered adequate for a specific building and a specific ground shaking. [8], however, by rapid development of science applications, structural analyses methods had been widely spread to adopt most needs in line with the performance and request behavior and investment of buildings.

1.3 Structural analysis

There are many sorts of structural analysis that describes the behavior of the structure against vertical and lateral external forces in both fields Static & Dynamic. Which those forces are being converted to internal forces.
Selecting which sort of structural analysis is required is said to several factors, which are the sort geometrical shape of structure, behavior of structure, materials, geometrical style of the placement of the structure and most significantly designer point of view. Here are the foremost common structural analyses used:

1.3.1 Modal analysis

Modal analysis is employed to see the vibration modes of a structure. These modes are useful to grasp the behavior of the structure. They will even be used because the basis for modal super position in response-spectrum and modal time-history load cases. The varieties of modal analysis are Eigenvector and Ritz-vector.

1.3.2 Response-Spectrum analysis

Response-spectrum analysis may be a statistical kind of analysis for the determination of the likely response of a structure to seismic loading.
The dynamic equilibrium equations -as sociated with the response of a structure to ground motion- are given by:

$$K\,u(t) + C\,\dot{u}(t) + M\,\ddot{u}(t) = m_x\ddot{u}_{gx}(t) + m_y\ddot{u}_{gy}(t) + m_z\ddot{u}_{gz}(t) \qquad (1.1)$$

where K is the stiffness matrix; C is the proportional damping matrix; M is the diagonal mass matrix; u, $\dot{u}$ and $\ddot{u}$ are the relative displacements, velocities, and accelerations with respected to the ground; m_x, m_y, and m_z are the unit Acceleration Loads; and $\ddot{u}_{gx}$, $\ddot{u}_{gy}$, and $\ddot{u}_{gz}$ are the components of uniform ground acceleration.

1.3.3 Time-History analysis

Time-history analysis is a step-by-step analysis of structure's dynamic response to a specific load which may vary according to time. This analysis could be bilinear or nonlinear.

Time- history analyzing is employed to work out the structure's dynamic response to capricious load cases. The dynamic equilibrium equations to be solved are given by:

$$K\,u(t) + C\,\dot{u}(t) + M\,\ddot{u}(t) = r(t) \tag{1.2}$$

Where M is the diagonal mass matrix; C is the damping matrix; K is the stiffness matrix; u, $\dot{u}$, and $\ddot{u}$ are the displacement, velocity, and acceleration values of the structure; and r represents load. When load includes ground acceleration, displacements, velocities, and accelerations are relative to the present ground movement.

1.3.4 Nonlinear Static analysis

Using of nonlinear static analysis may be for a large style of purposes, which includes: to research a structure's nonlinear geometric and nonlinearity of material; to create the P-delta stiffness of sub sequent linear analyses; to analyze incremental construction of time-dependent behave of material; to implement cable analysis and static push over analysis.

1.3.5 Frequency-Domain analysis

Frequency-domain type analysis relies on the structure's dynamic response to harmonical varying types load. Styles of frequency-do main load can be divided in to two categories cases which are available: probabilistic Power-Spectral-Density and deterministic Steady-State analyses. This analysis is predicated on the structure's dynamic response to harmonically varying load. The analysis can be done at one or more frequencies of vibration. For every frequency, load varies with time in the shape of sine and cosine functions. There are two varieties of frequency-domain load cases: power-spectral-density and steady-state analyses. [10]

Nonlinear Static Analysis (Pushover) will be the type of study case for the structural analysis used in this research

.

CHAPTER II

DAMPING SYSTEMS

2.1 Concept of damping

The structural system damping concept may have lots of meanings to the different disciplines of engineering. For civil engineer, damping means a reference note on a seismic or spectral diagram of wind, “5 % damped spectra” is usually the most jointly indicator. Damping in general means ant type of changes in stress as overall within a structure to the structural engineer, which is subjected to shocks and vibrations, with frequent pretexts whether a structure have “2 %, 3 %, 4 %, but not more than 5 %” structural damping.

It is obvious from previous descriptions that a damper is an element which can be added to a system to provide forces can be resistible against motion, thus providing a dissipation of energy. The most convenient and common functional output equation for a damper comes from classical systems theory, and is that of the so-called “linear” or “viscous” damping element:

$$F = C\dot{X} \tag{2.1}$$

Where F is resistive force from the damping element, C is the damping constant, $\dot{X}$ is end-to-end velocity across the element. [4]

2.2 Effects generalized by adding damping to a structure

Damping is considered as one among many alternative ways that are proposed to allow a structure attaining its optimal performance while it is subjected to seismic forces, windstorm, or any other styles of lateral vibration disturbances. Regular approach would instruct that the structure should passively attenuate the consequences of transient inputs within a mix of deformability, flexibility, strength, and energy absorption. Regular structures in usual can deform properly beyond their elasticity limits and stay intact to their ability to non-elastically deforms during strong movements, as earthquakes. According to that, large amount of the energy is absorbed by the structure through local damage. Adding dampers within a structure supposes that a number of energy inputs from transient loads to the structure are going to be dissipated, not only within the structure itself, but by added supporting damping members.

An idealized damping member would act as a form which producing force by itself that is of such a magnitude and may occur at a time which the damping member forces don't increase general stresses in the structure.

Nowadays structural protective systems are designed to be divided into three categories, which are passive energy dissipation, seismic isolation, and semi active and active systems. This research has considered passive dissipation of energy for metallic yielding dampers.[5]

2.3 Hysteretic damping modality

Hysteresis loops are considered as a succession of loops between force and displacement (resistance-deformation relationship). They are caused by successive loading and offloading on structures, during a severe earthquake striking a structure, the deformations which are beyond the elastic limits. These inelastic deformations depend on the earthquake's magnitude and the structure's load-deformation characteristics. Figure 2.1 shows a typical hysteretic damping loop.

Hysteresis loops are measuring the structure's capacity of energy dissipation. The orientation and form of the loop is determined by the structural yield displacement and stiffness. structural material, structural system and connectivity are factors that affect hysteretic behavior of structure. [5]

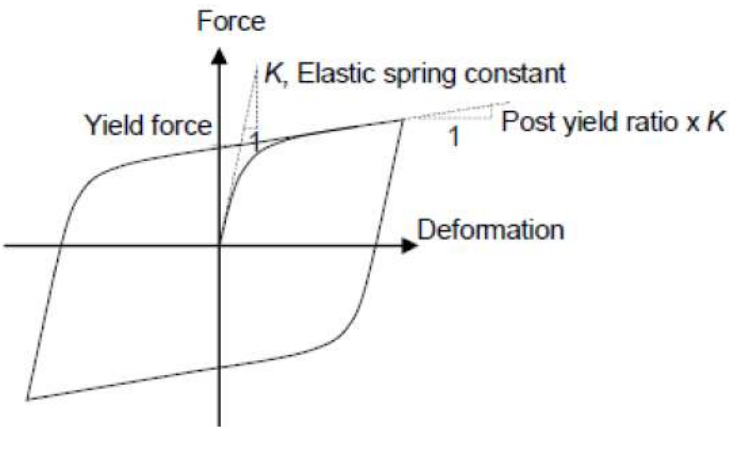

Figure 2.1 Hysteretic damping loop

2.4 Metallic yielding dampers

The metallic yielding dampers' mechanism used to dissipate a seismic activity' energy can be done by hysteretic behavior during metal elements' plastic deformation. This was proposed by Kelly in 1972. A yielding of metallic structures absorbs larger quantities of the vibration energy. Such dampers are placed at specified locations of the structure and not lodged to the rest of the structure, therefore replacement after an sever seismic forces is easy. These dampers must have reasonable characteristics like adequate elastic stiffness and strength, so the damper doesn't reach inelastic limits during service loads.

Selection of the parameters governing the behavior of the yielding devices improves the earthquake resistance of a structure and increases the effectiveness of the dampers themselves. The results investigations in numerical and analytical points of view have detected that the main parameters included within the dampers' design are the ratios of bracing stiffness to damper stiffness, brace-damper assembling stiffness to damper stiffness, and assembling stiffness of the relevant story. [5]

2.4.1 Added damping and stiffness (ADAS)

This device has several X-shaped plates. Shape of plates make sure that all plates yield over the total length of the damper. ADAS devices improv the main structure's behavior by stiffness increasing strength and dissipating energy's ability.

2.4.2 X-Plates metallic dampers

Such type of damping devices have high capability of bearing and huge initial stiffness. But experimental results have displayed that stresses are focused at the centers and the corners of the dampers and experiments have also shown that bending deformations are less than stresses deformations, as it illustrated in Figures 2.2 and 2.3.

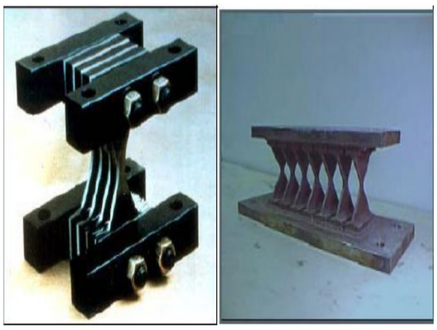

Figure 2.2 Samples of X plates dampers (ADAS)

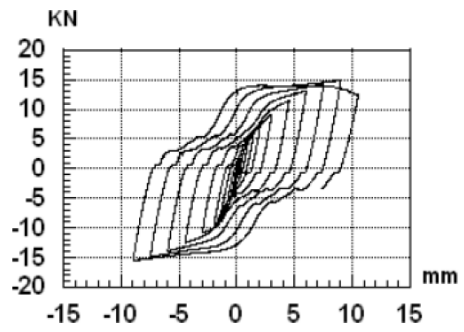

Figure 2.3 Example of hysterical damping loops of (ADAS)

X-plate dampers are usually made of multilayers of X-shaped steel plates, every plate have a double curvature arranged parallelly; the number of plates relays upon required quantity of energy which must be dissipated in the given system. Materials used to manufacture the X-plates can possibly be any kind of metal that allow huge deformations such as mild steel. Such dampers can be used with a suitable supporting mechanism to reduce the structure response by dissipating the input seismic energy, substantially in building structure combining of bracing and XPDs. When such system exposed to lateral forces as earthquakes, high winds, the seismic energy is dissipated through the flexural yielding deformation. They also can bear many cycles of fixed yielding deformation, resulting as a high level of energy dissipation. The target of using X- dampers is they have a constant strain variation over its height and ensures that yielding occur at the same time and uniformly through the full height of the damper. [6]. A typical XPD with holding device used in the present work as shown in Figure 2.4.

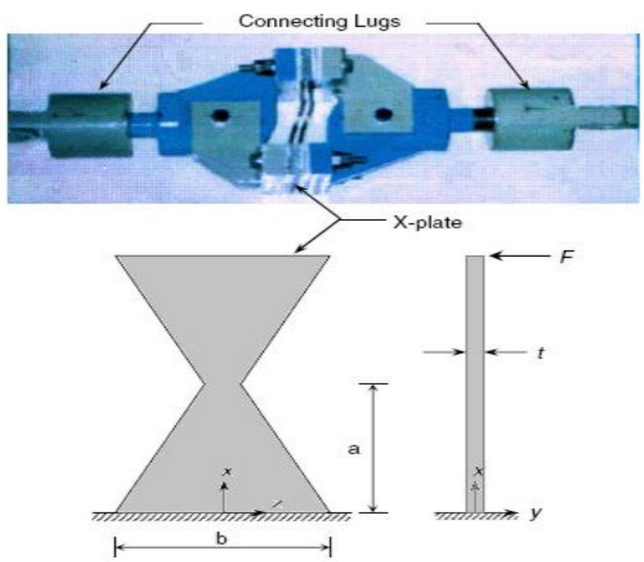

Figure 2.4 Sample of single X-plate experiment

Using beam theory, the properties of XPD are expressed as,

$$F_y = \frac{\sigma_y b t^2}{6a} n \tag{2.2}$$

$$q = \frac{2\sigma_y a^2}{Et} n \tag{2.3}$$

$$K_d = \frac{F_y}{q} \tag{2.4}$$

$$K_d = \frac{Ebt^3}{12a^3} n \tag{2.5}$$

$$P = \frac{\sigma_y b}{12Ea}\left\{\left((4{y_0}^2 - 3t^2)(H - E) + \frac{Ht^3}{y_0}\right)\right\} \tag{2.6}$$

$$y_0 = \frac{\sigma_y a^2}{Ed} \tag{2.7}$$

Where;

N is the number of plates

K_d is the initial stiffness,

F_y is the yield load and

q is the yield displacement of the XPD.

E and σ_y are elastic modulus and yield stress of the damper material, respectively.

a, b and t are height, width and thickness of the XPD.

P is the plastic force in XPD due to given displacement *d*

H is the rate of strain hardening and strain hardening and is the elastic depth given by this equation:

$$H = \begin{cases} \frac{\left(\sigma_u/\sigma_y\right)-1}{1.3-1} E & if\ \sigma_u/\sigma_y \le 1.3 \\ 0.003 + 0.007\left(\frac{\left(\sigma_u/\sigma_y\right)-1}{1.6-1.3}\right) E & if\ 1.3 < \sigma_u/\sigma_y \le 1.6 \\ 0.01E & Otherwise \end{cases} \tag{2.8}$$

[3]

σ_u is the failure stress of damping material.

2.5 Wen plasticity property (For X plates dampers, XPD)

For each deformational degree of freedom, it may specify independent uniaxial plasticity properties. The plasticity model is based on the hysteretic behavior proposed by Wen (1976). Figure 2.5 shows the nonlinear relationship between force and deformation for Wen plasticity property.

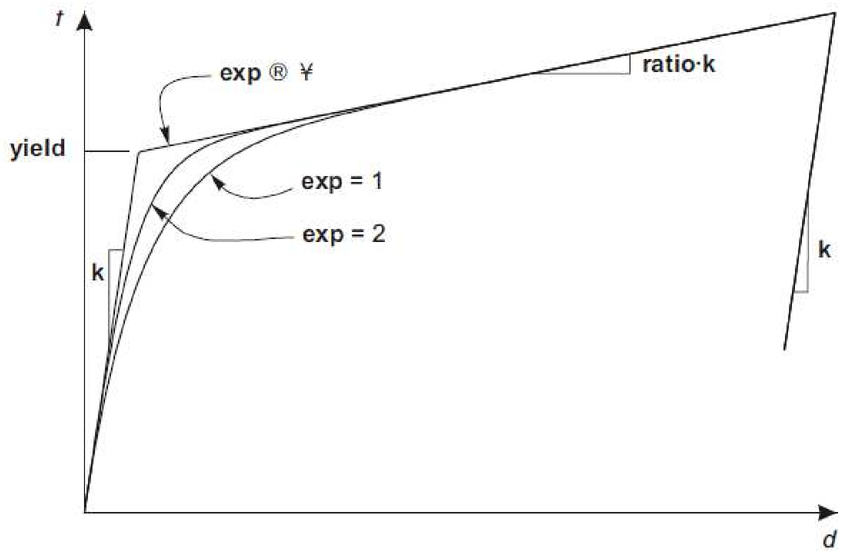

Figure 2.5 Nonlinear force- deformation relationship of Wen plasticity

Most of internal deformations usually work independently. The yielding at one degree of freedom has no effects at the behavior of any other deformations. When nonlinear properties are not specified for a specific freedom degree, that freedom degree is acting as linear usage of the effective stiffness, that can be also zero.

The nonlinear force- deformation relation is provided as:

$$f = ratio\ k\ d + (1 - ratio) yield\ z \tag{2.9}$$

where **k** is the constant of elastic spring, **yield** is a yield force, **ratio** is the determined ratio of post- yield stiffness to elastic stiff ness (**k**), and **z** is an internal hysteretic variable. This variable has a range of | z | ≤ 1, with the yield surface represented by | z | =1. The initial value of z is zero, and it evolves according to the differential equation:

$$\dot{z} = \frac{\mathrm{K}}{\mathrm{yield}} \begin{cases} \dot{d}(1 - |z|^{exp}) & if\ \dot{d}\ z > 0 \\ \dot{d} & otherwise \end{cases} \tag{2.10}$$

where **exp** is an exponent greater than or equal to unity. Larger values of this exponent increase the sharpness of yielding. The practical limit for **exp** is about 20. The equation for $\dot{z}$ is equivalent to Wen's model with A =1 and $\alpha = \beta = 0.5$. [10]

2.6 Global energy response

Usually, the entire structure is being provided by energy response for specific types of load cases, that includes linear and nonlinear modal time-history, nonlinear static and, and nonlinear direct-integration time, staged construction.

Considering three basic sorts of energy: the work done by external loads that apply energy over the structure, conservative energy in the structure that can be recovered, and unconservative energy that is dissipated by the structure. Looking at the energy response can show the behavior of the structure, and it is useful to determine the effectiveness of energy-dissipation mechanisms which are provided as part of the design. This can include dampers, isolators, and plastic hinging in structural elements.[10]

CHAPTER III

RUBBER CONCERETE

3.1 Rubber as a material

Powdered rubber could be a general term, or an expression given to recycled rubber that's generated from scrap tires. The outline of the rubber source is incredibly important and may always be laid out in the literature because it's an influence on the feel and also the shape, and consequently, on the characteristics of the concrete that's adjusted by the addition of the desired percentage of the rubber. [1]

It appears that because of the brittle nature of concrete, the foremost direct and effective approach in creating damage tolerant concrete structures would be to embed intrinsic tensile ductility into concrete. Also Crumb Rubber have the ability of absorbing sudden impacts by taking control of the transmission of waves done by movement loads. Rubber can be compressed and deformed in easy way, but the speed of deformation reduces by increasing applied loads, that makes it a decent muffler. Rubber also has the property of small hysteresis and take control of energy dissipation in an exceedingly system exposed to vibration relevant forces. Aggregates are considered as the smallest quantities' expensive component in concrete and their usage is related to economics. [2]

3.2 Research on rubberized concrete

Crumb rubber is obtained by chopping scrap tires, it had been added in two different sets named SET 1 - Treated Crumb Rubber (TCR) and concrete and SET 2 - TCR with Ultra Fine GGBS as admixture in concrete. Coarse aggregate replaces Rubber in each of the two Sets in proportions of 5, 10, 15 and 20%. Properties like Compressive Strength, Young's Modulus, Direct and Semi direct Ultrasonic Pulse Velocity, Sorbtivity damping ratio and Frequency were discovered. Deformation and mode shape were studied with modal analysis and static analysis by applying a regular pressure appreciate the best compressive strength of the slab, using ANSYS. [2]

3.3 Methodology and results

3.3.1 Compressive strength

Compressive strength is considered as an essential parameter for any material which is given like a ratio between applied load and surface of the sample. Many tests had conducted on cube specimens after 28 days. Test cube patterns of size 100 x 100 x 100 mm were applied having a ultimate capacity of 2000kN, as illustrated in Figure 3.1.

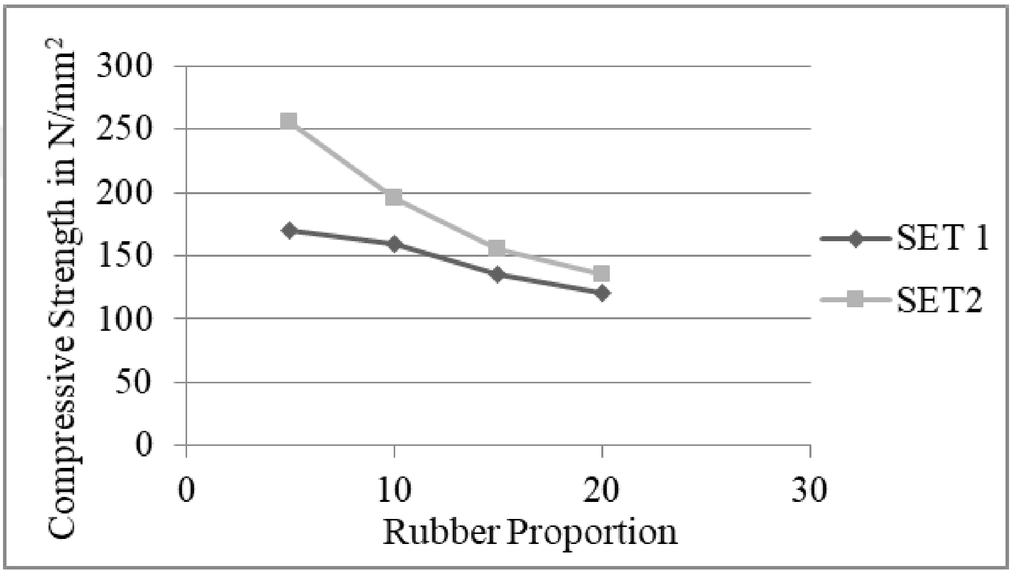

Figure 3.1 Rubber concrete's compressive strength test.

3.3.2 Damping ratio and natural frequency analysis

Crumb rubber has the property of dissipating energy and this property can be exploited to improve energy absorbing characteristics of concrete. Accident loadings, wind, seismic loads, or vehicle passage are causing vibrations in concrete structure and crack formation. Damping and frequency analysis helps to analyze and early mitigate of such hazards. Frequency and damping tests were applied on cube samples of size 100 x 100 mm. By using a Fast Fourier transform (FFT) device, damping properties were specified, as shown in Figures 3.3 and 3.4.

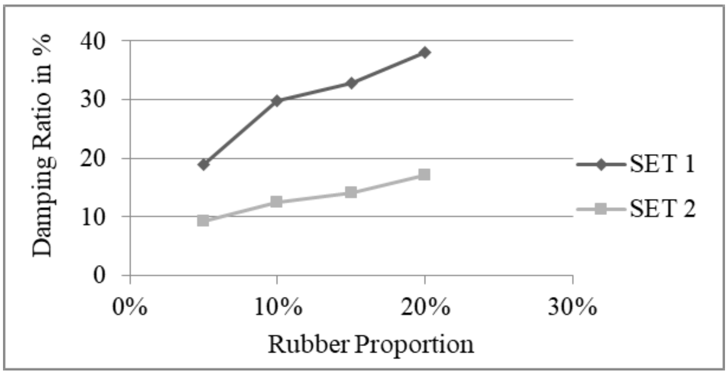

Figure 3.2 Damping ratio of rubber concrete test.

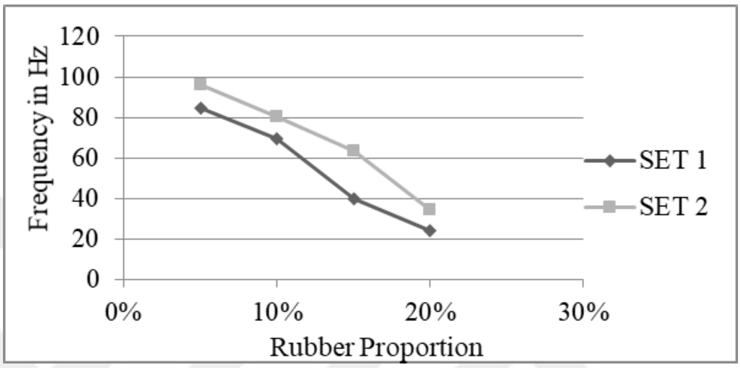

Figure 3.3 Natural frequency variation of with rubber.

3.3.3 Young's modulus (E)

Young's Modulus is one of the most significant properties that analysis concrete's elastic specifications and deformations' samples and measures the stiffness of a material. Young's Modulus is a proportional constant in Hooke's Law and is given as a ratio of stresses to strains. Test was applied on a 100 x 200 mm cylindrical specimen, that the drift was found by fixing a dial gauge parallel to the side of the specimen as it shown in Figure 3.6.

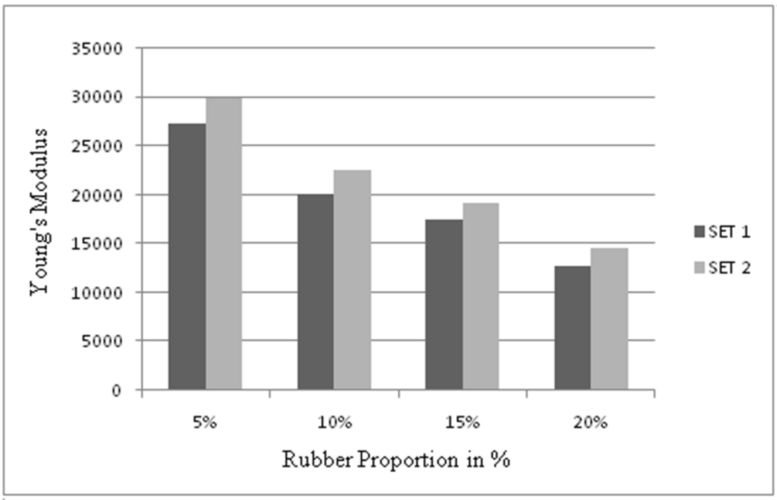

Figure 3.4 Young's modulus value of rubber concrete Test.

3.4 Summary

The summary of rubberized concrete used in this research will be according to the values in Table 3.1

Table 3.1 Parameters' values of rubber concrete

Parameters	Normal	5%	10%	15%	20%
Compressive Strength *Mpa*	27.58	17	16	14	12.5
		16	13.82	11.88	8.97
		26.88	24.13	19.53	13.65
		30.07	28.15	22.13	16.3
		39.95	34.63	23.96	18.93
Average	**27.58**	**25.9**	**22.78**	**17.83**	**14.05**
Material Damping Ratio *%*		18	30	33	38
		9	12	14	18
Average	**5**	**13.5**	**21**	**23.5**	**28**
Young Modulus *Mpa*	24,855	27,000	20,000	17,500	12,500
		30,000	22,500	19,000	14,000
Average	**24,855**	**28,500**	**21,250**	**18,250**	**13,250**
Tensile Strength *Mpa*		2.492	1.799	1.789	1.319
		2.675	2.49	2.295	2.066
		3.393	3.215	2.837	2.39
		4.327	3.656	3.015	2.361
		3.22175	2.79	2.484	2.034
Average		**2.492**	**1.799**	**1.789**	**1.319**

CHAPTER IV

PUSHOVER (NONLINEAR STATIC ANALYSIS)

4.1 Definitions

Both elastic and inelastic, are the types of the analysis of recent buildings. The elastic type analyses methods include the code of static lateral force prosses, demand capacity ratios using elastic procedures and code dynamic lateral force procedures. The fundamental inelastic analysis way is a nonlinear time history analysis, that have consideration of very complex and impracticable for common usage.

The simple methods of nonlinear analysis which known as nonlinearity of static analysis processes like Capacity Spectrum Method (CSM) which is the interchange of the curve of capacity and a reduced response spectrum to assess ultimate displacements.

The capacity spectrum method provides a graphical representation of force-displacement capacity curve of the structure comparing it to the response spectrum lines of the seismic demands, is a useful tool in the evaluation design of concrete buildings. [8]

4.2 Methods to perform nonlinear analysis

The keys factors of overall performance-based totally design method are the demand and capability. Demand is an impersonation of the earthquake ground movement. Capacity is the impersonation of the structure's capability to resist the ground motion loads. The overall performance is depending on the manner that the capability can manage the Demand.

Capacity: the total capacity of a structure relies on the deformations and strengths capacity of the individual elements of the structure. To determine capacities going far than elastic limits.

Demand: ground motions usually during the earthquake produce complex horizontal displacements in structures that vary with time. Tracking such motion at every time-step to determine structural design requirement is impractical. For arbitrary building and ground motion, displacement request is the ultimate expected response of the structure when earthquake occurs.

Performance: A performance control had to be done when curve of capacity and required displacement are determined. This performance control confirms that both structural with nonstructural members are not harmed beyond the accepted ranges of the performance target for displacement and forces. Figure 4.1 shows a typical capacity curve between base shear forces and roof displacement.

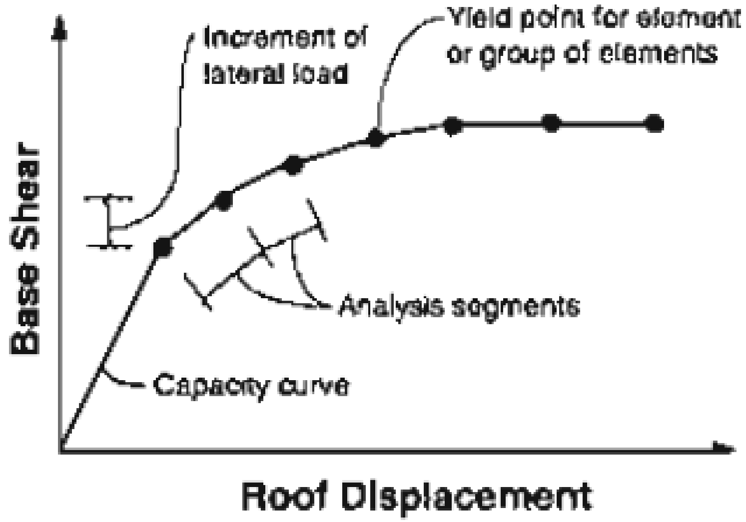

Figure 4.1 Capacity curve

4.2.1 Determining capacity

Structure capacity representation can be plotted by pushover curve. Best way to plot the force displacement curve is tracking base shear and the roof displacement. Curve is drawn to represent first mode response of the structure considering the presumption that essential mode of the motion is the predominantly response of the structure. This is valid for structures with essential period of ground motion less than one second.

4.2.2 Determining demand

Calculating of Demand by using the Capacity Spectrum Method:

Position of the performance point should be complied with the followings: 1) it must be on the capacity spectrum curve for specific displacement, and 2) should lie at the same time on spectral demand curve, reduced from the elastic, 5% damped design spectrum, that represents the nonlinear demand at the same structural displacement. According to what mentioned previously, factors of spectral reduction are given as effective damping. Which calculated according to capacity curve form, the displacement demand, and the hysteresis loops. Predictable defections in actual structure hysteresis loops, with degradation and duration effects, are accounted by reductions of equivalent viscous damping values.

The methods used for this determination are:

- **The concept of developing of the capacity spectrum:**

To apply capacity spectrum in nonlinear analysis, it is necessary to transform capacity curve from base shear forces and roof displacements to a capacity spectrum represented by the capacity in Acceleration-Displacement Response Spectra (ADRS) format. The required equations to make the transformation are:

$$PF_1 = \left[\frac{\sum_{i=1}^{N} \frac{w_i \Phi_{i1}}{g}}{\sum_{i=1}^{N} \frac{w_i \Phi_{i1}^2}{g}}\right] \tag{4.1}$$

$$\alpha_1 = \frac{\left[\sum_{i=1}^{N} \frac{w_i \Phi_{i1}}{g}\right]^2}{\left[\sum_{i=1}^{N} \frac{w_i}{g}\right]\left[\sum_{i=1}^{N} \frac{w_i \Phi_{i1}^2}{g}\right]} \tag{4.2}$$

$$S_a = \frac{V/W}{\alpha_1} \tag{4.3}$$

$$S_d = \frac{\Delta_{roof}}{PF_1 \Phi_{roof1}} \tag{4.4}$$

PF_1 modal participation factor for the first natural mode.

α_1 modal mass coefficient for the first natural mode.

$\frac{w_i}{g}$ mass assigined to to level i.

ϕ_{i1} amplitude of mode1 to level i.

N level N, the level which is the uppermost in the main portion of the structure.

V base shear.

W building dead weight plus likely live loads.

Δ_{roof} roof displacement (V and the associated Δ_{roof} make up points on the capacity curve).

S_a spectral acceleration

S_d spectral displacement (S_a and the associated S_d make up points on the capacity spectrum).

Figure 4.2 shows an example of modal participation factors and mass coefficients.

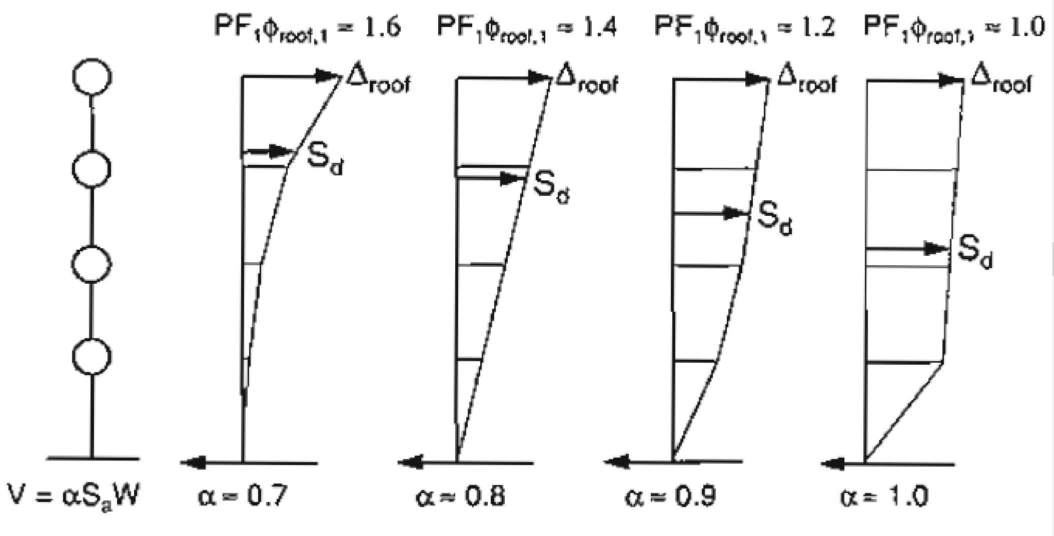

Figure 4.2 Example of modal participation factors and modal mass coefficients

The overall procedures of transforming capacity curve to capacity spectrum are by transforming capacity curve into ADRS format, at the beginning it must calculate the modal PF_1 and the modal coefficient α_1. Then every point on capacity curve, V, Δ_{roof}, calculating the related point S_a, S_d on the capacity spectrum as it could be seen in Figure 4.3.

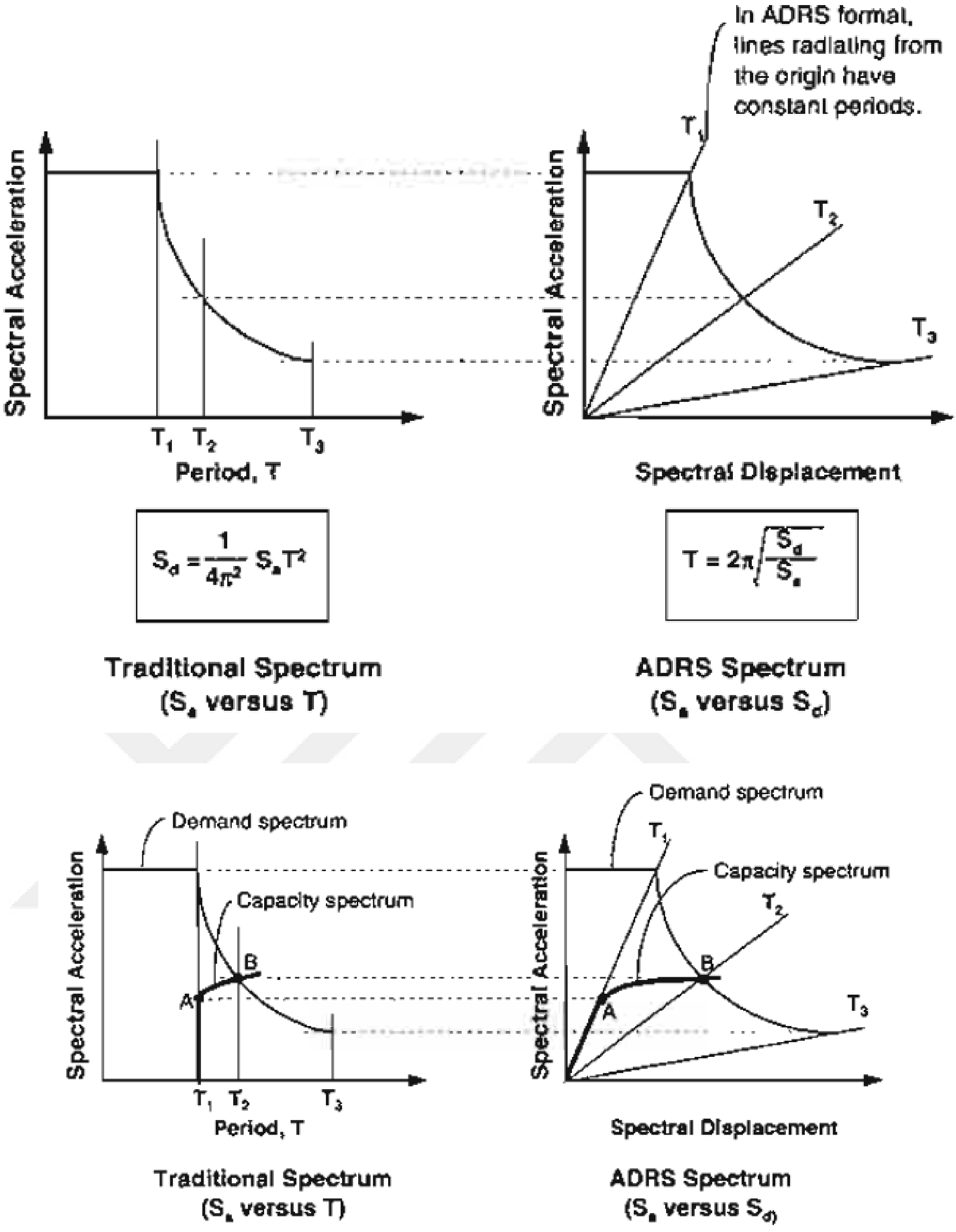

Figure 4.3 Capacity Spectrum Superimposed Over Spectra in Traditional and ADRS Format

4.2.3 Bilinear representing of capacity spectrum

The dual linear capacity spectrum representation is important for estimating effective damping and spectral demand reduction. Establishment of the bilinear representation needs specification of the point a_{pi}, d_{pi}. This point can be considered as the trial performance point to improve a reduced demand response spectrum. If the reduced response spectrum is found to intersect the capacity spectrum at the estimated a_{pi}, d_{pi}

point, then that point is the performance point. The first level of estimating of point a_{pi}, d_{pi} is appointed a_{p1}, d_{p1}, the second a_{p2}, d_{p2}, and so on. Figure 4.4 shows how the bilinear representation of capacity spectrum is constructed, while Figure 4.5 shows the dissipated energy by damping E_D and the maximum strain energy E_{S_0}.

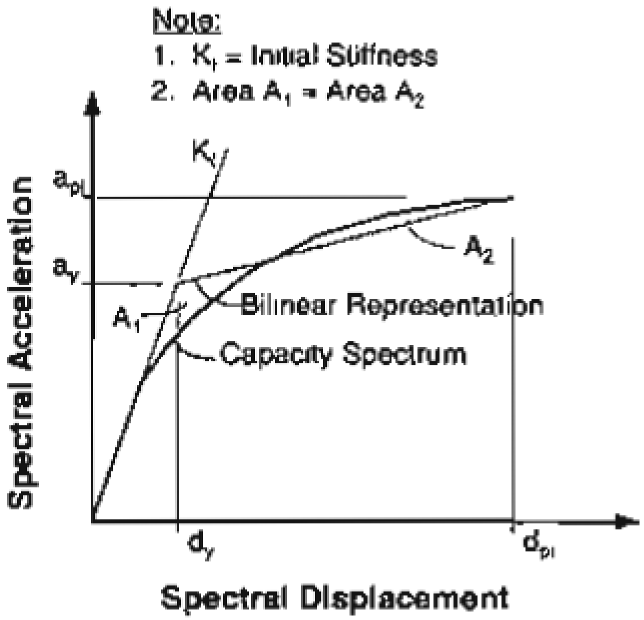

Figure 4.4 Bilinear representation of capacity spectrum for capacity spectrum method

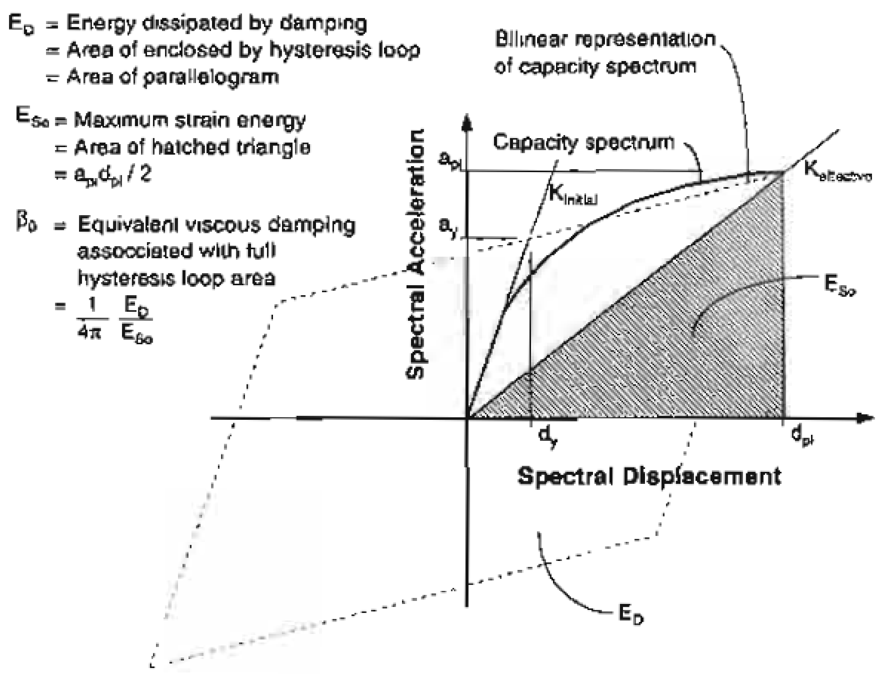

Figure 4.5 Derivation of damping for spectral reduction

The area designated A1 in Figure 4.4 is approximately equal to the area designated A2. The intent of setting area A1 equal to area A2 is to have equal area under the capacity spectrum and its bilinear representation, that is, to have equal energy associated with each curve.

4.2.4 Estimation of damping & reduction of 5% damped response spectrum:

Damping which happens when seismic forces lead a building into the nonelastic limit may be shown as a mixing of viscous damping which is essential in the hysteretic damping and structure. Hysteretic damping is associated to the loops' area formed when seismic forces (base shear force) is plotted against displacement. It also can be represented as equivalent viscous damping β_{eq}, associated with a maximum displacement of d_{pi} can be estimated from the following equation:

$$\beta_{eq} = \beta_0 + 0.05 \qquad (4.5)$$

Where,

β_0 hysteretic damping represented as equivalent viscous damping

0.05 5% viscous damping essential in the structure

The term β_0 may be calculated

$$\beta_0 = \frac{1}{4\pi}\frac{E_D}{E_{S_0}} \qquad (4.6)$$

Where,

E_D the energy dissipated by damping

E_{S_0} the maximum strain energy

E_D is the energy dissipated by the structure in a single cycle of motion, that is, the area enclosed by a single hysteresis loop. E_{S_0} is the maximum strain energy associated with that cycle of motion, that is, the area of hatched triangle.

The term E_D can be derived as

$$E_D = 4 * \left(a_{pi}\, d_{pi} - 2\mathrm{A}_1 - 2\mathrm{A}_2 - 2\mathrm{A}_3\right)$$
$$= 4\left(a_{pi}\, d_{pi}.\, a_y\, d_y - (d_{pi} - d_y)(a_{pi} - a_y) - 2d_y(a_{pi} - a_y)\right)$$

$$= 4\left(a_y d_{pi} - d_y a_{pi}\right) \tag{4.7}$$

The term E_{S_0} can be derived as

$$E_{S_0} = \frac{a_{pi} d_{pi}}{2} \tag{4.8}$$

$$\beta_0 = \frac{2}{\pi} \frac{a_y d_{pi} - d_y a_{pi}}{a_{pi} d_{pi}} \tag{4.9}$$

$$\beta_{eq} = \frac{63.6\left(a_y d_{pi} - d_y a_{pi}\right)}{a_{pi} d_{pi}} + 5 \tag{4.10}$$

Figures 4.6 and 4.8 show how energy dissipated by damping E_D is derivate.

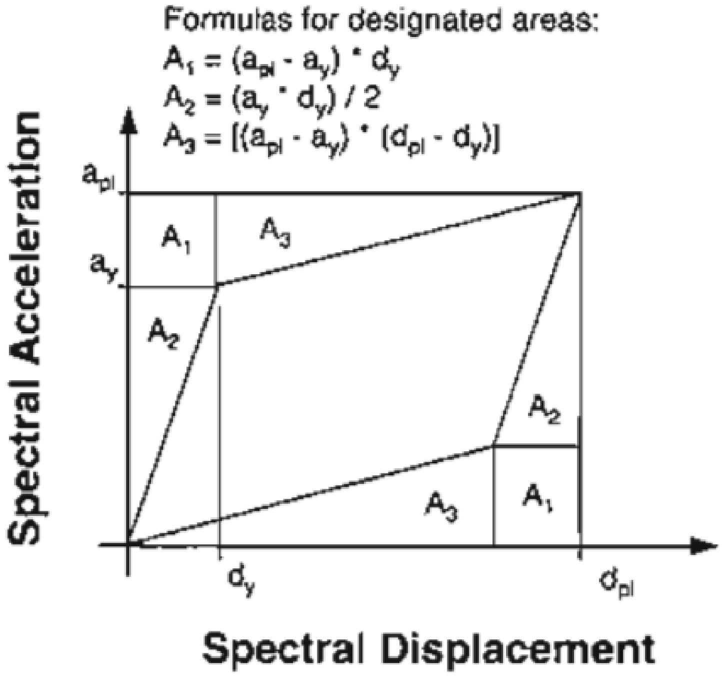

Figure 4.6 The derivation of dissipated energy by damping E_D

β_{eq} values obtained are used to estimate factors of spectral reduction, which are used to reduce elastic (5% damped) response spectrum with damping greater than 5% of critical damping. For damping values less than 25%, factors of spectral reduction calculated using β_{eq} equation is harmonious with analogous factors in the FEMA Guidelines. Figure 4.7 illustrates how response spectrum as a demand curve can be reduced.

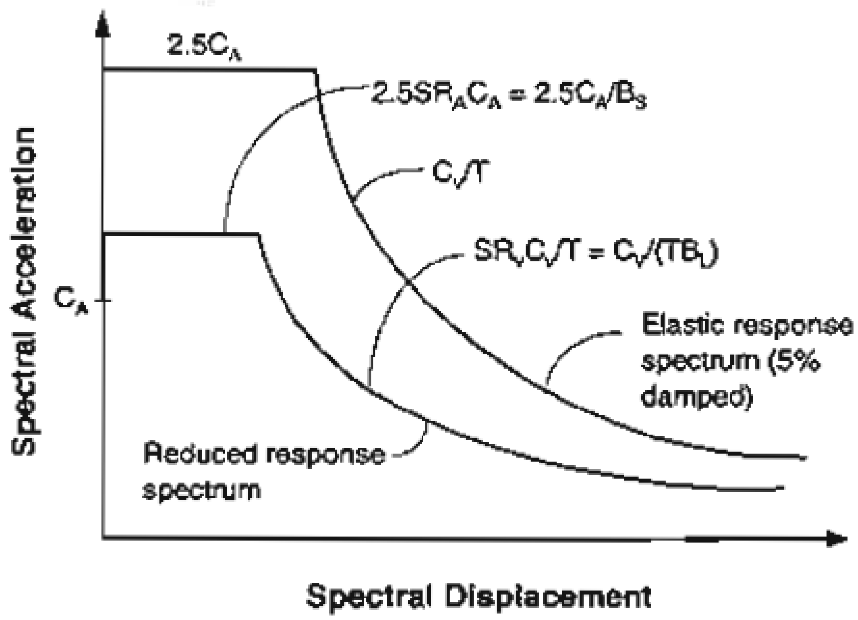

Figure 4.7 Reduced response spectrum

Those damping factors indicated that spectrum must not be increased to this limit at higher values of damping and judgmental way of increasing the coefficients beginning at almost 25% the damping, in addition to set an absolute range on decreases at β_{eq} of about 50%.

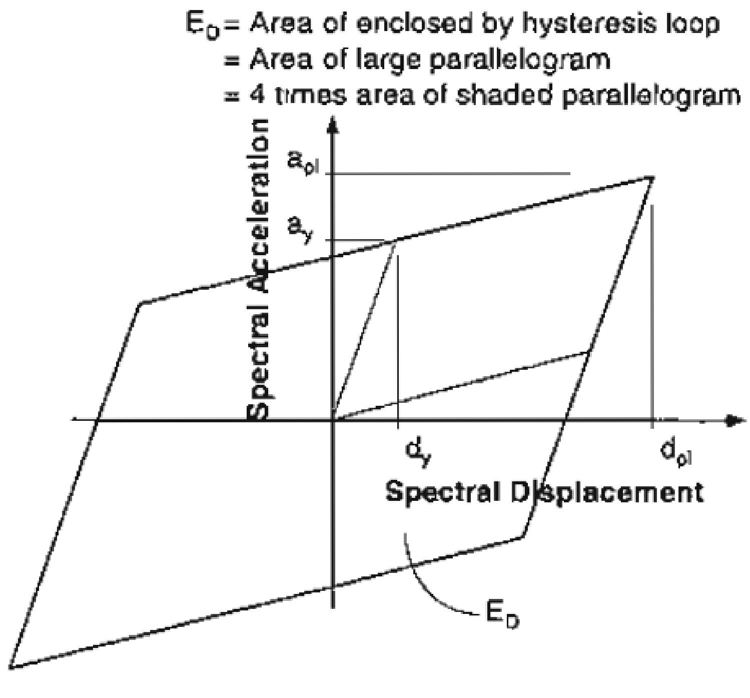

Figure 4.8 Derivation of energy dissipated by damping E_D

The idealized hysteresis loop is a rational estimation for a ductility detailed building exposed to shortened duration vibration (cycles are not sufficient to exceptionally degrade members) and with no more than 30% of equivalent viscous damping ratio.

To consistent with damping coefficients, B, and to simulate non perfect hysteresis loops (that is decreased in areas), the term of effective viscous damping to use a factor of modifying the damping, k, so effective viscous damping, β_{eff} is concluded by:

$$\beta_{eff} = k, \beta_o + 5 = \frac{63.6k(a_y d_{pi} - d_y a_{pi})}{a_{pi} d_{pi}} + 5 \quad (4.11)$$

k-factor is a measure of the actual structure hysteresis is well represented, either initially, and/or after degradation. Also, it depends on the structural behavior of the building, and on the quality of the seismic resisting system and the duration of ground motion. Numerical spectral reductions conclusion:

Reduction factors equations for the of response spectrum SR_A and SR_V are given by:

$$SR_A = \frac{3.21 - 0.68 \ln(\beta_{eff})}{2.12} = \frac{3.21 - 0.68 \ln\left[\frac{63.6k(a_y d_{pi} - d_y a_{pi})}{a_{pi} d_{pi}} + 5\right]}{2.12} \quad (4.12)$$

$$SR_V = \frac{2.31 - 0.41 \ln(\beta_{eff})}{1.65} = \frac{2.31 - 0.41 \ln\left[\frac{63.6k(a_y d_{pi} - d_y a_{pi})}{a_{pi} d_{pi}} + 5\right]}{1.65} \quad (4.13)$$

When Figure 4.9 shows a typical demand spectra family in ADRS format.

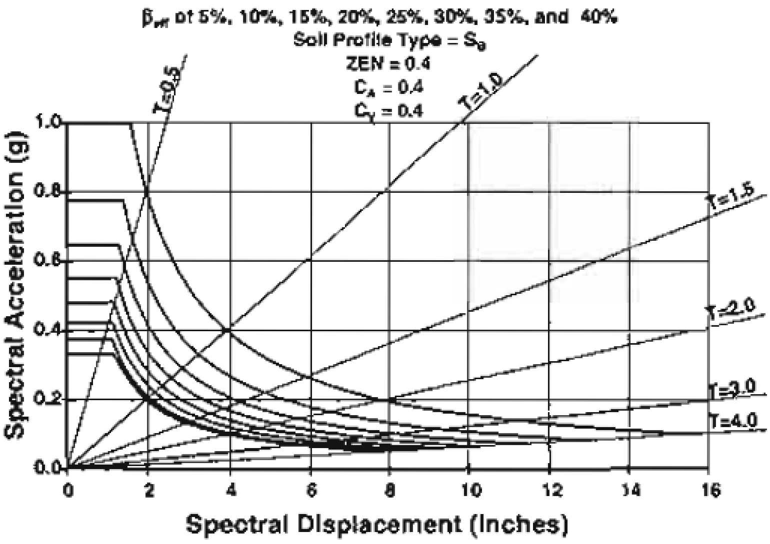

Figure 4.9 Family of demand spectra in ADRS format

4.3 Intersection of Capacity Spectrum and Demand Spectrum:

If the displacement at the demand spectrum and the capacity spectrum intersection, d_i is within 5% ($0.95d_{pi} \leq d_i \leq 1.05d_p$) of the displacement of trial point of performance. a_{pi}, $d_{pi,}$ d_{pi} becomes the exact point of performance. if the demand spectrum and the capacity spectrum intersection is not within the acceptable rang, then a new a_{pi}, d_{pi} point had to be selected, and repeating the process.

A maximum structural displacement represented by performance point is expected for the demand earthquake ground motion as it represented in Figure 4.10.

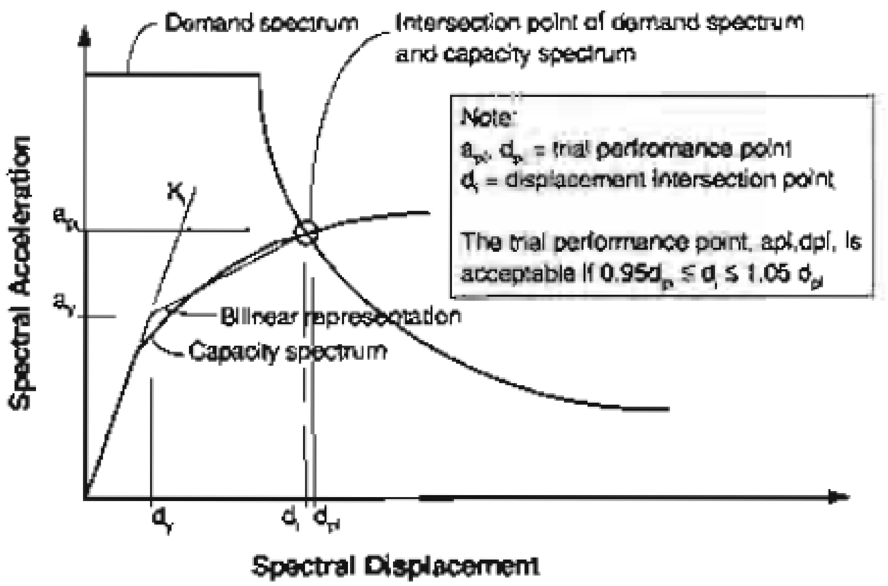

Figure 4.10 Intersection Point of demand and capacity spectrums within acceptable tolerance

CHAPTER V

SEISMIC REQUIRMENTS AND PERFORMANCE OBJECTIVES

5.1 Performance objective

A performance objective determines the requested ground motion performance of structure. Earthquake performance is determined by nominating the most allowable harm situation (performance level) for a specified seismic danger. A performance objective includes sight of harm situations for multiple levels of earthquakes and can then be turned into multiple levels of performance objective.

5.2 Performance levels

A performance levels usually must describe limits of damage conditions that may be taken into consideration of satisfactory for a specified structure and ground motion. The limitation conditions which might be considered by the physical harm within the structure, threatening the life safety of establishment's occupants done by means of the damage, and the after-earthquake service situation of the structure.

Required performance levels of structural systems can be determined one by one. Building performance levels are a mixture of a structural and nonstructural performance levels.

5.2.1 Levels and ranges of structural performance

Limits and levels of structural performance are assigned a little for reference simplicity and a number. The Structural Performance Levels- Immediate Occupancy, Life Safety and Structural Stability- are separated harm situations and can be used immediately in estimation and prosses of retrofit to determine technical criteria.

Immediate Occupancy (IO): The post-earthquake damage state in which only very limited structural damage has occurred. The basic vertical and lateral force resisting systems of the building retain nearly all of their pre-earthquake characteristics and capacities.

Damage Control (DC): this term is actually not a specific level but a range of post-earthquake damage states that could vary from Immediate Occupancy to Life Safety. It can provide a placeholder to several states where it can be obligatory to limit structural harm exceeding the level of Life Safety, but there is no issue of occupancy.

Life Safety (LS): The after-earthquake harm situation that important injury to the structure can occur so some boundary against partial or total structural failure leftovers. The scale of harm can be lower than that which is for the level of Structural Stability. Main structural elements are not become expelled and fell, intimidating life safety inside and outside structure members. But damages through the ground motion may occur, the hazard of life-threatening harm from structural harm is very low.

Limited Safety: this term is not limited to a determined scale but a limit of post-earthquake harm situation which are less than Life Safety and more than Structural Stability. It is providing a placeholder for the state where a retrofit may not bear whole structural needs of the Life Safety level but is better than the level Structural Stability.

Structural Stability (SS): This level is the limiting post-earthquake structural damage state in which the building's structural system is on the verge of experiencing partial or total collapse. Potentially including significant degradation in the stiffness and strength of the lateral force resisting system. However, all significant components of the gravity load resisting system continue to care their gravity demands.

Not Considered (NC): this can provide a placeholder for states which only seismic of nonstructural estimation or retrofit is done.

5.2.2 Building performance levels:

Combinations of a structural performance level and a nonstructural performance level form a Building Performance Level to completely describe the desired limiting damage state for the building. The four most referenced building performance levels are:

Operational: This is the performance level related to functionality. Damage to the building structure is limited so that safe occupancy is not an issue, and any requested repairs are secondary, damage to nonstructural systems related to operations is minor and will not venture operations in the building.

Immediate Occupancy: This corresponds to the almost closely used criteria for fundamental facilities. The structure's areas and systems are predictable to be rationally usable, but continuity of all services, major or secondary, is not certainly provided.

Life Safety: It is the level which is intended by achieving a damage situation that presents an intense low possibility of threats to life safety, either from structural damage or from falling of nonstructural components of the building. This performance level can be called as intended to be less than the performance which is predicted of code designed new buildings.

Structural Stability: This state addresses only the major building frame gravity load carrying system and demands only settlement under vertical loads. No reservoir against failure in aftershock can be available.

5.3 Earthquake ground motion

Earthquake ground motion and the desired performance objective are combined with each other. earthquake ground motion may express by determining a scale of shaking related to a specific probability of occurrence, or by ultimate shaking predictable from a one event of determined size on determined source fault. A response spectrum or an equivalent series of simulated ground motion records are used for this purpose.

5.4 Seismic hazard

5.4.1 Earth ground shaking hazard levels

Serviceability Earthquake

The Serviceability Earthquake (SE) is the situation of ground movement that has 50 present chance of being exceeded in a 50-year period. This situation of seismic motion is almost 0.5 times the level of ground motion of the Design Earthquake.

Design Earthquake

The Design Earthquake (DE) is the level of ground shaking that has a 10 percent chance of being exceeded in a 50-year period.

Maximum Earthquake

The Maximum Earthquake (ME) is the maximum level of earthquake found shaking which may ever be expected at the structure location within the known geologic framework. Intensity of ground shaking in Seismic Zones 3 and 4, might be calculated by the severe of shaking that has a 5% probability that can be exceeded in 50-year time-period. This scale of earthquake is about 1.25 to 1.5 times the scale of ground motion shaking of the Design Earthquake.

5.4.2 Primary ground shaking criteria

Site-specific hazard analysis studies

The analysis of site hazard analysis processes must be done for building placed on the profile of site soil type S_F, it is preferred for structures on the profile of site soil type S_E. Specific research must also be taken for specific structures placed near active seismic sources of and for building that have needs of special design.

Construction of an elastic response spectrum

For each seismic level of attention at a location an elastic spectrum is depended on the site earthquake coefficients C_A and C_V. C_A represents the effective peak acceleration (EPA) of the ground. A factor about 2.5 times C_A represents the average value of peak response of 5 percent-damped short period system in the acceleration domain. Coefficient Cv represents 5 percent-damped of 1 second system and when divided by period defines acceleration response in the velocity domain. Figure 5.1 illustrates the construction of elastic response spectrum. [9]

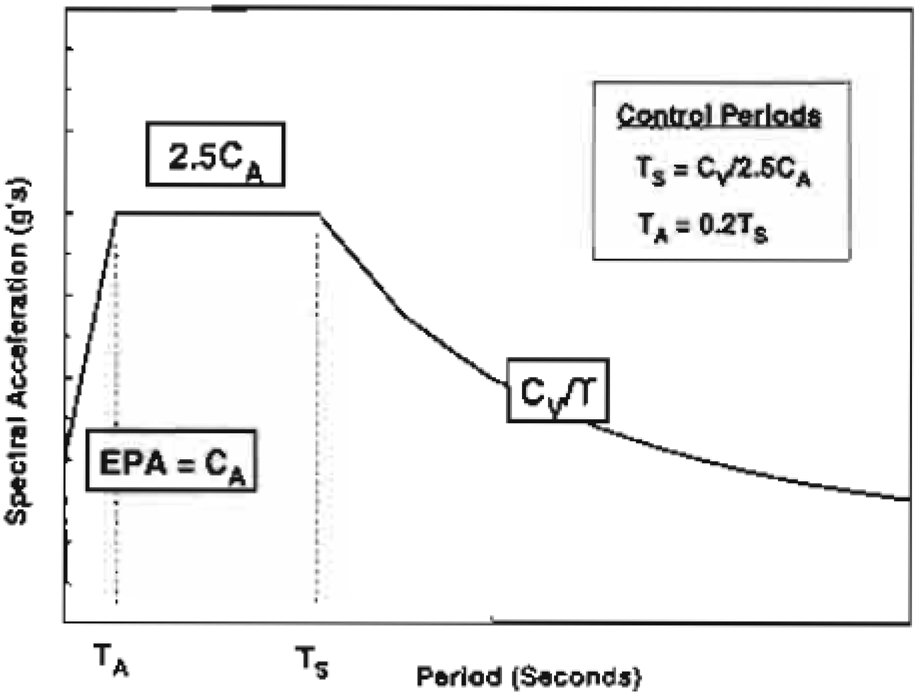

Figure 5.1 Construction of a 5% damped elastic response spectrum

Earthquake ground motion records

Two sets of 10 earthquake records each have been identified as suitable candidates for time history analysis. One set contains records at sites at least 10 km from fault rupture. These records are appropriate for time history analysis of structures at all sites after appropriate scaling.

CHAPTER VI

MODELING, RESULTS, COMPARISONS AND DISCUSSIONS

6.1 Introduction

Nowadays, technological development and the practical solutions which provides to complex issues have become an urgent necessity that cannot be dispensed with.

Accordingly, the development of damping systems used in modern structures and high-rise buildings depends on the software provided by computers mainly, programs such as SAP2000 and ETABS that allow any structural engineer to participate in the development of such systems. As a result of this, two-dimensional and three-dimensional models were assumed and represented on SAP2000 and ETABS programs to support this research based on the assumptions and equations that were mentioned in the previous chapters.

Mixing damping systems or modalities like (using rubber concrete and the benefits of damping due to absorbing energy through treated crumbed rubber articles in concrete aggregates, chevron braces with X plates dampers “Wen plasticity property dampers” or (XPDs, ADAS)) may lead to new achievement and development of structural damping.

The main objective of this paper is to make a comparison between crumbed rubber added to the aggregates of concrete with multiple percentages according to assumptions mentioned in **Chapter III** without adding any extra dampers, the second scenario will be a comparison between rubber concrete systems in accompany with adding chevron braces elements with X plate dampers.

Since dampers absorb energy of vibrations, in nonlinear analysis like push over, the effect of damping is related to the materials and systems used, so in pushover analysis

β_0 -the hysteretic damping- which is represented as equivalent viscous damping, it is changing according to materials of structure and damping systems added. Also, it is related to E_D the energy dissipated by damping and E_{S_0} which is the maximum strain energy.

Since the equivalent viscous damping β_{eff} bounds with the hysteretic damping β_0 of the structural system and its obtained values can be used to evaluate factors of spectral reduction in order to decrease the elastic (5% damped) response spectrum with damping ratio which is greater than 5% of critical damping.

Proposed and formed models in both two dimensions and 3D were to observe how equations 5, 6, 7, 8, 9, 10, 11, 12 and 13 mentioned in **Chapter IV** will react according different damping systems in the model. Since this research will deal with two types of damping modalities applied to models, the first one will be by the construction material which is related to the crumbed rubber added to the concrete aggregates with different properties which includes damping ratio ζ. As we can see in Figures 4.5, 4.6 and 4.8 every loop in direct integration solving mechanism while analyzing models, is generated according to material damping ratio and model's defined dampers like the XPDs. Hence new energy dissipation E_D and strain energy E_{S_0} is stored in the energy data base or matrix of the program as it explained in **Chapter IV** for each of material, modal, viscous and hysteretic damping to generate β_{eff} accordingly, hence new reduction of damping ratio of defined response spectrum is generated.

Meanwhile, during analysis the performance curve is being created according to equations 1, 2, 3 and 4 also motioned in **Chapter IV**. Those equations are related directly with the concepts of modal analysis mentioned in **Chapter I** since the pushover analysis depends firmly upon the first mode participation ratio of the model or structure as we saw in **Chapter IV**.

The Wen XPDs used in this research were selected due simplicity of manufacturing and availability in local markets. As we saw in **Chapter II** Wen or XPDs act as both linear and nonlinear behaviors according to the degree of freedom direction in which the link or the damper was modeled, thus plasticity property of the added links or dampers will add extra energy absorption as a nonlinearity action.

To see how all those factors affecting the pushover analysis in structures with multiple damping systems this research was made.

6.2 Two dimensional models' specifications

Objective of two-dimensional models is to build initial vision about how those aspects mentioned in previous clause will react, for two-dimensional models' comparison criteria were based on comparing the following parameters for each model with the rest of other models. Those parameters were:

- V, V_x Performance Point Base Shear Force (tonf) with/out XPDs.
- D, D_x Performance Point Displacement (cm) with/out XPDs.
- S_a, $S_{a\,x}$ Pseudo Spectrum Acceleration (m.sec^{-2}) with/out XPDs.
- S_d, $S_{d\,x}$ Pseudo Spectrum Displacement (m) with/out XPDs.
- T_{eff}, $T_{eff\,x}$ Performance Point Effective Period (sec) with/out XPDs.
- B_{eff}, $B_{eff\,x}$ Performance Point Effective Damping Ratio with/out XPDs.
- A-IO HN, A-IO HN_x # of formed hinges within IO criteria with/out XPDs.
- IO-LS HN, IO-LS HN_x # of formed hinges between IO & LS criteria with/out XPDs.

Figures 6.1, 6.2, 6.3, 6.4 and 6.5 show sections and properties of two-dimensional models, locations and types of hinges assigned.

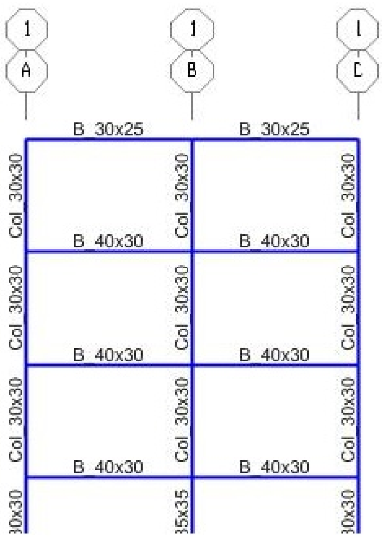

Figure 6.1 Sections, bays and heights of the two-dimensional model without XPDs

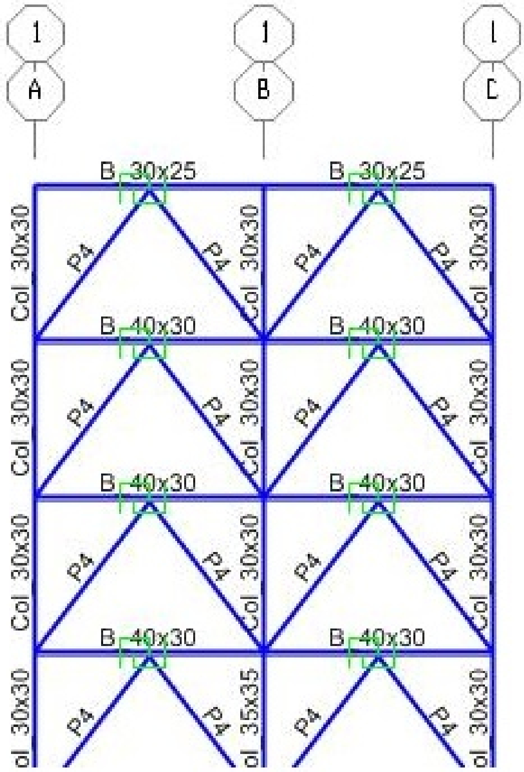

Figure 6.2 Sections, bays and heights of the two-dimensional model with XPDs

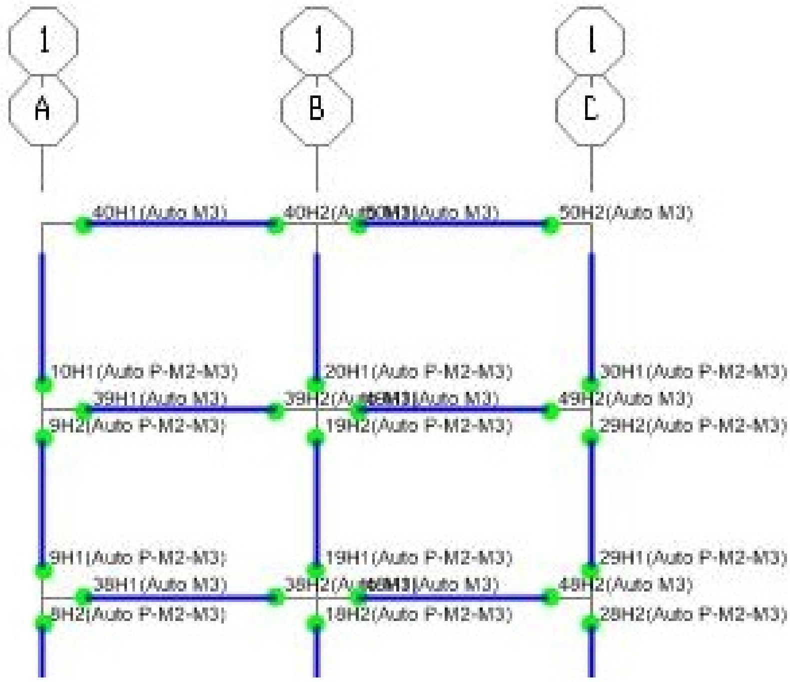

Figure 6.3 Hinges types in two dimensional models without XPDs

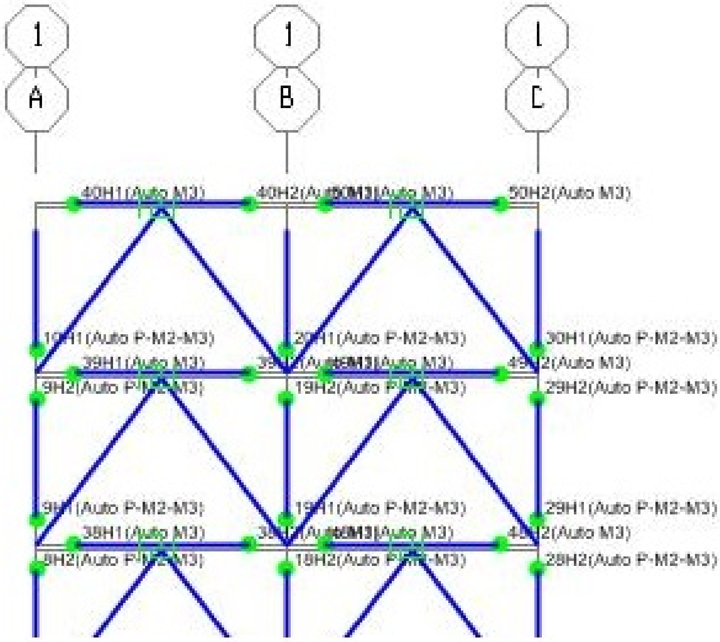

Figure 6.4 Hinges types in two dimensional models with XPDs

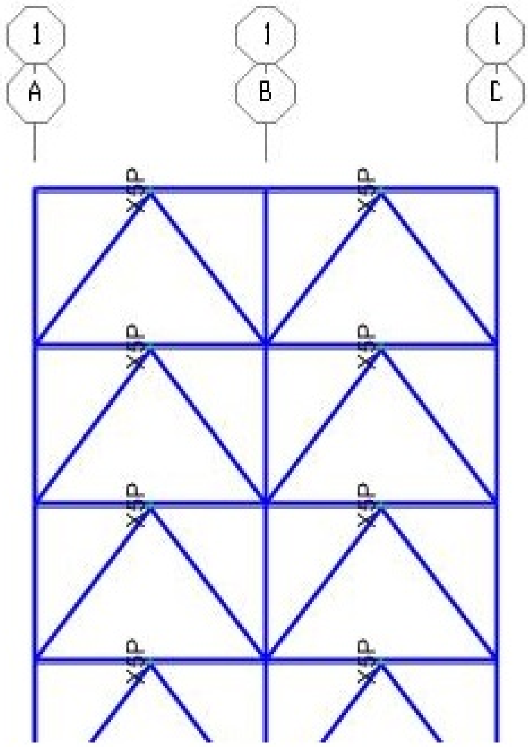

Figure 6.5 Defined XPDs in two dimensional models

All those comparisons will be done under single defined response spectrum function which all two-dimensional models exposed to.

Ten stories frame with two bays was to be analyzed according to the following specifications by using SAP2000 program:

- Geometrical specifications, Table 6.1 shows the two-dimensional models' specifications.

Table 6.1 two-dimensional model geometrical specification

Parameters	Value	Number
Bay Dimensions (m)	5	2
Story Hight (m)	3.4	10

- Materials used in the models are as defined in **Chapter III** for rubber percentage in the aggregates of used concrete. The properties of materials used in the two-dimensional models are specified in Table 6.2.

Table 6.2 Materials used in two dimensional models

Parameters	Normal	5%	10%	15%	20%
Compressive Strength MPa	27.58	25.9	22.78	17.83	14.06
Material Damping Ratio %	5	13.5	21	23.5	28
Young Modulus MPa	24,855	28,500	21,250	18,250	13,250
Tensile Strength MPa		3.22	2.79	2.48	2.03

- Sections are rectangular for all elements used in the models and they are fixed regardless the type of model and designing aspects as the target of the research is to compare the results of changing the materials and damping systems proposed, as in Table 6.3.

Table 6.3 Sections' definitions and locations for two dimensional models

Section	Height (cm)	Width (cm)	Location (Story)	Number
Col_40x40	40	40	1,2,3	7
Col_35x35	35	35	3,4,5,6,7	12
Col_30x30	30	30	7,8,9,10	11
B_40x30	40	30	1,2,3,4,5,6,7,8,9,10	20
P4 (Circular)[1, 2]	11.43	0.602	1,2,3,4,5,6,7,8,9,10	40

[1] This pipe section is used for 2D models Wen (Xplates) dampers height value in the table expresses diameter of section and width values expresses the thickness

[2] Material used of this section is the default Steel A99Fy50 in Sap2000 E 20,389,019 Tonf/m2, Fy 35,153.48 Tonf/m2, Fu 45,699.53 Tonf/m2

- Hinges properties, an interacting hinges P-M2-M3 for column elements and M3 Moment Hinges for beam elemets, which are specified in Table 6.4 and Table 6.6.

Table 6.4 Properties and fields of P-M2-M3 hinges used in two dimensional models

Hinge Type P-M2-M3 Moment/ Rotation Data	**A**	**B**	**C**	**D**	**E**
Moment/Yield Mom	0	1	1.1	0	0
Rotation/scale factor	0	0	0.01	0.01	0.01

The acceptance properties of P-M2-M3 hinges (column hinges) are specified in Table 6.5 which specifies at which range the hinge is acting after analysis.

Table 6.5 Acceptance criteria of P-M2-M3 hinge types used in two dimensional Models

Hinge Type P-M2-M3 Acceptance Criteria	**Value** (Deformation/SF)
Immediate Occupancy	3.000E-3
Life Safety	9.000E-3
Collapse Prevention	0.01

Table 6.6 Properties and ranges of M3 hinges used in two dimensional models

Hinge Type M3 Moment/ Rotation Data	**Moment/Yield Mom**	**Rotation/scale factor**
E-	-0.2	-0.0457
D-	-0.2	-0.0259
C-	-1.1	-0.0239
B-	-1	0
A	0	0
B	1	0
C	1.1	0.025
D	0.2	0.0269
E	0.2	0.05

The acceptance properties of M3 hinges (beams hinges) are specified in Table 6.7 which specifies at which range the hinge is acting after analysis.

Table 6.7 Acceptance criteria of M3 hinge types used in two dimensional models

Hinge Type M3 Acceptance Criteria (Plastic Rotation/SF)	**Value (Positive)**	**Value (Negative)**
Immediate Occupancy	0.01	-8.92E-3
Life Safety	0.025	-0.0269
Collapse Prevention	0.05	-0.0457

- Mass Sources: the mass sources of models were taken from elements self mass, with additional mass of assigned 1 supper dead loads and 0.25 multiplier of assigned live loads.
- Load Patterns, the loads patterns used in the two-dimensional models are used as in the Table 6.8.

Table 6.8 Load patterns used in two dimensional models

Load Pattern Name	**Type**	**Self-Weight Multiplier**	**Location** ***(Story)***
Dead	Dead	1	All elements
SDL	Super Dead	0	Beam elements
LL	Live	0	Beam elements
Push	Other	0	All elements
Gravity NL	Other	0	All elements

- Response Spectrum Functions: the response spectrum defined for all two dimensional models was taken according to ASSHTO 2012 with those parameters with Tonf, m, C units, as in Table 6.9.

Table 6.9 Design response spectrum parameters used in two dimensional models

Parameters Name	**Value**
0.2 Sec Spectral Accel, Ss $_{(m.sec^{-2})}$	4
1 Sec Spectral Accel, S1 $_{(m.sec^{-2})}$	2.25
Peak Ground Accel, PGA $_{(m.sec^{-2})}$	1.5
Site Class	C
Site Coefficient, Fa	1
Site Coefficient, Fv	1.3
Site Coefficient, Fpga	1
SDS=Fa*Ss $_{(m.sec^{-2})}$	4
SD1=Fv*S1 $_{(m.sec^{-2})}$	2.925

- Load cases:
    - **Dead load** was defined as linear static load and applied for all models as self-weight load for all structural elements.
    - **Modal load** was defined as Eigen vectors with zero initial conditions and the defined mass sources of the model.
    - **Super Dead load** defined as liner static load and applied as 1.5 Tonf/m for all beam elements of the models with zero initial conditions.
    - **Live load** defined as linear static load and applied as 2.5 Tonf/m for all beam elements of the models with zero initial conditions.
    - **Push Over Load** which is a nonlinear static typecontinue from the state at end of another nonlinear case which is Gravity nonlinear, where all modal loads applied using modes from the defined modal load with the same mass sources, load application will be Displacement Control with multiple states.
    - **Garvity Nonlinear load** is nonlinear static loadcase starting from zero initial conditions (Unstressed State) contains 1 Dead, 0.25 Live and 1 Super Dead load patterns, the load will be applied fully with final state result.
- Load combinations will be taken as the default Sap2000 conbination for concerete RFS (Rigid Frame Structures).
- Analysis Options were taken to be XZ plane with advanced solver with auto process.
- Wen plasticity dampers specifications (XPlate Dampers) and it is functional into U2 direction. Where the Wen properties of used dampers in the two-dimensional models are as in Table 6.10.

Table 6.10 Parameter of Wen (XPDs) used in two dimensional models

Parameters Name	Value
n (Number of plates)	5
E (Young Modulus od plates) $(N.mm^{-2})$	199947.98
b (Width of plates) (mm)	100
t (thickness of plates) (mm)	8
a (height of plates) (mm)	50
σy (Yielding strength of plates) (N/mm^2)	248.2113
σu (Failure strength of plates) (N/mm^2)	399.896
H (rate of strain hardening)	3450.95
d (yield displacement) (mm)	30

Kd (the initial stifness) (N.mm^{-1})	34124.45
Fy (N/mm^{2})	26475.87
q (yield displacement) (mm)	0.77586
Y_0	0.10344
P (plastic force for d displacement) (N)	65977.44
Kp (plastic stiffness) (N.mm^{-1})	2199.248
Kp/Kd	0.06444

6.3 Geometrical properties of two-dimensional models using Sap2000 analysis

The porperties of higes assigned to the frames of two dimensional models are illustrated in Figure 6.6

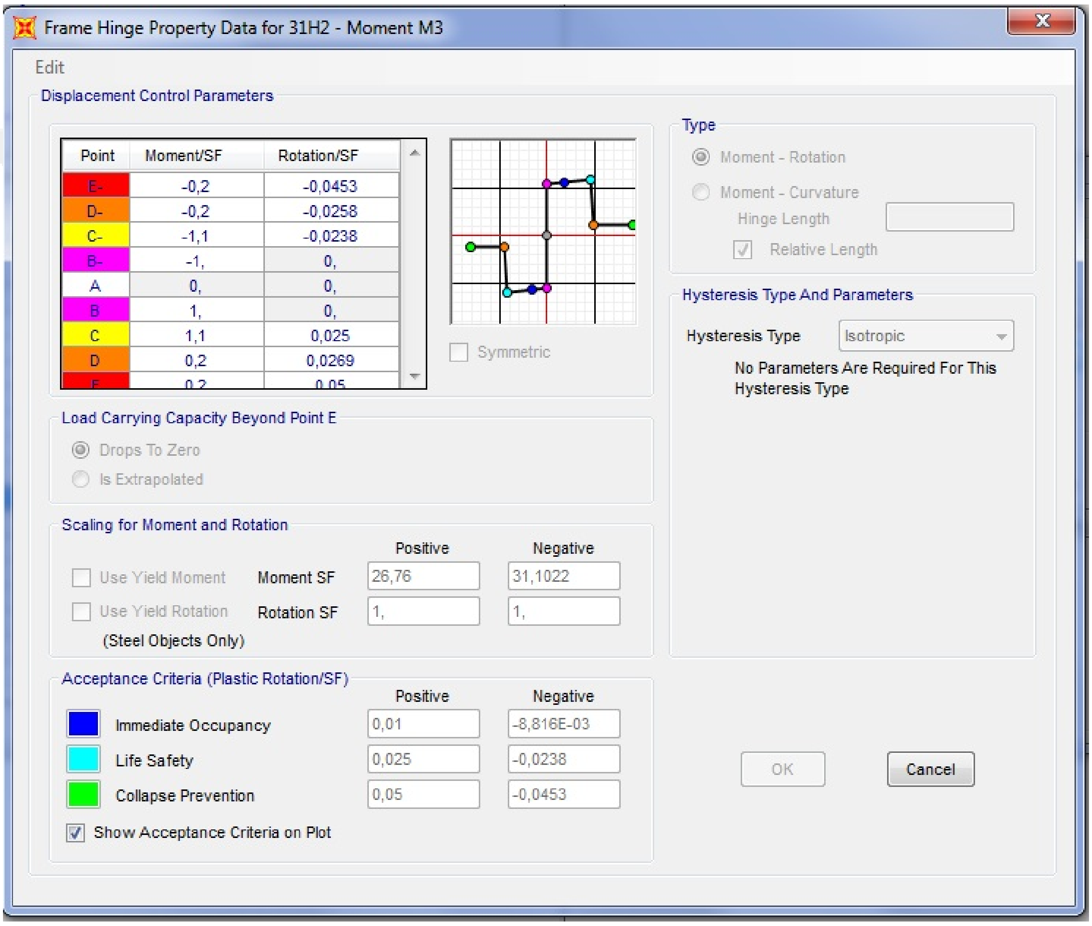

Figure 6.6 Hinge Properties for Beams

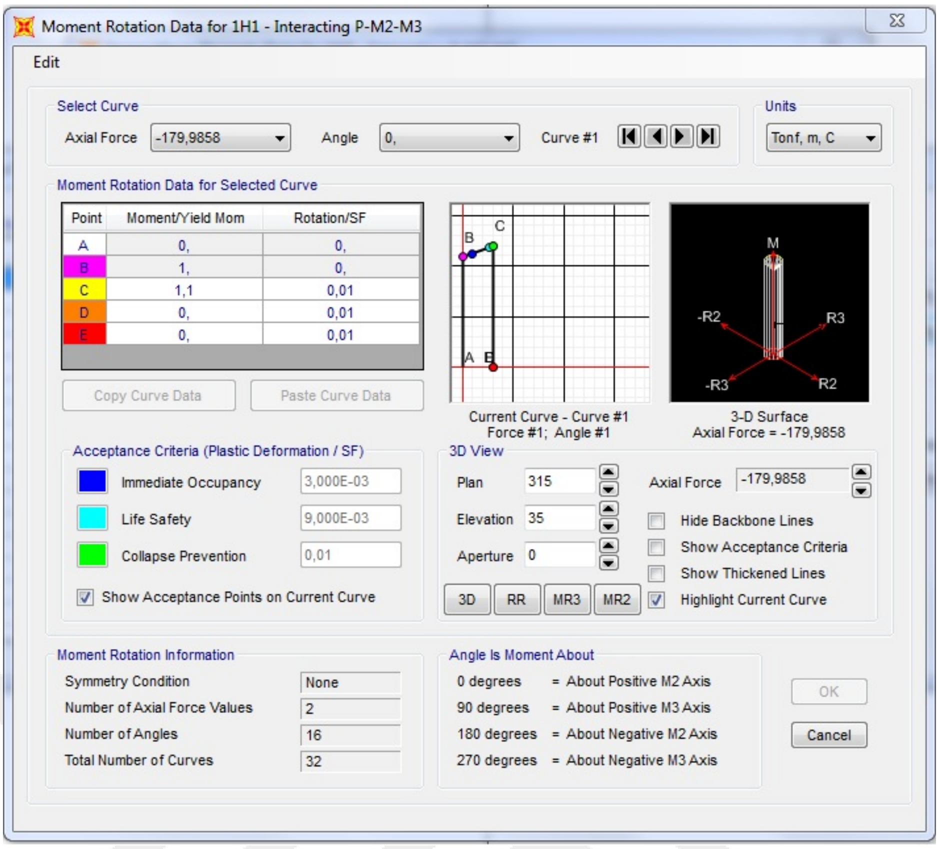

Figure 6.7 Hinges' definition

Also, we can see the plots of pushover analysis for two-dimensional model with XPDs in Figures 6.7 and 6.8

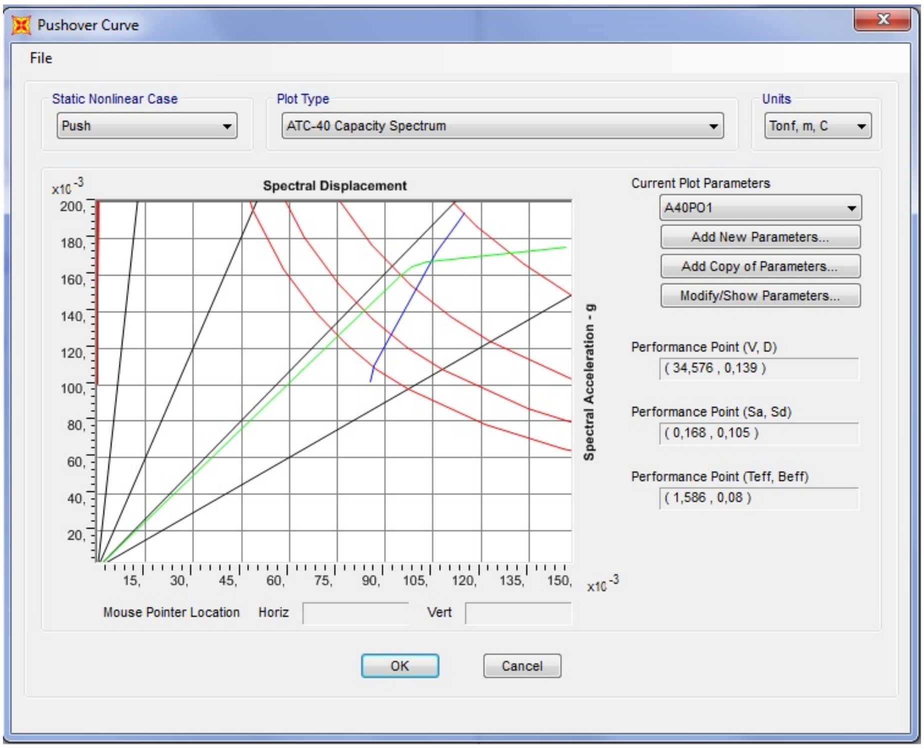

Figure 6.8 Pushover plot for two dimensional model with XPDs

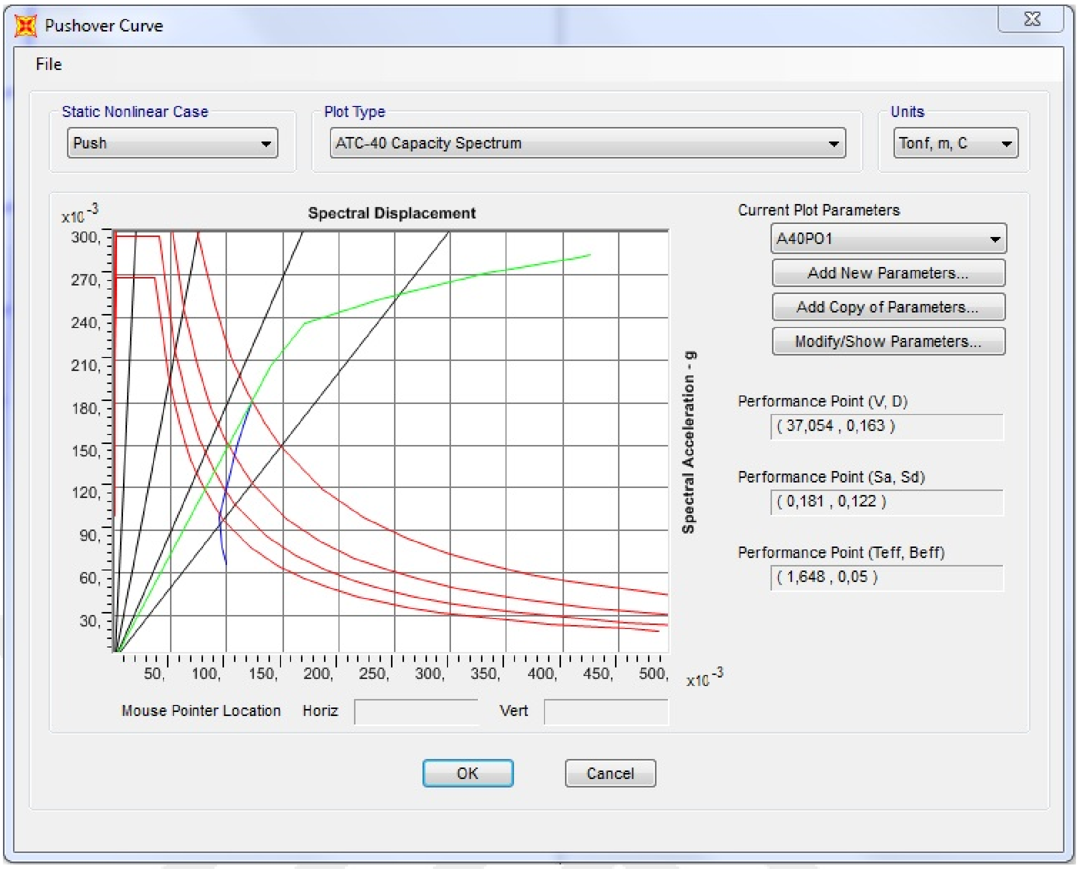

Figure 6.9 Pushover plot for two-dimensional model without XPDs

6.4 Results and Comparisons for the two-dimensional models using Sap2000 analysis

Tables 6.11 and 6.12 show the results of two-dimensional model analysis for all types of rubber concrete used with and without XPDs.

Table 6.11 Results of pushover analysis of two-dimensional models without XPDs

Parameters	Normal	5%	10%	15%	20%
V (tonf)	37.054	32.199	28.808	27.612	24.424
D (cm)	16.3	13.2	15.7	17.3	20.8
S_a (m.sec^{-2})	0.181	0.157	0.14	0.134	0.119
S_d (m)	0.122	0.1	0.119	0.13	0.157
T_{eff} (sec)	1.648	1.603	1.845	1.974	2.303
B_{eff}	0.05	0.095	0.087	0.079	0.073
A-IO HN	77	85	82	104	66
IO-LS HN	68	74	58	160	101

Table 6.12 Results of pushover analysis of two-dimensional models with XPDs

Parameters	Normal	5%	10%	15%	20%
V_{x} (tonf)	37.262	34.576	31.164	29.534	25.508
D_{x} (cm)	16.4	13.9	16.5	18.1	21.5
$S_{a\,x}$ (m.sec^{-2})	0.181	0.168	0.151	0.143	0.124
$S_{d\,x}$ (m)	0.123	0.105	0.124	0.136	0.161
$T_{eff\,x}$ (sec)	1.654	1.586	1.817	1.954	2.293
$B_{eff\,x}$	0.05	0.08	0.069	0.064	0.065
A-IO HN_{x}	77	0	144	123	119
IO-LS HN_{x}	82	80	159	161	172

A comparison of base shear force comparison for two dimensional models is illustrated in Figure 6.9

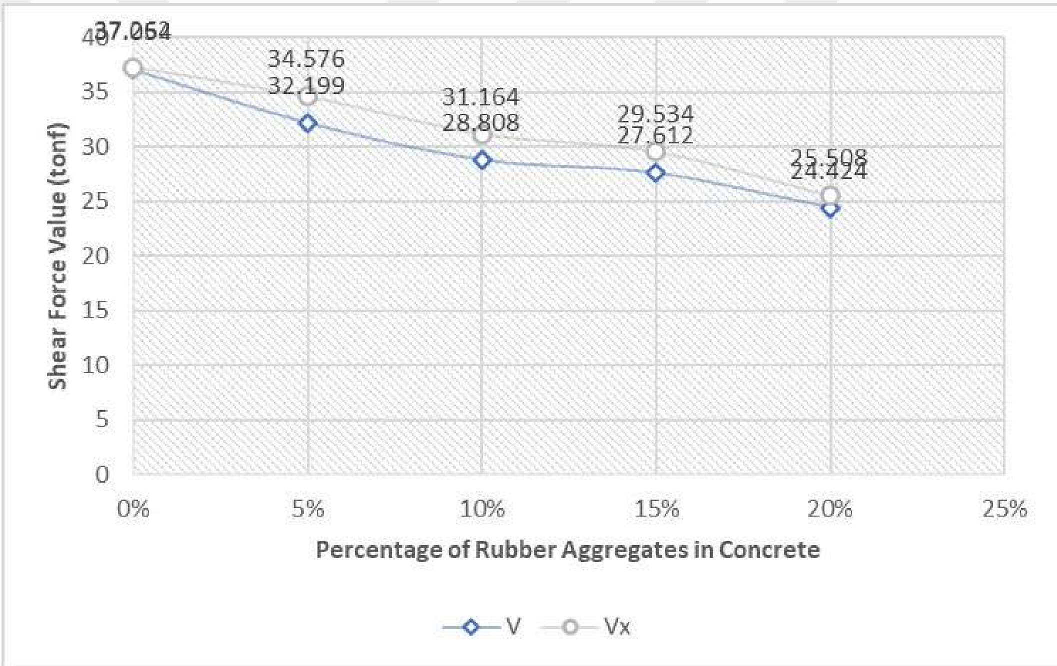

Figure 6.10 Base shear force comparison for two dimensional models with/out XPDs

Figure 6.10 shows results of performance curve displacement values for two dimensional models

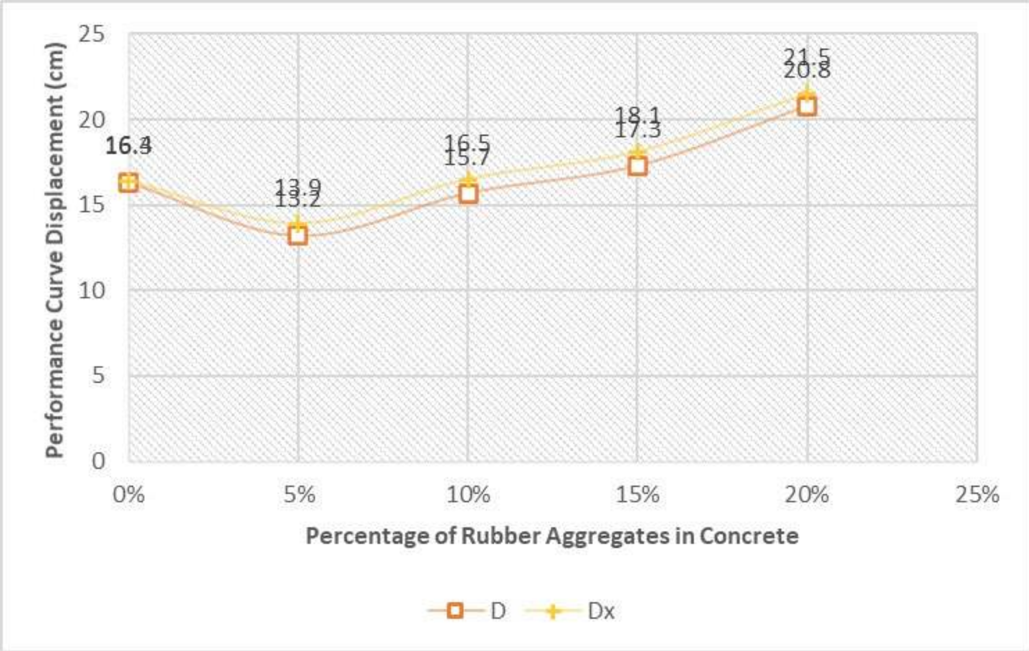

Figure 6.11 Performance curve displacement comparison for 2D models with/out XPDs

A comparison of spectrum acceleration for two dimensional models is illustrated in Figure 6.11

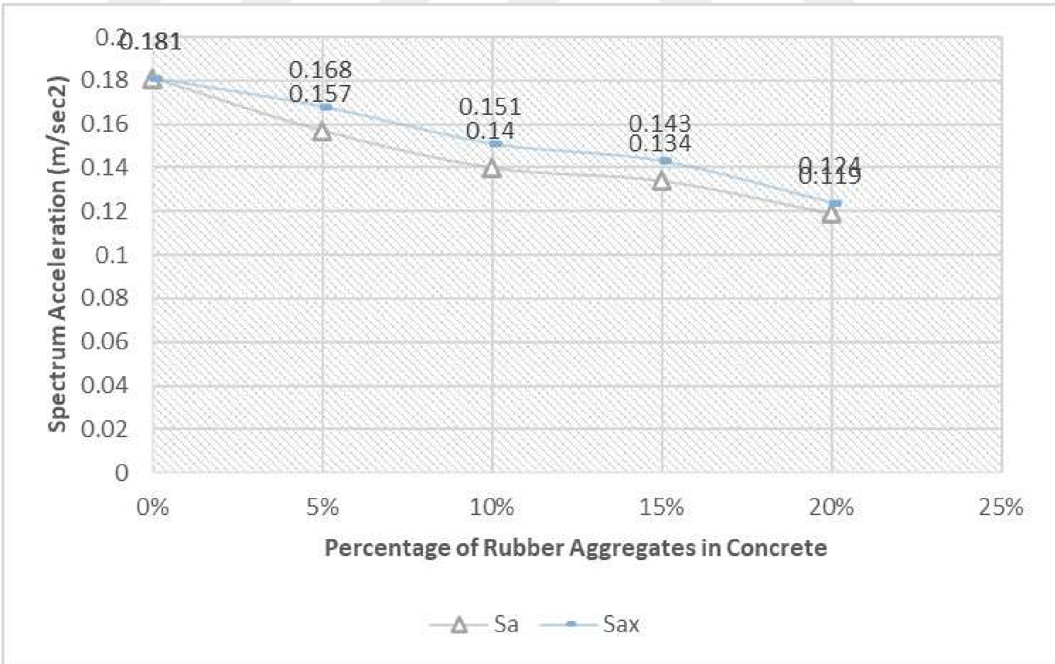

Figure 6.12 Spectrum acceleration comparison for two dimensional models with/out XPDs

Figures 6.12 and 6.13 show results of spectrum displacement and effective period values for two dimensional models.

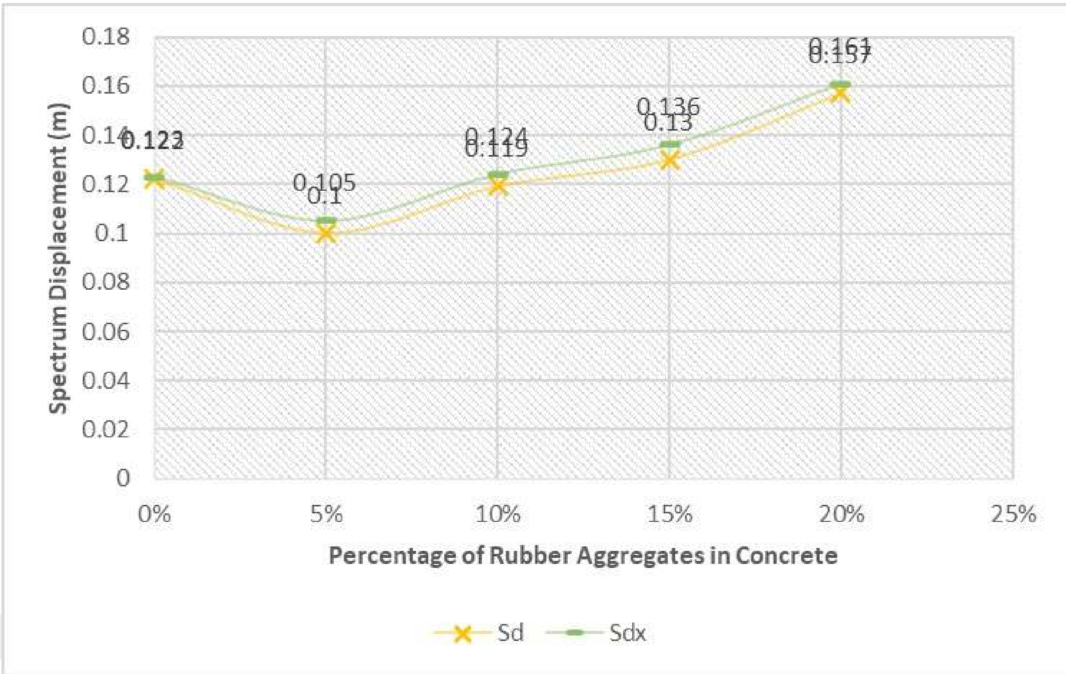

Figure 6.13 Spectrum displacement comparison for two dimensional models with/out XPDs

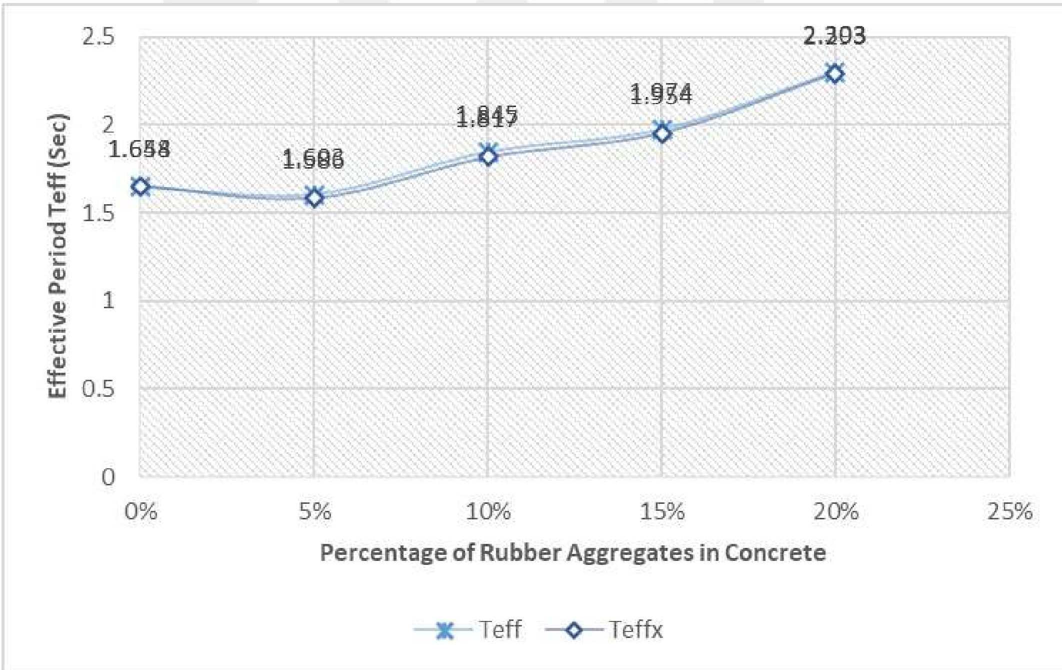

Figure 6.14 Effective period comparison for two dimensional models with/out XPDs

A comparison of effective damping ratio and number of formed hinges for two dimensional models are illustrated in Figures 6.14, 6.15 and 6.16

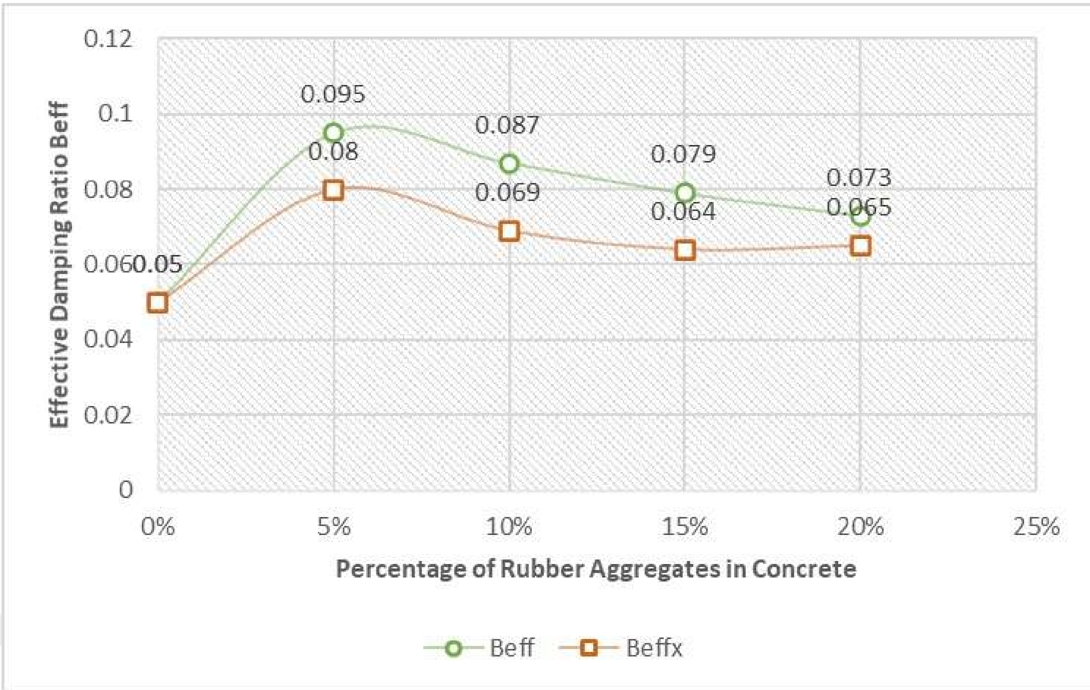

Figure 6.15 Effective damping ratio comparison for two dimensional models with/out XPDs

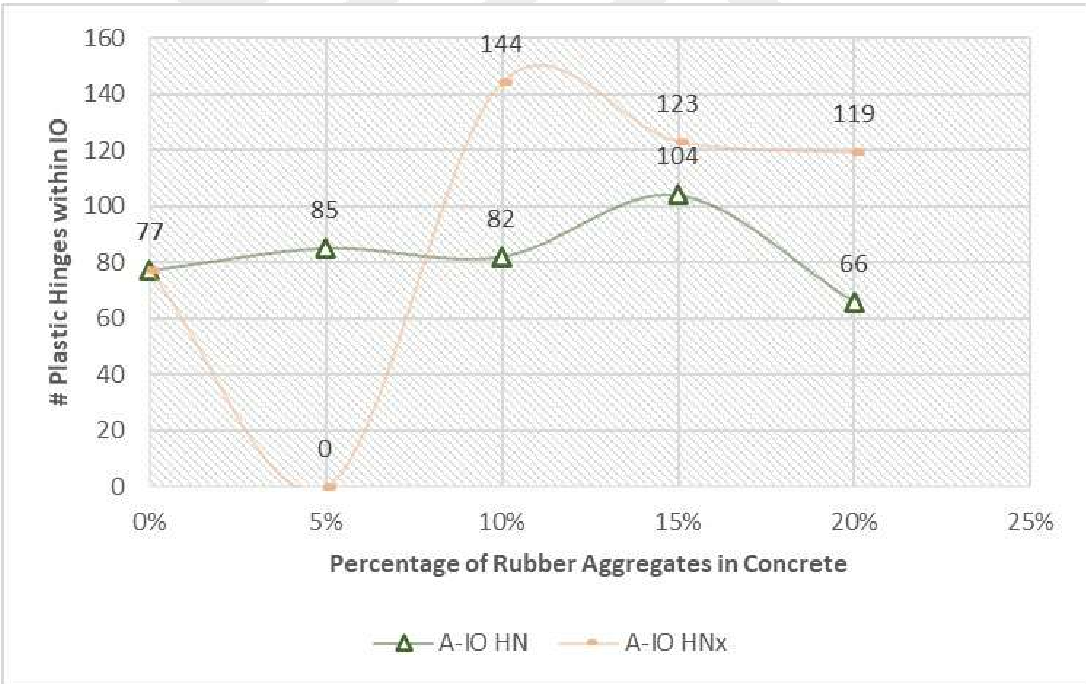

Figure 6.16 # Formed plastic hinges within IO comparison for two dimensional models with/out XPDs

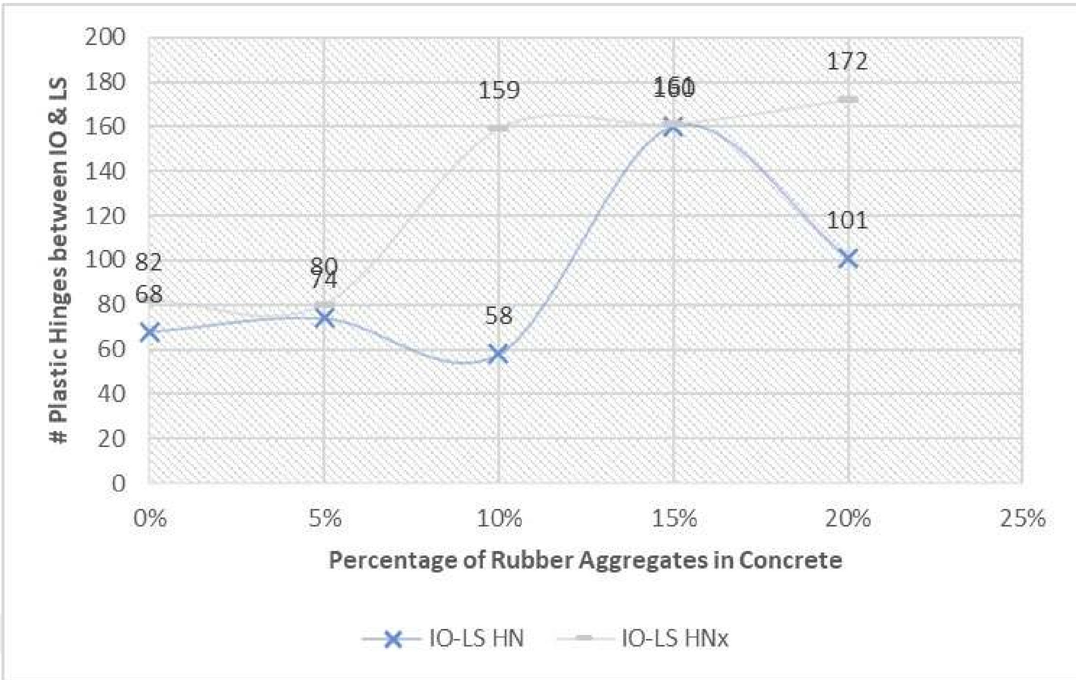

Figure 6.17 # Formed plastic hinges between IO & LS comparison for two dimensional models with/out XPDs

Here we can see that base shear forces of the performance points are decreasing by increasing crumbed rubber ratio in concrete aggregates for both with/out XPDs models, Figure 6.9 also shows that models with XPDs transmit lager number of forces to the base for -with XPDs models- than -without XPDs models-.

Same approach is projected to displacements of the performance points of the models except that displacements are increasing by increasing crumbed rubber in concrete aggregates for both with/out XPDs models.

Also, from Figures 6.13 and 6.14 it seems that Effective Periods are increasing proportionally with increasing ratio of crumbed rubber but still too close to each other comparing as with/out XPDs models, while Effective Damping Ratio decreases by increasing rubber ratio, yet the Effective Damping Ratio at the same rubber ratio for -with XPDs models- is lower than those of without XPDs.

Figures 6.15 and 6.16 show that hinges formed in both IO criteria or even between IO and LS for all models are arbitrary and not bounded to damping type of the models so it will not be taken as comparison criteria in 3D models. From the discussed herein above we will notice that adding XPDs to two dimensional models did not bring enhancement results for the behavior of the structure against the earthquake response spectrum function and that may be for the following reasons:

- Adding chevron braces with XPDs in the two bays for all stories will increase mass and loads in the system as a whole which will affect transition, distribution of loads, and increasing stiffness of the model. Thus, it will increase the transmitted base shear forces and displacements.
- Increasing stiffness of the model will delay plasticity behavior of the model when conduction pushover analysis.
- Restraining stories number of models by 10 is not enough to show fair results for nonlinear analysis and plastic behavior.

6.5 3D models specifications

Like the two-dimensional models, the XPDs or ADAS will be added to the structural system of the 3D models, now we will see wider range of results applied for 3D models where the analysis will be done by ETABS Program. Where the 3D models will be compatible with the following:

- The proposed model will consist of symmetrical 26 stories with 3 bays in X and 3 bays Y directions.
- Wen Plasticity dampers (XPDs) will be added to the 3D models in the middle bays in X and Y directions as first step, next step added (XPDs) will be removed from the middle bays and will be added to corner bays.
- Chevron braces without XPDs will be considered in the analysis as a comparison like it is shown in Table 6.13.
- The first and second step will be repeated for all rubber concrete ratios used in this thesis which explained in previous chapters.
- In 3D models the Response Spectrum will be amplified for each model by scale factor to reach the ultimate performance point for each case separately.

Here Figures 6.18 and 6.19 show the geometrical definitions of one of the 3D models with the chevron XPDs in the middle bays.

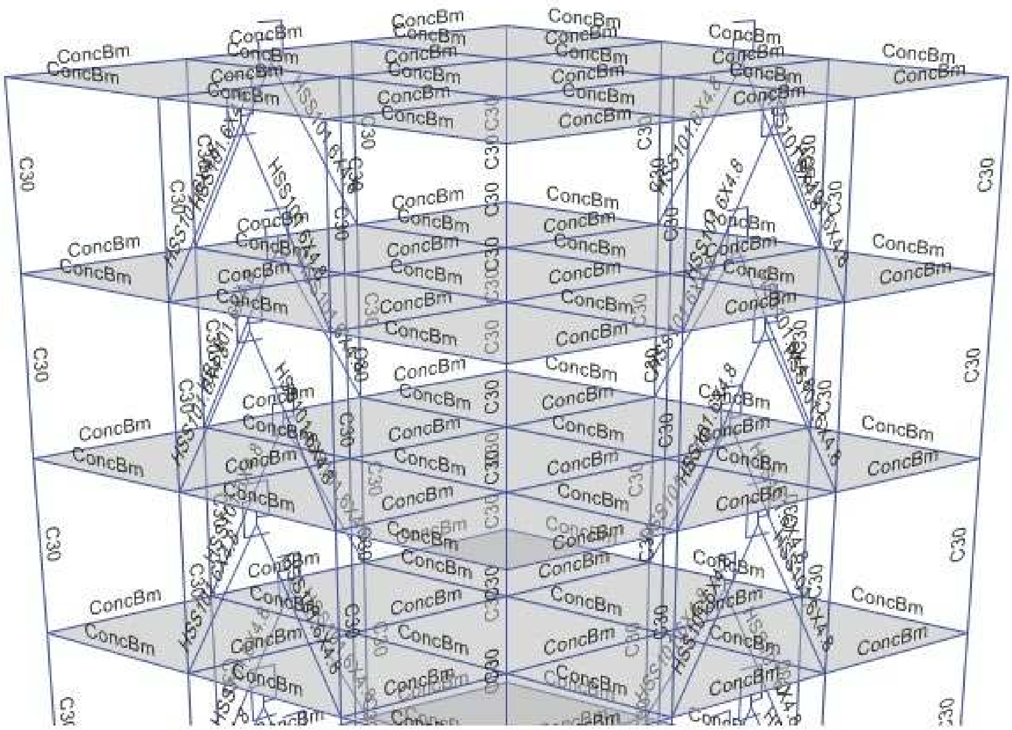

Figure 6.18 Sections, bays and heights of the 3D model with middle XPDs

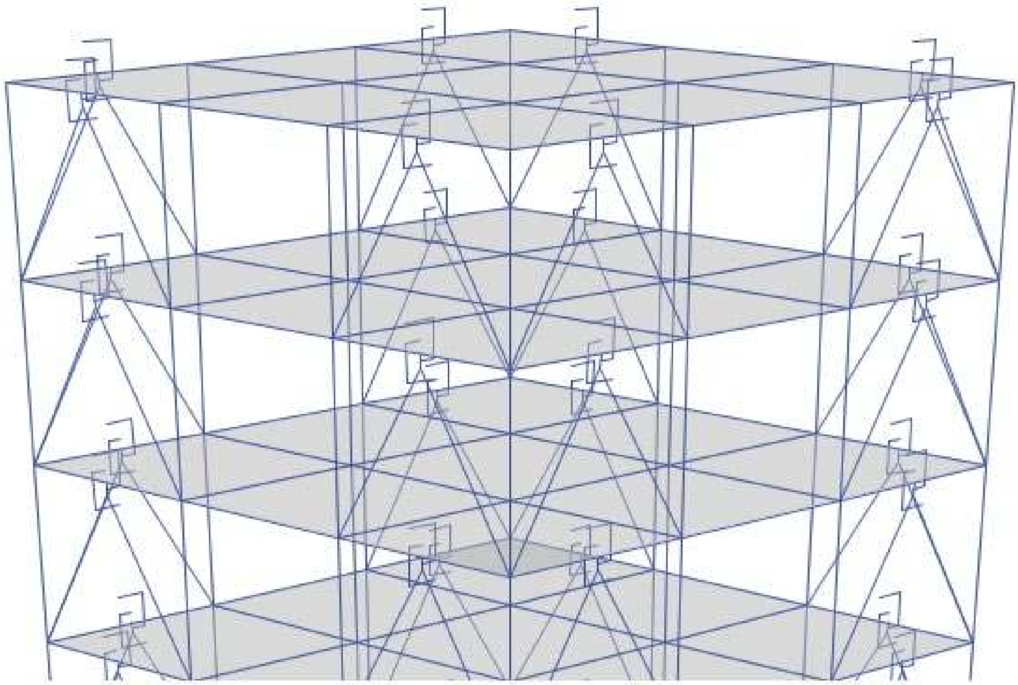

Figure 6.19 Bays and heights of the 3D model with Corner XPDs

Table 6.13 shows the types of 3D models used on the research.

Table 6.13 Definitions of 3D models

Crumbed Rubber Ratio in Concrete Aggregates %	**0 %**	**5 %**	**10 %**	**15 %**	**20 %**
Model Code	**Type of Braces and Dampers Used in the Model**				
ND	NA				
XB_1	1 Middle Chevron Brace without XPDs				
XB_2	2 Corner Chevron Braces without XPDs				
XD_1	1 Middle Chevron Brace with XPDs				
XD_2	2 Corner Chevron Braces with XPDs				

- Geometrical specifications are shown as in Table 6.14

Table 6.14 3D Models' geometrical specification

Parameters	**Value**	**Number**
Bay Dimension, X Direction (m)	4	3
Bay Dimension, Y Direction (m)	4	3
Story Hight (m)	3	26

- Materials used in the models are same as defined for two dimensional models and in **Chapter III** for crumbed rubber percentage in the aggregates of used concrete.
- Sections are rectangular for all elements used in the models, they are fixed regardless the type of model and designing aspects as the target of the research is to compare the results of changing the materials and damping systems proposed, as in Table 6.15.

Table 6.15 Sections' definitions and locations for 3D models

Section	**Hight** *(cm)*	**Width** *(cm)*	**Location** *(Story)*	**Number of elements**
C40	40	40	1 to 10	160
C35	35	35	11 to 19	144
C30	30	30	20 to 26	112
ConcBm	50	30	All Stories	624
HSS101.6*4.8 (Circular)	10.2	4.8	Middle/Corner Bays	208/416
Slab 1 (thin shell)	20		1 to 26	26

- Hinges properties, ranges and acceptance criterias are same as defined in the two dimensional models as Tables 6.4, 6.5, 6.6 and 6.7 shows for both types M3 and P-M2-M3 Hinges.
- Mass Sources: the mass sources of were models taken from elements self mass and additional mass and 1 multiplier of assigned super dead loads and 0.25 multiplier of assigned live loads.
- Load Patterns used in the 3D models are as in Table 6.16.

Table 6.16 Load patterns used in 3D Models

Load Pattern Name	Type	Self-Weight Multiplier	Location *(Story)*
Dead	Dead	1	All elements for self-weight/Shells for Dead
LL	Live	0	Shell elements
Push	Other	0	All elements

- Response Spectrum Functions: the response spectrum defined for all 3D models was taken according to American Society for Civil Engineering ASCE7-16 code with those parameters with Tonf, m, C units, as in Table 6.17.

Table 6.17 Design response spectrum parameters used in 3D models

Parameters Name	Value
0.2 Sec Spectral Accel, Ss $(m.sec^{-2})$	2.29
1 Sec Spectral Accel, S1 $(m.sec^{-2})$	0.869
Long Period Transition (sec)	8
Site Class	C
Site Coefficient, Fa	1
Site Coefficient, Fv	1.3
SDS=2/3*Fa*Ss $(m.sec^{-2})$	1.5267
SD1=2/3*Fv*S1 $(m.sec^{-2})$	0.7531

- Load cases:
    - **Dead load** was defined as linear static load and applied for all models as self-weight load for all structural elements in addition to 2 KN/m^2 for each Area element (Slabs).
    - **Modal load** was defined as Eigen vectors with zero initial conditions and the defined mass sources of the model.

- **Live load** defined as linear static load and applied as 2 KN/m^2 for all Area elements (Slabs) of the models with zero initial conditions.
- **Push Over Load** which is a nonlinear static typecontinue from the state at end of another nonlinear case which is Gravity nonlinear, where all modal loads applied using modes from the defined modal load with the same mass sources, load application will be Displacement Control with multiple states.
- **Garvity Nonlinear load** is nonlinear static loadcase starting from zero initial conditions (Unstressed State) contains 1 Dead, 0.25 Live and 1 Super Dead load patterns, the load will be applied fully with final state result.

- Load combinations will be taken as the default ETABS 17 conbination for concerete RFS (Rigid Frame Structures).
- Analysis Options were taken to be 3D analyzing with advanced solver with auto process.
- Wen plasticty dampers specifications (XPlate Dampers) are functional into U2 direction with nonlinear properties for XPDs which acting in X axis and fixed for all other degrees of freedom, otherwise, all XPDs are functional into U3 direction with nonlinear properties for XPDs which acting in Y axis and fixed for all other degrees of freedom, those properties are given are in Table 6.18.

Table 6.18 Parameter of Wen (XPDs) used in 3D models

Parameters Name	Value
n (Number of plates)	8
E (Young Modulus od plates) (N.mm^{-2})	199947.98
b (Width of plates) (mm)	100
t (thickness of plates) (mm)	8
a (height of plates) (mm)	100
σy (Yielding strength of plates) (N/mm^2)	248.2113
σu (Failure strength of plates) (N/mm^2)	399.896
H (rate of strain hardening)	3450.95
d (yield displacement) (mm)	30
Kd (the initial stiffness) (N.mm^{-1})	6824.89
Fy	21180.69
q (yield displacement) (mm)	3.103448
Y_0	0.413793
P (plastic force for d displacement) (N)	31650.50
Kp (plastic stiffness) (N.mm^{-1})	1055.016
Kp/Kd	0.154583

6.6 Results and Comparisons for the 3D models using ETABS analysis

Figures 6.20 and 6.21 show the plot of pushover analysis of one of the 3D models (XD_2_R15%)

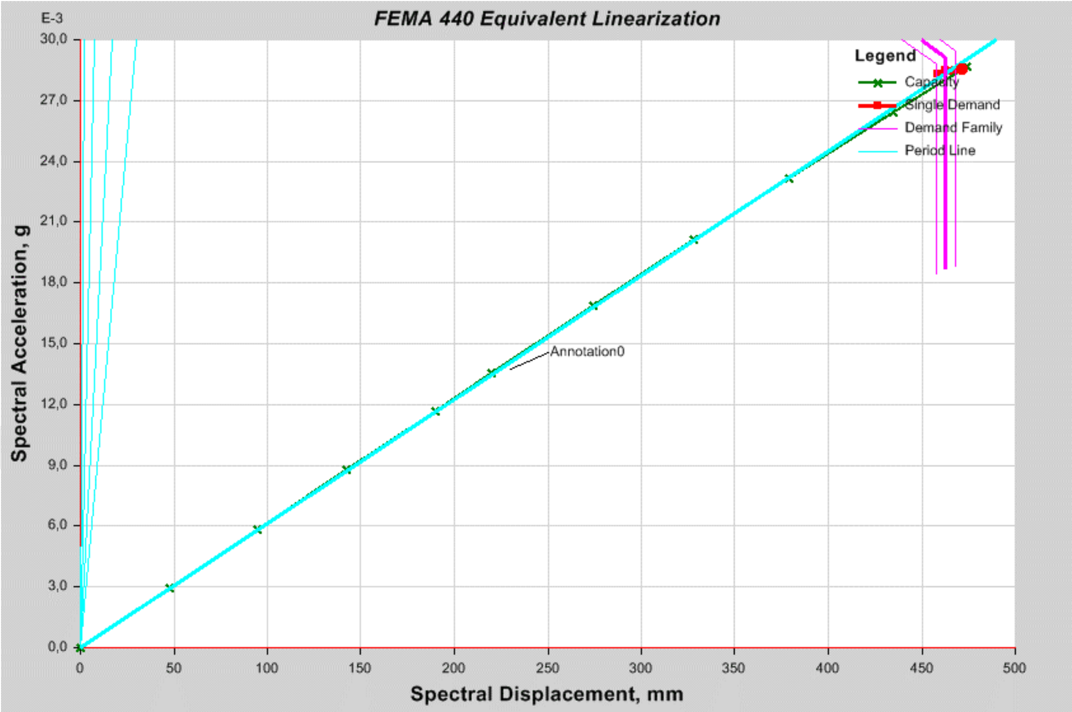

Figure 6.20 Pushover plot for model XD_2_R15%

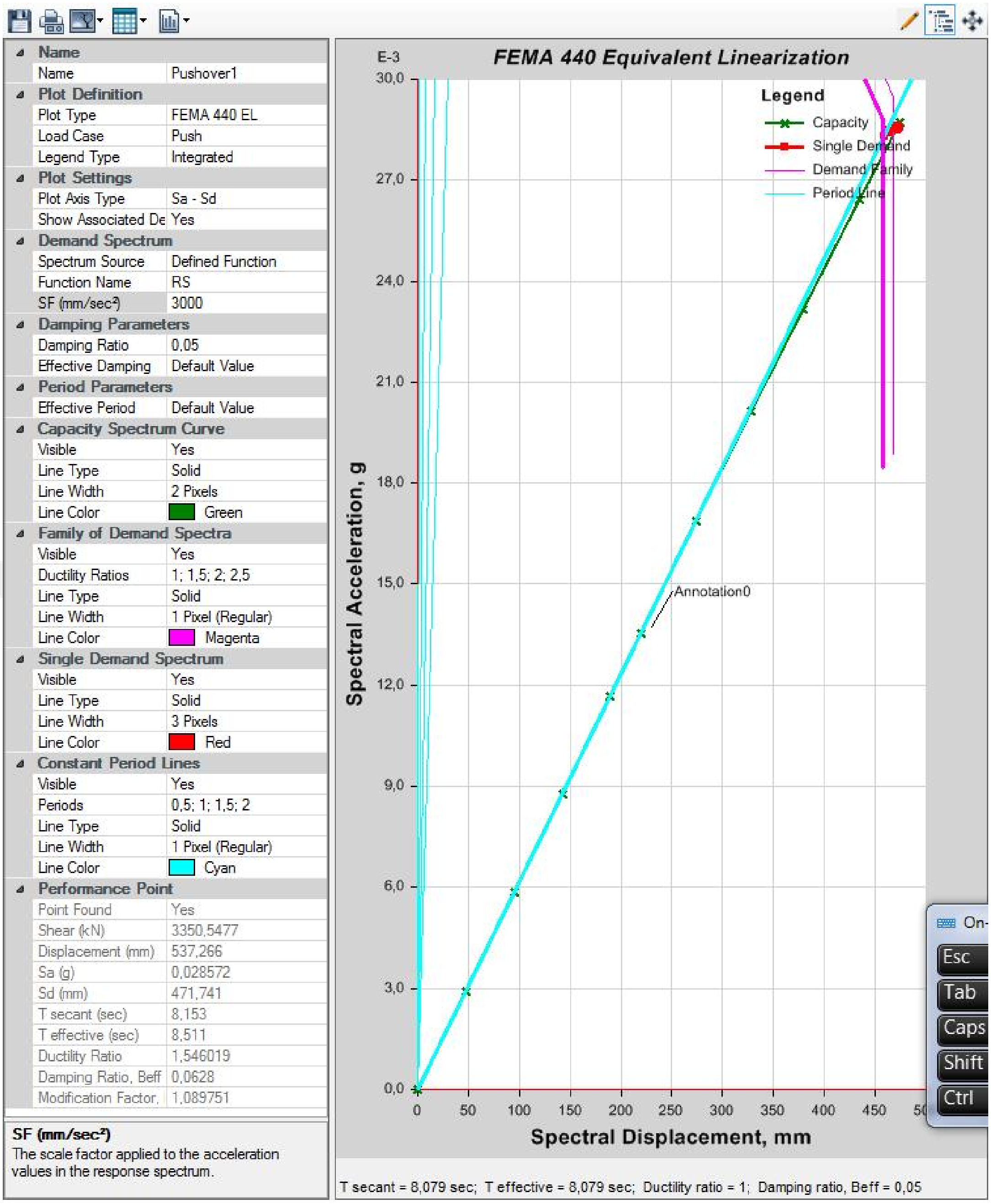

Figure 6.21 Pushover plot with detailed for model XD_2_R15%

Figure 6.22 shows the definition of design response spectrum.

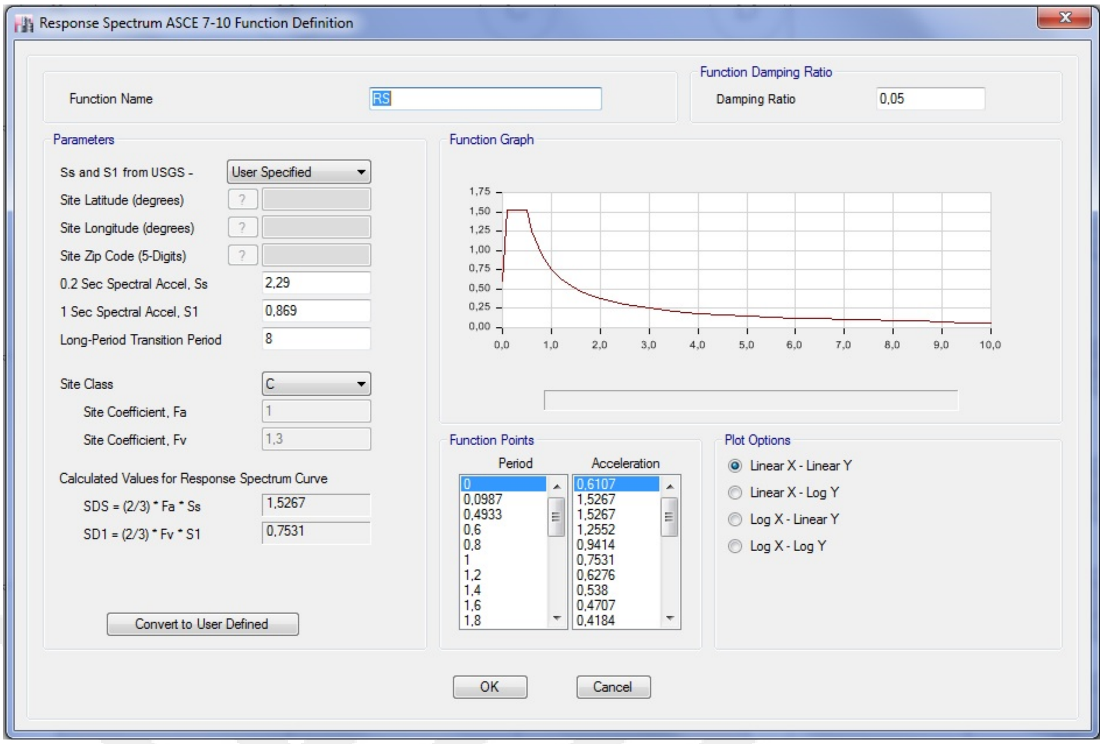

Figure 6.22 Response spectrum definition in 3D Models

Table 6.19 shows the results and comparison of all 3D models pushover performance point parameters, base shear forces, roof displacements, pseudo acceleration, pseudo displacement, periods, effective periods, ductility ratios and damping ratios.

Table 6.19 Results of pushover analysis of 3D models with/out XPDs

Model Code	SF	Shear (KN)	Dis (mm)	Sa (g)	Sd (mm)	T sec (Sec)	T eff (Sec)	Duct %	Beff %
ND	2975	4213	596	0.03575	526	7.701	9.584	2.406	11.63
NDR_5%	2725	3724	458	0.03163	404	7.173	8.07	2.2	8.93
NDR_10%	2775	3200	497	0.02729	437	8.035	8.497	1.656	6.80
NDR_15%	3150	3013	539	0.02579	473	8.6	8.685	1.368	5.63
NDR_20%	3400	2557	596	0.02195	524	9.806	9.941	1.291	5.39
XB_1	3250	6251	696	0.05340	612	6.794	9.613	2.897	15.12
XB_1_R5%	2800	4560	442	0.03891	388	6.337	7.425	2.14	9.74
XB_1_R10%	3200	4497	544	0.03857	477	7.058	7.965	1.94	8.46
XB_1_R15%	3675	4955	678	0.04261	593	7.489	8.601	2.012	8.88
XB_1_R20%	4175	4485	786	0.03878	687	8.444	9.216	1.761	7.35
XB_2	3100	6058	614	0.05210	538	6.451	8.913	2.775	14.28

XB_2_R5%	2900	6113	562	0.05245	493	6.155	8.689	2.885	15.04
XB_2_R10%	3725	8608	998	0.07422	865	6.852	11.12	3.535	18.56
XB_2_R15%	6575	19873	2483	0.16566	2172	7.281	13.95	6.332	20.71
XB_2_R20%	8650	23467	3855	0.20400	3365	8.149	16.121	6.69	20.48
XD_1	2900	3837	494	0.03260	435	7.333	8.095	1.89	8.11
XD_1_R5%	2250	3080	349	0.02614	307	6.88	7.356	1.73	7.19
XD_1_R10%	3075	3801	555	0.03236	488	7.794	8.513	1.814	7.66
XD_1_R15%	3150	3488	574	0.02977	504	8.262	8.806	1.666	6.85
XD_1_R20%	5475	4826	1046	0.04130	918	9.461	10.545	1.869	7.98
XD_2	2825	3727	469	0.03165	412	7.246	7.865	1.805	7.60
XD_2_R5%	2675	3801	429	0.03221	378	6.879	7.652	1.94	8.42
XD_2_R10%	3125	3871	553	0.03293	486	7.71	8.342	1.77	7.40
XD_2_R15%	3000	3350	537	0.02858	471	8.153	8.511	1.546	6.28
XD_2_R20%	4225	3637	758	0.03113	665	9.276	9.653	1.5	6.10

Figures 6.23 and 6.24 show a comparison of ground shear forces and performance curve displacement of the 3D analyzed models.

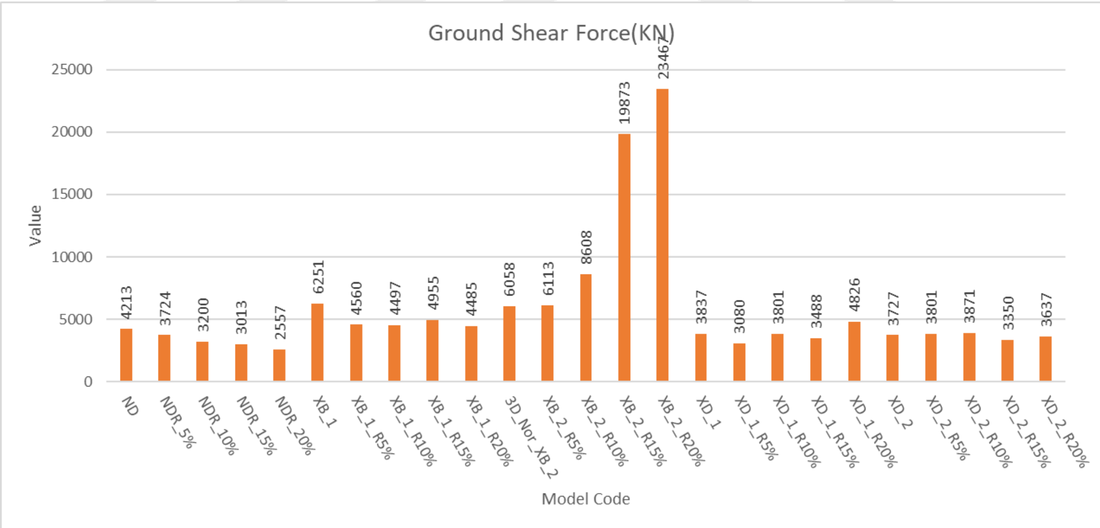

Figure 6.23 Ground shear force comparison for 3D models with/out XPDs

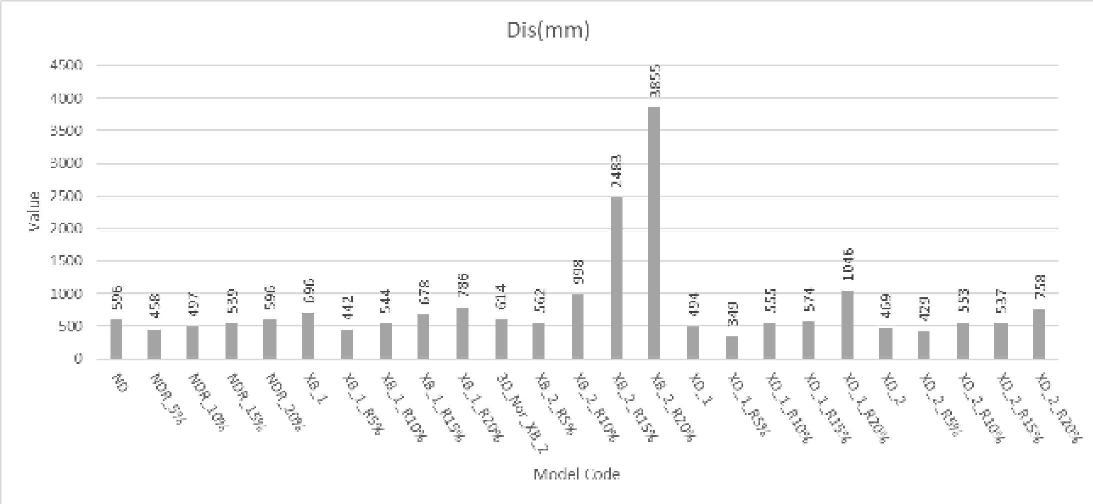

Figure 6.24 Performance curve displacement comparison for 3D models with/out XPDs

Figures 6.25 and 6.26 show a comparison of pseudo acceleration values and pseudo displacements of the 3D analyzed models.

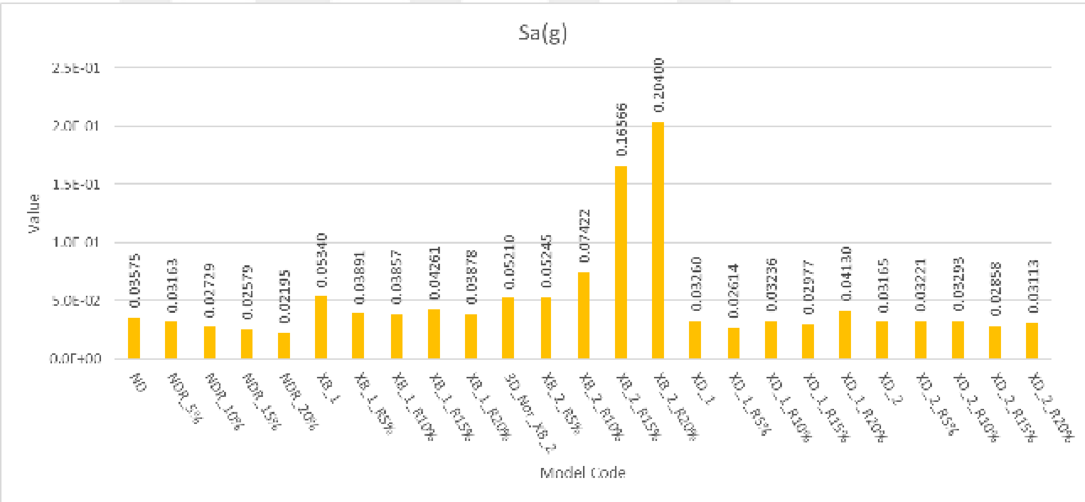

Figure 6.25 Spectrum acceleration comparison for 3D models with/out XPDs

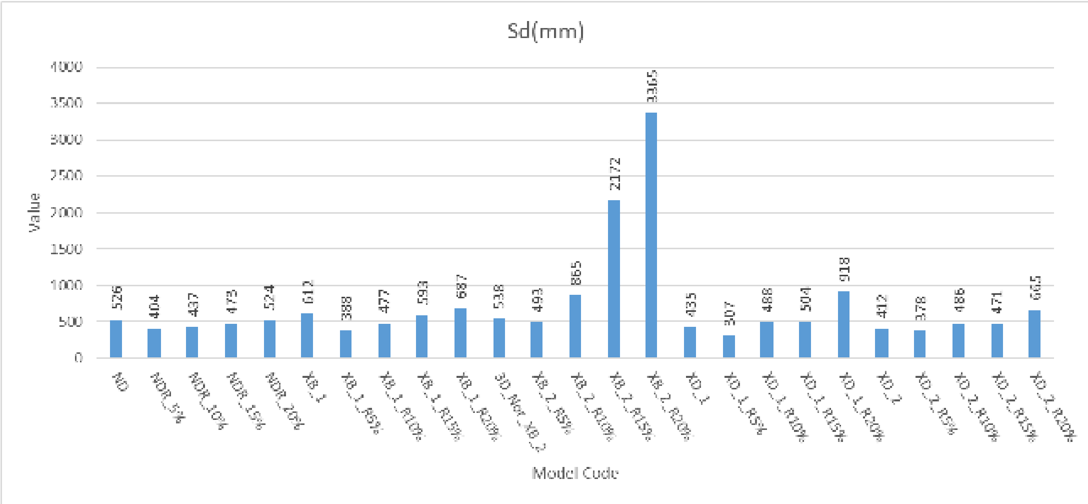

Figure 6.26 Spectrum displacement comparison for 3D models with/out XPDs

Figures 6.27 and 6.28 show a comparison of design response spectra amplifying scale factor values and the values of effective period of the 3D analyzed models.

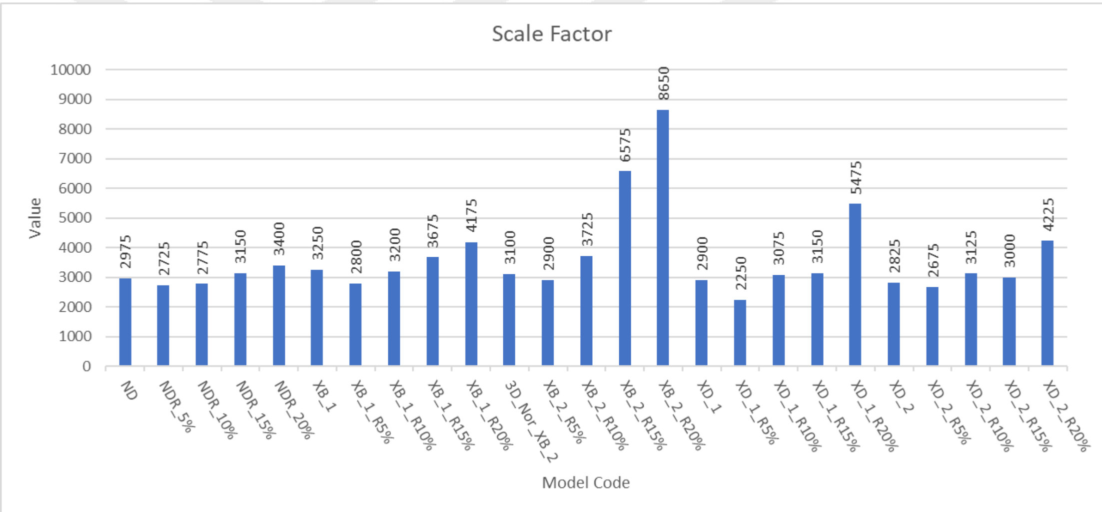

Figure 6.27 Scale factor of applied response spectrum comparison for 3D models

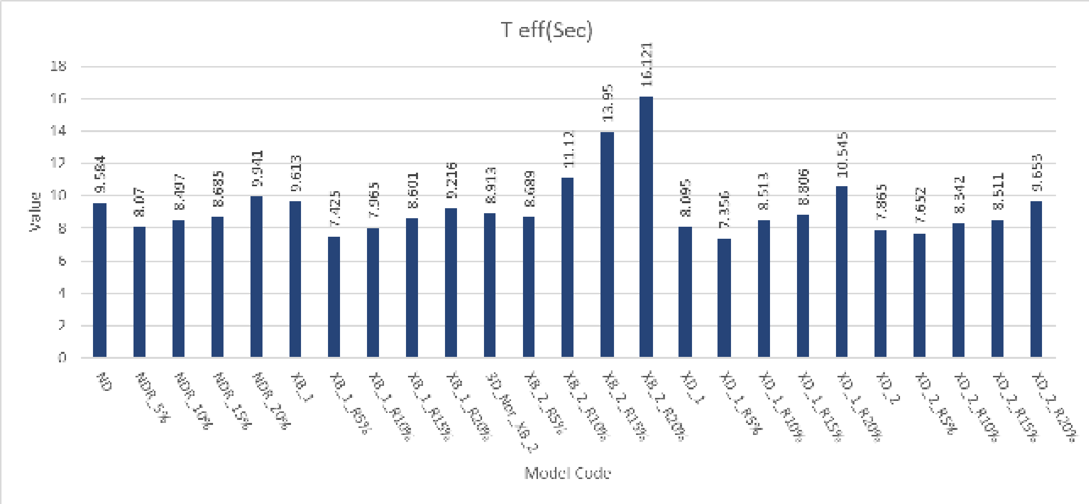

Figure 6.28 Effective period comparison for 3D models with/out XPDs

Figures 6.29 and 6.30 show a comparison of ductility ratios and the effective damping values of the 3D analyzed models.

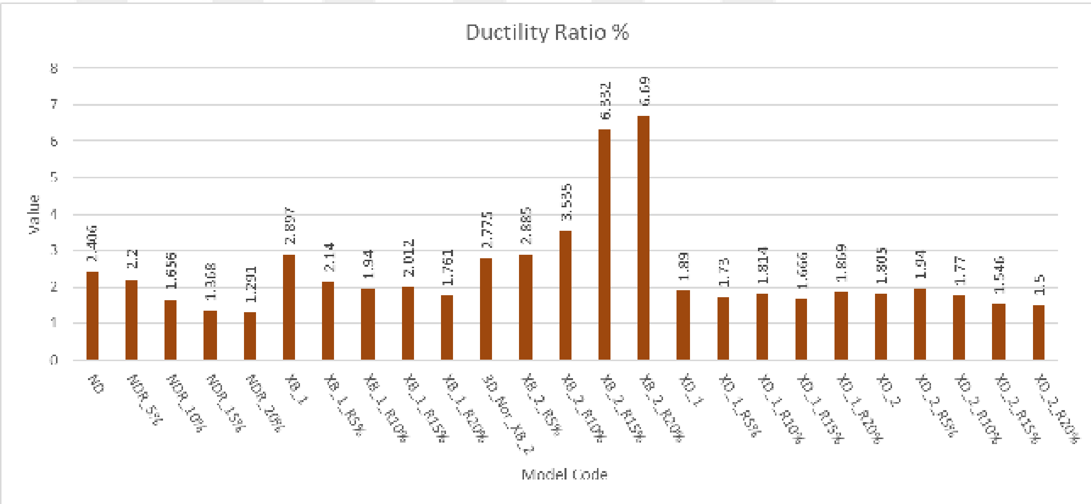

Figure 6.29 Ductility ratio comparison for 3D models with/out XPDs

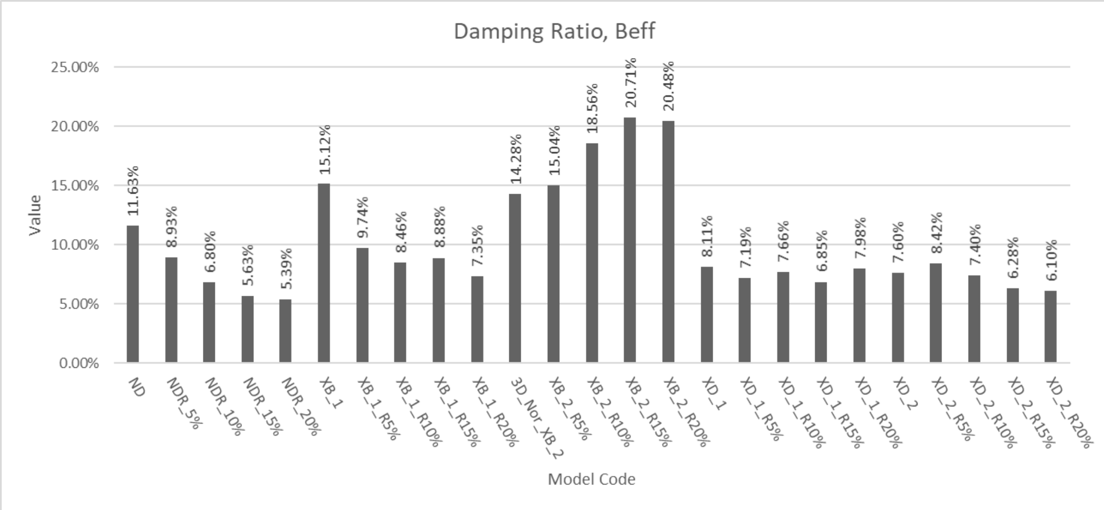

Figure 6.30 Effective damping ratio comparison for 3D models with/out XPDs

Figures from 6.23 to 6.30 show that models with corner chevron braces (XB_2 group) can bear extremely earthquakes with large, transmitted base shear forces, roof displacements for the performance point, high ductility and effective damping ratio, on the other hand, high values of pseudo acceleration and displacement which are nonrealistic and not preferable in building designs and cause collapsing to nonstructural elements.

For models without braces or XPDs (NRD group) we can see that pseudo acceleration values of performance point, base shear forces, ductility ratio and effective damping ratio are decreasing by increasing crumbed rubber concrete in the aggregates. Meanwhile pseudo displacement, effective period and scaling factors of defined earthquake response spectrum increase. Same approach is applied for models that have middle chevron brace without XPDs (XB_1 group), yet scale of all comparison parameters is a bit larger than those without braces or XPDs.

The diagrams of models with middle and corner chevron XPDs show that effective damping ratio, ductility ratio is increasing proportionally with increasing of crumbed rubber ratio in aggregates, meanwhile effective period, scale factor of defined earthquake response spectrum, pseudo accelerations and displacements and base shear forces and displacements for performance curves increase.

Comparing models of middle chevron XPDs with corner chevron XPDs we can notice slight enhancement in the behavior of middle chevron XPDs models than the corner XPDs ones, whereas, values of effective damping ratio, ductility ratio are slightly higher. In the same time, values of effective period, scale factor of response spectrum, pseudo acceleration, pseudo displacement, base shear force and displacement of performance points are slightly lower in the middle XPDs models than the corner ones.

As general overview for the results of 3D models, models with corner chevron braces without XPDs -especially for 15% and 20% of rubber ratios- give unrealistic results comparing to other types. Models with middle XPDs and corner XPDs can stand more severe earthquakes since scale factor for the response spectrum reaches maximum values with minimum values of transmitted base shear force, displacement, pseudo acceleration, pseudo displacement and effective period especially the models with 10% and 15% ratio of added crumbed rubber models. Thus, this leads us to conclude that an optimum situation can be obtained from mixing the damping systems such adding crumped rubber to aggregates of concrete or installing other types of dampers like the XPDs or viscous liquid dampers or mass dampers, etc

CHAPTER VII

CONCLUSION

During the last three decades, great attention has been paid to increase and develop buildings and high structures' resistance against earthquakes, and there is no doubt that the development of damping systems in structures is one of the most important and most used mechanisms, unless they are the most used and popular ones at all, due to their good availability in the market, ease of manufacturing, maintenance and logical costs compared to other structure's resistance -against earthquakes- systems.

We found through this research that the development of some tools and materials that contribute to increasing the effective damping ratio during non-linear static analysis, such as adding treated crumbed rubber to aggregates concrete, and that the combination of these materials by adding extra damping systems or dampers may increase the effectiveness of the effective damping ratio. Consequently, the demand curve of the design response spectrum decreases.

The dampers' locations that were used in the research played a major role in the mechanism of the model's response and behavior towards the design response spectrum curve and also played a major role in raising the scale factor in some cases compared to other models and the ratios of crumbed rubber added to aggregates in the concrete used in the models.

Rubber concrete may be a good option to raise the damping ratio in concrete structures it contributes to absorbing energy by a certain amount, but it is not sufficient as individual, adding extra damping systems can form a hybrid system that gives greater effectiveness to raise the damping ratio of the structure as a whole and thus greater absorption capacity of seismic energy, also proportions of crumbed rubber added should be carefully studied.

The use of engineering simulation programs such as SAP2000 and ETABS, and the mechanisms of linking the damping ratio of assigned materials to structural elements and linking them to the general damping matrices in the structure with other dampers' properties, choosing the comparison mechanisms based on the results of nonlinear analysis such as the scale factor and the base shear force transmitted to the ground, roof displacement, pseudo spectral acceleration, pseudo spectral displacement and effective damping ratio allows structural engineer or designer to move towards choosing the optimal state of the design by preference the most appropriate selection criteria for the design according to the function, investment of the facility as it may be the cost factor, safety factor, or higher value of effective damping ratio or scale factor of applied design response spectrum could be the desired goals to be reached considering that the rest of the comparison criteria can be defined as conditions in the optimization process.

Research in this field still need a lot of development and continuation, so there is no fixed case, as we have seen that the experiment in two-dimensional models did not yield a desired result, and it is also possible to conduct research and scrutiny using other analysis types such as dynamic analysis, time history analysis or nonlinear dynamic analysis. It can be use other systems such as tuned mass dampers, water dampers or viscous liquid damping systems or combined to produce hybrid systems and choose the optimal state for them, which ultimately depends on the unique geometrical case in addition to the conditions and criteria of comparison.

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