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Response of steel buildings under near and far field earthquakes

Serkan Etli^{a,b} and Esra Mete Güneysi^{a,*}

^aDepartment of Civil Engineering, Gaziantep University, 27310, Gaziantep, Turkey

^bDepartment of Civil Engineering, Munzur University, 62000, Tunceli, Turkey

*Corresponding author, e-mail: eguneyisi@gantep.edu.tr

Abstract

In this study, the seismic behavior of steel framed buildings under the effect of near-field with pulse (NF-P), near-field without pulse (NF-NP), and far-field (FF) ground motions was evaluated in a comparative manner. For this, 8 and 12 story steel moment-resisting frames (SMRFs) having four equal bays and identical story height were taken into account and then subjected to a series of NF-P, NF-NP and FF earthquake motions based on the nonlinear dynamic analysis. Each earthquake series included 10 different acceleration records. They were scaled to match the target design spectrum. Numerical modeling and analysis of the SMRFs were performed through Seismostruct software. The response quantities utilized in the comparisons of the case study SMRFs were the inter-story drift ratio, roof drift ratio, base shear coefficients, and time history of displacement in different story level. These responses were examined comparatively.

Keywords: Earthquake, Modeling, Nonlinear analysis, Seismic behavior, Steel frame

1. Introduction

In the course of the last century, many studies have been conducted on the destructive effects of the earthquake on the structure and how to minimize its subsequent damage. Even though these researches have important and positive results, the earthquake event with complex natural structure and the results obtained do not show the end of scientific discussions. It is reported that since the water reserves are located near the faults and the urbanization is concentrated in the areas close to the water resources, cities have been built near the faults as the main sources of the seismic waves. Investigations into the occurrence and effects of the earthquakes in these regions, called near field (NF) earthquakes, have become necessary [1].

Moreover, the collapsing structures caused by the last devastating earthquakes such as Northridge (1994), Kobe (1995), Kocaeli (1999), and Chi-Chi (1999), and the number of victims damaged in these structures have had great negative effects on the society. As a matter of fact, the common denominator for all these situations is that the source of the earthquake, as mentioned above, is located near or below the urban area. Moreover, the devastating effects of the earthquakes are observed in the near-field area, in the immediate vicinity of the epicentral region. It should be noted that NF ground motions are quite different from normal far field (FF) motions if the station is placed in the fault breaking direction. In fact, near-field areas, the normal-fault component of ground motion, is a pulse-type component with a long-term pulse in the acceleration time history. Such a significant blow is absent in ground motion recorded in locations far from NF. Therefore, the importance of considering the effects of NF motions in the seismic design of the buildings is clear. On the contrary, when most of the national and international seismic codes are examined, it is observed that the properties of ground motion are better documented and studied more in relation to the problems of FF or intermediate

epicentral distances and design procedures involving such problems [2]. On the other hand, NF earthquakes have different characteristics from FF ones. NF earthquakes have higher acceleration and more limited frequencies compared to the frequency of FF earthquakes. Seismographs of such earthquakes include long-pulsed high intervals periods observed at the beginning of the earthquake recordings, especially when subjected to progressive direction effects [3]. It is known that the seismic ground movements recorded in the NF region of an earthquake are quite different from the area in the remote FF, which is confirmed by the researchers in the literature [4–7].

Following the strong earthquakes mentioned above, the scientists, who examined the failure modes of the nearby frame structures, observed that in many cases these structures reached a collapse state, even though they were designed and detailed according to the criteria of the earthquake design codes. This means that there is a new situation that is not foreseen in the design practice. As a result, it is possible to underline that in the NF and FF cases, sometimes new dynamic states arise from the interaction between ground motions and structural behavior [7–10]. The behavior of structures under regular FF strong ground motions has been studied in the literature for a number of parameters such as stiffness distribution along with the supporting structure, location, damping ratio, and non-structural component, and higher mode effects. Mainly, the term NF represents permanent static displacement due to source, and the term FF represents dynamic response or transient seismic waves emitted by the source that does not cause permanent displacement. Owing to the unique characteristics of NF ground motions, the structural responses under NF ones have received much attention in recent years [11–17].

Furthermore, in the literature, there are some useful investigations regarding the actual response of the buildings under severe earthquakes. For example, in the study of Youssef et al. [18], the level and location of the damage observed in the steel buildings after 1995

*Corresponding Author: eguneyisi@gantep.edu.tr
(E.M.Güneysi ORCID: 0000-0002-4598-5582)

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Kobe earthquake was examined, and they reported that most of the damaged steel structures had moment resisting frame system as structural system rather than the braced ones. Depending on the structural system of the buildings having unbraced or braced frames, it was noted that the location of damage, namely columns, beams, beam-to-column connections, or column bases was varied [19, 20].

In the light of the above discussion, in this study, the effects of near and far field earthquakes on the behavior of steel moment-resisting frames (SMRFs) were investigated comparatively. Two SMRF structures having 8 and 12 story levels were used. Considering the characteristics of the earthquake records to be utilized in the nonlinear dynamic analysis, a total of 30 records were selected from Pacific Earthquake Engineering Research (PEER) database [21]. They were categorized as near-field with pulse (NF-P), near-field without pulse (NF-NP), and far-field (FF) ground motions. Seismostruct software [22] was used in the design and performance analysis of the structures. The obtained inelastic responses were given and discussed accordingly.

2. Structural Modeling and Analysis

In this study, two SMRF models were considered as 8 and 12 stories. In Fig. 1, the plan and elevation views of the structures are shown. For all frame models, the opening width is 7 m and the typical floor height is 3 m. The frames are designed in accordance with Eurocode-3 [23] and Eurocode-8 [24]. Since the case study SMRFs had a double symmetrical plan arrangement, a single plane frame was obtained from the structure and numerical modeling and analysis of the frames were conducted by using Seismostruct software [22]. For the structural modeling of beam and column elements, the force-based element model was used. This element model is defined as "infrmFB" in the software. Moreover, this element model uses the distributed plasticity with the integration sections located along the length of the member. Secondary deformations are also considered due to the P-Δ effect. Numerical simplifications have been made in numerical modeling; (1) the beam-column connections are assumed to be rigid; (2) the base columns are fixed to the foundation; the effect of soil-structure interaction is neglected; and (3) the shear effects of the beams are in the linear elastic range [25].

Fig.1 a) Plan and b) elevation views of 8 story and 12 story SMRF structures

In the design of SMRFs, PGA value was chosen as 0.3g. In addition to this value, ground type C and type-1 spectra were used in the response spectra. The modal analysis method as described in Eurocode-8 [24]

was utilized. The frame models designed with gravity loads for preliminary analysis then SMRFs were checked for seismic performance for predicted seismic situation. Therefore, designed structures were checked for ultimate limit state (ULS) and serviceability limit state (SLS) according to the analysis and design criteria given in Eurocode-8 [24]. While the design of the structure under the influence of earthquake forces, the strength requirement of the elements and the inter-story drifts of the structure, the stability of the structure were also taken into consideration. In addition to these, the sensitivity coefficient given in Eurocode-8 [24] was used to reflect the second-order effects on the element design.

In the design process, square hollow section steel box profiles (SHS) were selected for the steel column members as 650x45 and 800x60 mm and I profile were used for beams as IPE 550 and HEA 550 for 8-story and 12 story SMRFs, respectively. Modulus of elasticity (E), Poisson ratio (ν), and hardening coefficient (μ) values of the materials used in the element models were taken as 210×10^3 N/mm², 0.3 and 0.5%, respectively. The structural steel grade was taken as S235. The fundamental period and modal participation factors of the frames are summarized in Table 1.

Table-1 Period and modal participation factors

Frames	T ₁ (s)	T ₂ (s)	T ₃ (s)	U ₁ (%)	U ₂ (%)	U ₃ (%)
8STRY	1.119	0.328	0.161	76.765	11.362	5.077
12STRY	1.301	0.391	0.199	76.162	10.750	4.698

Earthquake records were selected to perform the nonlinear dynamic time history analysis. For this, three groups of the earthquake acceleration records were used. The first group was far field (FF), the second group was near-field without pulse (NF-NP), and the last one was near-field with pulse (NF-P) effects. Each group had 10 different acceleration records. Totally, 30 ground motion records were employed in the analysis. The characteristics of these earthquake records are presented in Table 2.

Table-2 Earthquake records used

No.	Name	Year	Component
Far-Field Earthquake Record Set			
TH-1	Friuli, Italy	1976	0
TH-2			270
TH-3	Kobe, Japan	1995	0
TH-4			90
TH-5	Kocaeli, Turkey	1999	0
TH-6			90
TH-7	Chi-Chi, Taiwan	1999	E
TH-8			N
TH-9	Manjil, Iran	1990	L
TH-10			T
Near-Field Earthquake Record Set			
No Pulse Records Subset			
TH-11	Gazli, USSR	1976	0
TH-12			90
TH-13	Cape Mendocino	1992	0
TH-14			90
TH-15	Northridge-01	1994	270
TH-16			360
TH-17	Loma Prieta	1989	0
TH-18			90
TH-19	Loma Prieta	1989	0
TH-20			90
Pulse Records Subset			
TH-21	Loma Prieta	1989	0
TH-22			90
TH-23	Cape Mendocino	1992	0
TH-24			90
TH-25	Landers	1992	260
TH-26			345
TH-27	Northridge-01	1994	90
TH-28			360
TH-29	Chi-Chi, Taiwan	1999	E
TH-30			N

All records scaled according to Eurocode 8 elastic response spectra were utilized for evaluating the inelastic response of the structures. For this, SeismoMatch software [26] was utilized to scale the earthquake acceleration records. During the scaling process of the earthquakes to be employed during inelastic analysis, the design

spectrum characteristics were taken into consideration and performed in the 0-4 s period. In addition, Al Atik and Abrahamson [27] method was used as a scaling method in the software. The elastic acceleration spectrum of the selected FF, NF-NP and NF-P type earthquakes with the elastic response spectra (Eurocode 8) are given in Fig. 2. By means of these earthquake motions, the nonlinear dynamic analysis (NDA) of the frames was performed, and their responses were discussed comparatively.

Fig.2 Acceleration response spectrum of a) original and b) scaled earthquakes

3. Results and Discussion

The inter-story drift ratio (IDR) is an important parameter used to evaluate the seismic performance of civil engineering structures [28]. As shown in Figs. 3a and 3b, the average of IDR_{max} values for the 8-story SMRF building based on NF-NP and NF-P earthquake records is higher than the average value of the IDR_{max} obtained for the 12-story SMRF building considering the same earthquake record groups. Furthermore, the average of the IDR_{max} values gathered under FF type earthquake records is nearly the same for 8-story SMRF and for 12-story SMRF structures, which is approximately 0.011.

In the NDA analysis of the 8-story SMRF building considering FF earthquakes, the greatest IDR_{max} value was obtained under the TH-1 (Friuli, Italy) earthquake as 0.0143, whereas in the NDA of the 12-story SMRF building the greatest value was obtained under TH-3 (Kobe, Japan) earthquake as 0.0180. Thus, based on FF-type recordings, the resulting greatest IDR_{max} value observed in 12-story SMRF is higher than 8-story SMRF. However, from the result of NDA made using NF-NP and NF-P earthquake records, in 8-story SMRF the greatest IDR_{max} values were obtained as 0.0164 for TH-18 (Loma Prieta) and 0.01527 for TH-22 (Loma Prieta), while in 12-story SMRF those were obtained as 0.0134 for TH-19 (Loma Prieta) and 0.0118 for TH-24 (Cape Mendocino), for NF-NP and NF-P earthquake sets, respectively. Therefore, the highest IDR_{max} value obtained as a result of NDA from the FF earthquake group for 8 story SMRF is smaller than that was obtained for 12-story SMRF structure. On the other hand, for the 8-story SMRF structure, the IDR_{max} value obtained as a result of NDA from the NF-NP and NF-P earthquake group is higher than the IDR value obtained for the 12-story SMRF structure.

The smallest IDR_{max} values under FF, NF-NP and NF-P earthquake records were obtained for 8-story SMRF structure as 0.0093 under TH-2 (Friuli, Italy), 0.0098 under TH-16 (Northridge-01) and 0.0107 under TH-21 (Loma Prieta), respectively. The smallest IDR_{max} values for the 12-story SMRF structure were obtained as 0.0088 under TH-9 (Manjil, Iran), 0.0077 under TH-16 (Northridge-01) and 0.0086 under TH-21

(Loma Prieta) for the analyzed earthquake sets of FF, NF-NP and NF-P, respectively. From the results, it was also observed that the lowest value of IDR_{max} for 8 story SMRF was obtained among FF earthquake set while for 12-story the lowest value was obtained among NF-NP earthquake set.

Fig.3 Variation of the maximum inter-story drift ratio (IDR) for each earthquake group: a) 8 story and b) 12 story SMRFs

In Fig. 4, the variation of IDR_{ave} values of the structures at each story level for each earthquake record group is given. When these values were examined, the IDR_{ave} varied from 0.2 to 1.3%, depending on earthquake type and building properties. The IDR_{ave} value in the 8-story SMRF ranged from 0.6 to 0.7% for the FF, NFP and NF groups, while in the 12-story SMRF structure these values decreased and reached 0.2 and 0.3% at the roof story. The IDR_{ave} values of these earthquake groups in the middle stories vary between 1.0% and 1.3% in the 8-story structure, while they vary between 0.80% and 1.1% in the 12-story structure (Fig. 4a and 4b). In addition to these, when we investigated the highest IDR_{ave} values which were obtained for the middle story of the 8 story SMRF, the order from the higher to lower is obtained NF-NP, NF-P and FF, showing that under near pulse earthquakes greater IDR was observed. On the other hand, when the highest IDR_{ave} values obtained for the middle story of the 12 story SMRF were investigated, the order from the higher to lower is obtained under FF, NF-NP and NF-P earthquake sets, displaying that unsimilar to the 8 story building under far field earthquakes greater IDR was observed.

Evaluation of performance level of the structures varies depending on the guidelines that result from intensive research. Based on FEMA-273 [29] and Vision 2000 [30], the performance level can be divided into several performance stages which include fully operational, operational, immediate occupancy, damage control, life safety, collapse prevention and near collapse. The drifts proposed for these performance levels are 0.2% for fully operational (FO), 0.5% for operational (OP), 1% for immediate occupancy (IO), 1.5% for damage control (DC), 2% for life safety (LS), and 2.5% for collapse prevention (CP), and 3% for near collapse (NC). Considering the above limit values in terms of the performance limits, the 8-story SMRF structure mostly

reaches the DC limit under the effects of FF, NF-NP and NF-P earthquakes. However, under some earthquakes such as TH-18 (Loma Prieta), IDR value is 0.0164 and TH-22 (Loma Prieta), the IDR value is 0.0153, slightly exceeding DC and within LS performance level. Therefore, even if the number of earthquakes with LS performance is limited, 8-story SMRF structure may have more seismic demand under the effects of NF-NP and NF-P earthquakes. By examining the 12-story SMRF building in terms of performance limits by considering the above limit values, 50% of NDA analysis satisfies DC and the rest satisfies IO limit under the effects of FF, NF-NP and NF-P earthquakes for each group records. As a result, under the effects of FF, NF-NP and NF-P earthquakes, the seismic demand of 8-story SMRF structure was observed to be more than that of the 12-story SMRF structure.

Fig.4 Variation of average value of the maximum inter-story drift ratio (IDR) with story level for SMRFs

For the FF, NFP and NF earthquake groups, the base shear coefficient (V_{base}/W) values obtained as a result of dynamic time-history analysis are given in Fig. 5. This coefficient, which is defined as the ratio of the maximum base shear force obtained during the analysis to the structure weight (W) which is the basis of the design of the structures, are examined in this section.

For the 8-story SMRF building, the highest (V_{base}/W)_{max} values as a result of NDA using the earthquake records in the FF, NF-NP and NF-P groups were obtained as 0.3384 (TH-1, Friuli, Italy), 0.3483 (TH-14, Cape Mendocino) and 0.3381 (TH-28, Northridge-01), respectively. For the 12-story SMRF building, the greatest (V_{base}/W)_{max} values obtained under earthquake records within FF, NF-NP and NF-P groups were as 0.3400 (TH-7, Chi-Chi, Taiwan), 0.3552 (TH-12, Gazli, USSR) and 0.3390 (TH-28, Northridge-01), respectively. On the other hand, when the lowest values of (V_{base}/W)_{max} were examined, the values obtained for the 8-story SMRF building from the NDAs under the FF, NF-NP and NF-P earthquake record groups were found to be 0.2259 (TH-7, Chi-Chi, Taiwan), 0.2551 (TH-15, Northridge-01) and 0.2577 (TH-30, Chi-Chi, Taiwan), respectively. In addition, the smallest values of (V_{base}/W)_{max} for the 12-story SMRF structure obtained as a result of the NDA with earthquake records within FF, NF-NP and NF-P groups were found as

0.2502 (TH-4, Kobe, Japan), 0.2517 (TH-17, Loma Prieta) and 0.2580 (TH-25, Landers), respectively.

Fig.5 Variation of the base shear coefficients (V_{base}/W) for each earthquake group: a) 8 story and b) 12 story SMRFs

When the average value of (V_{base}/W)_{max} are examined, the values obtained for the 8-story SMRF building under the earthquakes within FF, NF-NP and NF-P sets were found to be 0.2866, 0.2972 and 0.2981, respectively. In addition, the average values (V_{base}/W)_{max} obtained for the 12-story SMRF structure were observed as 0.3004, 0.2940 and 0.2921 under the NDA with FF, NF-NP and NF-P earthquake groups, respectively. As expected, all the average values of (V_{base}/W)_{max} is very close to the initial design assumption of 0.3 for both 8 and 12 story structures.

While evaluating the structural performance of the structures against seismic activity loads, measures will be taken for critical damages such as cracks, displacements and drifts that may lead to collapse of the structures. As building heights increase, forces of nature begin to dominate the structural system and gain importance in the general building system. The analysis and design of especially tall building is influenced by lateral loads, in particular by drift or shaking caused by these loads. Drift or swing is the magnitude of lateral displacement at the top of the building relative to the base [31]. Several terms are used to describe displacements with different meanings. The following definitions will clarify their use here. Global displacements, Δ , represent displacement relative to the base of an equivalent single degree of freedom system representing the structure. The roof displacement refers to the lateral displacement of the roof of the building relative to the base and the term Δ_{roof} is used to describe this. The drift between stories is the relative horizontal displacement between two adjacent stories and the term Δ_i is used here to define the value of drift at i^{th} story. The drift ratio corresponds to the drift between the stories divided by the story height Δ_i/h_i ; where h_i is story height of the i^{th} story. The roof drift ratio is related to the division of the roof displacement by the total height of the building [32].

The graphs of the roof drift ratio (Δ_{roof}/H) are given in Fig. 6a and 6b for 8-story and 12-story SMRFs, respectively. When the (Δ_{roof}/H)_{max} values are analyzed, the values obtained as a result of NDA made with the earthquake records in the FF, NF-NP and NF-P group in the 8-story SMRF structure were obtained as 0.0101 (TH-1, Friuli, Italy), 0.0121 (TH-18, Loma Prieta) and 0.0106 (TH-22, Loma Prieta), respectively. In the 12-story SMRF structure, the (Δ_{roof}/H)_{max} values obtained as a result of the NDA made with the earthquake records in the FF, NF-NP and NF-P groups were obtained as 0.0076 (TH-1, Friuli, Italy), 0.0084 (TH-19, Loma Prieta) and 0.0078 (TH-28, Northridge-01), respectively. Thus, results of the NDA of both buildings displayed that the maximum values for the roof drift were observed under earthquake record within the NF-NP group. As expectedly, for all types of earthquakes, the greatest (Δ_{roof}/H)_{max} values obtained for the 8 story building is higher than those obtained for 12 story SMRF.

Fig.6 Variation of the roof drift ratio (Δ_{roof}/H) for each earthquake group: a) 8 story and b) 12 story SMRFs

When the lowest values for (Δ_{roof}/H)_{max} were examined, the values obtained from the NDA of the 8-story SMRF building under the sets of FF, NF-NP and NF-P earthquakes were found to be 0.0065 (TH-5, Kocaeli, Turkey), 0.0068 (TH-16, Northridge-01) and 0.0072 (TH-21, Loma Prieta), respectively. In addition, the minimum (Δ_{roof}/H)_{max} values obtained for 12-story structure under FF, NF-NP and NF-P earthquakes were determined as 0.0055 (TH-9, Manjil, Iran), 0.0050 (TH-13, Cape Mendocino) and 0.0054 (TH-23, Cape Mendocino), respectively. Similar to the greatest values, when we investigated the lowest (Δ_{roof}/H)_{max} values, those obtained for the 8 story building is higher than those obtained for 12 story building. The minimum (Δ_{roof}/H)_{max} value for the 8-story SMRF building was obtained as a result of the NDA made by earthquake records in the FF group, while the lowest value for (Δ_{roof}/H)_{max} for the 12-story SMRF building was obtained as a result of the NDA made by the earthquake records in the NF-NP group.

When the average (Δ_{roof}/H)_{max} values of the 8-story SMRF building under the earthquake records in the FF, NF-NP and NF-P group were examined, the obtained values were found to be 0.0079, 0.0087 and 0.0085, respectively. In addition, average (Δ_{roof}/H)_{max} values as a

result of NDA under earthquakes within FF, NF-NP and NF-P groups were observed as 0.0063, 0.0064 and 0.0065 for 12-story SMRF structure, respectively.

As an example, the time history response of the 12-story SMRF exposed to Chi-Chi, Taiwan (TH-7), Northridge-01 (TH-16) and Cape Mendocino (TH-24) earthquakes are given in Fig.7. When the results of the displayed response histories of the first, sixth and last floors of the structure were examined, it was observed that the difference between the sixth and twelfth floor displacements in FF type earthquake records is more apparent than that of NF-NP and NF-P type earthquakes.

Fig.7 Time history response of first story, six story and roof displacements (Δ) for 12 story SMRF under a) Chi-Chi (FF, TH7), b) Northridge-01 (NF-NP, TH16), and c) Cape Mendocino (NF-P, TH24) earthquakes

4. Conclusions

The performance of steel moment resisting frame (SMRF) buildings under earthquake effects having different characteristics was

studied. For this, 8 and 12 story SMRFs were modeled and analyzed by using Seismostruct software. In the nonlinear dynamic analysis (NDA), ground motions classified in three groups as near-field with pulse (NF-P), near-field without pulse (NF-NP), and far-field (FF) were utilized. The seismic response of the structures was examined. The conclusions based on the findings of this study could be summarized as follows:

- It was pointed out that the response of frames varied under the earthquakes with different features.
- When the IDR values were examined, it was observed that the average of the IDR_{max} values obtained as a result of the NDA of the 8 story and 12 story structures were almost the same under the FF group earthquakes.
- However, when the average IDR_{max} values obtained by using NF-NP and NF-P group earthquake records were evaluated, the average of the IDR_{max} values for the 8-story SMRF structure were observed to be greater than the average of the IDR_{max} values obtained for the 12-story SMRF structure.
- From the average of the maximum Δ_{roof}/H values obtained as a result of the NDA of the case studied structures, for all earthquake types, the average of the $(\Delta_{roof}/H)_{max}$ values for the 8-story SMRF were higher than the average of the $(\Delta_{roof}/H)_{max}$ values obtained for the 12-story SMRF type structures.
- For both 8 story and 12 story structures, even though there were differences in $(V_{base}/W)_{max}$ observed under different earthquakes, the average of $(V_{base}/W)_{max}$ calculated for each earthquake set gave close to 0.3, which is concordant with the initial design assumptions.
- To represent more general conclusions, the number of buildings could be increased, and so the effect of various numbers of stories and also different structural systems could be investigated as a further research.

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