P-Delta Effects on Nonlinear Seismic Behavior of Steel Moment-Resisting Frame Structures Subjected to Near-Fault and Far-Fault Ground Motions

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Abstract: This paper presents a comparison of P-Delta effects on the nonlinear seismic behavior of the steel moment-resisting frame structures (MRFs) subjected to near-fault and far-fault ground motions. The 3-, 9- and 20-story MRFs designed for the American SAC Phase II Steel Project are used as benchmark models. The 40 near-fault ground motions with large velocity pulses, as well as ten typical far-fault ground motions, are selected and scaled for the nonlinear time-history analysis. The P-Delta effect is quantified based on peak inter-story drift ratio (PIDR) demands. The displacement demands of the whole structure and the distortion of the structural components are compared and analyzed. It was found that, at each floor, the P-Delta effect under near-fault ground motions is more significant than that under the far-fault ground motions. The P-Delta effect under near-fault ground motions also increases more rapidly with decreasing structure height even for low-rise structures or low earthquake intensity. It was also found that the P-Delta effect cause the PIDR demands to increase by 10% for all three structures subjected to far-fault ground motions. In contrast, considering the P-Delta effect, the PIDR demands rapidly increase by 45% for the high-rise building subjected to near-fault ground motions. Note that the increasing PIDR demands occur at the weakest floor and with the stronger earthquake intensity. However, the P-Delta effect does not change the location of the weakest floor and the yield sequence of components. The seismic behaviors under far-fault and near-fault ground motions are significantly different, because near-fault ground motions not only have velocity pulse but also possibly trigger structural higher vibration modes. In addition, the P-Delta effect may change the distortion direction of the components so that the prediction of the structural collapse direction may be affected. In addition, it was found that if the structure’s period is near the pulse period, the P-Delta effect becomes more significant with the increase of earthquake intensity, and accordingly, it should not be ignored. Moreover, the P-Delta effect cannot be neglected either for the structures susceptible to near-fault ground motions, even if those structures are not tall or the earthquake intensity is not strong.

Keywords: P-Delta effects; steel moment-resisting frame structures; near-fault ground motions; far-fault ground motions; inter-story drift ratio; velocity pulse; pulse period

1. Introduction

P-Delta effects can be very severe for steel moment-resisting frame structures (MRFs) because the structures are usually flexible and may be subjected to large lateral displacements. Thus, P-Delta effects should be considered in the structural design. There are many
studies on the influence of P-Delta effects on seismic behavior. However, these studies have mainly focused on high-rise structures subjected to far-fault ground motions [1–10]. Most practitioners also believe that P-Delta effects need to be considered only for high-rise buildings. Thus, P-Delta effects on low-rise buildings can be neglected at lower earthquake intensity.

However, the characteristics of near-fault ground motions are very different from the far-fault ground motions. The main difference can be attributed to the inherent velocity pulses that can result in large displacement demands and higher collapse risks of the structures. There is also still very severe damage for lower-rise buildings subjected to near-fault ground motions at lower earthquake intensity [11–18].

At present, there is a lack of research on comparing the P-Delta effects on dynamic responses of MRFs subjected to near-fault ground motions and far-fault ground motions. Yang et al. [19] focused on the inter-story drift ratio demands of MRFs subjected to near-fault ground motions. They analyzed the displacement demands of MRFs using the distinct property of near-fault impulsive ground motions and generalized drift spectral analysis. However, Yang et al. did not involve P-Delta effects. Domizio et al. [20] quantified P-Delta effects on the structural collapse subjected to near-fault ground motions based on the experimental and numerical analyses. However, the structural example in the study of Domizio et al. [20] was not a multiple degrees of freedom steel structure.

The main objective of this paper is to determine and compare P-Delta effects on MRFs subjected to near-fault and far-fault ground motions. For this purpose, the 3-, 9- and 20-story MRFs designed for the American SAC Phase II Steel Project are used as benchmark numerical models [21–23] and two models are built. In the first model, P-Delta effects are considered, while in the other model, P-Delta effects are neglected. Three near-fault ground motions with large velocity pulses, as well as three typical far-fault ground motions, are also selected and scaled to three levels of the peak ground acceleration (PGA). Then, these motions are used as input for the nonlinear time-history analysis. Note that the inter-story drift ratio demands are used not only as an important damage demand parameter of buildings under earthquake loads but also to quantify the P-Delta effect. In addition, the displacement demands of the whole structure and the distortion of the structural components subjected to near-fault ground motions are compared and analyzed. Moreover, the relationship between the pulse periods of the near-fault ground motion records and P-Delta effects on the structural nonlinear earthquake behavior is established.

2. Near-Fault and Far-Fault Ground Motions

The potential importance of P-Delta effects on the seismic response of MRFs necessitates the nonlinear time-history analysis to properly evaluate a structural response under time-varying loads. In this study, 40 near-fault and 10 far-fault ground motion records are selected as input ground motions. Near-fault ground motion records have many kinds of the mechanisms, so the number of the near-fault ground motion records is larger than that of the near-fault ground motion records in order to consider more scenarios. The key properties for characterizing these ground motions are given in Table 1. The 40 near-fault ground motion records are from the Pacific Earthquake Engineering Research Center (PEER) transportation database [23]. They all have velocity pulses, and their $T_p$ has been recognized using wavelet analysis [24]. The 10 far-field ground motions, whose recorded distances are ranged from 39 to 82 km, include some records selected from [25] and some representative records, such as I-ELC180 from El Centro Array #9 and TAF021 from 1095 Taft Lincolnl School, which are commonly used by designers and scholars.
Table 1. Properties of selected near-fault and far-fault ground motion records.

| ID | Earthquake       | Year | Station                        | Mw  | dR up (km) | Tp  (s) | PGA (g) | V S30 (m/s) |
|----|------------------|------|-------------------------------|-----|------------|--------|--------|-------------|
|    | Near-fault ground motions                          |      |                               |     |            |        |        |             |
| 1  | Imperial Valley | 1979 | EC County Center FF           | 6.5 | 7.3        | 4.4    | 0.18   | 192         |
| 2  | Imperial Valley | 1979 | EC Meloland Overpass          | 6.5 | 0.1        | 3.4    | 0.38   | 265         |
| 3  | Imperial Valley | 1979 | El Centro Array #4            | 6.5 | 7.1        | 4.8    | 0.36   | 209         |
| 4  | Imperial Valley | 1979 | El Centro Array #5            | 6.5 | 4.0        | 4.1    | 0.38   | 206         |
| 5  | Imperial Valley | 1979 | El Centro Array #6            | 6.5 | 1.4        | 3.8    | 0.44   | 203         |
| 6  | Imperial Valley | 1979 | El Centro Array #7            | 6.5 | 0.6        | 4.4    | 0.46   | 211         |
| 7  | Imperial Valley | 1979 | El Centro Array #8            | 6.5 | 3.9        | 5.4    | 0.47   | 206         |
| 8  | Imperial Valley | 1979 | El Centro Differential Array  | 6.5 | 5.1        | 6.3    | 0.42   | 202         |
| 9  | Morgan Hill     | 1984 | Coyote Lake Dam               | 6.2 | 0.5        | 1.1    | 0.81   | 561         |
| 10 | Loma Prieta     | 1989 | Gilroy-Gavilan Coll.          | 6.9 | 10.0       | 1.8    | 0.29   | 730         |
| 11 | Loma Prieta     | 1989 | LGPC                          | 6.9 | 3.9        | 4.4    | 0.94   | 595         |
| 12 | Landers         | 1992 | Lucerne                       | 7.3 | 2.2        | 5.1    | 0.70   | 1369        |
| 13 | Landers         | 1992 | Yermo Fire Station            | 7.3 | 23.6       | 7.5    | 0.24   | 354         |
| 14 | Northbridge     | 1994 | Jensen Filter Plant           | 6.7 | 5.4        | 3.2    | 0.52   | 373         |
| 15 | Northbridge     | 1994 | Jensen Filter Plant           | 6.7 | 5.4        | 3.5    | 0.52   | 356         |
| 16 | Northridge      | 1994 | Newhall-Fire Sta              | 6.7 | 5.9        | 1.4    | 0.72   | 269         |
| 17 | Northridge      | 1994 | Newhall W Pico Canyon Rd.     | 6.7 | 5.5        | 3.0    | 0.43   | 286         |
| 18 | Northridge      | 1994 | Rinaldi Receiving Sta         | 6.7 | 6.5        | 1.2    | 0.87   | 262         |
| 19 | Northridge      | 1994 | Sylmar-Converter Sta          | 6.7 | 5.4        | 3.0    | 0.59   | 251         |
| 20 | Northridge      | 1994 | Sylmar-Converter Sta          | 6.7 | 5.2        | 3.5    | 0.83   | 371         |
| 21 | Northridge      | 1994 | Sylmar-Olive View             | 6.7 | 5.3        | 2.4    | 0.73   | 441         |
| 22 | Kobe, Japan     | 1995 | KJMA                          | 6.9 | 1.0        | 1.1    | 0.85   | 312         |
| 23 | Kobe, Japan     | 1995 | Takarazuka                    | 6.9 | 0.3        | 1.8    | 0.65   | 312         |
| 24 | Kocaeli, Turkey | 1999 | Gebze                         | 7.5 | 10.9       | 6.0    | 0.24   | 792         |
| 25 | Chi-Chi, Taiwan | 1999 | CHY028                        | 7.6 | 3.1        | 2.2    | 0.66   | 543         |
| 26 | Chi-Chi, Taiwan | 1999 | CHY101                        | 7.6 | 9.9        | 5.3    | 0.38   | 259         |
| 27 | Chi-Chi, Taiwan | 1999 | TCU049                        | 7.6 | 3.8        | 10.2   | 0.29   | 487         |
| 28 | Chi-Chi, Taiwan | 1999 | TCU052                        | 7.6 | 0.7        | 8.4    | 0.38   | 579         |
| 29 | Chi-Chi, Taiwan | 1999 | TCU053                        | 7.6 | 6.0        | 13.1   | 0.22   | 455         |
| 30 | Chi-Chi, Taiwan | 1999 | TCU054                        | 7.6 | 5.3        | 10.5   | 0.16   | 461         |
| 31 | Chi-Chi, Taiwan | 1999 | TCU068                        | 7.6 | 0.3        | 12.3   | 0.56   | 487         |
| 32 | Chi-Chi, Taiwan | 1999 | TCU073                        | 7.6 | 0.9        | 5.0    | 0.33   | 57          |
| 33 | Chi-Chi, Taiwan | 1999 | TCU076                        | 7.6 | 2.7        | 4.7    | 0.31   | 615         |
| 34 | Chi-Chi, Taiwan | 1999 | TCU082                        | 7.6 | 5.2        | 8.1    | 0.23   | 473         |
| 35 | Chi-Chi, Taiwan | 1999 | TCU087                        | 7.6 | 7.0        | 10.4   | 0.13   | 539         |
| 36 | Chi-Chi, Taiwan | 1999 | TCU101                        | 7.6 | 2.1        | 10.3   | 0.21   | 389         |
| 37 | Chi-Chi, Taiwan | 1999 | TCU102                        | 7.6 | 1.5        | 9.6    | 0.30   | 714         |
| 38 | Chi-Chi, Taiwan | 1999 | TCU103                        | 7.6 | 6.1        | 8.7    | 0.13   | 494         |
| 39 | Chi-Chi, Taiwan | 1999 | TCU122                        | 7.6 | 9.3        | 10.9   | 0.22   | 475         |
| 40 | Chi-Chi, Taiwan | 1999 | WGK                           | 7.6 | 10.0       | 4.4    | 0.30   | 259         |
|    | Far-fault ground motions                          |      |                               |     |            |        |        |             |
| 1  | Imperial Valley | 1940 | El Centro Array #9(-1ELC180)  | 6.9 | 6.1        | -      | 0.31   | 213         |
| 2  | Kern County     | 1952 | School(TAFFO21)               | 7.7 | 41.0       | -      | 0.05   | 385         |
| 3  | Colinga         | 1983 | Parkfield-Vineyard Cany       | 6.4 | 32.2       | -      | 0.04   | 309         |
| 4  | Taiwan Smart1   | 1986 | Smart1E02                     | 7.3 | 51.4       | -      | 0.04   | 672         |
| 5  | Imperial Valley | 1979 | Coachella Canal # 4           | 6.5 | 50.1       | -      | 0.15   | 337         |
| 6  | Cape Mordenico  | 1992 | Butler Valley Station         | 7.0 | 45.4       | -      | 0.09   | 525         |
| 7  | Northridge      | 1994 | Manhattan-Beach               | 6.7 | 39.2       | -      | 0.12   | 352         |
| 8  | Chi-Chi         | 1999 | TCU045-W                      | 7.6 | 24.1       | -      | 0.16   | 705         |
| 9  | Iwate           | 2008 | Rifu Town                     | 6.9 | 57.8       | -      | 0.16   | 521         |
| 10 | Darfield        | 2010 | CSFS                          | 7.0 | 43.6       | -      | 0.47   | 638         |

Note that the near-fault ground motion records considered in this study are significantly different from those at far-fault region. Figure 1 compares the acceleration, velocity, and displacement time-history of TCU052 and TCU045-W. TCU052 shows two important characteristics, which are a pulse-like velocity wave form and a permanent ground displacement. Though TCU052 and TCU045-W have very similar PGA, those characteristics above are not included in TCU045-W, which has significantly lower velocity and displacement. It has been verified that all three near-fault ground motions have large velocity pulses, according to the essential conditions mentioned by Baker [24].
In order to investigate the difference between the near-fault and far-fault ground motions at the same earthquake intensity, three near-fault ground motions, including the 28th, 30th, and 37th records of the group (c), and three far-fault ground motions, including the 1st, 2nd, and 8th records of the group (d), are scaled to the same PGA, i.e., 0.2, 0.4 and 0.6 g, and used as the input records for the additional nonlinear time-history analysis. The three near-fault ground motion records of the 1999 Chi-Chi earthquake are taken from station numbers TCU052, TCU054, and TCU102. The other three far-fault ground motions records are from Imperial Valley, Kern County and the 1999 Chi-Chi earthquake and from station numbers I-ELC180, TAF021 and TCU045-W, respectively. These far-fault ground motions records are representative and very commonly used by designers and scholars. The scaled acceleration, velocity, and displacement response spectra of the six records are shown in Figure 2, in which the PGA is equal to 0.2 g. It can be noticed that all response spectra are similar in the short period range, i.e., less than 1.0 s. However, response spectra of the three near-fault ground motions are higher than that of the three far-fault ground motions in the long period range, i.e., larger than 1.0 s. The response spectra of the three near-fault ground motions also have bumps in the period range of 1.3–2.3 s. Note that the bumps are mainly attributed to their pulse-type characteristics. For example, as shown in Figure 1, the velocity and time-history of TCU052 has one or two pulses in the time region of 35–40 s. Therefore, the structural response excited by the near-fault ground motions would be larger than that of the far-fault ground motions. In addition, the effects of higher modes that occurred due to the near-fault ground motions should be examined. Moreover, it should be expected that the near-fault ground motions would affect longer buildings more than shorter ones due to large response spectra at longer periods.
Figure 2. The acceleration, velocity, and displacement response spectra of the six records.

3. Case Studies

The 3-, 9- and 20-story MRFs conforming to the seismic code are designed for the American SAC Phase II Steel Project under the assumption that they are located in Los Angeles. Although these three buildings have not been actually constructed, they, respectively, represent the typical low, medium, and high-rise buildings in regions with a high risk of earthquake. Note that the detailed plan and elevation in the north-south direction of the moment-resisting frame structures were previously published [22].

Following previously published structural details [22], the three buildings are modeled using the Finite element method (FEMs) of the ABAQUS 6.12 platform. It should be noted that the moment-resisting frames are placed on the structural perimeter, resulting in the presence of many interior frames, which can be designed to carry gravity loads only. Thus, in the analysis, considering the interior frames is not required. Only moment-resisting frames that are resistant to the horizontal forces, such as wind and earthquake loads, are modeled as two-dimensional frames representing half of the structure in one direction [22]. It is worth mentioning that B22 elements are used to model the beams and columns. The B22 element in the ABAQUS 6.12 platform is developed based on the Timoshenko beam element, which can consider the shear deformation. There are two integration points in the element, and both the deep beam and slender beam can be properly simulated by the B22 element, whether or not the shear deformation plays a major contribution to the structural response. The bilinear material constitutive model with a strain-hardening ratio of 0.03 is also applied to the steel material. In addition, Rayleigh damping with a damping ratio of 2% is used for the first and second vibration modes. Following [21], in the FEM model, the shear behavior of the Panel Zone is considered. Note that this model uses rigid links to hold panel zones in all dimensions. In addition, this model controls the panel zone’s deformation using one spring, which simulates a tri-linear behavior.

P-Delta effects can be reflected by many methods, among which the geometric nonlinearity with the element-by-element basis is the most common method in structural finite
element analysis. However, this method is only suited to a three-dimensional (3D) model, thus, it is not appropriate for the two-dimensional (2D) FEM model that was built in the present study. For the sake of simplification, 2D numerical models of the steel moment-resisting frame structures are only established according to the peripheral frames. Indeed, applying all gravity loads directly to the elements is not reasonable because it affects their capacities, especially, the columns’ elements will carry overlarge vertical loads. In this case, there are no components resisting the whole structural gravity loading, let alone considering the P-Delta effect. Hence, it is necessary to establish other components to carry the whole structural gravity loading. One simple way to reflect P-Delta effects in a 2D model is that the building’s gravity load is carried by a leaning column rather than the moment frame [21,26]. As shown in Figure 3, rigid links are used to connect the column to the moment frame with hinges at both top and bottom ends. Therefore, only an additional overturning moment of the lateral displacement will be induced. In addition, the leaning columns do not carry lateral loads because they are pinned. Note that the learning columns have the same material with the MRF columns and also have the following properties at each story level. The moment of inertia of the leaning column is equal to half the sum of moment of inertia of all gravity columns and orthogonal MRF columns, with the correct bending axis represented.

![Figure 3. Modeling interior columns for P-Delta effect.](image)

The present study focuses on P-Delta effects on the seismic response of the steel moment-resisting buildings. Two different building models are investigated, which build nonlinear springs for yielding of panel zones according to [21]. The first model, denoted as M1-PD, has a leaning column attached to the moment-resisting frame to correctly account for the building’s P-Delta effects. For comparison, the second model, denoted as M2-noPD, does not have a leaning column and does not consider the P-Delta effects.

For the 3-, 9- and 20-story structures, both models are developed and analyzed dynamically. The lower three vibration periods of the three structures obtained by both models are listed and compared in Table 2. The periods for both models are extremely similar, mainly because of their consistent mass and stiffness. Additionally, in the present paper, the periods are compared with the results of a previous study [22]. Among these models, the largest relative error was less than 13%. Note that this error may be a result of ignoring the shear deformation of the panel zone [22].

The M1-PD and M2-noPD have almost similar lower vibration modes for the three structures. The inter-story drift ratios over the height of the M1-PD model for the first three–five vibration modes are shown in Figure 4. For the 9-story and 20-story structures, at each story, there is less difference in the first vibration. However, the inter-story drift ratio over the height of the structures undulates more obviously in the higher vibration modes. In other words, the higher vibration modes, the more waves exist along the height of the structures.
Table 2. Vibration periods of the 3-story, 9-story and 20-story structures.

| Model  | Vibration Mode | M1-PD (s) | M2-noPD (s) | Relative Error to M2-noPD (%) | Reference (s) [22] | Relative Error to Reference [22] (%) |
|--------|----------------|-----------|-------------|-------------------------------|---------------------|-------------------------------------|
| 3-story | 1st            | 1.01      | 1.00        | 1.00                          | 1.01                | 0.0                                 |
|        | 2nd            | 0.31      | 0.31        | 0.0                           | 0.33                | −6.1                                |
|        | 3rd            | 0.15      | 0.16        | −6.3                          | 0.17                | −11.8                               |
| 9-story | 1st            | 2.17      | 2.17        | 0.0                           | 2.27                | −4.4                                |
|        | 2nd            | 0.81      | 0.82        | −1.2                          | 0.85                | −4.7                                |
|        | 3rd            | 0.45      | 0.46        | −2.2                          | 0.49                | −8.2                                |
| 20-story| 1st            | 4.13      | 4.12        | 0.2                           | 3.85                | 7.3                                 |
|        | 2nd            | 1.47      | 1.47        | 0.0                           | 1.33                | 10.5                                |
|        | 3rd            | 0.87      | 0.86        | 1.2                           | 0.77                | 13.0                                |

Figure 4. Inter-story drift ratios over the height of the three M1-PD models for the first three–five vibration modes.

4. Quantification of P-Delta Effects

Story shear forces are usually defined as the sum of inertial forces above a certain story. In addition, the base shear forces are measured as the sum of products of the mass of the floor plate and the absolute acceleration. Figure 5a shows the relationship between peak inter-story drift ratio (PIDR) demands at the first floor and normalized base shear forces of the M2-noPD model applied to the 20-story structure subjected to TCU052 (PGA = 0.6 g). Note that, in Figure 5, where W is virtually the half of the total structure weight. The base shear forces obtained by numerical analysis (i.e., effective story shear forces) are consistent with the sum of inertial forces above the first floor. However, as shown in Figure 5b, for the M1-PD model, the effective story shear forces (black solid curve) are no longer equal to the sum of inertial forces above the first story (blue dashed curve). The remarkable difference between the two quantities is just due to P-Delta effects.

Figure 5. Base shear-first story PIDR relationship for both structural models subjected to TCU052.

Furthermore, this involves the assumption that the moment diagram in all elements due to $P \cdot \delta$ is identical to that caused by $V' \cdot h$, where $P$ is the total vertical load tributary to the
frame, \( \delta \) is the story displacement, \( V' \) is an equivalent story shear force, and \( h \) is the height of the story under consideration. Note that the above assumption has approximately minimal effect on the seismic response of frame structures in the inelastic range [27]. \( P \cdot \delta_1 / h_1 \) is added to the sum of inertial forces above the first story (i.e., the blue dashed curve), and then the red dotted curve in Figure 5b is obtained. This red dotted curve is very close to black solid curve (the shear forces). Therefore, it confirms that P-Delta effects can be represented reasonably well by \( P \cdot \delta / h \) so that \( P_i \delta_i / h_i \) can be used to calculate the shear force in the \( i \)th story induced only by P-Delta effects. Note that \( P_1 \) is the sum of gravity load in the stories above the \( i \)th story, and \( \delta_i / h_i \) is the inter-story drift ratio in the \( i \)th story. It is further deduced that PIDR would be closely related to P-Delta effects.

5. Nonlinear Earthquake Behavior

5.1. MIDR Demands

Since the inter-story drift ratio can be used to quantify P-Delta effects, it is necessary to analyze the maximum inter-story drift ratio (MIDR) demands. Figures 6 and 7 show the MIDR demands of the three structures under the 40 near-fault and 10 far-fault ground motions.

![MIDR demands for the 3-, 9- and 20-story structures subjected to the 40 near-fault ground motions.](image)

Figure 6. MIDR demands for the 3-, 9- and 20-story structures subjected to the 40 near-fault ground motions.
Figure 7. MIDR demands for the 3-, 9- and 20-story structures subjected to the ten far-fault ground motions.

The MIDR demands under the near-fault ground motions are much larger than that under the near-fault ground motions and about three or four times as great as that under the near-fault ground motions. The MIDR demands of the 20-story structure are the largest among the three structures subjected to the near-fault ground motions. However, under the far-fault ground motions, the MIDR demands of the 3-story structure are the largest. The MIDR demands of the M1-PD is about 5–8% greater than that of the M2-noPD for all the structures and ground motions. Comparing the variance of the MIDR demands, M1-PD is lightly larger than M2-noPD for the 3-story and 9-story structures. However, the variance of the M1-PD of the 20-story structure is obviously larger than that of the M2-noPD (i.e., about 1.5 times).

Furthermore, Figure 8 shows the relationship between the MIDR demands and the PGA levels of the 40 near-fault and the 10 far-fault ground motions. The MIDR demands increase with the PGA levels. At the same PGA level, the MIDR demands of the M1-PD is larger than that of the M2-noPD for all the ground motions. The PGA levels of the 10 far-fault ground motions are not in the high earthquake intensity. At the similar PGA levels, the MIDR demands of the far-fault ground motions are less than that of the near-fault ground motions.
5.2. PIDR Demands over the Height of Structures

In order to investigate the difference between the near-fault and far-fault ground motions at the same earthquake intensity, three near-fault and three far-fault ground motions are scaled to the same PGA, i.e., 0.2, 0.4 and 0.6 g, and used as the input records for the nonlinear time-history analysis. Figures 9–11 show the peak inter-story drift ratio (PIDR) demands over the height of the three structures with and without P-Delta effects subjected to the six records at the three PGA levels (i.e., PGA = 0.2, 0.4 and 0.6 g). The PIDR demands of the M1-PD and M2-noPD models have a similar distribution along the height of the structures subjected to the same ground motion. In other words, P-Delta effects have not influenced the prediction of the weakest floor, which is defined as the floor where the largest PIDR occurs. Whereas, at the same PGA level, for the 9- and 20-story structures, the location of the weakest floor subjected to the near-fault ground motions is not identical to that subjected to the far-fault ground motions. For example, the weakest floor of the 9-story structure is the eighth or ninth floor subjected to the far-fault ground motions, while under the near-fault ground motions, the weakest floor is the fourth or fifth floor. The weakest floor of the 20-story structure is also the eighteenth or nineteenth floor for the far-fault ground motions, but for the near-fault ground motions, the weakest floor is the second or third floor.
Figure 9. PIDR demands for the 3-story structure subjected to the three near-fault ground motions and the three far-fault ground motions.
Figure 9. PIDR demands for the 3-story structure subjected to the three near-fault ground motions and the three far-fault ground motions.

Figure 10. PIDR demands for the 9-story structure subjected to the three near-fault ground motions and the three far-fault ground motions.

It has been found that the location of the weakest floor is different at different earthquake intensities [21,23]. The near-fault ground motions have stronger earthquake energy and result in a more severe nonlinear response of the structures than the far-fault ground motions. The scaled acceleration, velocity, and displacement response spectra of the three near-fault ground motions are significantly larger than the three far-fault ground motions at around the first vibration mode periods of the 9 and 20-story structures, which are 2.17 and 4.13 s, respectively. In addition, as shown in Figure 2, the spectrum of TCU052 is the largest at the long periods, so that it results exactly in the greatest displacement demands.
Figure 11. PIDR demands for the 20-story structure subjected to the three near-fault ground motions and the three far-fault ground motions.

As shown in Figure 12, to further clarify P-Delta effects, amplification factors are introduced. They are defined as the ratio of the PIDR demands of the M1-PD model to the demands of the M2-noPD model. If the three structures are only subjected to the three far-fault ground motions, the mean amplification factors due to the three records at the same PGA level are almost uniform over the height of the structures even though they are less than one in several stories. The average amplification factors over the height of the structures are nearly less than 1.1 at all PGA levels. Comparatively, for the near-fault ground motions, the amplification factors of the 3-story and 9-story structures are still uniform over the height of the structures and have a similar mean, around 1.1, with the far-fault ground motions. However, the near-fault ground motions affect the 20-story structure more seriously than the other two structures, which can be predicted with the
spectra shown in Figure 2. Particularly, the amplification factors of the 20-story structure represent a more obvious S-shape over the height of the structure. The amplification factor increases rapidly as the PGA level increases. The amplification factors in the lower six stories occurring due to the near-fault ground motions are apparently larger than those occurring due to the far-fault ground motions, especially at the larger PGA level. For example, the amplification factor in the second story of the 20-story structure is 1.08 at the PGA = 0.2 g, but it rises to 1.45 at the PGA = 0.6 g. The greater PIDR demands are attributed to the P-Delta effect and the pulse-type characteristics of the near-fault ground motions, which have possibly triggered structural higher vibration modes.

![Graphs showing amplification factors for 3, 9, and 20-story structures.](image)

**Figure 12.** Amplification factors for 3, 9, and 20-story structures.

5.3. **Quantification of P-Delta Effects over the Height of the Structures**

The term $P_i \cdot \delta_i/h_i$ represents the inter-story shear force, which is only induced by P-Delta effects. Thus, this term can represent the value of P-Delta effects directly. Figure 13 shows the inter-story shear forces over the height of the 3-, 9- and 20-story structures. Under the far-fault ground motions, the shear forces increase linearly as the floors decline because $\delta_i/h_i$ in the upper stories are greater than that in the lower stories. It should also be noted that $P_i$ at the lower stories are larger than the upper stories and $P_i$ values are much more than $\delta_i/h_i$ values. Comparatively, the shear forces under the near-fault ground motions are greater than that under the far-fault ground motions at each story. Note that both $P_i$ and $\delta_i/h_i$ increase more rapidly as the story declines so that the shear forces in the lower stories outdistance the shear forces in the upper stories. The difference between shear forces in lower and upper stories increases with the PGA level. For example, at the PGA = 0.6 g, the mean shear force in the first story subjected to the three near-fault ground motions is greater by about four times in the 3-story structure, three times in the 9-story structure, and ten times in the 20-story structure than that subjected to the three far-fault ground motions. According to the above analysis, P-Delta effects influence the structural response more obviously under the near-fault ground motions even though the structure is very low (i.e., a 3-story structure). Thus, it is clearly seen that P-Delta effects should not be
neglected for low, middle, and high-rise buildings as long as the structure will be subjected to near-fault ground motions, even if the earthquake intensity is not strong.

![Figure 13](image)

Figure 13. Inter-story shear forces over the height of the 3, 9, and 20-story structures.

A specification for the stability coefficient of a structure has been introduced by the NEHRP 2020 code (Equation (1)), and the P-Delta effect should be considered when the stability coefficient exceeds 0.1.

$$\theta = \frac{P_i \Delta_i}{V_i h_i}$$  \hspace{1cm} (1)

where $P_i$ is the total vertical load acting above floor level $i$, $\Delta_i$ is the design story drift occurring when the story shear is equal to the design shear $V_i$, and $h_i$ is the height of story $i$.

When the structures are subjected to the ground motions at large PGA levels, the structures should be in a nonlinear stage. Table 3 shows the stability coefficients of the first floor of the three MRFs under the six records, but the $\Delta_i$ used to calculate $\theta$ is the largest story drift and the $V_i$ is the story shear when the story drift reaches the maximum. When subjected to the far-fault ground motions, only the stability coefficients of the 20-story structure exceed 0.1 and the P-Delta effect should be considered, but there is no need for the 3-story and 9-story structures. However, when subjected to the near-fault ground motions, except for the 20-story structure, the stability coefficients of the 3-story and 9-story structures are also larger than 0.1 at PGA = 0.4 g and 0.6 g. Some of the stability coefficients of the 20-story structure in particular are larger than the upper limit in the code (i.e., 0.25). At these situations, the structure should have collapsed. Comparing the values of the M1-PD and M2-noPD, they are very similar under the far-fault ground motions. However, the stability coefficients of the M1-PD are larger than that of the M2-noPD under the near-fault ground motions. For the 3-story structure under TCU052 at PGA = 0.4 g, the stability coefficients of the M1-PD is 0.10 and the P-Delta effect should be considered. However, the stability coefficients of the M2-noPD is only 0.08 and the P-Delta effect may not be necessary to be considered. Obviously, this is not reasonable. Therefore, even for the lowest structure (i.e., 3-story structure) and the minor earthquake intensity (i.e., PGA = 0.4 g), the
P-Delta effect should be considered as long as the structure would be subjected to near-fault ground motions.

### Table 3. Stability coefficients of the first floor.

| Structure | PGA (g) | Model | Near-Fault | Far-Fault |
|-----------|---------|-------|------------|-----------|
|           |         |       | TCU052     | TCU054    | TCU102    | I-ELC180 | TAF021 | TCU045-W |
| 3-Story   | 0.2     | M1-PD | 0.05       | 0.03      | 0.03      | 0.03      | 0.03    | 0.02     |
|           |         | M2-noPD | 0.04      | 0.03      | 0.03      | 0.03      | 0.02    | 0.02     |
|           | 0.4     | M1-PD | 0.10       | 0.06      | 0.07      | 0.03      | 0.03    | 0.03     |
|           |         | M2-noPD | 0.08      | 0.05      | 0.06      | 0.03      | 0.03    | 0.03     |
|           | 0.6     | M1-PD | 0.14       | 0.11      | 0.15      | 0.05      | 0.05    | 0.03     |
|           |         | M2-noPD | 0.13      | 0.07      | 0.14      | 0.04      | 0.04    | 0.03     |
| 9-Story   | 0.2     | M1-PD | 0.08       | 0.06      | 0.07      | 0.06      | 0.05    | 0.05     |
|           |         | M2-noPD | 0.08      | 0.06      | 0.07      | 0.06      | 0.05    | 0.05     |
|           | 0.4     | M1-PD | 0.12       | 0.08      | 0.08      | 0.06      | 0.06    | 0.06     |
|           |         | M2-noPD | 0.12      | 0.08      | 0.08      | 0.06      | 0.06    | 0.06     |
| 20-Story  | 0.2     | M1-PD | 0.19       | 0.10      | 0.12      | 0.06      | 0.06    | 0.06     |
|           |         | M2-noPD | 0.19      | 0.10      | 0.11      | 0.06      | 0.06    | 0.06     |
|           | 0.4     | M1-PD | 0.22       | 0.15      | 0.14      | 0.10      | 0.10    | 0.10     |
|           |         | M2-noPD | 0.22      | 0.15      | 0.14      | 0.10      | 0.10    | 0.10     |
|           | 0.6     | M1-PD | 0.35       | 0.39      | 0.23      | 0.10      | 0.11    | 0.10     |
|           |         | M2-noPD | 0.35      | 0.31      | 0.17      | 0.10      | 0.11    | 0.10     |
|           |         |         | 0.53      | 0.59      | 0.34      | 0.10      | 0.14    | 0.11     |
|           |         | M2-noPD | 0.48      | 0.49      | 0.33      | 0.10      | 0.13    | 0.10     |

### 5.4. Component Distortion Subjected to a Single Ground Motion Records

Since there is a huge difference between the seismic response subjected to the far-fault ground motions and the near-fault ground motions, the responses of the components, such as panel zone, beams, and columns, should be analyzed in detail [28].

First, the nonlinear dynamic response of the M1-PD and M2-noPD models for the 20-story structure at the PGA = 0.6 g level subjected to TCU045-W, which is a far-fault ground motion record, are investigated as an example. Figure 14 shows the distortion time-history of the springs of the panel zones and the elements of the beams and columns in all stories.

The distortion time-history of the M1-PD has a similar pattern to that of M2-noPD, but has a larger value, especially in the second half of the time course. The panel zone in the eighteenth story of the center column, the beam in the middle span, and the side column in the first story have the largest distortion among all structural components. Moreover, as shown in Figure 1, in the time-history of TCU045-W, these components reach the peak at around 46 s, which is almost the same time as the largest acceleration peak, i.e., 44.79 s. Because of this great acceleration peak, the components are in a nonlinear stage, followed by a large distortion, even though, after 44.79 s, the acceleration time-history of the ground motion goes back to smaller values. These components, i.e., the panel zone in the eighteenth story of the center column, the beam in the middle span, and the side column in the first story, would be the first batch of yield components.
Comparatively, Figure 15 shows the distortion time-history of the components at the PGA = 0.6 g level subjected to TCU052, which is a near-fault ground motion record. Obviously, M1-PD is very different from M2-noPD. Note that at about 40 s, the peak of the distortion time-history of M1-PD is larger than that of M2-noPD. After 40 s, the direction of the distortion time-history of M1-PD is opposite to that of M2-noPD. Almost all distortion peaks of the components occur at about 40 s, which is the time of the largest velocity pulse and the displacement time-history of TCU052, as shown in Figure 1. Because of this large velocity pulse, the components reach yield and generate a large distortion. P-Delta effects then further cause the distortion to increase rapidly, which probably would be more likely to cause the structure to collapse. Even though the acceleration of the record goes back rapidly after 40 s, the substantial deformation of the components still cannot be recovered completely and the deformation still has the same direction as that at the moment of the largest pulse. Therefore, the components of the M1-PD model have the opposite direction distortion to the M2-noPD model. In particular, the results of the columns, shown in
Figure 15e,f indicate that the whole structure would collapse in the opposite direction when the P-Delta effect is considered compared to when the P-Delta effect is not considered.

![Figure 15e](image1) ![Figure 15f](image2)

(a) Torsional moments of Panel zone for M1-PD. (b) Torsional moments of Panel zone for M2-noPD.

![Figure 16a](image3) ![Figure 16b](image4)

(c) Curvature of beam-end elements for M1-PD. (d) Curvature of beam-end elements for M2-noPD.

![Figure 16c](image5) ![Figure 16d](image6)

(e) Curvature of column-end elements for M1-PD. (f) Curvature of column-end elements for M2-noPD.

**Figure 15.** Distortion time-history of structural components of M1-PD and M2-noPD models subjected to far-fault ground motion TCU052 at the PGA = 0.6 g level for the 20-story structure.

From the components’ distortion time-history, shown in Figures 14 and 15, the moment when the peak occurs can determine the yield sequence of the components along the height of the structures. Figure 16 shows the peak distortion of the panel zones and the beams over the height of the 3-, 9- and 20-story structures subjected to TCU045-W and TCU052 at PGA = 0.6 g level. Both M1-PD and M2-noPD models have a similar distribution along the height of the three structures. In other words, P-Delta effects do not change the yield sequence of the components along the height of the structures. P-Delta effects of the three structures also do not obviously increase the distortions under TCU045-W. However, the values of M1-PD are obviously larger than M2-noPD under TCU052. For the 3- and 9-story structures, the values of M1-PD are significantly larger than M2-noPD subjected to TCU052 at all floors. For the 20-story structure, the values of M1-PD subjected to TCU052 are less
than M2-noPD at some middle floors, i.e., the 8th–15th floors, the value at the weakest floor, i.e., the second floor, for M1-PD is nearly twice of the value for M2-noPD. It implies that the P-Delta effects cause worse problems when structures are subjected to near-fault ground motion.

In addition, the yield sequence of the components along the structure height under the near-fault motion is obviously different from that under the far-fault motion, except for 3-story structure. For the 9-story structure, the panel zones at the 8th floor and the beams at the 9th floor yield first, subjected to TCU045-W, but the panel zones at the 5th floor and the beams at the 3rd floor yield first, subjected to TCU052. For the 20-story structure, the panel zones and the beams near the 18th floor yield first, subjected to TCU045-W, but the panel zones at the 5th floor and the beams at the 2nd floor yield first, subjected to TCU052. Since the inter-story shear forces due to earthquakes increase as the floor declines and P-Delta effects influence the lower floors more significantly, as shown in Figure 13,

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**Figure 16.** Peak distortion of the panel zones and the beams over the height of the 3, 9, and 20-story structures subjected to TCU045-W and TCU052 at the PGA = 0.6 g level for the 20-story structure.
the peak curvature of the beams mainly locates on the bottom floors. However, the panel zones are mainly affected by the structural vibration modes. Under the far-fault ground motions, the distortion of the panel zone is mainly influenced by the first vibration mode. However, the pulse characteristic of the near-fault ground motions triggers the higher vibration modes. Comparing Figure 16a,c,e, with Figure 4, the 5th floor and 6th floor reach maximal inter-story drift ratio in the 3rd and 4th vibration modes for the 9-story structure, respectively. Similarly, the distortion of the panel zone at the 6th floor reaches maximum. In the same way, for the 20-story structure, maximal inter-story drift ratios in the 5th vibration modes locate on the 5th floor and 15th floor, where the distortion of the panel zone also reaches maximum.

As shown in Figures 9–11, the PIDR demands of the structures are the combination of the peak torsional moment of the panel zones and the peak curvature of the beams. For example, in the 20-story structure, both torsional moment of the panel zones and the curvature of the beams reach maximum near the upper floors under the far fault ground motions. Thus, the weakest floor is the 18th floor. However, under the near-fault ground motions, the PIDR demands of the structure exhibit an S-shape along the structure height and have the maximum effect in the lower stories. The S-shape implies that P-Delta effects intensify higher vibration modes more significantly. When the P-Delta effect is considered, the values at the floors where the peaks of the S-shape appear are significantly larger than that where the P-Delta effect is ignored. For example, in Figure 7, the PIDR demands reach a peak at or near the fifth floor for the four and five vibration modes. Comparing that with Figure 11a–c, there is a rapid increase at the fifth floor and the PIDR demand of the M1-PD at the weakest floor (i.e., the 2nd floor) is larger than that of the M2-noPD. Then, the displacement demands of the weakest floor(s) and the collapse probability of the structures would increase significantly.

5.5. Relationship between P-Delta Effects and Pulse Periods

According to the above analysis, P-Delta effects closely correlate with inter-story drift ratio, and they are larger at lower stories. Thus, the PIDR demands at the first story can represent the P-Delta effects and they depend on the pulse periods of these near-fault ground motions, as shown in Figure 17. The PIDR demands of the ground floor tend to decrease with the increase of $T_p/T_1$, which can be deduced from the downward sloping trend lines. In other words, the closer $T_p$ and $T_1$ are, the greater the P-Delta effects are. For example, the trend lines of M1-PD between PGA = 0.4 and 0.6 g have a larger distance than M2-noPD. Thus, the P-Delta effects should increase significantly with the earthquake intensity.

![Figure 17](image_url)

**Figure 17.** Relationship between $T_p/T_1$ and PIDR at the first story.

6. Conclusions

MRFs are typical steel-frame structures with excellent seismic performance. They are modeled as two-dimensional frames considering the structural symmetry and the efficiency of finite element methods (FEMs). Since MRFs only carry their own gravity loads and
interior frames, carrying gravity loads has not been modeled in FEMs. Thus, the influence of the global P-Delta effect would not be accounted only by considering the geometric nonlinearity in FEMs. Therefore, an additional leaning column was introduced to carry the gravity loads of the whole structure so that FEMs can reflect P-Delta effects properly.

The present study mainly focuses on the influence of P-Delta effects on the nonlinear earthquake behavior of MRFs. The P-Delta effects caused by near-fault ground motions and far-fault ground motions were compared and analyzed. The 3-, 9- and 20-story MRFs designed for the American SAC Phase II Steel Project were used as case studies. The 40 near-fault ground motions with large velocity pulses and 10 far-fault ground motions were inputted to the nonlinear time-history analysis, and then the three near-fault ground motions and three typical far-fault ground motions were selected from the 50 ground motions and scaled based on three PGA levels. The P-Delta effect was quantified based on PIDR demands. The displacement demands of the whole structures and the distortion of the structural components were compared and analyzed. The main conclusions of this study are briefly summarized below:

1. It was found that the term \( P \cdot \delta / h \) can quantify P-Delta effects. The P-Delta effect subjected to near-fault ground motions is significantly larger than that under the far-fault ground motions at each floor. The P-Delta effect subjected to near-fault ground motions also increases more rapidly as the floor declines. Note that the above two features of the P-Delta effect were observed even for the lowest structure, i.e., the 3-story structure, and even for the weak earthquake intensity, i.e., PGA = 0.4 g. It is concluded that P-Delta effects should be considered for low-rise buildings or at low earthquake intensity as long as the structure would be subjected to near-fault ground motions;

2. P-Delta effects can increase the PIDR demands of the structures. The PIDR demands considering P-Delta effects are about 1.1 times that without considering P-Delta effects for all three structures subjected to both near-fault ground motions and far-fault ground motions. The seismic responses will be significantly different when the high-rise structure (e.g., 20-story structure) is subjected to near-fault and far-fault ground motions. The average PIDR considering P-Delta effects at the weakest floor would be up to 1.45 times that without considering P-Delta effects at the stronger earthquake intensity, i.e., PGA = 0.6 g. Therefore, P-Delta effects would significantly increase collapse probability under near-fault ground motions for longer period structures and at a stronger earthquake intensity;

3. P-Delta effects do not change the location of the weakest floor and the yield sequence of the components. However, the location of the weakest floor and the yield sequence of the components would be different when the structures are subjected to near-fault ground motions or far-fault ground motions. It is mainly because of the velocity pulses of the near-fault ground motions, which induce the larger response spectra than far-fault ground motions. Furthermore, the near-fault ground motions possibly trigger higher structural vibration modes, which were realized from the yield sequence of the panel zones. The difference between near-fault ground and far-fault ground motions would be more obvious for the longer period structures at the stronger earthquake intensity. Additionally, P-Delta effects would change the direction of the components’ distortion even though it would not change the yield sequence. Thus, P-Delta effects would influence the prediction of the structural collapse direction;

4. If the period of the structure is near to the pulse period, P-Delta effects should increase significantly with the earthquake intensity and should not be ignored.

It was clearly shown that P-Delta effects should not be neglected for low, middle, and high-rise buildings as long as the structure will be subjected to near-fault ground motions, even if the earthquake intensity is not strong. It is worth mentioning that this study is only preliminary for the influence of P-Delta effects on MRFs. For future studies, the near-fault ground motions could be grouped based on forward directivity and fling-step effects. Then
the relationship between P-Delta effects and the forward directivity and fling-step effects of the near-fault ground motions will be analyzed in detail.

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