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Performance-based framework for soil-structure systems using simplified rocking foundation models

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Abstract. Results from nonlinear time-history analyses of wall-frame structural models indicate that the condition of vulnerable foundations -for which uplifting and reaching the bearing capacity of the supporting soil can occur before yielding at the base of the shear walls- may not be necessarily detrimental to the drift response of buildings under strong ground motions. Analyses also show that a soil-foundation system can inherently have deformation capacity well in excess of the demand and thus act as a source of energy dissipation that protects the structural integrity of the shear walls.

Keywords: soil-structure interaction; vulnerable foundations; performance-base framework; nonlinear analysis; seismic strengthening

1. Introduction

Seismic evaluation and seismic rehabilitation standards (ASCE 31-03 and ASCE 41-06) support the notion that the expected performance of a building under major ground motions largely depends on the deformation capacity of members and connections of the lateral-load-resisting system. As elegantly captured in a quote by the late Professor Hardy Cross "strength is essential but otherwise unimportant", the importance of deformation capacity in performance-based frameworks has been recognized for decades. Unfortunately, there still appears to be a major disconnect to this philosophy when it comes to soil-foundation systems because current standards only go as far as recognizing the finite stiffness and the finite bearing capacity of foundations with no consideration to their deformation capability. Sufficient foundation deformation capacity can be advantageous to the seismic performance of structures because of the energy dissipation associated to yielding of the soil and period shifting associated to rocking and uplifting (Gajan *et al*. 2008, Harden and Hutchinson 2009, Ugalde *et al*. 2010), however, drift demands may be more significant.

Foundation rotational capacity is relevant to the seismic performance of a building when foundations are vulnerable either to uplifting and/or to reaching the bearing capacity of the supporting soil during a strong ground motion. A practical case in which foundations can become vulnerable is illustrated Fig. 1 and consists of a reinforced concrete moment frame building deemed to be inadequate for current code's seismic design provisions and subsequently rehabilitated through the installation of shear walls. Originally, foundations are proportioned for demands that are

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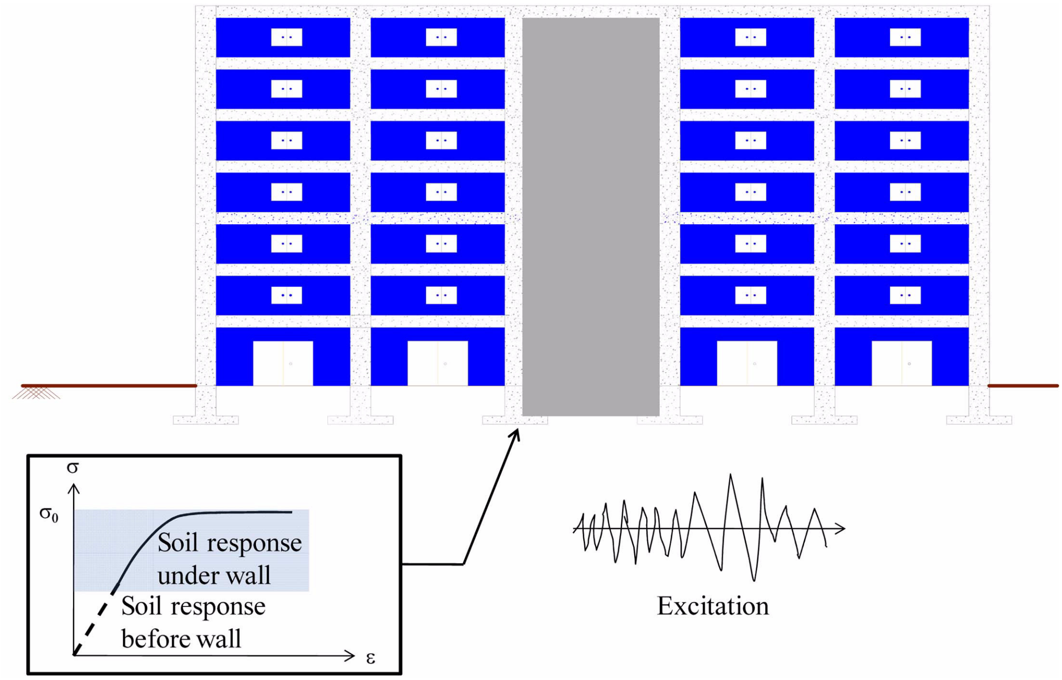


Fig. 1 Building with vulnerable foundation after placement of shear wall

consistent with the stiffness and strength of the columns and thus the bearing capacity of the soil is unlikely to be reached during a major seismic event (capacity protected foundation). But after placement of the shear walls and in absence of major modifications to the footings, the foundations under the wall may become vulnerable because they cannot fully transfer the large moment demands imposed by the stiff and strong slender members (Smith-Pardo 2008). Once it is determined that foundations are vulnerable, the critical issue is shifted to determining how much rotation the footings can sustain before losing their ability to carry load. It is proposed with this manuscript, therefore, that the definition of performance levels in terms of displacements/rotations should exist for soil-foundations in current standards as it does for actual structural members and connections.

In order to evaluate the implication of having a limited foundation rotational capacity, two case studies are presented of wall-frame structures with vulnerable shallow foundations under strong ground motions. Rotational capacities of the foundations under the walls are hypothetically assumed to be equal to those measured by the author in a previous experimental program of footing models on fine and well-graded gravel under combined axial load and moment (Smith-Pardo 2004).

2. Foundation rocking parameters

Four parameters are used in this paper to describe the rocking behavior of shallow foundations in a simplified manner that is consistent with a preliminary design/evaluation process. The intent of this paper is not so much to precisely predict the foundation response but to evaluate the implication of having a limited rotational capacity and the effect of foundation yielding and/or uplifting on the overall seismic response of a structure. More rigorous soil-foundation models using beam-on-nonlinear-Winkler-foundations have been presented elsewhere (Gajan *et al*. 2008, 2010, Harden and Hutchinson 2009, Raychowdhury 2008). In addition, the dynamic response of footings has been described by using macro models that capture nonlinear material and uplift response at the soil-foundation interface (Cremer *et al*. 2001, Grange *et al*. 2009, Chatzigogos *et al*. 2009). Macro models consist of joint elements in global coordinates and variables (forces and displacements) located at the base of columns and walls, which can be directly incorporated in nonlinear finite element models of the entire soil-foundation-structure system. Nonlinear Winkler-based and macro model formulations are still too complex to implement and thus not readily available to engineering practitioners working in a consulting company under a tight design schedule. The proposed model parameters have practical significance as they relate to physical properties widely known to engineers, namely strength, stiffness, softening, and deformation capacity- but applied to the case of a soil-foundation system.

2.1 Foundation moment capacity

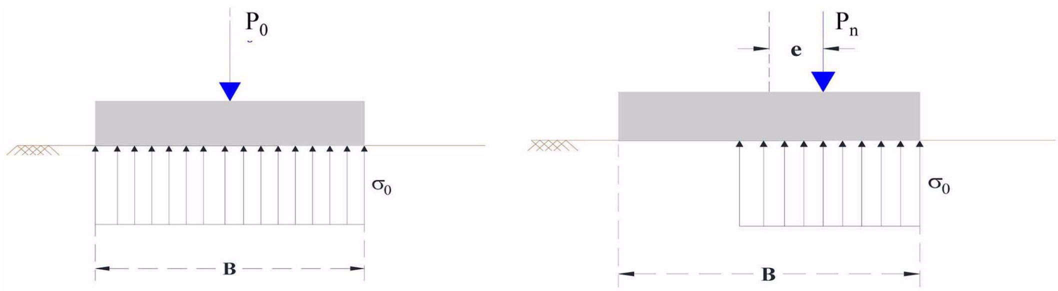
The eccentric load capacity, P_n, of an isolated shallow foundation can be estimated using the equivalent width concept proposed by Meyerhof (1953). The expression Eq. (1) is obtained from equilibrium of the footing assuming that the soil can fully plastify at a stress equal to the bearing capacity under concentric loading, σ_0 (Fig. 2)

$$P_n = P_0\left(\frac{B-2e}{B}\right) \tag{1}$$

where

P_n = bearing capacity under eccentric loading
P_0 = bearing capacity under concentric loading = σ_0A, where A is the foundation horizontal area
B = foundation width (dimension in the direction of the applied moment)
e = load eccentricity

The foundation moment capacity, M_n, for a given axial load can therefore be determined by substituting $e = M_n / P_n$ in Eq. (1) and rearranging



(a) Yielding under concentric loading (b) Yielding under eccentric loading

Fig. 2 Concentric and eccentric bearing capacity of foundations

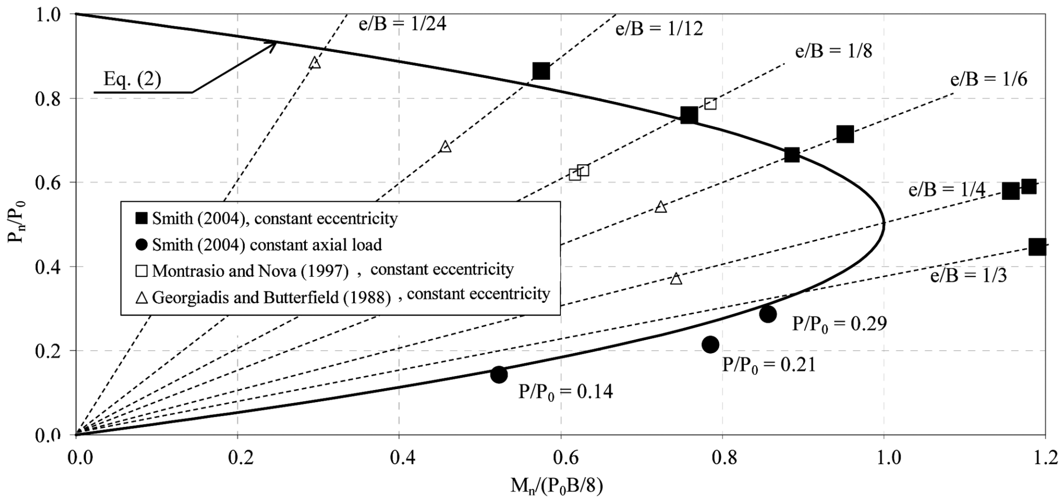


Fig. 3 Moment capacity of foundation models

$$M_n = P_n \frac{B}{2}\left(1 - \frac{P_n}{P_0}\right) \tag{2}$$

This expression is plotted in Fig. 3 in terms of normalized variables together with the experimental results of footings on sand under combined axial load and moment reported by several researchers (Montrasio and Nova 1997, Georgiadis and Butterfield 1998, Smith-Pardo and Bobet 2007). It is apparent that regardless of the loading condition (constant axial load or constant eccentricity) Eq. (2) provides reasonable estimates of the moment capacity of foundation models.

2.2 Foundation rotational stiffness

For small settlements it is customary to assume that the distribution of contact stresses under a rigid footing is linear. If uplifting is also precluded, it can be shown that the rotation of a foundation under combined axial load, P, and moment, M, is given by

$$\theta = \frac{M}{K_{s0} I} \tag{3}$$

where

K_{s0} = initial (tangent) subgrade modulus
I = moment of inertia of the foundation = $LB^3 / 12$, with L: foundation length

The initial subgrade modulus, K_{s0}, can be obtained from the initial slope of the mean stress-vs.-normalized settlement response from tests of foundations/plates under concentric loading. The normalization of the settlement serves to alleviate size effects as shown by Briaud and Gibbens (1994, 1999), Consoli *et al.* (1998), Smith-Pardo and Bobet (2007). It is often convenient to fit the measured response to a hyperbolic function such as

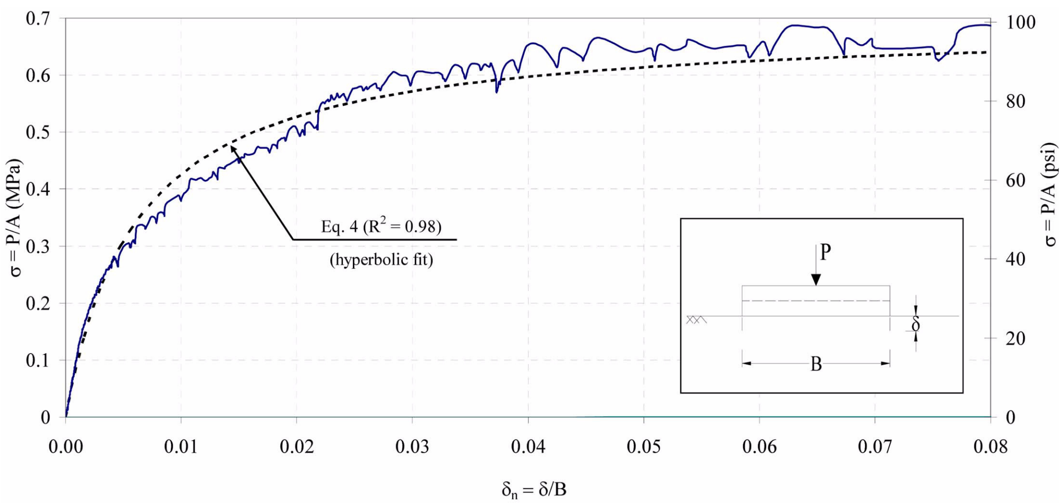


Fig. 4 Response of foundation model under concentric loading

$$\sigma = \frac{a\delta_n}{1+b\delta_n} \tag{4}$$

where

δ_n = normalized settlement = δ/B
δ = settlement
σ = mean stress for concentrically loaded plate/foundation

As an example, the measured response of a 0.3 m (12-inch) square concentrically loaded foundation model tested by Smith-Pardo (2004) was fitted to a hyperbolic function (Fig. 4), which produced $a = 110$ MPa (16 ksi) and $b = 160$. The initial subgrade modulus, K_{s0}, can therefore be estimated, in this particular case from

$$K_{s0}B = \left.\frac{\partial\sigma}{\partial\delta_n}\right|_0 = a = 110\ MPa \tag{5}$$

2.3 Uplifting moment

Assuming a linear contact stress distribution under a rigid footing, the moment corresponding to the onset of uplifting, M_1, is given by

$$M_1 = \frac{PB}{6} \tag{6}$$

The equation is valid only if the maximum contact stress under the footing is less than σ_0. It can be proven that this occurs when the applied axial load, P, is equal or less than half the bearing capacity load under concentric loading, P_0. In reference to Fig. 3, this means that the portion of the axial load-moment interaction diagram for which $P_n / P_0 > 1/2$ corresponds to the condition of reaching the bearing capacity of the supporting soil, σ_0, prior to foundation uplifting (compression-

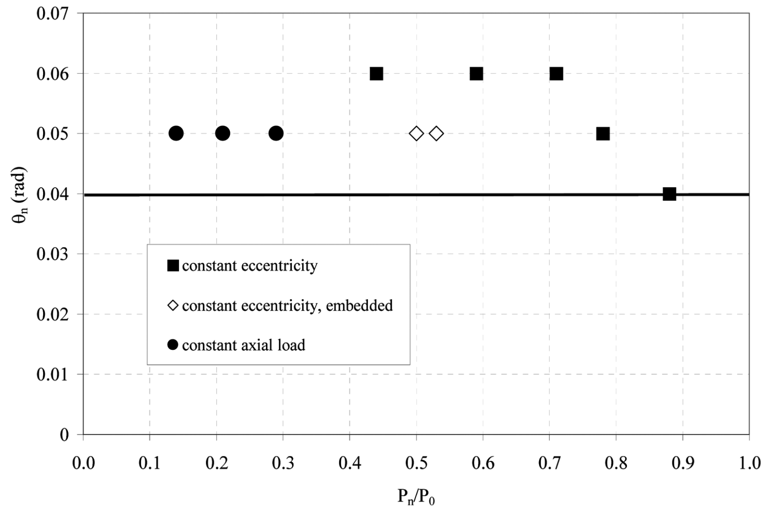


Fig. 5 Rotational capacity of foundation models

controlled foundation response), whereas the portion of the interaction diagram for which $P_n / P_0 < 1/2$ corresponds to foundation uplifting occurring prior to reaching the soil bearing capacity (uplifting-controlled response). Because of this, $P_n / P_0 = 1/2$ can be considered as a balance point.

2.4 Rotational capacity

Realistic estimates of the rotational capacity of foundations need to be based on experimental results as closed-form solutions may be difficult to obtain from basic principles. Unfortunately, test results reported in the literature mostly focus on studying the stiffness and bearing capacity and thus the amount of experimental information on the deformation capacity is almost nonexistent. Fig. 5 shows the measured foundation rotational capacity obtained in the experimental program conducted by the Smith-Pardo (2004) for footings under combined pseudo static axial load and moment. For these tests, the measured rotational capacity was defined as that for which a further increase in the induced rotation required a decrease in the applied moment. It is interesting to notice from Fig. 5 that foundation rotational capacity tends to decrease with the applied axial load at ultimate, P_n, and that the minimum value measured for this series of tests is $\theta_n = 0.04$ rad.

3. Simplified foundation rocking models

Using the parameters mentioned above, proposed upper bound and lower bound models describing the nonlinear behavior of vulnerable foundations are as shown in Fig. 6. The first model accounts for yielding of the foundation (full plastification of the soil) alone, whereas the second model accounts for uplifting and the subsequence softening of the moment rotation response. In the first model (yielding) the response is assumed elastoplatic with the linear component given by Eq. (3) and the plateau defined by Eq. (2). In the second model (softening) the moment-rotation

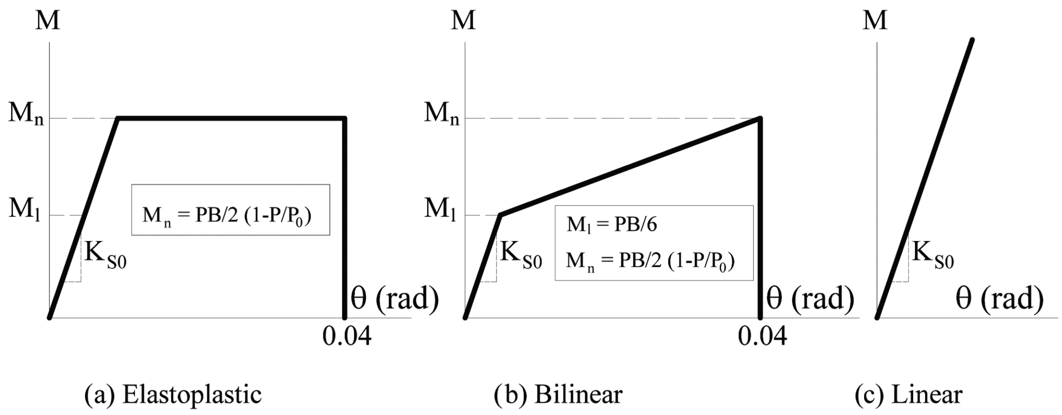


Fig. 6 Alternative foundation models

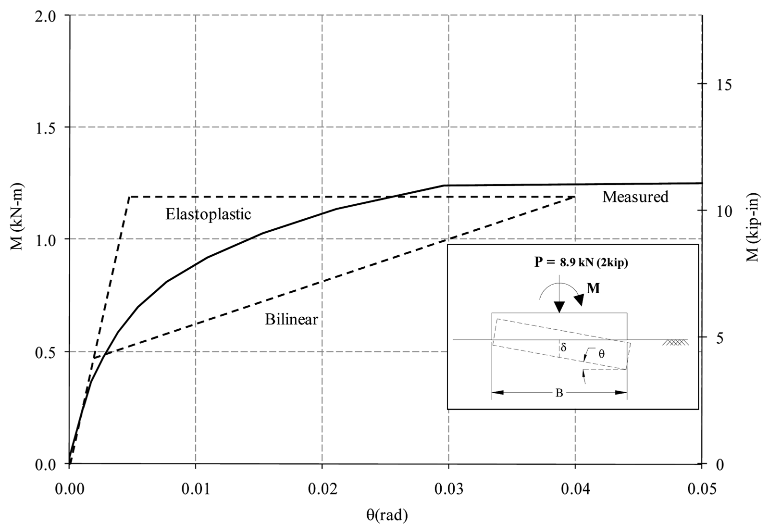


Fig. 7 Moment rotation foundation response and simplified models ($P = 8.9$ kN)

response of the foundation is assumed bilinear with the first segment also defined by Eq. (3) up to the uplifting moment given by Eq. (6). The subsequent linear relation in the second model is defined by the uplifting moment and a point whose coordinates are the rotational capacity, θ_n, and the foundation moment capacity defined by Eq. (2). It is assumed that the cyclic response of the foundation models is also elastoplastic or bilinear, as justified by the stable hysteresis moment-rotation response recorded in various centrifuge tests on dense granular soils (Gajan *et al*. 2008, Algie *et al*. 2010)

In both simplified models, foundation failure –defined as the loss of the ability to carry moment- is assumed to occur when the rotation of the foundation equals θ_n. For the particular case of the foundation models tested by Smith-Pardo (2004), a rotational capacity $\theta_n = 0.04$ rad can be considered conservative.

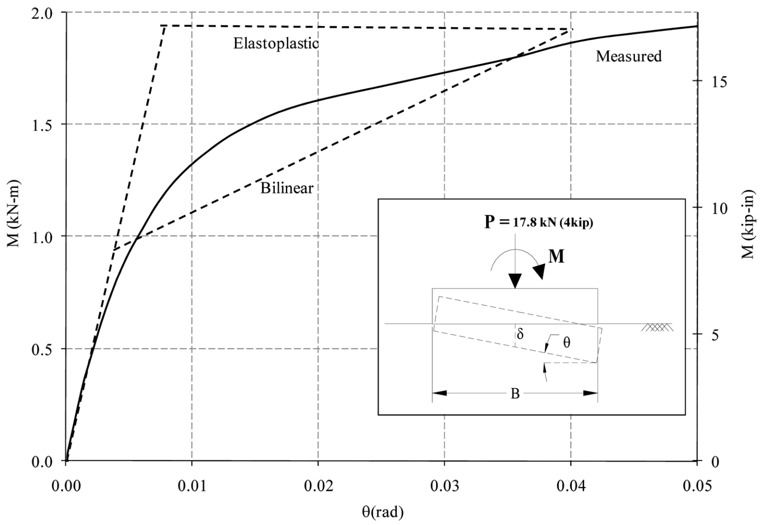


Fig. 8 Moment rotation foundation response and simplified models ($P = 17.8$ kN)

Figs. 7 and 8 show the simplified foundation models in relation to measured moment-rotation responses for two 0.3 m (12-inch) square footings on fine and well-graded gravel (Smith-Pardo 2004). Notice that the required input parameters for the simplified models are only the slope and asymptote from a plate load test result (Fig. 4); that is the subgrade mdulus times the foundation/plate size $K_{s0}B = 110$ MPa (Eq. 5) and the bearing capacity under concentric loading, $\sigma_0 = 0.65$ MPa (Fig. 4), in addition to the footing size ($B = L = 0.3$ m). As expected, none of the models by itself provide a precise estimate of the measured response; however, they provide reasonable lower and upper bounds.

4. Analysis of two buildings with vulnerable foundations

Nonlinear time history drift responses were calculated for the numerical models of two reinforced concrete structures subjected to unidirectional seismic excitation. It was hypothetically assumed that the supporting soil for the buildings is identical to that in the experimental program of footing models under axial load and moment conducted by Smith-Pardo (2004). Therefore in reference to Fig. 5, the rotational capacity of the foundations is conservatively assumed equal to $\theta_n = 0.04$ rad. Additional foundation characterization parameters include a bearing-capacity stress, σ_0, equal to 0.65 MPa (Fig. 4), and an initial subgrade modulus calculated, in reference to Eq. (5), as

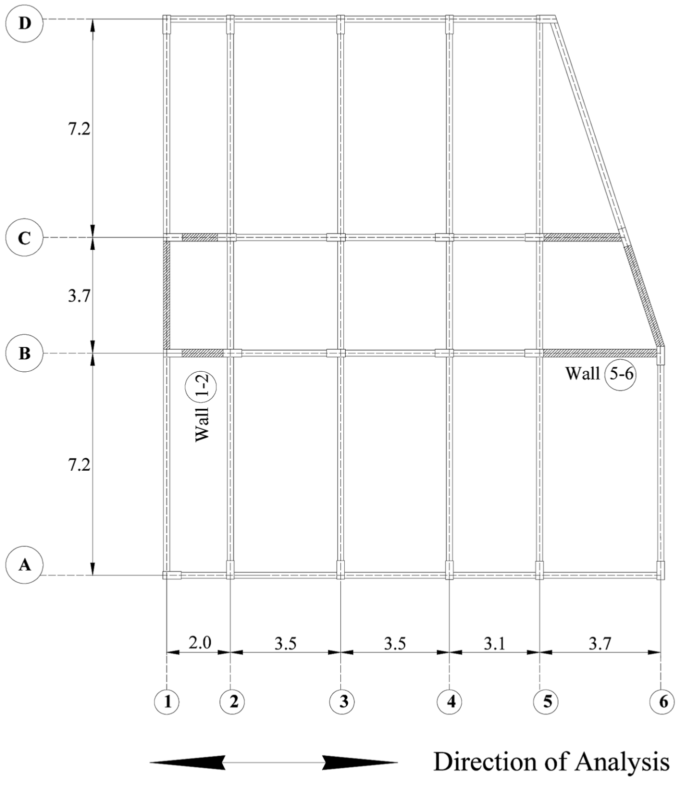
$$K_{s0}B = (K_{s0}B)_{\text{plate load test}} = 110 \text{ MPa} \tag{7}$$

Three alternative moment-rotation models were used to represent the rocking response of the foundations (Fig. 6); the elastoplastic and bilinear models described in the previous session –which separately account for soil yielding and uplifting- and a linear model that serves as a reference for conventional soil-structure analyses in which the foundations are considered non-vulnerable.

Table 1 Characteristics of selected ground motions

Event Name	Recording Station	Date	PGA_1 (g)	PGV_2 m/sec (ft/s)
Imperial Valley	Bonds Corner (USA)	15 Oct 1979	0.78	0.46 (1.5)
Duzce	Duzce (Turkey)	12 Nov 1999	0.50	0.89 (2.9)
Kobe	Takatori (Japan)	16 Jan 1995	0.70	0.86 (2.8)
Loma Prieta	16 LGPC (USA)	18 Oct 1989	0.56	0.94 (3.1)
San Fernando	Pacoima Dam (USA)	9 Feb 1971	1.17	1.17 (3.8)

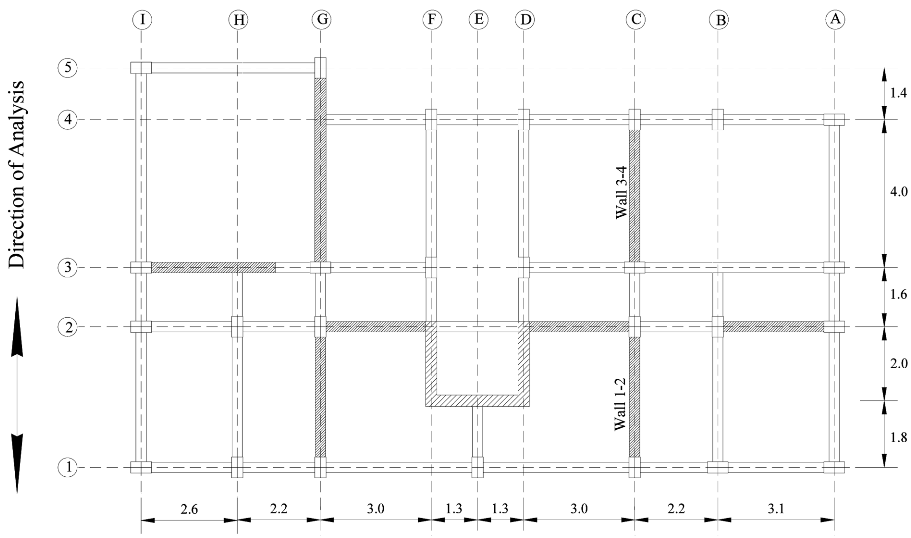
1. PGA: Peak Ground Acceleration
2. PGV: Peak Ground Velocity



Note: all dimensions in meters. (1 m = 3.28 ft)

Fig. 9 Plan view of four-story building

The case study structures, described in more detail by Gur (1999), correspond to actual four-story and eight-story moment frames in Turkey that were strengthened through reinforced concrete shear walls. The numerical models of the two structures were subjected to five strong ground motion records whose characteristics are listed in Table 1. The records comprise a wide range of peak ground velocities [0.46 to 1.17 m/s (1.5 to 3.8 ft/s)] , peak ground accelerations (0.50 to 1.17 g, where g is the acceleration of gravity), and sites (United States, Japan, and Turkey). For each



Note: all dimensions in meters. (1 m = 3.28 ft)

Fig. 10 Plan view of eight-story building

building and for each ground motion, four independent nonlinear time-history analyses were performed in correspondence with the three alternative foundation models described above and a fixed based condition (in which the foundation subgrade modulus, K_{s0}, becomes infinite).

The general structural floor plans of the example structures are indicated in Figs. 9 and 10. The typical floor area of the four-story building is 310 m^2 (3340 ft^2) and the story height is 3.8 m (12.5 ft) for the first floor and 3.5 m (11.5 ft) for the remaining floors. Cross sections are 0.25 × 0.60 m (10 × 24 in.) for columns and 0.25 × 0.70 m (10 × 28 in.) for beams, while reinforced concrete walls are 0.25 m-thick (10 in.). The typical floor area of the eight-story building is 227 m^2 (2440 ft^2) and the story height is 3.0 m (10 ft). Columns are rectangular with dimensions of 0.25 × 0.60 m (10 × 24 in.), beams are 0.20 × 0.60 m (8 × 24 in.), and reinforced concrete shear walls are 0.25 m-thick (10 in.).

Material properties for the structures are based on information provided by Gur (1999). The compressive strength of concrete was equal to 12 MPa (1,700 psi) and the corresponding modulus of elasticity was calculated as 16,000 MPa (2,300 ksi) following section 8.5.1 of ACI 318-08. The specified yielding stress of the reinforcing steel was 220 MPa (32 ksi). The amount of longitudinal reinforcement in columns and walls was assumed equal to 1.0 and 0.2 percent of the respective cross-sectional area.

A total story weight, including live load, equal to 1.0 ton/m^2 (200 lbs/ft^2) and a minimum factor of safety of four against exceeding the bearing capacity of the soil [σ_o = 0.65 MPa (94 psi)] were used to estimate the foundation sizes since actual dimensions were not known. The moment capacity and the rotational stiffness of the foundations were calculated using Eqs. (2) and (3). Moment capacities for structural members, M_{ns}, were determined using conventional strain compatibility and a limiting concrete compressive strain equal to 0.003. Columns, beams and walls were assumed to exhibit an elastoplastic moment-curvature response with the linear portion given by the product of the modulus of elasticity of concrete and the cracked moment of inertia of the section.

Table 2 Calculated model parameters for four-story building

Frame[1]	Axis[1]	Column	Foundation			
		M_{ns} kN-m (k-ft)[2]	BxL, mxm (ft×ft)	$K_\theta = K_{s0}I$ kN-m (k-ft)	M_n kN-m (k-ft)	M_l kN-m (k-ft)
A	1	110 (81)	1.5×1.5 (4.9×4.9)	31E3 (23E3)	74 (54)	27 (20)
	2	50 (37)	1.5×1.5 (4.9×4.9)	31E3 (23E3)	160 (118)	63 (46)
	3	60 (44)	2.0×2.0 (6.6×6.6)	73E3 (54E3)	300 (220)	120 (88)
	4	60 (44)	2.0×2.0 (6.6×6.6)	73E3 (54E3)	300 (220)	120 (88)
	5	60 (44)	2.0×2.0 (6.6×6.6)	73E3 (54E3)	300 (220)	120 (88)
	6	45 (33)	1.5×1.5 (4.9×4.9)	31E3 (23E3)	110 (81)	43 (32)
B	3	180 (133)	2.5×2.5 (8.2×8.2)	140E3 (103E3)	700 (526)	280 (206)
	4	180 (133)	2.5×2.5 (8.2×8.2)	140E3 (103E3)	700 (516)	280 (206)
B (walls)	1-2	1800 (1330)	3.2×1.5 (10.5×4.9)	140E3 (103E3)	1000 (737)	410 (302)
	5-6	3500 (2580)	5.9×2.0 (19.3×6.6)	640E3 (470E3)	2000 (1470)	770 (567)
Girders:		$M_n^+ = 150$ kN-m (110 k-ft)		$M_n^- = 200$ kN-m (147 k-ft)		

1: Refer to Fig. 9 for gridline notation
2: Column/wall moment capacity

Tables 2 and 3 summarize the parameters used to define the numerical models of the four-story and the eight-story structures in the direction of analysis. The calculated moment capacities of reinforced concrete walls were 70 to 80 percent higher than those of the corresponding foundations. This implies that foundation uplifting and yielding of the supporting soil may occur before yielding at the base of walls; i.e., the case study buildings have vulnerable foundations under the walls.

The numerical model of each building under the simulated ground motions was carried out using the computer program Drain-2Dx® (Prakash *et al*. 1993). It was assumed that the floor slabs provide full diaphragm action, and that proper reinforcement detailing precludes the occurrence of brittle modes of failures in shear or anchorage loss. In addition, because of the approximately symmetrical configuration of the structures, only frames A and B were considered in the analyses of the four-story building, whereas only frames A through D were considered in the numerical model for the eight-story building. The structures were modeled as two-dimensional wire frames connected with axially rigid links at each floor, as depicted in Figs. 11 and 12. Rigid offsets at the ends of the beams were defined to account for the width of columns and walls. It must be noticed that the use

Table 3 Calculated model parameters for eight-story building

Frame[1]	Axis[1]	Column	Foundation			
		M_{ns} kN-m (k-ft)[2]	BxL mxm (ft×ft)	$K_\theta = K_{s0}I$ kN-m (k-ft)	M_n kN-m (k-ft)	M_l kN-m (k-ft)
A	1	55 (41)	1.5×1.5 (4.9×4.9)	31E3 (23E3)	120 (88)	45 (33)
	2	60 (44)	1.5×1.5 (4.9×4.9)	73E3 (54E3)	170 (125)	68 (50)
	3	60 (44)	1.5×1.5 (4.9×4.9)	73E3 (54E3)	180 (133)	73 (54)
	4	55 (41)	1.5×1.5 (4.9×4.9)	31E3 (23E3)	140 (103)	53 (39)
B	1	60 (44)	1.5×1.5 (4.9×4.9)	31E3 (23E3)	190 (140)	80 (59)
	2	190 (140)	2.5×2.5 (8.2×8.2)	140E3 (103E3)	800 (590)	330 (243)
	4	170 (125)	2.0×2.0 (6.6×6.6)	73E3 (54E3)	410 (302)	170 (125)
C (walls)	1-2	3500 (2580)	5.8×1.5 (4.9×4.9)	460E3 (339E3)	1900 (1400)	710 (523)
	3-4	3500 (2580)	5.6×1.5 (18.3×4.9)	430E3 (317E3)	2000 (1470)	780 (475)
D	1-2	620 (457)	3.0×1.0 (9.8×3.3)	83E3 (61E3)	600 (442)	270 (199)
	3	170 (125)	2.0×2.0 (6.6×6.6)	73E3 (54E3)	430 (317)	180 (133)
	4	170 (125)	2.0×2.0 (6.6×6.6)	73E3 (54E3)	410 (302)	170 (125)
Girders:		$M_n^+ = 120$ kN-m (88 k-ft)		$M_n^- = 250$ kN-m (184 k-ft)		

1: Refer to Fig. 10 for gridline notation
2: Column/wall moment capacity

of symmetry is only a simplifying assumption that is required given that the proposed foundation models are only for two-dimensional analyses.

5. Results from nonlinear time history analyses

The fundamental period of the four-story building was calculated as 0.42 seconds for the structure with flexible supports (elastoplastic, bilinear, or linear) and 0.26 seconds for the fixed-base structure ($K_{s0} \rightarrow \infty$). For the eight-story building, on the other hand, the period of the flexible base structure was calculated as 0.29 seconds, whereas the period for the fixed-base structure was calculated as

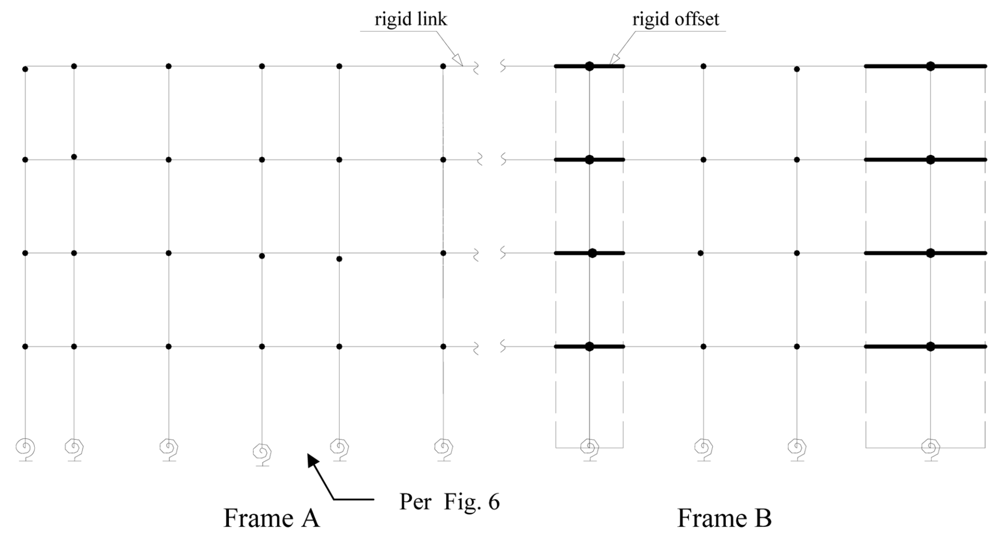


Fig. 11 Wire frame model for four-story building

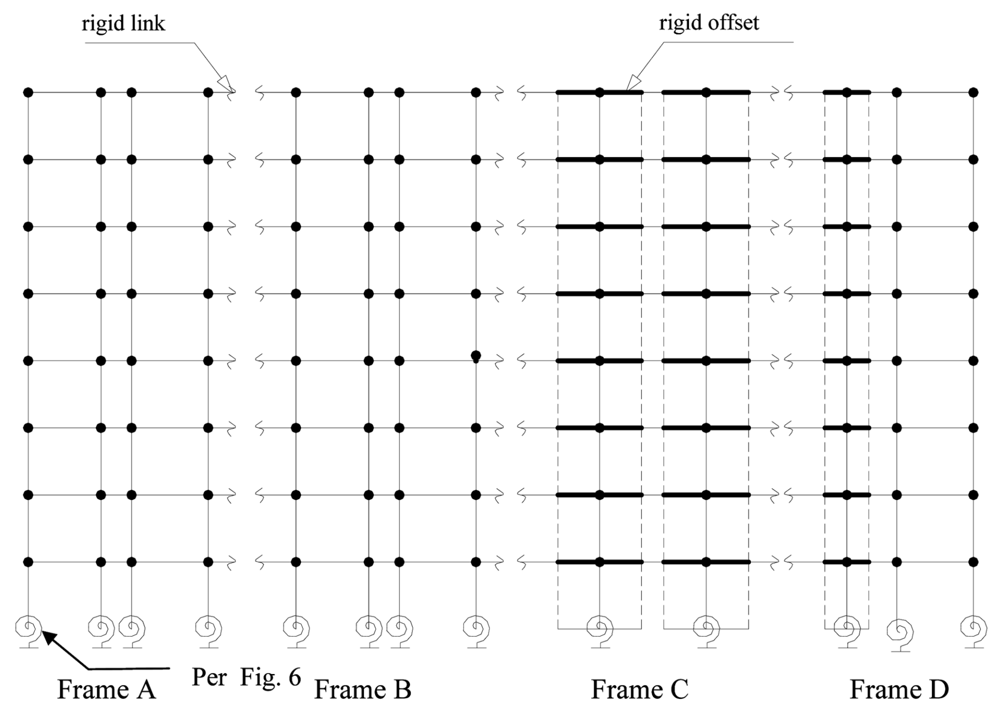


Fig. 12 Wire frame model for eight-story building

0.25 seconds.

Depending on the ground motion and foundation model, calculated maximum interstory drift ratios were 2 percent or less for the 4-story structure and 1.1 percent or less for the 8-story structure

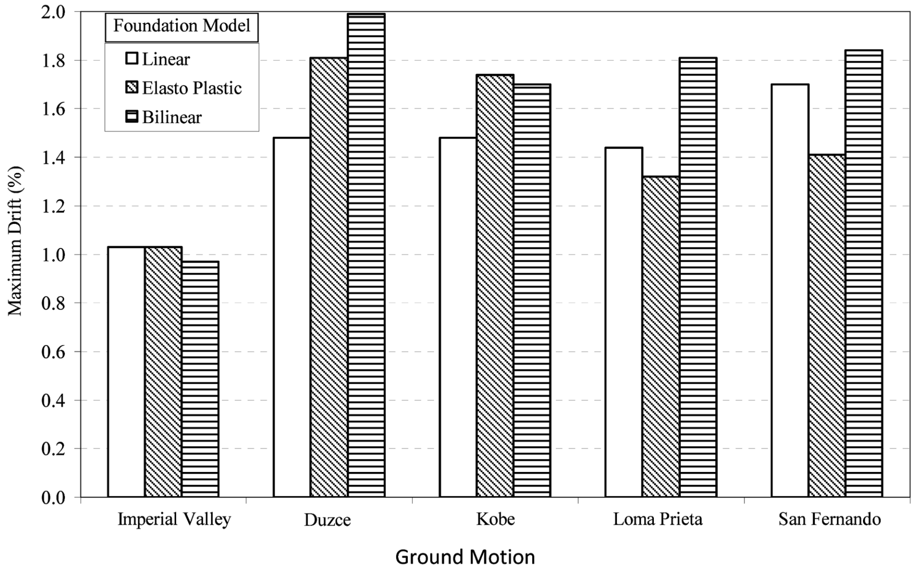


Fig. 13 Calculated maximum interstory drift response for four-story building

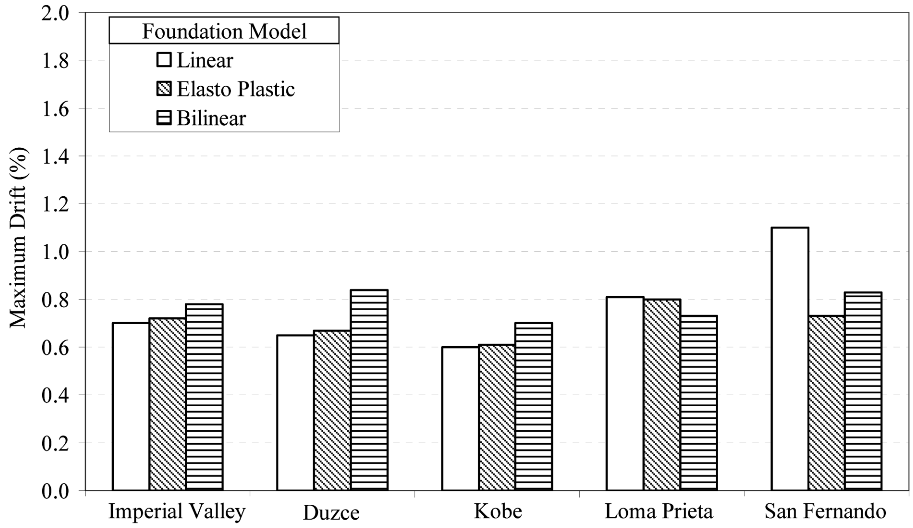


Fig. 14 Calculated maximum drift response for eight-story building

as shown in Figs. 13 and 14. Such levels of interstory drift ratios could cause excessive damage/collapse to nonstructural elements and moderate damage to structural elements, however, it should be realized that the values include rotation of the foundation itself thus the actual story distortion would be less. For both structural models, it is noticed that consideration of foundation yielding (elastoplastic model) does not produce a consistent increase in the maximum calculated drift ratio. Consideration of foundation uplifting and response softening (bilinear model), on the other hand,

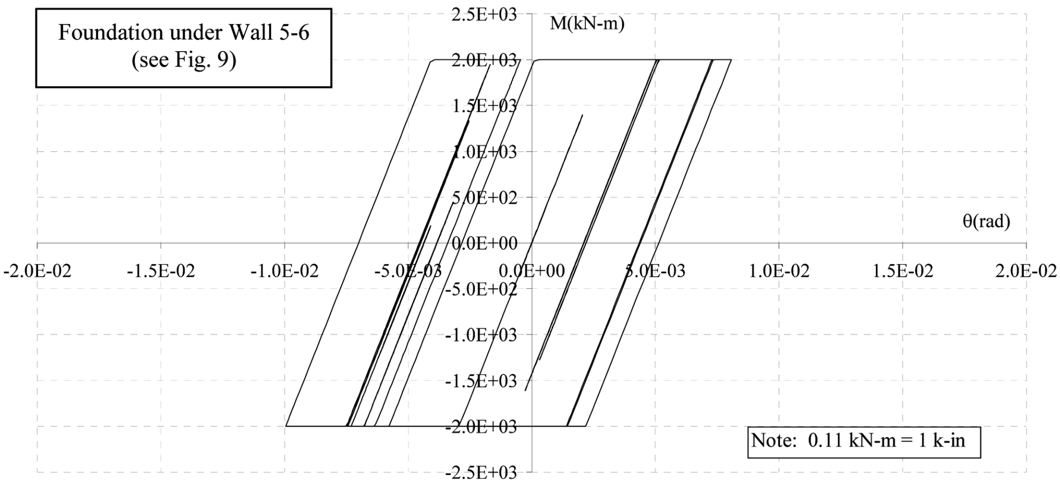


Fig. 15 Calculated wall foundation response for four-story building under Imperial Valley record– elastoplastic model

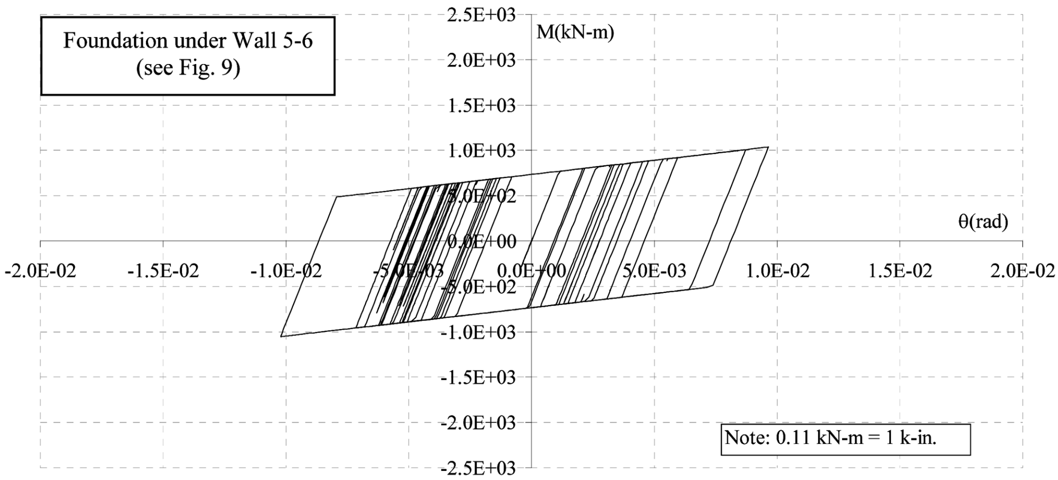


Fig. 16 Calculated wall foundation response for four-story building under Imperial Valley record - bilinear model

produces maximum interstory drifts ratios that are at most 30 percent higher than those calculated with a conventional (linear) foundation model. Thus given that the actual building response is expected to be between that of the elastoplastic and that of the bilinear foundation model, it can be concluded that the presence of vulnerable foundations does not significantly increase the drift demand of the buildings.

For the two case study structures, yielding of the wall foundations occur under all the ground motions listed in Table 1 and thus the evaluation of the rotational demand becomes critical to the seismic performance of the buildings. In fact, the calculated drift responses reported above are valid only if the rotational demand on the foundation does not exceed the capacity of the soil, $\theta_n = 0.04$ rad. Figs. 15 and 16 show the alternative moment-rotation response history of a shear wall

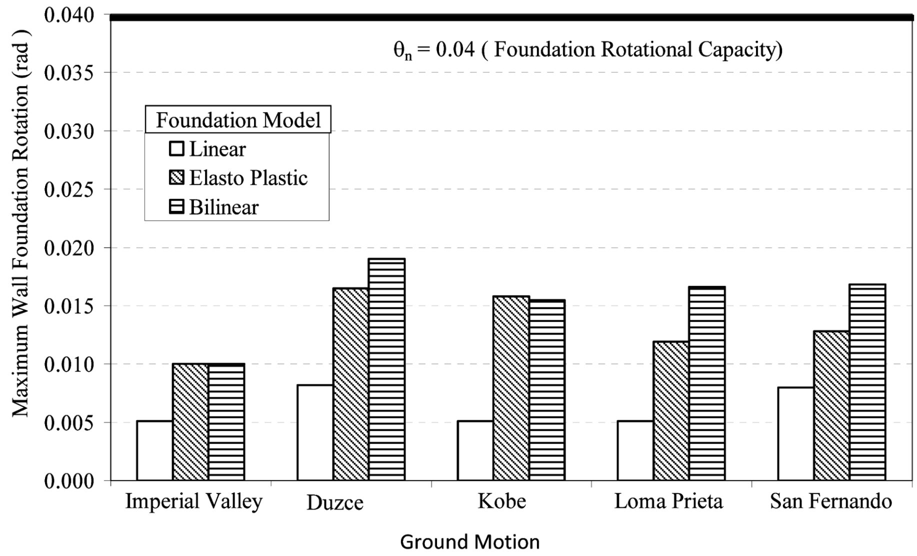


Fig. 17 Calculated maximum wall foundation rotation for four-story building

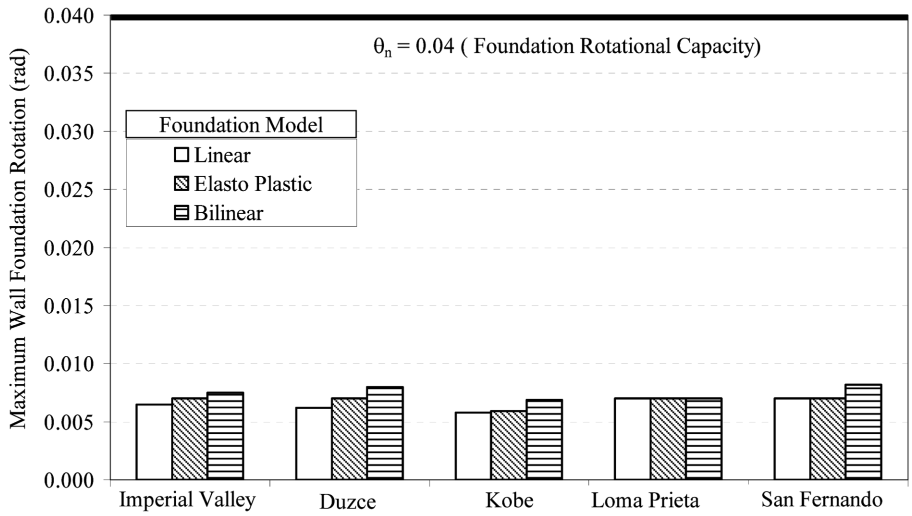


Fig. 18 Calculated maximum wall foundation rotation for eight-story building

foundation for the 4-story structure under the Imperial Valley record. In this case, the maximum rotational demand is about 0.01 rad only regardless of the vulnerable foundation model used.

A summary of the maximum calculated wall foundation rotational demands is shown in Fig. 17 for the 4-story building and in Fig. 18 for the 8-story structure. It is apparent that the minimum rotational capacity measured in the laboratory (whose soil conditions are assumed to apply to the case study structures), $\theta_n = 0.04$ rad., exceeds the calculated rotational demand by a factor of two or more in the case of the 4-story building and by a factor of four or more in the case of the 8-story

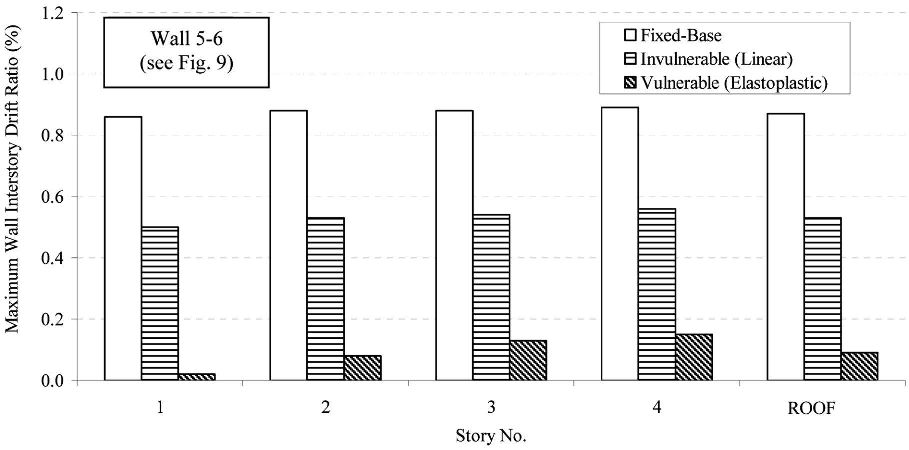


Fig. 19 Calculated maximum wall interstory drift ratio for four-story building subjected to the imperial valley ground motion

building. Therefore, foundation failure (loss of bearing capacity) for the two case structures is unlikely to occur.

Comparing the results from Fig. 13 and Fig. 17 it can be noticed that the calculated maximum rotation demands at the level of the foundation are similar in magnitude to the maximum interstory drift ratios of the 4-story building. For Imperial Valley ground motion, for example, the maximum calculated interstory drift ratio is about 0.95 to 1.05% depending on the vulnerable foundation model used, while the maximum wall foundation rotation under the same ground motion is about 0.01 rad. This is an indication that drift demands are mostly associated to rocking of the foundation and thus yielding of the supporting soil decreases the distortion demands on the shear walls. To further prove the benefit of foundation yielding on the distortion demand on structural elements, Fig. 19 shows the calculated maximum wall interstory drift ratios for three alternate support conditions of the four-story structure subjected to the Imperial Valley ground motion. Wall interstory drift ratio is calculated as the difference between the interstory drift ratio and the rotation of the foundation (Fig. 20). The calculated maximum wall interstory drift ratio demand is 0.9 percent for the model with fixed bases, 0.5 percent for the model with non-vulnerable foundations (linear moment-rotation response), and only 0.15 percent for the model with vulnerable foundations (elastoplastic moment-rotation response).

6. Discussion on permanent deformation of foundations

The condition of vulnerable foundations can be an effective energy-dissipation mechanism to soil-structure system under severe ground motions; however, yielding and uplifting may lead to permanent deformations rotations that are undesirable to the integrity of structural and nonstructural members. Based on centrifugal tests, Gajan *et al.* (2005) concluded that these permanent deformations accumulate with the number of loading cycles, but the rate of accumulation of settlement decreases as the footing embeds itself during shaking. In some respect, vulnerable foundations are similar to elastomeric bearings which can provide flexibility and energy dissipation

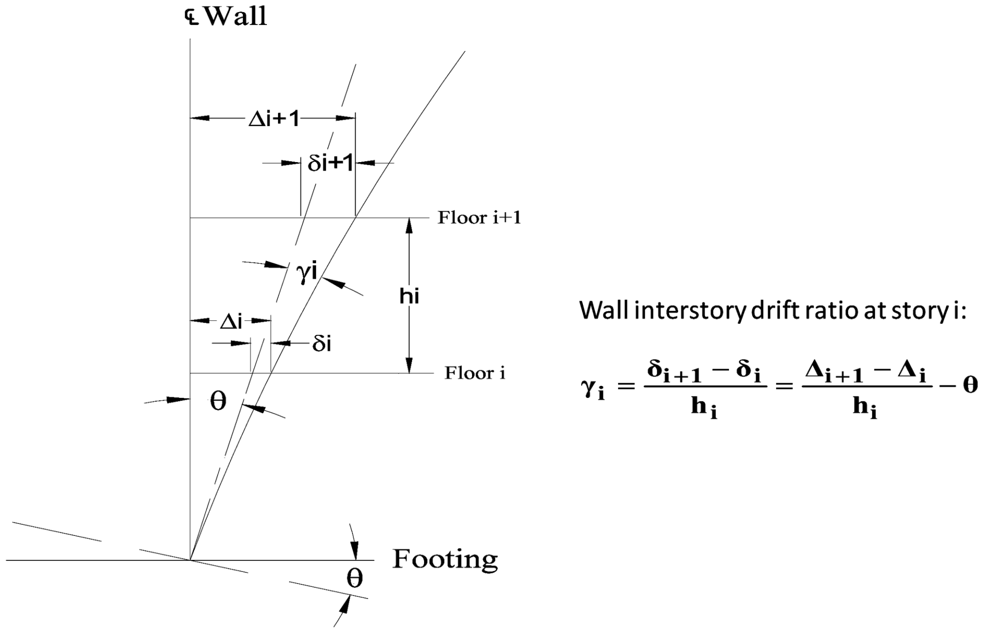


Fig. 20 Wall interstory drift ratio

ability at the base of the buildings but that do not have the centering capability.

A strict determination of the effect of permanent foundation deformations on the seismic response of buildings is beyond the scope of this manuscript. Nevertheless, in an attempt to investigate such effect, the bases of the shear walls in the four-story building with vulnerable foundation were subject to an imposed deformation given by

$$\theta_{\text{permamemt}} = \theta_{\max} - \frac{M_n}{K_{s0} I} \tag{8}$$

where

$\theta_{\text{permanent}}$ = maximum expected permanent wall foundation rotation
$\theta_{\max}$ = maximum calculated wall foundation rotational demand for all the selected ground records
$K_{s0}I$ = linear rotational stiffness of the wall foundation
M_n = moment capacity of the foundation (Eq. (2))

From the time history analyses of the four-story building, $\theta_{\text{permanent}}$ was calculated to be 0.011 rad under the smaller shear wall (wall 1-2 in Fig. 9) and 0.015 rad under the larger shear (wall 5-6).

Fig. 21 depicts the deflected shape of the structure when the foundations under the walls are subject to a nonlinear static application of $\theta_{\text{permanent}}$ and no other loading exists in the structure. It is observed that in this particular case the imposed permanent deformation induce a very modest mean interstory drift ratio (0.16%) and only minor flexural hinging on beams framing into the first story shear walls thus indicating that permanent deformation alone, as imposed by compatibility requirements, do not significantly compromise the structural integrity of the building.

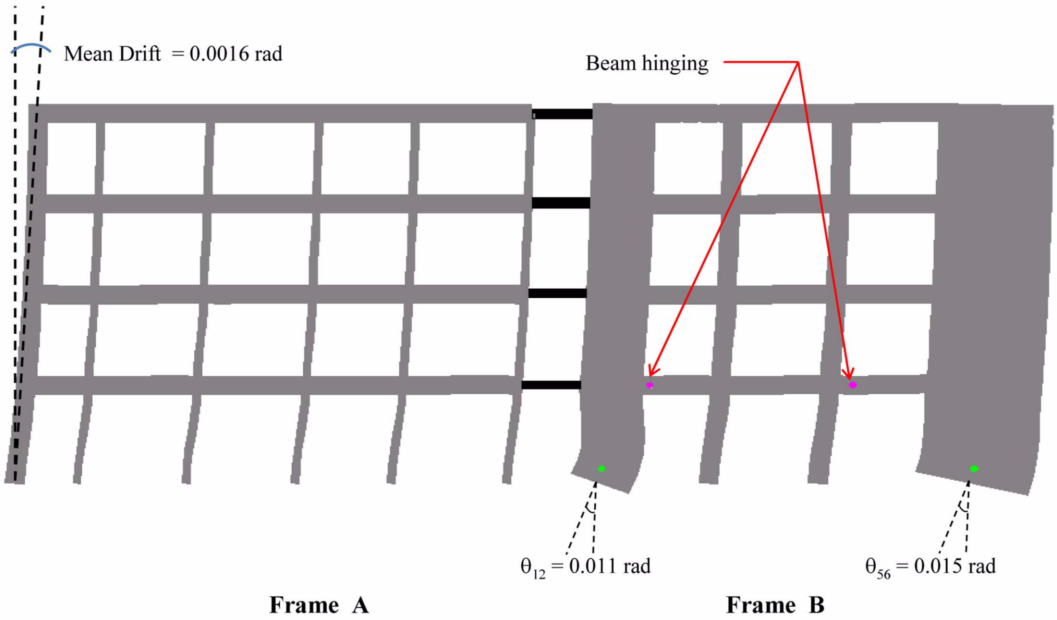


Fig. 21 Deflected shape of the four-story building subject to permanent foundation rotations

7. Conclusions

Based on the results from the analyses of two structures subject to five selected ground motions, it is concluded that reaching either the moment corresponding to foundation uplift or the moment capacity of the wall-foundation does not significantly increase the calculated maximum interstory drift response. Because the calculated maximum rotation demands were less than half of the minimum rotational capacity obtained in an experimental program, it can be concluded that the condition of vulnerable foundation may not be detrimental to the performance of a wall-frame building subjected to strong ground motions because the supporting soil can become a source of energy dissipation that protects the structural integrity of walls.

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