BULETINUL INSTITUTULUI POLITEHNIC DIN IAȘI
Publicat de
Universitatea Tehnică „Gheorghe Asachi” din Iași
Volumul 62 (66), Numărul 4, 2016
Secția
CONSTRUCȚII. ARHITECTURĂ

INCREASINGLY DYNAMIC ANALYSIS OF STRUCTURE WITH VISCOUS DAMPERS UNDER EARTHQUAKE NEAR AND FAR FIELD FAUL

BY

HAMID REZA ASHRAFI*, SORUSH DADGAR and PEYMAN BEIRANVAND

Technical and Engineering Faculty, Razi University , Kermanshah , Iran

Received: October 01, 2016
Accepted for publication: November 11, 2016

Abstract. The study shows the effect of past strong earthquakes that acceleration of near fault earthquakes due to having impact like movement with long periods in the beginning of the record; "directional" have significant differences with far-fault field earthquakes. Adverse effects of such earthquakes increases by approaching cycle time for structures to the period of these earthquakes. In this study, the behavior of steel moment frames equipped with linear and nonlinear viscous fluid dampers with different damping percentage under the effect of far field earthquakes and near and far fault were studied and evaluated. For this purpose, structural models of the 9 floors with moment frame system were selected. With the incorporation of viscous dampers and increasingly dynamic analysis (IDA), the results obtained in the form of diagrams maximum cutting base, relative displacement of floor and residual displacement was introduced.

Keywords: increasingly dynamic analysis; viscous damper; residual displacement; near fault field.

1. Introduction

The recent earthquakes in Turkey (1999), Taiwan (1999), India (2001), Iran (2003), China (2008), New Zealand (2011) and Japan (2011), have made

*Corresponding author: *e-mail:* peyman51471366@gmail.com

significant financial and life losses. Induction powers to large the structure by earthquakes to the factors such as mass of the structure, materials and built structure type, structure damping, ductility and the energy dissipation capacity is dependent. Among the energy dissipation devices can be noted to viscous dampers (FEMA-440 (ATC-55), 2005). The use of energy absorbing systems the first time after the earthquake of 1989 Loma Prieta was raised. Dargoosh in 1997 and sung to provide a history of energy absorbing systems have attempted (Soong & Dargoush, 1997). In 1998, Constantino and Song presented the how to design elements used in the energy absorbing system to cope with the lateral forces caused by the earthquake (Constantinou *et al.*, 1998). After the earthquake in 1971 in San Diego, studies and findings of earthquakes engineers and seismologists led to determine the coefficient for the near field earthquakes in UBC97 regulations. Later, more accurate coefficients for the near field earthquakes were considered in UBC2000 regulations. During the study of damping, a structural system should the structural behavior from elastic state to plastic changes in the way in the analysis be considered, that this is possible with increasingly dynamic analysis (Vamvatiskos & Cornell, 2002). Increasingly dynamic analysis is one of the nonlinear dynamical analysis methods (Luco & Cornell, 1998). Over the years this method became to one of the most popular methods among researchers in studies such as, Bazooro and Cornell (1994), Lee and Fooch (2002) and Mahani and Dirlin (2000) was used [8-11].

2. Concept of Increasingly Dynamic Analysis

After performing analysis of the results can be displayed as curves, in which the scaling coefficient associated with each record of earthquake, to its corresponding response is related that this response can be structural relative displacement. If the relative displacement, as a basis for evaluating the performance we choice, when building becomes unstable that the relative displacement for a small amount of increasing in the accelerated the move of the earth to increase sharply or that the difficulty equivalent of building an amount less than 20% of hardness in elastic state is reduced. For example, in the form of (1) the initial hardness for elastic state, and hardness equivalent between the S_A = 1.3 g and S_A = 1.4 g has been shown. For a structure, the overall capacity of the record usually is determined so that the structure is unstable. When using the relative displacement Lee and Fotch (2002) suggest that this displacement be limited to 0.1.

To obtain the full range of possible answers must be performed increasingly dynamic analysis for an appropriate number of earthquake records. By doing so, several curves for analysis performed is obtained that can be displayed together on a graph in which case diagram of Fig. 2 can be obtained.

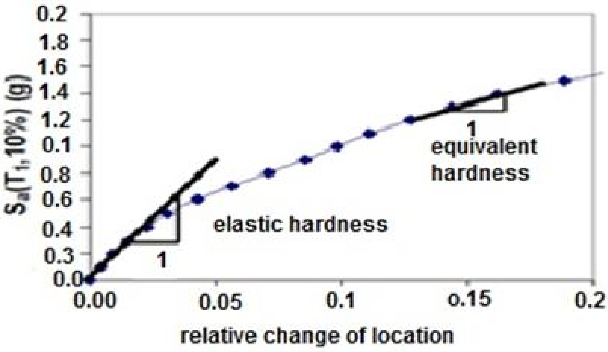

Fig. 1 – IDA curve for a record of earthquake

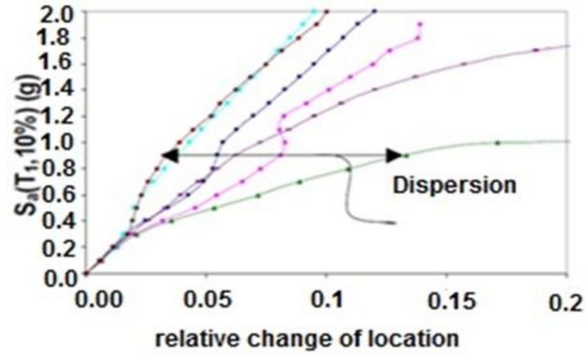

Fig . 2 – IDA curve for several record of earthquake

3. Introduction of the Structural Model

In this study, instead of designing a new model, the model used in studies and Kravinkler and Gupta (1999) has been used (Gupta & Krawinkler, 1999). System used is in this Moment frame model. reason of the using this model is that aforementioned model has been designed so that against earthquakes of moderate to high has nonlinear performance that this is due to role of dampers in reducing the amount of hysteretic energy dissipated by the other members of structural is essential (Powell, 2000). Model used is two-dimensional frame with five openings. A view of the plan of structure used in this model in Fig. 3 has been brought as mentioned earlier in the design of this structure for bearing lateral loads from the moment frame system has been used. It should be noted that the column in the F line under the influence of lateral forces exerted on weak axis, become bent and beam connection to its column is as articular connection. The rest of the frames have a moment frame connections. It is worth noting that this study only one of the moment frames related to this building is examined that in Fig. 3 has been shown.

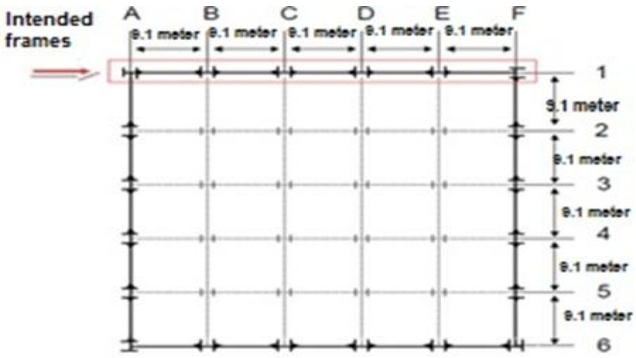

Fig. 3 – 9 floors structure plan.

The yield stress for beam and column respectively equal to 2,500 kilograms per square centimetre and 3,500 kilograms per square centimetre, and the Poisson coefficient of 0.3 has been considered. In Fig. 4 a view of the frame number (1) and sections used in it has been shown.

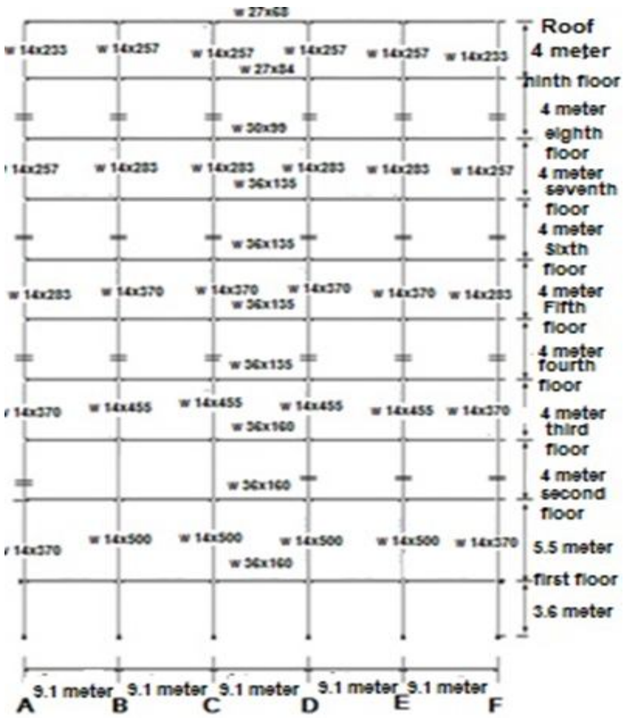

Fig. 4 – Used sections in frame number 1.

A dynamic characteristic of model that the modal analysis carried out, in the Perform software has been obtained in Table 1 has been presented. In this table *T* indicates the cycle time, and φ_i is the representative mode shapes.

Table 1
Periodicity Time and Participation Coefficient for First Primary Models

Periodicity time	T_1	T_2	T_3	T_4	T_5
	2.31	0.86	0.49	0.32	0.22
	φ_1	φ_2	φ_3	φ_4	φ_5
roof	1	1	0.968	-0.81	-0.52
9th	0.922	0.524	-0.15	0.837	1
8th	0.821	0.232	-0.93	1	0.108
7th	-0.71	0	-0.98	-0.11	-0.99
6th	0.616	-0.40	-0.46	-0.97	-0.25
5th	0.507	-0.63	0.248	-0.86	0.833
4th	0.394	-0.73	0.804	0.029	0.632
3rd	0.282	-0.59	1	0.819	-0.35
2nd	0.171	-0.39	0.78	0.907	-0.83
1st	0	0	0	0	0

4. Specifications of the beam elements

Beams used in this model have made by using three components which are: linear elastic section, nonlinear joints and the terminal rigid regions. In Fig. 5 the view of the used beam elements is presented View of the beam elements used has been brought. Here as approximate joint position, because on the basis of studies Kravinkler and Gupta (1999), the structural behavior is not sensitive to the position of the joints. In this research distance of plastic joints from the column has been considered equal to 15 cm. In perform of anchor relation the period for plastic joints is as (Fig. 5).

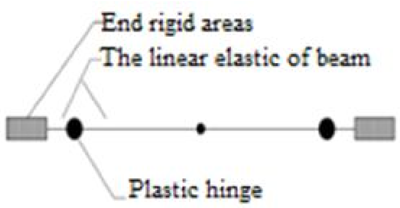

Fig. 5 – Beam elements schema in perform for non-linear joints.

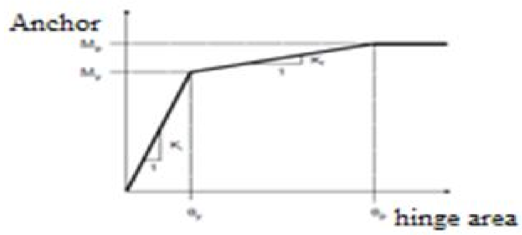

Fig. 6 – Anchor - rotation relation.

Anchor-rotation relation presented is as three linear and joint also in the last section of this diagram is as complete plastic. At the beginning of the diagram, initial hardness of joint is very high, when the joint flow and after that hardness significantly is reduced. So as long as plastic joint in the beam spreads and beam reaches to flowing stage, elastic beam behavior will be linear. Table 2 shows the parameters of initial hardness, surrender anchor and final hardness related to beams used in this study.

Table 2

Values of Anchor-Rotation Curve for Plastic Joints of Beam Elements

θ_p rad	M_P kN.m	K_f kN.m/rad	M_y kN.m	K_i kN.m/rad	Beam
0.15	4,995	11,904	3,210	Very much	W36×160
0.15	4,048	9,564	2,614	Very much	W36×135
0.15	2,322	4,615	1,630	Very much	W30×99
0.15	1,762	3,209	1,280	Very much	W27×84
0.15	1,231	1,972	935	Very much	W24×68

5. Specifications of Column Element

Modeling columns, except cases that in the following is expressed, is roughly the same of beams. The first difference in modeling the column compared to beam caused by the formation of plastic joints on beam-column connections. Another difference is that the plastic joint in the column are as the combination of anchor - axial force relation.

It is expected that the columns compared to beams, plenty of axial load bear, so the joints used in the columns will be P-M joints type. Table 3 specifications related to anchor-rotation diagram for the sections used in the columns are shown. Joints used in the columns can under the combination of loads include anchor and compressive force or anchor or tensile force to surrender and therefore joints related to column would have a kind of interaction behavior caused by anchor- axial force. To enter result of this interaction, column plastic joints by using interaction P-M diagram as shown in Fig. 7 are modelled.

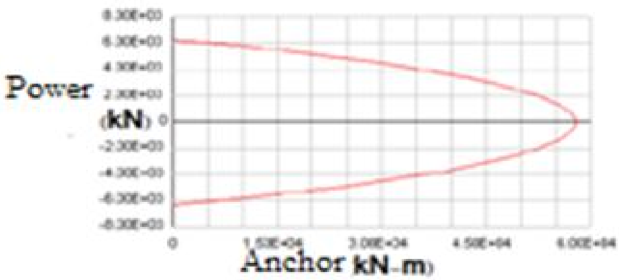

Fig. 7 – Diagram of interaction P-M for columns joints.

From the column interaction diagram can be seen that by increasing the axial load, bending strength values are reduced and vice versa. Perform Software the column interaction diagram is drawn automatically.

Table 3

Curve Values of Anchor- Rotation for Joints Plastic Element of Column

θ_p rad	M_P kN.m	K_f kN.m/rad	M_y kN.m	K_i kN.m/rad	Column
0.15	9,550	22,701	6,145	Very much	W14×500
0.15	8,183	13,867	6,103	Very much	W14×455
0.15	6,543	14,552	4,360	Very much	W14×370
0.15	4,634	7,337	3,533	Very much	W14×283
0.15	4,147	6,435	3,182	Very much	W14×257
0.15	3,702	5,696	2,847	Very much	W14×233

6. Specifications of Applied Records and Description of Scaling

Increasingly dynamic Analysis used in these research by nonlinear dynamical analysis on the 9 floors and in the presence of a viscous damper, especially under the effect of 12 records of the earthquake has been done which of these 12 different records related to the field of earthquake 6 records related to near field and 6 other records is related to far fault field. Specifications related in this acceleration mapping in Table 4 have been shown.

Table 4

Earthquake Records of Near and Far Fault Field

Name	earthquake	Station	Date	PGA	Scale Coefficient
NF01	Tabas	Tabas station	1978	0.49g	0.546
NF02	C.Mendocino	Petrolia	1992	0.22g	0.458
NF03	Landers	Lucerne	1992	0.55g	0.772
NF04	Erizinican	Erzinican	1992	0.218g	0.505
NF05	Northridge	Rinaldi	1994	0.506g	0.569
NF06	Kobe	Takatory	1996	0.204g	0.259
FF01	Imperial Valley	IrrDist	1940	0.346g	0.752
FF02	Landers	Yermo	1992	0.545g	1.515
FF03	northridge	Newhall,	1994	0.739g	1.091
FF04	Landers	Barstow	1992	0.519g	1.233
FF05	Loma Prieta	Gilroy Ary 3	1998	0.416g	0.626
FF06	Northridge	Olive View FF	1994	0.315g	0.533

To perform increasingly dynamic analysis, any earthquake intensity scale with 20 different levels, that these levels are designed by using the spectral acceleration level considered in a critical period of the main building and for attenuation of 10% has been measured. Intensity domain from 0.1 g to 2 g is varied that with 0.1 g minor development is changed. Increasingly dynamic analysis for the case of aforementioned frame to add various types of dampers and in the absence of the frame set and damper be under the effects of earthquake records field far and near fault field are used. In this study, the incremental dynamic analysis process on the structural model of dynamic analysis, nonlinear time history and in the case of a certain type of damper is added to the structure, the main earthquake by using the acceleration whole of the main cycle time and for attenuation of 10% to 20 levels different of intensity has been scaled. Intensity earthquake applied from 0.1g to 2.0 g and steps have taken to distances. To perform increasingly dynamic analysis, desired records should be brought to scale. Records desired with the use of first mode of spectral acceleration for critical damping equivalent to 10% we scale. To find out the first mode spectral acceleration records of earthquake, the software was used to name of NONLIN. This software by doing the spectral analysis for critical damping of 10%, an Excel file as output provide for user that contains diagrams of spectral acceleration – cycle time. After that the displacement of spectral can be found through the relationship (1) to convert the spectral acceleration and to obtained spectral acceleration – cycle time figures.

$$S_A = \omega^2 D. \qquad (1)$$

In this relation, $\omega = 2\pi/T$, where T is cycle time and D is displacement. It is worth noting that to get the whole response to critical damping of 10% from the whole response based on the damping of 5% will be based on the provisions of Chapter 4 related to NEHRP, the values of the response range of 5% to the damping coefficient of BD = 1.2. The main cycle time of structure under the studied in this research is 2.3 s that for this cycle time values of SA (T_1, 10%) is equal to 0.217 g. The following records used to such a way were scaled that has the same value at T = = 2.3 (FEMA, 2000). In the Figs. 8 and 9 can be seen the range of response scaled for desired records.

When the records were scaled, perform software has this ability that re-scaled them, in a way that intensity be vary in the range of 0.1 g,...,2.0 g and with 0.1 g development. Then the results of increasingly dynamic analysis on the frame are checked. For each type of damping, the structure under the effect of 12 different records of earthquake located that these earthquakes to levels of intensity that in the range of 0.1 to 2.0 g = S_A (T_1, 10%) with development equal to 0.1 g covers. This means Structural analysis on a large number of dynamic time histories it has been done. Damper are considered in this research, including linear and nonlinear structural dampers with α = 0.5, 1, 1.5 with additional critical damping ratio is 10 and 20%. It should be noted in this paper, the amount of relative displacement element of 1% is considered between the

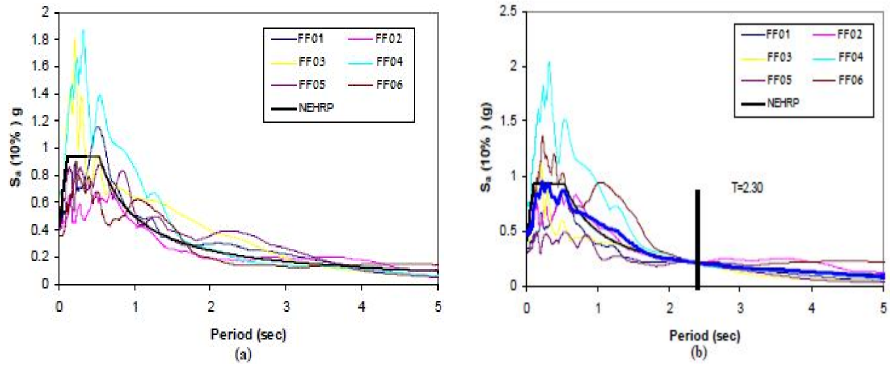

Fig. 8 – Not scaled response range (a). The scaled response range (b), for the records of far field (FF).

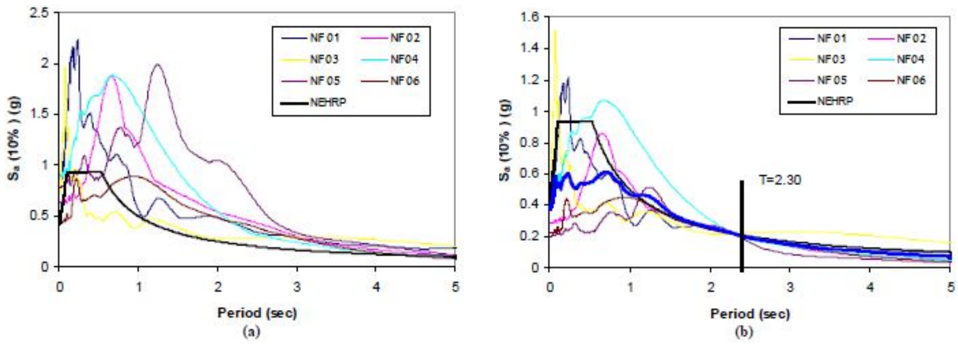

Fig. 9 – Not scaled response range (a). Scaled response range (b), for near field records (NF).

two ends. After increasingly dynamic analysis, Using records from near and far field On the structural model And for different modes of decay, results The maximum base shear in the form of graphs, change the location of waste And the relative displacement of the first floor were drawn. Figures obtained, The IDA figures are known, for each case brought in part by the same title and have been discussed. And at the end of the conclusion of the results Charts has been provided.

7. Maximum Cutting Base

Researchers such as Constantine & Symans (1993), Miyamoto & Singh (2002) and Lin & Chopra (2002), Put viscous dampers The structures, The effects of this energy dissipation systems on Cutting base have studied (Myiamoto & Singh, 2002; Lin & Chopra, 2002) [17-18]. The purpose of this research is intended to carry out; this issue is what impact that different type's damper on the relative displacement and displacement residual obtained from loads applied of earthquake far and near fault field on structure. The study of these cases, regardless of the use of dampers The rationale is not cutting rates, Because reducing the amount of relative displacement and displacement by the use of the damper can increase the amount of basic cutting and consequently the plan uneconomical. So before considering any case, we pay to review the charts cutting base. Figures obtained from the use of near field records in the form of (10) and (11) have been shown.

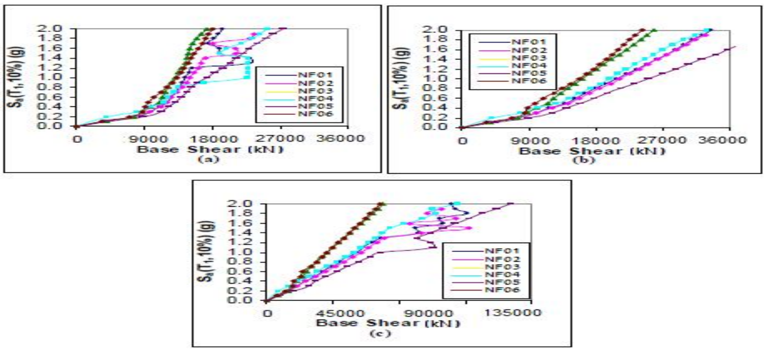

Fig. 10 – IDA diagrams Maximum cutting base by the additional damping 10% and dampers with damping component 1.5, 1, 0.5 = α (near fault field).

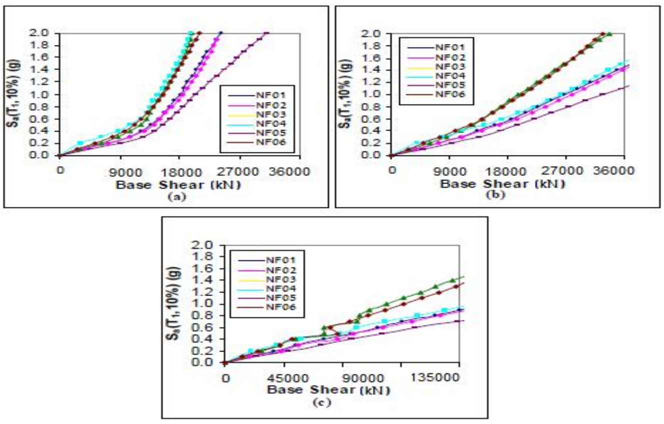

Fig. 11 – IDA diagrams Maximum cutting base by the additional damping 20% and dampers with damping component 1.5, 1, 0.5 = α (near fault field).

In the following and in Figs. 12 and 13. Figures related to the use of the records in far field from the fault has been shown.

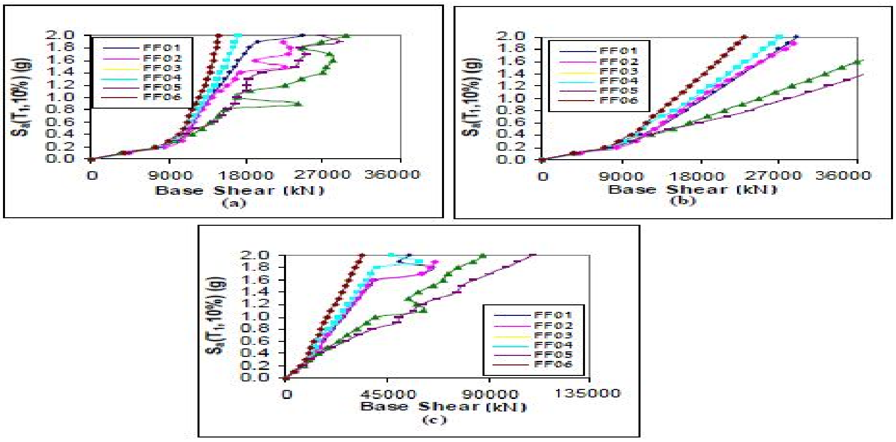

Fig. 12 – IDA diagrams maximum cutting base by the additional damping 10% and dampers with damping component of 1.5, 1, 0.5 = α (far fault filed).

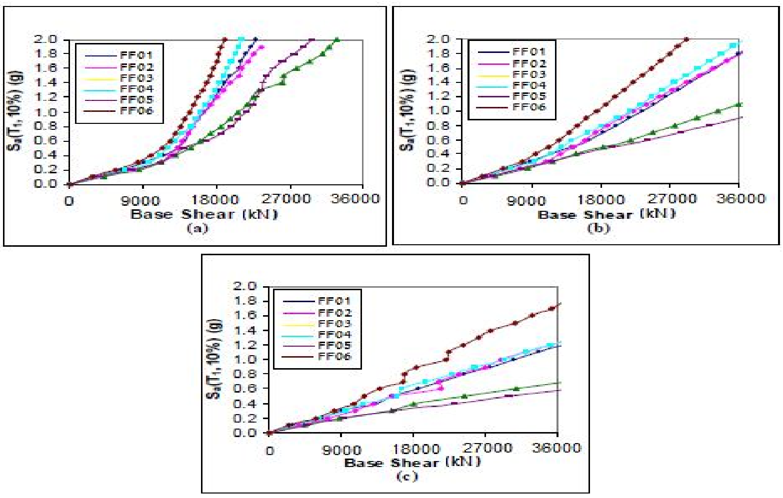

Fig. 13 – IDA Diagrams maximum cutting base the additional damping 20% and dampers with damping component of 1.5, 1, 0.5 = α (far fault field).

As the Figs. 10,...,13 is found, regardless of the type of earthquake actions ranging from far-field or near, with the percentage of critical damping the additional of the 10% to 20% cutting base values on the structure is increased. It can be seen that in both near field and far-field fault diagrams, with an increase in the values of damping component of α values recorded for cutting base is increased. The highest amount of values recorded in case of using nonlinear dampers with α = 1.5 and the lowest amount of cutting in case of using non-linear dampers with α = 0.5 is obtained and values recorded for linear dampers is between these two values.

8. Relative Displacement

From relative displacement can be regarded as one of the criteria suitable for assessing the damage to the structures named. In this study, maximum relative displacement values first floor of frame under studied has been presented. With accuracy of these figures can be realized by increasing the critical damping from 10 to 20%, relative displacement values (IDA) is reduced.

With accuracy in Figs. 14,...,17 can be seen that with increasing critical damping, from 10 to 20% relative displacement values is reduced. Behavior observed in the curves (IDR) to cutting base curves near and far field earthquakes are more diverse. By passing the yield point (starting point dispersion in curves) curves of relative displacement in case of using the damper with α <1 a have less steep and more diffuse, while for the values α > 1 More curves slope and curves are gathered in less space. The greatest value of relative displacement, in case of using dampers with α = 0.5 and lowest values using dampers with α = 1.5 is obtained. By comparing diagrams near and far field earthquakes can be seen that the distribution among related diagrams of near field is more.

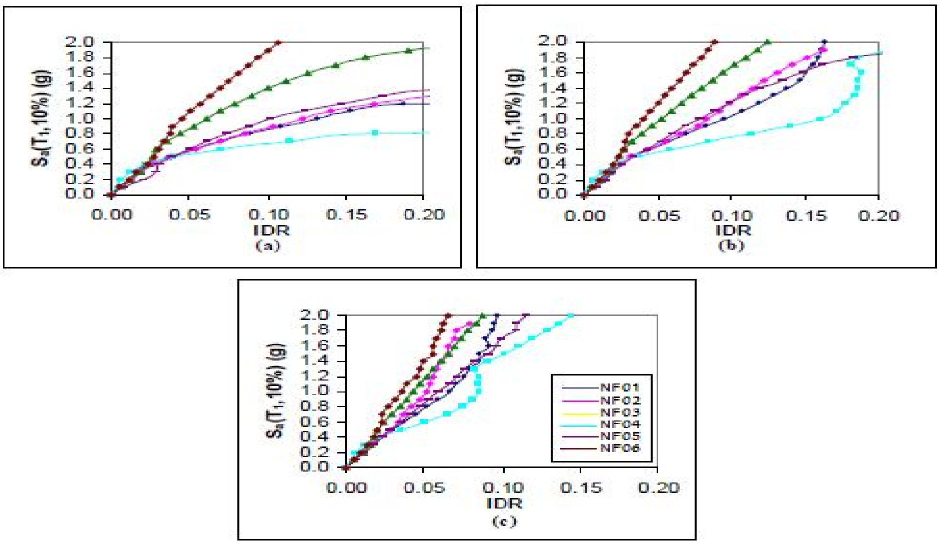

Fig. 14 – IDA diagram of the relative displacement of the first floor for the additional damping of 10% and dampers with damping component of α = 0.5,1, 1.5 (near fault field).

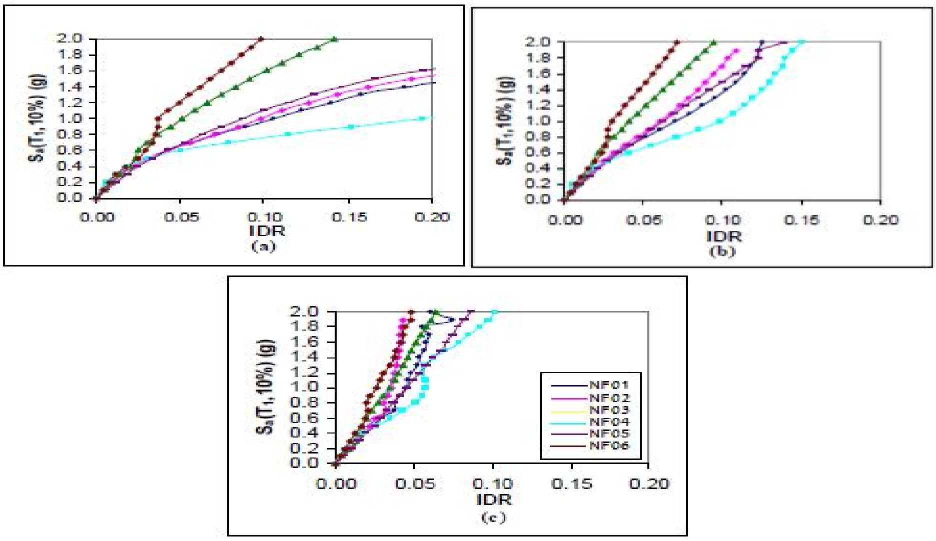

Fig. 15 – IDA diagram of the relative displacement of the first floor the additional damping of 20% and dampers with damping component of α = 0.5,1, 1.5 (near fault field).

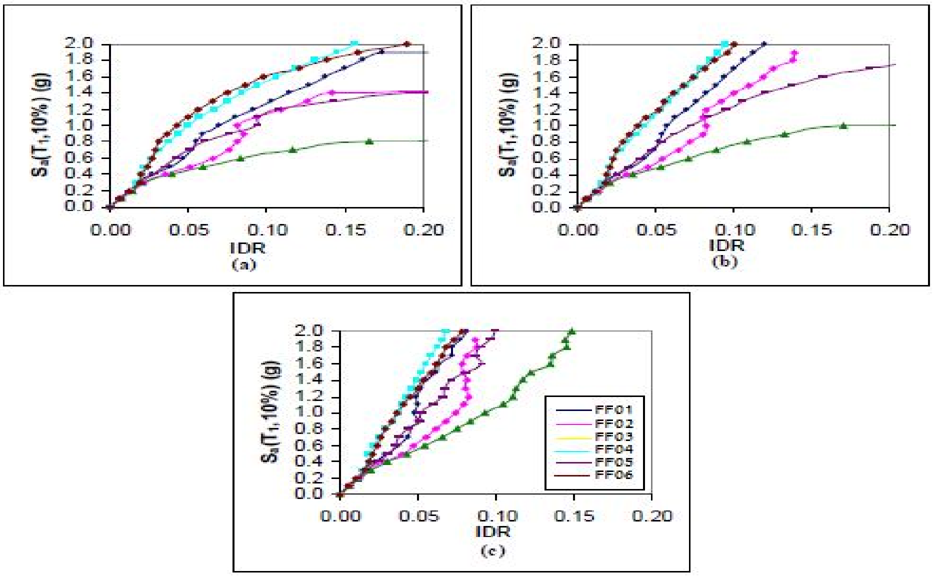

Fig. 16 – IDA diagram of the relative displacement of the first floor for the additional damping of 10% and dampers with damping component of α = 0.5,1, 1.5 (far fault field).

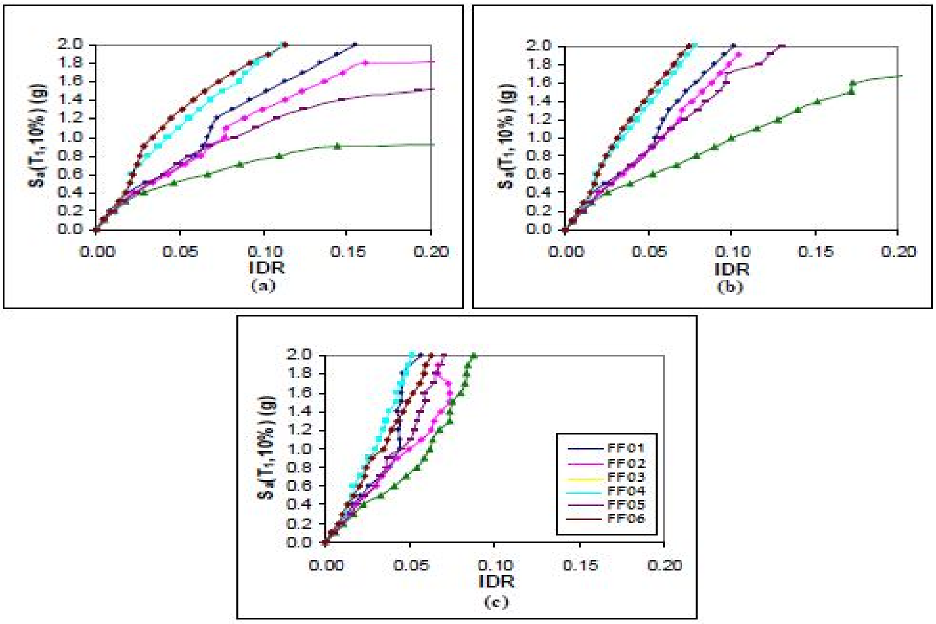

Fig. 17 – IDA diagram of the relative displacement of the first floor for the additional damping of 20% and dampers with damping component of α = 0.5,1, 1.5 (far fault field).

9. Residual Displacement

Residual displacement occurs when the structures were affected by the earthquake forces moved in one direction and without returning to the original position, be vibrated at the new location. Displacement values residual, for frame of 9 floors of, studied affected by earthquakes near and far field in the Tables 5 and 6 has been shown.

With viewing values presented in the Tables 5 and 6 can be realized the difference among values residual displacement for near and far field earthquakes. This difference in values can be observed in diagrams of Figs. 18 and 19 related to near and far fault field earthquakes. Here, for example, results related to records of NF1, NF2, FF1, FF2 has been presented. After drawing the diagrams of residual displacement was observed that in most cases, similar to the drawn for each damper has more similarity to force-speed diagram of that damper. Thus it can be stated that the damper ability to reduce residual displacement of structure to force-speed relationship of damper is dependent.

Table 5
Values Recorded of Residual Displacement for Near Faults Field Earthquake, [cm]

Earthquake acceleration	NF01			NF02			NF03			NF04			NF05			NF06		
	α=0.5	α=1	α=1.5	α=0.5	α=1	α=1.5	α=0.5	α=1	α=1.5	α=0.5	α=1	α=1.5	α=0.5	α=1	α=1.5	α=0.5	α=1	α=1.5
0.1g	0	1	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0.3g	3	3	3	3	2	0	1	0	0	5	3	3	4	3	0	1	0	0
0.5g	8	5	5	6	3	2	2	1	1	18	13	10	10	5	4	3	2	1
0.7g	19	13	8	18	8	3	0	0	2	46	28	16	17	10	2	4	3	2
0.9g	35	19	11	34	17	5	3	1	1	97	38	17	28	15	9	4	3	1
1.1g	51	25	10	47	19	5	6	2	2	116	44	13	43	19	10	3	2	3
1.3g	76	30	9	64	23	3	10	3	4	146	48	8	61	29	10	2	1	7
1.5g	145	36	10	93	28	3	18	4	3	135	51	3	98	35	17	2	3	9
1.7g	156	38	8	111	30	1	32	5	3	99	51	1	114	43	18	5	5	10
1.9g	156	38	18	134	35	1	44	8	4	35	53	1	123	46	16	7	8	13

Table 6
Values Recorded of Residual Displacement for Far Fault Field Earthquake, [cm]

Earthquake acceleration	FF01			FF02			FF03			FF04			FF05			FF06		
	α=0.5	α=1	α=1.5	α=0.5	α=1	α=1.5	α=0.5	α=1	α=1.5	α=0.5	α=1	α=1.5	α=0.5	α=1	α=1.5	α=0.5	α=1	α=1.5
0.1g	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.1
0.3g	1	0	0	1	0	0	2	1	1	1	1	1	1	1	1	0	0	0.2
0.5g	8	5	3	5	4	2	14	9	6	2	2	2	6	5	5	1	0	0.2
0.7g	18	12	8	8	5	4	28	18	8	4	3	3	15	13	10	1	1	22.5
0.9g	25	18	10	13	9	9	35	9	5	9	5	3	23	19	12	1	2	4
1.1g	28	21	10	10	15	18	47	25	3	14	7	5	37	20	16	6	5	7.6
1.3g	30	24	11	12	15	20	54	30	2	17	10	6	58	28	16	11	6	10
1.5g	30	28	13	10	6	23	63	39	6	31	12	6	64	34	18	15	9	11
1.7g	27	33	16	34	1	18	72	45	17	39	15	7	71	36	18	22	12	13
1.9g	19	38	18	51	8	10	81	56	22	46	16	7	78	53	18	30	14	14

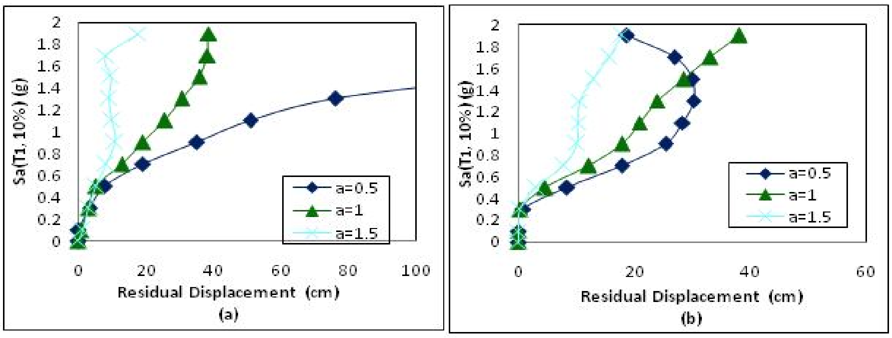

Fig. 18 – Residual displacement diagram of the structure affected by NF1 and FF1 earthquakes.

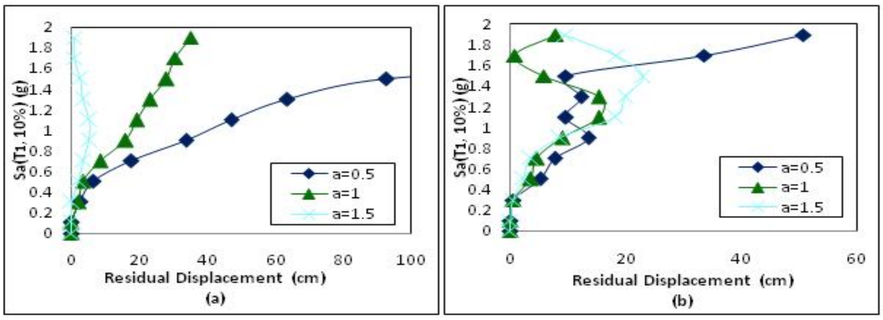

Fig. 19 – Residual displacement diagram of the structure affected by NF2 and FF2 earthquakes.

10. Conclusion

1. Linear viscous dampers under the effect of records of near and far field are reduced the residual displacement as well as the comparable cutting base values are produced under the effect of these records that it is contrary to the findings of researchers that the role of linear viscous damper in reducing the responses of the structure affected by near field records are insignificant.

2. Dampers with α = 1.5 show the best performance to reduce the relative displacement and residual displacement , Although in the same proportion increase the cutting base values that it may, the design of structures faced with a problem. Dampers with α = 0.5 create the lowest amount of reduction in relative displacement and residual displacement of structure and while the lowest values of cutting base produced in the structures by using this damper has been obtained.

3. Structure has a linear damper; with increasing amounts of critical the additional damping, responses obtained from actions of near and far field

records (cutting base, relative displacement and residual displacement) are improved. It was also observed that when the using the linear viscous damper, the general form of IDA diagrams is fixed that this represents a predictable behavior for this type of dampers are affected by earthquakes of near and far fields.

4. In most cases, residual displacement diagrams, is very similar to diagrams indicates the relationship of power- speed used damper. This clearly shows the structural response dependency to relationship of power-speed of damper.

5. Most of the dispersion among the relative displacement curves in case of using the damper with α = 0.5 and lowest dispersion in case of using damper with α = 1.5 was created.

REFERENCES

Bazzuro P., Cornell C.A., *Seismic Hazard Analysis for Non-Linear Structures I: Methodology*, Journal of Structural Engineering, ASCE, **120**, *11*, 3320-3344 (1994a).

Bazzuro P., Cornell C.A., *Seismic Hazard Analysis for Non-Linear Structures II: Methodology*, Journal of Structural Engineering, ASCE, **120**, *11*, 3345-3365 (1994b).

Constantinou M.C., Soong T.T., Dargush G.F., *Passive Energy Dissipation Systems for Structural Design and Retrofit*, Monograph Series No.I, Multidisciplinary Center for Earthquake Engineering Research, University at Buffalo, State Universityof New York at Buffalo, Buffalo, NY, 1998.

Constantinou M.C., Symans M.D., *Experimental Study of Seismic Response of Buildings with Supplemental Fluid Dampers*, The Structural Design of Tall Buildings, **2**, 93-132 (1993).

Gupta A., Krawinkler H., *Seismic Demands for Performance Evaluation of Steel Moment Resisting Frame Structures (SAC Task 5.4.3)*, Report No. *132*, The John A Blume Earthquake Engineering Center, Stanford University, 1999.

Lee K., Foutch D.A., *Performane Evaluation of New Steel Frame Buildings for Seismic Loads*, Earthquake Engineering and Structural Dynamics, **31**, *3*, 653-670 (2002).

Lin W.H., Chopra A.A., *Earthquake Response of Elastic SDF Systems with Non-Linear Fluid Viscous Dampers*, Earthquake Engineering and Structural Dynamic, **31**, *9*, 1623-1642 (2002).

Luco N., Cornell C.A., *Effects of Random Connection Fractures on Demand and Reliability for a 3-Story Pre-Northridge SMRF Structure*, Proc. of the 6th US National Conf. on Earthquake Engineering, EERI, Seattle, WA, 1998.

Makris N., Constantinou M.C., *Viscous Dampers: Testing, Modeling and Application in Vibration and Seismic Isolation*, Tech. Rep. NCEER-90-0028, National Center for Earthquake Engineering. Research State Univ. of New York (SUNY) at Buffalo, N.Y., 1990.

Mehanny S.S., Deierlein G.G., *Modeling and Assessment of Seismic Performance of Composite Frames with Reinforced Concrete Columns and Steel Beams*, Report No. *136*, The John A. Blume Earthquake Engineering Center, Stanford University, 2000.

Myiamoto H.K., Singh J.P., *Performance of Structure with Passive Energy Dissipators*, Earthquake Spectra, EERI, **18**, 105-119 (2002).

Powell G.H, PERFORM 3D Users Manual: Version 5, 2000.

Soong T.T., Dargoush G.F., *Passive Energy Dissipation Systems in Structural Engineering*, John Wiley & Ltd., London (UK) and New York (USA), 1997.

Vamvatiskos D., Cornell C.A., *Incremental Dynamic Analysis*, Earthquake Engineering and Structural Dynamics, **31**, *3*, 491-514 (2002).

* * * *Improvement of Nonlinear Static Seismic Analysis Procedures*, Applied Technology Council, FEMA- 440 (ATC- 55), Federal Emergency Management Agency, Washington DC, 2005.

* * * International Conference of Building Officials, Uniform Building Code, Whitter, California (1997).

* * * International Conference of Building Officials, Uniform Building Code, Whitter, California (2000).

* * * *NEHRP Recommended Provisions for Seismic Regulation for New Buildings and Other Structures*, FEMA, 2000.

ANALIZA DINAMICĂ INCRMENETALĂ A UNEI STRUCTURI CU AMORTIZARE VÂSCOASĂ CONSIDERÂND INFLUENȚA ACȚIUNII SEISMICE ÎN FUNCȚIE DE DISTANȚA EPICENTRALĂ

(Rezumat)

Lucrarea prezintă efectul cutremurelor de pământ asupra structurilor de construcții şi difrențele care apar în funcție de poziționarea structurii față de epicentru. Efectul mişcărilor seismice cu perioade mari de vibrație, în special în faza de început a mişcării seismice, este cu atât mai important cu cât se apropie de perioada proprie de vibrație a structurii analizate.

În lucrare de față se studiază efectul cutremurelor de pământ asupra sistemelor în cadre cu noduri rigide, cu şi fără amortizoare seismice, situate la diferite distanțe față de epicentru. Pentru acest lucru, s-a ales o structură în cadre cu noduri rigide având regimul de înălțime P+9E. În urma analizei comportamentului dinamic al structurii selectate, s-au trasat diagrame ale forței tăietoare la baza structurii, deplasările relative de nivel precum şi identificarea deformațiilor remamnente la nivelul structurii.