EFFECT OF NONLINEARITY IN PIER AND WELL FOUNDATION ON SEISMIC RESPONSE OF BRIDGES

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ABSTRACT:
Nonlinear seismic analysis of soil-well-pier system of a typical bridge supported on well foundation is carried out considering nonlinearity in piers and well. Bi-linear kinematic element is used to model nonlinearity in piers and well. Separation at the interface of soil and well is considered using compression-only gap elements. Analyses have been performed in two steps. In the first step, for a given acceleration time-history one-dimensional free-field analysis of the site is performed using SHAKE2000 to obtain the motion at the base of the soil profile. In the next step, this motion is applied at the base of finite element model of soil-well-pier system in SAP2000. The analysis has been carried out for different values of depth of scour and for two different earthquake motions in longitudinal direction. It is found that bending moment demand exceed the capacity by 20% to 70% in piers and 30% to 75% in well when piers and well are assumed to behave linearly. Subsequently, nonlinearity in piers is introduced when well is considered as linear. The analysis results show that nonlinearity in piers does not considerably reduce the force response of well. Therefore, nonlinearities in both piers and well are introduced in the next step. In this case, 15% to 50% reduction in rotational ductility demand in piers is observed but now the well must have adequate rotational ductility.

KEYWORDS:
Bridge, well foundation, caisson foundation, seismic analysis, nonlinear analysis

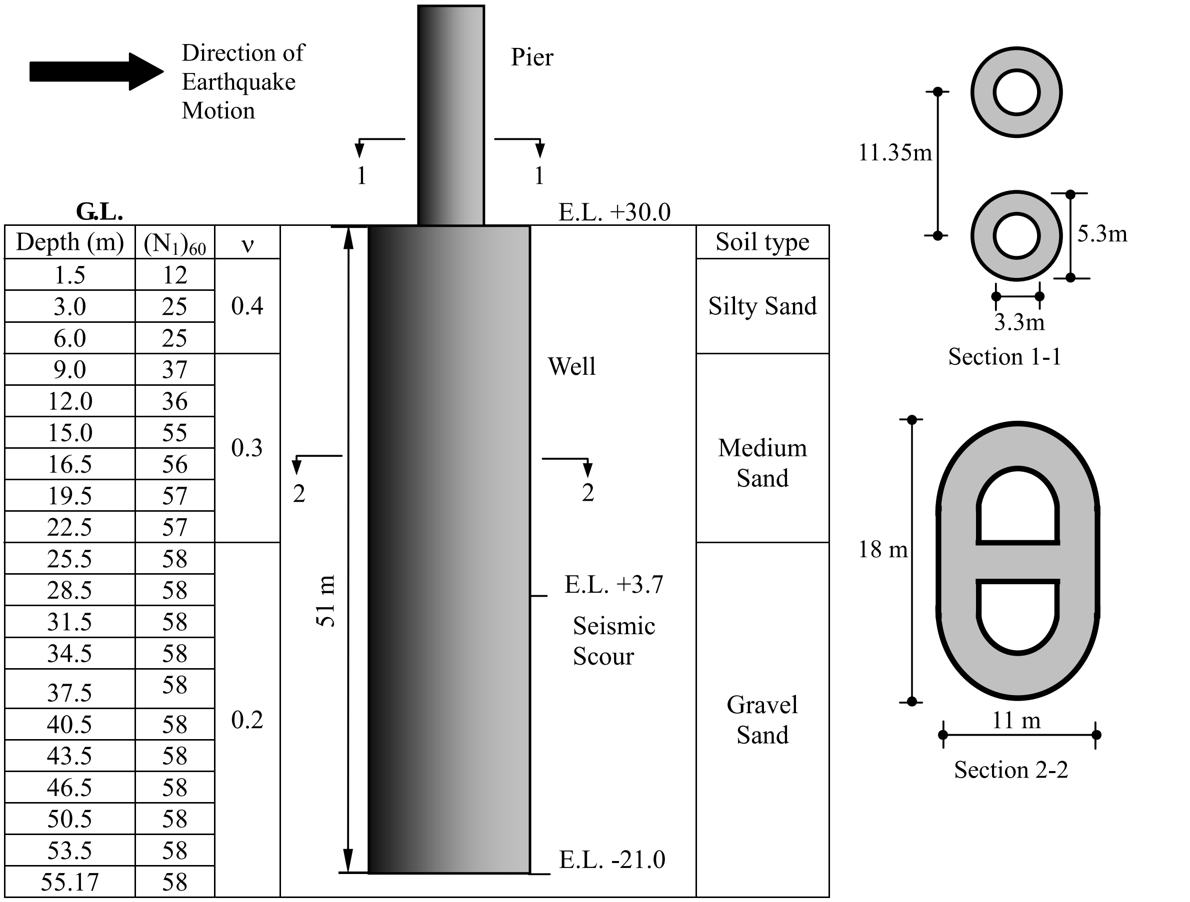
1. INTRODUCTION

Well foundations are commonly used in the Indian subcontinent for both railway and road bridges on river streams. Seismic response of such foundations depends on several factors namely, shear modulus and hysteresis damping in soil, radiation damping, spatial variations of earthquake motion at different depths, nonlinearity at soil-well interface, nonlinearity in pier and well, hydrodynamic force, etc. Several researchers have analysed well foundation where the soil was modelled as lumped (Arya and Thakkar, 1970; Thakkar *et al.*, 1991) or discrete springs (Arya and Thakkar, 1986). These approaches partially account for soil-well-pier interaction effects with nominal computational effort. More advanced methods account for soil-structure interaction (SSI) of well foundation; however, these methods do not account for nonlinearity in pier and well and/or at the interface of well and soil (Chang *et al.*, 2000; Tsigginos *et al.*, 2008; Zheng and Takeda, 1995).

In the present study, nonlinear dynamic analysis of a typical bridge supported on well foundation is performed considering pier and well nonlinearity. Apart from that, interface nonlinearity and hydrodynamic effect are also considered. However, yielding of soil surrounding the well is not accounted for. The analysis has been performed for different values of depth of scour and for two different earthquake motions in longitudinal direction.

2. DESCRIPTION OF THE BRIDGE

A typical bridge, simply supported on well foundations is considered for the present study (Figure 1). The substructure consists of two hollow circular reinforced concrete piers supported on a double-D hollow reinforced concrete well foundation (Figure 1). The bridge decks are assumed to be simply supported on piers, and therefore, an isolated well and two piers are considered as a vibration unit during seismic analysis.



Depth (m)	$(N_1)_{60}$	ν	Soil type
1.5	12	0.4	Silty Sand
3.0	25		
6.0	25		
9.0	37	0.3	Medium Sand
12.0	36		
15.0	55		
16.5	56		
19.5	57		
22.5	57		
25.5	58	0.2	Gravel Sand
28.5	58		
31.5	58		
34.5	58		
37.5	58		
40.5	58		
43.5	58		
46.5	58		
50.5	58		
53.5	58		
55.17	58		

Figure 1 Average soil profile along bridge alignment and schematic drawing of well foundation

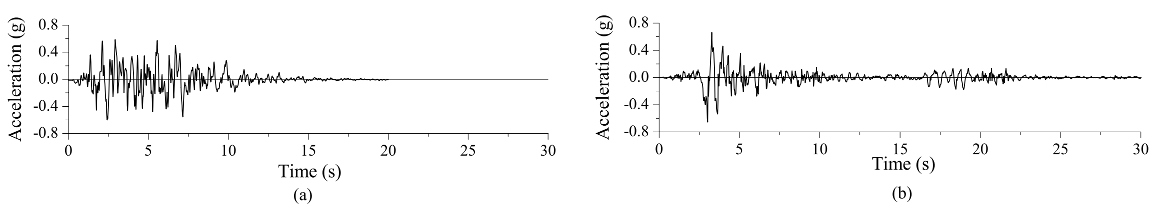


Figure 2 Acceleration time history for (a) SEE and (b) CME

3. ANALYSIS PROCEDURE

The bridge is analysed for two different earthquake motions: Safety Evaluation Earthquake (SEE) and 1992 Cape Mendocino earthquake (CME) (Figure 2). SEE is specified with PGA of 0.6g caused by a magnitude 7.0 earthquake while CME is an independent near-field earthquake motion (epicentral distance 4.5 km) with PGA 0.662g. In this paper, these motions are applied in longitudinal direction only. Analysis has been performed in two steps (Figure 3). In the first step, one dimensional free-field analysis of the site is performed in SHAKE2000 using the above ground motions at the ground level of the soil column to obtain the motion at the base of the model. In this analysis strain-dependent shear modulus and damping are used to evaluate effective shear modulus and damping of soil at each layer. In the next step, these effective properties of soil are used to perform nonlinear modal time history analysis of the soil-well-pier FE model in SAP2000 using the time history obtained from the previous step (Figure 3). Initially, both piers and well are assumed as linear. Later, pier nonlinearity in is introduced while well is assumed to behave as linear structure in order to examine the effect of pier nonlinearity on the response of well. If bending moment demand in well is more than the capacity, nonlinear analysis of the soil-well-pier model is performed considering nonlinearity in both piers and well.

4. FE MODELLING OF THE SYSTEM

Figure 3 shows a schematic of idealisation of the entire structure. In the analysis, only mass of the superstructure is modelled and is applied at the pier cap. The mass of water in the enveloping cylinder of the submerged part of the well above the ground level is added to the structural mass. The mass of water and sand inside the well have been appropriately considered in the analysis.

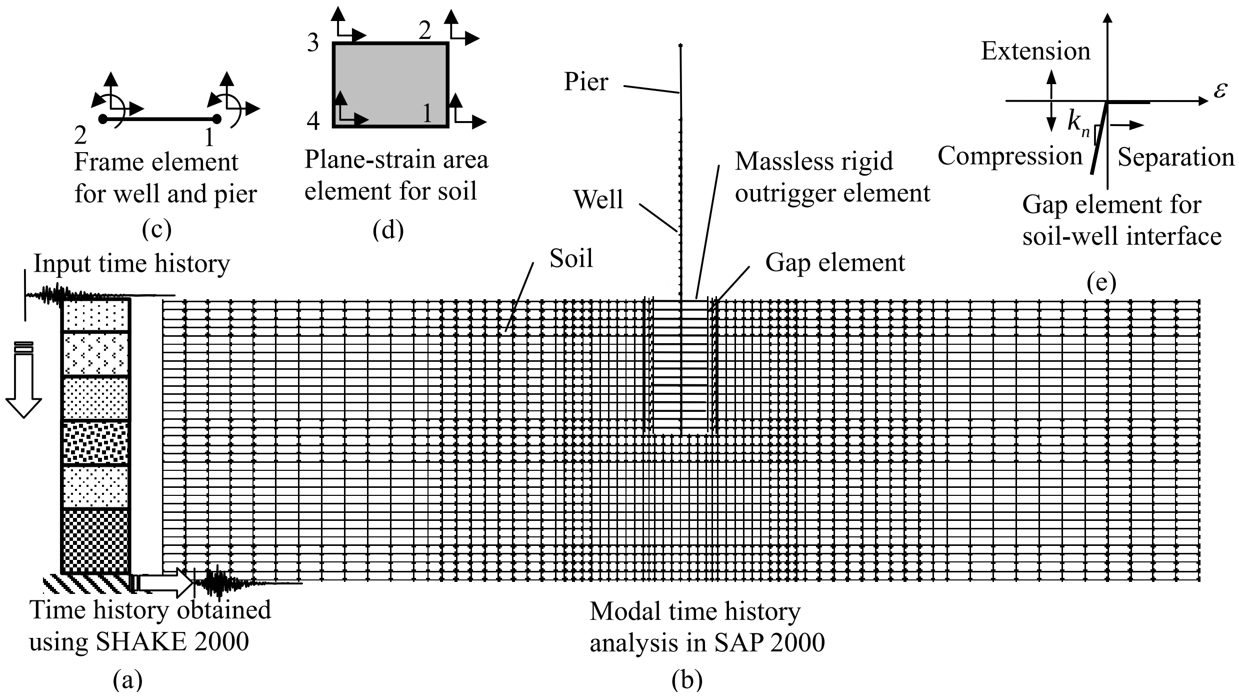


Figure 3 Schematic representation of the analysis procedure and modelling of 2-D soil-well-pier model

Piers and well are modeled as two-noded frame elements. Massless rigid-outrigger elements are added in the embedded part of the well to account for the breadth of the well when interacting with soil. Plastic hinges in piers are modelled by lumped plasticity model in the form of bi-linear kinematic rotational springs with degrading hysteretic loop recommended by Takeda *et al.* (1970). These springs are lumped at a distance $L_p/2$ from the bottom end of the piers where L_p is defined as (Priestley *et al.*, 1996):

$$L_p = 0.08l + 0.022 f_y d_b \geq 0.044 f_y d_b \tag{4.1.1}$$

where, l is the height of the piers (m), d_b is the diameter of longitudinal reinforcement (m), and f_y is the yield strength of reinforcement (MPa). These springs are assumed to be rigid under shear and axial forces. Flexural deformation of the plastic hinge is taken care of by the spring elements and all the shear and axial deformations are taken by linear frame elements. Stiffness values of spring elements are defined by moment-rotation (M-θ) curves which are derived from the moment-curvature (M-φ) curves of the piers. M-φ curves have been derived from the moment–curvature analysis of piers and well section considering confinement model proposed by

Mander *et al.* (1988).

Soil surrounding the well is modelled as four-noded two-dimensional plane-strain element (Figure 3). Element size in FE model is chosen such that it can satisfactorily represent propagating waves of desired frequency. However, as major part of the motion consists of vertically propagating waves with horizontal wave front, horizontal dimension of the elements can usually be chosen several times the vertical dimension (Lysmer *et al.*, 1975). Here, the vertical dimension of the element is considered as 1.5 m while horizontal dimension gradually increases from 1.5 m at center to 5 m towards the boundary of the soil medium. Thickness of each soil element is taken as 18 m which is same as the well dimension perpendicular to the direction of motion. Vertical soil boundaries are restricted at 300 m away from the centre of well assuming that response of well will be unaffected by the boundary condition at the two vertical sides. Vertically fixed and horizontally free boundary conditions are applied at the two vertical boundaries of the model since this boundary condition converges faster to the infinitely long model (Agarwal, 2006). Bottom boundary is restricted at 35 m below the bottom of well and is restrained in both horizontal and vertical directions since it is considered that soil stratum is rested on hard rock.

Interface nonlinearity in the form of gapping between soil and well surface has been modelled by compression-only nonlinear springs (Figure 3b). Infinite stiffness of rigid-plastic gap element causes numerical problem. Therefore, elasto-plastic gap elements with very high initial stiffness have been used during the actual numerical computations (Figure 3e).

5. MATERIAL PROPERTIES

Figure 1 shows the soil properties of different layers used in the present study. Shear moduli reduction (G/G_{max}) curves proposed by Sun *et al.* (1988) are used to obtain strain-dependent shear modulus of soil. Similarly, strain-dependent damping curve proposed by Seed and Idriss (1970) are used for silty sand and that proposed by Lysmer *et al.* (1971) are used for medium and gravel sand. Small strain shear moduli, G_{max} (in kN/m^2) have been estimated from the following equation (Seed *et al.*, 1986):

$$G_{\max} = 3610\left(N_1\right)_{60}{}^{0.33}\left(\sigma_0'\right)^{0.5} \tag{5.1.1}$$

where, σ_0' is the effective stress in kN/m^2, and $\left(N_1\right)_{60}$ is the SPT N-value corrected for energy and overburden pressure.

Modulus of elasticity of concrete (E_c) for both the substructure and the foundation is taken as 27,400 MPa. The Poisson's ratio and mass density of concrete are taken as 0.15, and 2,500 kg/m^3, respectively. The bridge is allowed to undergo inelastic action and properties of cracked section are used. The stiffness of the cracked section is considered by the secant slope of the M-φ curve corresponding to 60% of the ultimate moment capacity. In the moment-curvature analysis, the effect of confinement in enhancing the strength and ductility of the concrete is taken into account. Confinement model proposed by Mander *et al.* (1988) to quantify the effect of transverse steel is used. Unconfined compressive strength of concrete ($f_c^{'}$) is taken as 28 MPa.

HYSD steel bars of grade Fe 415 are used. Such bars often exhibit actual yield strength much higher than their specified yield strength (f_y) of 415 MPa. In the present study, mean yield strength of 440 MPa and overstrength of $1.27\,f_y$ are used in this analysis. Section Designer in SAP2000 is used to perform moment-curvature analysis of the pier section.

6. ANALYSIS CASES

Seismic analysis has been performed for three cases. In Case 1 it is assumed that at the time of earthquake, full scouring has taken place corresponding to the annual mean discharge in the river. Therefore, embedment depth of well is taken as 24.7 m. Case 2 does not consider seismic scour and 51 m embedment depth of well is adopted in this case. It is assumed in Case 3 that at the time of earthquake motion scouring will take place only up to half the depth

Table 1 Force responses of piers considering linear piers and well

Earthquake	Analysis Cases	Response Cases	Axial Force (MN)	Shear Force (MN)		Bending Moment (MN-m)	
			P_m	V_m	V_u	M_m	M_u
SEE	Case 1	P_{max}	22	13	25	156	181
		V_{max}	22	15	25	**194**	181
		M_{max}	22	15	25	**194**	181
	Case 2	P_{max}	21	1	25	10	180
		V_{max}	21	17	25	**217**	180
		M_{max}	21	17	25	**217**	180
	Case 3	P_{max}	21	16	25	195	180
		V_{max}	21	23	25	**295**	180
		M_{max}	21	23	25	**295**	180
CME	Case 1	P_{max}	22	11	25	139	181
		V_{max}	22	21	25	**275**	181
		M_{max}	22	21	25	**275**	181
	Case 2	P_{max}	21	12	25	147	180
		V_{max}	21	21	25	**266**	180
		M_{max}	21	21	25	**268**	180
	Case 3	P_{max}	21	15	25	182	180
		V_{max}	21	24	25	**309**	180
		M_{max}	21	24	25	309	180

Table 2 Force responses of well considering linear piers and well

Earthquake	Analysis Cases	Response Cases	Axial Force (MN)	Shear Force (MN)		Bending Moment (MN-m)	
			P_m	V_m	V_u	M_m	M_u
SEE	Case 1	P_{max}	225	5	129	425	1744
		V_{max}	191	106	127	735	1606
		M_{max}	172	46	126	**2590**	1532
	Case 2	P_{max}	351	16	136	248	2232
		V_{max}	265	126	131	1741	1902
		M_{max}	308	43	134	**2630**	2070
	Case 3	P_{max}	269	22	131	387	1918
		V_{max}	244	107	130	250	1819
		M_{max}	199	59	127	**2709**	1639
CME	Case 1	P_{max}	216	13	128	710	1709
		V_{max}	178	114	126	1155	1556
		M_{max}	177	43	126	**2700**	1551
	Case 2	P_{max}	347	29	136	241	2217
		V_{max}	283	118	132	1648	1972
		M_{max}	283	72	132	**2692**	1972
	Case 3	P_{max}	266	14	131	203	1906
		V_{max}	245	113	130	769	1823
		M_{max}	194	36	127	**2686**	1620

of full seismic scour. Therefore, embedment depth of well is taken as 38.75 m. Added mass of water is considered in Case 1 but is neglected in Case 2 and Case 3.

7. RESULTS AND DISCUSSION

Analysis of soil-well-pier system has been carried out for three analysis cases (Case 1, Case 2 and Case 3) and for two earthquake motions, i.e., SEE and CME in longitudinal direction (along traffic). Responses of piers and well have been taken assuming that maximum demand of axial force, shear force and bending moment will not occur simultaneously. Therefore, shear force and bending moment demands of any member have been obtained when axial force demand is maximum (P_{max}) (Table 1). Similarly, axial force and bending moment demands have been obtained at the instant when maximum shear force (V_{max}) is observed. Similarly, when bending moment is maximum (M_{max}), axial force and shear force demands have been obtained.

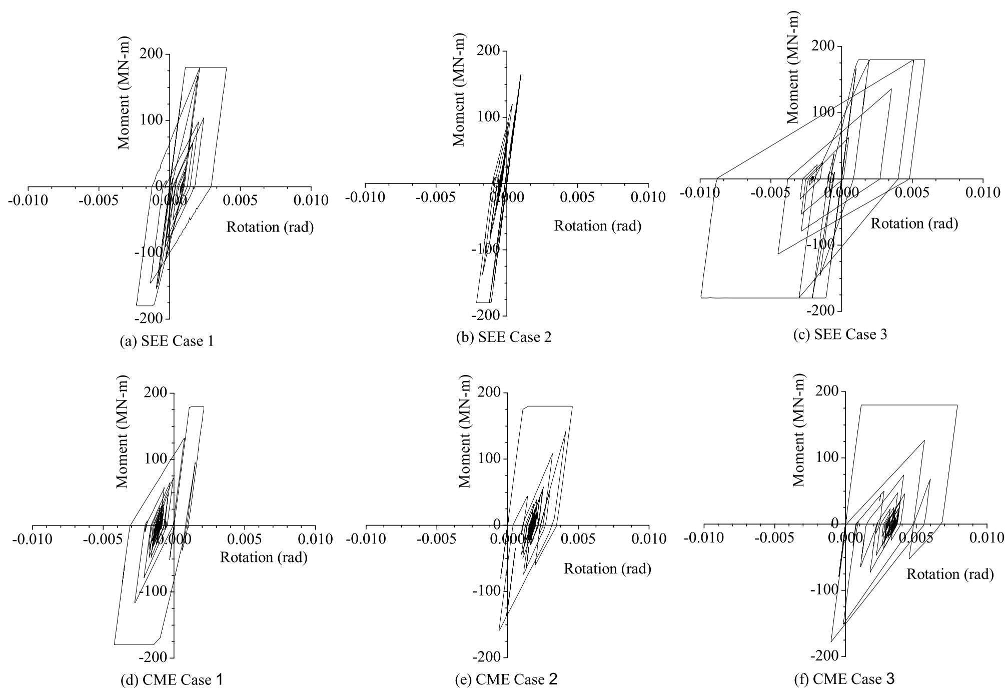


Figure 4 Hysteretic responses of piers (assuming well remains linear)

The bridge is first analyzed assuming piers and well both are linear. It is found that bending moment demands (M_m) in piers are exceeding the capacity (M_u) by 20% to 70% except in Case 1 under SEE, where bending moment demand in piers is exceeding the capacity by 7% only (Table 1). Moreover, the well demands are exceeding the capacity by 30% to 75%. Therefore in the next step, nonlinear analyses are carried out considering pier nonlinearity while well is assumed to behave linearly. Hysteretic behaviour of piers illustrates that significant nonlinearity occurs in piers during strong ground motion (Figure 4). The rotational ductility demand in piers for the two ground motions and three cases ranges from 2.0 to 8.5. From the comparison of Table 2 and Table 3, it is found that effect of pier nonlinearity on the response of well is insignificant and the maximum bending moment in well reduces by not more than 6% due to nonlinearity in the piers. As a result, the bending moment demands in well still exceed the corresponding capacities by 30% to 75%.

Therefore in the next phase, nonlinear analyses of the models are carried out considering nonlinearity in both piers and well. It is observed that nonlinearity in well considerably reduces bending moment demand in the piers (Figure 5). For model in Case 1 under SEE, the bending moment demand is less than the capacity of the piers and the piers behave linearly (Figure 5). In other models, hysteretic behaviour is observed in the piers and the ductility demand in piers now ranges from 1.5 to 4.3. However, well goes to nonlinear region for all the cases and the rotational ductility demand in the well ranges from 1.5 to 5.0 (Figure 6). In other words, nonlinearity in the well reduces the rotational ductility demand in the piers by 15% to 50%. Normally, foundation is not allowed to yield due to difficulty in inspection, repair and replacement. Moreover, the wells

Table 3 Force responses of well considering nonlinearity in piers only

Earthquake	Analysis Cases	Response Cases	Axial Force (MN)	Shear Force (MN)		Bending Moment (MN-m)	
			P_m	V_m	V_u	M_m	M_u
SEE	Case 1	P_{max}	221	60	129	597	1729
		V_{max}	184	110	127	884	1580
		M_{max}	178	49	126	**2584**	1556
	Case 2	P_{max}	351	6	136	194	2232
		V_{max}	263	124	131	**2373**	1894
		M_{max}	307	70	134	**2629**	2065
	Case 3	P_{max}	269	8	131	831	1918
		V_{max}	244	111	130	276	1819
		M_{max}	196	50	127	**2599**	1628
CME	Case 1	P_{max}	216	12	128	713	1709
		V_{max}	164	112	125	587	1498
		M_{max}	170	50	125	**2691**	1516
	Case 2	P_{max}	347	26	136	261	2217
		V_{max}	261	117	131	1478	1886
		M_{max}	292	58	133	**2655**	2007
	Case 3	P_{max}	266	8	131	118	1906
		V_{max}	245	108	130	750	1823
		M_{max}	203	47	127	**2531**	1656

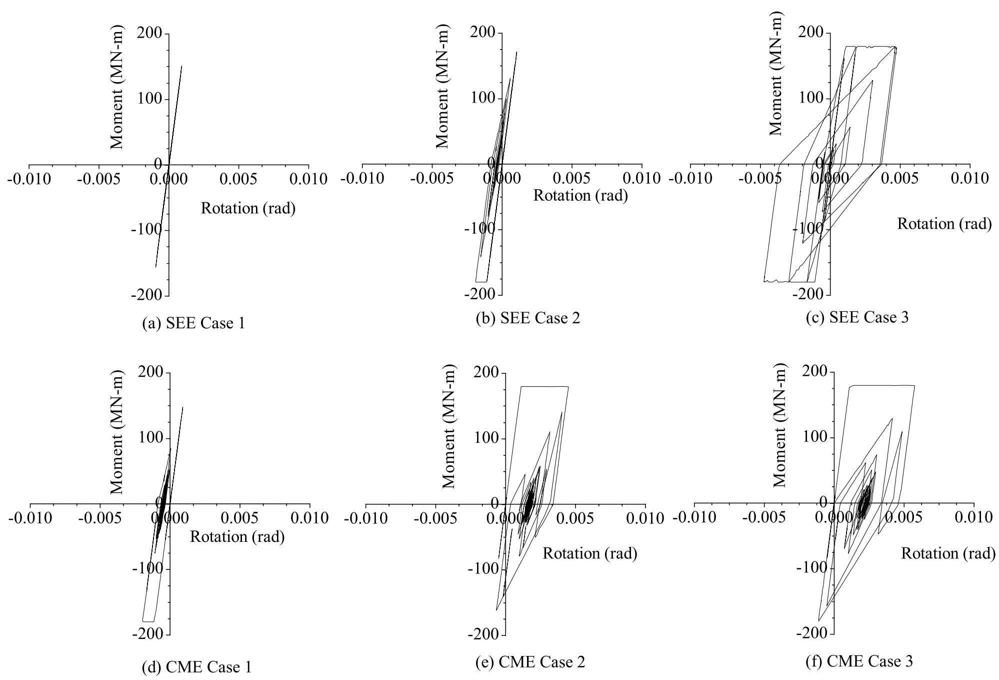


Figure 5 Hysteretic responses of piers considering nonlinearity in piers and well

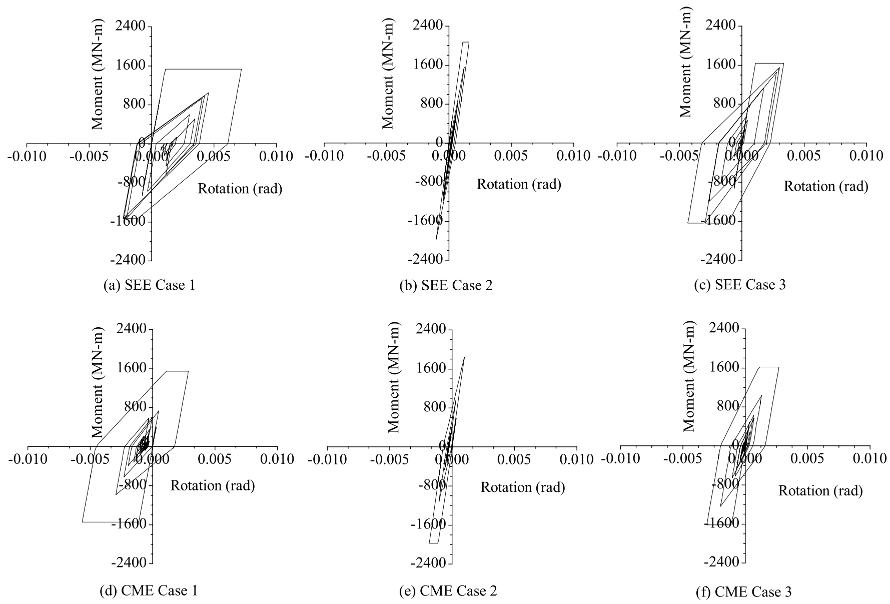


Figure 6 Hysteretic responses of well considering nonlinearity in both piers and well

are usually provided only nominal reinforcement and hence it may be impractical to design and detail the well for substantial ductility. Therefore, one needs to increase the capacity of the well. Else, one may carry out more sophisticated analysis to see if consideration of other sources of energy dissipation (such as yielding of soil adjacent to the well) shows the well to be safe.

8. SUMMARY AND CONCLUSION

Well foundation is a popular foundation system in Indian subcontinent for bridges on rivers especially where the scour in river bed is a major concern. Many of these bridges are located in high seismic region. In the present study, seismic analysis of 2-D soil-well-pier system is performed for three possible cases of embedment length and for two earthquake motions in longitudinal direction considering structural and interface nonlinearity. The bridge is analysed assuming the piers and the well as linear structure and in the subsequent steps nonlinearity in piers and in well are added to see their effect on the response of the bridge. It is found that pier nonlinearity does not substantially reduce the response of well while nonlinearity in well reduces the rotational ductility demand in the piers. However, in such a situation, the well must possess adequate rotational ductility that in most situations may be impractical to ensure. Therefore, either one should increase the capacity of well or carry out more sophisticated analysis.

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