A complete simplified framework in determination of seismic racking response of shallow buried structures

A.G. Özcebe
European School for Advanced Studies in Reduction of Seismic Risk, ROSE School-IUSS, Pavia, Italy

K.Ö. Çetin
Department of Civil Engineering, Middle East Technical University, Ankara, Turkey

SUMMARY
Recently, an alternative application has been proposed to determine soil strains or displacements from non-linear shear mass participation factors (Bilotta et al., 2007; Gingery, 2007). Yet, the accuracy of this procedure has not been studied in detail. This paper aims at providing a sensitivity study for Çetin and Seed (2004) r_d method by comparing its predictions with the results obtained from a set of 2880 different 1-D equivalent linear site-response analyses carried out in software STRATA (Kottke and Rathje, 2009). This work also provides the practising engineers with a complete and relatively straightforward framework in the seismic design of shallow underground structures with simplified method making use of r_d factor developed by Çetin and Seed (2004) and quasi-static simplified frame analysis (SFA) method in determining soil-structure interaction developed by Wang (1993). This paper also includes the possible implementation of structural non-linearity into the available SFA methods in the literature.

Keywords: seismic design, shallow buried structures, non-linear shear mass participation factor

1. INTRODUCTION

Unlike a "regular" structure supported by shallow foundations, deformation response of a buried structure is more dependent on complex soil-structure-earthquake interaction. Depending on the level of this interaction, several different modes of deformation may be exhibited by the buried structure [Owen & Scholl (1981)]. Among them, in-plane deformation mode due to vertically propagating S-waves has been widely referred to as the most critical one [Wang (1993), Penzien & Wu (1998), Penzien (2000), Huo et al. (2006), Bobet et al. (2008), Özcebe (2009)]. This mode of deformation is called “racking” for and “ovaling” for rectangular and circular sections, respectively (Figure 1).

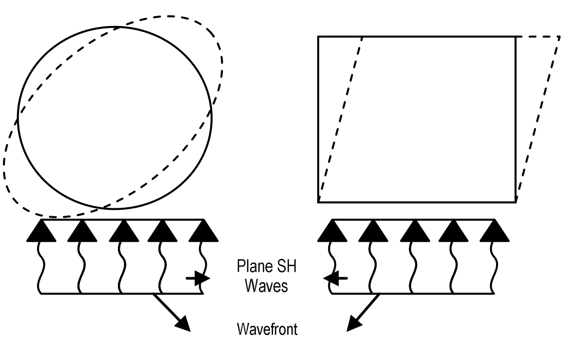

Figure 1. Ovaling and racking type of deformations for buried structures with circular and rectangular geometry

In the chronological development of the evaluation for racking/ovaling deformations, researchers initially ignored both the kinematic and inertial components of soil-structure interaction, and assumed that the embedded structure will deform compatibly with the "free field" soil layer. Newmark (1967) and Kuesel (1969) provided analytical closed-form solutions for the estimation of free-field strains in the soil medium subjected to harmonic waves. St. John & Zahrah (1987) further developed the formulations by adding shear and Rayleigh wave patterns. Later, Hendron & Fernandez (1983) and Merritt et al. (1985) pointed out that in the cases of presence of a cavity inside the soil continuum (analogous to soft structure - stiff soil conditions), free-field approach severely under predicts the displacement demands in the vicinity of the cavity and they suggested the use of a more rigorous (e.g. finite element based seismic soil-structure interaction model) assessments techniques. As part of this progress, a simplified framework (Simplified Frame Analysis, SFA) to assess racking displacements of buried structures is proposed on the basis of quasi-static soil-structure interaction solutions [Wang (1993)]. Figure 2 schematically expresses racking response of a buried structure as compared with free field response of a free field soil site. Amplification or de-amplification of the displacement demand on the buried structure as compared with free field displacements (the ratio of which is defined as R: racking ratio) is expressed as a function of lateral relative flexibility of the replaced soil with respect to the immersed structure (the ratio of which is defined as F: flexibility coefficient).

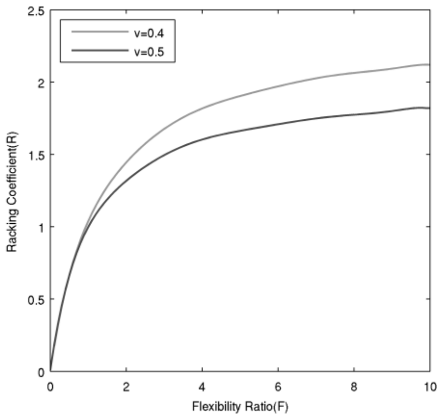

Figure 2. Racking coefficient (R) versus flexibility ratio (F) (redrawn after Wang, 1993)

Formulation of the methodology for rectangular tunnels is presented through Eq. (1.1) to (1.2).

$$F = \frac{G_m L}{S_1 H} \qquad (1.1)$$

$$\Delta = R \cdot \Delta_{FF} \qquad (1.2)$$

Where;
G_m= Strain compatible shear modulus of the soil continuum; L= Span of the embedded tunnel; S_1= Lateral structural stiffness; H= Height of the structure; Δ= Displacement demand on the structure of the combined system; R= Racking ractor; Δ_{FF}= Free-field displacement demand on the structure in free-field conditions

Racking factor presented in Eq. (1.2) can be determined from Fig. 2 for a given Poisson's ratio and flexibility ratio if the following assumptions of the SFA framework [Wang (1993)] are fulfilled:

- Rigid base (i.e. base rock) is overlaid by the horizontal soil profiles,
- Soil profile consisting of stiffer layers are overlaid by softer layers,
- Linear elastic structural frame, and
- No slip condition between the soil-structure interface

Benefitting from the original SFA framework, several improved versions have been developed [Penzien & Wu (1998), Penzien (2000), Huo et al. (2006)]. Bobet et al. (2008) pointed out that strain compatible shear modulus of the surrounding medium is not same as free-field strain compatible shear modulus due to free-field strains affected by soil-structure interacting system. They have proposed an iterative scheme to estimate strain compatible shear moduli values, separate sets for the free field and soil-structure interacting systems. Recently, Özcebe (2009) proposed an alternative framework to assess racking ratios benefitting from Winkler springs and non-linear shear mass participation (r_d) concepts. Among all those developments in this area, only the use of shear mass participation factor has not been discussed in detail in the last decade. In fact; only few research works are present [Bilotta et al. (2007), Gingery (2007)]. Up to the knowledge of the authors, only Bilotta et al. (2007) mentions about the implementation of SFA methodology by taking soil-structure interaction into account.

Current work is devoted to provide a complete simplified framework, further improve the methodology suggested by Bilotta et al. (2007) and Gingery (2007) approaches by taking soil-structure interaction with structure non-linearity, and illustrate the accuracy of the simplified methodology by numerous case studies.

2. ADOPTED METHODOLOGY

Showing similarities with Bilotta et al. (2007) and Gingery (2007), general framework adopted to compute simplified analyses are listed by the bullet points defined below:

- Obtaining PGA_{rock} at the site considered (preferably from PSHA or hazard maps),
- Estimating PGA_{soil} by a code-specified or any scientifically approved procedure (preferably from the amplification factors defined at intensity and $V_{s,30}$ sensitive NGA relationships),
- Finding shear mass participation factor (r_d) from Çetin and Seed (2004) formulation by using Eq. (2.1) to Eq. (2.4):

For d<20 m (≈65 ft):

$$r_d(d, M_w, a_{max}, V_{s,12m}) = \frac{1+\dfrac{-23.013-2.949a_{max}+0.999M_w+0.0525V_{s,12m}}{16.258+0.201e^{0.341(-d+0.0785V_{s,12m}+7.586)}}}{1+\dfrac{-23.013-2.949a_{max}+0.999M_w+0.0525V_{s,12m}}{16.258+0.201e^{0.341(0.0785V_{s,12m}+7.586)}}} \mp \sigma_e \qquad (2.1)$$

Where; d= Depth from the ground surface; M_w= Moment magnitude; a_{max}= Maximum horizontal acceleration (rock outcrop); $V_{s,12m}$= Average shear wave velocity of first 12 m; e= Exponential; σ_e= Error term

For d≥20 m (≈65 ft):

$$r_d(d, M_w, a_{max}, V_{s,12m}) = \frac{1+\dfrac{-23.013-2.949a_{max}+0.999M_w+0.0525V_{s,12m}}{16.258+0.201e^{0.341(-20+0.0785V_{s,12m}+7.586)}}}{1+\dfrac{-23.013-2.949a_{max}+0.999M_w+0.0525V_{s,12m}}{16.258+0.201e^{0.341(0.0785V_{s,12m}+7.586)}}} - 0.0046(d-20) \mp \sigma_e \qquad (2.2)$$

In Eq. (2.1) and Eq. (2.2), error term (σ_e) is defined as in Eq. (2.3) and Eq. (2.4):

For d<12 m (≈40 ft):

$$\sigma_e(d) = d^{0.850}0.0198 \qquad (2.3)$$

For d≥12 m (≈40 ft):

$$\sigma_e(d) = 12^{0.850}0.0198 \qquad (2.4)$$

- Obtaining maximum shear stress at the center of gravity of the soil layers at which the structure is planned to be constructed by using Eq. (2.5).

$$\tau_{max,rd} = \frac{a_{max}}{g} \gamma h r_d \qquad (2.5)$$

Where; $\tau_{max,rd}$= Maximum shear stress of deformable soil column at depth d; a_{max}= Peak ground acceleration (soil free surface, in g's); g= Earth's gravitational acceleration; h= Depth of deformable soil column (=d); r_d= Non-linear shear mass participation factor developed to represent soil deformability ; γ= Unit weight

- Finding small strain shear modulus of the soil layers at which the structure is planned to be constructed by using Equation (2.6).

$$G_{max} = V_s^2 \times \rho \qquad (2.6)$$

Where; G_{max}= Initial (small-strain) shear modulus; V_s= Shear wave velocity; ρ= Soil density

- Iterating cyclic shear strain through each layer under interest until error term is small enough (< 1-2% is recommended)

  <u>At any step i and for any layer j which is in contact with the structure</u>:

- Maximum shear strain is obtained by using Eq. (2.7):

$$\gamma^j_{max,i} = \gamma^j_{c,i} / 0.65 \qquad (2.7)$$

Where; $\gamma^j_{max,i}$= Maximum shear strain in j[th] layer at iteration step i; $\gamma^j_{c,i}$= Cyclic shear strain in j[th] layer at iteration step i.

- Equivalent linear strain dependent shear modulus is found by Eq. (2.8):

$$G^j_{eq}(\gamma_{c,i}) = \frac{G^j_{eq}}{G^j_{\max}}(\gamma^j_{c,i}) \times G^j_{\max} \qquad (2.8)$$

Here; $G^j_{eq}(\gamma^j_{c,i})$= Cyclic shear compatible equivalent linear modulus of j[th] layer at step i; G^j_{max},= Initial (small-strain) shear modulus of j[th] layer, its value is obtained from Eq. (2.6) for any j[th] layer.

- Maximum shear stress is obtained by Eq. (2.9):

$$\tau^j_{\max}(\gamma^j_{c,i}) = G^j_{eq}(\gamma^j_{c,i}) \cdot \gamma^j_{\max,i} \qquad (2.9)$$

Where; $\tau^j_{max}(\gamma^j_{c,i})$= Cyclic shear compatible maximum shear stress in j[th] layer at step i

- Error term is the relative difference between the shear stresses found by Eq. (2.5) and Equation (2.9), as illustrated in Eq. (2.10):

$$\varepsilon^j_i = \frac{\tau^j_{\max,rd} - \tau^j_{\max}(\gamma^j_{c,i})}{\tau^j_{\max,rd}} \qquad (2.10)$$

Where; $\tau^j_{max,rd}$= Maximum shear stress obtained by r_d method, found from Eq. (2.5) in layer j; ε^j_i= Relative error of j^{th} considered at step i

- Having reached the convergence with a desired level of accuracy at the previous step, free-field deformation can be calculated as defined in Eq. (2.11) and Eq. (2.12)

$$\Delta_{FF} = \sum_{j=1}^{j=n} [\gamma^j_{\max} \times h_j] \quad (2.11)$$

Where; γ^j_{max}= Maximum shear strain in j^{th} layer after obtaining reasonable convergence at Eq. (2.10); h_j= Layer thickness for j^{th} layer (only the part that is in contact with the structure)

$$\sum_{i=1}^{i=n} h_i = H \qquad (2.12)$$

Where; H= Height of the building.

- Shear modulus of the continuum is the equivalent shear modulus found at the end of the iteration at two steps before. In case of the presence of multiple soil layers around the structure, then representative soil shear modulus may be obtained by Equation (2.13):

$$G_s = 0.5 \times (G_{top} + G_{bottom}) \qquad (2.13)$$

Where; G_s= Equivalent shear modulus of the soil medium surrounding the structure; G_{top}= Cyclic strain compatible equivalent linear shear modulus of the extreme top layer that is in contact with the structure, its value is obtained from Eq. (2.8) whenever Eq. (2.10) shows reasonable convergence; G_{bottom}= Cyclic strain compatible equivalent linear shear modulus of the extreme bottom layer that is in contact with the structure, its value is obtained from Eq. (2.8) whenever Eq. (2.10) shows reasonable convergence

- After the previous step, all soil related parameters have been obtained, all needed is to find structure related parameter (S_1: structural stiffness). This may be done by several methods via structural analysis programs or via analytical methods. In order to make the proposed scheme, ready to apply without any sophisticated use, authors propose the use of widely popular Matrix Stiffness Method in this paper. If matrix stiffness method is formulated on a 9 degree of freedom system of single barrel buried structure (Figure 3) following matrix occurs as stiffness matrix (Eq. 2.14).

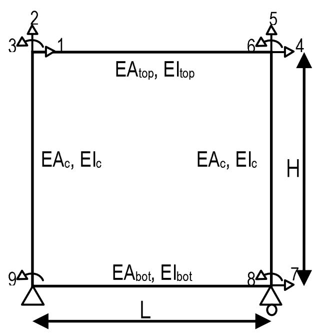

Figure 3. 9-dof single-storey, single-bay rectangular shallow buried structure

$$[K^G]=\begin{bmatrix}
\frac{EA_{top}}{L}+\frac{12EI_c}{H^3} & 0 & \frac{6EI_c}{H^2} & -\frac{EA_{top}}{L} & 0 & 0 & 0 & 0 & \frac{6EI_c}{H^2} \\
0 & \frac{12EI_{top}}{L^3}+\frac{EA_c}{H} & \frac{6EI_{top}}{L^2} & 0 & -\frac{12EI_{top}}{L^3} & \frac{6EI_{top}}{L^2} & 0 & 0 & 0 \\
\frac{6EI_c}{H^2} & \frac{6EI_{top}}{L^2} & \frac{4EI_{top}}{L}+\frac{4EI_c}{H} & 0 & -\frac{6EI_{top}}{L^2} & \frac{2EI_{top}}{L} & 0 & 0 & \frac{2EI_c}{H} \\
-\frac{EA_{top}}{L} & 0 & 0 & \frac{EA_{top}}{L}+\frac{12EI_c}{H^3} & 0 & \frac{6EI_c}{H^2} & -\frac{12EI_c}{H^3} & \frac{6EI_c}{H^2} & 0 \\
0 & -\frac{12EI_{top}}{L^3} & -\frac{6EI_{top}}{L^2} & 0 & \frac{12EI_{top}}{L^3}+\frac{EA_c}{H} & -\frac{6EI_{top}}{L^2} & 0 & 0 & 0 \\
0 & \frac{6EI_{top}}{L^2} & \frac{2EI_{top}}{L} & \frac{6EI_c}{H^2} & -\frac{6EI_{top}}{H^2} & \frac{4EI_{top}}{L}+\frac{4EI_c}{H} & -\frac{6EI_c}{H^2} & \frac{2EI_c}{H} & 0 \\
0 & 0 & 0 & -\frac{12EI_c}{H^3} & 0 & -\frac{6EI_c}{H^2} & \frac{12EI_c}{H^3}+\frac{EA_{bot}}{L} & -\frac{6EI_c}{H^2} & 0 \\
0 & 0 & 0 & \frac{6EI_c}{H^2} & 0 & \frac{2EI_c}{H} & -\frac{6EI_c}{H^2} & \frac{4EI_c}{H}+\frac{4EI_{bot}}{L} & \frac{2EI_{bot}}{L} \\
\frac{6EI_c}{H^2} & 0 & \frac{2EI_c}{H} & 0 & 0 & 0 & 0 & \frac{2EI_{bot}}{L} & \frac{4EI_{bot}}{L}+\frac{4EI_c}{H}
\end{bmatrix} \quad (2.14)$$

Where; $[K^G]$= Global stiffness matrix for structure; E= Modulus of elasticity; H= Height of the structure; L= Width of the structure; A_{top}= Area of the top slab (for unit length along the out of plane direction); A_{bot}= Area of the bottom slab (for unit length along the out of plane direction); I_c= Cracked inertia of a single vertical column (for unit length along the out of plane direction); I_{top}= Cracked inertia of top slab (for unit length along the out of plane direction); I_{bot}= Cracked inertia of bottom slab (for unit length along the out of plane direction)

- To provide a neat solution, static condensation can be used to condense 9x1 displacement vector into 2x1, provided that master degrees of freedoms are selected as 1 and 4 (Eqs. 2.15-2.22). Equations 2.23 and 2.24 are used to obtain stiffness of the structure.

$$K_{mm}=\begin{bmatrix} K_{11}^G & K_{14}^G \\ K_{41}^G & K_{44}^G \end{bmatrix} \quad (2.15) \qquad K_{ms}=\begin{bmatrix} K_{12}^G & K_{13}^G & K_{15}^G & K_{16}^G & K_{17}^G & K_{18}^G & K_{19}^G \\ K_{42}^G & K_{43}^G & K_{45}^G & K_{46}^G & K_{47}^G & K_{48}^G & K_{49}^G \end{bmatrix} \quad (2.16)$$

$$K_{sm}=\begin{bmatrix} K_{21}^G & K_{24}^G \\ K_{31}^G & K_{34}^G \\ K_{51}^G & K_{54}^G \\ K_{61}^G & K_{64}^G \\ K_{71}^G & K_{74}^G \\ K_{81}^G & K_{84}^G \\ K_{91}^G & K_{94}^G \end{bmatrix} \quad (2.17) \qquad K_{ss}=\begin{bmatrix} K_{22}^G & K_{23}^G & K_{25}^G & K_{26}^G & K_{27}^G & K_{28}^G & K_{29}^G \\ K_{32}^G & K_{33}^G & K_{35}^G & K_{36}^G & K_{37}^G & K_{38}^G & K_{39}^G \\ K_{52}^G & K_{53}^G & K_{55}^G & K_{56}^G & K_{57}^G & K_{58}^G & K_{59}^G \\ K_{62}^G & K_{63}^G & K_{65}^G & K_{66}^G & K_{67}^G & K_{68}^G & K_{69}^G \\ K_{72}^G & K_{73}^G & K_{75}^G & K_{76}^G & K_{77}^G & K_{78}^G & K_{79}^G \\ K_{82}^G & K_{83}^G & K_{85}^G & K_{86}^G & K_{87}^G & K_{88}^G & K_{89}^G \\ K_{92}^G & K_{93}^G & K_{95}^G & K_{96}^G & K_{97}^G & K_{98}^G & K_{99}^G \end{bmatrix} \quad (2.18)$$

$$F_m = < 0.5 \quad 0.5 >^T \quad (2.19) \qquad F_s = < 0 \quad 0 \quad 0 \quad 0 \quad 0 \quad 0 \quad 0 >^T \quad (2.20)$$

$$[K_{mm}^*]=[K_{mm}]-[K_{ms}][K_{ss}]^{-1}[K_{sm}] \quad (2.21) \qquad [F_m^*]=[F_m]-[K_{ms}][K_{ss}]^{-1}[F_s] \quad (2.22)$$

$$[\Delta_m^*]=[K_{mm}^*]^{-1}[F_m^*] \quad (2.23) \qquad S_1 = 1/\Delta_1^* \quad (2.24)$$

Where; K^G_{ij}= Component of i[th] row and j[th] column of global stiffness matrix presented in 2.14; s= Slave degree of freedom, m= Master degree of freedom; F= Force vector; K^*_{mm}= Condensed stiffness matrix; F^*_m= Condensed force vector; Δ^*_m= Structure displacement due to unit force acting on the top slab

3. EQUIVALENT LINEAR SITE RESPONSE ANALYSES

3.1. Determination of random soil profiles

Determination of random soil profiles can be categorized into three steps:

(i) Following the framework provided by Toro (1995), total of 18 median depth models (9 for medium stiff soil and 9 for soft soil) are created (by assuming 6 discrete soil sub-layers) and their shear wave velocity profiles are determined

(ii) Soil types are assigned to each predefined layer model (at *step (i)*) by using uniform distribution with coefficients 0.4, 0.15, 0.15, 0.15, 0.15 for sand, clay with PI=5, clay with PI=15, clay with PI=25 and clay with PI=35, respectively

(iii) Total of 18 shear wave velocity profiles (9 for soft and 9 for medium stiff) are randomized and obtained 360 (20 randomizations for each 18 median shear wave velocity profiles from obtained from *step (i)*)by using a standard deviation, σ=0.10 in STRATA [Kottke and Rathje (2009)] and modulus degradation and damping curves are assigned.

3.1.1. Determination of depth model with shear wave velocity realizations

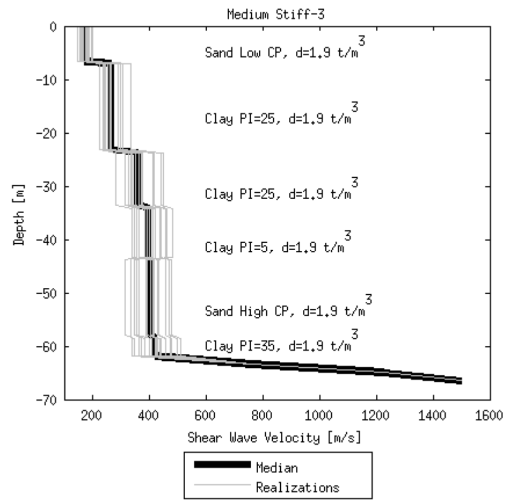

Figure 4. An example 6 soil sub-layer model with 20 shear wave velocity realizations

3.1.2. Soil models

Table 3.1. Soil models used in equivalent-linear site response analyses

Soil type	Modulus Degradation Curve (G/G_{max})	Damping Ratio Curve (DR)
Sand (σ'_m<1ksf)	Sun et al. (1988) with (σ'_m<1ksf, PI=0-10)	Seed and Idriss (1970) with (Average)
Sand (σ'_m=1-3ksf)	Sun et al. (1988) with (1 ksf≤σ'_m≤3 ksf, PI=0-10)	Seed and Idriss (1970) with (Average)
Sand (σ'_m>3ksf)	Sun et al. (1988) with (σ'_m>3 ksf, PI=0-10)	Seed and Idriss (1970) with (Lower)
Clay PI= 5	Vucetic and Dobry (1991) with PI=5	Vucetic and Dobry (1991) with PI=5
Clay PI= 15	Vucetic and Dobry (1991) with PI=15	Vucetic and Dobry (1991) with PI=15
Clay PI= 25	Vucetic and Dobry (1991) with PI=25	Vucetic and Dobry (1991) with PI=25
Clay PI= 35	Vucetic and Dobry (1991) with PI=35	Vucetic and Dobry (1991) with PI=35

3.1.3. Earthquake records

Variability in the seismic input was simulated by 8 rock outcrop acceleration ground motions, selected from PEER/NGA Strong Ground Motion Database. A list of these records was presented in Table 3.2.

Table 3.2. Earthquake set used in equivalent linear site response analyses

Earthquake	Station	File Name	Mw	D	PGA_{scaled}
Kocaeli (Turkey)	Izmit	IZT 090.AT2	7.5	1.000	0.22g
Chi-Chi (Taiwan)	ILA063	ILA063-N.AT2	7.6	1.000	0.09g
Chi-Chi (Taiwan)	HWA003	HWA003-N.AT2	7.6	1.000	0.14g
Coyote Lake (USA)	Gilroy Array #1	G01320.AT2	5.7	1.000	0.13g
Denali (Alaska)	Carl Carlo	5595-090.AT2	7.9	1.000	0.10g
Tabas (Iran)	Tabas	TAB-TR.AT2	7.4	0.470	0.43g
Loma Prieta (USA)	Gilroy Array #1	G01320.AT2	6.9	0.840	0.40g
Northridge (USA)	LA00	LA0090.AT2	6.7	0.780	0.30g

3.2. Comparison of results

Accuracy and precision of the proposed simplified scheme is verified on the basis of comparisons with the results of a total of 2880 equivalent linear site response analyses. Natural logarithm of the residuals of the simplified method and equivalent linear analyses are presented in Figures 5-7, in terms of (i) maximum shear stress, (ii) maximum shear strain, (iii) maximum quasi-static structure racking drift, respectively. It should be noted that the buried structure assessed in the verification analysis is a linear-elastic, single barrel and single bay structure with geometrical properties of W=L=5.5 m and S_l= 7040 m/kN buried at 5.5m. Almost zero mean residuals and their relatively smaller standard deviations confirmed that the proposed simplified framework can be an unbiased and relatively precise alternative to currently used SFA which requires the input of equivalent linear site response analyses.

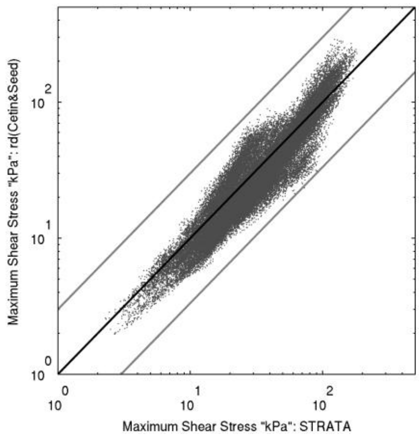

Figure 5. Comparison shear stresses (black line shows 1:1 line, grey lines show 1:3 and 3:1 lines)

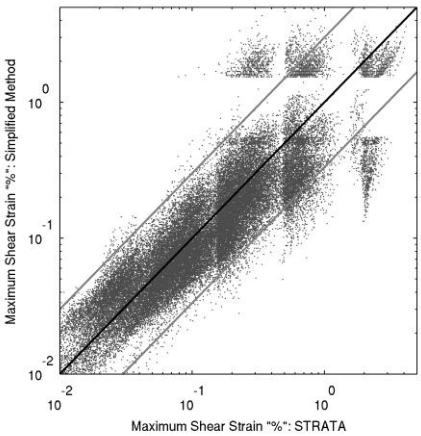

Figure 6. Comparison shear strains (black line shows 1:1 line, grey lines show 1:3 and 3:1 lines)

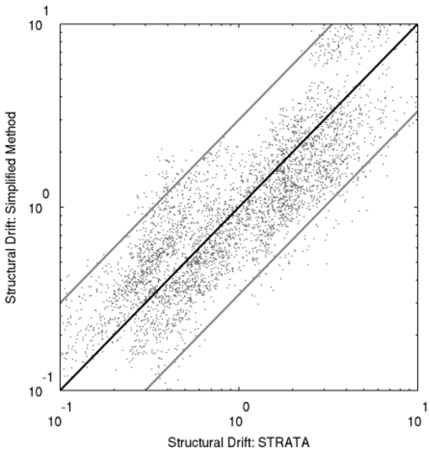

Figure 7. Comparison of structural drifts (%) (black line shows 1:1 line, grey lines show 1:3 and 3:1 lines)

4. CLOSURE

4.1. Summary and conclusions

Within the confines of this manuscript, after a brief review of currently available simplified methodologies to assess seismic displacement demands in simple rectangular buried structures, an alternative simplified framework was introduced valid for shallow depth (*depth*≤30m). Currently available SFA frameworks require a site-specific seismic response analysis to estimate free field shear stresses and induced shear strains (or deformation). However such requirements may become an obstacle for rapid assessment of displacement demand on buried structures, especially during preliminary design stages of the systems. The proposed procedure can produce unbiased and relatively precise estimates of racking coefficients with the input of relatively easy to estimate parameters of PGA_{rock}, modulus degradation and damping curves, etc. Accuracy and precision of the proposed simplified scheme is verified on the basis of comparisons with the results of a total of 2x9x20x8= 2880 equivalent linear site response analyses. Almost zero mean residuals and their relatively smaller standard deviations confirmed that the proposed simplified framework can be an unbiased and relatively precise alternative to currently used SFA which requires the input of equivalent linear dynamic site response analyses, during the preliminary design stage of conventional projects. However, in the detailed design stage, fully nonlinear dynamic time history analysis, incorporating both the kinematic and inertial components of soil-structure-earthquake interaction is strongly recommended.

4.2. A suggestion to consider structural nonlinearity in simplified frame analyses

Valid for particular cases where yielding displacement levels of buried structures are exceeded, strain-dependent secant stiffness can be used in the proposed scheme as shown in Figure 8. A detailed discussion on nonlinear response of structural systems is available in Priestley et al. (2007) and will not be repeated herein.

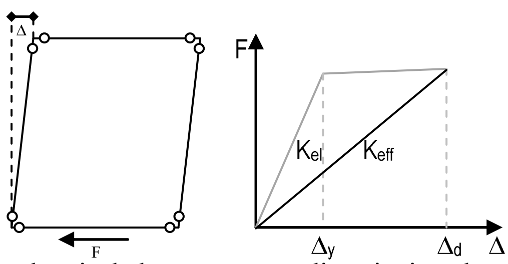

Figure 8. Qualitative sketch to include structure non-linearity into the present SFA methods

AKCNOWLEDGEMENT
First author is supported by a Ph.D. scholarship provided by IUSS/Rose School; this financial support is gratefully acknowledged. First author also wants to express his thankfulness to Dr. Albert Kottke, who had been the TA in Geotechnical Earthquake Engineering course in Rose School thought in 2009-2010 semester, for his advice on making a parametric analysis by making use of randomization process modelled by STRATA.

REFERENCES

Owen, G.N., Scholl, R.E. (1981). Earthquake engineering of large underground structures. Report no. FHWA RD-80 195, Federal Highway Administration and National Science Foundation.
Wang, J. (1993). Seismic design of tunnels. Monograph 7, Parsonns Brinckerhoff Quande & Douglas Inc., New York, USA.
Penzien, J., Wu, C.L. (1998). Stress in linings of bored tunnels. *Earthquake Engineering and Structure Dynamics* **27**, 283-300.
Penzien, J. (2000). Seismically induced racking of tunnel linings. *Earthquake Engineering and Structure Dynamics* **29**, 683-691.
Huo, H., Bobet, A., Fernández, G., Ramirez, J. (2005). Load transfer mechanisms between underground structure and surrounding ground: Evaluation of failure of Daikai Station. *Journal of Geotechnical and Geoenvironmental Engineering* **131:12**, 1522-1533.
Bobet A., Fernández, G., Huo, H., Ramirez, J. (2008). A practical iterative procedure to estimate seismic-induced deformations of shallow rectangular structures. *Canadian Geotechnical Journal* **45:7**, 923-938.
Özcebe, A.G. (2009). A comparative assessment of available methods for seismic performance evaluation of buried structures. Dissertation submitted in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering, Middle East Technical University, Ankara, Turkey.
Newmark, N.M. (1967). Problems in wave propagation in soil and rock. *International Symposium on Wave Propagation and Dynamic Properties of Earth Materials, Albuquerque, New Mexico.* 7-26.
Kuesel, T.R. (1969). Earthquake design criteria for subways. *Journal of Structural Divisions, ASCE* **95:ST6**, 1213-1231.
St. John, C.M., Zahrah, T.F. (1987). Aseismic design of underground structures. *Tunneling and Underground Space Technol.* **2**, 165-197.
Hendron A.J., Fernandez, G. (1983). Dynamic and static design considerations for underground chambers. *Seismic Design of Embankments and Caverns, ASCE, New York*, 157-197.
Merritt, J.L., Monsees, J.E., Hendron, A.J. (1985). Seismic design of underground structures. *Rapid Excavation Tunneling Conference* **Vol I:** 104-131.
Bilotta, E., Lanzano, G., Russo, G., Santucci de Magistris, F., Aiello, V., Conte, E., Silvestre, F., Valentino, M. (2007). Pseudostatic and dynamic analyses of tunnels in transversal and longitudinal directions. *4th International Earthquake Geotechnical Engineering, Thessaloniki, Greece.*
Gingery, J.R. (2007). A simplified method for estimating shear strains for ovaling and racking analysis of tunnels. *4th International Earthquake Geotechnical Engineering, Thessaloniki, Greece.*
Seed, R.B., Çetin, K.Ö., Moss, R.E.S., Kammemer, A.M., Wu, J., Pestana, J.M., Riemer, M.F. (2001). Recent advancements in soil liquefaction engineering and seismic site response evaluation. *4th International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics and Symposium in Honor of Professor W.D. Liam Finn, San Diego, California.* 1-45.
Çetin, K.Ö., Seed, R.B. (2004). Nonlinear shear mass participation factor (rd) for cyclic shear stress ratio evaluation. *Soil Dynamics and Earthquake Engineering* **24:2**, 103-113.
Toro, G.R. (1995). Probabilistic models of site velocity profiles for generic and site-specific ground-motion amplification studies, Brookhaven National Library, Upton, New York.
Sun, J.I., Goleshkorkhi, R., Seed, H.B. (1988). Dynamic moduli and damping ratios for cohesive soils. Report No. UCB/EERC-88/15. University of California, Berkeley, California.
Seed, H.B., Idriss, I.M. (1970). Soil moduli and damping factors for dynamic response analyses. Earthquake Engineering Research Center, Report No. EERC 70-10, University of California, Berkeley, California.
Vucetic, M., Dobry, R. (1991). Effects of soil plasticity on cyclic response. *Journal of Geotechnical Engineering, ASCE* **117:1**, 89-107.
Kottke, A., Rathje, E.M. (2009). Technical Manual for STRATA, PEER Report 2008/10, Pacific Earthquake Engineering Research Center, College of Engineering, University of California, Berkeley.
Priestley, M.J.N., Calvi, G.M., Kowalsky, M.J. (2007). Direct Displacement Based Design of Structures. IUSS Press, Pavia, Italy.