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Effect of SMA on the concrete bridge piers subjected to fling step containing records

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ABSTRACT

Near-fault earthquakes have shown how they impose different demand on structures than the far-fault ones. Due to the pulse-like essence of near-fault ground motions, such ground motions apply a large amount of energy in one big pulse and the structure has to dissipate and withstand this energy. Conventional buildings, through conventional structural design cannot meet such demands and novel material such as SMA are being investigated to resolve these shortcomings. The main objective of this study is to investigate the effects of fling step pulses on the seismic behavior and inelastic response of steel reinforced concrete (RC) bridge piers in the near-fault area by comparing it to far-fault ground motions. Then the same pier will be reinforced with SMA in the plastic hinge area and the behavior under near-fault and far-fault records will be compared to that of steel-reinforced concrete bridge piers. Thus, this paper will investigate the efficacy of using shape memory alloys to reduce seismic responses of reinforced concrete bridge piers when subjected to fling step containing records. To this end, a reinforced concrete bridge pier reinforced with SMA has been considered. The ground motion database consists of near-fault records originally containing forward directivity pulses and fling step pulses. These records have all been scaled to the design target spectrum. Each ground motion is then imposed on bridge pier models using a nonlinear time history analysis. The results show that the drift demand imposed by fling containing records are much higher than the far-fault ones. Also, SMA-reinforced pier resulted in less shear demand, less residual drift, yet an increased maximum drift demand.

Keywords: Near-Fault ground motion, Fling step, SMA-reinforced concrete bridge piers, Nonlinear time history analysis.

INTRODUCTION

Bridge piers built close to seismic sources can undergo large residual displacements due to near-fault ground motions. In the recent near-fault earthquakes, major destructions were observed in concrete structures. Long period pulses such as fling step and forward directivity could be the source of such destructions. Studies conducted by Anderson and Bertero [1], Hall et. al [2], Heaton et al. [3], Bray and Rodriguez-Marek [4], Kalkan and Kunnath [5], Ventura et al. [6] Jamnani et al [7] have shown the destructive effects of fling step on building structures. In the case of bridge structures, Park et al [8] showed that records containing fling step subjected to the superstructure of the Bolu Viaduct caused large displacements that exceeded its capacity. This indicates the insufficiency of code criteria and the need for further studies on the behavior of bridges in the near-fault area and also the need for innovative solutions to overcome the increased demands of near-fault records.

There could be many innovative ways for reducing residual displacement of an structure after undergoing near-fault earthquakes, the ground motion of which contain fling step. One of the methods could be to use super-elastic Shape Memory Alloy (SMA) reinforcement in the plastic hinge region of bridge piers instead of the conventional longitudinal steel reinforcement. Billah and Alam [9] have conducted research on the behavior of SMA-reinforced piers under near-fault ground motions. However, that study does not cover fling step. Current study investigates the efficacy of the use of SMA reinforced concrete bridge piers for mitigating residual displacements. To this end a bridge pier is chosen and validated and near-fault ground motions containing fling step pulse are selected as well to conduct nonlinear time history analyses. To study the effect of fling step, the same ground motion set subjected to base line correction has been used. A far fault record set has been considered as a benchmark. The studied bridge pier with and without SMA reinforcement is subjected to ground motion sets and the seismic responses including shear, maximum drift and residual drift are investigated. The results of this study show the severity of seismic responses when subjected to near-fault records. It is also depicted that using SMA can reduce the shear demand as well as the residual drift experienced by the structure significantly. However, the maximum drift experienced is much higher for a SMA-reinforced bridge pier as opposed to a conventional one.

EXPERIMENTAL STUDY

Test Specimen Details

The one quarter scale bridge pier tested on a shake table by Saiidi and Wang [10] is selected as the test specimen. This pier employs 356 mm NiTi rods with a diameter of 12.7 mm in the plastic hinge length of the column. Figure 1 shows the geometry and reinforcement details of this pier. The measured concrete strength was 43.8 MPa. The yield strength of transverse reinforcement is 469. The mechanical properties of SMA are presented in Table 1.

Table 1. Material properties for steel and SMA [10].

Material	Property	Value
Longitudinal steel	Yield Strength (MPa)	469
Transverse steel	Young’s Modulus (MPa)	199000
Transverse steel	Yield Strength (MPa)	469
	Ultimate strength (MPa)	540
Super elastic SMA	Modulus of elasticity (MPa)	48300
	Austenite to martensite starting stress (MPa)	379
	Austenite to martensite finishing stress (MPa)	405
	Martensite to austenite starting stress (MPa)	180
	Martensite to austenite finishing stress (MPa)	100
	Superelastic plateau strain (%)	5.5

The axial load applied to the column was equal to 624 kN, corresponding to an Axial Load Index (ALI) of 0.25, defined as the ratio of the axial load and the product of the gross column section and the specified concrete compressive strength.

Analytical Model

The nonlinear evaluations were conducted using a two-dimensional model of the pier. SeismoStruct finite-element platform [11] is used to construct the finite element model of the pier and carry on the simulations. A displacement-based nonlinear frame element with the use of a layered fiber section is used for the Finite Element Model of the column. Centerline dimensions were used in the element modeling. For the time-history evaluations, a concentrated mass of 444.8 kN was applied to the top node of the column as inertial mass. SMA is modeled using the SMA uniaxial material in SeismoStruct [11] and the properties of the SMA were converted to get the appropriate properties for this material model.

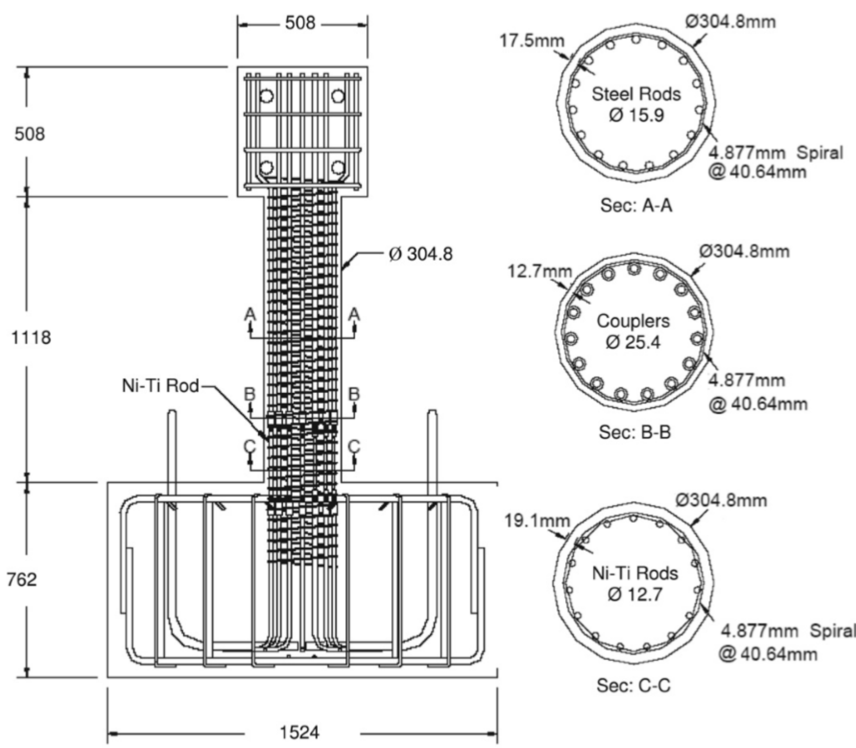

Figure 1. Details of SMA-RC column (Saiidi and Wang, 2006)

Comparison of Experimental and Analytical Results

To validate the model, the column was subjected to a synthetic ground motion compatible to the Applied Technology Council 32 document [12] for medium soil (ATC-32-D) with a Peak Ground Acceleration (PGA) of 0.44g. Validation of the model was based on the 11 runs conducted on the specimens, with the amplitude normalized to 15% for the first run to 300% for the last run of the ATC-32-D record amplitude. Comparison between the measured and the calculated cumulative hysteretic curves of the specimen are depicted in Figure 2. Displacement histories of experimental and numerical results for run11 are presented in Figure 2. In the figure, the red and black lines indicate the analytical and experimental results, respectively. The presented result shows very close match between the experimental and numerical results.

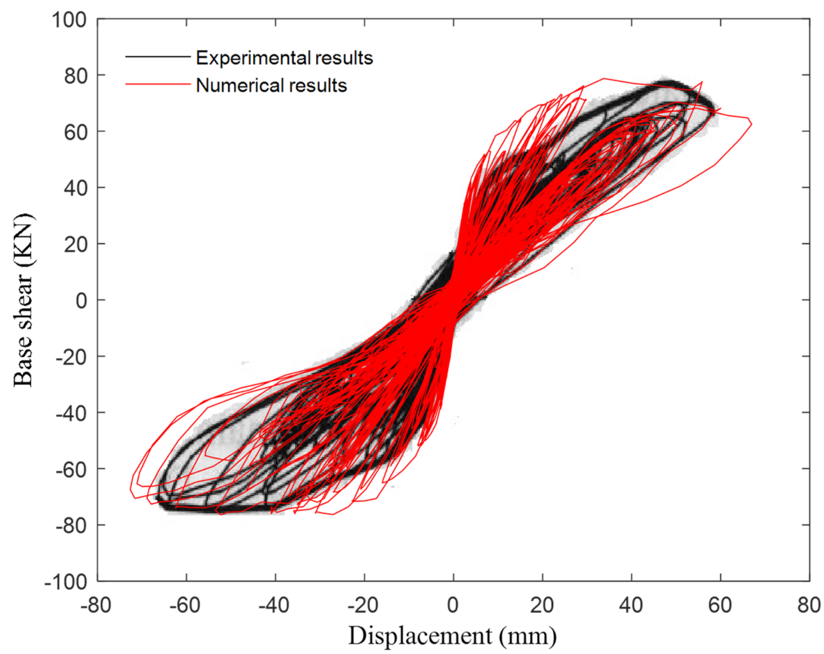

Figure 2. Comparison of the numerical and experimental results.

GROUND MOTION DATABASE

The ground motion database considered for this study includes 7 far fault records, 7 near-fault records containing fling step pulse. The ground motions were chosen so as to cover a reasonable range of frequency content, duration time and amplitude [5]. Table 2 shows information such as event name, station, inclusion of fling step pulse (Yes or No), Peak Ground Acceleration (PGA), Peak Ground Velocity (PGV) and Peak Ground Displacement (PGD).

Table 2. Ground motion data base.

Earthquake	Fling step pulse	Fault distance	Mw	Station	PGA (g)	PGV (cm/s)	PGD (cm)
Northridge	No	23.7	6.7	Century CCC	0.26	21.19	7.85
Kern county	No	36.2	7.5	Taft	0.18	17.50	8.79
Imperial valley	No	-	6.5	Calexico	0.27	42.49	36.12
Northridge	No	-	6.7	Lakewood	0.14	11.17	1.97
Landers	No	-	7.3	Baker	0.11	9.42	5.76
Kern county	No	-	7.5	SantaBarbara Courthouse	0.13	15.48	4.27
Loma prieta	No	67.4	7.0	Presidio	0.10	12.91	4.32
Loma Prieta	Yes	4.5	7.0	Gilroy STA #2	0.36	32.91	7.15
Loma Prieta	Yes	6.3	7.0	Gilroy STA #3	0.37	44.67	19.33
Northridge	Yes	6.1	6.7	Slymar Converter Sta East	0.83	117.50	34.48
Northridge	Yes	6.4	6.7	Sylmar Olive View Hospital	0.84	129.37	31.92
Northridge	Yes	7.1	6.7	Newhall Pico Canyon	0.45	92.76	56.64
Loma Prieta	Yes	5.1	7.0	Corralitos	0.64	55.15	10.82
Cape mendecino	Yes	15.9	7.1	Petrolia, General Store	0.66	90.16	28.99

In order to conduct Nonlinear Time History Analyses (NTHA), the ground motion set presented in Table 2 were scaled in a way that the spectrum of each record matches the 5 percent damped target design spectrum of Vancouver [13] with minimum error in the period range of 0.2T-1.5T, in which, T is the fundamental period of the pier. The period range is [0.22s, 3.74s]. This scaling procedure has been proposed by Alavi and Krawinkler [14] results from the different ground motion sets in a consistent manner.

NONLINEAR TIME HISTORY ANALYSES

In this study, Nonlinear Time History Analysis (NTHA) is used to evaluate the performance of the bridge pier. First analyses were conducted to investigate the effect of fling step containing records on the pier in comparison with that of the far fault records. The mean of the maximum drift, mean of residual drift and mean of maximum shear for the two records sets are depicted in Figure 3. It is shown that in all responses, fling step imposes larger demands on the buildings than the far fault records. This increased demand is highly significant in residual displacement demand, due to the essence of the fling step pulses.

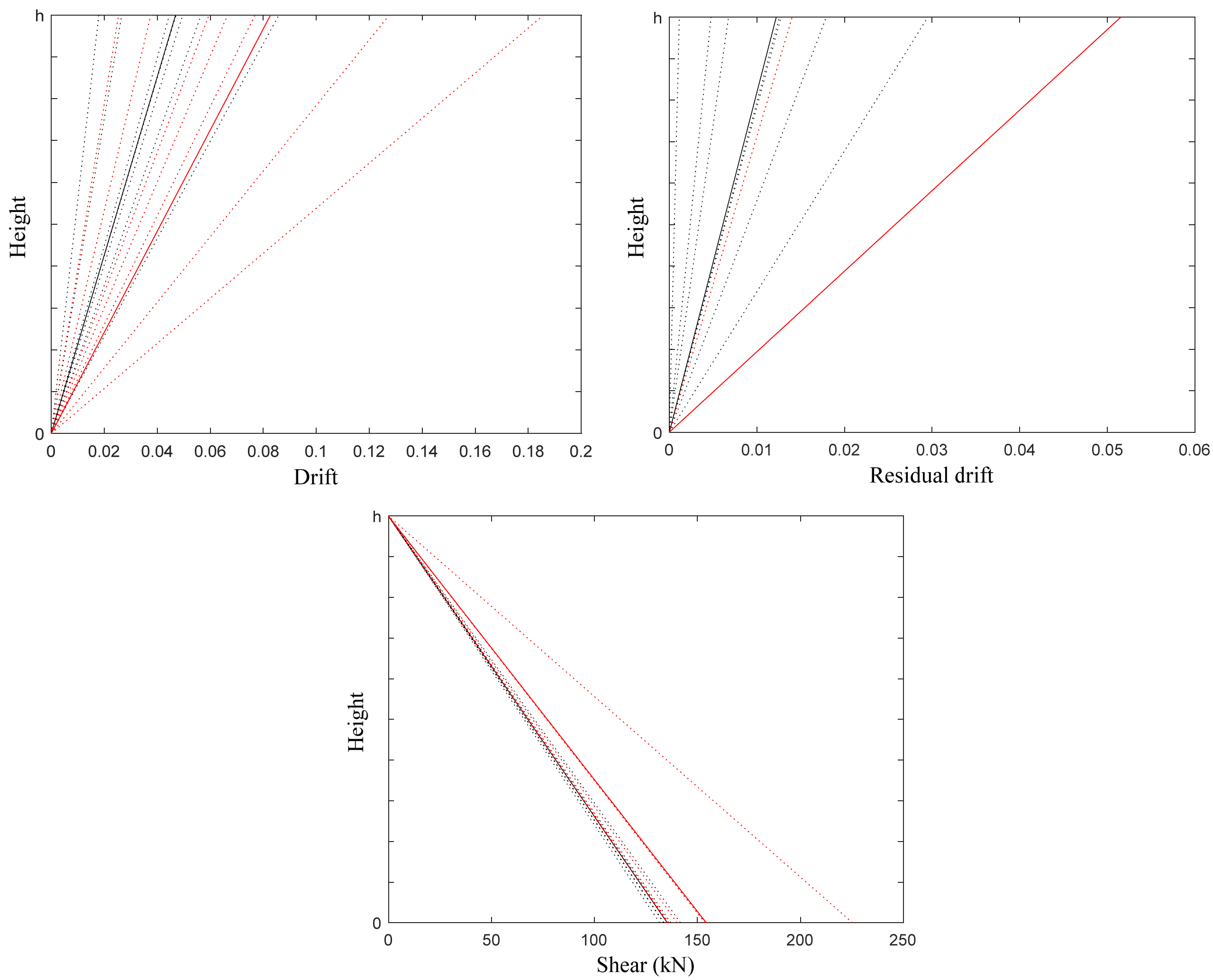

Figure 3. Maximum drift, residual drift and maximum shear of the column subjected to each ground motion (dotty line) and the mean of the responses of the 7 cases (solid line), (Fling step containing: red lines, Far-fault: black lines)

Nonlinear Time History Analysis (NTHA) was also used to evaluate the performance of the bridge pier with and without SMA reinforcement. Figure 4 shows the displacement time history of the top node of the pier subjected to 6 of the fling step containing ground motions. As shown in the figure, in most cases the SMA-reinforced pier experienced larger displacements than the conventional one during the whole earthquake. However, having experienced lower displacements, the conventional pier yields and gives larger residual displacements in the end.

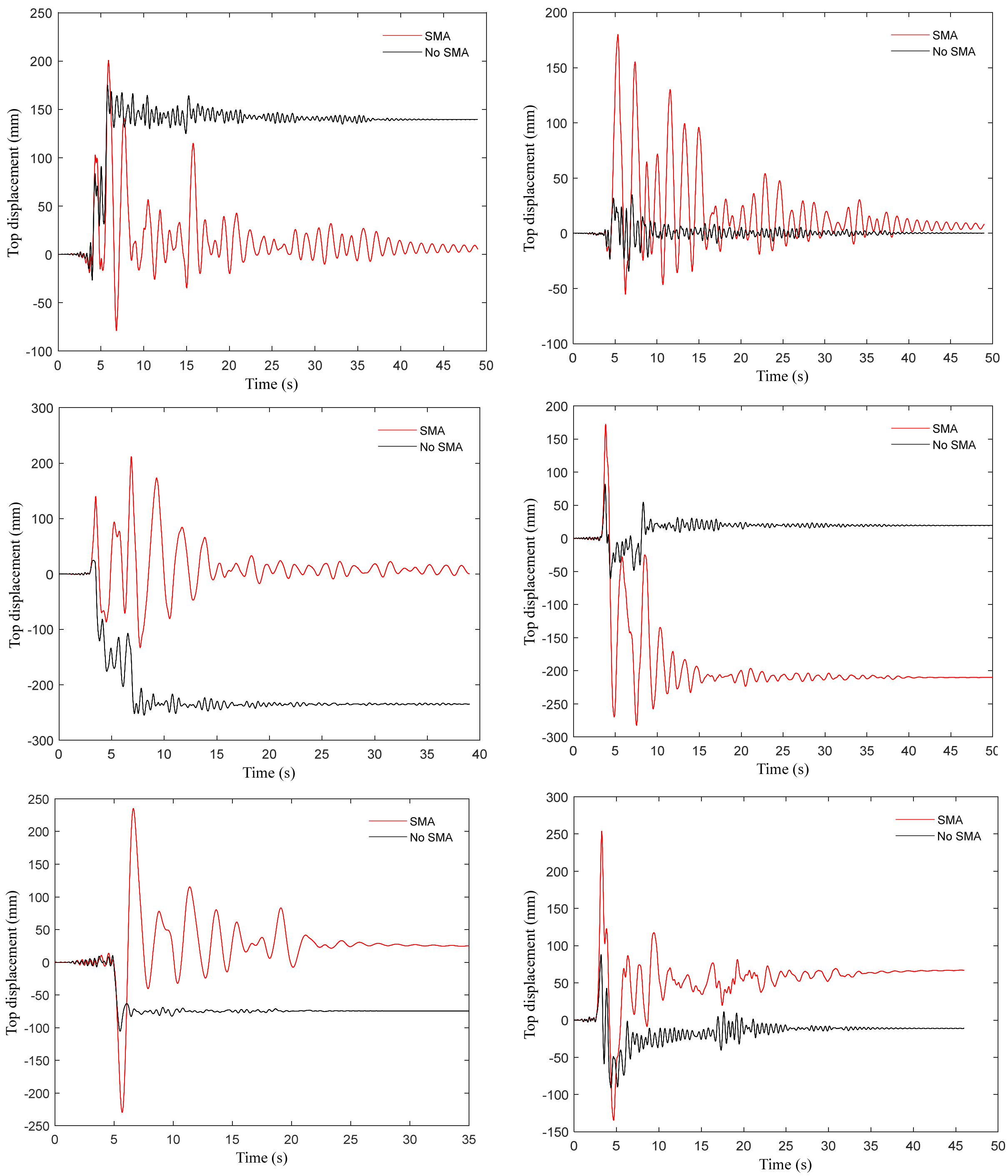

Figure 4. Top displacement vs. time

Maximum drift, residual drift and maximum shear demand are taken as indicators of seismic response of the pier. Piers are subjected to the seven ground motions and the mean of the aforementioned responses are computed and presented. Figure 5 shows the maximum drift, residual drift and maximum shear of the column subjected to each of the 7 fling containing ground motions are depicted in dotty line and the mean of the responses of the 7 ground motions in solid lines. The red lines show the responses for the SMA-reinforced pier and the black lines represent the conventional pier. As shown in figure, the mean of the maximum drift for the SMA-reinforced pier is 225mm; however, the mean for the conventional pier is 113mm. This shows that the use of SMA doubled the maximum displacement. However, comparing the residual displacements, SMA-reinforced pier yields a mean residual drift of 46 mm, whereas the conventional pier results in a 70 mm mean of residual drift. This shows that the SMA decreased the residual displacement by 35%. Results for mean of the maximum shear show that the use of SMA decreased the maximum shear from 154kN for the conventional pier to 99.7kN for the SMA-reinforced pier. This is about 35% decrease in shear demand.

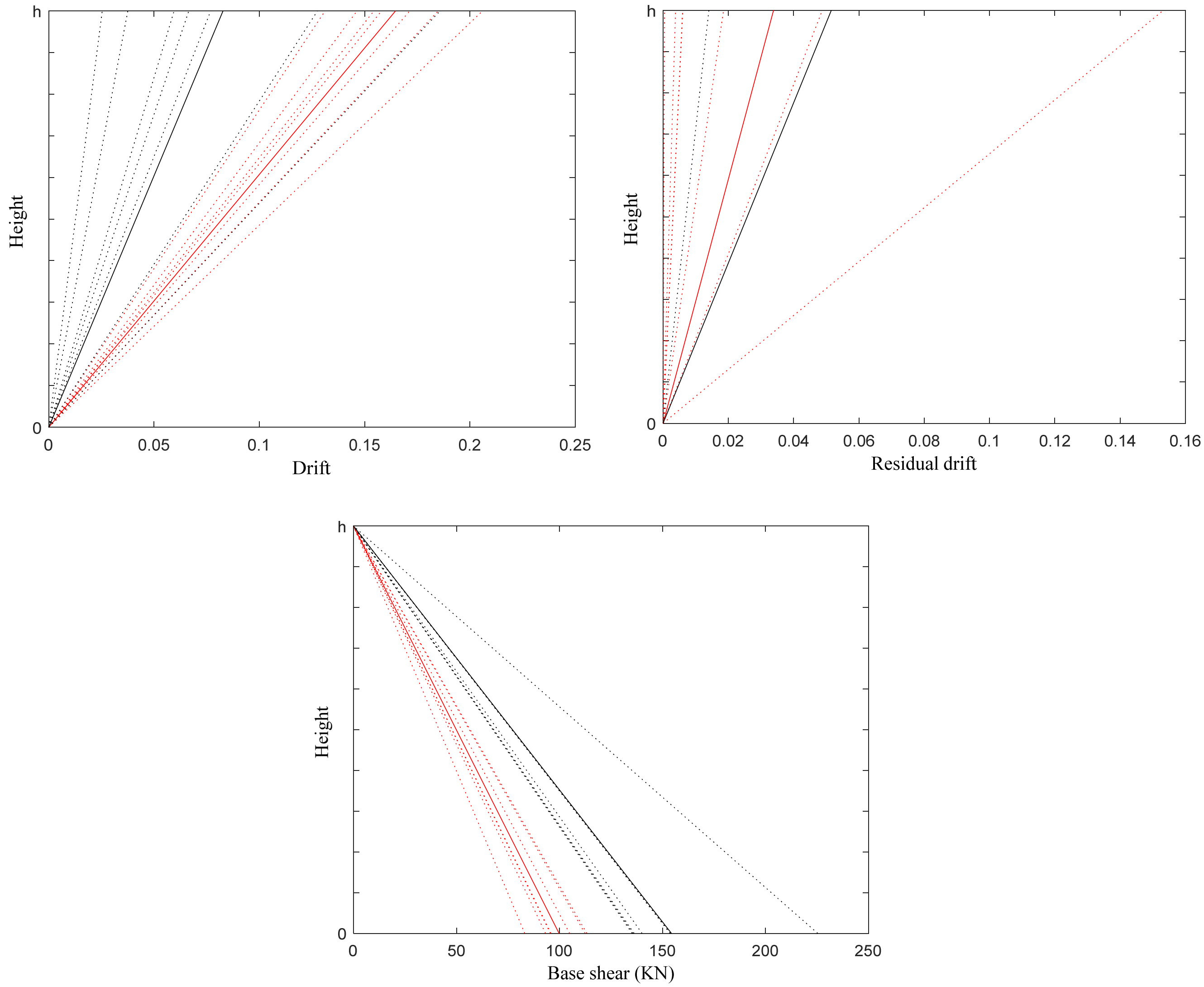

Figure 5. Maximum drift, residual drift and maximum shear of the column subjected to each ground motion (dotty line) and the mean of the responses of the 7 cases (solid line), (SMA: red linnes, No SMA: black lines)

CONCLUSIONS

The results of the nonlinear time history analyses show that:

- There is a meaningful increase in structural demands when subjected to fling step containing near-fault ground motion compared to far-fault ground motions.
- This demand increase is much bolder when it comes to residual drift. This shows the necessity of more studies and the need for using novel technologies to satisfy this demand.
- When studying for the effect of using SMA, the results also show the SMA-reinforced pier results in much larger drifts. However, it reduces the residual drift due to its super-elastic characteristic that makes it resistant to residual strains. Meanwhile, the post-yield strain recovery of steel is fairly small and leads to large residual displacements in piers.
- In cases where the structure can undergo large displacements from an occupancy point of view, it is very much suggested to use SMA-reinforced as it experiences large displacements, yet very low residual displacements.

- The SMA-reinforced pier experiences lower shear forces. This comes from the difference between the yield stresses of steel and SMA which are 469 and 379 respectively.

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