DAMPER SYSTEM DESIGN FOR POUNDING EFFECT BETWEEN EXISTING HIGH-RISE BUILDING AND ITS EXTENSION

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Abstract- This paper describes damper system design for pounding between existing building and its extension. In this system, passive viscous dampers are used by which the existing building is connected to its extension. To improve the seismic performance of the adjacent buildings, it is necessary to perform sufficient reinforcement in spaces that are already used. In metropolitan cities such as Yangon and Mandalay, these spaces are very limited and are not sufficient. Furthermore, in high-rise buildings, seismic retrofit is much more difficult not only for the above reasons but also because of the difficulties of the construction work and changes in the structural system due to the structural reinforcements. Seismic pounding is defined as the collision of adjacent buildings during earthquakes due to insufficient separation gap between buildings. Pounding Effect between existing building and its extension is studied by non-linear dynamic analysis. Firstly, the required ground motion records are obtained from PEER ground motion database by using input parameters. Secondly, the adjacent two buildings are separately analysed and the maximum displacements of two buildings at connecting corner points are resulted. Thirdly, two buildings connected with dampers are continuously analysed for pounding effect by using non-linear time history analysis. Finally, it is observed that the maximum displacements between two adjacent buildings by using dampers are reduced as compared to the case of the independent system.

Keywords- Maximum Displacement, Non-linear Time History, PEER Ground Motion Database, Seismic Pounding, Viscous Damper

I. INTRODUCTION

Seismic pounding is defined as the collision of adjacent buildings during earthquakes. The principal reason for seismic pounding is insufficient separation between the buildings. The phenomenon is mostly observed in old buildings that were constructed before the advent and popularity of earthquake resistant design principles. Although many current building codes specify a minimum seismic gap, it is still sometimes inadequate as the codes necessarily lag behind current research and fail to include the effect of other parameters that affect the structural deformation [2]. The gap is often seen as a waste of prime real estate by developers and has been reduced in some newer versions. It also seems to unfairly penalize a property owner whose neighbor has already constructed a building at the boundary of their properties, as the new owner will have to provide the gap to accommodate the relative deformation of both buildings. A different problem is posed by existing buildings which were designed with sound structural methodology but before the prevalence of earthquake-resistant designs. The study raised the awareness in the engineering community for the need to assess the pounding forces, so as to better design new buildings, and retrofit the older constructions to withstand a possible seismic collision with adjoining structures [4]. Three key factors for a good retrofitting method are efficiency, cost and applicability to existing structures. Firstly, a good retrofitting method must be able to reduce the risk of damage to an acceptable level. Another important factor that must be considered is that a good retrofitting method must be economical. And finally, since these methods are to be used for existing structures, it is very crucial to take into consideration the amount of work needed to be done in order to employ these techniques. Among the retrofitting methods, passive coupling of adjacent structures is known to be an effective method to reduce undesirable vibrations and structural pounding effects [1].

II. DESCRIPTION OF THE CASE STUDY

The proposed structure is Palace Medical Centre located in Mandalay City of Myanmar. This hospital includes existing 7-storeyed RC building and 12-storeyed extension steel building. Both buildings have one normal stair and two elevators. The structural dimensions of two buildings are shown in Table-I. Ground floor plans of the proposed structures are shown in Figure 1 and Figure 2. The proposed medical center is located between 71 and 28x29 streets in Chan Aye Thar Zan Township in Mandalay. The required data source is from Mandalay City Development Committee (MCDC).

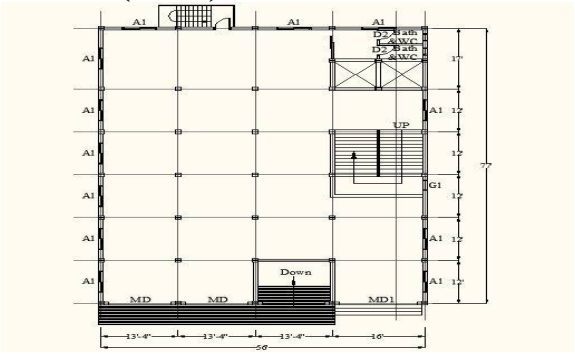


Figure 1 – Ground Floor Plan of 12-storeyed Steel Building

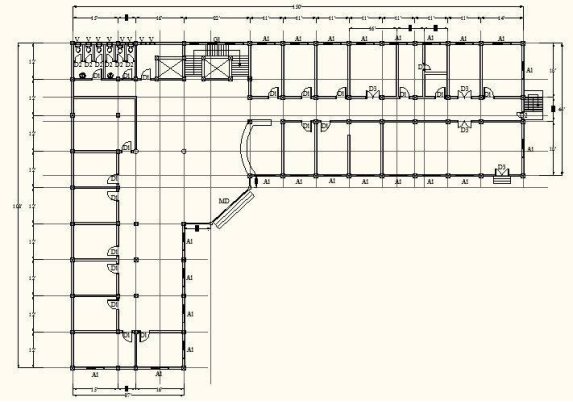


Figure 2 – Ground Floor Plan of 7-storeyed RC Building

Table -I Structural Dimension of Palace medical centre

Type of Buildings	12-storeyed Extension Steel Building	7-storeyed Existing R.C Building
Shape of building	Rectangular shape	L-shape
Length of building	56-ft	154,37-ft
Width of building	77-ft	108,44-ft
Ground floor height	14-ft	12-ft
Typical story height	10-ft	10-ft
Stair roof height	9-ft	7-ft
Foundation depth	15-ft(including basement)	10-ft
Overall height	99-ft	77-ft
Ground Beam size	24" x 24"	20" x 20
Column size(up to plinth)	24" x 24"	20" x 20"
Column size(GF to 3F)	W14x233	14" x 16"
Column size(3F to 7F)	W14x90	14" x 16"
Column size(7F to 11F	W12x65	
Column size(12F)	W10x49	
Beam size	W12x40,W10x22, W8x15,W6x9	12" x 12" 10" x 12"

III. METHODOLOGY

In this study, 3D models of 12-storeyed steel and 7-storeyed RC buildings are created by using Structural Analysis Program Software to undertake static and dynamic analysis. Firstly, two independent buildings are constructed and designed by non-linear time-history analysis and maximum displacements of both buildings are obtained. Secondly, In order to investigate dynamic behavior of coupled structures, multi-degree-of-freedom (MDOF) system is constructed to model coupled adjacent buildings. Modeling is to formulate mechanical behavior of structures in terms of mathematical equations. Thirdly, the appropriate stiffness and damping coefficient of viscous damper are calculated by using equation of motion. And then two adjacent buildings are joined by passive viscous dampers to withstand the pounding effect. The two structures joining with dampers are designed by non-linear dynamic analysis to withstand the pounding force. In time-history analysis, time series records from Pacific Earthquake Engineering Research Centre-National Geospatial Agency (PEER NGA) Spectrum approximately equalled to Myanmar Earthquake Intensity are used. Finally, the maximum displacements between two adjacent buildings by using dampers are reduced as compared to the case of the independent system [1].

A. Modelling Approach

In this study, two buildings are considered as independent system and dependent system to study the pounding effect between adjacent structures. The selected buildings are designed to resist earthquake and pounding effect by using time-history analysis. The applied loads considered in this structural analysis and the design codes applied are AISC-LRFD-99, ACI-318-99 and UBC-97. The proposed structures are considered as locating near Sagaing fault and suffering high seismic hazard, seismic zone 4. Soil profile type is S_D and seismic source type is B. The material properties for structural data are as follows:

Weight per unit volume of concrete = 150pcf
Weight per unit volume of steel = $0.28 lb/in^3$
Specified concrete compressive strength, f_c' = 3ksi
Bending reinforcement; yield stress, f_y = 50ksi
Bending reinforcement; yield stress, f_s = 50ksi
Modulus of Elasticity, E_s = 29000 ksi
Modulus of Elasticity, E_c = $57000\sqrt{fc'}$
Poisson's ratio, μ = 0.3
Seismic Importance Factor =1.25
Wind Importance Factor =1.15

B. Methods of Connecting Dampers

Coupling building method includes following methods to reduce seismic vibration.

Passive methods
Active methods
Semi-active methods
Hybrid methods

Passive methods are those which do not use any external power. They basically change mass, stiffness, or damping of the structure. Among those, one can refer to viscous dampers, friction dampers, metallic yield braces, fiber reinforced concrete, and tuned mass dampers and so on. Passive methods are generally effective, very economical, and since they do not need external power, they are reliable during an earthquake that may cause electricity loss[2].

C. Multi-degree-of –freedom System

Three major types of modeling are available: single-degree-of freedom (SDOF), multi-degree-of-freedom (MDOF), and finite element (FE). Each of them has its own advantages and disadvantages. In general, the more accurate the model is, the more complexity will be involved in the model and its solution. Except for simple SDOF systems, closed-form solutions are almost impossible to achieve. For most cases, MDOF models provide enough information for researchers and engineers to predict the dynamic behavior of buildings.

This study uses a simple MDOF model in order to model the dynamic behavior of the buildings. Each floor is modelled via a lumped mass which includes the mass of each floor as well as the mass of the walls connecting that floor to the upper floor. Then each mass is connected to the upper and lower mass with a set of linear springs and linear dampers. A linear spring is a mechanical component that generates a force which linearly depends on the relative displacement at the ends of the spring. And a linear damper is a mechanical component that generates a force which linearly depends on the relative velocity at the ends of the damper. After modeling each structure individually, we may now connect these two structures using linear dampers.

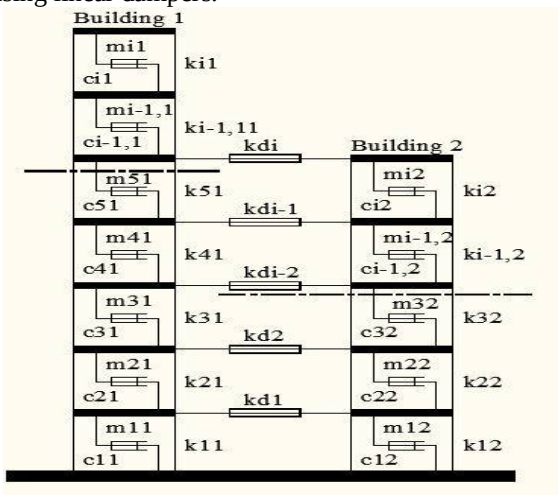


Figure 3- Schematic dynamic model of two adjacent buildings

D. Mathematical Equation(Equation of Motion)

As shown in Figure 2-1, buildings 1 and 2 have twelve and seven stories, respectively. The mass, shear stiffness, and damping coefficients for the i^{th} story are mi1, ki1, and ci1 for building 1 and mi2, k_{i2}, and c_{i2} for building 2. The damping coefficient and stiffness coefficient of the damper at the i^{th} floor are c_{di} and k_{di}, respectively. Let $x_{i1}(t)$ and $x_{i2}(t)$ be the displacement of the i^{th} floor of buildings 1 and 2 in the time domain, respectively. Consequently, $\dot{x}_{i1}(t)$ and $\dot{x}_{i2}(t)$ represent the velocity, and $\ddot{x}_{i1}(t)$ and $\ddot{x}_{i2}(t)$ represent the acceleration of the i^{th} floor of buildings 1 and 2, respectively. The governing equation of the system is as follows.

$$M \ddot{X}(t)+ (C+ C_d) \dot{X} (t) + (K+ K_d) X (t)= M g(t) \quad (1)$$

Where, M, C and K are the given mass, damping and stiffness matrices of the buildings, C_d and K_d represent damping and stiffness matrices of the connectors, and X is the displacement vector of the system. Also, g(t) is the ground acceleration during the earthquake. The details are given as follows:

$$M=\text{diag}\,[m_{11}, m_{21},\ldots, m_{i1}, m_{12}, m_{22},\ldots,m_{i2}]$$

$$K=\begin{bmatrix} K1 & 0 \\ 0 & K2 \end{bmatrix}$$

$$C=\begin{bmatrix} C1 & 0 \\ 0 & C2 \end{bmatrix}$$

$$K_1=\begin{bmatrix} k_{11}+k_{21} & -k_{21} & & 0 \\ -k_{21} & k_{21}+k_{31} & -k_{31} & \\ & \ddots & & \\ 0 & & k_{i-1}+k_{i1} & -k_{i1} \\ & & -k_{i1} & k_{i1} \end{bmatrix}$$

$$K_2=\begin{bmatrix} k_{12}+k_{22} & -k_{22} & & 0 \\ -k_{22} & k_{22}+k_{32} & -k_{32} & \\ & \ddots & & \\ 0 & & k_{i-1}+k_{i2} & -k_{i2} \\ & & -k_{i2} & k_{i2} \end{bmatrix}$$

$$C_1=\begin{bmatrix} c_{11}+c_{21} & -c_{21} & & 0 \\ -c_{21} & c_{21}+c_{31} & -c_{31} & \\ & \ddots & & \\ 0 & & c_{i-1}+c_{i1} & -c_{i1} \\ -c_{i1} & & -c_{i1} & \end{bmatrix}$$

$$C_2=\begin{bmatrix} c_{12}+c_{22} & -c_{22} & & 0 \\ -c_{22} & c_{22}+c_{32} & -c_{32} & \\ & \ddots & & \\ 0 & & c_{i-1}+c_{i2} & -c_{i2} \\ & & -c_{i2} & c_{i2} \end{bmatrix}$$

$$K_d=\begin{bmatrix} \overline{Kd} & 0 & -\overline{Kd} \\ 0 & 0 & 0 \\ -Kd & \cdots & \overline{Kd} \end{bmatrix}$$

$$C_d=\begin{bmatrix} \overline{Cd} & 0 & -\overline{Cd} \\ 0 & 0 & 0 \\ -Cd & \cdots & \overline{Cd} \end{bmatrix}$$

$\underline{K_d}$= diag $[k_{d1}, k_{d2}, k_{d3}, k_{d4}, k_{d5}, k_{d6}, k_{d7}]$

$\underline{C_d}$= diag $[c_{d1}, c_{d2}, c_{d3}, c_{d4}, c_{d5}, c_{d6}, c_{d7}]$

$X=[x_{11}, x_{21},\ldots,x_{i1}, x_{12}, x_{22},\ldots,x_{i2}]^T$

It is clear that displacement x_{i1} and x_{i2} for a set of dampers is highly dependent on the mechanical properties of the damper connectors since damping and stiffness matrices of the connectors, C_d and K_d are dependent on c_{di} and k_{di}. If damping coefficient of the connectors, c_{di} is optimal, the stiffness of the connectors, k_{di} is small. Moreover, the value of x_{i1} and x_{i2} will increase if k_{di} has a very large value, i.e., rigid connectors. Thus, it is assumed that k_{di} =0 for all dampers. This simplifies the problem while keeping the final result accurate. Furthermore, it is reasonable to assume that all damping coefficients c_{di} are equal. Therefore, it is assumed that c_{di} lies between 0 and 10^{16}[5].

Table–II Storey stiffness and damping coefficient of 12-storeyed steel building

No of Storey	Storey Stiffness k_{i1}(k/in)	Total Mass (k-sec2/in)	Damping Coefficient,ci 1 (k-sec/in)
1	727448.6144	867.71102	2512.3996
2	835225.9061	825.51978	2625.8246
3	835225.9061	1001.1300	2891.6599
4	381939.8700	934.4274	1889.1667
5	381939.8700	934.4274	1889.1667
6	380995.5575	934.1940	1886.5942
7	380051.2450	933.9606	1884.0194
8	310306.2450	924.2636	1693.5311
9	309361.9325	924.0302	1690.7388
10	308417.6200	923.7968	1687.9432
11	307473.3075	923.5634	1685.1442
12	217794.1324	639.1674	1179.8598

Table–III Storey stiffness and damping coefficient of 7-storeyed RC building

No of Storey	Storey Stiffness ki2(k/in)	Total Mass (kips)	Damping Coefficient,ci2 (k-sec/in)
1	923366.4054	2121.7744	4426.2571
2	1011129.2940	2067.9917	4572.7530
3	1009464.2280	1978.7256	4469.2871
4	1002855.9940	2007.5236	4486.9333
5	994530.6610	2004.1902	4464.5589
6	994530.6610	2004.1902	4464.5589
7	994530.6610	1399.8541	3731.2167

Passive viscous dampers used to connect every two neighboring floors are assumed to have the same damping coefficient of $1x10^6$ k-sec/in and the same elastic stiffness. The stiffness of damper is calculated by using the following equation [3].

$$C = 2\zeta\sqrt{km} \quad (2)$$

Where, k = stiffness of damper(k/in)
C = damping coefficient (k-sec/in)
ζ = damping ratio (5%)
m = mass of damper (300-600kip)

E. Time Series Records

In this study, non-linear dynamic behaviors of adjacent buildings are studied by time-history analysis to investigate pounding effect. The required ground motion records approximately equal to Myanmar earthquake are obtained from PEER ground motion database beta version application. The Pacific Earthquake Research Centre (PEER) ground motion database provides tools for searching, selecting and downloading ground motion data. The GMPE is considered to be valid for estimating ground motions from worldwide shallow continental earthquakes in active tectonic regions for magnitudes ranging from 3.3 to 7.5-8.5, depending on source mechanism, and rupture distances ranging from 0-300 km.

The PEER-NGA West 2 allows the users three options to define target response spectrum for selecting and scaling of ground motion records:
1. ASCE 7 Code Spectrum
2. PEER-NGA Spectrum
3. User-defined Spectrum

In this study, the responses of RC buildings are studied for ground motions based on PEER-NGA Spectrum.
The "PEER-NGA Spectrum" model creates a target response spectrum using the PEER-NGA ground motion prediction equations (GMPEs). Five empirical models are employed in PGME: Abrahamson-Silva(A&S,2014), Boore-Atkinson(B&A,2014),Campbell-Bozorgnia(C &B,2014), Chiou-Youngs (C&Y,2014) and Idriss (2014). PEER-NGA spectrum is based on deterministic seismic hazard analysis (DSHA). The resulting hazard statement is basically a scenario. A scenario based assessment computes the response of a building to user-specified earthquake event, which is typically defined by earthquake magnitude and the distance between the earthquake source and the building site.

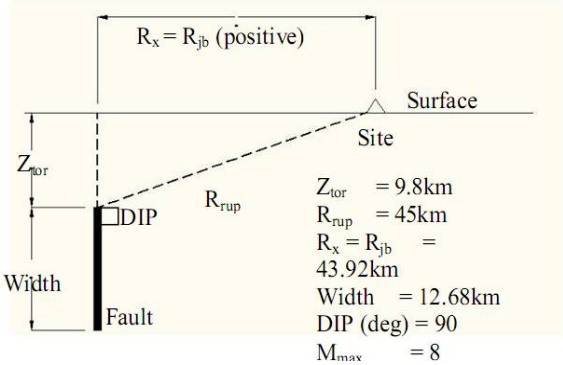


Figure 4 – Parameters for strike slip faulting system

Parameters to be used in searching ground motions (based on recent earthquake data 2012 Thabeikkyin Earthquake near the Sagaing fault) are as follows:

Damping Ratio = 5%
Fault Type = Strike Slip
Magnitude = 8 (min:=6/max:=8)
R_{rup}(km) = 45km (min:=30/max:=60)
R_x(km) = 43.92km (min:=30/max:=60)
R_{yo}(km) = 0km
R_{jb}(km) = 43.92km (min:=30/max:=60)
Z_{tor} = 9.8km
Width = 12.68km
DIP (deg) = 90
Vs30 (m/s) = 750 (min:=360/max:=750)
Vs30 Type = Estimated
GMM Average = Geometric
Max: No. Records = 1

IV. RESULTS AND DISCUSSIONS

In this study, non-linear dynamic analysis is used to determine pounding between adjacent buildings. Time-history analysis method is used to check the maximum displacements of dependent and independent structures. In time-history analysis, time

series records are resulted from PEER-NGA Spectrum. The scaled spectrum of specified record is used for input data of non-liner time-history analysis. In this study, the proposed medical centre neglect soil structure interaction. 3D Model of adjacent buildings connecting with dampers is shown in Figure 5. In this case, four joints at adjacent corners of two buildings are considered as critical points or joints because maximum displacements appear at these joints. The displacements of connecting corners of two adjacent buildings are shown in Figure 6-9.

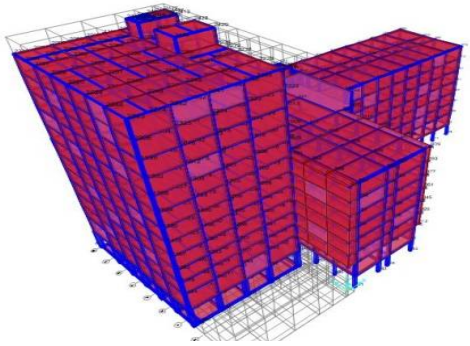


Figure 5 – 3D Model of Adjacent buildings connecting with damper

Table–IV Displacement at Joint 969 according to non-linear time-history analysis without damper and with damper

No. of storey	Max; displacement without damper (in)	Max; displacement with damper (in)
Twelfth Storey	39.3242	5.5617
Eleventh Storey	36.2202	6.1697
Tenth Storey	33.5260	5.0737
Ninth Storey	30.3681	4.5369
Eight Storey	26.7416	3.9496
Seventh Storey	22.6932	3.3181
Sixth Storey	19.0043	2.7534
Fifth Storey	15.1032	2.1658
Fourth Storey	11.1485	1.5800
Third Storey	7.8543	1.0803
Second Storey	5.7945	0.8059
First Storey	3.7496	0.5159

Displacement (in)

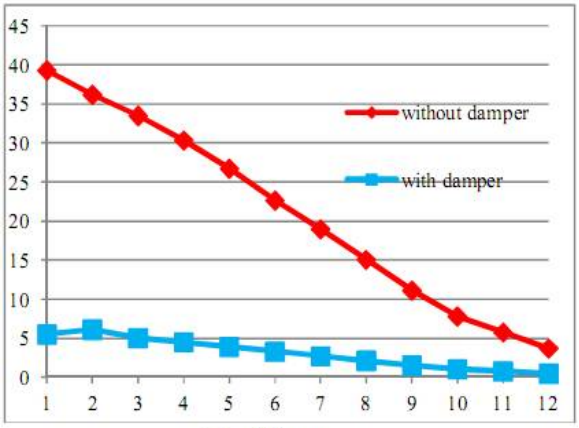


Figure 6 – Displacement diagram of Joint 969

From this Figure, it can be seen that displacement of proposed structure at Joint 969 with viscous damper is less than that of structure without damper. The average percentage of reduction is 73% at Joint 969.

Table -V Displacement at Joint 949 according to non-linear time-history analysis without damper and with damper

No. of storey	Max; displacement without damper (in)	Max; displacement with damper (in)
Twelfth Storey	39.3447	5.5638
Eleventh Storey	36.2183	6.1558
Tenth Storey	33.5183	5.0758
Ninth Storey	30.3616	4.5389
Eight Storey	26.7345	3.9515
Seventh Storey	22.6931	3.3202
Sixth Storey	18.9982	2.7545
Fifth Storey	15.0989	2.1665
Fourth Storey	11.1521	1.5798
Third Storey	7.87328	1.1005
Second Storey	5.7874	0.8061
First Storey	3.7517	0.5156

Displacement (in)

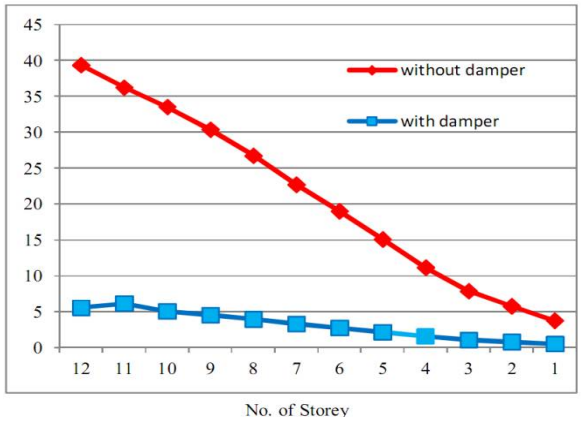


Figure 7 – Displacement diagram of Joint 949

From this Figure, it can be seen that displacement of proposed structure at Joint 949 with viscous damper is less than that of structure without damper. The average percentage of reduction is 86% at Joint 949.

Table -VI Displacement at Joint 656 according to non-linear time-history analysis without damper and with damper

No. of storey	Max; displacement without damper (in)	Max; displacement with damper (in)
Seventh Storey	1.8030	1.0021
Sixth Storey	1.6856	0.9510
Fifth Storey	1.5252	0.8704
Fourth Storey	1.3217	0.7612
Third Storey	1.0821	0.6278
Second Storey	0.8346	0.5008
First Storey	0.6002	0.3617

Displacement (in)

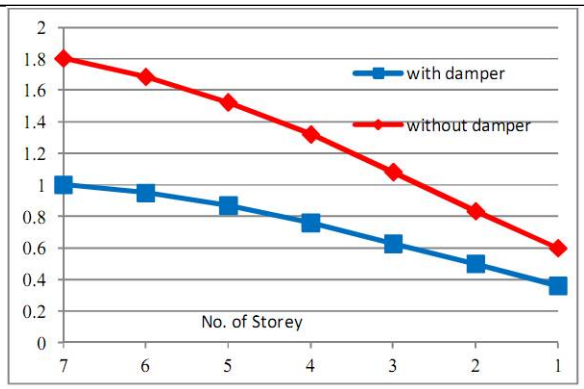


Figure 8 – Displacement diagram of Joint 656

From this Figure, it can be seen that displacement of proposed structure at Joint 656 with viscous damper is less than that of structure without damper. The average percentage of reduction is 44% at Joint 656.

Table -VII Displacement at Joint 629 according to non-linear time-history analysis without damper and with damper

No. of storey	Max; displacement without damper (in)	Max; displacement with damper (in)
Seventh Storey	2.2932	1.4501
Sixth Storey	2.1609	1.3701
Fifth Storey	1.9646	1.2506
Fourth Storey	1.7096	1.0932
Third Storey	1.4059	0.9034
Second Storey	1.0672	0.6895
First Storey	0.6996	0.4548

Displacement (in)

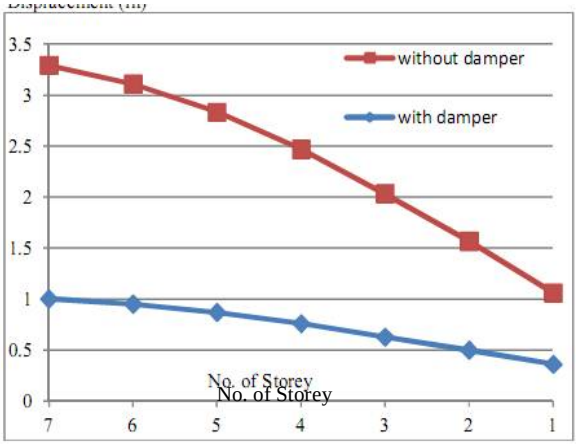


Figure 9 – Displacement diagram of Joint 629

From this Figure, it can be seen that displacement of proposed structure at Joint 629 with viscous damper is less than that of structure without damper. The average percentage of reduction is 37% at Joint 629.

From time-history analysis, it is clearly seen that the joint displacements of connected buildings are reduced to limitable seismic separation gap. The response of adjacent buildings by using dampers can be reduced as compared to the case of the independent system by seeing the corner joint displacements at adjacent corners of two buildings.

There are four types of damper such as passive, active, semi-active and hybrid dampers. Among them, coupling building method uses passive viscous damper by considering the economic point of view. The adjacent buildings excluding damper connection between them become pounding distances or displacements causing pounding behavior because joint displacements of two isolated adjacent buildings exceed the minimum separation gap. This separation gap can cause pounding or colliding force between each other. Therefore this study uses coupling building method which two adjacent buildings are connected with passive viscous damper. Joint displacements of connected adjacent buildings are smaller than the required seismic separation gap and so it cannot cause pounding force between two buildings.

CONCLUSION

Firstly the main assumptions were explained and the type of connection such as passive damping system was discussed. The system dependency from various parameters, such as seismic separation was discussed and commented for each type of connecting damper system. Unlike the case of rigid links, coupling two structures with dissipative connections is a very efficient method for the improvement of their seismic behavior in terms of displacement and response reduction. The behavior of the system depends on the type of dampers. As an added value, passive viscous connections maintain the dynamic characteristics of the unlinked buildings.

From time-history analysis, it is clearly seen that the displacements between two adjacent buildings by using dampers are reduced as compared to the case of the independent system by seeing the corner joint displacements. The average displacement reduction percentages of adjacent buildings at Joint 969, Joint 949, Joint 656 and Joint 629 are 73%, 86%, 44% and 37% respectively.

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