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RESPONSE OF STRUCTURES TO NEAR-FAULT GROUND MOTION

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SUMMARY

Near-fault earthquake (NFE) records are rich in high frequencies because the short travel distance of the seismic waves would not allow enough time for the high frequency content to be damped out of the record. In addition, in the forward directivity zone NFE records may contain large amplitude velocity pulse of long duration. These characteristics impact the response of both high frequency and long period structures. The objective of this study is to characterize near-fault records by idealizing the velocity pulse component of the record and to investigate their effect on the response of structures in terms of displacements and damage. Four reinforced concrete moment resisting frames: 3, 6, 12 and 20-storeys were designed and subjected to idealized pulses and a set of near-fault earthquake records. The nonlinear static and dynamic responses of the frames were determined using inelastic time history analysis. It was found that the response of structures to near-fault ground motion is substantially different from the response to far-field earthquake records. For the same base shear, the static pushover analysis gives conservative estimates of the displacement of the structure. The nonlinear static pushover approach is particularly suited for displacement-based design of structures to NFE. The response of long period structures to idealized pulses was found to be comparable to the response to NFE records. The results are not as accurate for short period structures.

INTRODUCTION

It is recognized that the characteristics of near-fault earthquake (NFE) ground motions are different from those records in the far-field. The fault normal component is of higher peak ground acceleration than the fault parallel component at the same recording station. In the forward directivity zone, the velocity record is characterized by pulse type motion of long duration. The effect of this pulse type motion on the response is important in the design of structures for near-fault events. In the near-fault region, the short travel distance of the seismic waves does not allow enough time for the high frequency content to be damped out of the record as is normally observed in far field records. Near fault effects were observed in failures during the 1994 Northridge and 1995 Kobe earthquake events.

Available research on the response of structures to near-fault earthquakes is fairly limited. The response of inelastic structures to near-fault ground motion was studied using a shear-beam model [1]. It was concluded that the single-mode analysis provides misleading results for long period structures subjected to pulse-type records. The shear beam model representation of structures has limited applications such as moment resisting frames. Alavi and Krawinkler [2] studied the elastic and inelastic response of frame

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structures subjected to NFE. They showed that for structures with long period T in comparison to the pulse period T_p ($T > T_p$), the distribution of elastic storey shear forces over the height are sensitive to the ratio of natural period of the structure to the pulse duration. Short period structures are not affected as much by the long period velocity pulse. It was shown that the traveling wave effect causes highly non-uniform distribution of ductility demands over the height of the structure. An attempt was made to match the near-fault ground motions to equivalent pulses based on statistical procedure to evaluate the effective peak ground acceleration for each near field earthquake record.

The effect of NFE records on the response of reinforced concrete frames was recently studied by Liao et al. [3]. Five and twelve-storey moment resisting frames were designed according to the Taiwan building code. Four near-fault ground motion records made during the 1999 Chi-Chi (Taiwan) earthquake were used as well as another set of earthquake records at the same sites recorded from past events with distant epicenters representing far-field ground motions. All the records were scaled to the same PGA of 3 m/s^2. The NFE records caused higher storey drift for both 5 and 12-storey frames than that due to far-field ground motion.

NFE may contain large amplitude long period pulses in addition to high frequency content. Available codes and design procedures take the high frequencies into account as ground motion with high frequency content was used in the development of code design spectra. However, structures designed according to current procedures may be vulnerable to the high amplitude long period velocity pulse type ground motion. The limited number of structures and near-fault records used in the published research emphasize the need for a comprehensive study of the effect of NFEs on a wide range of actual structures designed to current codes.

Approximating the time history by a series of sine pulses to represent low frequency content is widely used approximation of near-fault records [4]. This approximation is not based on rigorous analysis. Trying to best fit the velocity pulse generated unrealistic permanent displacement. By adjusting the displacement record, the peak ground velocity (PGA) is reduced.

The objective of this study is to evaluate the response of reinforced concrete moment resisting frame structures of various dynamic characteristics to near-fault ground motion. Four frames designed according to current codes were subjected to selected earthquake time histories recorded by stations located in the near-fault region.

FRAME DESIGN

Four reinforced concrete moment resisting frames of 3, 6, 12 and 20-storeys were designed to current Canadian codes NBCC [5] and CSA [6]. The buildings were assumed to be located in the city of Victoria on Canada's west coast. The four buildings have the same symmetrical floor plan of 3 by 3 bays as shown in Figure 1. Each bay is 6.00 m wide. The storey height is 3.6 m, for total building heights of 10.8, 21.4, 43.2 and 72.0 m for the 3, 6, 12 and 20-storey structures, respectively. The elevation of the 20-storey frame and details of the steel reinforcement are shown in Figure 2. The 3, 6 and 12 storey frames are similar to the top 3, 6 and 12-storeys of the 20-storey frame shown in Figure 2. All interior frames of each building were subjected to the same dead, live, inertia and wind loads. However, they were subjected to different seismic loads due to accidental torsional effect. The design base shears were 14.9, 10.1, 7.8 and 6.4 % of the total weight for the 3, 6, 12 and 20-storey frames, respectively. The frames were designed to ensure that the columns are stronger than the beams. The total column moments is taken ≥1.1 the total moments of beams framing into the beam-column joint. The first three frequencies of free vibration of the four structures are listed in Table 1.

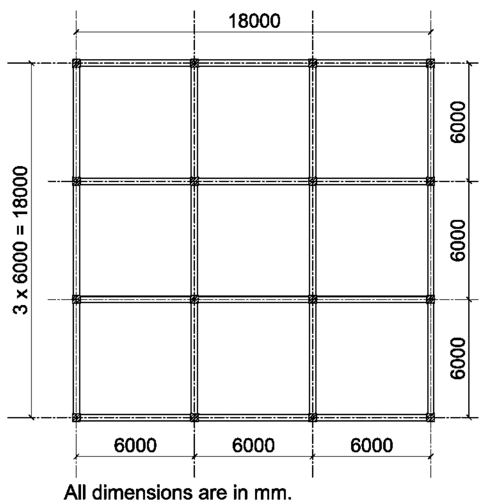

Figure 1 Floor plan of selected Buildings

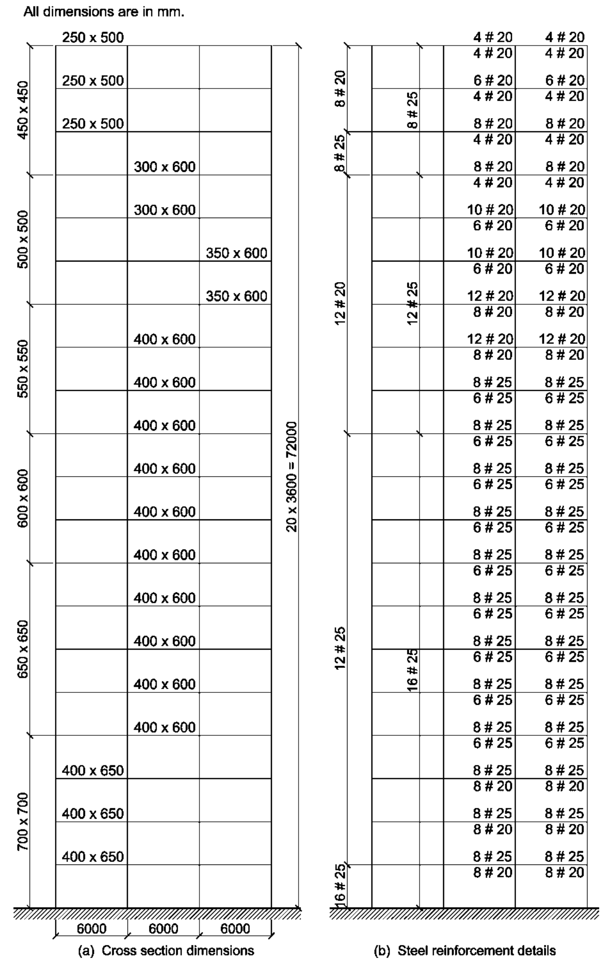

Figure 2 Elevation of the 20-storey frame.

Table 1 Frequencies of free vibration of frames

No. of storeys	Frequencies of free vibrations (Hz)		
	First	Second	Third
3	1.04	4.00	9.09
6	0.68	1.96	4.00
12	0.46	1.11	2.04
20	0.30	0.81	1.35

GROUND MOTION

The designed moment resisting frames are subjected to a set of selected near-fault earthquakes. The selected earthquakes are shown in Table 2. All the earthquakes are larger than magnitude 6 with short epicentral distances of less than 5 km. The peak ground acceleration PGA, peak ground velocity PGV and peak ground displacement PGD of each record are liste in Table 2. For near-fault records in forward directivity zone, the ratio of the PGA (in g) to the PGV (in m/s) was found to be normally low. This is because the ratio is now dominated by the high PGV of the pulse. In this particular case, the high frequency content of the record is not represented by high PGA/PGV ratio.

Table 2 Properties of selected near-fault ground motion records [7]

Earthquake	Date	M_w	Station	Fault Distance km	Comp. Deg.	PGA g	PGV m/s	PGD m
Superstition Hills, US	24 Nov. 87	6.7	5051 Parachute Test Site	0.7	225	0.46	1.12	0.53
Erzincan, Turkey	13 March 92	6.9	95 Erzincan	2.0	NS	0.52	0.84	0.27
Tabas, Iran	16 Sept. 78	7.4	9101 Tabas	3.0	TR	0.85	1.21	0.95
Landers, US	28 June 92	7.3	24 Lucerne	1.1	275	0.73	1.47	2.63

IDENTIFICATION OF PULSE TYPE

The type of pulse in the ground motion is characterized by filtering the high frequencies from the record. In the Fourier amplitude plot, the amplitude spectra are roughly constant over the middle frequency range. At higher frequencies, there is a tendency for the motion to attenuate after f_{max}, defined as the frequency at which attenuation begins. This frequency is used as a cut off frequency in the filtering process of the velocity record. After filtering of the high frequencies from the velocity record, the shape of the pulse can be identified as one full sine cycle or one and half cycles. The one full sine velocity cycle is labeled as P1 while the one and half cycle pulse is labeled P2. After the type of pulse is identified, the pulse amplitude and duration are determined.

The nonlinear dynamic analysis of the frames was conducted with the frames are subjected to the full actual earthquake record. Similar analysis was performed using the velocity pulses as input. The response quantities such as the maximum interstorey drift, the maximum roof drift and the base shear are calculated and compared. As an example of the results, Tables 3 and 4 show the response parameters of the 20-storey and 12-storey frames, respectively, when subjected to two NFE records and their corresponding pulse. The two records are the Superstition Hills which was found to reduce to pulse type P1 of 2 s duration when filtered and the Erzincan record which reduces to pulse type P2 of 3 s duration. It can be seen from Table 3 that the results of the dynamic analysis using the actual earthquake record as input are close to the response results of the frame when subjected to the pulse. For the 12-storey frame shown in Table 4 and, the results are not as accurate. The results for the lower height frames are more divergent.

Table 3 Response of the 20-storey frame to NFE record and the idealized pulse

Parameter	Superstition Hills	Idealized pulse type P1, T_p=2 s.	Erzincan	Idealized pulse type P2, T_p=3 s.
PGV (m/s)	1.12	1.00	0.83	0.85
Maximum interstorey drift (%)	5.18	5.40	3.35	3.40
Roof drift (%)	1.46	1.80	1.13	2.10
Base shear / Weight (%)	13.39	10.7	11.22	10.20

Table 4 Response of the 12-storey frame to NFE record record and the idealized pulse

Parameter	Superstition Hills	Idealized pulse type P1, T_p=2 s.	Erzincan	Idealized pulse type P2, T_p=3 s.
PGV (m/s)	1.12	1.00	0.83	0.85
Maximum interstorey drift (%)	5.60	5.70	3.64	5.00
Roof drift (%)	2.37	2.80	1.80	3.00
Base shear / Weight (%)	10.65	16.00	11.22	16.00

PUSHOVER ANALYSIS

The static pushover analysis using modal SRSS pattern was performed. The results of the analysis in the form base shear variation with maximum roof drift for the 3 and 20-storey frames are shown in Figures 3 and 4, respectively. The static and dynamic analyses were conducted using the CANNY program [8]. The dynamic nonlinear time history analysis for each frame was conducted using the ground motion records scaled by 0.1 g steps. For each case the maximum base shear and the maximum roof drift were determined. This gives one point on the dynamic capacity curves shown in Figures 3 and 4. In the figures, the shown dynamic capacity curves were calculated using the four earthquake records listed in Table 2.

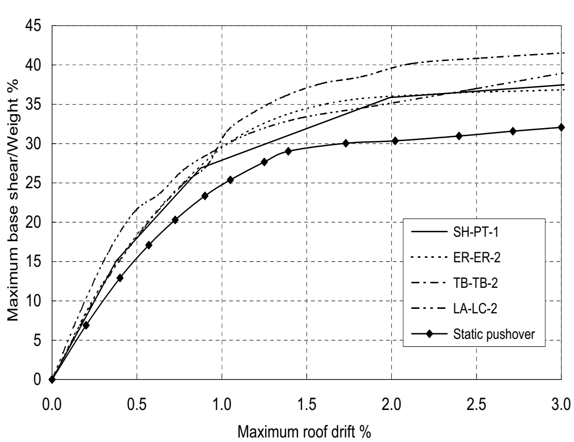

Figure 3 Static and dynamic capacity curves for 20-storey frame

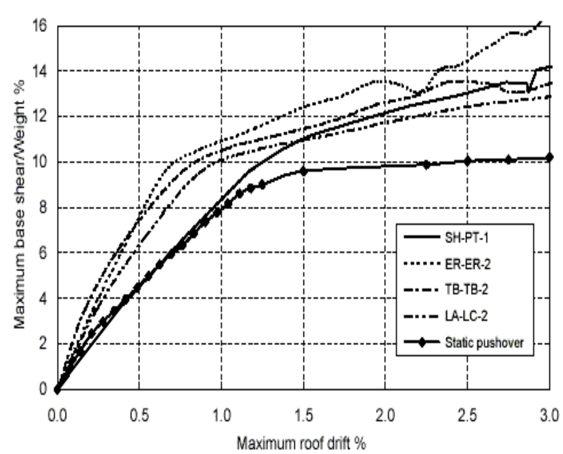

Figure 4 Static and dynamic capacity curves for 3-storey frame

The predicted behaviour of the structure using the static pushover analysis provides a lower bound for the dynamic capacity curve. For high frequency structures, the estimated base shear resistance using the static pushover analysis is approximately 0.8 of the dynamic load capacity. For long period structures the estimated static pushover capacity is approximately 0.67 of the dynamic capacity. In other words, for the same base shear the pushover analysis gives higher drift prediction than the dynamic capacity curve. Since the static capacity curve represents a lower bound for the dynamic curves, it is consistently conservative. It is not practical for the design process to include calculating dynamic capacity curves. Instead, the static pushover analysis can be used. If the frame design is based on the capacity from the static pushover analysis, then during an earthquake the frame will deflect less than the design limit. Capacity curves are particularly suited for displacement-based design approaches.

From the nonlinear dynamic analysis results of the four frames subjected to the selected earthquakes scaled by 0.1 g PGA steps, the base shear is plotted against the peak ground velocity. The results for the 3, 6, 12 and 20 frames are shown in Figures 5 to 8, respectively. There are some differences between the curves because the ground motion scaling was based on the PGA. The same graphs plotted using the peak ground velocity scaling are closer together. The graphs showing the base shear variation with the PGV are shown beside the plots of the base shear against the maximum roof drift. similar graphs can also be plotted in the form of base shear against the maximum interstorey drift. These graphs present a valuable tool in both the strength-based and displacement-based design approaches for NFE. Given a design peak ground velocity, the pair of graphs shown in figures 5 to 8 will give the base shear and the roof drift values to be used in strength-based and displacement-based design of the frames. Since it takes no additional effort to plot the capacity curves in terms of interstorey drift instead of roof drift, the maximum interstorey drift can also be used in performance-based procedures.

DESIGN OF RC FRAME STRUCTURES TO NFE

In current design procedures the ground motion is represented implicitly in the form of the design response spectrum. The development of the design spectra was based on the response of a single degree of freedom oscillator to the then available far-field records, which have stochastic characterizations over a long period. These spectra are not capable of describing the seismic demands imposed on the structure by near-fault pulses. These demands are similar to far-field demands with the added effect of simple high amplitude long period velocity pulse, which characterizes NFE records in the forward directivity zone.

A procedure was described in this study to represent the pulse effects of NFE records on the design by idealized velocity sine pulses. These representations are subject to some constraints. The idealized velocity pulses are identified by their amplitude and duration. These pulses can be applied as input ground motion time-history in a nonlinear dynamic analysis. The analysis will serve as a check on the performance of already designed structures to near-fault effects. If the response is inadequate, the design may be modified. From a practical design perspective, what remains then is to estimate the amplitude and duration of the velocity pulse for a given location.

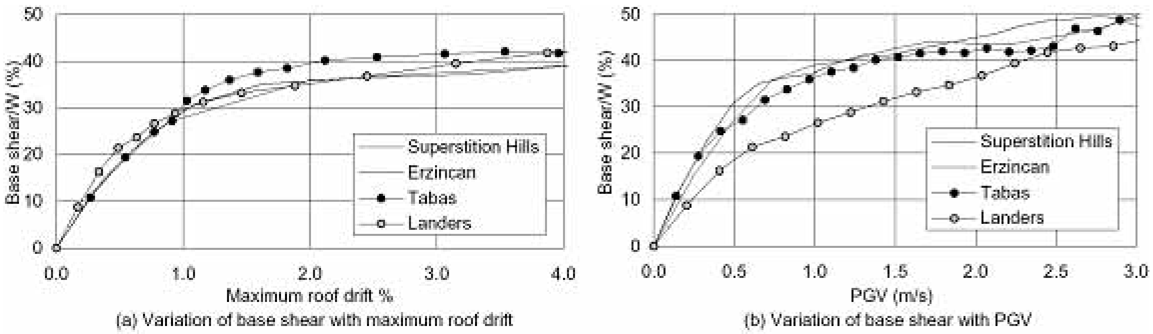

Figure 5 Response of 3-storey frame due to the selected NFE scaled to different PGV

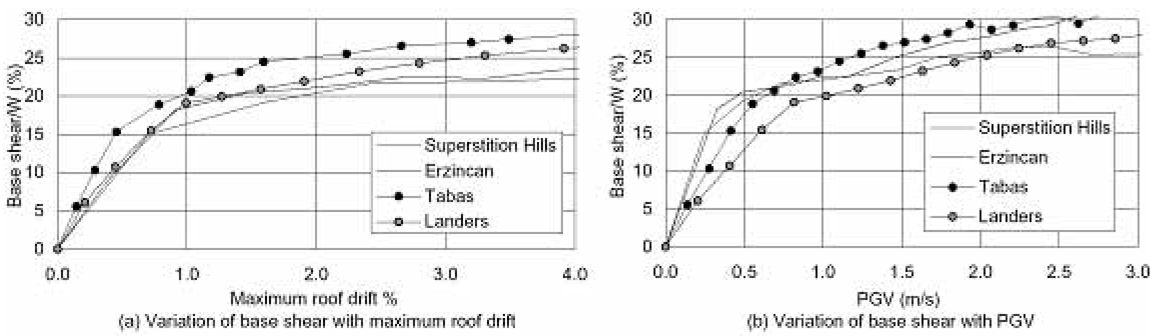

Figure 6 Response of 6-storey frame due to the selected NFE scaled to different PGV

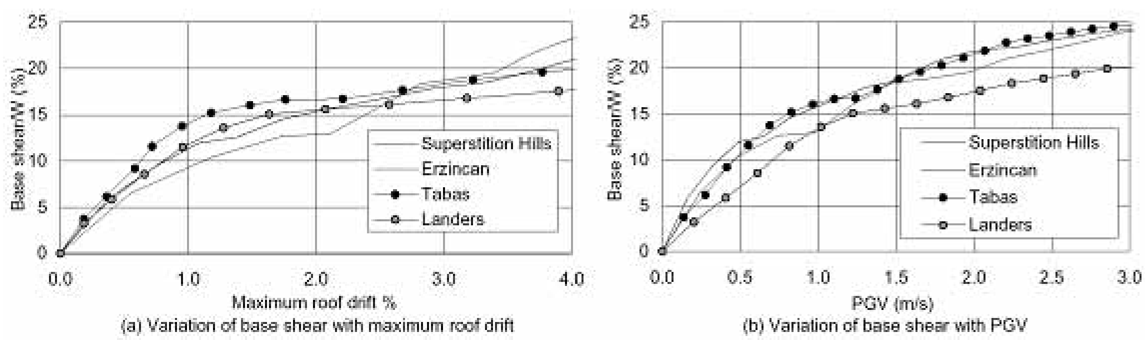

Figure 7 Response of 12-storey frame due to the selected NFE scaled to different PGV

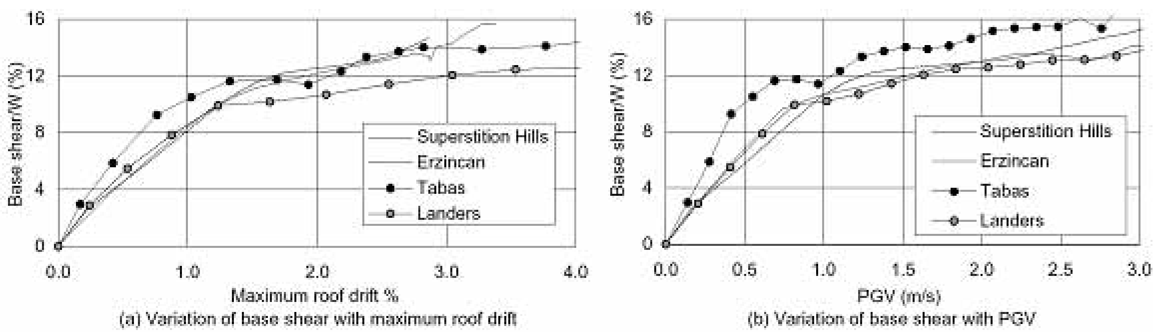

Figure 8 Response of 20-storey frame due to the selected NFE scaled to different PGV

In the near-fault, the velocity pulse amplitude and duration can be related to the magnitude of the earthquake. Several models are available for predicting the time domain parameters of the NFE ground motion pulse. Somerville [9] proposed a model that relates the pulse period T_p to the earthquake moment magnitude M_w. In the model development, 15 time-history records were used to represent records of earthquakes of magnitude in the range of 6.2 to 7.3. An additional 12 records were used to represent distance to fault in the range of 3 to 10 km of earthquake events of magnitudes 6.5 to 7.5. The pulse period is assumed independent of the distance to fault. The model assumes forward rupture directivity and produces a low frequency velocity pulse in the fault-normal direction. The relationship between the period of the pulse T_p of the near-fault, fault-normal forward directivity and the moment magnitude M_w is:

$$\mathrm{Log}_{10} T_P = -3.0 + 0.5 M_w \tag{1}$$

The other parameter of interest is the amplitude of the peak ground velocity of the pulse. The PGV of the near-fault fault-normal forward directivity pulse is related to the moment magnitude M_w of the earthquake by several models. Somerville [9] proposed the following relationship:

$$\mathrm{Log}_{10}\mathrm{PGV} = -1.0 + 0.5 M_w - 0.5 \mathrm{Log}_{10} R \tag{2}$$

where R is the closest distance between the site of interest and the fault.

In the equation derivation records made within 3 km distance from the fault were excluded. The reason is that for small R-values of less than 3 km, the logarithmic terms in equation (2) give unrealistic values for the peak ground velocity.

The pulse parameters (T_p and PGV) obtained using equations (1) and (2) should be treated with caution because the formulas are based on different earthquake events, which have different faulting mechanisms. In addition the number of NFE records used to develop this mode is not particularly large. It is expected that these formulas will be refined with the increase in available NFE records with increased confidence in the prediction of PGV and pulse duration for use in design.

As was demonstrated in this study, the use of the velocity pulse amplitude and duration in the design instead of the full record is applicable with reasonable accuracy to long period structures.

CONCLUSIONS

Near-fault records have high frequency content. In addition, in the positive directivity the records may contain large amplitude velocity pulse of long duration. These characteristics affect the response and design of both high frequency and long period structures. The long period pulse in near-fault records may cause strong fundamental mode response of long period structures. In addition, the high frequency content of the same record may coincide with the second (or higher modes) resulting in severe overall response of the structure. In traditional seismic design procedures, the high frequency content of near-fault records has been accounted for in the development of the seismic design spectra. However, structures designed according to current procedures may be vulnerable to the high amplitude long period velocity pulse type ground motion in the near-fault region.

NFE in the forward directivity zone include a large amplitude velocity pulse of long duration. In near-fault records, the ratio of the PGA (in g) to the PGV (in m/s) was found to be normally low. This is because the ratio is now dominated by the high PGV of the pulse. In this particular case, the high frequency content of the record is not represented by high PGA/PGV ratio. The PGV is a measure, which if combined with the pulse duration can give a complete description of the near-fault pulse type characteristic of the record.

The static pushover analysis underestimates the building capacity to resist dynamic loads due to NFE, which will result in a conservative design. For the same base shear, the static pushover analysis gives conservative estimates of the displacement of the structure. The nonlinear static pushover approach is particularly suited for performance or displacement based design of structures to NFE.

The response of long period structures to idealized pulses was found to be comparable to the response to NFE. However, the results are not as good for intermediate and short period structures.

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