Rocking Isolation of Frames on Shallow Footings : Design Limitations

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ABSTRACT

Taking advantage of mobilization of inelastic foundation response has been recently proposed as a novel seismic protection scheme. Such a design concept has been termed *"rocking isolation"*, and consists of intentionally *under-dimensioning of the footings in order* to respond to strong seismic shaking through rocking, thus bounding the inertia forces transmitted to the superstructure. This paper attempts to shed light in the possible limitations of rocking isolation, mainly caused by the unavoidable uncertainties regarding the estimation of soil properties and ground motion characteristics. It is shown that even a gross overestimation of the soil properties may not cancel the favorable role of rocking–isolation, although at the cost of increased residual deformations of the structure. In contrast, its effectiveness may be limited in case of un-symmetric structures although the prevailing rocking

1. Introduction

Recorded accelerations during strong earthquakes over the last 20 years very often overly exceeded code provisions: in the 1994 Northridge earthquake (Ms = 6.8) the maximum recorded P.G.A. exceeded 0.90 g; the 1995 Kobe earthquake (Ms = 7.2) produced maximum recorded acceleration of a = 0.85 g, while the 2007 Niigata-ken Oki earthquake produced an acceleration of a = 1.20 g. Such events have demonstrated that non-linear foundation response is indeed inevitable during strong seismic shaking. In fact, ensuring elastic foundation response may even be totally undesirable since enormous ductility demands would be imposed on the superstructure. On the other hand, allowing "plastic hinging", in the form of foundation uplift could be beneficial for the superstructure as it would bound the inertial forces transmitted to it [*Yim & Chopra, 1985; Martin & Lam, 2000; Pecker, 1998; Faccioli et al., 2001; Kutter et al., 2003; Gajan et. al., 2005; Kawashima et al., 2007; Paolucci et al., 2008; Chatzigogos et al., 2009; Anastasopoulos et al, 2010*].

The potential effectiveness of the mechanisms of foundation uplifting on frame structures has recently been investigated by Gelagoti et al (2011) for a simple 2-storey 1-bay frame (**Fig. 1**). Since foundation plastic "hinging" is mainly in the form of rocking and uplifting of the footing, the proposed design concept is termed *rocking isolation*, following the terminology proposed by [Mergos and Kawashima, 2005].

The authors compared the seismic performance of a conventionally designed structure (with square footings of b = 1.7 m) to a specific rocking-isolation alternative (with smaller footings of b′ = 1.4 m). In this latter case footings were designed so that their moment capacity (M_{ult}) is smaller than that of the corresponding column. Hence, when the earthquake demand exceeds the footing capacity of the foundation, uplift is promoted. The Safety Factor against vertical loads was adequately high ($FS_v > 5$) so that foundation rocking prevail and soil yielding is impeded.

The performance of the rocking–isolated alternative was investigated by means of static pushover and nonlinear dynamic time-history analysis using an ensemble of 24 strong motion records (**Figure 1b**). The distress is expressed in terms of ductility demand over ductility capacity ratio (μ_{demand} /$\mu_{capacity}$) of the ground floor column. Based on the corresponding drift ratios (*Priestley et. al., 2007*), for μ_{demand} /$\mu_{capacity}$ ratios below 0.15 the structural response is considered very satisfactory maintaining the structure within *Serviceability Limits*. For intermediate values of the ratio 0.15< μ_{demand} /$\mu_{capacity}$<0.25, the frame response falls within the *Damage Control Limit State* in which the structure is expected to sustain repairable damage, but the cost of repair would be substantially lower than the cost of replacement. Finally for 0.25<μ_{demand} /$\mu_{capacity}$<1 the collapse of the structure may be marginally avoided, although structural damage will be excessive and replacement will be unavoidable, while values of the ratio higher than 1 signify failure. The earthquake records are described through their peak spectral velocities (maxSV).

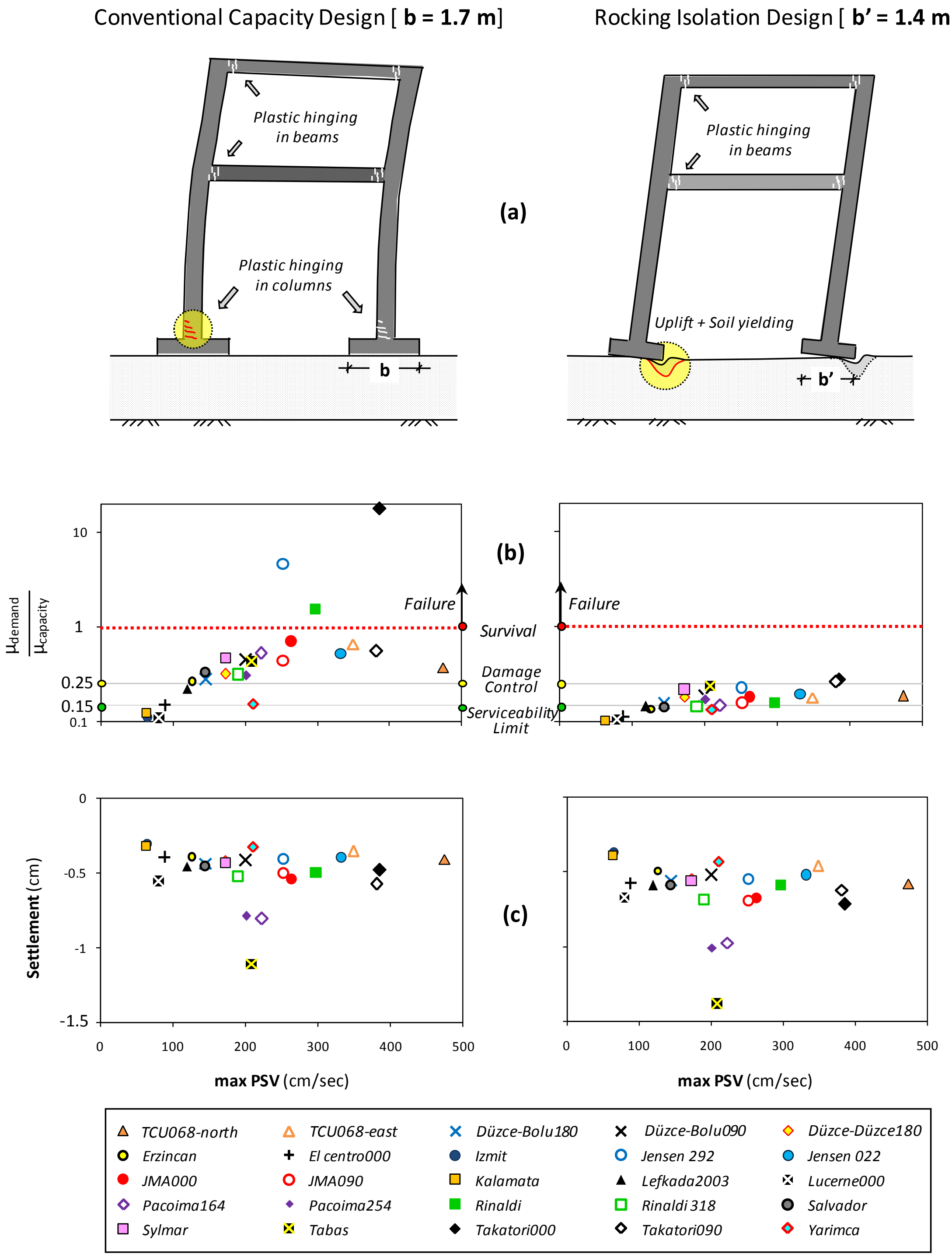

Figure 1. Comparison of the two design alternatives (conventional design versus rocking isolation design) : **(a)** Schematic illustration of the response; **(b)** Ductility consumption ratio; **(c)** Settlement w of the footing center against maximum spectral velocity SV of each earthquake.

As evidenced by these results, the response of the rocking-isolated alternative has been found to be advantageous in very strong seismic shaking, well in excess of the design limits (**Fig. 1b**) : the frame survives the earthquake demand sustaining non-negligible but repairable damage to its beams and non-structural elements (infill walls, etc.), while the residual settlements (due to soil yielding underneath the footing) were not substantially increased (**Fig. 1c**)

The aforementioned encouraging, though preliminary, results could by no means be endorsed with generalized applicability, before some possible limitations inherent to the concept have been explored. To this end, in the ensuing it is attempted to illustrate potential restrictions induced by uncertainties regarding the estimation of soil properties, as well as to investigate the possibility to extend the concept applicability to slightly more complex 2-storey 2-bay frames.

2. Analysis Methodology

The following sections compare the response of frames on conventionally designed foundations to that of their rocking-isolated counterparts. The superstructure is in all cases assumed to have been designed conventionally obeying the capacity-design principles prescribed by EC8 and has been performed utilizing the computer code ETABS [A^E = 0.36 g, q=3.5].

The soil-structure interaction problem is then analyzed utilizing the ABAQUS finite element (FE) algorithm (**Figure 2**), assuming plane-strain conditions, with due consideration to material (soil *and* superstructure) and geometric (*uplifting* and *P-Δ effects*) nonlinearities. Soil and footings are modeled with quadrilateral continuum elements, while nonlinear beam elements were used for the superstructure. To allow for detachment and sliding at the foundation-soil interface, appropriate gap-elements (of friction coefficient μ equal to 0.7) have been utilized. The lateral boundaries of the model are free to move horizontally so as to realistically reproduce the free-field kinematic soil response.

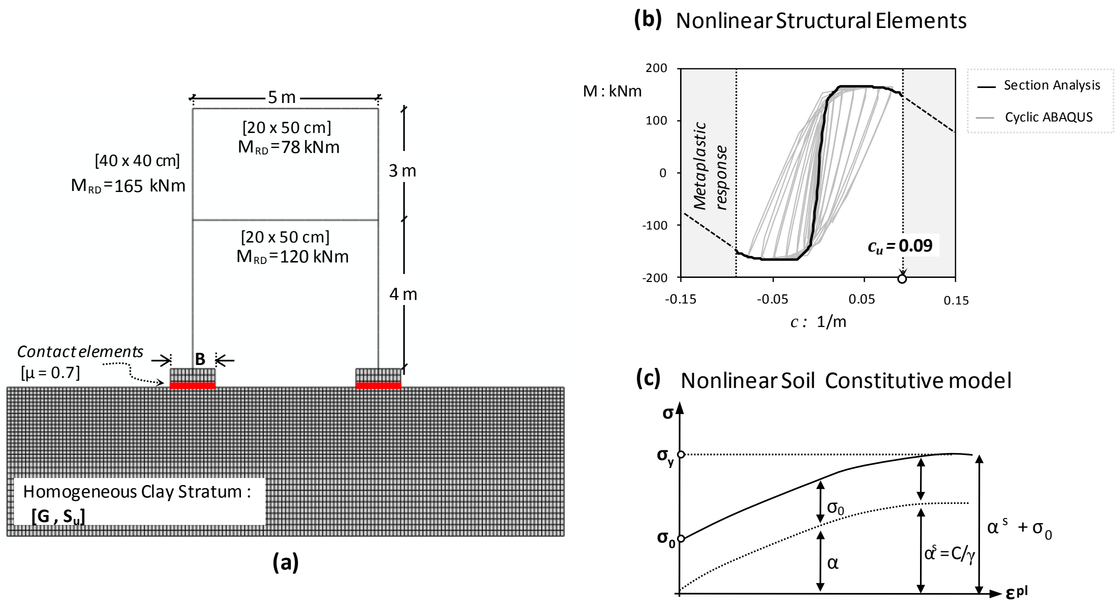

Figure 2. Outline of finite element modeling. Material (soil and superstructure) and geometric (uplifting and second order effects) are taken into account: (a) ABAQUS F.E. mesh, (b) description of the soil constitutive model and (c) simulation of the non-linear response of the structural members with properly calibrated kinematic hardening model**.**

Nonlinear soil behavior is modeled through a simple kinematic hardening model, with Von Mises failure criterion and associated flow rule [*Anastasopoulos et. al., 2011*]. This constitutive model is very efficient in capturing both the uplift-dominated response of the foundation (i.e $FS_V > 4$) and excessive soil yileding (when i.e.$FS_V < 4$).

The same kinematic hardening model, as suggested by *Gerolymos et al. (2005)*, is used to simulate the nonlinear moment–curvature response of the RC members of the superstructure. The constitutive model parameters are calibrated against the calculated by the section analysis software X-tract [*Imbsen Assoc*, *2004*] moment–curvature relationships under monotonic and cyclic lateral loading. As far as the metaplastic response (when $c_{max} > c > c_{ult}$) of each RC section is concerned, its residual bending moment (M_{res}) has been reasonably assumed to be equal to 30% of its bending moment capacity (*Vintzileou et al., 2007*), and is considered to be attained for a curvature 3 times larger (c_{max}) than the ultimate curvature c_{ult} calculated conventionally with X-tract.

The dynamic response of the system is simulated employing nonlinear dynamic time history analysis, applying the excitation time history at the base of the model. A quite comprehensive database of 24 recorded time-histories is used as an input to assess the seismic performance of the system at different earthquake scenarios. The selected records cover a wide range of strong-motion parameters such as PGA, PGV, SA, SV, frequency content, number of strong motion cycles, duration (*Kourkoulis et al., 2011*).

3. The effect of Uncertainties on the Estimation of Soil Properties

The efficiency of rocking isolation has been shown to greatly rely on the achieved safety factor against vertical loads (FS_v) which, in turn, strongly depends on the estimation of soil properties. Following this reasoning, the concept applicability may be questioned given that a possible over-prediction of the actual soil strength may finally lead to either augmented rocking-induced rotation (and subsequently enhanced distortion) or substantial soil yielding underneath the footing (and subsequently permanent deformations) even in case of a moderate earthquake scenario.

The effect of the possible over-prediction of the actual soil properties is investigated in this section considering an extreme example case. It is assumed that the soil actual strength is $S_u = 85\ kPa$ yielding a mere $FS_V \approx 3$, instead of the initially estimated Su=150kPa, (which yielded $FS_V \approx 5$). **Figure 3** compares the foundation response in these two cases ($FS_V = 3$ *and* $FS_V = 5$) for two earthquake events (**Fig. 3b**):

(i) A moderately strong seismic shaking. As such we have used the El Centro 1940 record, with a PGA of 0.31 g and a maximum SA of 0.9g (Figure 2, black dashed line) which fits the design spectrum, while it contains a multitude of strong motion cycles capable of producing rather augmented settlements. In this case the effectiveness of the design is judged mainly in terms of serviceability after the end of the earthquake.

(ii) A very strong seismic shaking. As such the devastating Takatori time history, which is considered one of the worst seismic motions ever recorded, exceeding (in terms of SA) the design by a factor of at least 2 over the whole period range, has been selected. This excitation corresponds to the case of an "unanticipated" event that substantially exceeds the design spectrum of the frame.

As evidenced by **Figure 3b,** indeed, when excited by the El Centro record, the *actual footing response* ($FS_V \approx 3$ instead of the assumed $FS_V \approx 5$) deviates substantially from the one *anticipated*: the footing keeps accumulating settlement at each cycle until an ultimate value of 1.7 cm which is considerably higher than the maximum expected settlement (that would not exceed 0.5 cm). Still however, the absolute magnitude of settlements is relatively low and does not jeopardize the structural integrity of the frame.

The most noteworthy discrepancies arise when the frame is subjected to the Takatori record. In both safety factor scenarios (**Figures 3b and c**), the footings develop significant rotations either due to uplifting with limited settlement (for $FS_V \approx 5$) or due to uplifting accompanied by noticeable soil yielding which unavoidably is associated with accumulation of permanent settlements. Although the uplifting-dominated response ensures the self centering of the footing which after the shaking remains nearly horizontal, the actual response leads to gradual development of irrecoverable residual rotation (**Fig. 4a**). This brings about residual rotational drift of the order of 8 cm as opposed to only 3 cm when $FS_V = 5$ (**Fig. 4b**). The observed flexural drift however is slightly lower (although in both cases maintained between the safety threshold of 1%). The latter is attributed to the lower moment capacity of the footing which bounds the amount of earthquake demand transmitted to the superstructure and thus limits the flexural distress of the column. Still, it should be noted that although the collapse is avoided (as opposed to a conventionally designed frame) through either of the two identified mechanisms, uplifting is definitely preferable than soil yielding since the latter would possibly result in unacceptable permanent deformation of the structure (i.e. irrecoverable damage to non-structural members).

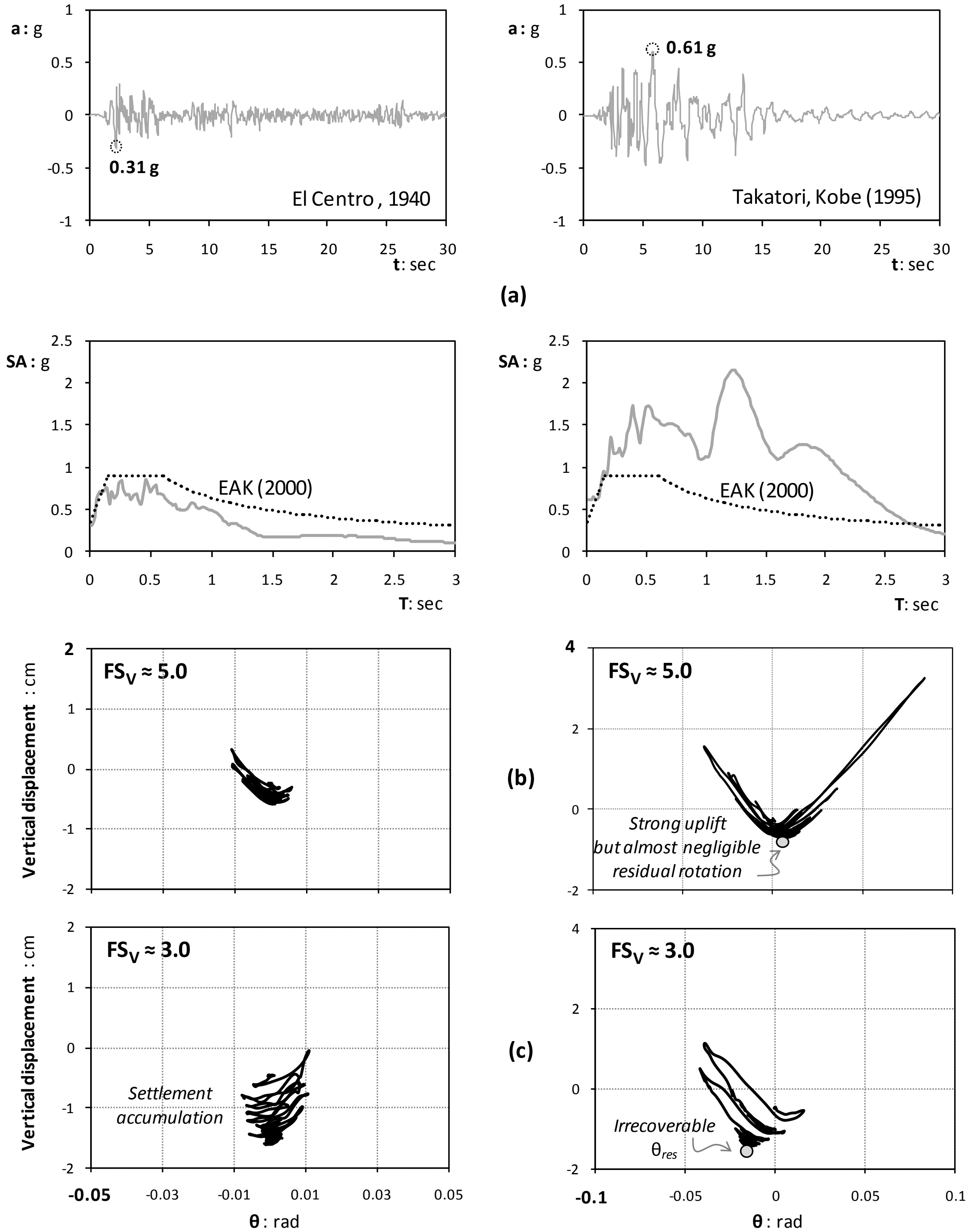

Figure 3. Investigation of the effect of Uncertainties on the estimation of soil properties for two seismic scenarios: (a) the moderately strong El Centro, 1940 record (left column) and the

extremely strong Takatori, 1995 earthquake scenario (right column). The frame response is studied in terms of Settlement-rotation loops of two identical rocking isolated frames (with footing width B = 1.4 m) founded on soil strata of different S_u : (b) S_u=150 kPa (resulting in FSv ≈ 5.0) and (b) and S_u=85 kPa (resulting in FSv ≈ 3.0).

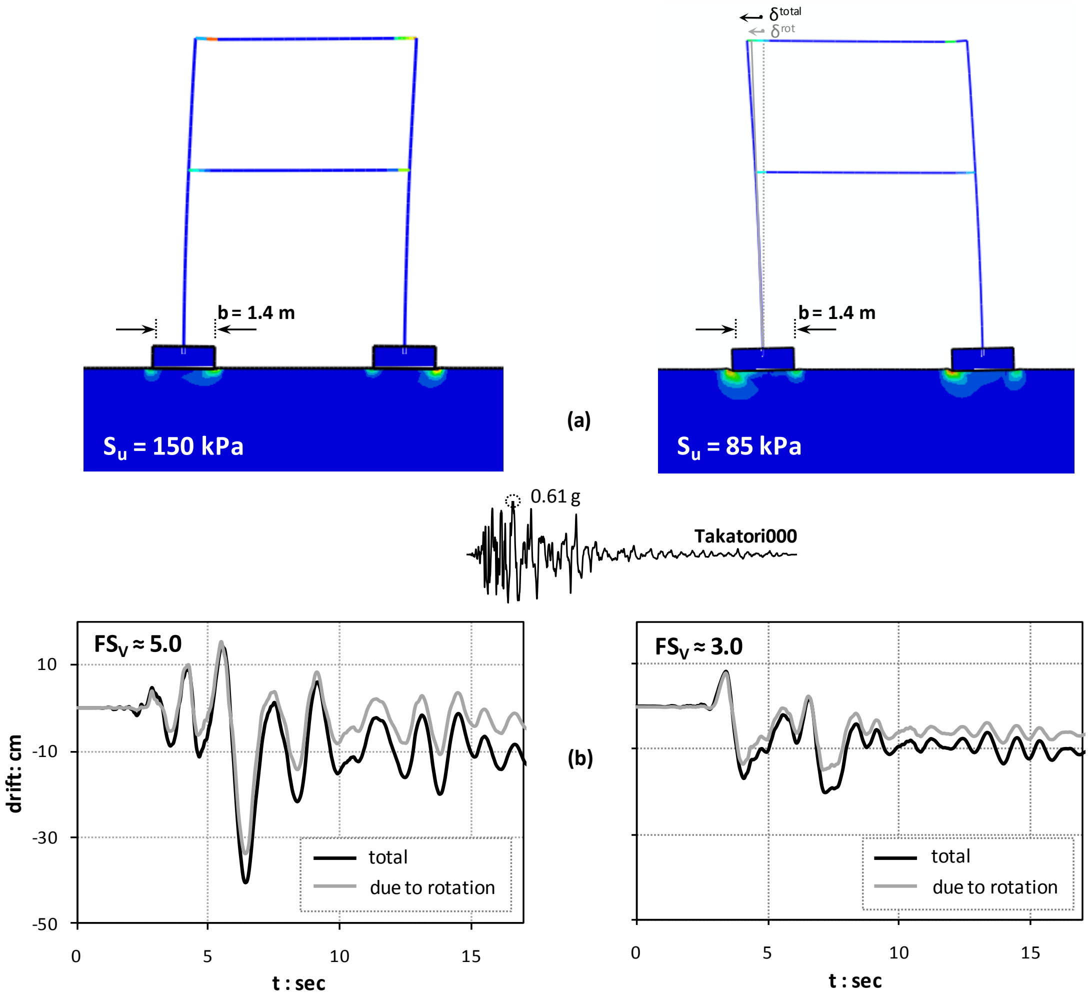

Figure 4. Investigation of the effect of Uncertainties on the estimation of soil properties: (a) Plastic Strain distribution of two identical frames founded on isolated (under-dimensioned) footings of b = 1.4 m when the supporting ground strength varies from S_u = 150 kPa (left figure) S_u = 85 kPa (right). Both frames are excited by the Takatori record (Kobe, 1995). (b) Time history of the ground floor drift. The black line corresponds to the total drift, and the grey line represents the drift associated with foundation rotation.

4. Application of the Rocking–Isolation Approach in case of 2-bay Frames

This section explores the applicability of rocking isolation design approach on a more complex 2 bay 2-storey frame structure on isolated square footings. The soil has been assumed homogeneous clay of undrained shear strength S_u = 150 kPa. The superstructure has been designed conventionally as described previously.

Following the code provisions, the minimum acceptable footing dimension was calculated to be B = 1.80 m, while in the rocking-isolated alternative, the footings had dimensions B’ = 1.30 m. The latter yields a safety factor against vertical loads $FS_V^{side} \approx 5.31$ for the two side footings (under the static loads combination 1.35g+1.5q) and therefore –according to the previous statements– it is expected to provoke intense rocking. The major difference between this case and the 1-bay frame, stems from the *un-symmetry* in the distribution of axial loads on the three columns; the middle column carries double the axial load of the two side ones. Given that all footings have the same dimensions, the different load results in lower safety factor of the middle column which equal to $FS_V^{middle} \approx 3.5$. A characteristic snapshot of the dynamic analysis of the two frames is portrayed in **Figure 5a.**

4.1. Response to earthquake Records

Figure 5b collectively portrays the response of the two design alternatives to the recorded time-histories database referred to previously. The left-column distress is expressed in terms of ductility demand over ductility capacity ratio (μ_{demand} /$\mu_{capacity}$), and, once more reflects the very favorable effect of rocking isolation.

For the conventional frame the μ_{demand} /$\mu_{capacity}$ ratio averages around 0.5, while even the lateral columns (that as mentioned previously are less distressed), consume a substantial portion of their available ductility during all strong earthquake scenarios. Besides, for more than 35 % of the excitation motions the μ_{demand} /$\mu_{capacity}$ ratio surpasses 1.0 which indicates total flexural failure of the central column and probable collapse of the frame. On the other hand, the unconventional alternative has safely sustained all examined earthquake scenarios, and in fact with minimal bending distortion on the columns (the μ_{demand} /$\mu_{capacity}$ ratio fluctuates around an average value of 0.1).

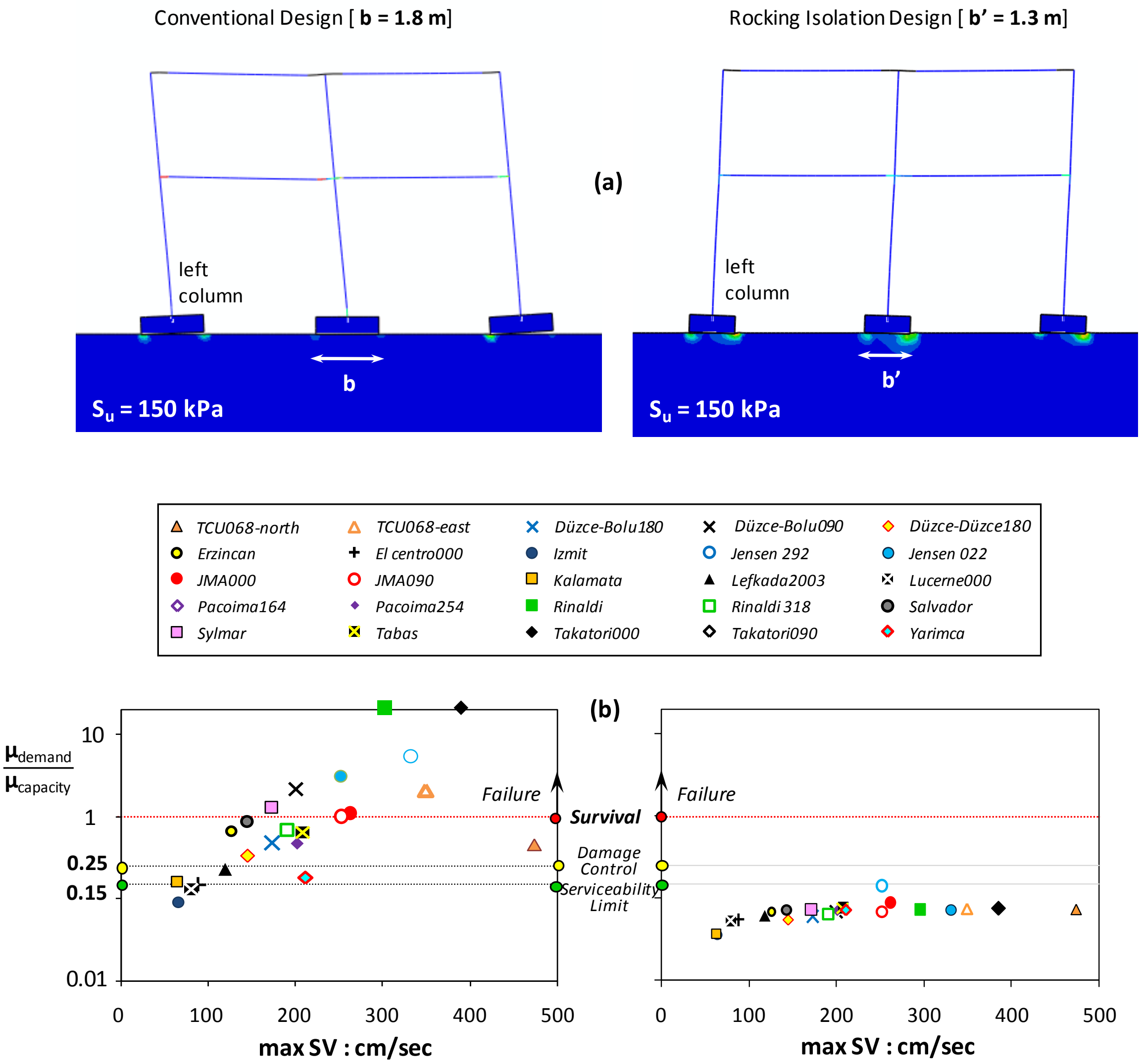

Figure 5. Application of the "Rocking-Isolation" Concept to 2-bay 2-storey frames. The rocking foundation alternative (b' = 1.3 m) is compared with its conventional counterpart (b = 1.8 m) in

terms of (a) plastic strain distribution when the two frames are excited by the Takatori record (Kobe, 1995) and (b) Ductility consumption ratios for various earthquake scenaria (described by their peak Spectral Velocity SV).

This picture is slightly modified when examining the non-structural distortion of the frame, expressed by the total (combined flexural and rotational) residual drift ratio of the ground floor. Two particular cases have been identified where the distortion of the frame is rather increased (**Figure 6**). These cases refer to the frame being subjected to *directivity "contaminated"* motions (*Abrahamson,2000; Sommerville, 2000; Mavroeidis and Papageorgiou, 2003*), namely the Jensen-292 and the Rinaldi-228 accelerograms (**Figure 7a**) recorded during the Northridge 1994 earthquake. Such motions contains a single coherent large period pulse which usually results to substantial impulsive deformation demand of the system (*Hall et al, 1995; Anderson et al, 1999; Makris and Chang, 2000; Chopra and Chintanapakdee, 2001; Alavi and Krawinkler, 2001*).

Remarkably, such phenomena are not evident in case of the symmetric 1-bay frame even when excited by these motions. This latter system, is able to recover even a highly impulsive displacement (produced by the directivity-affected motion) due to the inherent self-centering behavior of the rocking foundation (**Figure 7b**). Notice that while in this symmetric system both footings respond identically, this is not true when referring to the non-symmetric 2-bay frame. Since the middle footing carries double axial load than the two side ones, its rotation –when subjected to the strong motion pulse- produces a very significant soil yielding underneath it compared to that of the side footings whose rotation is primarily materialized through uplifting. Indeed, observe that although all three footings develop rotations of very similar magnitude **(Figure 7b**), the settlements do not follow the same pattern: after t = 6s the middle footing has permanently settled by about 1 cm (**Figure 7c)** which may not be retrieved during subsequent cycles of motion. Unavoidably, this plastic soil deformation produces an irrecoverable rotation of the footings of θ_{res} =0.03 rad.

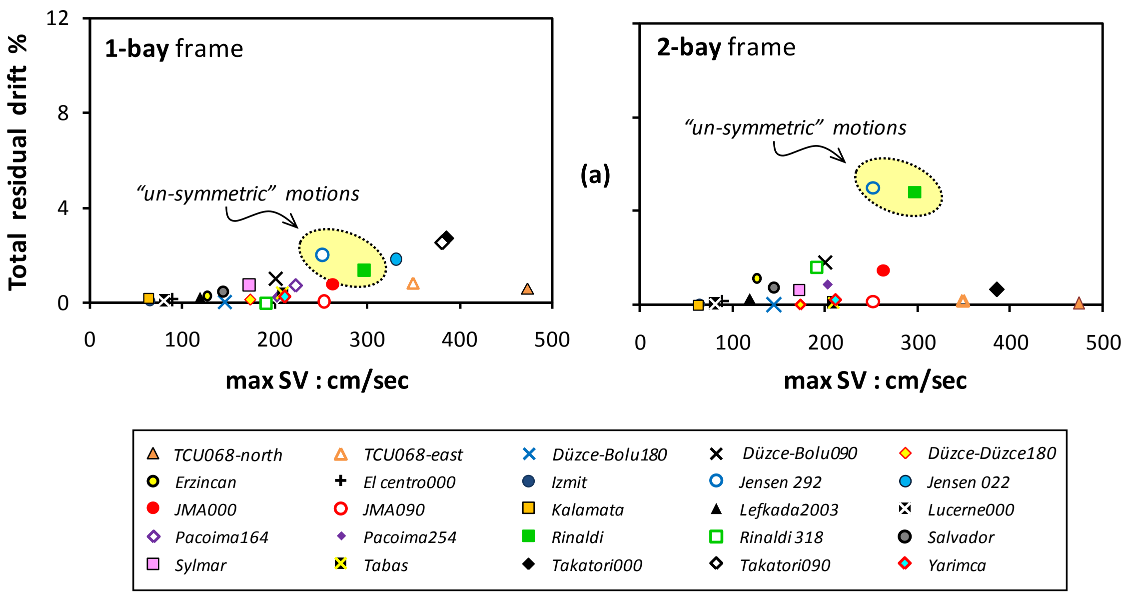

Figure 6. Collective results on the effectiveness of the "*Rocking-Isolation*" concept to 1-bay (left column) and 2-bay frames (right column): total residual drift ratio of the first floor as a function of the peak spectral velocity *SV* of each earthquake scenario.

5. Conclusions

This paper has incorporated non-linear finite element analyses to investigate the effectiveness of rocking-isolation in case the actually achieved Factor of Safety is substantially lower than the one initially required to provoke the desired uplifting behavior. It has been manifested that even in such a case involving considerable soil yielding underneath the footings, the effect of foundation under-design remains positive especially when it comes to severe ground shaking.

The applicability of the concept has been also further extended to the case of a more complex 2-bay 2-storey frames subjected to strong seismic shaking. In the course of this analysis a number of differences have been recognized between the response of 2-bay and simple 1-bay frames. The main discreapancy is associated with the response of the middle column which, due to its higher axial load experiences increased settlement compared to the two side-ones. This produces residual irrecoverable deformation when the frame is excited by motions affected by forward-directivity, but does not question the superiority of rocking-isolation over conventional foundation design.

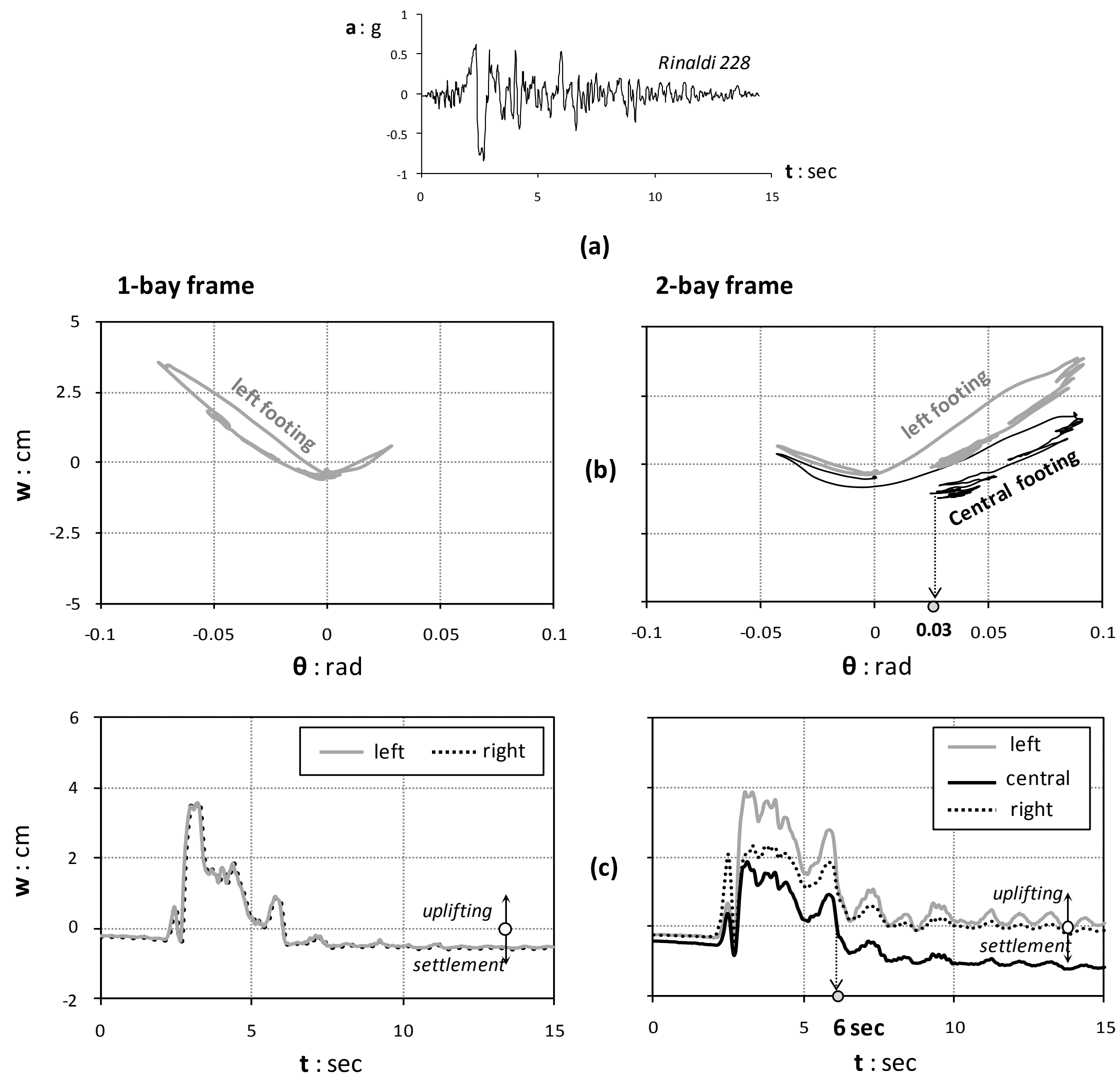

Figure 7. Comparison of the response of 1-bay (left column) and 2-bay (right column) "rocking isolated" frames when excited by the "un-symmetric" Rinaldi228 (Northridge, 1994) motion: **(a)** the earthquake record, **(b)** settlement-rotation loops and **(c)** time history of the vertical displacement at the center of each footing.

6. Acknowledgement

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