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SEISMIC BASED STRENGTHENING OF STEEL AND RC TELECOMMUNICATION POLES BASED ON FEM ANALYSIS

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ABSTRACT: This study describes a numerical projection of pre- and post-retrofitted conditions of telecommunication poles subjected to seismic hazards. The absence of explicit guidelines for the rehabilitation of existing poles motivated the investigation of the effectiveness of steel jacketing for the retrofitting of self-supporting steel and reinforced concrete (RC) communication poles. Effects of mast flexibility, variable damping on dynamic response, and significance of period on base shear amplification were investigated. The overall effectiveness of retrofitting against base excitation was assessed for RC and steel poles through the application of modal analyses and response spectrum approach based on a set of strong motion accelerograms recorded during the 1994 Northridge earthquake. Based on a serviceability approach, the analysis of results shows effectiveness of the steel jacketing in increasing load carrying capacity of the poles by enabling stress redistribution.

Keywords : Poles; Finite elements; Retrofitting; Steel jacketing; Response spectrum analysis;

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1. INTRODUCTION

Uninterrupted service is an essential design criterion for utilities and selected structures subjected to man-made or natural hazards. The 1995 Kobe earthquake in Japan and the 1999 Chi-Chi earthquake in central Taiwan highlighted the criticality of a rapid restoration of damaged telecommunication systems to both the recovery efforts and the normalization of business and civic life [1,2]. During these earthquakes, the damaged states of several telecommunication towers caused delays in national communication systems throughout the most critical rescue and recovery period, namely the hours immediately following the earthquakes. The importance of telecommunication towers, because of their rescue and recovery role, requires not simply post-incident survival but minimal overall damage, with no interruption of service.

In fact, self-supporting tower design provisions for the rehabilitation of existing towers against earthquakes are not yet addressed in current industry standards [3], although simplified procedures for the seismic design of new poles are presented in UBC, 1997 [4] and IBC, 2000 [5]. Additional provisions are needed to address the increasing prevalence and geometric complexity of existing telecommunication towers in seismic zones. As communities move to restrict the construction of new telecommunication poles, there is even more pressure to obtain higher capacity levels from existing poles by including new antennas and reflectors with additional carries. Attempts to increase loads of existing structures have further complicated efforts at seismic retrofitting because of the resulting, supplemental, non-uniform loads. More precise knowledge of pole structure behavior under seismic excitations is essential for proper code development. Detailed analyses are also necessary to evaluate the potential of various retrofitting solutions. For that reason, this paper presents the numerical assessment of a specific retrofitting technique, (i.e., steel jacketing), for the superstructure rehabilitation of steel and RC telecommunication poles prone to seismic events. The extensive application of steel jacketing for seismic retrofitting of building and highway columns and its good performance under seismic excitation provide a strong justification for its selection as a possible treatment for telecommunication poles. Consequently, its applicability for pole structures was assessed via numerical analysis of real cases. The global effects of remediation with a new composite retrofitted section, consisting of an existing pole, surrounded by a rubber sheet and a steel sleeve, were investigated using three-dimensional (3D) finite element models (FEM) under different seismic loading conditions. To this aim, 2 RC poles (38m and 50m high) and 2 steel poles (33m and 53m high) were studied. The overall effectiveness of retrofitting against base excitation was assessed by applying modal analyses and a response spectrum approach using a set of strong motion accelerograms recorded during the 1994 (M_L 6.7) Northridge earthquake.

In the light of the findings generated, the pre- and post-retrofitted condition of steel and RC poles were compared to investigate the flexibility of mast, variable damping on dynamic response, and the significance of period changes on base shear amplification. The geometric properties and capacity of the new composite section were presented and compared to the original RC and steel sections. Based on the results of the response spectrum analyses capacity and demand, variation on the section level were evaluated.

This paper presents the assessment of a specific retrofitting technique for superstructure rehabilitation of existing poles to decrease their seismic vulnerability. This study reflects results of FEM simulations, as currently there are no published case histories or experimental studies against which to compare these numerical projections.

2. STEEL JACKET BASED STRENGTHENING

Steel jacketing, as a retrofitting technique for steel and RC poles results from a combination of a synthetic rubber sheet placed around the existing structure surrounded by half-cylinder shaped steel sleeve segments (Fig. 1). The reason for the use of rubber sheet lies in the fact that it behaves as a gasket, thereby providing uniform friction between inner and outer sections. The rubber sheet transfers bending moments and shear forces induced by seismic or wind loads along the strengthened height of the structure. Neoprene and Nitrile are two synthetic rubber materials [6] that can be recommended as this intermediary layer between steel sleeve and existing pole due to the high frictional potential of the rubber, which helps to prevent slippage and maximize bonding between inner and outer layers. The main characteristic properties of the rubber materials are given in Table 1. The placement of rubber within the steel sleeve sections and the field application of steel-jacketing are presented in Figure 2.

The steel jacket should be mounted to the foundation, either through the use of a combination of steel base plate and anchor bolts (Fig. 3a) or embedded sleeve segments into the new concrete collar (Fig. 3b), even if this requires casting an extension from the existing foundation. Therefore the base shear and overturning moment carried by the sleeve segment can be transferred directly to the foundation. Although superstructure rehabilitation may in some cases entail further retrofitting of the foundation, this paper is strictly limited to superstructure rehabilitation.

3. FINITE ELEMENT MODELING

The accurate modeling of long poles is of particular importance because of the significant P-delta effects they exhibit. These second-order moment effects are exacerbated as the height of the pole and the number of antennas and platforms increase, as and the amplitude of motion intensifies. Summaries of the many techniques that have been proposed to evaluate the second-order behavior are readily available [7]. The application of the geometric stiffness matrix is a general approach to include these effects during the analysis of all types of structural systems [8], and followed in this study, thus an iterative approach was utilized during response spectrum analyses to consider second-order moment effects. All computer simulations were conducted using the commercially available FEM software, SAP2000 [9].

The four self-supporting telecommunication poles modeled in this study are exact models of the real poles used by the industry. Figure 4 illustrates their pre-retrofitted geometry. The poles were modeled in 3D finite element domain. The steel sleeve sections and existing poles were simulated 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 reinforcement within the RC poles was smeared over the section area as an additional equivalent concrete area. The rubber sheet was simulated with eight-noded brick elements to investigate its deformability, while transferring bending moments and shear forces between inner and outer layers. The typical 3D finite element model (for case 3 in Fig. 4) is presented in Figure 5. Originally existing poles were constructed on caisson foundations (a typical configuration is given in Fig 3c). A fixed base support for finite element simulations, however, was assumed, and effects of soil-structure interaction were ignored.

In common practice, steel jacketing is applied starting from the base level to variable heights of the poles depending on the loading and requirements of redesign considerations. To be consistent in all cases herein studied, and to investigate the overall effectiveness of the retrofitting along the full height of the poles, the steel jacketing was applied to the entirety of pole heights. For the retrofitted poles, thickness of 6.4mm was used in all cases for neoprene. For each of the 4 cases, a 6.4mm steel sleeve was utilized. Two additional cases were analyzed for the RC poles using a 12.8mm sleeve to reflect the standard practice.

For modeling, the following material properties were selected: for the RC: a compressive strength (*fc'*) of 40 MPa, a modulus of elasticity (E) of $3x10^4$ MPa, and a mass density (M) of 245 kgf-s^2/m^4. Those for the steel were fy = 345 MPa, E = $2x10^5$ MPa, and M = 798 kgf-s^2/m^4.

Since the serviceability of the poles is their primary design criterion, the material inelasticity was not considered. The general geometric properties of the pre- and post-retrofitted sections are given in Table 2. Since the pole structures have a tapered geometry along their heights, the presented results correspond to the bottom level sections. The retrofitting resulted in a considerable increase in section moduli of the sections, as well as the equivalent cross sectional and shear areas (Table 2).

4. DYNAMIC CHARACTERISTICS OF POLES

Prior to response spectrum analyses, modal analyses were performed in order to determine the elastic modal properties of the poles. Several seismic codes provide guidance on the participating mass and the number of modes to use for response calculations [4, 5]. Base on these 90 percent of the participating mass was used for principal horizontal directions and 12 modes were utilized, with the first three dominant modes presented herein. The periods, and effective modal masses computed for the pre- and post-retrofitted states of the poles are given in Table 3. The results indicate that the first two fundamental modes of the models are dominated by lateral translation with coupling effects of principal horizontal directions, despite their symmetrical configuration. That phenomenon might be attributed to closely spaced modes (i.e., the first two modes have almost the same period). Also worth noting is the difference in fundamental periods of the pre- and post-retrofitted conditions of the poles. The difference is most noticeable for the RC poles, where retrofitting resulted in an 8-13 percent reduction in the period (considering 2 different sleeve thicknesses), whereas, the difference was relatively negligible for the steel poles. This small difference for steel poles may due to the almost same proportional increase in the stiffness and mass of the original steel poles through a doubling of the original thickness (since the elastic vibration period of the system is $T_n=2\pi\sqrt{m/k}$, where m is the mass and k is the global system stiffness). For RC and steel poles, the first 2 modes in each direction were characterized by extremely low frequencies, in the range of 0.4 to 1.1 sec^{-1}. 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 to a considerable extent, and the analysis would lose its significance.

5. RESPONSE SPECTRUM ANALYSIS AND FINDINGS

The response spectrum analysis provided insight into the elastic response of the poles during their pre- and post-retrofitted conditions, yet, one of the main problems in seismic analysis is the selection of a proper input. As no recordings of earthquakes were available for the sites of the poles investigated, the corrected strong motion accelerograms recorded during the 1994 Northridge earthquake were adopted to construct 2, 5, 10 and 20 percent damped response spectrum for 2 horizontal and 1 vertical components of motion recorded in Slymar Country Hospital and Santa Monica City Hall. These records were selected only to test the performance of the poles under extreme seismic excitations irrespective of their site conditions.

In performing the response spectrum analysis, the horizontal components were applied bi-directionally and orthogonally to each other, while a vertical component was applied in the normal direction simultaneously. The seismicity levels considered correspond to peak horizontal accelerations of 0.84g and 0.88g and peak vertical accelerations of 0.54g and 0.23g recorded at Slymar and Santa Monica stations, respectively. The response spectra constructed for their three components with selected damping ratios are given in Figure 6 to show the amplitude of spectral acceleration during pre- and post-retrofitted stage of the poles. Since convenient damping ratios for communication poles are not justified with experimental studies, a range of damping levels were used to determine how the response varies at these levels. The selected ratios are commonly accepted values for standard engineering structures [8], therefore, one may expect that the damping of the pole structures and its corresponding response may lie in the range of the presented response curves.

With the response spectrum analyses, 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; (1) the resultant base shear; (2) horizontal shear force along the mast; (3) overturning moment at the base; (4) dynamic component of axial force along the mast; (5) bending moment along the mast; (6) top lateral displacement; (7) dynamic component of axial displacement along the mast; (8) tilting and rotation along the mast. All of them have significance in the process of analysis and redesign: the first five relate to strength and stability, and the other three relate to serviceability considerations. Self-supporting telecommunication poles must meet strict serviceability criteria that are pre-defined individually by their owners according to particular use of the pole. Seismic amplification of displacement and rotations may affect the top part of the pole, where the antennas are attached, but they should not result in any local permanent deformation after the earthquake. Such deformation may result in a loss of serviceability generating unacceptable signal attenuation [9]. This is the main constraint for the use of elastic analysis techniques in this study.

Table 4 summarizes the top drift ratio calculated based on the ratio of resultant of lateral top displacements to total pole height for different damping levels of pre- and post-retrofitted cases. Concerning the drift values, the applied retrofitting shows more remedial effects on RC poles than steel poles. The retrofitting particularly reduces the deformability of masts for RC poles, thus causing a diminishing in second-order moments. The difference in pre- and post-retrofitted states of the RC poles becomes more noticeable as the damping ratio decreases. That is due to fact that as the damping ratio increases, the structure becomes less sensitive to damping level. The pre-retrofitted maximum drift was 3.0 and 2.1 percent (at 5% damping level) for cases 1 and 2, respectively. Maximum drift decreased to 2.8 and 1.8 percent for the thin sleeve retrofit, and further decreased to 2.6 and 1.6 percent for the thick sleeve retrofit for case 1 and 2, respectively. In contrast to the RC poles, the maximum drift for the steel poles was 1.9 and 1.4 percent for cases 3 and 4 at 5% damping level for their existing and retrofitted conditions and did not show significant differences.

The results of response spectrum analyses demonstrated that masts of the poles are relatively flexible for all cases. The maximum flexural rotation (tilting) of the top of the mast was below 2.2° for pre-retrofitted cases and below 1.8° for post-retrofitted cases of RC poles (considering each damping level). For steel poles, maximum tilting was below 1.3° for both their pre- and post-retrofitted conditions. The maximum flexural rotation is an important parameter for the serviceability of most reflector antennas, as they must retain their horizontal and vertical position because full functionality depends on the specific tolerances of the equipment carried by the poles.

Another measure of the effectiveness of the applied retrofitting is the relative reduction of maximum drift (i.e., ratio of resultant top displacements to pole height) to total carried base shear. Figures 7 and 8 present the variation of base shear coefficient (i.e., ratio of total base shear (V) to weight (W) of the pole) for the maximum drift of four cases at different damping levels. Notably, the smallest damping ratio presents the higher drift. These figures show that RC retrofitted poles have the potential to carry relatively more base shear, with less top deflection.

No significant changes were observed for steel poles between their pre-retrofitted and retrofitted states.

Retrofitted poles displayed more stable behavior despite the increase in the seismic forces due to the increase in spectral acceleration conveyed by the reduction in their fundamental period. The maximum bending moment at the lower sections was decreased by 36 to 56 percent after retrofitting, due to stress redistribution between the inner and outer sections. The general conclusions presented herein were also supported via demand and capacity evaluation on pre- and post-retrofitted sections using the results of the 5 percent damped Slymar record response spectrum analysis. Demand calculations were carried out considering the axial load and biaxial moments simultaneously, and demand (i.e., resultant stress) was computed for existing sections and sleeve sections separately. The capacity evaluation is based on the elastic material properties. The resultant elastic capacity and demand ratio (*C/D*) for pre- and post-retrofitted conditions of the poles are presented in Table 5. The results correspond to the bottom section of the poles, where the stress concentrations reach maximum. The representative stress variation on the pre- and post-retrofitted sections are shown in Figure 9 for case 2 (Fig. 9a-b) and case 3 (Fig. 9c-d). The results obtained imply that retrofitting increases overall section capacity and helps to relieve the stress on the existing section by transferring the stress to the sleeve section. For the RC poles, pre-retrofitted sections near their elastic limits, where *C/D* is equal to 1.2 and 1.0 for case 1 and 2, respectively. On the other hand, retrofitting results in significant reduction on the stress acting on the existing section and increases *C/D* to 1.8 and 2.4 for case 1 and 2. After retrofitting of RC poles, sleeve sections still have high *C/D* that imply that the poles can carry more loads without yielding of the sections and losing their stability. The steel poles show higher *C/D* compared to RC poles in their pre-retrofitted conditions. After retrofitting, *C/D* values for steel poles double, which enable significant load carrying capacity for steel poles in the elastic range.

6. CONCLUSIONS

Self-supporting telecommunication poles are a vital component of post-disaster communication system and management needs to assess the reliability and safety of these poles to minimize the risk of distribution to service during a severe earthquake. Unfortunately, current tower design standards used by the industry (e.g., [3]) do not include necessary guidelines for the retrofitting of existing poles. In this paper the applicability of a steel jacketing, currently widely used for retrofitting of bridge piers and buildings columns, is described and its effectiveness is numerically assessed for existing steel and RC poles. The findings indicate that the composite section, consisting of a steel sleeve and synthetic rubber is effective to provide stress redistribution between the existing section and steel sleeve, and the retrofitting showed a potential to reduce top deformations particularly on RC poles. In fact, reduction in deformability is a convenient way to minimize the undesired second-order moment effects and to ensure continuous service of the telecommunication poles.

A contribution of the retrofitting to reduce the base level stresses was obtained for both steel and RC poles, which leads to an optimum utilization of the existing sections by increasing their shear and bending capacities. The applied retrofitting exhibited more pronounced effects on the fundamental periods of the RC poles, in contrast to those of the steel poles. The doubling of steel sections in the sense of equal thickness and material properties seems to be responsible for the insensitivity of rehabilitation on the period of the steel poles. While the effects of retrofitting to decrease the deformations and to decrease the maximum stress on the sections are valid for static systems, they are not always warranted for dynamic system depending upon the response characteristics of the input motion considered. As such, increasing stiffness of a structure subjected to earthquake loading may also result in an increase in maximum stress (i.e., change in period may results in higher spectral acceleration). Yet, the high capacity to demand ratios obtained by retrofitting for both steel and RC poles lead to the conclusion that steel jacketing can be used for retrofitting of poles.

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Figure Captions

Figure 1. Constructive details of steel jacketing based retrofitting

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

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

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

Figure 5. Acceleration, velocity and displacement time-histories for Slymar Country Hospital records

Figure 6. Acceleration, velocity and displacement time-histories for Santa Monica City Hall records

Figure 7. Elastic spectra for 2, 5, 10 and 20 percent damping ratios

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

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

Figure 10. Axial stress distributions on pre- and post retrofitted sections (units are in N. and mm)

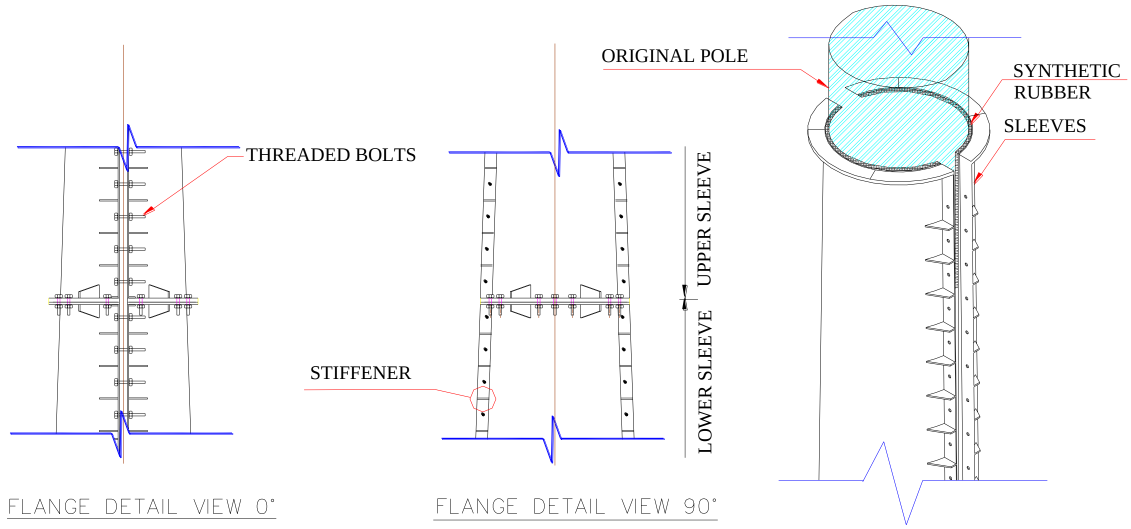
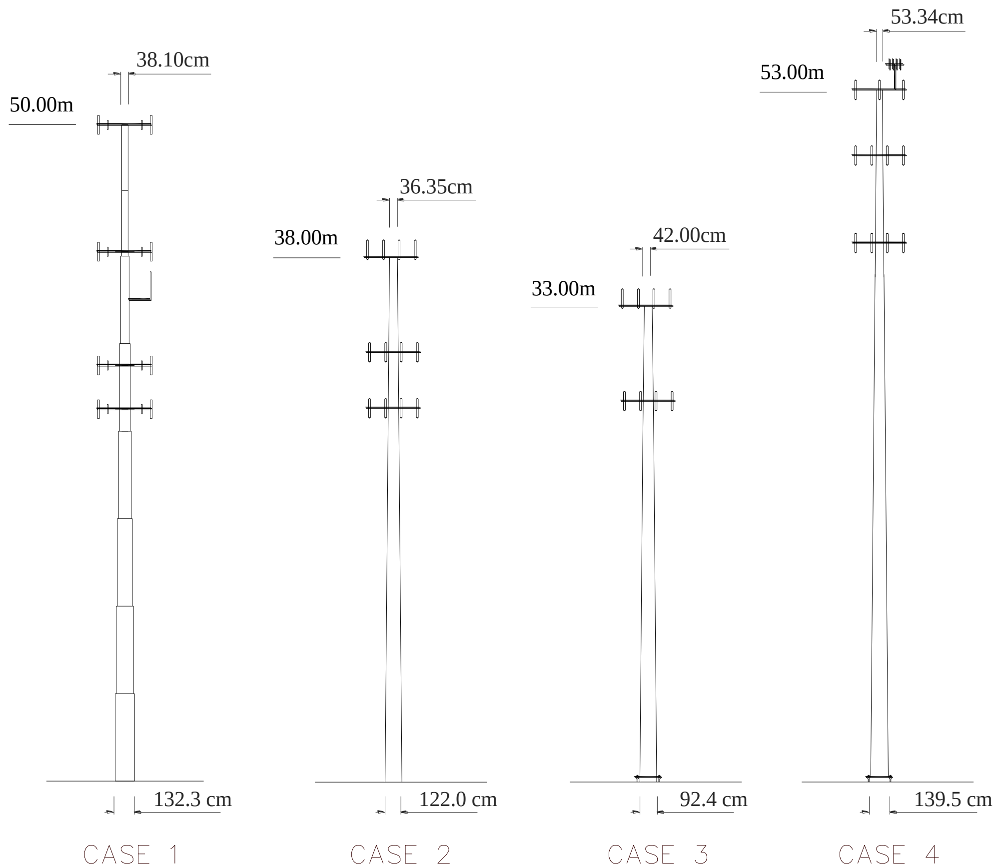

THREADED BOLTS
FLANGE DETAIL VIEW 0°
STIFFENER
UPPER SLEEVE
LOWER SLEEVE
FLANGE DETAIL VIEW 90°
ORIGINAL POLE
SYNTHETIC RUBBER
SLEEVES

Figure 1

(a)

(b)

Figure 2



38.10cm
50.00m
36.35cm
38.00m
42.00cm
33.00m
53.34cm
53.00m
132.3 cm
122.0 cm
92.4 cm
139.5 cm
CASE 1
CASE 2
CASE 3
CASE 4

Figure 3

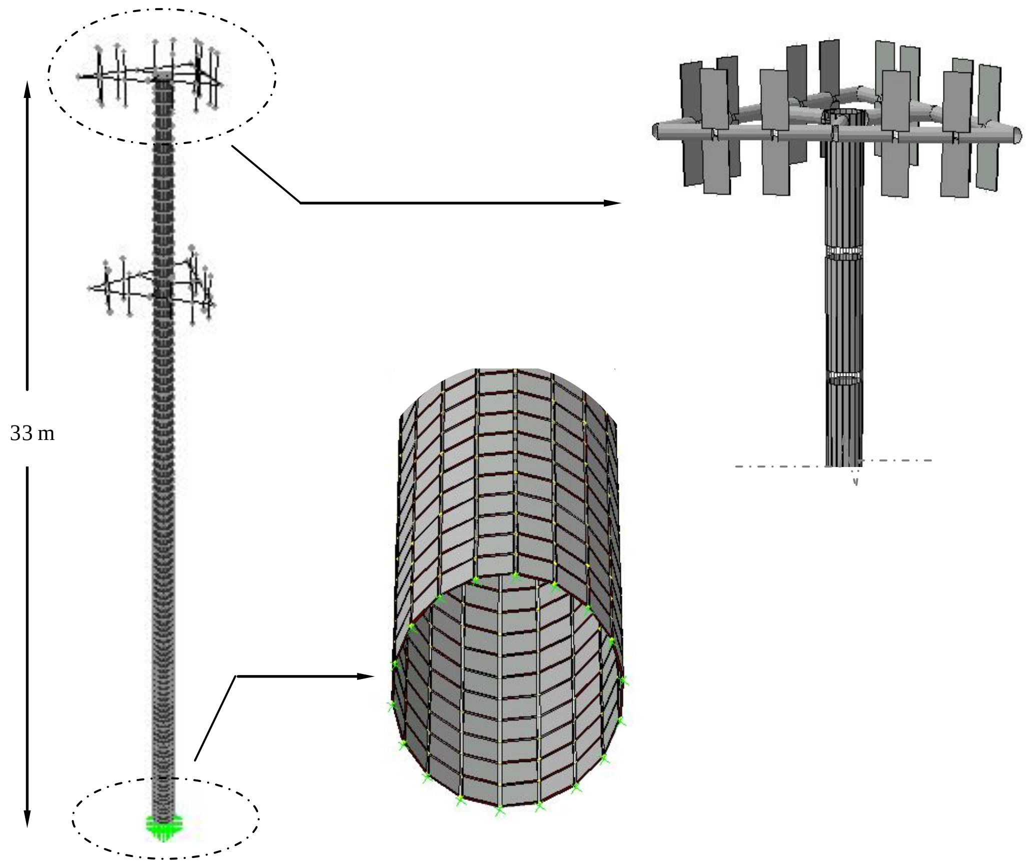


Figure 4

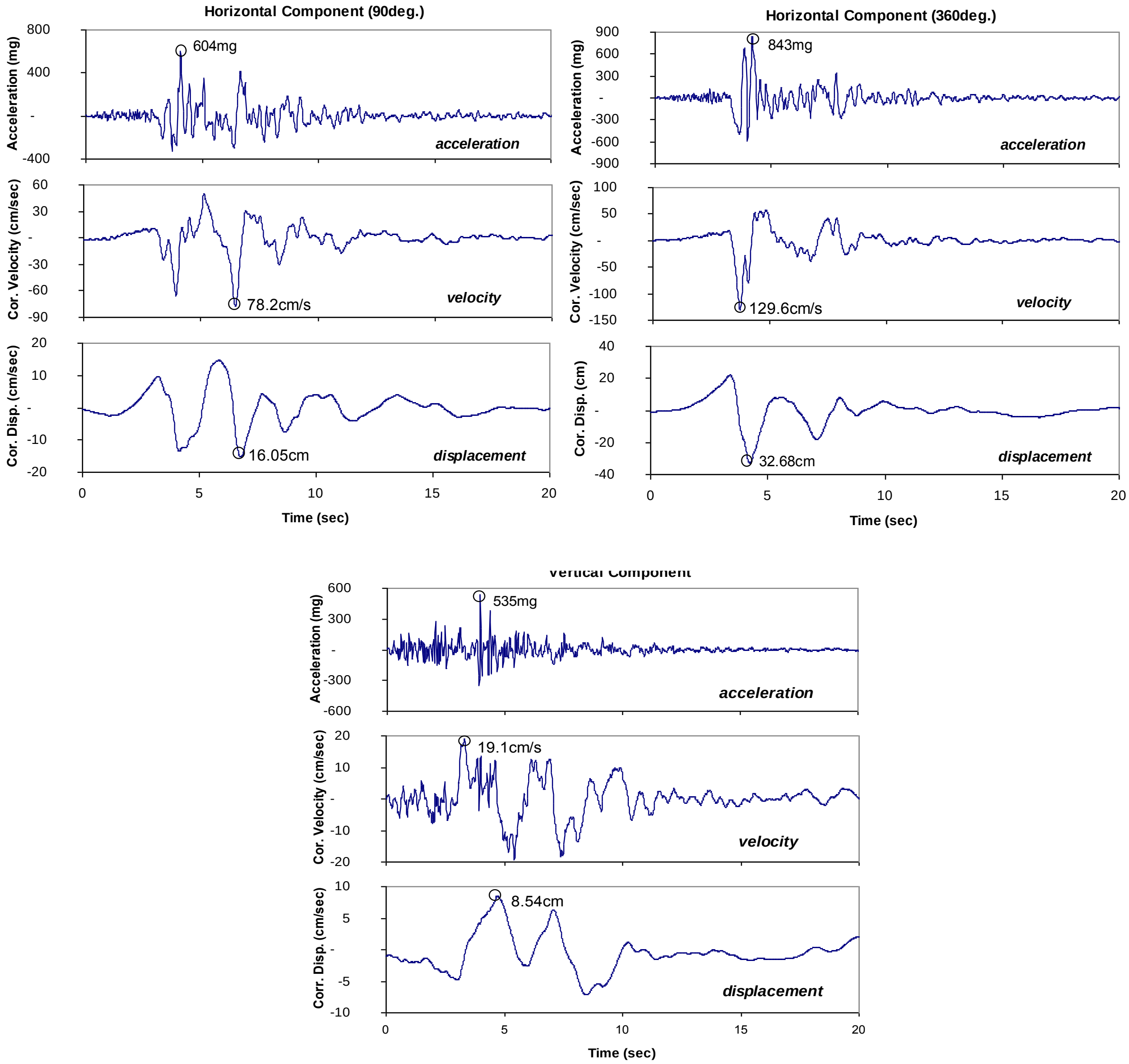

Horizontal Component (90deg.)
Acceleration (mg)
604mg
acceleration
Cor. Velocity (cm/sec)
78.2cm/s
velocity
Cor. Disp. (cm/sec)
16.05cm
displacement
Time (sec)
Horizontal Component (360deg.)
Acceleration (mg)
843mg
acceleration
Cor. Velocity (cm/sec)
129.6cm/s
velocity
Cor. Disp. (cm)
32.68cm
displacement
Time (sec)
Acceleration (mg)
535mg
acceleration
Cor. Velocity (cm/sec)
19.1cm/s
velocity
Corr. Disp. (cm/sec)
8.54cm
displacement
Time (sec)

Figure 5

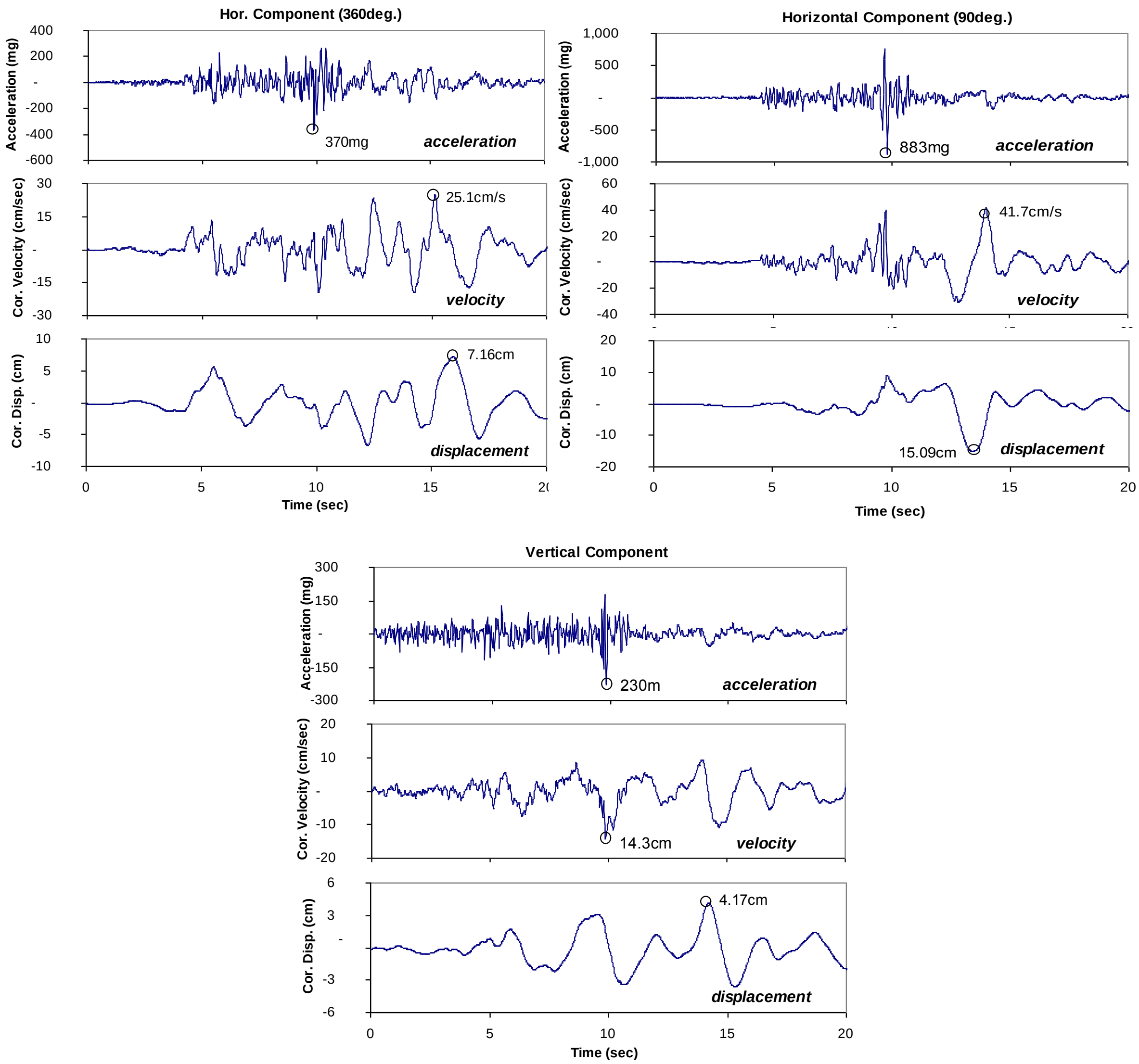
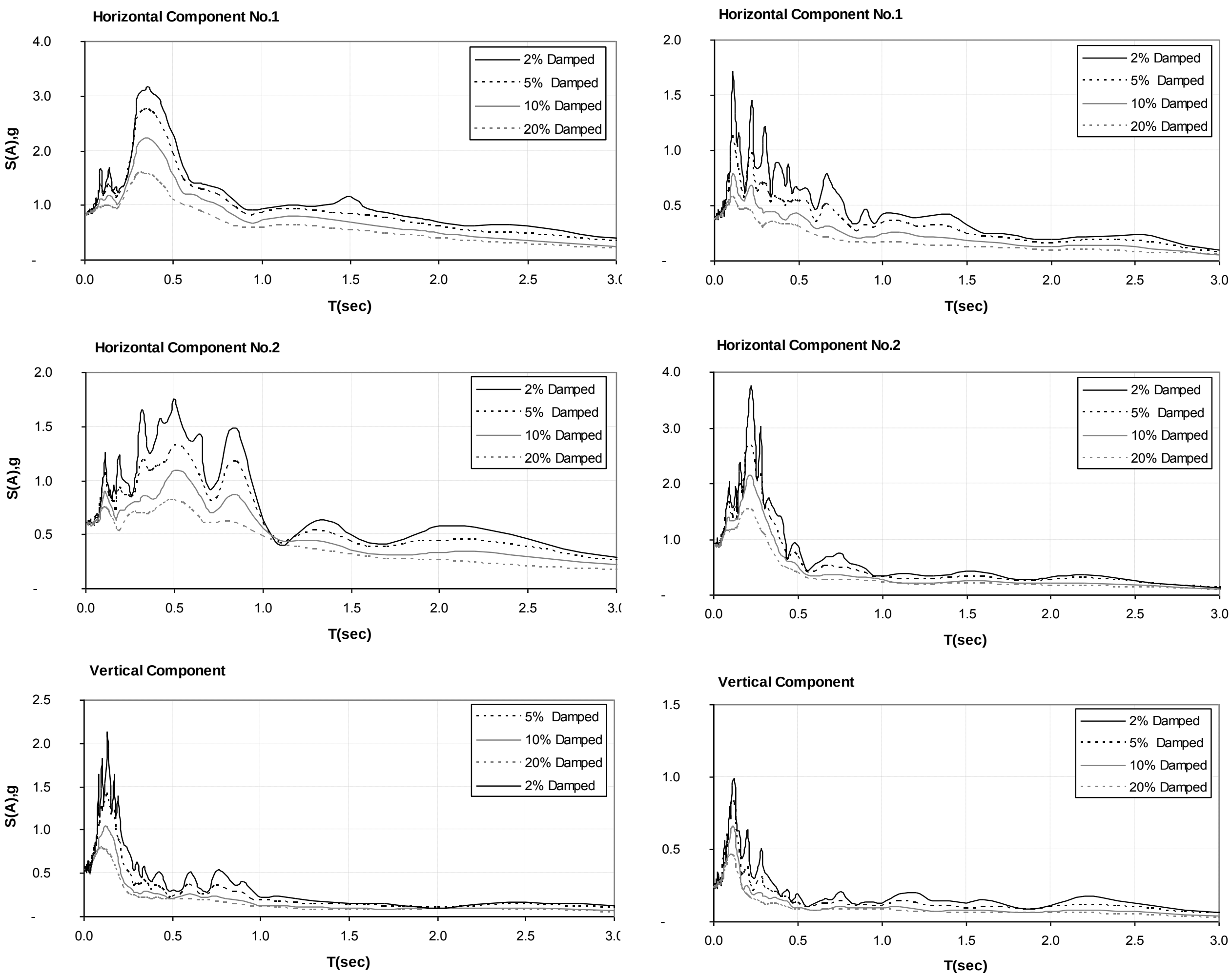
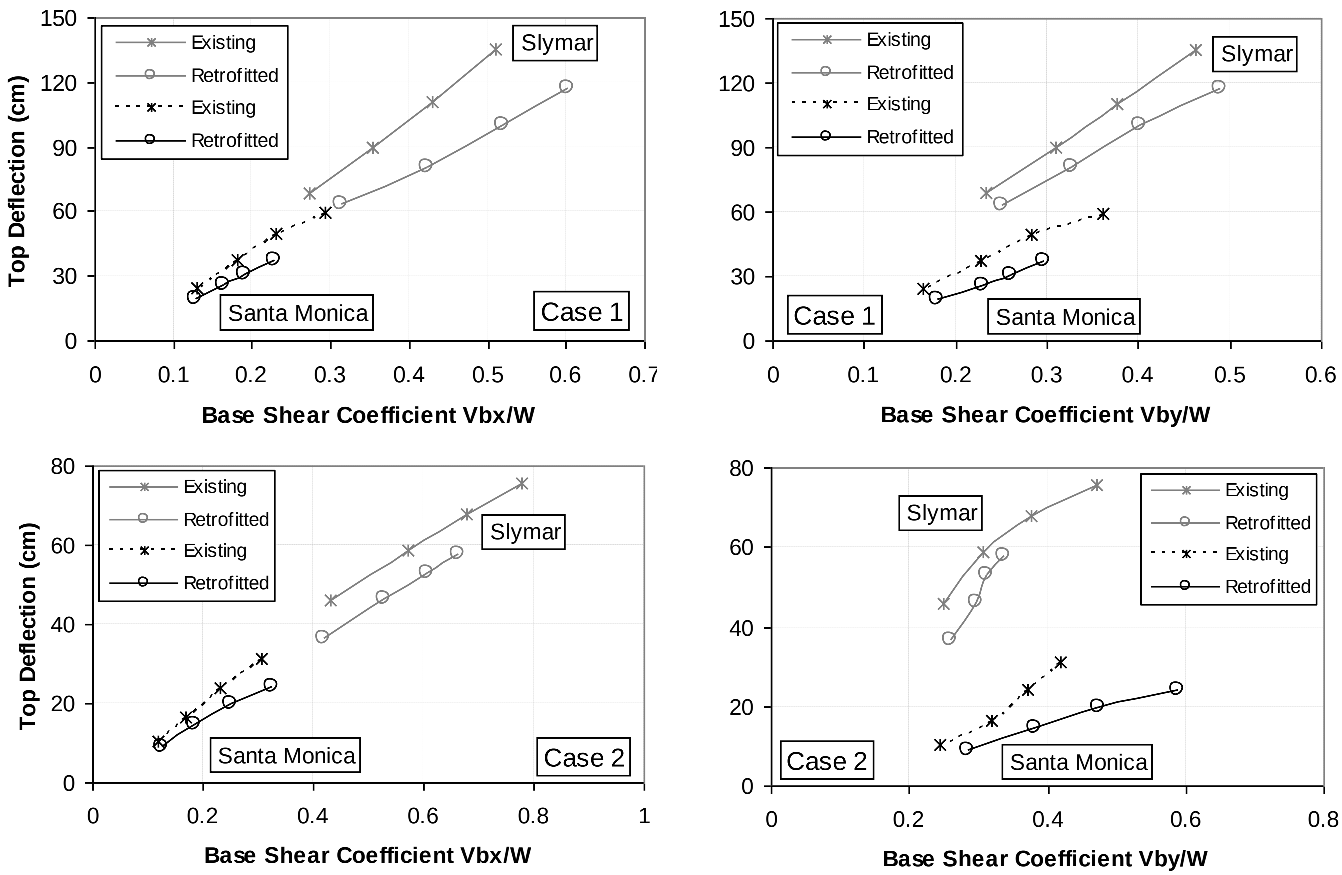

Hor. Component (360deg.)
Acceleration (mg)
Cor. Velocity (cm/sec)
Cor. Disp. (cm)
370mg
acceleration
25.1cm/s
velocity
7.16cm
displacement
Time (sec)
Horizontal Component (90deg.)
883mg
acceleration
41.7cm/s
velocity
15.09cm
displacement
Vertical Component
230m
14.3cm
4.17cm

Figure 6



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

Figure 7



Top Deflection (cm)
Base Shear Coefficient Vbx/W
Base Shear Coefficient Vby/W
Existing
Retrofitted
Existing
Retrofitted
Slymar
Santa Monica
Case 1
Case 2

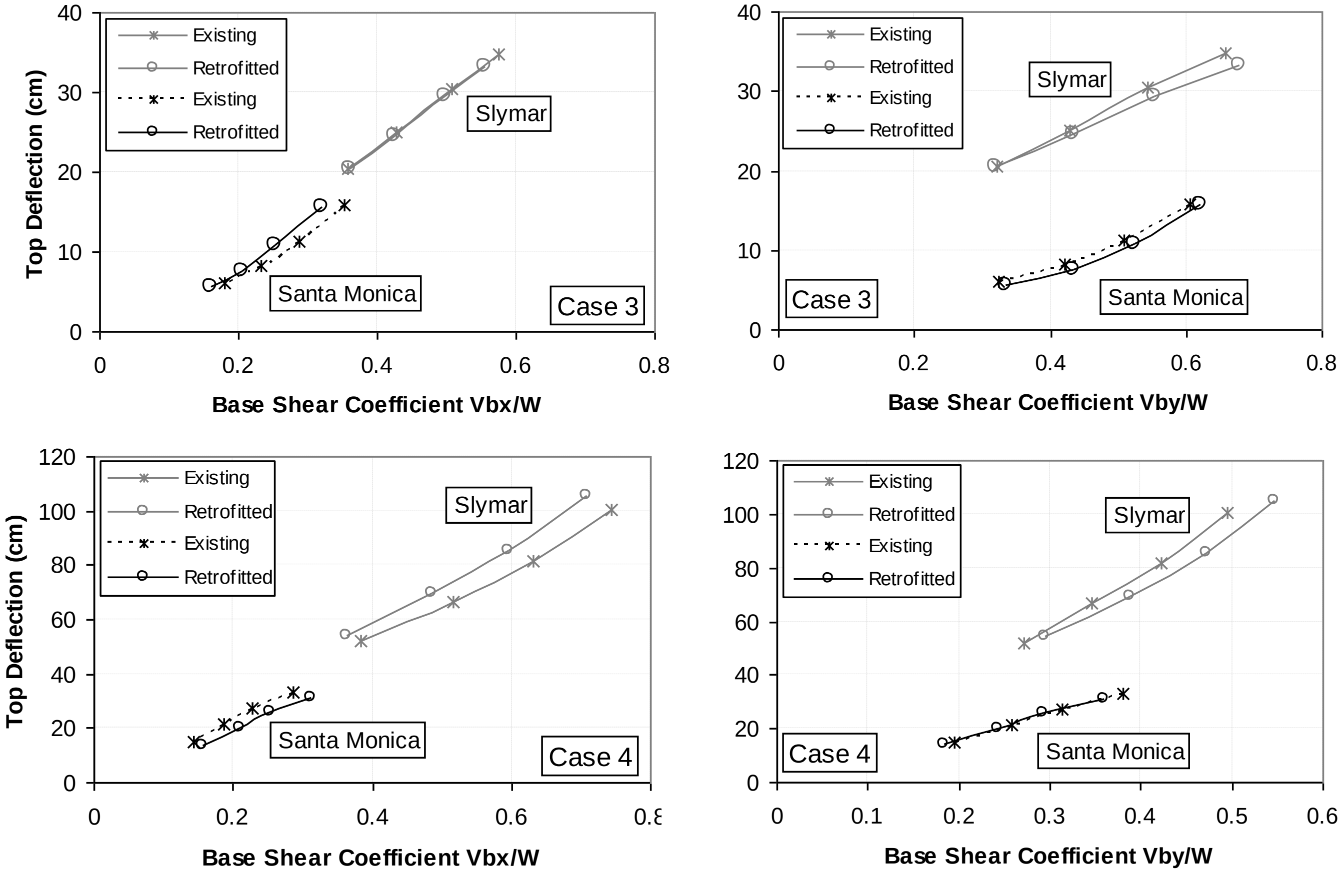
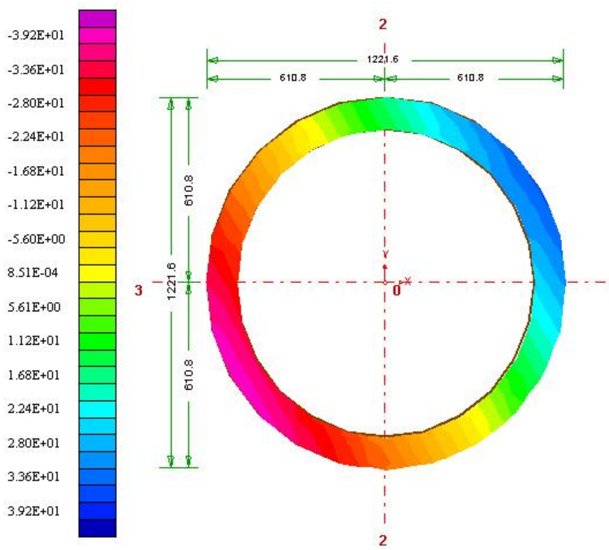
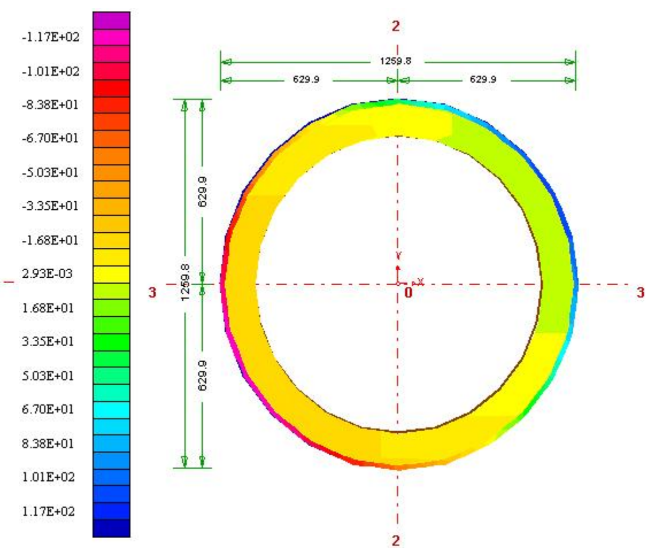

Existing
Retrofitted
Existing
Retrofitted
Slymar
Santa Monica
Case 3
Top Deflection (cm)
Base Shear Coefficient Vbx/W
Base Shear Coefficient Vby/W
Case 4

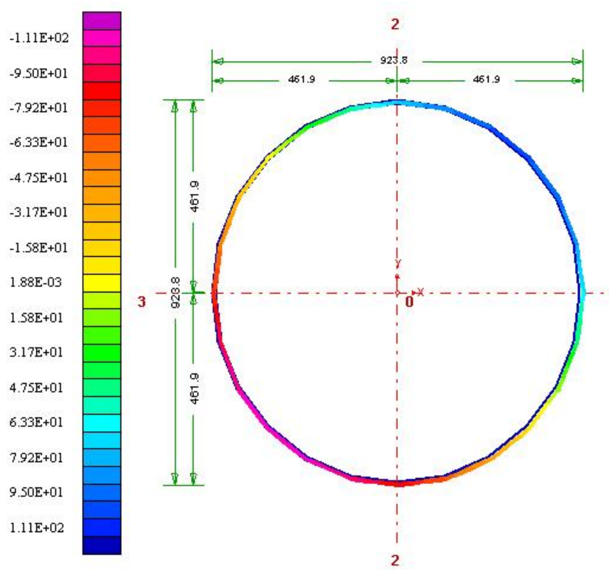
Figure 9



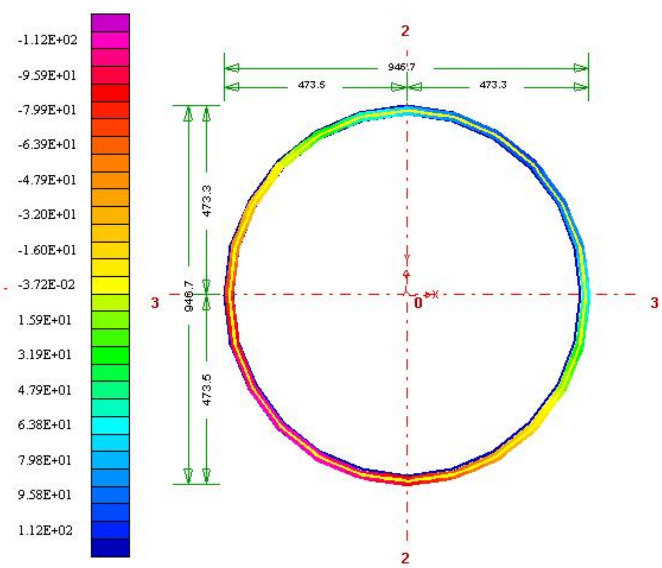
(a) Pre-Retrofitted Section (Case 2)



(b) Post-Retrofitted Section (Case 2)



(c) Pre-Retrofitted Section (Case 3)



(d) Post-Retrofitted Section (Case 3)

Figure 10

Table Captions

Table 1. General material properties of synthetic rubber sheets

Table 2. Structural and architectural configuration of existing pole structures

Table 3. Geometric properties of pre- and post-retrofitted sections

Table 4. Periods, frequencies and effective modal masses of pole structures

Table 5. Top displacement distribution for reinforced concrete and steel poles

Table 6. Axial stress calculations of pre- and post-retrofitted sections

	Neoprene	Nitrile
Durometer	50-60	60
Tensile Strength (Mpa)	6.9-8.3	6.9
Elongation (%)	350	300
Specific Gravity	1.42	1.51
Compressibility (%)	28	24.5

Table 1

	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	132.3 cm	122.0 cm	92.4 cm	139.5 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

Table 2

Reinforced Concrete Poles

	Case 1			Case 2		
	Pre-Retrofitted	Post-Retofitted [1]	Post-Retofitted [2]	Pre-Retrofitted	Post-Retofitted [1]	Post-Retofitted [2]
Outside Dia. (m)	1.323	1.362	1.349	1.222	1.260	1.247
Area* (m^2)	0.391	1.100	0.832	0.358	1.011	0.555
Shear Area* (m^2)	0.279	0.739	0.405	0.347	0.693	0.390
Section Modulus (m^3)	0.111	0.334	0.246	0.057	0.281	0.143
Plastic Modulus (m^3)	0.160	0.445	0.233	0.182	0.378	0.197

Steel Poles

	Case 1		Case 2	
	Pre-Retrofitted	Post-Retofitted [1]	Pre-Retrofitted	Post-Retofitted [1]
Outside Dia. (m)	0.924	0.947	1.395	1.420
Area* (m^2)	0.023	0.041	0.062	0.090
Shear Area* (m^2)	0.015	0.029	0.041	0.063
Section Modulus (m^3)	0.005	0.009	0.021	0.031
Plastic Modulus (m^3)	0.006	0.013	0.027	0.039

[1] Sleeve thickness is 12.8 mm, [2] Sleeve thickness is 6.4 mm

* Equivalent area based on the existing section material property

Table 3

Reinforced Concrete Poles

	Mode No	Pre-Retrofitted Period (sec)	Pre-Retrofitted Frequency (Hz.)	Pre-Retrofitted Mx/MI (%)	Pre-Retrofitted My/MI (%)	Post-Retrofitted [1] Period (sec)	Post-Retrofitted [1] Frequency (Hz.)	Post-Retrofitted [1] Mx/MI (%)	Post-Retrofitted [1] My/MI (%)	Post-Retrofitted [2] Period (sec)	Post-Retrofitted [2] Frequency (Hz.)	Post-Retrofitted [2] Mx/MI (%)	Post-Retrofitted [2] My/MI (%)
Case 1	1	2.208	0.453	9.40	38.68	1.918	0.521	6.91	41.27	2.024	0.494	41.29	6.86
	2	2.208	0.453	38.68	9.40	1.918	0.521	41.27	6.91	2.024	0.494	6.86	41.29
	3	0.523	1.912	2.18	18.64	0.455	2.197	3.09	17.70	0.480	2.083	15.58	5.23
Case 2	1	1.296	0.772	0.58	47.67	1.141	0.876	1.10	47.20	1.204	0.830	3.01	45.28
	2	1.296	0.772	47.67	0.58	1.141	0.876	47.20	1.10	1.204	0.830	45.28	3.01
	3	0.313	3.195	0.73	20.12	0.273	3.663	0.39	20.46	0.288	3.472	0.59	20.27

Steel Poles

	Mode No	Pre-Retrofitted Period (sec)	Pre-Retrofitted Frequency (Hz.)	Pre-Retrofitted Mx/MI (%)	Pre-Retrofitted My/MI (%)	Post-Retrofitted [2] Period (sec)	Post-Retrofitted [2] Frequency (Hz.)	Post-Retrofitted [2] Mx/MI (%)	Post-Retrofitted [2] My/MI (%)
Case 3	1	0.906	1.103	48.34	4.36	0.905	1.105	0.28	52.56
	2	0.906	1.103	4.36	48.34	0.905	1.105	52.56	0.28
	3	0.193	5.192	1.63	19.05	0.192	5.211	0.77	19.89
Case 4	1	1.570	0.637	43.97	5.65	1.570	0.637	47.32	2.37
	2	1.570	0.637	5.65	43.97	1.570	0.637	2.37	47.32
	3	0.355	2.815	17.99	2.57	0.354	2.825	6.77	13.80

[1] Sleeve thickness is 12.8 mm
[2] Sleeve thickness is 6.4 mm

Table 4

Reinforced Concrete Poles

		Case 1						Case 2					
		Pre-Retrofitted		Post-Retrofitted [1]		Post-Retrofitted [2]		Pre-Retrofitted		Post-Retrofitted [1]		Post-Retrofitted [2]	
	Damping	δ_x (cm)	δ_y (cm)	δ_x (cm)	δ_y (cm)	δ_x (cm)	δ_y (cm)	δ_x (cm)	δ_y (cm)	δ_x (cm)	δ_y (cm)	δ_x (cm)	δ_y (cm)
Slymar	2%	135.5	130.7	117.5	99.7	124.9	113.8	75.8	48.6	57.8	28.5	63.0	38.9
	5%	110.5	104.1	100.2	82.6	105.1	90.6	67.7	41.5	53.0	27.8	57.7	36.2
	10%	89.9	82.5	80.8	64.3	84.5	70.8	58.8	35.2	46.3	27.5	50.4	32.8
	20%	68.8	60.9	63.5	50.8	66.1	54.2	45.9	28.3	36.6	24.6	39.7	27.6
Santa Mon.	2%	59.4	70.8	37.1	44.4	44.1	55.0	31.3	27.5	24.4	23.6	25.7	24.8
	5%	49.4	58.0	30.8	36.9	35.3	44.5	24.1	22.7	19.8	17.8	20.2	19.7
	10%	37.7	43.9	25.9	32.7	28.9	37.2	16.4	17.3	14.8	13.8	15.2	14.8
	20%	24.7	28.7	19.6	25.0	20.7	26.6	10.4	14.0	9.3	11.5	9.9	11.9

Steel Poles

		Case 3				Case 4			
		Pre-Retrofitted		Post-Retrofitted [1]		Pre-Retrofitted		Post-Retrofitted [1]	
	Damping	δ_x (cm)	δ_y (cm)	δ_x (cm)	δ_y (cm)	δ_x (cm)	δ_y (cm)	δ_x (cm)	δ_y (cm)
Slymar	2%	34.8	41.6	33.2	42.9	100.6	66.1	105.4	57.7
	5%	30.4	34.3	29.5	35.1	81.6	55.3	85.4	49.2
	10%	25.1	27.0	24.6	27.3	66.7	46.0	69.7	41.3
	20%	20.4	19.9	20.5	19.7	52.2	38.1	54.4	35.0
Santa Mon.	2%	15.8	16.3	15.7	16.3	33.2	42.3	31.3	43.6
	5%	11.3	13.2	10.9	13.4	27.1	33.7	25.8	34.6
	10%	8.2	10.7	7.6	11.0	21.3	26.5	20.2	27.3
	20%	6.1	8.3	5.6	8.5	14.9	19.2	14.0	19.9

[1] Sleeve thickness is 12.8 mm
[2] Sleeve thickness is 6.4 mm

Table 5

			Axial Force (kN)	Mx (kN.m)	My (kN.m)	Stress Check for Original Sec.			Stress Check for Sleeve Sec.		
						Maximum Stress (Mpa)	Section Capacity (Mpa)	Original Sec. Factor of Safety	Maximum Sleeve Stress (Mpa)	Sleeve Sect. Capacity (Mpa)	Sleeve Factor of Safety
Reinforced Conc.	Case 1	Pre-retrofitted	274.6	2235.5	2427.3	33	40	1.21	-	-	-
		Post-Retofitted [1]	389.1	3337.8	4138.9	22	40	1.79	157	345	2.20
	Case 2	Pre-retrofitted	190.4	1248.9	2099.6	39	40	1.02	-	-	-
		Post-Retofitted [1]	269.8	1529.1	2946.7	17	40	2.38	117	345	2.95
Steel	Case 3	Pre-retrofitted	38.9	416.2	371.7	111	345	3.11	-	-	-
		Post-Retofitted [2]	70.2	781.2	663.6	112	345	3.08	112	345	3.08
	Case 4	Pre-retrofitted	159.7	1541.6	2290.1	133	345	2.59	-	-	-
		Post-Retofitted [2]	232.5	2166.0	3388.8	134	345	2.57	134	345	2.57

[1] Sleeve thickness is 12.8 mm, [2] Sleeve thickness is 6.4 mm

Table 6