Evaluating Deformation Demands for Mid-rise Steel Office Buildings Using Near-fault Records with and without Pulse Signals

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Abstract

This study investigates the near-fault pulse effects on the deformation demands of steel moment frames. Two mid-rise steel office buildings, which experienced connection fractures in the 1994 Northridge and 1995 Hyogo-ken Nan-bu (Kobe) earthquakes and have detailed damage investigation, were selected for study. The deformation demands of the 5-story and 13-story buildings, such as the maximum roof and interstory drift ratios and beam plastic rotations (i.e. δ_{max}/H, Δ_{max}/h and $\theta_{p,max}$) were evaluated for a total of 145 near-fault records with and without pulse signals. Under pulse excitation, the local deformation demands were found to increase by 14% (for Δ_{max}/h in the 5-story building) to 62% (for $\theta_{p,max}$ in the 13-story building). Despite that, most of the deformation demands were confirmed not to exceed the post-earthquake code requirements (e.g. $\Delta_{max}/h \leqq 4\%$ and $\theta_{p,max} \leqq 3\%$).

Keywords: deformation demands, steel structures, near-fault records, pulse signals

1. INTRODUCTION

This paper presents a quantified evaluation of the near-fault pulse effect on deformation demands of mid-rise steel structures. Two actual steel office buildings, which were respectively struck by the 1994 Northridge and 1995 Hyogo-ken Nan-bu (Kobe) earthquakes and have detailed damage investigation, were selected for study (Section 2). The deformation demands of these buildings, such as the maximum roof and interstory drift ratios and beam plastic rotations, were evaluated for a total of 145 near-fault ground motions with and without pulse signals. In order to achieve a specific verification, two separate evaluations were carried out for the near-fault ground motions with and without pulse signals (Section 3 & 4). Analysis results show that the uncertainty in near-fault ground motions may complicate the global and local deformation demands differently, and the variation can be interpreted as an interaction of structural periods and pulse periods of near-fault ground motions (Section 5).

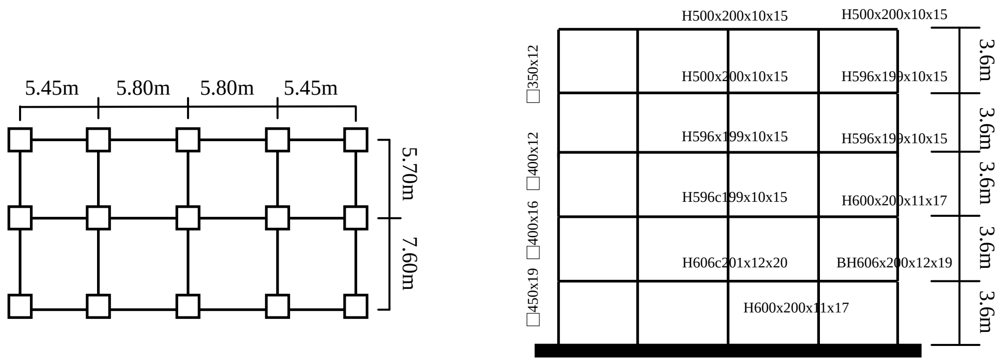
2. MODEL BUILDINGS

The analyzed buildings have been investigated as the representative cases of mid-rise steel office buildings that underwent severe earthquake ground motions in the epicentral areas of the 1994 Northridge and 1995 Kobe earthquakes (SAC 1995; Uang et al 1997; AIJ 1997; Chang et al 2006). The schematic plan, elevation and member size of the buildings is given in Figure 1. The 5-story Kobe-damaged building (designated as JP5) building was designed using the 1973 standard code procedures (AIJ 1997) and was constructed after the 1981 revision of Japan's architecture laws. The 13-story Northridge-damaged building (designated as N13) was designed for standard occupancy with the UBC 1973 and was constructed in 1975.

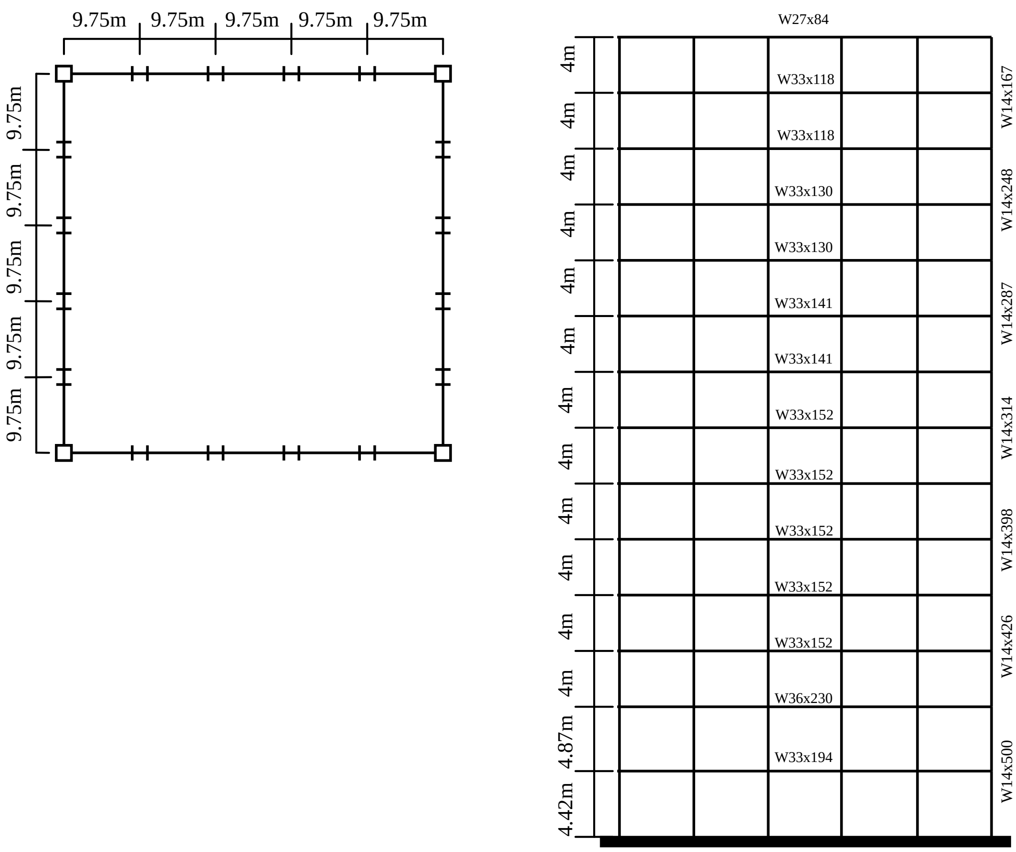
The JP5 building is regularly shaped in the two directions, in which hot-rolled H-section beams are rigidly connected to cold-formed box columns with through diaphragms. After the earthquake, a material test was taken on the damaged beams and it gives the yield stress of beam flange and web, ranging from 263 to 339 MPa. The yield stress of each beam flange and web has been used for estimating the yield strength of the corresponding beam. On the other hand, statistics show that the yielding stress of cold-formed box columns, on average, is 363 MPa. This average yield stress has been adopted to evaluate the yield strength of the columns.

The N13 building is also regularly shaped in the two directions, but has moment frames only at the perimeter. The beam-co-column connections are consistent to the standard qualified type indicated by the code. Beam sections are W27 at roof and W33 at the 2nd

to 12th floors, and W36 at the plaza and 1st floor. Typical column sections are W14x167 at the roof and W14x500 at the basement. Corner columns are built-up box columns. Beams and columns are A36 steel, and the nominal yield stress has been used for evaluation.



(a) Kobe 5-story building (JP5)



(b) Northridge 13-story building (N13)

Figure 1 Schematic plan, frame elevation and member size of model buildings

3. NEAR-FAULT RECORDS

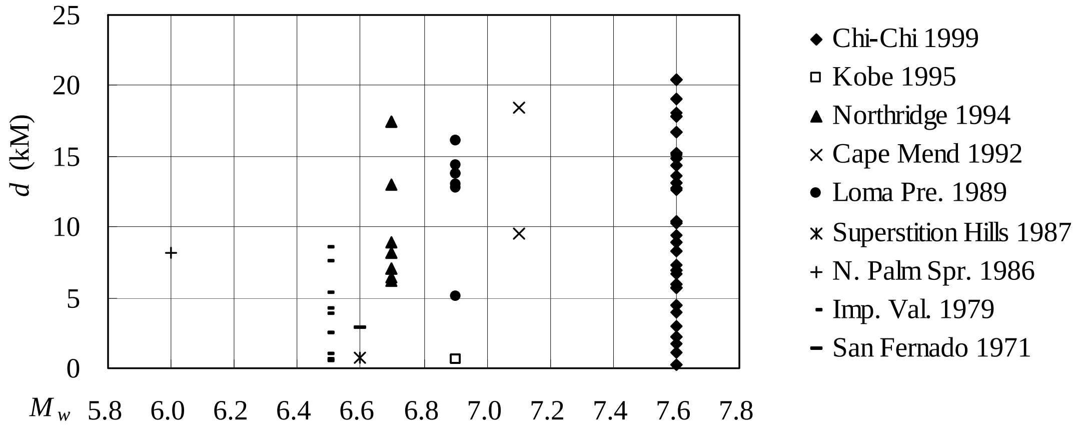


Figure 2 Moment magnitude M_w and closest site-to-fault distance d of the near-fault ground motions used

The connection fractures observed in the 1994 Northridge and 1995 Hyogo-ken Nan-bu (Kobe) earthquakes are thought to highly correlate with the high deformation demands submerged in the near-fault ground motion records in those earthquakes (Somerville et al 1997). Since that, many analytical and experimental studies have been made using near-fault ground motion records, especially those which contain pulse signals in the wave form and appear to be more critical to structures (e.g. Akkar et al 2005).

The deformation demands of the case study buildings were evaluated for a total of 145 near-fault ground motions. The data set has been used to estimate the drifts for frame buildings (Akkar et al 2005). The ground motions of the data set are records from dense-to-firm soil sites, and can be divided into two groups mainly depending on their waveform. 56 of the records exhibit a dominant pulse in their wave form, and the remanding 89 records don’t.

The ground motions are collected from 9 main earthquake events, including the 1994 Northridge, 1995 Kobe and 1999 Chi-Chi earthquakes. The moment magnitude M_w and the closest site-to-fault distance d of the ground motions are depicted in Figure 2. The closest site-to-fault distance d varies from 0.1 to 20 km, and the moment magnitude M_w ranges from 6.0 to 7.6.

Since the source and wave propagation effects for pulse dominant near-fault records are beyond the scope of this study, no detailed discussion will be given here to the complex seismological aspects of near-fault records.

4. DEMAND EVALUATION

The presented evaluation was based on the results of non-linear time history analysis using a general purpose computer program DRIAN2D+ (Tsai and Li, 1994). As mentioned, the model buildings are actual middle-rise steel office buildings, which experienced connection fractures in the earthquakes and have detailed damage investigation. That has permitted to study the correlation between the analyzed frame response and observed connection fractures, as well as verify the effectiveness of the analysis implemented (Chang et al 2006). The connection plastic rotation demands were confirmed to have a clear correlation with the actual spatial distribution of connection fractures. The analysis implemented can therefore be considered to give a reasonably good estimation to the actual deformation demands, even though the analysis doesn't include any connection fracture models.

The following conditions have been assumed in the analysis: (1) reactive weight including uniform partition load but no live load, (2) two-dimensional model with beams and columns modeled by the beam-column element, (3) 2% strain hardening for the beams, (4) axial force-moment interaction for column strength considered in the analysis for the N13 building, but neglected in the analysis for the JP5 building (because low-rise buildings usually don't have a strong axial force-moment interaction in column strength), (5) slab participation as composite beam not considered, (6) fixed-base columns at ground level (and infinity stiff springs to prevent translation at plaza level of the N13 building), (7) P-Δ effects included, and (8) Rayleigh damping of 2% and 5% for the first and second modes, respectively. The natural periods T of the JP5 and N13 buildings are 0.72 sec and 3.05 sec, respectively.

5. NEAR-FAULT PULSE EFFECTS

The object of this study is to make clear the near-fault pulse effects on deformation demands of middle-rise steel structures. The maximum roof drift ratio (i.e. the ratio of the maximum roof displacement δ_{max} to the building height H), the maximum story drift ratio (i.e. the ratio of the maximum story drift Δ_{max} to the story height h) and the maximum connection rotation $\theta_{p,max}$ were evaluated for the JP5 and N13 buildings using the 89 near-fault ground motions without pulse signals, and the 56 with pulse signals.

The evaluated deformation demands are summarized in Table 1. δ_{max}/H is a global measure that relates to structural and non-structural damage of a framing structure. Δ_{max}/h is a local and global deformation measure because it relates to both structural and nonstructural damage over the full height of the structure, as well as assesses the risk of global collapse due to dynamic instability. In contrast, the maximum connection rotation $\theta_{p,max}$ is a local measure relevant for fracture damage.

Comparing the deformation demands in Table 1 enables to know that near-fault pulses may increase local deformation demands to a greater extent. For the JP5 building, the average of δ_{max}/H, Δ_{max}/h and $\theta_{p,max}$ respectively increased by 3%, 14% and 43% under pulse excitation. For the N13 building, the average of δ_{max}/H, Δ_{max}/h and $\theta_{p,max}$ respectively increased by 36%, 33% and 62% under pulse excitation. Both the buildings have shown a similar trend in their deformation demands.

By normalizing the standard deviation by the average in Table 1 also leads to know that near-fault pulses may complicate local and global deformation demands differently. The normalized standard deviation of δ_{max}/H, Δ_{max}/h and $\theta_{p,max}$ are 0.46, 0.58 and 0.78 when the JP5 building are subjected to the 89 near-fault ground motions without pulse signals. The normalized standard deviation of δ_{max}/H, Δ_{max}/h and $\theta_{p,max}$ are 0.60, 0.65 and 0.73 when the JP5 building are subjected to the 56 near-fault ground motions with pulse signals. Out of expectation, the near-fault pulses did complicate the global deformation demands δ_{max}/H and Δ_{max}/h, but didn't changed and even reduced the complexity of the local deformation demands $\theta_{p,max}$. The N13 building also allows exploring this trend.

Table 1 Deformation demands of model buildings

Buildings	Records	Without Pulse δ_{max}/H (%)	Without Pulse Δ_{max}/h (%)	Without Pulse $\theta_{p,max}$ (%)	With Pulse δ_{max}/H (%)	With Pulse Δ_{max}/h (%)	With Pulse $\theta_{p,max}$ (%)
JP5	Maximum	1.69	2.95	2.39	1.94	3.36	2.87
	Average	0.65	0.90	0.68	0.67	1.03	0.97
	Standard deviation	0.30	0.52	0.53	0.40	0.67	0.71
N13	Maximum	1.59	7.35	6.65	3.60	6.67	7.86
	Average	0.83	1.30	0.94	1.13	1.73	1.52
	Standard deviation	0.25	0.87	1.31	0.62	1.12	1.28

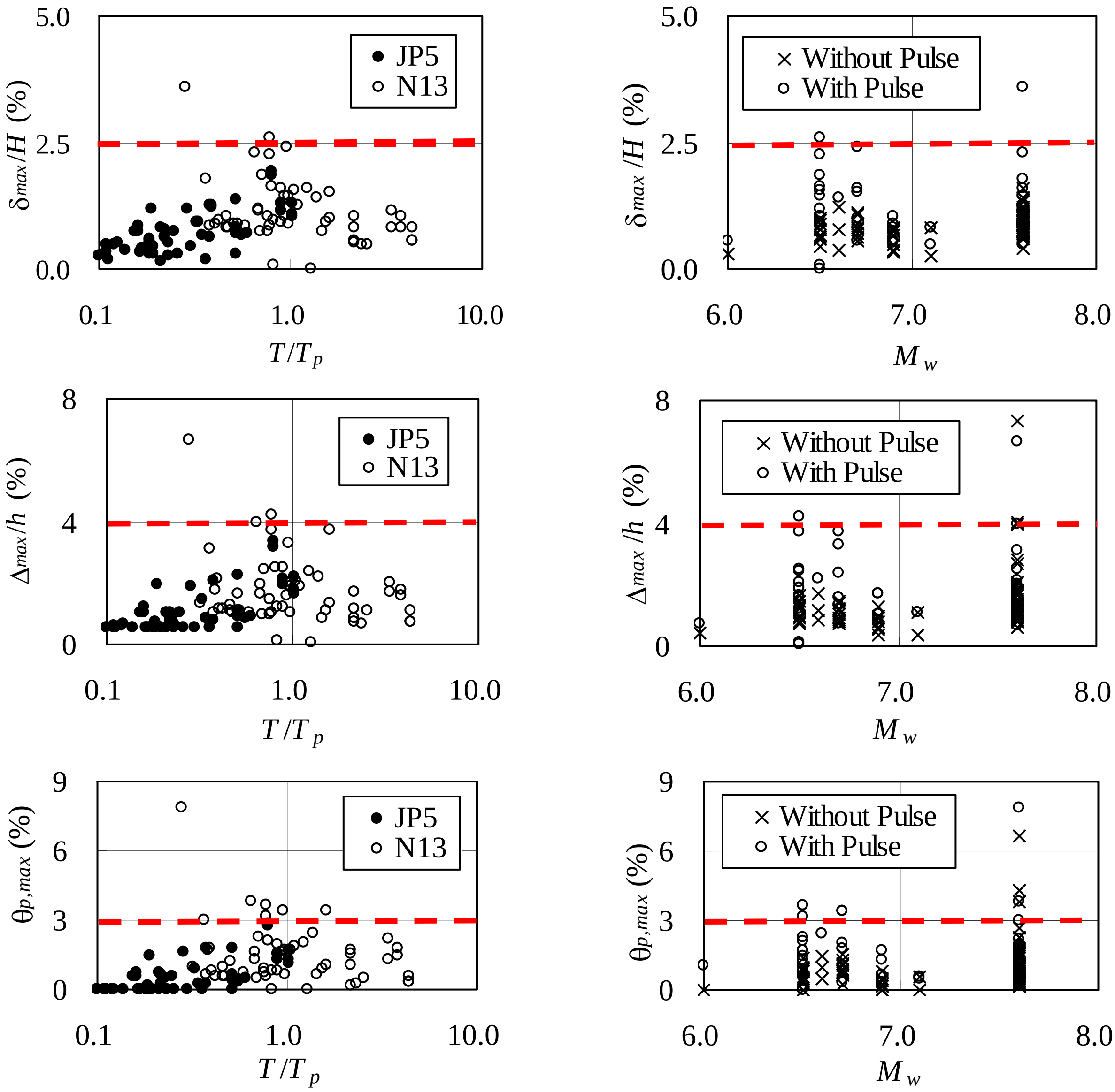


Figure 3 Near-fault pulse effects on deformation demands

The deformation demands for the near-fault pulses and the ratios of pulse periods T_p to structural periods T are depicted on the left side of Figure. 3. A result consistent with previous studies (e.g. Akkar et al 2005) was obtained that the responses drop down gradually as T/T_p moves away from 1.0. Besides, the structural periods T have been shifted to longer ones for the large inelastic deformations induced by the pulses. The deformation demands of the N13 building for the near-fault ground motions with pulses are compared to those without pulses on the right side in Figure. 3. The post-earthquake requirements, for example $\Delta_{max}/h \leqq 4\%$ and $\theta_{p,max} \leqq 3\%$, are also given. As shown by Table 1, Δ_{max}/h increased by 33% and $\theta_{p,max}$ increased by 62% under pulse excitation. Despite that, most of the deformation demands didn't exceed the code requirements.

6. CONCLUSIONS

The deformation demands of two steel middle-rise office buildings were evaluated for a total of 145 near-fault ground motions with and without pulse signals. The local deformation demands were found to increase by 14% (for Δ_{max}/h in JP5) to 62% (for $\theta_{p,max}$ in N13) under pulse excitation. Despite that, most of the demands did not to exceed the post-earthquake code requirements.

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