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SEISMICALLY INDUCED TILTING POTENTIAL OF SHALLOW MAT FOUNDATIONS

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ABSTRACT

Widespread displacements of shallow mat foundations resting on saturated fine soils occurred in Mexico City during 1985 Mexico Earthquake and in Adapazari during 1999 Kocaeli Earthquake. Soft surface soils, shallow ground water, limited foundation embedment and deep alluvial deposits were the common features pertaining to such foundation displacements in either case. Experience shows that, while tilting is particularly problematic, uniform foundation settlement, even when excessive, does not generally limit the post-earthquake serviceability of building structures.

In this study a simplified methodology is developed in order to estimate the seismically induced irrecoverable tilting potential of shallow mats on the saturated fine soils. Seismic response of the nonlinear soil-structure system is reduced to a single degree of freedom oscillator with elastic-perfectly plastic behaviour. Pseudo-static yield acceleration, which initiates the foundation bearing capacity failure when applied to the structural mass, can be estimated through utilizing analytical bearing capacity expressions or numerical analyses. The pseudo-static yield acceleration, besides other factors, is also related to the state of soil consolidation before seismic loading and the factor of safety against bearing capacity failure under static loading conditions. The irrecoverable displacement demand on the single degree of freedom oscillator is determined by inelastic transient response analyses of the oscillator. Predictive capability of the methodology is tested with actual case data from Mexico City and Adapazari.

Keywords: Bearing Failure, Foundation Displacement, Foundation Tilting, Mat Foundations, Seismic Performance.

INTRODUCTION

In association with September 19, 1985 earthquake that hit Mexico City, which is situated over deep alluvial soils, cases of excessive foundation displacements of shallow foundations were reported. Subsoil conditions under the failed foundations were predominantly silty clays with high compressibility (Zeevaert, 1991). The city of Adapazari, a greater part of which is located on a deep alluvial basin in the field of the ruptured North Anatolian Fault, was among the worst affected urban areas during the 17 August 1999 İzmit (Kocaeli), Turkey earthquake (M_w 7.4). A quite remarkable aspect from the geotechnical engineering point of view was the occurrence of numerous cases of displacements in various forms and levels at the foundations of, by and large, three- to six-story reinforced concrete buildings in the city. Based on the post earthquake observations and subsequent studies, the factors that might have contributed to certain extends in those displacements were listed as the variability of induced seismic excitation throughout the city, building height and foundation width, as well as the presence of adjacent buildings, and generally soft surface soils dominated by silt-

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clay mixtures with occasional apparent potential for liquefaction (Bakir et al., 2002, 2005; Gazetas, 2003; Karaca, 2001).

The observed tendency of increase in foundation displacements with increasing building height underlines the impact of inertial effects on the seismic response of shallow mat foundations. Association of the ground failures with shallow foundations and the lack of evidence of soil liquefaction in the free-field point out the seismic bearing capacity failure as a plausible mechanism. Besides, a number of comparable attributes between Mexico City and Adapazari regarding ground conditions, such as the presence of saturated soft deposits of fine surface soils and deep alluvial basins that amplify the long-period components of ground motion appear to be the influential factors, and hence should be considered in the investigation of available cases.

Based on experience, tilting mode of foundation displacements has a greater potential for adverse effect regarding post-earthquake serviceability of buildings structures compared to uniform settlement or lateral sliding. In Adapazari, buildings that experienced excessive tilts during the earthquake are observed to have poor aspect ratios (i.e., relatively greater ratios of building height to foundation width). Although a constraint can be imposed on the aspect ratio to bound the seismically induced tilting within acceptable limits, absence of a viable procedure to assess the potential of foundation tilting inhibits such a practice as of present. Considering the cases with limited foundation displacements, it is possible to obtain estimates of foundation tilting for a given acceleration-history utilizing the macro-element method (Paolucci, 1997; Cremer et al., 2002). Predictive capability of the macro-element approach, however, should be examined through model studies before performing parametric analyses in order to develop practical criteria for foundations.

In this paper a simple methodology is formulated for prediction of the seismically induced irrecoverable tilting demand on shallow mat foundations resting on the saturated fine soils. The methodology is based on an approximation of the nonlinear soil-structure response through equivalent nonlinear single degree of freedom (SDOF) oscillator. Although possible, stiffness or strength degradation of soils under successive load cycles is not considered in the form presented here.

METHODOLOGY

The in-plane response of common multistory buildings resting on shallow rigid mats to vertically incident SV waves is considered in the following formulation. Seismic demand is defined by the horizontal component of free-field motion, denoted as $u_g(t)$. Considering the degree of uncertainties involved, it is appropriate to assume that the building behaves linearly.

Equation of Motion of a Nonlinear SDOF Oscillator

The equation of motion of a single degree of freedom system with nonlinear stiffness is (Chopra, 1995)

$$m\ddot{u} + c\dot{u} + f_s(u,\dot{u}) = -m\ddot{u}_g \tag{1}$$

where, m is the mass, c is the damping constant and $f(u,\dot{u})$ is the recovery force. If the force-deformation behavior of the system is idealized as linearly elastic-perfectly plastic with yield strength f_y, the normalized form of equation of motion can be expressed as

$$\ddot{\mu} + 2\varsigma\,\omega_n\dot{\mu} + \omega_n^2\bar{f}_s(\mu,\dot{\mu}) = -\omega_n^2\,\frac{\ddot{u}_g}{a_y} \tag{2}$$

where, $\omega_n = k / m$ is the natural frequency, k is the elastic stiffness, $\zeta = c / \omega_n m$ is the damping ratio, $a_y = f_y / m$ is the pseudo-static yield acceleration, $\mu = u / u_y$ is the ductility ratio, $u_y = f_y / k$ is the yield displacement, and $\bar{f}_s(\mu, \dot{\mu}) = f_s(u, \dot{u}) / f_y$ is the normalized recovery force.

Several relationships between elastic spectrum and the maximum ductility ratio μ_{max} are reported in literature (Riddel et al., 2002). Through utilization of the elastic design spectrum for a specific site, it is possible to obtain an estimate of μ_{max}, which makes it possible to anticipate the displacement demand, u_{ir}, on a SDOF oscillator with linearly elastic-perfectly plastic behavior, via the relationship

$$u_{ir} = \begin{cases} (\mu_{max} - 1) \cdot \dfrac{a_y T_n^2}{4\pi^2} & \text{if } \mu > 1.0 \\ 0 & \text{if } \mu \leq 1.0 \end{cases} \tag{3}$$

Approximations for the Nonlinear Soil-Structure Response

Idealized inertial soil-structure interaction analysis model, described in Figure 1, is used for the formulation of natural frequency of the system: The lumped mass, m, representing the inertia of the structure, is located at a height h above the shallow foundation; k_h and k_θ are respectively the frequency-dependent horizontal translation and rocking stiffnesses of the foundation impedances, k_s is the structural stiffness ζ_h, ζ_θ, and ζ_s are their corresponding damping ratios. Formulations for foundation impedance are presented by Dobry and Gazetas (1986). Expressing the total displacement of the lumped mass as,

$$u = u_h + u_\theta + u_s \tag{4}$$

the equation of undamped motion of the lumped mass is obtained as,

$$\ddot{u} + \omega_n^2 u = -\ddot{u}_g \tag{5}$$

where, ω_n is the equivalent natural frequency of the linear system, formulated as

$$2\pi/\omega_n = T_n = \sqrt{T_s^2 + T_r^2 + T_h^2} \tag{6}$$

T_n is the natural period of equivalent SDOF oscillator. T_s, T_θ and T_r are expressed as follows:

$$T_s = 2\pi\sqrt{m/k_s} \qquad T_r = 2\pi\sqrt{mh^2/k_\theta} \qquad T_h = 2\pi\sqrt{m/k_h} \tag{7}$$

Hence, T_s is the natural period of fixed-base structure; T_r and T_h represent the contributions of soil deformability to the period elongation beyond T_s, due to the rocking and horizontal modes of foundation displacement, respectively.

Derivation of an expression of equivalent damping ratio is straightforward. However, the resulting frequency dependent form complicates the response computations. The peak response of nonlinear systems is respectively less sensitive to the damping coefficient when compared to linear systems, especially when a considerable part of the input energy is dissipated through hysteretic behaviour in the case of former (Chopra, 1995). In this study, a constant equivalent damping ratio of 5%, which is consistent with the most code based design spectra, is used.

Considering nonlinear behavior of the equivalent SDOF oscillator, the total irrecoverable displacement u_{ir} of the lumped mass, is the sum of the contributions of horizontal translation and tilting modes of irrecoverable foundation displacements. Determination of these distinct modes of displacement requires introduction of a reliable flow rule for plastic deformations. Accordingly, for practicality, it is assumed that only the rotational foundation spring in Figure 1 behaves nonlinearly, and hence, the irrecoverable displacement at lumped-mass level is solely due to the irrecoverable rotation of foundation:

$$u_{ir} \cong h \cdot \theta_{ir} \tag{8}$$

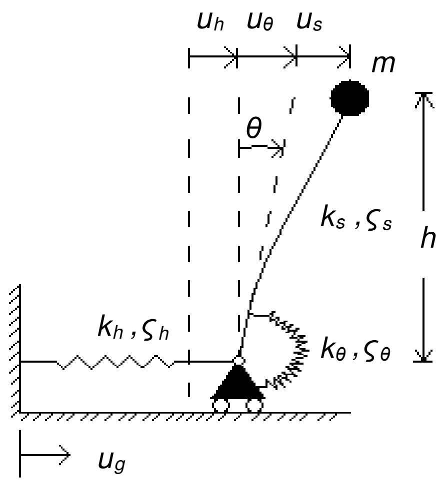


Figure 1. Idealized model for inertial soil-structure interaction analysis

Finally, for the utilization of equation (2), the pseudo-static yield acceleration is formulated as,

$$a_y = \frac{M_y}{mh} \tag{9}$$

The overturning moment capacity of the foundation, M_y, can be determined either by utilization of analytical formulae considering the idealized conditions, or through numerical approaches. The latter is to be preferred especially when soil heterogeneity and complicated stress paths limit the use of the former.

Assessment of M_y

Considering the seismic response of buildings with poor (i.e., large) aspect ratios, only the load eccentricity is accounted for estimation of M_y. Normally consolidated or lightly over-consolidated saturated soil deposits underlying foundations point out that the consolidated-undrained stress path is applicable during seismic loading. For soils with sufficient permeability to allow completion of consolidation during the construction period, or for existing foundations that have already completed their consolidation, the effective confining stresses beneath the foundations will be different from those on free field. Thus, the analyses for M_y should consider the heterogeneity in the undrained shear strength in the proximity of a foundation, as well as the heterogeneity imposed by soil stratification.

In this study, the finite element method was utilized for estimation of M_y. Plaxis v7.2, which is a finite element code particularly developed for plane-strain geotechnical analyses, was used. The basic principles followed in the analyses are based on the approach proposed by Taiebat and Carter (2002), with some minor modifications. The details of the finite element mesh employed for analyses are presented by Yilmaz (2004). A brief summary of the numerical procedure is provided in the following.

Soil medium and foundation were modeled by 15-node triangular elements and linear beam elements, respectively. Stiffness of the beam elements was set to a relatively large value in order to simulate a rigid foundation behavior. Elasto-plastic material model with Mohr-Coulomb yield criterion was assigned to represent the soil behavior. Foundation uplift capability, required for a realistic investigation of the seismic response, was provided through inclusion of a separate strip region located immediately beneath the foundation and it was set to drained condition with tension cut-off at zero effective stress. The thickness of the strip was selected as 1/200 of the foundation width, so that the bearing capacity of the foundation practically remained unaffected.

The computed overturning moment capacity of the foundation, M_y, was normalized by the vertical load V and foundation width B. Results were expressed as a function of factor of safety FS against bearing capacity failure, formulated as,

$$FS = \frac{V_{ult}}{V} \tag{10}$$

where V_{ult} is the ultimate bearing capacity of the foundation. Thus, an initial set of analyses to determine V_{ult}, for both the undrained (purely cohesive) and drained (cohesionless) stress paths, was performed. V_{ult} obtained through the drained stress path is also employed for the analyses regarding the consolidated-undrained stress path. V_{ult} was compared with the theoretical bearing capacity equations for verification. For the case of soil with undrained shear strength S_u, the value of V_{ult}/S_uB was computed as 5.12, which is consistent with the theoretical value of (π+2) calculated by Prandtl's formula. For the drained soil case with effective angle of internal friction ϕ' equal to 30°, and a dilatancy angle ψ equal to ϕ' (i.e., associative flow rule), the computed bearing capacity factor, N_γ, is 15.3, which is consistent with the value proposed by Hansen (1969). However, the associative flow rule implies unrealistically high values of the dilatancy angle when the behavior of loose cohesionless soils is considered. So, an additional analysis was performed to compute V_{ult} considering nonassociative flow rule ($\psi = 0$), and resulted in N_γ=10.9.

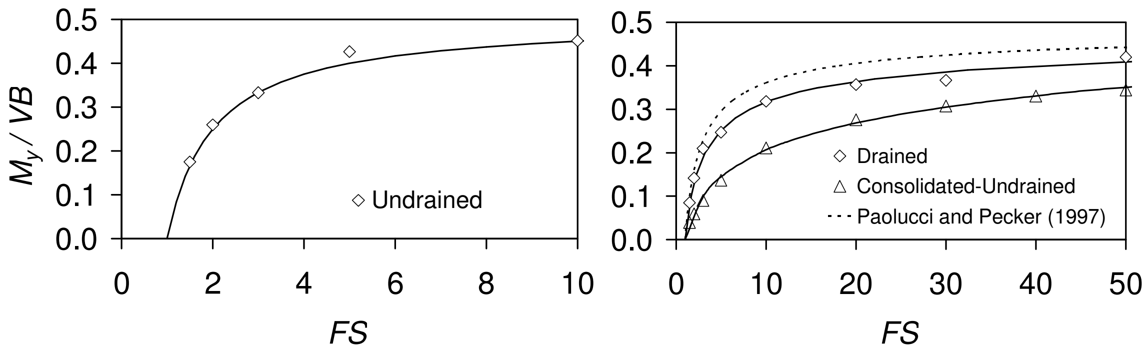
Each analysis aiming at determination of M_y for a given FS was composed of three computation steps: in the first step, the free-field stress distribution was computed. In the second step, the vertical load on stiff surficial mat was applied in the form of uniformly distributed pressure. The magnitude of this pressure was set in accordance with the target FS. In the final step, overturning moment was applied on the foundation through a linear vertical pressure distribution. The load representing the overturning moment was increased incrementally until the bearing failure is imminent. For the case of consolidated-undrained response, saturated cohesionless soil was set to undrained behavior during the third step. The pore pressure parameter A for an ideally elastic stress path is equal to 1/3. The value of ϕ' was set to 30°, and the value of ψ was set to zero for the cohesionless soils. The results were observed to be practically valid for values of ϕ' close to 30°, since an increase in ϕ' is observed to result in consistent change for both M_y and V_{ult}, and presumed to be representative for the behavior of loose cohesionless soils sustaining excessive shear strains. Analyses results are presented in Figure 2, and compared with the curves obtained by least-squares curve fitting to the results. Equations of the best-fit curves for each of the different stress-paths are the following:

Undrained:
$$FS = \left(1 - 2\frac{M_y}{VB} \right)^{-1} \tag{11.a}$$

Drained: $$FS = \left(1 - \frac{2 \cdot M_y}{VB}\right)^{-2.3} \quad (11.b)$$

Consolidated-undrained: $$FS = 266^{\left(2M_y / VB\right)} \quad (11.c)$$

Equation 11.a is consistent with the formulation proposed by Houlsby and Purzin (1999). On the other hand, equation 11.b provides slightly lower values for M_y/VB compared to those calculated by the seismic bearing capacity expression provided by Paolucci and Pecker (1997), such that the factors relevant to load inclination and soil inertia are ignored for the consistency of expressions. Analyses with the associative flow rule, however, provide consistent results. Thus, the finite element approach can be used to determine M_y for cases consisting of significant deviations from idealizations required for theoretical formulations, such as soil stratification or existence of adjacent buildings. Finally, a comparison of equation 11.c and 11.b reveals that a significant reduction in M_y occurs due to undrained behavior of saturated cohesionless soils, especially for the low *FS* range (Figure 2.b). This result also explains why seismic bearing capacity failures are more likely at sites with shallow ground water, rather than sites with unsaturated soils.



(a) (b)

Figure 2. Comparison of finite element analyses results with equations (a) 11.a, and (b) 11.b-c

Once M_y/VB is determined, the pseudo-static yield acceleration, which is the last parameter necessary for the utilization of equation 2, is calculated by the equation

$$\frac{a_y}{g} = \left(\frac{M_y}{VB}\right) \Big/ \left(\frac{h}{B}\right) \quad (12)$$

Equation 12 is obtained by substituting $V=mg$, where g is the gravitational acceleration, into the moment equilibrium equation derived for the simple system shown in Figure 1.

CASE STUDIES

Predictive capability of the presented methodology is investigated through utilization of available cases from Mexico City and Adapazari. However, recognizing the significant uncertainty involved in the definition of seismic demand and selection of relevant parameters, the attempt is to be considered as a preliminary investigation.

Adapazari Cases

Case data pertaining to the buildings with foundation displacements from Adapazari is provided by Karaca (2001), including the relevant CPT and SPT data. All cases are re-evaluated and supplementary borings were drilled where deemed necessary (Yilmaz, 2004).

Seismic Demand

Due to a malfunction the only lateral component recorded at Sakarya (Adapazari) station was EW during the 17 August earthquake, which is almost fault-parallel. The station is situated over stiff shallow soils in the south of the city, where ground conditions are not representative of the deep alluvium sites over which foundation failures occurred. In order to estimate the levels of seismic demand at a typical deep alluvium site in the city, one dimensional response analysis was performed assigning the EW record as the outcrop motion. Selection of geotechnical properties of the 150 m deep soil profile is based on the study by Bakir et al. (2002), in which the aftershock acceleration histories simultaneously recorded at the stiff and alluvium sites were used to determine the transfer function. A comparison between the spectral accelerations of the Adapazari record and the calculated surface motion on deep alluvium is presented in Figure 3.

Based on the study by Akkar et al. (2005) conducted on a RC building stock with comparable attributes to those in Adapazari, the equation T_n(s)=0.1N, where N is the number of stories, was presumed to provide a reasonable estimate of the fundamental periods. However, significant scatter exists in the periods for a given story number, which introduces further uncertainty in the calculations. For calculation of k_θ and k_h, considering the strain levels during seismic loading, the representative average shear wave velocity of shallow soils was estimated as 70 m/s. The total mass of a building is calculated considering a distributed mass of 1000 kg/m^2 per storey, and is lumped at two thirds of the building height. Thus, the estimated natural periods of the 4 to 6-story buildings, including soil-structure interaction, were around 0.8 s.

Irrecoverable displacement demand, u_{ir}, on a nonlinear SDOF oscillator was calculated for natural periods 0.6, 0.9, and 1.2 s (Figure 4). For a range of periods of about 0.8 s, the magnitude of u_{ir} is practically observed to be insensitive to T_n. The equation for the representative irrecoverable displacement demand obtained from a least-squares analysis is,

$$u_{ir} = \frac{0.75}{1300^{a_y/g}} \tag{13}$$

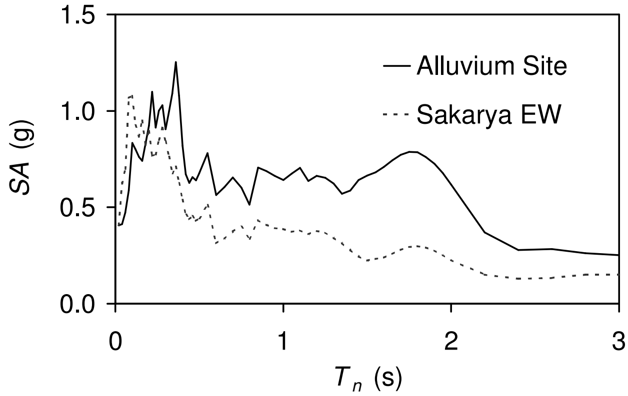


Figure 3. Response spectra of the Adapazari EW record (stiff site) and the simulated ground motion (deep alluvium site)

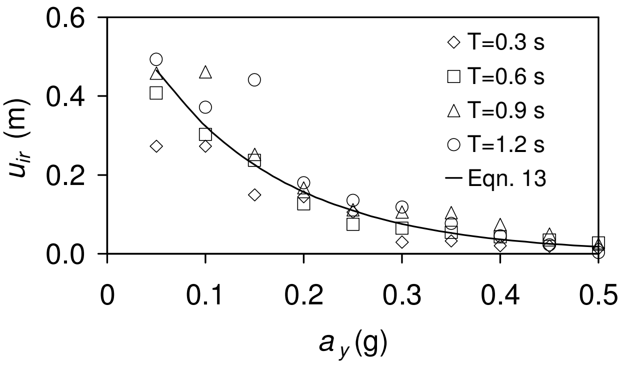


Figure 4. Relationship between u_{ir} and a_y for simulated motion

Estimation of θ_{ir}

Substituting equations 8, 11.c and 12 into equation 13 results in the following relationship between θ_{ir} and *FS*:

$$\theta_{ir} = \frac{0.75}{h \cdot 1300^{\left(\frac{B}{2h} \cdot \frac{\log FS}{\log 266}\right)}} \tag{14}$$

where θ_{ir} is in radians.

The *FS* was calculated by utilizing the conventional bearing capacity formula for strip foundations resting on cohesionless soils. In the calculations, both the weight of foundation and the surcharge due to embedment were neglected. Buoyant unit weight of soil was assumed to be 8 kN/m^3, and the N_γ factor is chosen as 10, considering soils with $\phi' \cong 30°$ and $\psi \cong 0$. Utilizing equation 14 for each case, predicted values of θ_{ir} were compared with those observed, in Figure 5. The measured angles of tilt of the foundations were truncated to the nearest 0.5° multiple, due to the uncertainties involved in field measurements. In the plot, the observed tilting angle (y-axis) is limited with 5°, including those of toppled buildings.

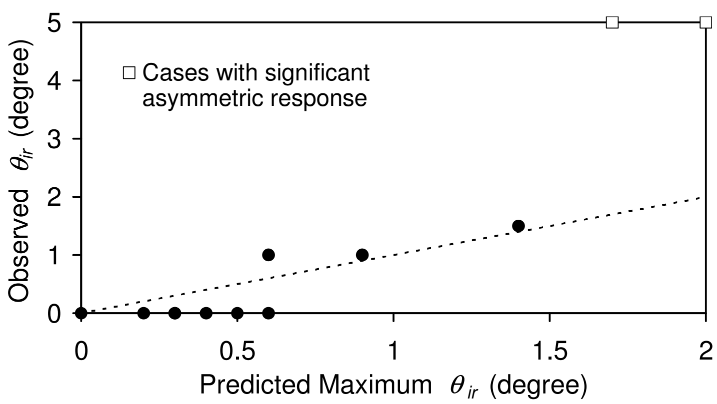


Figure 5. Comparison of the predicted and measured tilts for Adapazari cases

Predicted maximum irrecoverable tilts are seen to be consistent with the observations up to 1.5°. The two cases that suffered excessive foundation tilting (i.e., in excess of 5˚) were likely to be affected by the existence of adjacent comparable-size buildings.

Mexico City Cases

Documented cases of foundation displacements from Mexico City are presented by Mendoza and Auvinet (1988), and Auvinet et al. (1996). Four cases with shallow foundations consist of geotechnical information and foundation performance data, two of which have shallow mats (cases Ia and Ib) and the two others with compensated foundations (cases II and III). The fundamental periods for these mid-rise reinforced concrete buildings are in the range of 1-2 seconds, with very significant scatter (Gómez et al., 1989). The seismic demand at heavily damaged zones of Mexico City is somewhat chaotic, and extremely sensitive to small variations in the shear-wave velocity and thickness of the soft clay deposit on the surface. The SCT record is considered to be representative of the surface motion at the heavily damaged zone of the city (Seed et al, 1988).

Observed tilts are compared with those predicted in Figure 7 in terms of u_{ir}, calculated by utilizing equation 8, and SCT-EW record. Since the fundamental periods of the buildings are not precisely known, the irrecoverable displacement demand for a range of periods between 0.8 and 2.4 seconds are computed. Estimated foundation tilts for cases II and III are reasonably consistent with the observed values, whereas the reported tilting for cases Ia and Ib are considerably greater than the estimates. The latter two cases have relatively low a_y values (about 0.1g). The sensitivity effect due to such low pseudo-static yield acceleration levels is discussed in the following. Besides, these two buildings were adjacent; consequently, their seismic response could have been further influenced by asymmetric response.

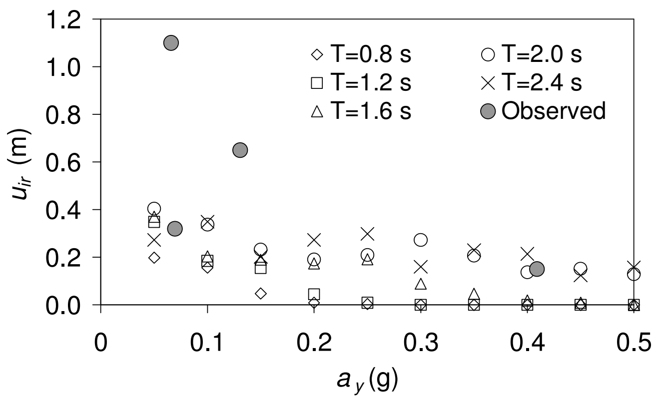


Figure 7. Comparison of the observed and predicted tilts of Mexico City cases

The Impact of Asymmetry on Response

A possible source of uncertainty involved in the estimation of irrecoverable tilting demand on foundations is the existence of an adjacent building as it was often the case in Adapazari. The neighboring building can introduce strong asymmetry in bearing strength in one direction and alter the seismic demand due to the interaction between the two foundations. The significance of this issue for the seismic response of toppled buildings was also discussed by Gazetas et al. (2003). Additionally, the soil heterogeneity is another possible source of asymmetric response for foundations.

In order to investigate the sensitivity of inelastic response to strength asymmetry, the yield strength of linearly elastic-perfectly plastic SDOF oscillator was doubled in one direction. Utilizing the synthetic record for Adapazari, the inelastic displacement demand on asymmetrically and symmetrically behaving ideally elastic-perfectly plastic SDOF oscillators is contrasted in Figure 8.a. Although significant amplification in inelastic displacement demand, in the order of 2 to 3 folds within the period range of interest, is observed for a_y within the range 0.1g to 0.3g, the amplification diminishes for greater values of a_y. This observation clearly shows that the cases with poor aspect ratios are more sensitive to the asymmetrical foundation response. On the other hand, sensitivity of the inelastic displacement demand to strength asymmetry for buildings in Adapazari is considerably lower than those in Mexico City, in general, as shown in Figure 8.b.

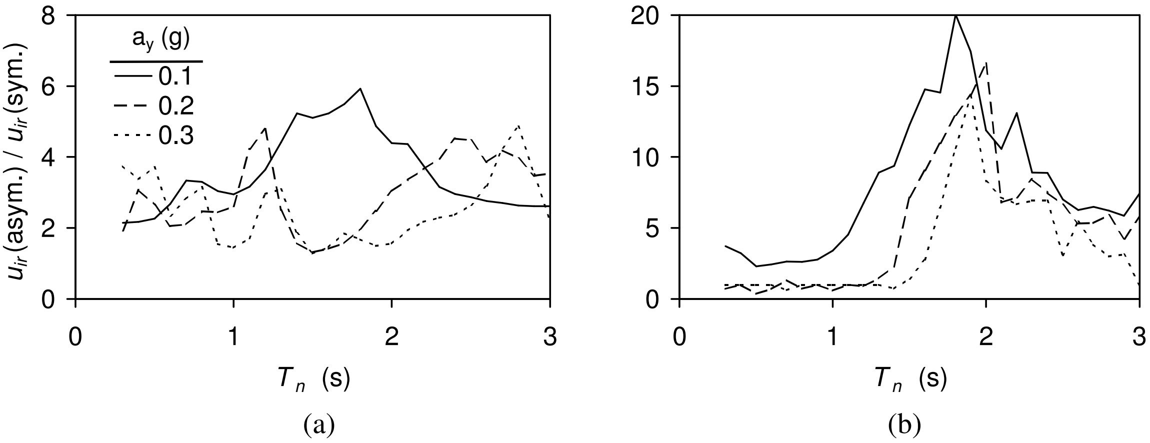


Figure 8. Amplification of u_{ir} due to asymmetric response of elasto-plastic oscillator, utilizing (a) synthetic Adapazari, (b) SCT-EW record

CONCLUSIONS

Seismically induced irrecoverable tilting of shallow mat foundations on saturated fine soils, such as those occurred in Adapazari and Mexico City, can be explained by a simple model of elasto-plastic SDOF oscillator. Tilting demand on a mat foundation due to a seismic excitation can also be reasonably estimated by the same model through calculated response. Utilizing an elastic spectrum, the empirical relationships can be used to estimate the inelastic displacement demand on an elasto-plastic SDOF oscillator. The natural period of the oscillator can be estimated through simple soil-structure interaction relationships; whereas the yield strength of the oscillator can be estimated by computing the overturning moment capacity of foundations employing numerical methods (e.g. the finite element method) so as to provide more realistic simulations of the stress paths followed prior to and during seismic loading. The consolidated-undrained stress path representative of the saturated cohesionless soils under seismic excitation can result in significant reductions in the foundation overturning moment capacity when compared to drained stress path.

Sensitivity to asymmetric response, uncertainties related to the geotechnical characteristics, as well as the seismic demand at investigated sites limit the further refinement of the methodology as of present. Future studies, including numerical and experimental phases, are planned for verification and modification of the proposed methodology.

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