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Optimization of Damper System Design for Pounding of High-rise Buildings

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Abstract-This study describes optimization of damper system design for pounding between existing building and its extension. In this system, passive coupling method is known to be an effective method to reduce undesirable vibrations and structural pounding effects and passive viscous dampers are used by which the existing building is connected to its extension. In this study, 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 analyzed and the maximum displacements of two buildings at connecting corner points are resulted. Thirdly, required minimum seismic separation gap is calculated by using Squared Root of Sum Squares (SRSS) method. A formulation of the multi-degree of freedom equations of motion for viscous damper-connected adjacent multi-story buildings under earthquake excitation is utilized to calculate the optimal damping coefficient and stiffness of dampers. The effectiveness of viscous dampers as known as optimal damper location is then investigated in terms of the reduction of displacement of adjacent buildings. And then, two buildings connected with selected passive viscous dampers are continuously analyzed 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 systemif damper properties are appropriately selected.

Index Terms-Maximum Displacement, Non-linear Time History; PEER Ground Motion Database**;** Seismic Pounding;Viscous Damper

I. INTRODUCTION

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. Thesebuildings, in most cases, are separated without any structural connections or are connected only at the groundlevel. Hence, wind-resistant or earthquake-resistantcapacity of each building mainly depends on itself. Ifthe separation distances between adjacent buildings arenot sufficient, mutual pounding may occur during an earthquake [2]. Seismic pounding is defined as the collision of adjacent buildings during earthquakes. The phenomenon is mostly observed in old buildings that were constructed before the advent and popularity of earthquake resistant design principles. 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.

Three key factors for a good retrofitting method are efficiency, cost and applicability to existing structures. Among the retrofitting methods, passive coupling of adjacent structures is known to be an effective method to reduce undesirable vibrations and structural pounding effects. This study focuses on the prevention of mutual pounding between existing high-rise building and its extension of Palace Medical Centre located in Mandalay, Myanmar. This study discusses the mitigation of earthquake responses of adjacent buildings, separated with a certain distance, by using fluid inertial joint dampers to connect them. Compared with other types of energy dissipaters which can be used as joint dampers to connect adjacent buildings, such as friction dampers, viscoelastic dampers, and yielding metal dampers, fluid inertial dampers have several inherent and significant advantages: linear viscous behavior; insensitivity to temperature changes; small size in comparison to stroke and output force; easy installation; almost free maintenance; reliability and longevity [6]. Furthermore, they can be manufactured less expensively to satisfy different requirements for damper parameters such as the maximum damping force, the maximum operating velocity, the maximum operating displacement, and no measurable stiffness for piston motions. To the best of the writers' knowledge, there are no applications of fluid dampers to connect adjacent buildings for reducing earthquake-induced response. The practical details such as how to manufacture the fluid dampers and how to install the fluid dampers between adjacent buildings are beyond the scope of this study.

The multi-degree of freedom equations of motion for fluid damper-connected adjacent multistory buildings under earthquake excitation are first formulated in both the frequency domain and time domain. Because of non-classical damping properties of the damper–building system, a pseudo-excitation algorithm for random vibration analysis in the frequency domain is introduced. An extensive parametric study is then carried out to investigate the effectiveness of joint dampers in terms of the reduction of displacement of adjacent buildings and to determine optimum damper properties to facilitate the integrated design of fluid damper-connected adjacent buildings. The practical details such as how to manufacture the fluid dampers and how to install the fluid dampers between adjacent buildings are beyond the scope of this study.The specific objectives of the study are (i) to choose the optimal damper system design to reduce the pounding effect, (ii) to improve the seismic performance of coupling buildings during earthquakes and (iii)to compare the maximum displacement of two buildings connected with dampers and that of independent system.

II. DESCRIPTION OF CASE STUDY

Mandalay is the second largest city and the last royal capital of Myanmar. It is located at 21˚ 58′ N and 96˚ 04′ E and 445 miles north of Yangon on the east bank of Irrawaddy River. The city has an estimated population of 1.3 million and is the capital of Mandalay Region. Mandalay is the main commercial, educational, health and economic hub of Upper Myanmar and considered as the center of Buddhism in Myanmar. In Mandalay city municipal area, there are five downtown townships; Aung Myae Thar San, Chan Aye Thar Zan, Chan Mya Thar Si, Mahar Aung Myae and PyiGyiTagon townships. In this study, the proposed structure is Palace Medical Centre located between 71 and 28x29 streets in Chan Aye Thar Zan Township in Mandalay. 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-1. Ground floor plans of the proposed structures are shown in Figure 1 and Figure 2. The required data source is from Mandalay City Development Committee (MCDC).

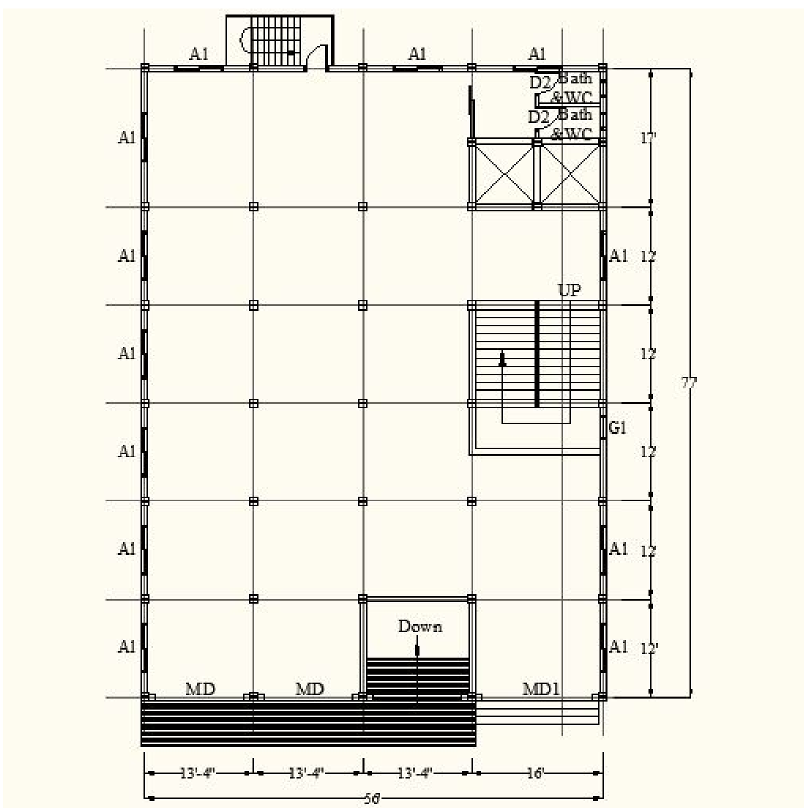

Figure 1: Ground floor plan of 12-storied steel building

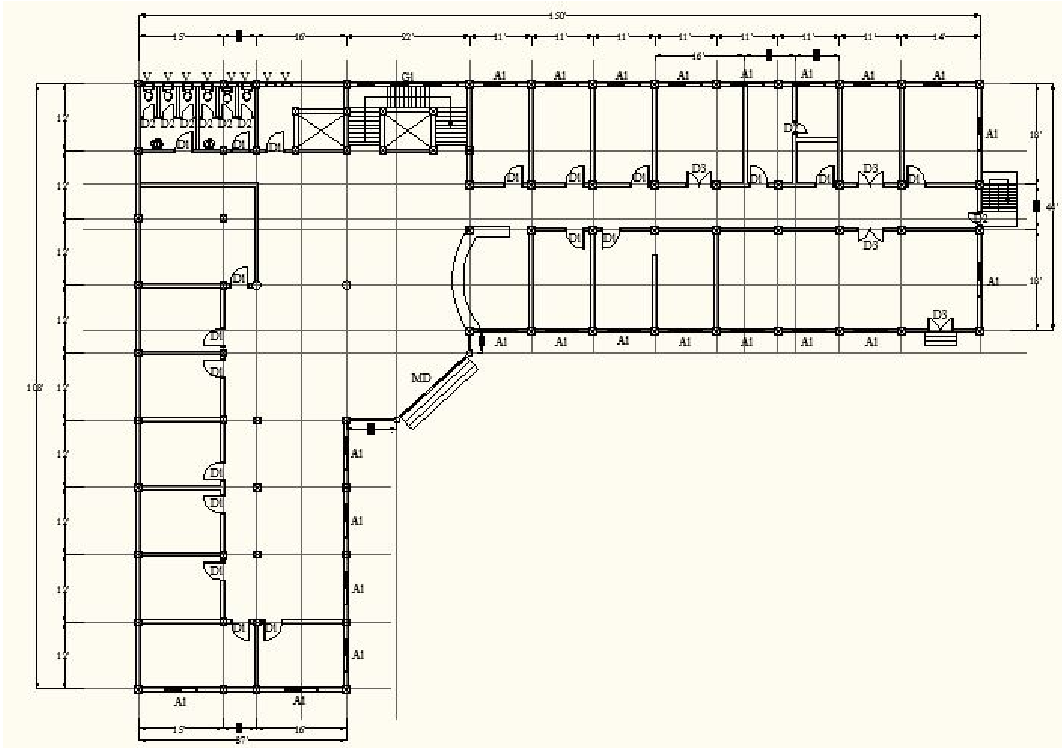

Figure 2: Ground floor plan of 7-storied RC building

Table 1: Structural dimension of Palace medical center

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
Length of building	56-ft	154,37-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

A. Bits and Pieces together

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 optimal coefficients of dampers and optimal dampers locations, 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 and suitable damper location 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 equaled 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. 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.

To highlight important characteristics of fluid damper-connected adjacent buildings and to make the problem manageable, some assumptions are necessary at present. Firstly, ground motion is assumed to occur in one direction as shown in figure 4 so that the problem can be simplified as a two dimensional case. Torsional effect s due to asymmetric buildings are not addressed to here. Each building is modelled as a linear multi-degree of freedom system where the mass is concentrated at each floor and the stiffness is provided by massless walls and columns. Both buildings are assumed to be subjected to same base acceleration and effects due to soil structure interaction is neglected.

B. Time Series Records

In this study, non-linear dynamic behaviours 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: (i) ASCE 7 Code Spectrum, (ii) PEER-NGA Spectrum and (iii) User-defined Spectrum. The responses of proposed 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 siteParameters to be used in searching ground motions (based on recent earthquake data 2012 Thabeikkyin Earthquake near the Sagaing fault) are shown in figure1 and Table 2.

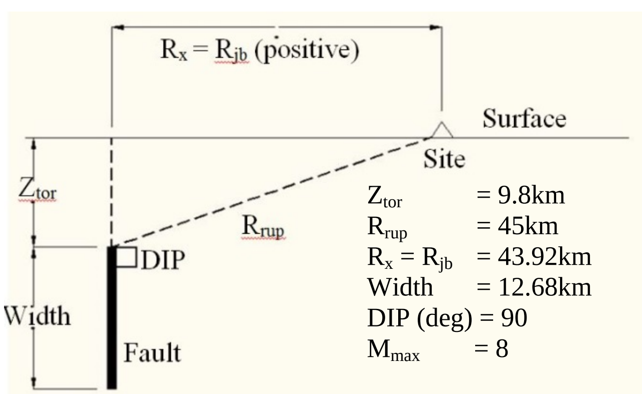

Figure 3: Strike slip faulting system

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 behaviour 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.

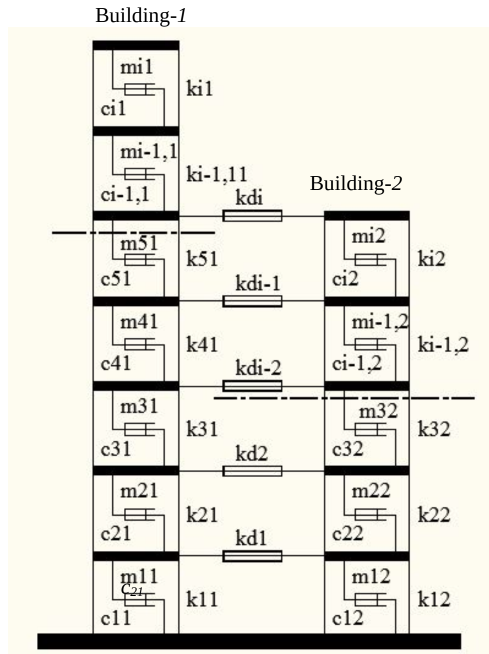

Figure 4- Schematic dynamic model of two adjacent buildings

D. *Mathematical Equation(Equation of Motion)*

As shown in Figure 4, buildings 1 and 2 have twelveand sevenstories, respectively. The mass, shear stiffness, and damping coefficients for the i^{th}story are m_{i1}, k_{i1}, and c_{i1} for building 1 and m_{i2}, 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.

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

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 ofthe system. Also, $g(t)$ is the ground acceleration during the earthquake. The details are given as follows:

$$M=diag\,[m_{11},m_{21},\ldots,m_{i1},m_{12},m_{22},\ldots,m_{i2}] \tag{2}$$

$$K=\begin{bmatrix}K1 & 0\\ 0 & K2\end{bmatrix},\ C=\begin{bmatrix}C1 & 0\\ 0 & C2\end{bmatrix} \tag{3}$$

$$K_1=\begin{bmatrix} k_{11}+k_{21} & -k_{21} & 0 & & \\ -k_{21} & k_{21}+k_{31} & -k_{31} & & \\ ` & & \ddots & & \\ & 0 & & K_{i-1}+k_{i1} & -k_{i1} \\ & & & -k_{i1} & k_{i1} \end{bmatrix} \tag{4}$$

$$C_1=\begin{bmatrix} c_{11}+c_{21} & -c_{21} & 0 & & \\ -c_{21} & c_{21}+c_{31} & -c_{31} & & \\ \text{`} & & \ddots & & \\ & 0 & & c_{i-1}+c_{i1} & -c_{i1} \\ & & & -c_{i1} & -c_{i1} \end{bmatrix} \tag{5}$$

$$C_2=\begin{bmatrix} c_{12}+c_{22} & -c_{22} & 0 & & \\ -c_{22} & c_{22}+c_{32} & -c_{32} & & \\ \text{`} & & \ddots & & \\ & 0 & & c_{i-1}+c_{i2} & -c_{i2} \\ & & & -c_{i2} & c_{i2} \end{bmatrix} \tag{6}$$

$$K_d=\begin{bmatrix} \overline{Kd} & 0 & -\overline{Kd} \\ \underline{0} & 0 & \underline{0} \\ -Kd & \cdots & Kd \end{bmatrix},\ C_d=\begin{bmatrix} \overline{Cd} & 0 & -\overline{Cd} \\ \underline{0} & 0 & \underline{0} \\ -Cd & \cdots & Cd \end{bmatrix} \tag{7}$$

$$\overline{K_d}= diag\ [k_{d1}, k_{d2}, k_{d3}, k_{d4}, k_{d5}, k_{d6}, k_{d7}] \tag{8}$$

$$\overline{C_d}= diag\ [c_{d1}, c_{d2}, c_{d3}, c_{d4}, c_{d5}, c_{d6}, c_{d7}] \tag{9}$$

$$X=[x_{11}, x_{21},\ldots,x_{i1}, x_{12}, x_{22},\ldots,x_{i2}]^T \tag{10}$$

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 coefficients of the connectors, $c_{di}$are optimal, the stiffness of the connectorsk_{di} are 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].

For the time domain analysis, the above equations (1 to 11) can be used for any given time history record of ground motion. For frequency domain analysis of multi-degree of freedom system to optimize the damper system design, pseudo-excitation method can be considered. In this method, the determination of random response of a structure is converted to the determination of response of the structure under a series of harmonic loads, i.e, the so-called pseudo-excitations. Let us assume that the ground acceleration g(t) is a stationary random process with zero mean of its power spectral density $S_g(\omega)$ given in equation (16). Considering ground excitation as a series of harmonic loads, one can write displacement ,velocity and acceleration vectors in the frequency domain as shown in equation (12 to 14).

$$X\,(t) = X(\omega)\, e^{i\omega t} \tag{11}$$

$$\dot{X}\,(t) = \frac{dX\,(t)}{dt} = i\,\omega X(\omega)\, e^{i\omega t} \tag{12}$$

$$\ddot{X}(t) = = \frac{d2X\,(t)}{dt2} = -\,\omega^2 X(\omega)\, e^{i\omega t} \tag{13}$$

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Substituting equations (11-13) into equation (1)

$$X(\omega) = [-M\,\omega^2 + (C + C_d)\, i\omega + (K + K_d)]^{-1} [M\sqrt{Sg(\omega)}] \tag{14}$$

$$S_g(\omega) = \frac{1+4\varepsilon 2\left(\frac{\omega}{\omega g}\right)2}{1+\left(\frac{\omega}{\omega g}\right)2+4\varepsilon 2\left(\frac{\omega}{\omega g}\right)2}\, So \tag{15}$$

Where, S_0 = intensity of rock acceleration (m^2/rad-s^3)

ω_g = angular frequency of rock (rad-s)

ε_g = damping ratio

Table 2: Parameter of Hu Spectral Model (by Hong Feng)

Site Condition	**So**(m^2/rad-s^3)	$\omega_{g\ (rad\text{-}s)}$	ε_g
Soft soil	$1.574x10^{-3}$	10.25	1.006
Medium soil	$1.076x10^{-3}$	17.07	0.7845
Firm soil	$4.65x10^{-4}$	15	0.6

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 such as 969, 949, 656 and 629 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 10-13 and Table 11-14. The stiffness, damping coefficient, natural angular frequency and total mass of 12-storeyed steel building and 7-storeyed RC building are calculated in Table 3-4. The optimal damping coefficient is selected by comparing minimum gap between adjacent buildings as shown in Figure 6-9 by using displacement function*X(ω)*;equation(14).

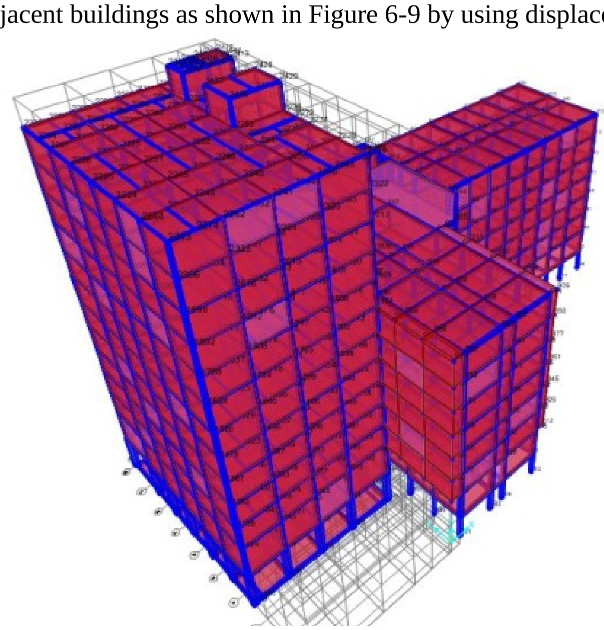

Figure 5 – 3D Model of Adjacent buildings connecting with damper

Table 3: Calculation of stiffness, damping coefficient , natural angular frequency and total mass of 12-Storeyed steel building

Floor of 12-storied Steel Building	Storey Stiffness ki1(k/in)	Storey mass (k-sec2/in)	Damping Coefficient,ci1(k-sec/in)	natural angular frequency ωn(rad/sec)	Total mass,mi1 (kips)
12	5164.0569	0.749923	6.223057964	4.221517693	289.7702472
11	7546.4703	0.83212	28.81165967	4.84462755	321.531168
10	8942.1035	0.841817	29.90335015	5.243151918	325.2780888
9	9954.2672	0.839023	29.93132219	5.54114041	324.1984872
8	10762.6181	0.840438	29.3429625	5.756885157	324.7452432
7	11630.0132	0.852395	28.53245387	5.942252447	329.365428
6	12282.7013	0.865112	27.14704547	6.061669006	334.2792768
5	12928.2486	0.865671	25.42464218	6.216913918	334.4952744
4	13603.3027	0.865671	23.32663945	6.377158155	334.4952744
3	15426.5098	0.964835	21.51267752	6.43263273	372.812244
2	13953.1532	1.037018	16.70518075	5.900984516	400.7037552
1	5756.3341	1.097789	7.587050876	3.683791149	424.1856696

Table 4: Calculation of stiffness, damping coefficient, natural angular frequency and total mass of 7Storeyed RC building

Floor of 7-storied RC Building	Storey Stiffness ki1(k/in)	Storey mass (k-sec2/in)	Damping Coefficient,ci1(k-sec/in)	natural angular frequency ωn(rad/sec)	Total mass,mi1 (kips)
7	16741.6616	4.404396	27.15454058	3.136444023	1701.858614
6	22504.6806	5.290679	36.74616764	3.317891361	2044.318366
5	24720.7375	5.2933	35.15731609	3.476553351	2045.33112
4	26173.009	5.308738	32.35614872	3.57200964	2051.296363
3	27496.863	5.336885	28.72117494	3.651565487	2062.172364
2	25036.436	5.354829	22.37696852	3.478523092	2069.105926
1	10354.2885	5.557922	10.17560244	2.195762421	2147.581061

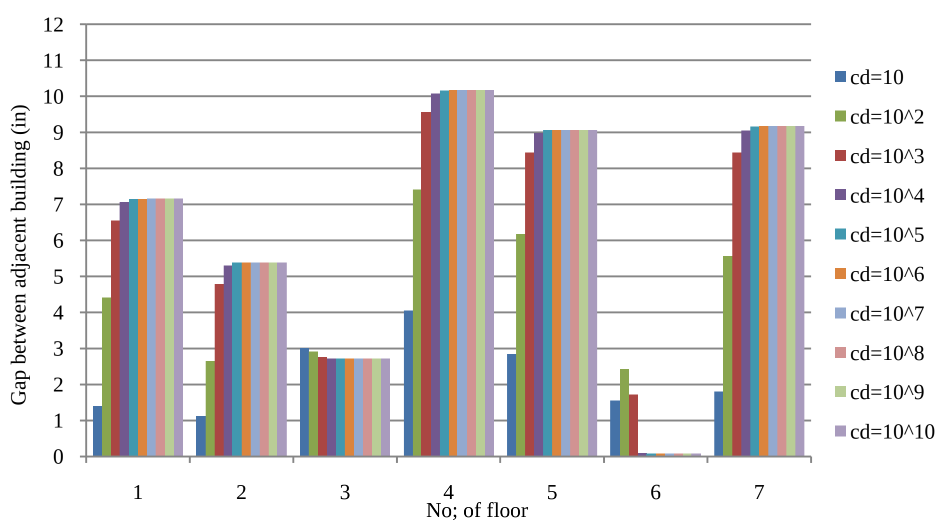

Figure 6 –Gap between adjacent buildings for each floor

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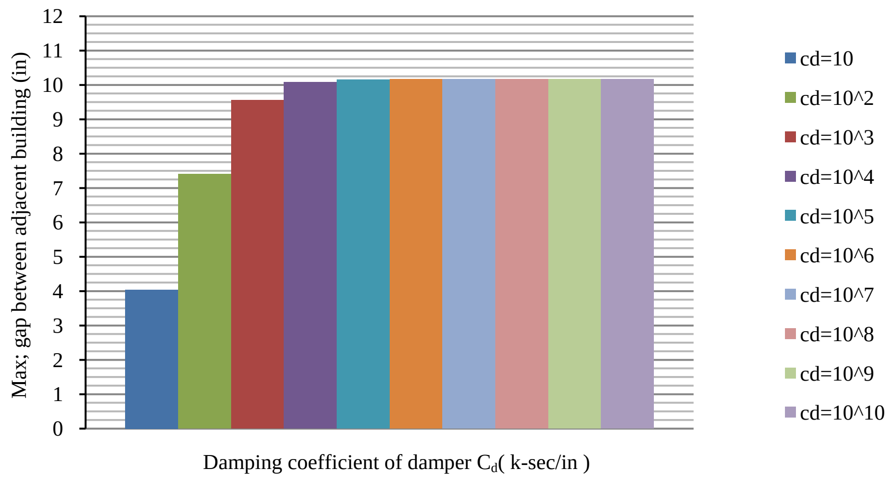

Figure 7 – Max; gap diagram between adjacent buildings for corresponding damping coefficient

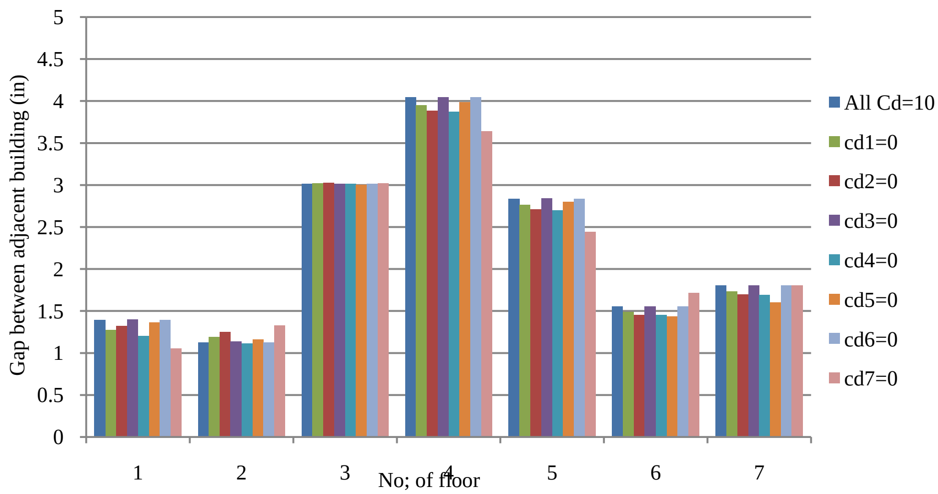

Figure 8 –Gap between adjacent buildings for each floor

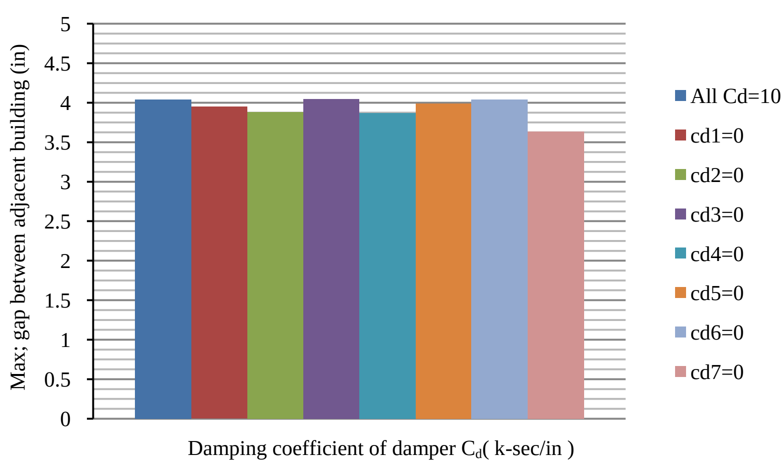

Figure 9 –Max; gap diagram between adjacent buildings for corresponding damping coefficient

Table 5: 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

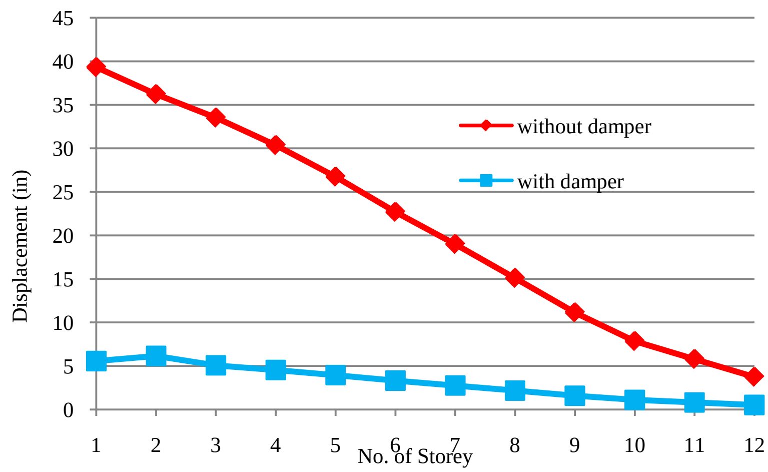

Figure 10 – Displacement diagram of Joint 969

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 6: 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

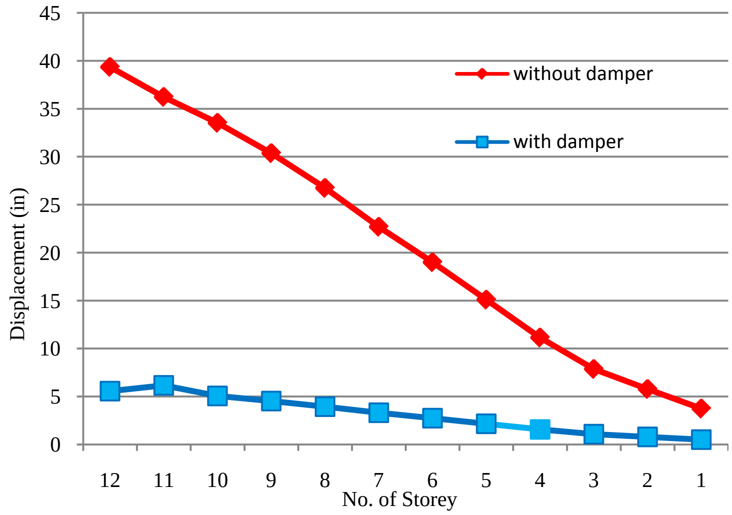

Figure 11 – Displacement diagram of Joint 949

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 7: 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

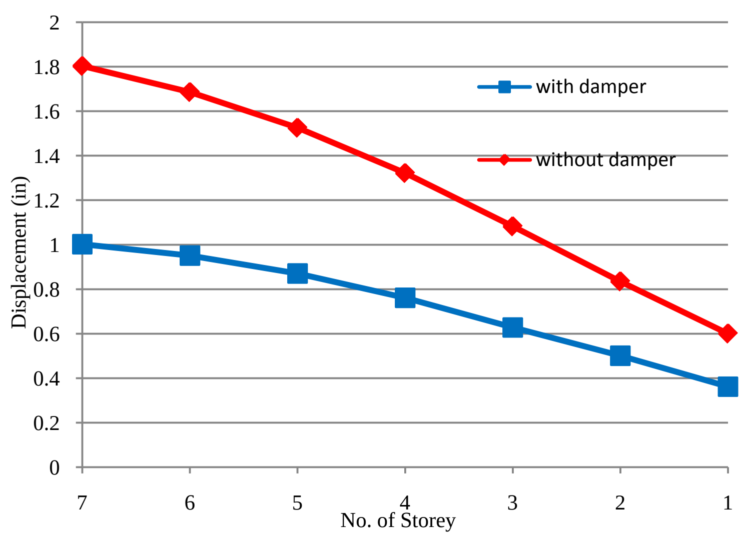

Figure 12– Displacement diagram of Joint 656

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 8: 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

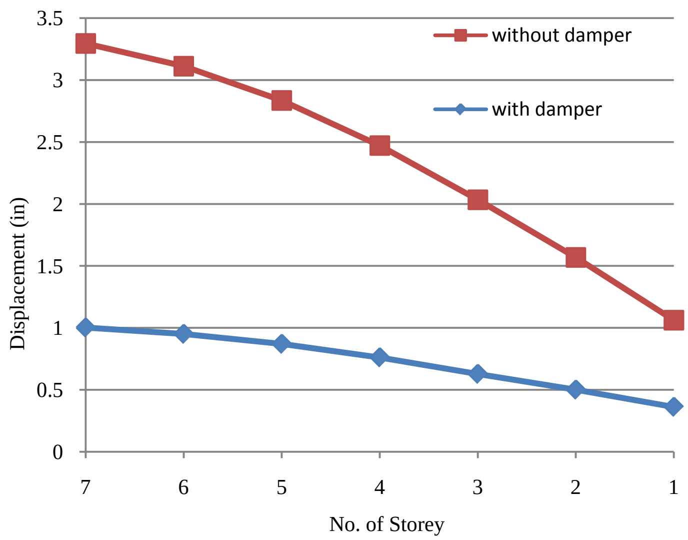

Figure 13 – Displacement diagram of Joint 629

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 behaviour 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.

V. Conclusion

In this study, damper system design for pounding between existing building and its extension was presented through time-history analysis. 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 behaviour in terms of displacement and response reduction. The behaviour 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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