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PUSHOVER AND INELASTIC-SEISMIC RESPONSE OF SHALLOW FOUNDATIONS SUPPORTING A SLENDER STRUCTURE

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ABSTRACT

The interaction between a surface foundation and the supporting inelastic soil under the action of monotonic, cyclic, and seismic loading is studied numerically. The foundation supports an elastic tall system, the horizontal loading of which induces primarily an overturning moment and secondarily a shear force. Starting from linear elastic behavior, the footing eventually uplifts from the soil, provoking strong inelastic soil response culminating in development of a bearing–capacity failure mechanism and progressive settlement. The substantial lateral displacement of the pier mass induces an additional aggravating moment due to P–δ effect. The paper outlines the moment–rotation–settlement relations under monotonic loading at the mass center, under cyclic loading, and under seismic excitation at the base.

THE PROBLEM AND THE KEY INVESTIGATED PARAMETERS

With the advent of performance–based design in structural earthquake engineering, the need has arisen for extending it to earthquake geotechnics. This calls for determining the complete inelastic response of the foundation-soil system (in the form of force–displacement or moment-rotation relation) to progressively increasing loads until collapse.

To this end, the paper investigates the response of a 2m–wide foundation supporting a 5m–high mass (Fig. 1) which undergoes, first, monotonic and cyclic lateral displacements, and is then subjected to seismic base excitation. Under progressively increasing loads the foundation uplifts from the ground (geometric nonlinearity) and failure mechanisms develop in the soil (material inelasticity). The interplay between these two mechanisms, affected by the unavoidable P–Δ effects, is governed primarily by the following factors:

- The vertical foundation load N in comparison with the ultimate vertical capacity N_u, expressed through the ratio $\chi = N/N_u$;
- The distance, R, of the mass centre of gravity from the base edge (Fig.1) ;
- The slenderness ratio h/b ;
- The intensity, frequency content and sequence of pulses of the seismic excitation ;
- The vibrational characteristics (natural period) of the structure.

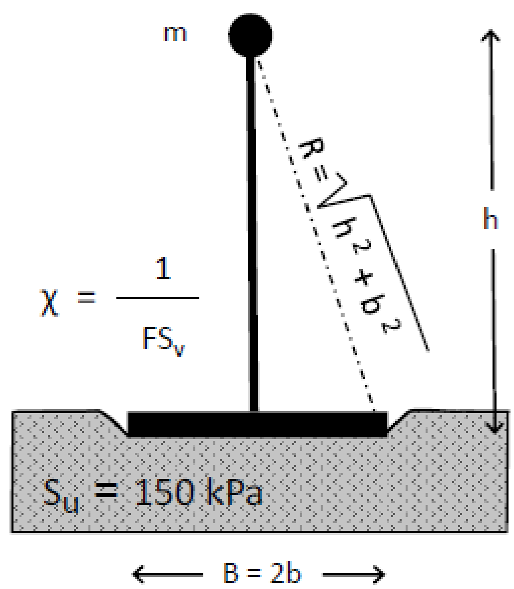

Fig. 1. Problem geometry

METHOD OF ANALYSIS

A series of two dimensional finite element analyses are performed using Abaqus for a single–degree–of–freedom oscillator on a foundation allowing uplift. The soil is saturated

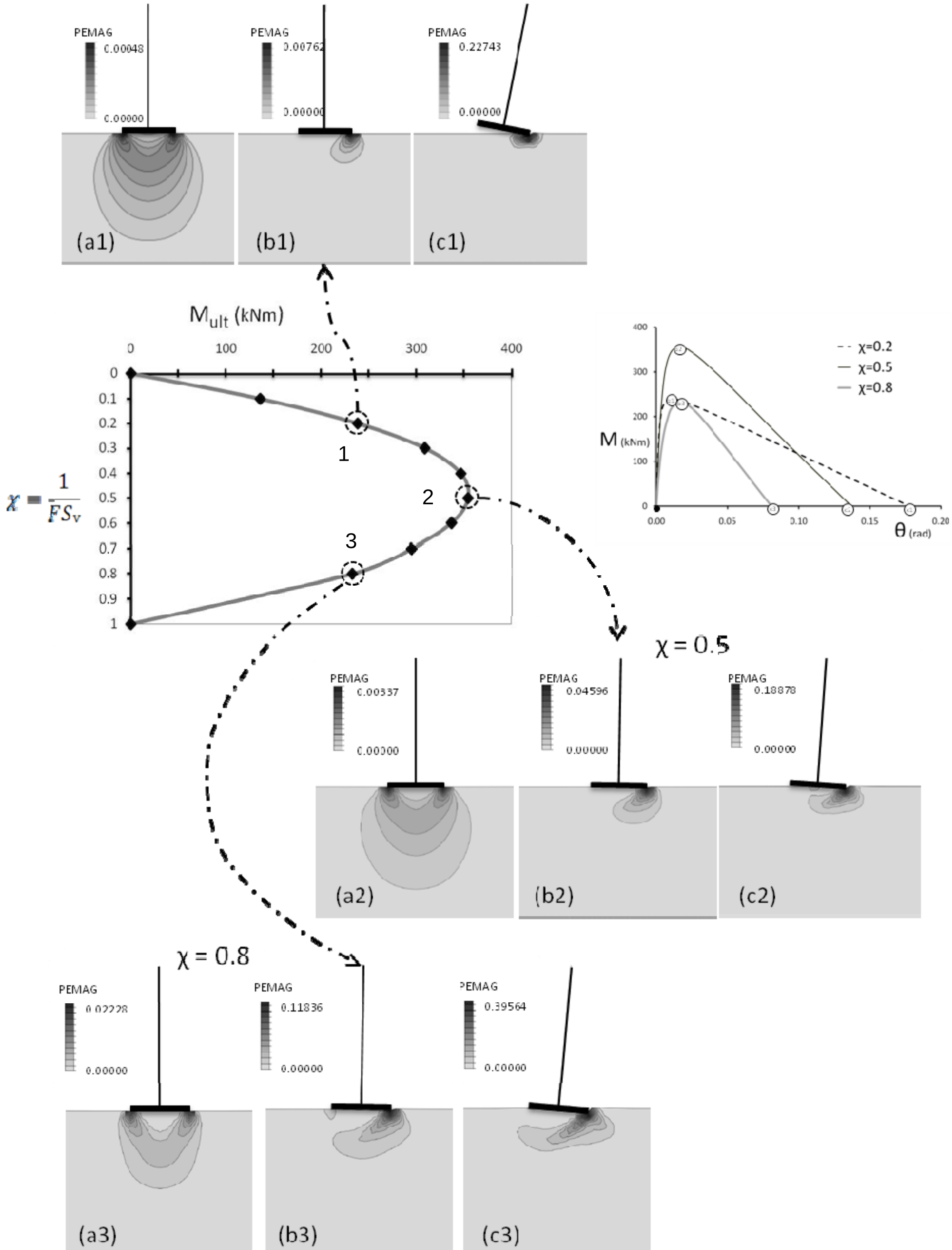

Fig. 2. Dependence of the moment capacity of the foundation on the static safety factor FS_v ($M - \chi$ diagram). For the three cases (1,2,3), corresponding to values of χ 0.2, 0.5, 0.8, we display three snapshots of the deformed system along with the contours of plastic deformation. Specifically a1,a2,a3 present the initial (static) state; b1,b2,b3 present the states at the peak moment, M_{ult} = 238, 354, 232 kNm, respectively; c1, c2, c3 show the states of imminent collapse. Also shown are the $M - \theta$ pushover curves for the three χ values.

stiff clay responding in undrained fashion with Su = 150 kPa. The bedrock is placed at a depth of 5 m below the foundation level. The mass element, which allows the introduction of lumped mass at a point, is located 5 m above the foundation level and is connected to the footing with linear elastic beam elements. The modulus of elasticity of the beam is selected such as to achieve either a rigid structure, or a structure, with a given fixed-base natural period. The footing is also modeled with linear elastic beam elements of rectangular section, with modulus of elasticity large enough to achieve structural rigidity.

The soil is modeled with continuum solid plane–strain 4-noded bilinear elements. An advanced contact algorithm has been adopted to incorporate potential uplifting of the foundation. Gap elements allow for the nodes to be in contact (gap closed) or separated (gap open). To achieve a reasonable stable time increment without jeopardizing the accuracy of the analysis, we modified the default hard contact pressure-overclosure relationship with a suitable exponential relationship. Finally, we used a significantly large coefficient of friction at the soil–footing interface to prevent sliding of the footing.

The elastoplastic soil behavior is described with Von Mises yield surfaces having nonlinear kinematic hardening and associative plastic flow rules. The model of Abaqus is calibrated using the methodology proposed by Gerolymos et al. (2005). It is worth noting that the soil plasticity begins at1/10 of its maximum yield stress, while P-Δ effects are computed during all steps of the analysis.

RESULTS : STATIC PUSHOVER ANALYSIS

For the static pushover analysis a horizontal displacement is applied on the mass center of the superstructure. The moment–rotation diagrams for the various χ factors (χ = the inverse of the static vertical safety factor FS_V) are portrayed in Fig. 2, along with the M–N interaction diagram (To be precise M – N/Nu.). As expected from the literature, the maximum value of moment capacity is reached for a static safety factor of about $FS_V \approx 2$ (i.e., $\chi \approx 0.5$) [Allotey & Naggar, 2003 ; Apostolou & Gazetas, 2005 ; Chatzigogos et al 2009, Gajan & Kutter, 2008]. The value of the critical rotation ,θc , is always lower than the one for a 1-dof rigid oscillator rocking on a rigid base :

$$\theta_c = \arctan {}^{b}\!/_{h}$$

This is due to soil compliance : as the safety factor diminishes (χ increases) the critical rotation before failure becomes smaller and smaller.

Examining the settlement–rotation curves (Fig.3), we may also observe that indeed the case of $FS_v = 2$ ($\chi = 0.5$) is in the middle between two different modes of response. Structures with $\chi < 0.5$ undergo predominantly uplifting, while with $\chi > 0.5$ they suffer mostly plastic deformation.

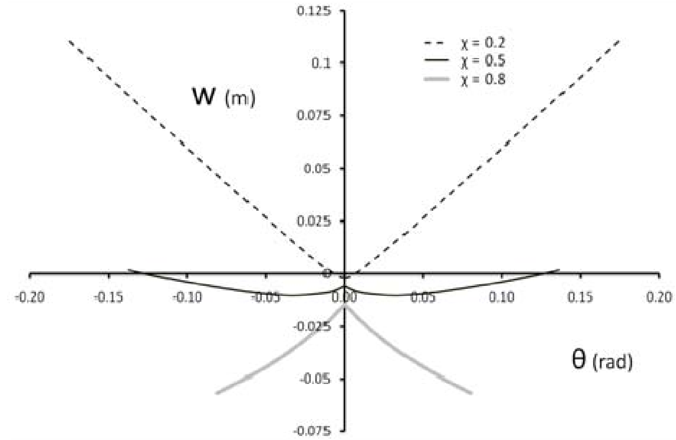

Fig. 3. Settlement-rotation envelopes for the three cases

RESULTS : CYCLIC PUSHOVER ANALYSIS

Slow cyclic results are shown for systems with low, high and medium factors of safety ($\chi = 0.8$, $\chi = 0.2$, and 0.5) respectively). As it can be seen in the moment-rotation diagrams, the envelopes of the cyclic analyses for safety factors greater than 2 ($\chi < 0.5$) are well enveloped by the monotonic pushover curves [Fig.4(a1)]. This can be explained by the fact that the plastic deformations, which take place under each corner of the foundation during the deformation controlled cyclic loading, are too small to affect to any appreciable degree the response of the system when the deformation alters direction (Fig.5). The key factor of this response is the low ground compliance due to the lightly loaded foundation. Effectively, the soil is nearly underformable at such small χ values.

However, the response of the heavily loaded structures ($\chi \geq 0.5$) is remarkably different. The M – θ loops are no longer enveloped by the monotonic pushover curves. It seems that the moment capacity of the system depends on the rotation of the previous step, and as the rotation increases and the safety factor diminishes the difference between the two curves (cyclic and monotonic pushover) increases [Fig.3(b1)]. This striking behavior can be attributed once again to soil compliance. As χ increases, the footing remains practically in full-contact state even for great rotation angles. The displacement loading at the mass center [Fig.6(a)] transmits a moment on the footing, let us say in clockwise direction, which mobilizes the bearing capacity type failure mechanisms. The mechanisms involve :

(a) a shallow rotational failure under the pushed–in right edge of the footing; the sliding surface passing through the zone of excessive shearing deformation [dark line in Fig.6(b)] extends a small distance beyond the footing; and
(b) a deeper rotational movement under the upward moving left side of the foundation, without a well–defined failure surface extending beyond the edge of the footing, and producing a significant bulge of the soil–footing interface [Fig.6(b)].

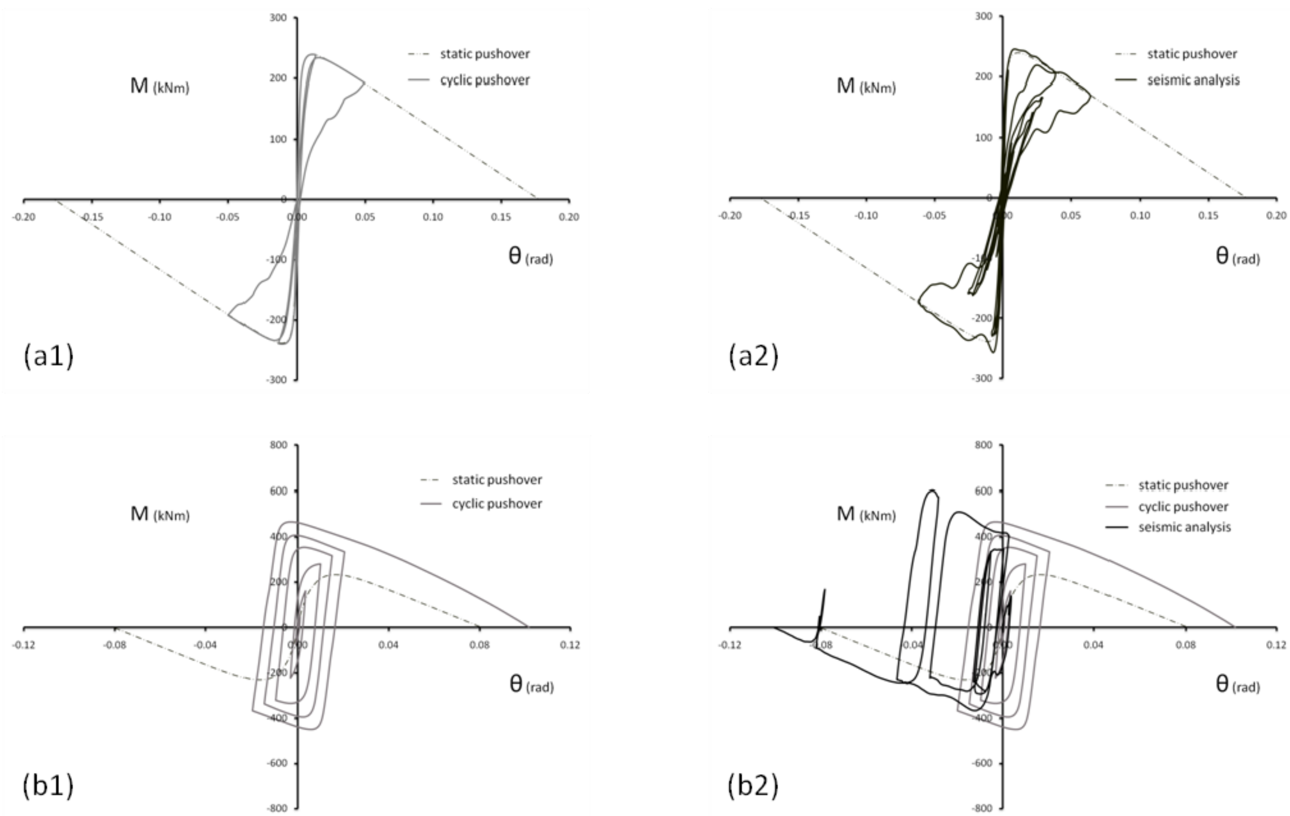

Fig. 4. Moment – Rotation relations : static (pushover), slow cyclic, and seismic (a) for χ = 0.2, (b) for χ = 0.8.

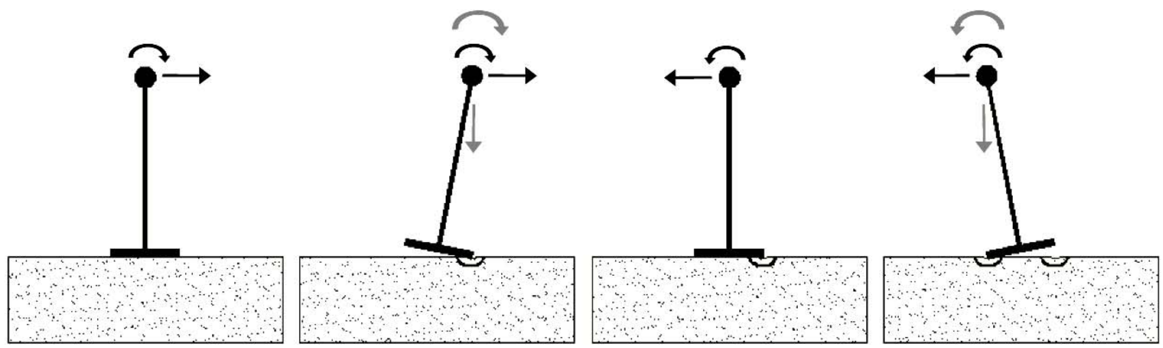

Fig. 5. Schematic snapshots of the displacement / rotation of the system and the corresponding bearing capacity failure mechanisms. Lightly loaded foundation (χ < 0.5).

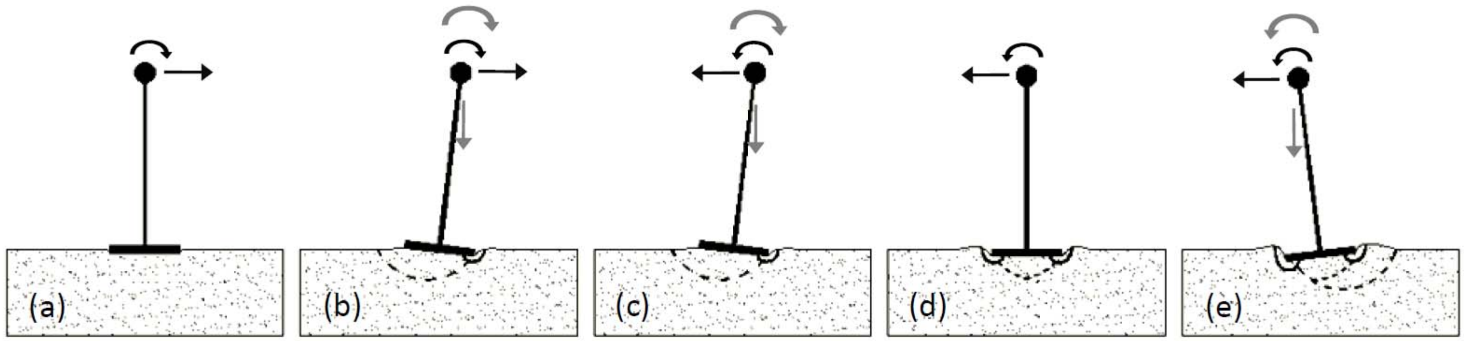

Fig. 6. Schematic snapshots of the displacement / rotation of the system and the corresponding bearing capacity failure mechanisms. Heavily loaded foundation (χ ≥ 0.5

As a result, when the loading direction is reversed, the foundation has to surmount the "hill" created in the preceding cycle. Moreover, the imposed external moment is no longer compromised by the P–Δ effects, but rather increased by the moment of the weight of the structure, which is still acting in the clockwise direction [Fig.6(c)]. Two new failure mechanisms in the soil on the opposite side start developing [Fig.6(d)]. After exceeding the point of zero rotation, the weight starts also acting in the counter–clockwise direction, thus again aggravating the tendency for overturning [Fig.6(e)].

RESULTS : SEISMIC ANALYSIS

The Takatori accelerogram (Kobe, 1995) was used as rock excitation. Since the fundamental (elastic) period of the soil stratum (V_S = 400 m/s) is only 0.05 sec no soil amplification takes place with this base motion (T_p = 1 ÷ 1.5 sec).

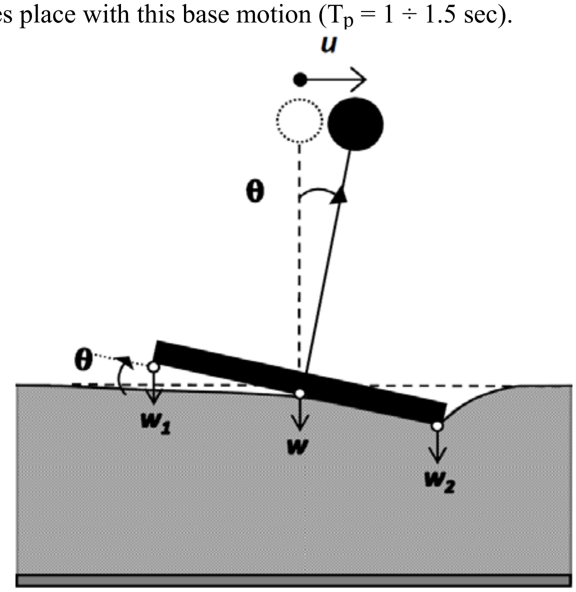

Fig. 7. Definition of displacement and rotation variables of the system.

The results for the moment–rotation and relations settlement–rotation are shown in Figures 3 [(a2) and (b2)] and 8. The following observations are noteworthy :

(a) The moment–rotation diagrams confirm the behavior already noted with cyclic loading. For $\chi < 0.5$, the M–θ relation is confined within the envelope of the static pushover analysis. On the other hand, for $\chi \geq 0.5$, the loops that are produced in the seismic analysis exceed substantially the static pushover curves. Only the first half cycle is indeed enveloped by the monotonic curve. Thereafter, as the soil exhibits large deformations due to its high compliance, the moment bearing capacity failure mechanisms become apparent. The development of these mechanisms affects the behavior of the system in the opposite direction when the acceleration changes sign. The system exhibits an a-symmetric behavior. If, for example, the first large deformation takes place to the right edge of the footing then the system displays "overstrength" when the acceleration changes to the left; yet it is more vulnerable to the next pulse that will push it again to the right. In conclusion for the majority of structures that have safety factors greater than 2 the monotonic pushover curves are representative of the moment capacity of the system even under dynamic loads. For the structures that have safety factors less than 2, the maximum moment cannot be determined a priori as it is a function of the preceding rotation and of the magnitude of the pulse. The cyclic pushover curves are representative of the behavior of the system only approximately.

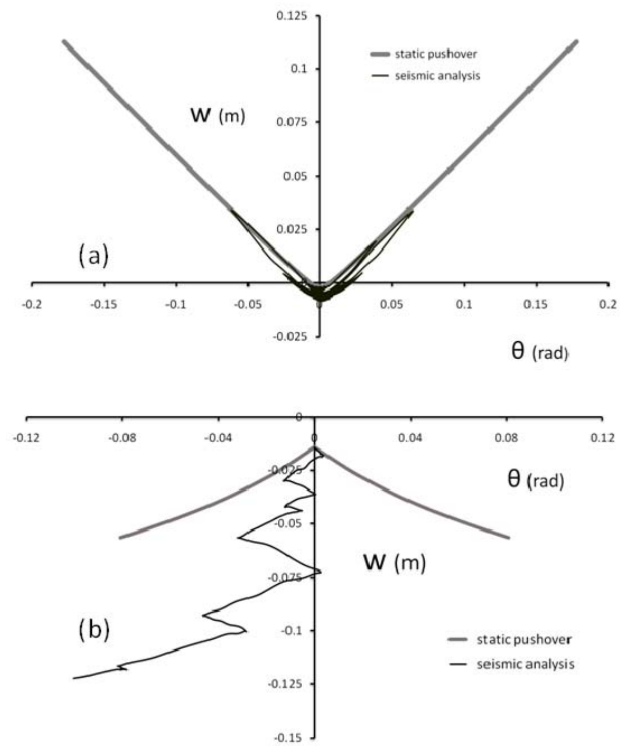

Fig. 8. Settlement – rotation diagrams for : (a) χ = 0.2 (stable system), (b) χ = 0.8 (system overturns).

(b) Regarding the rotation of the footing and consequently the horizontal displacement of the mass center, we generally conclude that the higher the safety factor the larger the rotation (Fig.8). However, it is important to note that the deformation of the lightly loaded systems is nearly elastic while the deformation of the heavily loaded systems is strongly inelastic. This leads to a progressive accumulation of plastic deformations of the heavily loaded systems, resulting to higher residual rotations.

(c) As expected, the heavily loaded structures exhibit larger settlement due to accumulation during shaking.

(d) The parameter λ, defined the effective contact area ratio :

$$\lambda = \frac{\beta}{B}$$

which represents the part β of the oscillating footing B still in contact with the deformed soil, reaches its lowest value for the higher safety factor. Moreover, when the oscillation has ceased the part of the footing still in contact with the deformed soil is greater for the low safety factor! Additionally, for the same safety factor, as the intensity of motion increases the residual λ diminishes. It is noticeable that the system can avoid overturning, while reaching values of λ as low as 0.1 due to the dynamic nature of the loading.

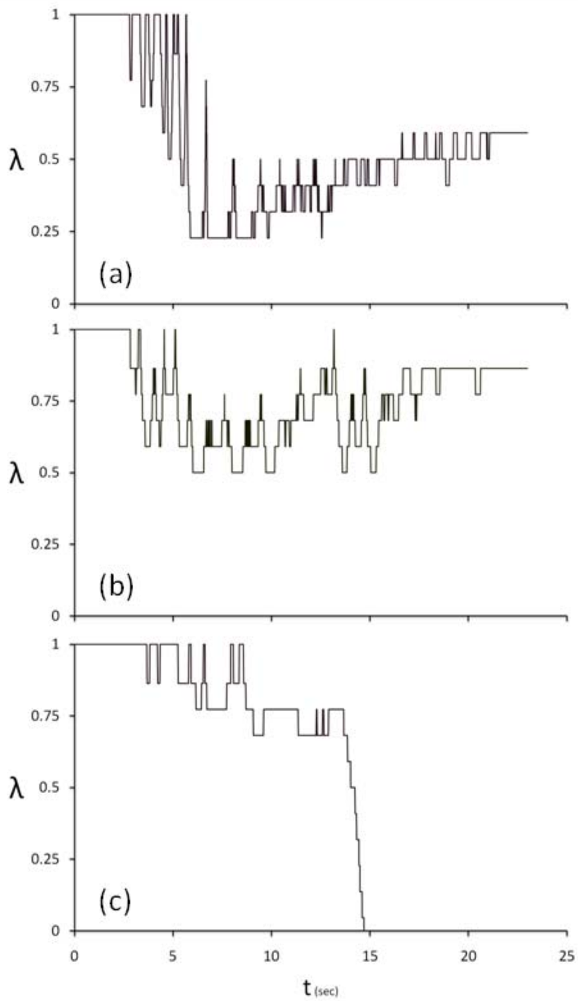

Fig. 9. Times histories of parameter λ for : (a) χ = 0.2 (stable system), (b) χ = 0.5 (stable system), (c) χ = 0.8 (system overturns).

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