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POTENTIAL RELEVANCE OF DIFFERENTIAL SETTLEMENTS IN EARTHQUAKE-INDUCED LIQUEFACTION DAMAGE ASSESSMENT

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ABSTRACT

The assessment of vulnerability of buildings subjected to earthquake-induced liquefaction requires the definition of an integrated damage scale accounting both for ground motion damage and ground permanent movements, which cause rigid-body settlement and tilt of the building but also flexural demand on members due to differential settlement of pad footings. Nevertheless, most of the existing procedures for the estimation of differential settlements rely only in soil characteristics, thus neglecting the influence of building stiffness on the soil-structure interaction. In the present work, based on simplified modelling of soil-structure variability and on preliminary assumption of force distributions, representative values of members' demand due to differential settlement are proposed. A simple approach relying on the structure-to-soil stiffness ratio and the equivalent soil degradation extent under pad footings is adopted. The methodology is calibrated by means of a parametric linear analysis for a set of planar frames. Relative flexural demand due to differential settlements normalised to the seismic flexural demand are obtained. Results show that their relevance may not be very severe, thus damage assessment of differential settlements could be likely accounted separately from flexural and rigid-body demand.

Keywords: Differential settlement; Soil-structure interaction; Earthquake; Liquefaction; Structural stiffness

1. INTRODUCTION

In recent years, increasing attention has been paid to earthquake-induced liquefaction damages on buildings after the evidences of their large influence on the overall losses. However, there is still an important lack of systematization regarding the assessment of vulnerability of buildings on potential liquefiable sites (Bird et al. 2006).

In general, assessment of seismic vulnerability of buildings requires the definition of damage levels. In liquefiable deposits, those levels should account not only for the damage in structural members but also account for the loss of functionality due to rigid-body movements which do not cause structural damage. Usually, flexural damage is related to shaking, while rigid-body movements are attributed to ground deformation, exacerbated by the rapid degradation of soil strength and stiffness due to excess pore pressure buildup in liquefiable deposits. However, frames founded in pad footings instead of rigid mat foundation can experience flexural damage due to differential settlements. Most seismic vulnerability approaches so far do not consider differential settlements except for liquefiable sites.

Different physical interpretations for the settlement of foundations in liquefiable soil have been proposed (e.g., Dashti et al 2010, Karamitros et al. 2013), which show the difficulty for considering accurately the character of SSI influence. According to Karamitros et al. (2013), there might be an

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initial, probably narrow “window” of time in which both sources of flexural damage –due to ground shaking and to differential settlements— may coexist. However, procedures with coupling of both actions are not usually considered due to their inherent difficulty. According to Bird et al. (2006), “all damage due to ground shaking occurs in the first part of the earthquake and the liquefaction-induced ground deformation will occur towards the end of, or subsequent to the earthquake”. The availability of some preliminary estimation of the relative importance of lateral seismic action vs. differential settlements could provide insight.

In Boscardin and Cording (1989), different numerical analyses and field data on brick-bearing-wall and some small frame structures subjected to vertical settlement combined with lateral movements are studied in order to evaluate the relevance of the structural stiffness. Yet, systematic approaches regarding representative reinforced concrete (RC) buildings are seldom found in literature.

Hence, the present work provides some preliminary, simplified estimation of the relevance of the differential settlements and their corresponding elastic flexural demand on members, accounting for SSI, within a wider methodology of assessment of vulnerability of RC frames subjected to earthquake-induced liquefaction. A rather simple approach relying on the structure-to-soil stiffness ratio and the soil degradation extent under footings are adopted. Then, the methodology is calibrated by means of a parametric linear analysis for a set of planar frames. Finally, results of relative flexural demand due to differential settlements are obtained for different levels of ground motion.

2. ESTIMATION OF DIFFERENTIAL SETTLEMENTS

Modelling permanent ground deformation under foundations is not straightforward. Pad footings are sized in order to warrant acceptable absolute settlements (S) which should be rather similar for all the footings. This condition is commonly chosen to be satisfied only for gravitational case. Many sources of uncertainty related to the soil and the structure cause a demand of differential settlement (ΔS) on the structure: the choice of only one load case for equal S, the homogeneisation of bearing capacity (q) for the whole site, the neglect of variations in the geometry of soil layers; the homogeneisation of measures of footings; the use of simplified formulations for the estimation of S; the neglect of group effect between close footings; the variability of loads on the building; the dimensional tolerance in the construction; the heterogeneity of soil mechanical properties; the position of water table; etc.

Traditionally, a rule of thumb is used in which ΔS within assumed homogeneous soil layers are equal to a fraction of the total settlement: $\Delta S = a \cdot S$. Many authors have proposed different characteristic (i.e., conservative) values for $0.3 \leq a \leq 1.0$ (Coduto 2001, Akbas and Kulhawy 2009). Then, those values of ΔS (with or without addition of theoretical ΔS contribution corresponding to each load case) might be imposed alternatively to the different column bases in order to design the superstructure. Those proposals arise from empirical observation of adjacent isolated footings without any structural connection between them or conversely from observation of buildings (e.g., Bjerrum 1963). If any measurement of ΔS in real or simulated structures is carried out in order to obtain regression values of a for further linear design purposes, real movements causing nonlinear incursion in structural members should not be accounted for.

On the other hand, most probabilistic approaches suffer from two main drawbacks (Akbas and Kulhawy 2009): (i) only inherent heterogeneity of soil is accounted as a source of uncertainty; and (ii) the stiffness of the structure is neglected. Any methodology relying on the imposition of vertical movements sequentially to each single footing, regardless of the stiffness of the building, does not return homogeneous safety factors for all the cases. Analogously to design approaches, most vulnerability methodologies assume that all the potential of the soil to settle in a differential manner is fully becoming effective to the frame (Bird et al. 2005, Negulescu and Foerster 2008), i.e., neglecting the stiffness of the building.

In liquefiable sites, ground movements are not only a consequence of the building mass; free-field displacements can be important. Vertical settlement due to volumetric reconsolidation and horizontal displacements due to lateral spreading of sloping ground can be feasibly estimated. The existence of those free-field movements, unlike in non-liquefiable sites, may suggest the use of free-field differential displacements for design or assessment purposes—as proposed in Bird et al. (2005)—instead of using other probabilistic-based approaches. Lower degree of conservativeness due to the lack of any consideration of variability is expected when compared with the "*a*-approach". In fact, differential settlements considering SSI could reach higher values rather than in free-field.

In the case of significant post-shaking liquefaction-induced volumetric settlement—or also if sand ejecta occurs—, the foundation of buildings with sufficient stiffness can experience a detachment from the ground surface due to incompatibility of deformations (Cubrinovski et al. 2011), which is not possible in non-liquefiable case; see Figure 1c for more details.

Conversely to the assumption of free-field displacements, other works explore the "*a*-approach" for liquefiable sites. In Stuedlein and Bong (2017), a random field model of inherent soil variability has been adopted for the analysis of settlements in free-field after shaking, for increasing seismic demand. Results show that potential a values may get reduced dramatically for large ground motions. For sufficiently high peak ground acceleration (PGA), able to cause severe stiffness and strength degradation of soil, any variability of soil mechanical properties gets greatly overwhelmed, while for lower values of PGA the degradation is not so homogeneous and differences may exacerbate.

3. METHODOLOGY FOR THE SIMPLIFIED ESTIMATION OF FLEXURAL DEMAND DUE TO DIFFERENTIAL SETTLEMENTS

In this section, a preliminary estimation of the relative importance of the equivalent elastic flexural demand caused by differential settlements when compared to the demand corresponding to lateral seismic action is carried out. RC frames with one-way slab, founded on pad footings, are considered, and non-structural infill panels are neglected. The choice of a force-based methodology relies on the need of a basic demand index compatible with linear approach. Besides, the proposed simplified procedure could also be used for design purposes instead of the "*a*-approach", regardless of the eventual seismic demand or soil liquefiability.

3.1 Equivalent soil degradation

In order to properly account for SSI, the proposed methodology is based on "causing" equivalent linear ΔS instead of imposing them, thus different ΔS are obtained depending on the structural stiffness. The whole tendency of a footing to settle differentially, due to soil and structure variability, is modelled by an equivalent reduction of stiffness of the soil below the footing. It is represented by a "degradation factor" $c = K_s'/K_s = k_{sb}'/k_{sb}$, $0 \leq c \leq 1$; the prime (') symbol refers to equivalent degraded parameters. $K_s = N/S$ is the stiffness of the equivalent single springs under footings, which can be related to the corresponding soil ballast coefficient (k_{sb}) as $K_s = k_{sb} \cdot A$, being $A = B^2$ the contact area of assumed square footings, considering strict design values of $B = (N/q)^{0.5}$.

The definition of a degradation factor c is consistent with the "*a*-approach" only if structural stiffness is neglected and all the sources of uncertainty are considered. An equivalent parameter $a^* = \Delta S^*/S$ is defined; the asterisk (*) symbol refers to hypothetical magnitudes if the structure stiffness tends to zero. Considering that the axial load can be expressed as $N = q \cdot A$, then the spring stiffness is formulated as $K_s = q \cdot A/S$. Rearranging the expression and considering $A = K_s/k_{sb}$, the total settlement can be expressed as $S = q/k_{sb}$. Hence, a simple relation between a^* and c is obtained as $a^* = 1/c - 1$.

The relation between the different parameters is qualitatively shown in Figure 1. Firstly, average k_{sb} is chosen; then, c is defined based on a probabilistic assumption of a^*, so a degraded k_{sb}' is imposed under only one footing. Subsequently, springs are modelled with equivalent stiffness K_s and K_s' for average and degraded soil, respectively. Finally, elastic ΔS are obtained from linear analysis, and hence the real parameter $a = \Delta S/S$, $0 \le a \le a^*$, can be obtained. For liquefiable sites, $c = 0$ is not unsuitable (see last example of Figure 1). Apart from the ability of that case to represent suitable situations, such a value of ΔS for complete soil degradation could be also understood as an upper bound, i.e., the "maximum free differential settlement" ($\Delta S_{\max}$) that a column base can experience, which is different depending on their position within the frame—exterior or interior.

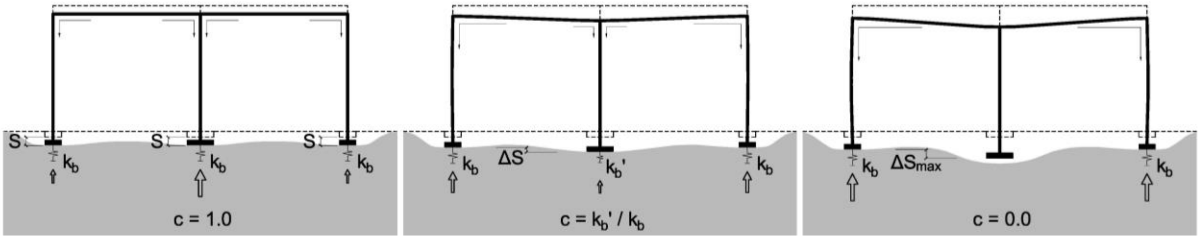

Figure 1. Conceptual configurations of equivalent models and ΔS for decreasing values of c

3.2 Estimation of member forces

In order to estimate the relative damage caused by the differential settlements with respect to the seismic case—horizontal loading and gravitational action—, an index of flexural contribution of ΔS is conceptually defined in the form $\zeta \equiv$ Force[ΔS/(Gravitational+Seismic)]. Within a linear approach, the member force chosen as the flexural index could be either shear force (V) or bending moment (M), regardless of the most suitable failure mechanism that can be expected in members of sub-standard or code-conforming frames, i.e., brittle shear failure or plastic hinge development, respectively. For beams, slightly higher values of ζ would be expected for M rather than for V.

The simplified methodology is based on the application of equivalent soil deformation under only one single footing, in order to cause ΔS between that footing and the contiguous one(s). Real cases composed of several adjacent ΔS in any direction (e.g., degradation of the soil under a large part of the building) would be characterised by larger force redistribution to the non-settled part but conversely the probabilistic nature of the problem would return lower soil degradation under the settled footings. Those situations of increasing complexity are out of the scope of this work.

Regarding the relative position (interior or exterior) of the more settled footing, it is necessary to make a distinction between single- and multi-bay frames. For multiple bays, the rotational demand in beams is generally larger rather than in columns if only one footing is causing ΔS, even when it is exterior. For exterior footings, ΔS is much larger (up to twice) rather than in the case of interior one due to the rotation of the adjacent column, regardless of the number of bays. Still, the forces induced by ΔS to the adjacent beams are roughly independent on the interior or exterior position, because in all the cases each adjacent column assumes a transfer of axial load from the more settled one which corresponds to a tributary area roughly equivalent to $L/2$, being L the span of the beam.

Rather different behaviour is observed in 1-bay frames (Bird et al., 2005), where the absence of counterbalancing bays causes an increase of rotational demand on column bases which may rule the collapse mechanism. Also, similar pattern is observed if increasing soil degradation alongside the building width is assumed, which is also out of the scope of this work. However, most vulnerability methodologies do not consider 1-bay frames to be sufficiently representative of the existing RC building stock, according to on-site after-event observation (Ricci et al. 2011).

Hence, the proposed methodology is based on the application of equivalent soil degradation to interior footings of multi-bay frames and the consideration of maximum $|V|$ in 1st storey beams as the flexural index of the whole frame. In Figure 2, $|V|$ is decomposed as the sum of the shear forces corresponding to gravity action, ΔS and seismic action (V_g, V_s and V_d, respectively).

The estimation of V_g, V_s and V_d requires the assumption of simplified aprioristic distribution of forces in members. As usual, for gravitational action, tributary areas corresponding to L and $L/2$ for interior and exterior columns, respectively, are considered. For seismic action, a linear distribution of forces with height is assumed. However, regarding settlements, the only way to estimate V_s is to consider it as a fraction of their upper-bound value, $V_{s,\max}$, which is the force corresponding to the maximum free differential settlement $\Delta S_{\max}$ for complete soil degradation $c = 0$; in this case the gravitational tributary areas are simply redistributed to the adjacent columns. Hence, the upper-bound of the flexural index parameter is obtained as $\zeta_{V,\max} = V_{s,max}/(V_g + V_d)$.

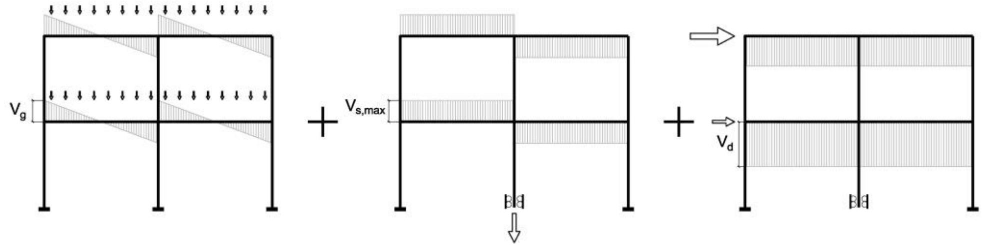

Figure 2. Maximum $|V|$ in beams considering superposition of gravitational loading (V_g), maximum free differential settlement of interior footing ($V_{s,\max}$) and seismic action (V_d)

In the transverse direction, similar span than the beams is assumed to be representative enough, thus V_g is simply obtained as in Equation 1a, where $w = m \cdot g$ is the superficial weight, being m the superficial mass in seismic situation and g the gravity acceleration.

$$V_g = wL^2/2 \quad ; \quad V_{s,\max} = wnL^2 / \left[U_b\left(n + f_b\right)\right] \quad ; \quad f_b = I_{fb}/I_b \qquad (1\text{ a, b, c})$$

The estimation of $V_{s,\max}$ (see Equation 1b, being n the number of storeys) depends on the number of beams with sufficient stiffness and strength framing the settled column at each storey level, which is represented by the parameter U_b. The eventual collaboration of one-way slab in resisting differential settlements is neglected, because the resistance to sagging moment in the joists framing the settled column is usually low, especially for pre-cast elements. Also, if there are foundation beams, it results in lower demand on the superstructure. The factor f_b is accounting for this reduction (see Equation 1c, where I_{fb} and I_b are the gross moment of inertia of the cross-section of foundation beams and superstructure beams, respectively). The collaboration of footings may be disregarded, although their elastic contribution to the overall stiffness result in more than twice the values only considering foundation beams, if the whole width is considered to be effective. Footings are usually only resistant to sagging moment, so their contribution under adjacent columns to the settled one is neglected.

For the estimation of V_d (see Equation 2, being h the interstorey height, V_i the storey shear force at the i^{th} storey and $V_b = V_1$), a portal-frame distribution of bending moments is assumed, and the first mode is considered to be representative of the deformed shape. The base shear V_b can be expressed as in Equation 3a, being λ the first mode participating mass ratio of the multi-degree-of-freedom, M the total mass in seismic situation, and $S_a(T)/g$ the 1st mode elastic spectral acceleration normalised to the gravity acceleration. For the evaluation of the fundamental period (T), proposal of Crowley and Pinho (2010) is followed. Besides, λ can be estimated as in Equation 3b (Gómez-Martínez 2015).

$$V_d = h/L \cdot \left(V_1 + V_2/2\right) = h/L \cdot V_b\left(1 - 1/2n^2\right) \qquad (2)$$

$$V_b = \lambda M S_a(T) = \lambda n w L^2\, S_a(T)/g \quad ; \quad \lambda \approx \left(2.95^n + 0.99\right)^{-1} + 0.75 \qquad (3\text{ a, b})$$

3.2 Structure-to-soil stiffness ratio

The representative flexural index parameter for $c > 0$ is calculated as $\zeta_V = \chi \cdot \zeta_{V,\max}$. Factor χ depends on the structure-to-soil stiffness ratio $\alpha = K_{st}/K_s$, being K_{st} the “vertical” stiffness of the structure. The more rigid is the structure with respect to the soil, the larger the shear forces due to the differential settlements are. A simple mechanical model can be adopted in which all the vertical stiffness of the frame, including foundation elements, is concentrated in a beam element (see Figure 3a). Considering the rest of the assumptions made so far, factor α can be calculated as in Equation 4) where f_K is the coefficient of stiffness (equal to 12 or 3 for fix- or pin-end beam, respectively). The upper bound of K_{st} can be considered, i.e., the sum of the vertical stiffness of all the beams and foundation beams above the settled footing, considering fix-end conditions.

$$\alpha = \frac{K_{st}}{K_s} = \frac{U_b\left(n+f_b\right) f_K E_c I_b / L^3}{N k_{sb}/q} = \frac{f_K U_b q E_c I_b \left(n+f_b\right)}{w n k_{sb} L^5} \tag{4}$$

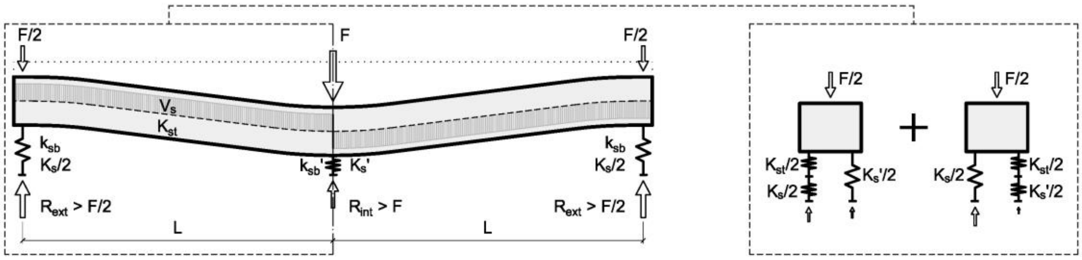

Figure 3. Conceptual model adopted for the calculation of the structure-to-soil stiffness weighting factor χ (a) and decomposition of the static problem of each half (b)

In order to obtain χ, the static problem is solved through a decomposition of each half of the beam in two combined systems, one for each applied force in the member ends (see Figure 3b). Thus, χ is obtained as function of the soil degradation factor c and the stiffness-to-soil factor α (see Equation 5 and Figure 4). For typical values of c = 0.5, upper bound of χ is lower than 35%.

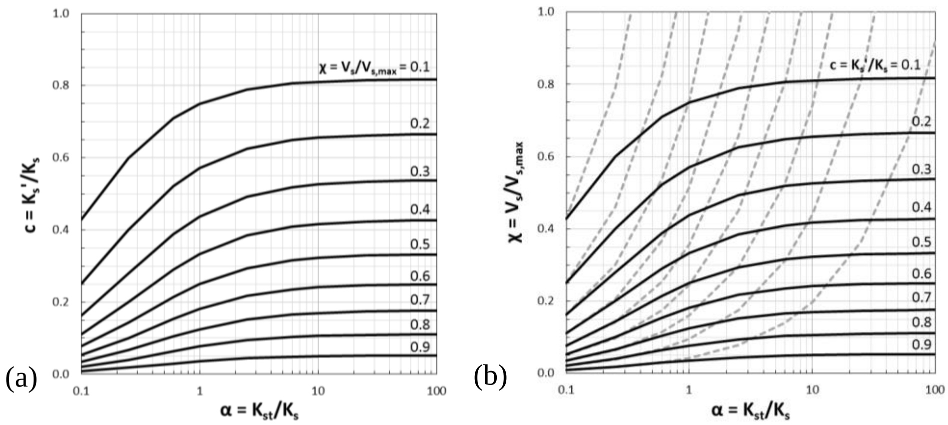

Figure 4. Charts of χ depending on c and α; the dashed grey line shows qualitatively the greater increase of resistance (capacity) with respect to the increase of demand when the stiffness is increased

$$\chi = \frac{\left(K_{st}^{-1}+K_s^{-1}\right)^{-1}}{K_s' + \left(K_{st}^{-1}+K_s^{-1}\right)^{-1}} - \frac{\left(K_{st}^{-1}+K_s'^{-1}\right)^{-1}}{K_s + \left(K_{st}^{-1}+K_s'^{-1}\right)^{-1}} = \frac{\alpha\left(1-c\right)}{c+\alpha\left(1+c\right)} \tag{5}$$

The shear demand on the superstructure due to the differential settlement increases with the stiffness of the frame, especially for the range of low α values. However, it does not mean that rigid frames are more vulnerable to differential settlements regardless of the rest of the parameters, because the increment of demand is slight if compared to the increase of strength for similar increment of member stiffnesses (see Figure 4b). Aimed at the evaluation of the damage contribution of real differential settlements, the procedure conclude by calculating $\zeta_V = \chi \cdot \zeta_{V,\max}$. However, for equivalent elastic design purposes, it could be useful to obtain particularised a values for each frame, so that they can be imposed to the footings instead of the generalised—and perhaps nonlinear-based—values proposed in literature (see section 1). In Equation 6, the expression of a is developed, where $\Delta S_{\max} = N/K_{st}$ and S_0 is the settlement for no soil degradation far away from the more settled footing.

$$a \equiv a_0 = \frac{\Delta S}{S_0} = \frac{\chi \cdot \Delta S_{\max}}{S_0} = \frac{\chi \cdot N/K_{st}}{N/K_s} = \frac{\chi}{\alpha} \tag{6}$$

3. CALIBRATION BY MEANS OF A PARAMETRIC NUMERICAL ANALYSIS

The procedure presented in the precedent section is herein calibrated by means of a parametric numerical analysis. A set of 36 planar frames, representative of sub-standard buildings (which are more likely to form the existing RC building stock), are analysed for different situations: (i) with and without foundation beams; (ii) c equal to 0.0, 0.2 and 0.5; and (iii) PGA equal to 0.20, 0.30 and 0.40g. Geometrical parameters of the frames are: number of bays (2 and 4), L (4 and 6m) and n (3, 6 and 9).

The simulated design of the frames has been carried out without any capacity design and adopting a basic acceleration $a_g = 0.08g$, which is the threshold of low seismicity according to Eurocode 8.
The possible bias caused by the differences between the estimated and real elastic fundamental period has been removed by adopting in the prediction the values of $S_a(T)$ corresponding to the numerical analysis. The scope of the calibration is more focused in the evaluation of V_s rather than V_d; moreover, the resulting T is highly dependent on the design approach.

Correction factors aimed at a proper calibration are proposed. Those values should be understood only as indicative ones, considering the reduced amount of frames analysed; further research is needed.
Numerical values of V_s are, on average, 12% higher than predicted, because the decompression of the settled column and the consequent loss of strain with respect to the adjacent ones causes an increase of shear force in lower storeys and a decrease in higher storeys. This phenomenon is more relevant for increasing n, and its evaluation is not straightforward since it depends on whether the sequential shortening of columns along the construction phase has been accounted or not in the design phase.

On the other hand, V_d gets slightly overestimated especially for tall buildings (5% on average) due to the “cantilever effect”: an increase on the shear span of the columns at lower storeys, which causes a decrease in the shear demand in beams at the corresponding storeys and also a slight decrease of λ due to the higher concavity of the deformed shape (see Figure 5a). The lack of consideration of higher vibrational modes show low counterbalance.

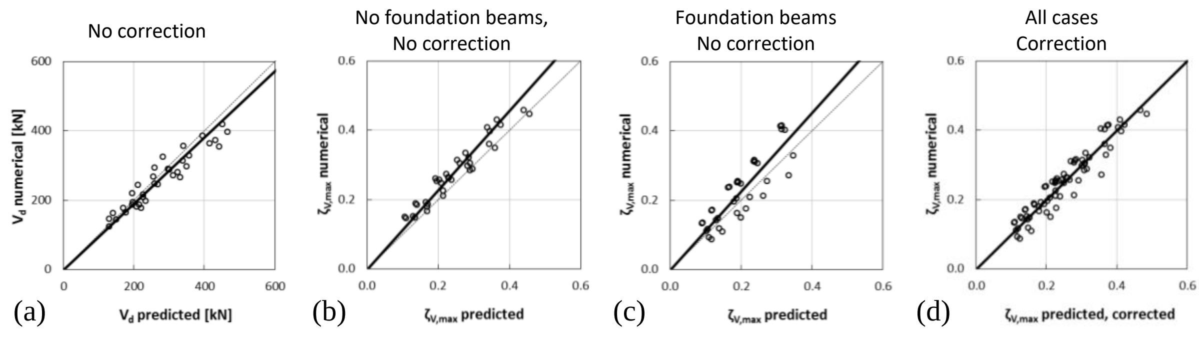

Figure 5. Predicted vs. numerical results of V_d and $\zeta_{V,\mathrm{mas}}$

Consequently, numerical results of $\zeta_{V,\max}$ are, on average, 13% larger rather than predicted ones, both for the consideration of foundation beams or not (see Figure 5 b and c). Considering that both sources of bias show a clear trend with n, a correction factor β^n is searched for best fit. Values of $\beta = 1.02$ (see

Figure 5d) show lower scatter than a trivial constant correction factor equal to 1.13.

Regarding ζ_V, predicted values show increasing overestimation with increasing *c* (Figure 6), because in that case the accuracy in the estimation of the superstructure stiffness play an important role, especially when foundation beams are present. In fact, values of ζ_V for superstructures of low height, low number of bays and large span show real stiffness much lower than predicted due to the rotation of the adjacent columns. However, the overestimation of beam damage could somehow be qualitative indicator of the increasing damage of columns, from a very simplified point of view. Values of $f_K \approx 8$ (intermediate between fix- and pin-end) are in good agreement with the whole database. Scatter is high (CoV = 0.24) but still acceptable considering the very simplified nature of the procedure.

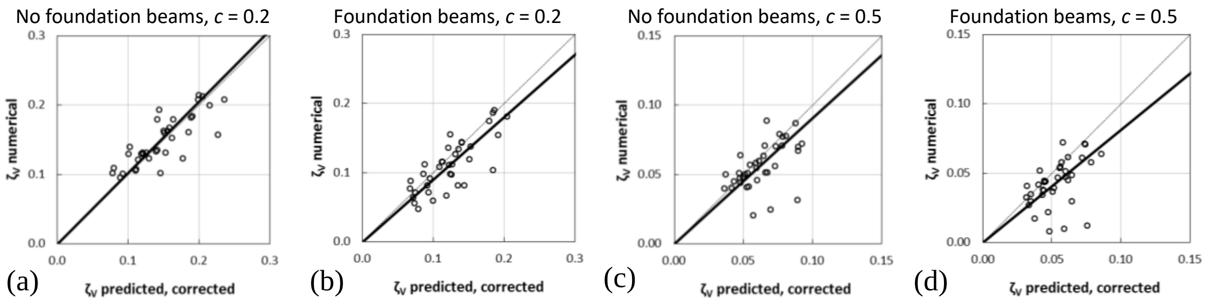

Figure 6. Predicted (corrected by β^n and considering maximum frame stiffness) vs. numerical results of ζ_V

Besides, predicted values of *a* show consistent underestimation of settlements (30% on average) for $f_K = 8$, because moment redistribution within the frame alters substantially the axial load demand on contiguous footings. However, for equivalent elastic design purposes, values of *a* should be consistent with the best-fit of forces instead of displacements, thus *a* should not be calculated considering reduced α; otherwise it would result in large overstrength.

4. ESTIMATION OF RELATIVE DEMAND DUE TO DIFFERENTIAL SETTLEMENTS

Once that the proposed methodology has been satisfactorily validated, the relative flexural demand due to differential settlements and their relevance within a framework of seismic and liquefaction vulnerability can be evaluated. Results of the parametric analysis show decreasing values of flexural demand for decreasing soil degradation. Still, the values of *c* have been chosen in order to cover a range in which the calibration of the different variables becomes critical (i.e., large degradation) rather than being representative of the soil behaviour of liquefiable sands.

4.1 Representative soil characteristics

Regarding soil quality, two primary variables should be stated: ballast coefficient k_{sb} and degradation factor *c*. Typical liquefiable sand in non-liquefied state may show a friction angle of 33-35º, which could correspond to a ballast coefficient for a 30 x 30 cm plate $k_{sb,30}$ = 60 MN/m^3. The decrease of stiffness caused by full liquefaction in homogeneous, infinite soil layers has been assessed, among others, by Ishihara and Cubrinovski (2004), which suggest a range between 1/30 and 1/80. However, typical multilayered soils with superficial crust may show much less softening. In the present work, equivalent softening is obtained by assuming that the foundation has been designed for a total settlement of 25mm, while the final settlement after liquefaction is estimated according to the procedure proposed by Karamitros et al. (2013) for liquefiable layers under a non-liquefiable superficial crust. Results corresponding to different situations return values of softening typically ranging between 1/5 and 1/20; hence, effective $k_{sb,30}$ may be lower than 10–15 MN/m^3.

On the other hand, factor *c* should be estimated through a probabilistic approach accounting for the soil inherent variability. The procedure suggested by Schneider et al. (2015) is followed, although the rest of the sources of uncertainty are not considered. Three different criteria are followed. Firstly, a "representative" value of c is defined as $c_{rep} = 1/(1 + a^{*}_{rep})$, consistently with the assumption of a^{*}_{rep}; i.e., that among two adjacent footings, one of them experiences mean value of settlement while the other one exceed it in magnitude equal to the standard deviation. Then, in order to estimate a suitable characteristic value, and assuming that ΔS follow a normal distribution, $a^{*}_{0.95} \approx 1.96 \cdot a^{*}_{rep}$ is adopted for the calculation of $c_{0.05} = 1/(1 + a^{*}_{0.95})$.Finally, a lower bound $c_{min} = (1 - a^{*}_{rep})/(1 + a^{*}_{rep})$ is obtained when two adjacent footings are assumed to show opposite addition of a standard deviation respect to the mean value.

In order to have a proxy of those c values, some assumptions can be done. Representative ranges for geometric variables L = [3–7] m and B = [1–3] m are considered; combinations of both lower bounds together and both upper bounds together are considered provide more probable values, while alternate combinations are considered to provide extreme values. Finally, H adopt values so that $\Delta H = [4\text{–}6] \cdot B$. Results (see Figure 7, in which ordinates of $c = 0.2$ and 0.5, adopted in the parametric numerical analysis, are also plotted) show that the probability for $c < 0.5$ is extremely low. Thus, shear demand on members obtained in parametric numerical analysis corresponding to $c = 0.5$ might be conservative enough even when not all the sources of uncertainty are considered. However, lower values should be at least qualitatively studied because full detachment is possible in liquefied soil.

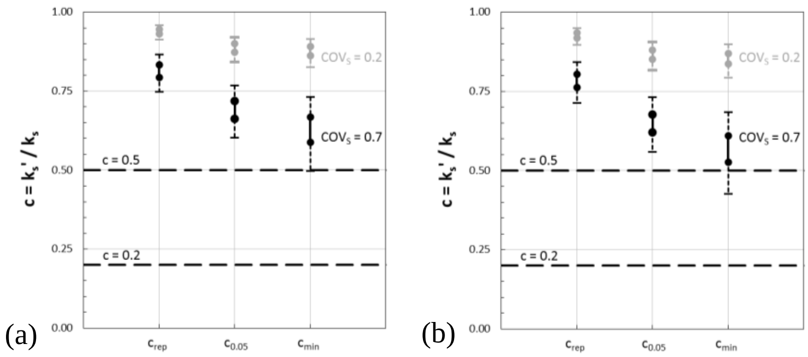

Figure 7. Representative (c_{rep}), characteristic ($c_{0.05}$) and minimum (c_{min}) equivalent soil degradation parameter according to different geometric and probabilistic assumptions, for $\Delta H = 4B$ (a) or $6B$ (b)

4.2 Flexural demand

By means of the proposed methodology, ζ_V are obtained for different frames. Sub-standard buildings are assumed to be arranged without beams in the transverse direction, thus $U_b = 2$; conversely, $U_b = 4$ for code-conforming frames. In order to estimate superstructure stiffness, slenderness of beams L/h_b has been estimated to be 14 for sub-standard buildings, which is the most conservative value for avoiding the evaluation of deflection in the design phase to gravity loads according to Eurocode 2 (BSI 2004), and thus considered to be a representative value. For code-conforming buildings, designed to Damage Limitation Limit State, L/h_b is likely 9 according to Gómez-Martínez (2015) when conventional deep beams are used; typical cross-section aspect ratio of 3:2 have been assumed. Only code-conforming buildings are considered to present well-connected foundation beams, and cross-section of 40 x 40 cm is considered. Concrete 25/30 is considered, and contact pressure q = 100 kPa for pad footings is adopted. Different spectral type (1 or 2 according to Eurocode 8) are considered, as well as soil type, even when sand may present rather common characteristics.

In Figure 8, values of ζ_V and $\zeta_{V,\max}$ against n and L for sub-standard buildings considering $c = 0.5$ are shown. Both variables are almost proportional, given that α is almost constant (only k_{sb} changes slightly, see Equation 4, because K_s refers to the equivalent spring and also increases with n), thus χ is rather constant itself. Both parameter ζ increase with n, because seismic demand increase while gravitational and settlement demands remain constant; and conversely increase with L, for the opposite reason. Values of ζ_V are always lower than 10%, because the reduced values of $\alpha < 1.0$ lead to $\chi \approx 0.25$. Thus, flexural demand of ΔS may be almost negligible if compared to seismic demand.

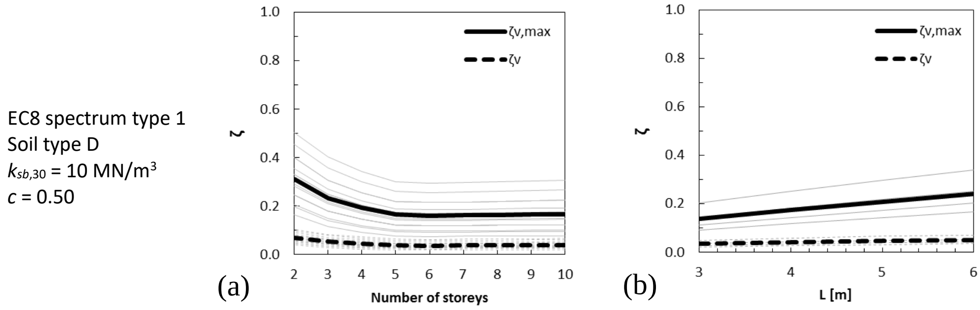

Figure 8. Values of relative shear force depending on n (a) and L (b) for sub-standard buildings

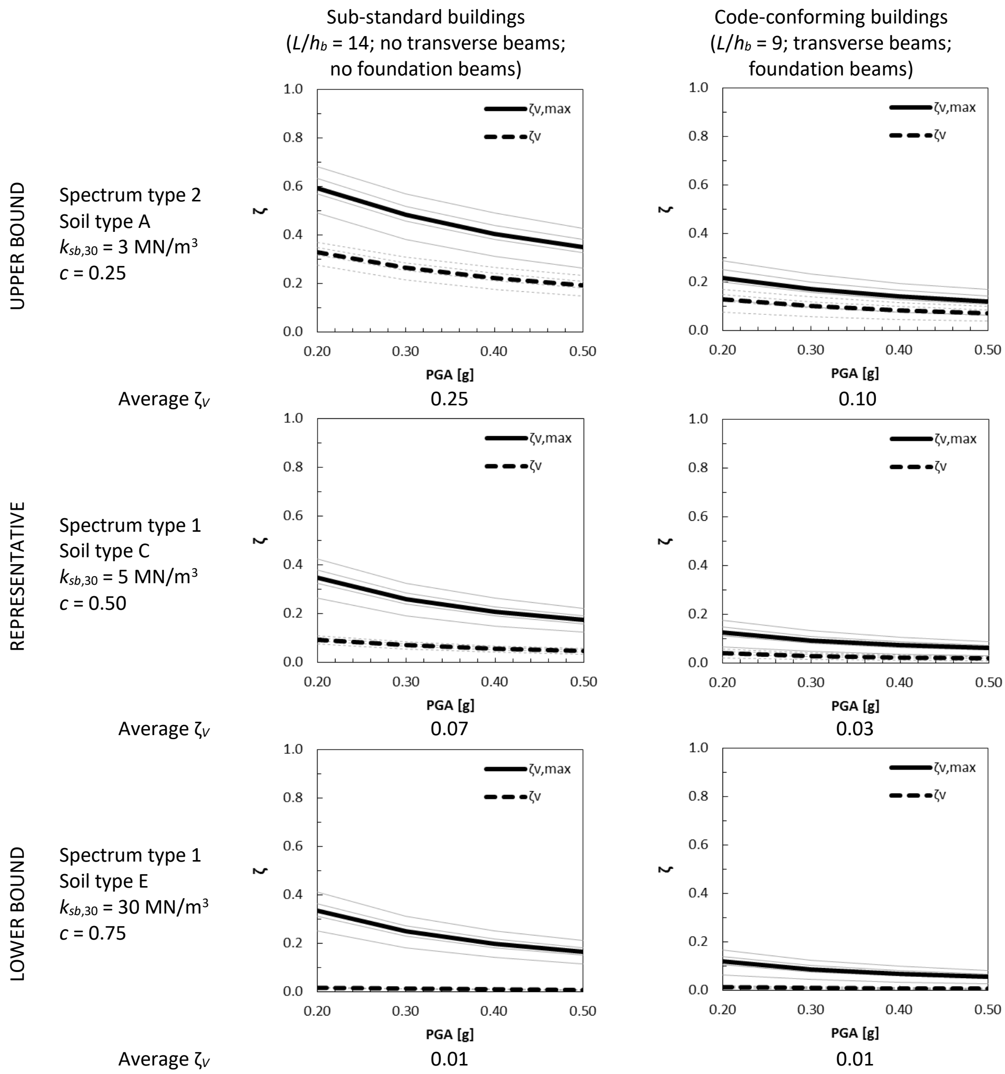

Figure 9. Values of relative shear force (ζ_V) due to settlement depending on the PGA of the seismic event for different situations; h = 3 m, h_b/b_w = 1.5, w = 6.6 kN/m^2, q = 100 kN/m^2; thick line denotes average values

In Figure 9, three different collections of variables are chosen aimed at reflecting upper bound, representative and lower bound values of relative shear force. Graphics show in all the cases the evolution of ζ_V with increasing PGA, which increase V_d and thus cause lower ζ_V. In almost all the cases, relative shear force is lower than half for code-conforming frames with respect to sub-standard ones, due to the collaboration of transverse beams and foundation beams in resisting V_s, even when such an increase of stiffness cause a rise of χ up to 50%. Only in extreme cases (i.e., upper bound), ζ_V can reach non-negligible values; still, for representative cases, average values of ζ_V are 7% and 3% for sub-standard and code-conforming buildings, respectively. Finally, in Figure 10 expected *a* values for different cases are plotted. The higher stiffness of code-conforming frames (α values up to 15 times those corresponding to sub-standard frames) is the responsible of the very low values of *a*, smaller than 0.1. Even for sub-standard buildings, values of *a* are lower than 0.5, which represents an important reduction from the assumed $a^* = 1.0$, thus frame stiffness is definitely not negligible.

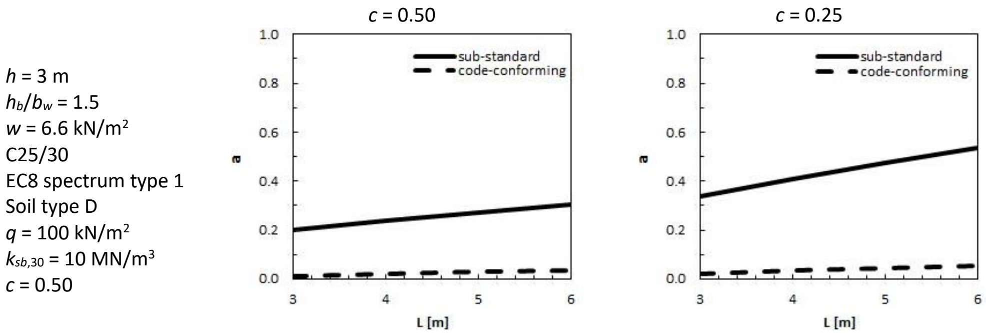

Figure 10. Average values of *a* for different *c* values

Based on the presented simplified, equivalent elastic methodology, the relevance of differential settlements in seismic situation appears to be minor, thus the evaluation of their damages could be considered separately from ground motion damages. Nevertheless, such an assumption should be supported by a more complete body of analyses considering nonlinearity. The results obtained by means of the proposed tool should be carefully understood only as a preliminary estimation.

5. CONCLUSIONS

A simplified methodology for the preliminary estimation of the relevance of the flexural demand caused by the differential settlements due to liquefaction with respect to the seismic demand, is proposed and calibrated against a set of numerical analyses. The goal of the proposal rely on its simplicity and its ability to account in a simple manner for the soil-structure interaction. The procedure estimates the relative shear force demand on the first storey beams (chosen as a proxy) as a fraction of the maximum potential value corresponding to the full degradation of the soil below each pad footing settling individually, through an estimation of the structure-to-soil stiffness ratio and the equivalent soil degradation extent according to probabilistic estimation of soil variability. Simplified assumption of force distribution are required. The methodology can be summarised in the following formulation of the relative shear demand due to differential settlements (Equation 7):

$$\zeta_V = \frac{\chi \beta^n n L}{U_b \left(n + f_b\right) \left[\frac{L}{2} + \lambda h \frac{S_a(T)}{g} \left(n - \frac{1}{2n} \right) \right]} \tag{7}$$

- χ switches from the maximum free settlement to the real one, and is calculated from Equation 5 depending on the degradation factor *c* (whose lower bound may be 0.5) and the structure-to-soil stiffness ratio α, obtained from Equation 4, where $3 \leq f_K \leq 12$ accounts for the rotational restraint of beam ends and is suggested to be conservatively 8 or higher;
- β, suggested to be 1.02, corrrects the overestimation of seismic demand due to the “cantilever effect” and the distortion of V_s due to decompression of the settled column;
- U_b is the number of effective beams joining the settled column at each level, in any direction;
- f_b accounts for the presence of foundation beams, and is calculated from Equation 1c;
- λ is the relative participating mass of the MDOF, and can be obtained through Equation 3b;
- *n*: number of storeys; *L*: beam span length; *h*: interstorey height; $S_a(T)/g$: elastic seismic demand spectral acceleration.

Results show that, in most cases, the relevance of the potential increment of the demand on members due to differential settlements may not be very severe, thus leading to some alternatives for damage assessment in which differential settlements could be accounted separately from flexural and rigid-body demand, to some extent. The proposed tool should be carefully understood only as a preliminary, linear estimation; further research, including nonlinear analyses, is required.

6. ACKNOWLEDGMENTS

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