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Computer-Aided Strengthening of Steel and Reinforced Concrete Telecommunication Poles

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ABSTRACT: The absence of explicit guidelines for the rehabilitation of existing pole structures was the motivation to investigate the effectiveness of steel jacketing for the retrofitting of self-supporting steel and reinforced concrete telecommunication poles. The present study describes a numerical simulation of pre- and post-retrofitted conditions of the pole structures subjected to seismic hazards. Effects of superstructure flexibility, variable damping on dynamic response and significance of flexural period on base shear amplification were investigated. The overall effectiveness of retrofitting against base excitation was assessed on two reinforced concrete and two steel poles through the application of nonlinear dynamic analyses, and a response spectrum approach based on a set of strong motion accelerograms recorded during the 1994 (M_S 6.8) Northridge earthquake. Analysis of results shows substantially greater effectiveness of the steel jacketing by decreasing seismic vulnerability for the reinforced concrete poles than that for the steel poles.

1 INTRODUCTION

Uninterrupted service is an essential design criterion for utilities and selected structures subjected to man-made or natural hazards. The 1995 Hyogoken-Nambu (Kobe) earthquake in Japan and the 1999 Chi-Chi earthquake in central Taiwan highlighted the criticality of a rapid restoration of damaged service facilities to recovery efforts and normalization of business and civic life. During these earthquakes, the collapsed or damaged states of several telecommunication towers caused delays in national telecommunication systems throughout the most critical rescue and recovery period immediately following the earthquakes (Schiff 1998, ASCE-TCLEE 1999). The importance of telecommunication towers, because of their rescue and recovery role, requires not simply post-incident survival of these structures should allow only minimal overall damage with no interruption of service.

Unfortunately, self-supporting tower design provisions for earthquake effects and rehabilitation are not yet addressed in current industry standards (TIA/EIA 1996) and building codes. Provisions are acutely needed in recognition of the increasing prevalence of seismic zones in which many more telecommunication towers than previously realized exist. Besides that, seismic retrofitting is further complicated by the additional pressures to place evermore carries onto the poles as communities move to restriction the construction of new poles. Without proper guidance, tower designers may be tempted to directly apply building code procedures for the rehabilitation of exiting poles. However, more precise knowledge on the behavior of the pole structures under seismic excitations is essential. This also necessitates detailed analyses to evaluate the potential of alternative retrofitting solutions. Therefore, the objective of this study is to present a numerical simulation of the superstructure rehabilitation of steel and reinforced concrete telecommunication poles subject to seismic events. For this purpose, the effectiveness of steel jacketing was investigated by using discrete finite element models of poles. The structural behavior of a new composite section consisting of an existing pole, surrounding rubber sheet and steel sleeve was modeled three-dimensionally under different seismic loading conditions for two reinforced concrete poles with heights of 50m and 38m, and two steel poles having heights of 33m and 53m, respectively.

In this study, the impacts of superstructure flexibility, variable damping on dynamic response and significance of flexural period of vibration on base shear amplification were investigated. The overall effectiveness of retrofitting against base excitation was assessed by applying nonlinear dynamic analyses and response spectrum approach based on a set of strong motion accelerograms recorded during the 1994 (M_S 6.8) Northridge earthquake.

2 STEEL JACKET BASED STRENGTHENING

Steel jacketing, as a retrofitting alternative for the existing pole structures, is a combination of a synthetic rubber sheet around the existing structure and surrounding steel sleeve (Fig. 1). The rubber sheet behaves as a gasket and provides uniform friction between inner and outer sections. As such, it transfers bending moments and shear forces induced by seismic or wind loads from the outermost portion of the sleeve to the inner pole along the strengthened height of the structure. The field installation and its placement within the steel sleeve sections are presented in Figure 2. Neoprene and Nitrile are two synthetic rubber materials (Skeist 1999) recommended as an intermediary layer between steel sleeve and existing pole due to their high frictional potential to prevent slippage, and to maximize bounding between inner and outer layers.

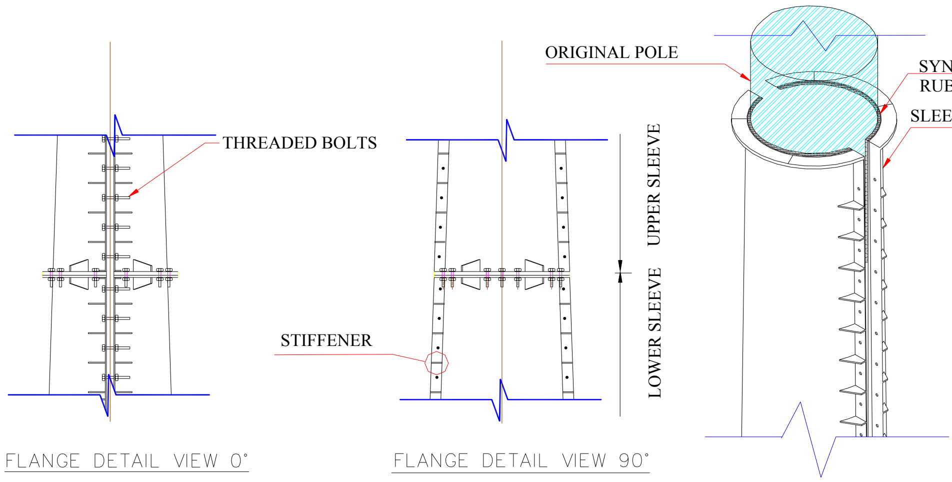


Figure 1. Constructive details of steel jacketing retrofitting

(a) (b)

Figure 2. Field application of steel jacketing (a) Installation of steel sleeve, (b) Close up to rubber sheets placed in sleeve segments

3 STRUCTURAL MODELING

The accurate modeling of long poles is of particular importance because of the large geometric nonlinearities they exhibit. These nonlinear effects are exacerbated as the height of the pole and the number of carries increases and the amplitude of motion intensifies. These geometric nonlinearities can be addressed by using a sufficiently fine mesh during modeling (McClure and Guevara 1994). For that purpose, numerical simulations of the poles were conducted by using discrete finite element modeling to generate detailed stress distributions and deformed shapes of the structures. All simulations were carried out by using a commercially available finite element software (SAP2000 2001), which allowed geometrically nonlinear dynamic and response spectrum analyses.

The four self-supporting telecommunication poles modeled in this study are representative of the real structures used by the industry. Figure 3 shows their pre-retrofitted geometry and Table 1 presents their architectural details. All models were sufficiently detailed based on applied mesh sensitivity analysis to better represent their actual dynamic behavior. The steel sleeve sections and existing poles were modeled by four-noded quadrilateral shell elements which combined separate membrane and plate-bending behaviors. Each shell element consisted of four nodes and six degrees of freedom per node. The rubber sheet was simulated with eight-noded solid elements to investigate its deformability while transferring bending moments and shear forces between inner and outer layers. Since the existing poles were constructed on caisson foundations, they were modeled as sitting on rigid base supports. The typical three-dimensional finite element mesh generation (e.g., for case 3 in Fig.3) is presented in Figure 4.

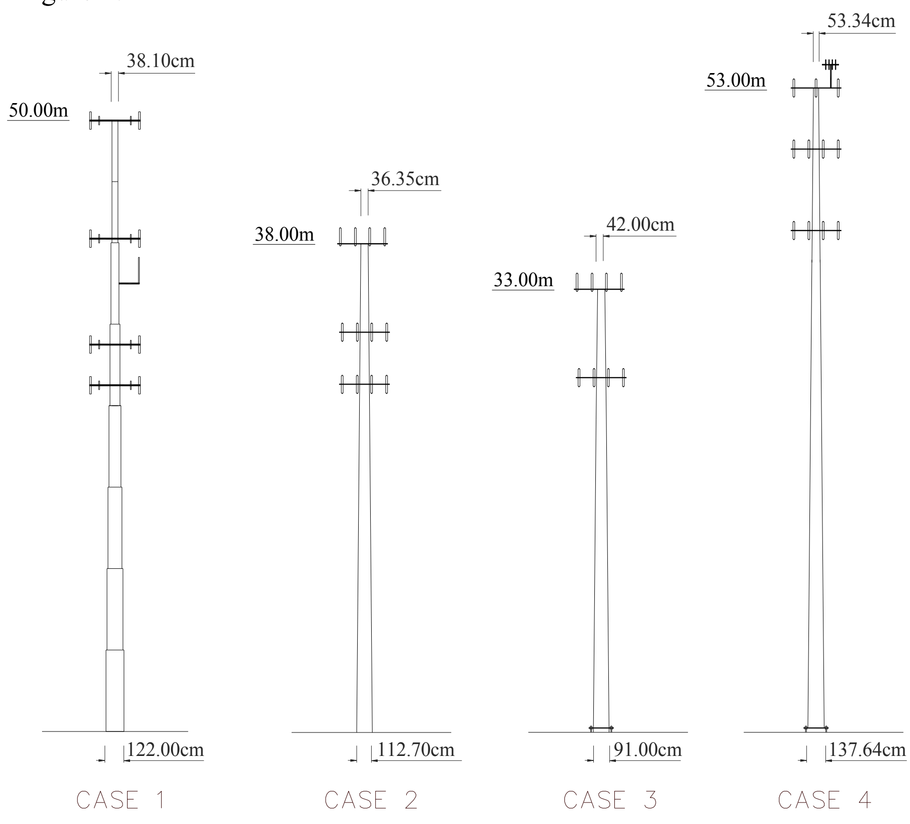


Figure 3. Self-supporting steel and reinforced concrete telecommunication poles

In common practice, steel jacketing can be applied starting from the base level to variable heights of the poles (e.g., 2/3 of pole height) depending on the loading and design considerations. However, to be consistent in all studied cases and to investigate the effectiveness of the retrofitting along the full height of the poles, the steel jacketing was applied to the entirety of their heights. During retrofitting, neoprene thickness of 6.4mm was used for all cases, and the thickness of steel sleeves was taken as 6.4mm for the steel poles and 12.8mm for reinforced concrete poles, respectively. There was no variation in these thicknesses along the height of the poles. The material properties for the reinforced concrete were a modulus of elasticity (E) of 2.48×10^4 MPa and a mass density (M) of 245 kgf-s^2/m^4. Those for steel were E = 2×10^5 MPa and M = 798 kgf-s^2/m^4. Other variables such as the number of platforms, the pole configuration and the geometry were also varied (Table 1).

Table 1. Structural and architectural configuration of existing pole structures

	Reinforced Concrete		Steel	
	Case 1	Case 2	Case 3	Case 4
Pole Height	50 m	38 m	33 m	53 m
Pole Configuration	Tapered & Stepped	Tapered	Tapered	Tapered
Base Diameter	122.0 cm	112.7 cm	91.0 cm	137.64 cm
Top Diameter	38.1 cm	36.35 cm	42.0 cm	53.34 cm
Section Thickness (bottom)	10.16 cm	10.16 cm	0.81 cm	1.27 cm
Section Thickness (top)	10.16 cm	10.16 cm	0.56 cm	0.63 cm
Number of Platforms	4	3	2	3

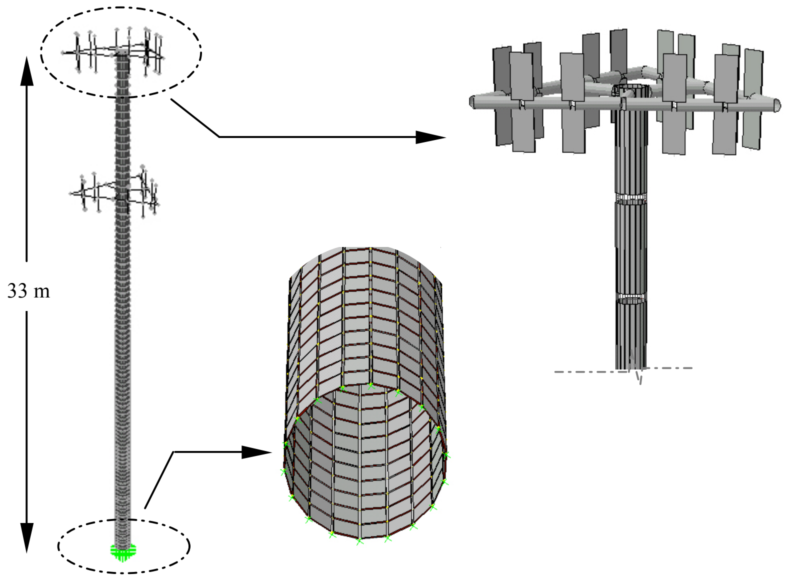


Figure 4. Typical three-dimensional finite element mesh generation (Case 3)

4 NONLINEAR DYNAMIC ANALYSIS

Priori to more complex response spectrum analyses, nonlinear dynamic analyses were performed in order to determine the eigen-values and eigen-modes of each case study. The number of modes used for response calculations is recommended in several seismic codes to include at least 90% of the participating mass for principal horizontal directions. In a similar fashion, 12 modes were used for each dynamic analysis; however, for the purpose of discussion, only the first few dominant modes are presented. The frequencies, periods and effective modal masses of the first three eigen-modes, computed for the pre-and post-retrofitted conditions of each case, are given in Table 2 for comparison. The results indicate that the first three fundamental modes of all models are dominated by lateral translation with coupling effects of principal horizontal directions. Also worth noting is the difference in fundamental periods of the pre-retrofitted and post-retrofitted conditions of the poles. This difference is particularly noticeable for the reinforced concrete poles, where retrofitting resulted in approximately 17 percent reduction in the fundamental period, whereas this difference is relatively negligible for the steel poles. It is also important to observe that the first two eigen-modes in each direction were characterized by extremely low frequencies, in the range of 0.4 to 1.1 Hz. In the following response spectrum analysis, it was, therefore, essential to choose a seismic input with significant frequency content in the low frequency range. Otherwise, the poles would not be excited by a considerable seismic event, and the analysis would lose its significance.

Table 2 Periods, frequencies and effective modal masses of poles

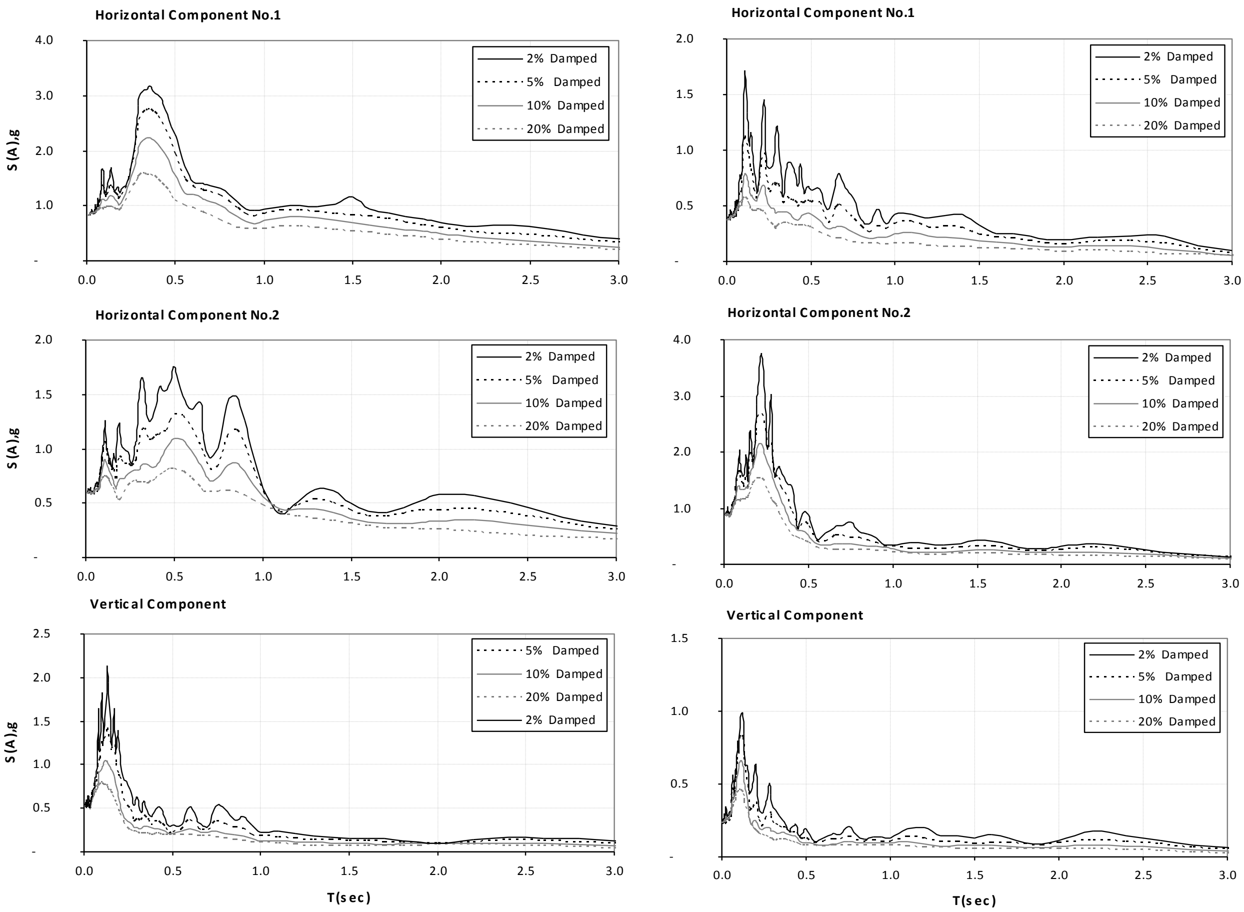
			Pre-Retrofitted				Post-Retrofitted			
		Mode No	Period (sec)	Frequency (Hz.)	Mx/MI (%)	My/MI (%)	Period (sec)	Frequency (Hz.)	Mx/MI (%)	My/MI (%)
Reinforced Concrete	Case 1	1	2.440	0.410	5.79	42.28	2.017	0.496	44.59	3.79
		2	2.440	0.410	42.28	5.79	2.017	0.496	3.58	44.60
		3	0.576	1.737	3.72	17.11	0.476	2.101	16.70	4.11
	Case 2	1	1.445	0.692	3.68	44.45	1.199	0.834	3.21	45.11
		2	1.445	0.692	44.45	3.68	1.199	0.834	45.11	3.21
		3	0.345	2.899	11.19	9.68	0.287	3.488	19.22	1.65
Steel	Case 3	1	0.906	1.103	5.66	47.06	0.906	1.104	30.64	22.20
		2	0.906	1.103	47.06	5.66	0.906	1.104	22.20	30.64
		3	0.198	5.058	16.12	4.57	0.192	5.208	0.70	19.97
	Case 4	1	1.571	0.636	1.03	48.60	1.570	0.637	47.33	2.38
		2	1.571	0.636	48.60	1.03	1.570	0.637	2.38	47.33
		3	0.355	2.814	1.17	19.40	0.355	2.814	6.77	13.80

5 RESPONSE SPECTRUM ANALYSIS

The response spectrum analysis provided more insight into the earthquake response of the poles during their pre- and post-retrofitted conditions. However, one of the main problems in seismic analysis is the selection of a proper input. As no recordings of earthquakes were available for sites with accessible pole structure responses recorded, the corrected strong motion accelerograms recorded during the (M_S 6.8) Northridge earthquake in 1994 (PEER 1994) were adopted to construct 2, 5, 10 and 20 percent damped response spectra for two horizontal and one vertical components recorded in Slymar Country Hospital and Santa Monica City Hall. Although these data were not free-field records, they can be considered to test the effectiveness of the steel jacketing under extreme conditions.

During the analysis, the horizontal components were applied bi-directionally, while a vertical component was applied in the orthogonal direction. The elastic spectra constructed for the two recording sites and their three components with various damping ratios are presented in Figure 5. Since a convenient damping ratio for pole structures was not so certain, different ratios were selected to determine the variation of structural response. With response spectrum analysis, estimates of the total response were calculated as the square-root-of-sum-of-squares (SRSS) of the modal responses. The earthquake responses studied included total base shear, axial forces and overturning moments at the base level, and top displacements. Figure 6 and 7 present the maximum top displacements corresponding to various damping ratios of pre- and post- retrofitted conditions of the poles for two horizontal directions. These figures show that retrofitting resulted in more significant improvements for reinforced concrete poles than for the steel poles in terms of reducing their top displacements. Besides that, the stress redistributions between inner and outer layers in all cases caused a considerable amount of reduction in axial and shear forces and bending moments acting on the existing sections. The retrofitting also decreased the deformability of the poles and consequently lessened undesired secondary moment effects. The difference in pre- and post-retrofitted conditions of the reinforced concrete poles becomes more noticeable as the damping ratio decreases. In general, the average reduction in the top displacement is 23% and 29% for case 1 and 2, respectively (the 2 reinforced concrete poles). On the other hand, contribution of the retrofitting to case 3 and 4 (the 2 steel poles) are almost insignificant. In interpreting these results for the steel poles, recall that retrofitting has almost negligible minor effects on their fundamental periods.

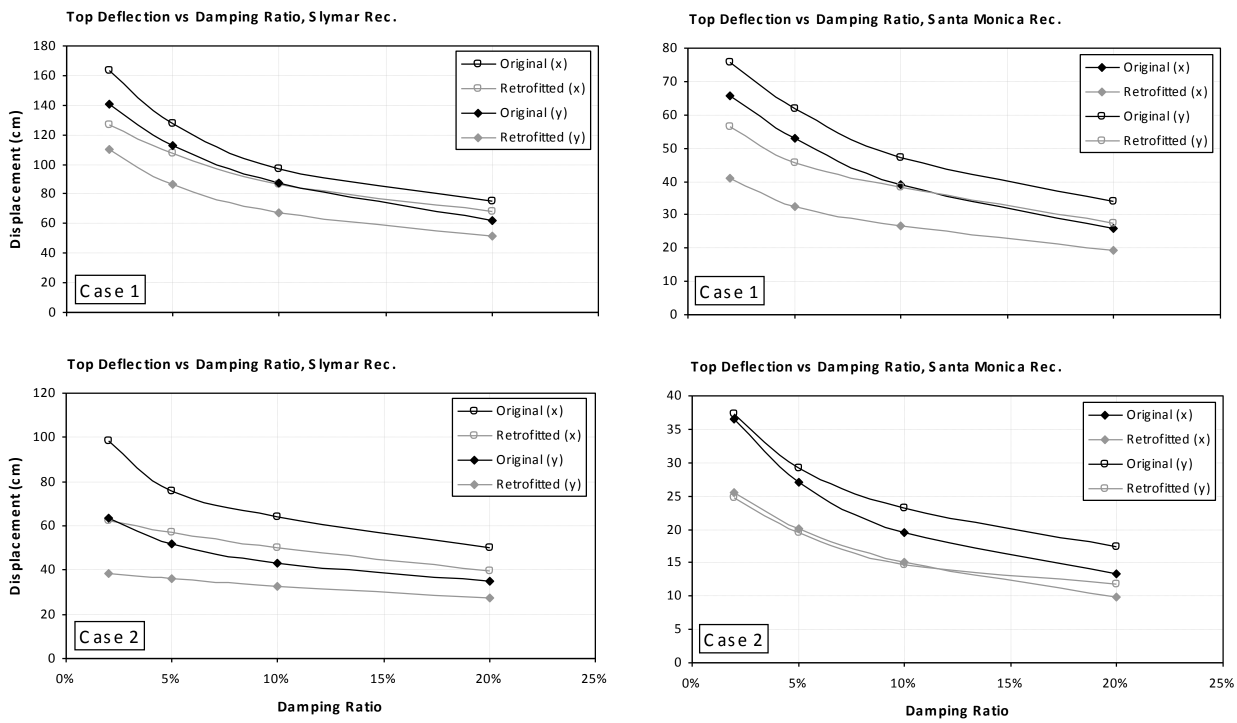
The relative reduction of top deflection according to total base shear may also reflect the effectiveness of the retrofitting. Figure 8 and 9 present the variation of the base shear coefficient (total base shear / total weight) with respect to the top displacement of four case studies. A good agreement between all these curves highlights that retrofitted poles have a potential to carry relatively more base shear by displaying less deformation. The reduction in the maximum bending moment at the base level was



found to be between 36% and 56% after retrofitting.

(a) Slymar Country Hospital (b) Santa Monica City Hall

Figure 5. Elastic spectra for 2, 5, 10 and 20 percent damping ratios



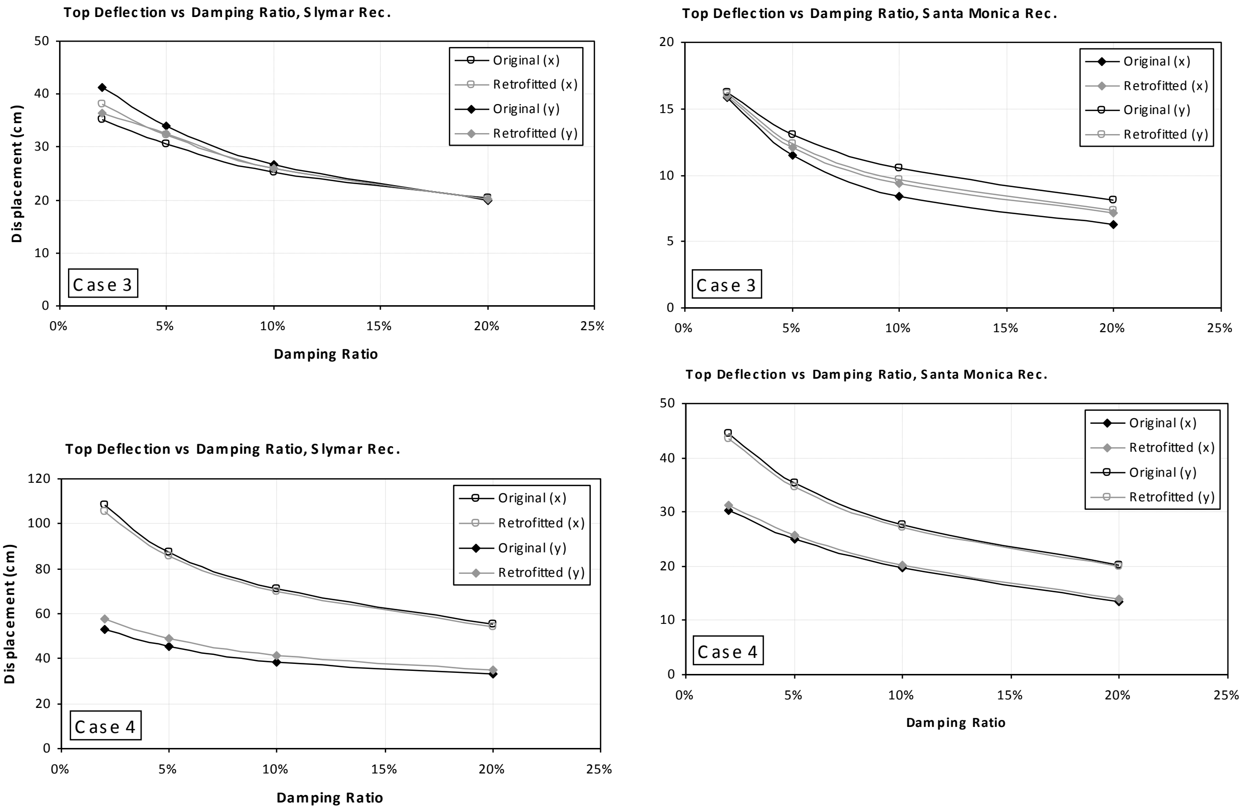
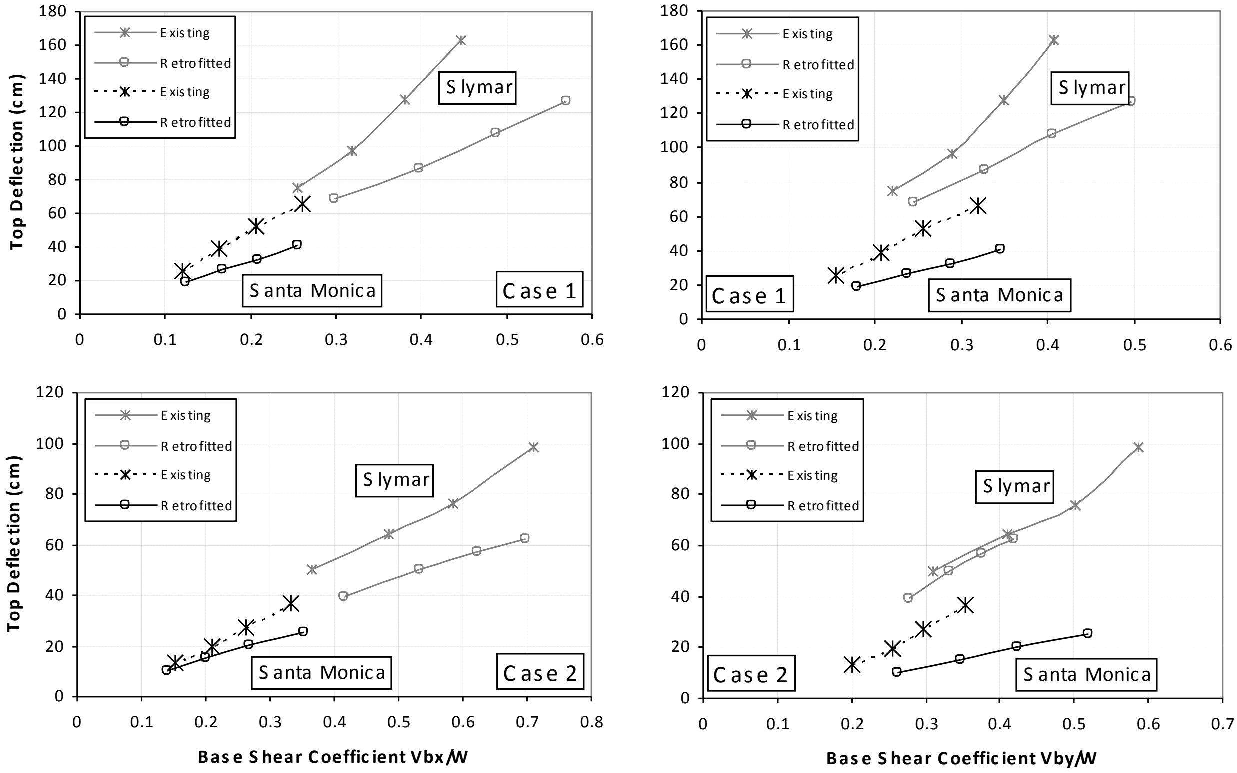


Figure 7. Top displacement and damping ratio distributions for steel poles

Figure 8. Top displacements and base shear coefficient distributions for reinforced concrete poles



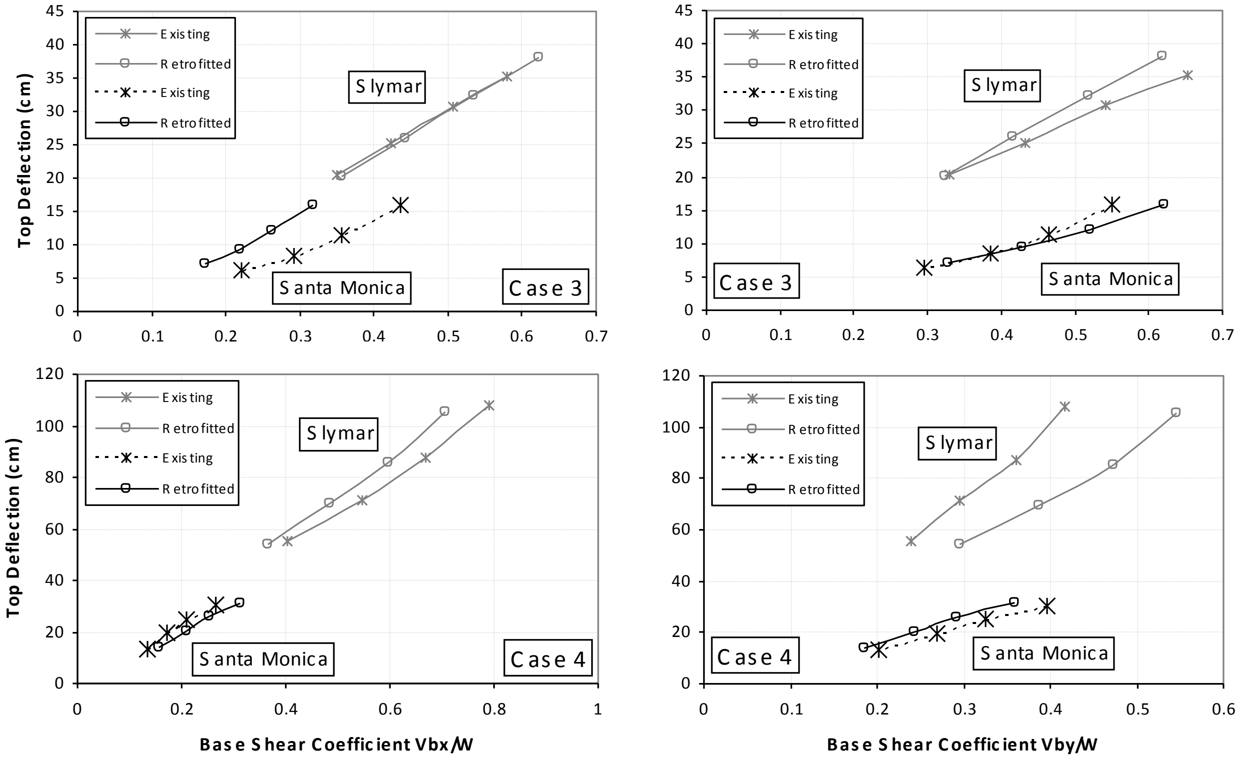


Figure 9. Top displacements and base shear coefficient distributions for steel poles

6 CONCLUSIONS

New seismic and usage requirements have heightened retrofitting needs of self-supporting telecommunication poles. For that purpose, steel jacketing as a retrofitting method was investigated by means of detailed finite element models. Such models included discrete modeling of the pole structures, their platforms and appurtenances allow for the account of the most important phenomena affecting the dynamic response of the poles, such as geometric nonlinearity and interaction between inner and outer layers. The analysis results indicated that the composite section consisting of the steel sleeve and synthetic rubber sheet is effective in reducing the deformability of the pole by increasing its lateral stiffness. A significant contribution of the retrofitting was obtained for reinforced concrete poles. This can be attributed to fact that the retrofitting exhibited more pronounceable effects on their fundamental periods compared to those for steel poles. The steel jacketing provided an optimum utilization of the sections by increasing their shear and bending capacities. The new composite structures became less vulnerable to vibrations as well.

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