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# A SIMPLIFIED METHOD FOR ESTIMATING SHEAR STRAINS FOR OVALING AND RACKING ANALYSIS OF TUNNELS

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## ABSTRACT

Seismic analysis has become an important aspect of a prudent tunnel design, particularly when the tunnel is considered a lifeline whose post-earthquake functionality is important. A deformation-based approach to seismic design of underground structures, which differs from the inertia-based approach used predominately for above ground structures, has been developed over the past several decades. One mode of deformation evaluated is the tunnel's response to vertically propagating shear waves (VPSW), which results in an "ovaling" distortion of circular tunnels or a "racking" distortion of rectangular tunnels. An important parameter for ovaling and racking analyses is the free field shear strain within the tunnel horizon that results from the VPSW. While simple closed form solutions are available that permit rapid evaluation of ovaling/racking strains, axial thrust, shear and moment in tunnel linings, developing the free field shear strain used in the closed form solutions by performing site response analyses is disproportionately more time consuming and involved. An existing simple method based on wave propagation theory is deficient in that it does not account for soil softening and layered soil deposits. This paper presents a new simple method for estimating free field shear strains that is quick and convenient, can be used for layered soil deposits, and accounts for soil softening.

Keywords: tunnels, shear strain, site response, ovaling, racking

## BACKGROUND

Tunnels have historically performed better under seismic loading compared to above ground structures. However, damage to tunnels from earthquakes has been reported that ranges from minor spalling of tunnel linings to the Daikai subway station collapse in the 1995 Kobe earthquake. Performance of tunnels in major metropolitan areas of the U.S. during recent larger events such as the 1989 Loma Prieta and 1994 Northridge earthquakes has generally been good. However, the ground motion levels experienced by tunnels during these events were significantly lower than typical design levels. For these reasons it is good practice to incorporate seismic evaluations into the design of tunnels in seismically active regions.

The mass of a tunnel relative to its volume and to the mass of the soil surrounding it is small. Moreover, tunnels are embedded below the ground and are not prone to larger amplitude vibration as are above ground structures. Therefore, the inertial response of tunnels is typically not significant. Instead, tunnel response to ground shaking is governed by deformations.

The three primary modes of deformation that are typically evaluated in the seismic analysis of tunnels are: (1) axial compression and extension; (2) longitudinal bending; and (3) ovaling/racking. Wang (1993), Power et al. (1996) and Hashash et al. (2001, 2005) provide comprehensive treatment of the state of knowledge and typical practices for evaluating these three modes of deformation. This paper presents a simplified method for estimating free field shear strain for use in ovaling and racking

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analyses. Figure 1 provides a schematic representation of the racking and ovaling deformations modes.

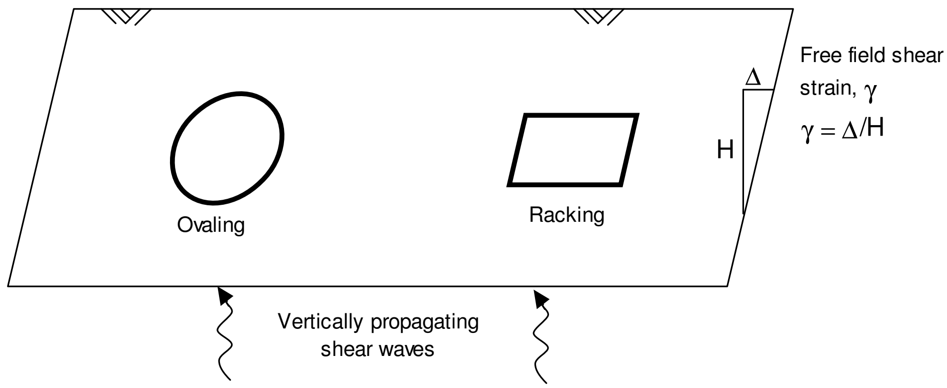


**Figure 1. Diagram showing ovaling and racking modes of displacement**

Preliminary and/or sensitivity analyses typically involve use of simple closed form solutions that require an estimation of the free field shear strain from VPSW. In these simple equations liner thrust, shear and moment in circular tunnels are proportional to free field shear strain from VPSW. Power et al. (1996) and Hashash et al. (2001) presented a simple equation to estimate free field shear strain that is based on wave theory:

$$\gamma = \frac{V_s}{C_s} \qquad (1)$$

Where $V_s$ is the peak particle velocity $C_s$ is the shear wave velocity of the. The Vs used in this equation should take into account degradation of shear wave velocity due to softening of soil during shaking (i.e., modulus degradation). However, no guidance is provided on how to reduce the small strain $V_s$ to account for softening. Also, equation (1) does not account for the soil layering that is typical of most profiles.

A good estimate of the free field shear strain from VPSW can be obtained by performing equivalent linear or nonlinear site response analyses. Compared to the simple closed form solutions for thrust, shear and moment in the tunnel lining, site response analyses are disproportionately more labor intensive and complicated. Performing site response analyses in preliminary design or for smaller projects is often not practical, or possible within budget constraints. A need exists for a simple method to estimate free field shear strain that is applicable for layered soil profiles and accounts for modulus degradation.

## PROPOSED METHOD

There are several analogous cases in geotechnical earthquake engineering where simplified methods for incorporating site effects have been developed so that full site response analyses can be avoided. In their methods for estimating seismic slope displacements, Makdisi & Seed (1978) and Bray & Rathje (1998) each provided simplified methods to incorporate site response without having to perform full site response analyses. The "stress reduction factor" or "shear mass participation factor", $r_d$, has been used in analyses of liquefaction triggering (Seed & Idriss, 1971) and seismic settlement of unsaturated sands (Tokimatsu & Seed, 1987). The $r_d$ factor was developed based on averaging of numerous site response analyses and permits an estimate of the average induced shear stress with depth.

In the following section equations are developed for estimating shear strain versus depth for seismic racking or ovaling analysis of tunnels. We begin with equation (2) for the average earthquake induced shear stress, $\tau_{ave}$, from Seed & Idriss (1971):

$$\tau_{ave} = 0.65 \cdot \frac{a_{\max}}{g} \cdot \sigma_0 \cdot r_d \qquad (2)$$

Where $a_{max}$ is the peak horizontal ground acceleration at the ground surface, g is the acceleration of gravity and $\sigma_0$ is the total vertical stress. Note that the factor 0.65 was incorporated into equation (2) by Seed & Idriss (1971) to account for observations that $a_{max}$ typically occurs as a single peak impulse during the acceleration time history, whereas repeating peak accelerations during significant shaking occur as about 65 percent of $a_{max}$. Equation (3) relates $\tau_{ave}$ to the effective shear strain, $\gamma_{eff}$.

$$\gamma_{eff} = \frac{\tau_{ave}}{G} = \frac{\tau_{ave}}{G_{\max} \cdot \left( \dfrac{G}{G_{\max}} \right)} \qquad (3)$$

Where G is the effective secant shear modulus at the induced shear strain level. The quantity $G/G_{max}$ varies with shear strain level.

Many researches have proposed $G/G_{max}$ versus shear strain relationships, with some of the more widely used in practice (Kramer & Paulsen, 2004) being Seed & Idriss (1970), and Vucetic & Dobry (1991). Darendeli (2001) presented a new family of modulus reduction (and damping) curves that were developed using statistical methods and vary with plasticity index (PI), mean initial effective stress ($\sigma'_m$) and overconsolidation ratio (OCR). The Darendeli (2001) $G/G_{max}$ curves are described using a variation of the hyperbolic equation of Hardin & Drnevich (1972a, 1972b) that is presented in equation (4).

$$\frac{G}{G_{\max}} = \frac{1}{1 + \left( \dfrac{\gamma}{\gamma_r} \right)^{0.919}} \qquad (4)$$

Where $\gamma_r$ is the reference strain which is computed using equation (5):

$$\gamma_r = \left(0.0352 + 0.0001 \cdot PI \cdot OCR^{0.3246}\right) \cdot \left(\sigma'_m\right)^{0.3483} \qquad (5)$$

*PI* is plasticity index in percent, *OCR* is the overconsolidation ration, and $\sigma'_m$ is the mean initial effective stress in atmospheres. The Darendeli $G/G_{max}$ curves have several advantages that make them attractive for this application. First, $G/G_{max}$ is presented in the form of an equation that permits direct calculation rather than reading values from charts. Second, $G/G_{max}$ curves can be calculated for a relatively wide range of soil types, stress conditions, and stress histories, i.e., $0 \le PI \le 100$, $0.25 \le \sigma'_m \le 16$ atm and $1 \le OCR \le 16$. A sampling of modulus reduction curves computed using the Darendeli equations are presented in Figure 2.

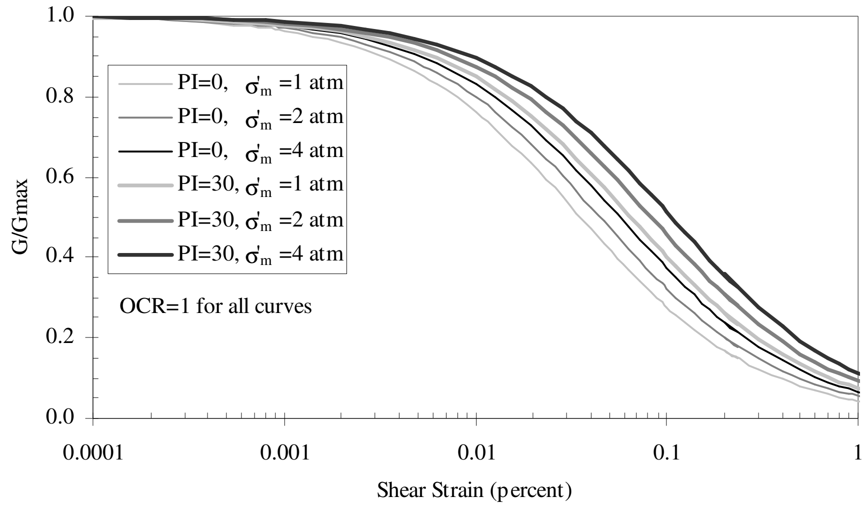


**Figure 2. Sample modulus reduction curves from Darendeli (2001)**

Combining equations (2), (3), and (4) yields the following equation 6a.

$$\gamma_{eff} = \frac{0.65 \cdot \frac{a_{\max}}{g} \cdot \sigma_0 \cdot r_d}{G_{\max} \cdot \frac{1}{1+\left(\frac{\gamma_{eff}}{\gamma_r}\right)^{0.919}}} \qquad (6a)$$

Rearranging terms yields:

$$\gamma_{eff} = \frac{0.65 \cdot \left(\frac{a_{\max}}{g}\right) \cdot \sigma_0 \cdot r_d \left[1+\left(\frac{\gamma_{eff}}{\gamma_r}\right)^{0.919}\right]}{G_{\max}} \qquad (6b)$$

The stress reduction factor, $r_d$, is an important quantity that partially accounts for site response effects. Cetin et al. (2004) and Idriss & Boulanger (2004) have proposed updates to the original depth dependent $r_d$ of Idriss & Seed (1971). Recognizing that site response is dependent upon several factors other than depth, Cetin et al. developed equations for $r_d$ that vary with depth, earthquake magnitude, peak ground acceleration, and average shear wave velocity. Idriss & Boulanger (2004) have proposed an $r_d$ relationship that is depth and magnitude dependent. The Idriss & Boulanger $r_d$ equations have been used in this paper and are presented in equations (7a,b,c) below. A plot of the Idriss & Boulanger $r_d$ versus depth for various magnitudes is shown in Figure 3. The Cetin et al. (2001) $r_d$ relationships could also be used in lieu of equations (7a,b,c).

$$r_d = \exp\left[\alpha(z) + \beta(z) \cdot M_w\right] \qquad (7a)$$

*where:*

$$\alpha(z) = -1.012 - 1.126 \cdot \sin\left(\frac{z}{11.73} + 5.133\right) \qquad (7b)$$

$$\beta(z) = 0.106 + 0.118 \cdot \sin\left(\frac{z}{11.28} + 5.142\right) \qquad (7b)$$

$M_w$ is the earthquake moment magnitude and $z$ is the depth in meters.

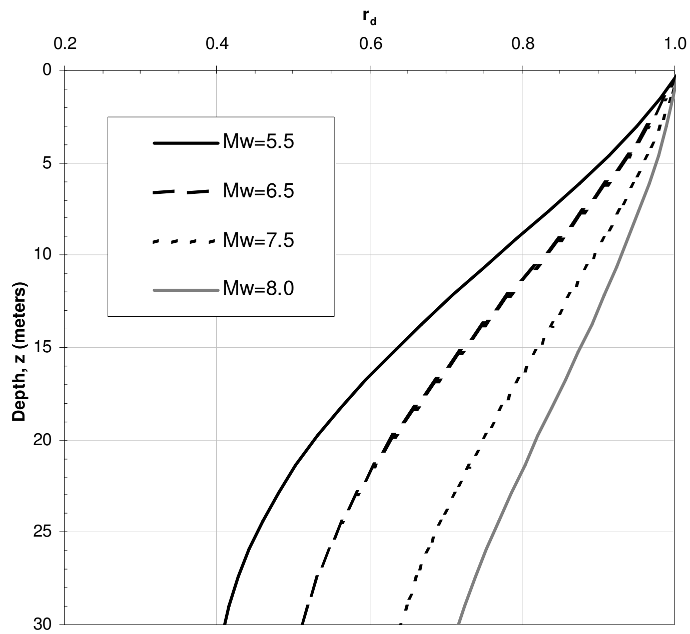


**Figure 3 – Stress reduction curves based on Idriss and Boulanger (2004)**

Where shear wave velocity data is available for a site, $G_{max}$ can be calculated using $G_{max} = (\gamma_t/g)V_s^2$; where $\gamma_t$ is the total unit weight of the soil. Where shear wave velocity is not available, $G_{max}$ can be estimated using correlations proposed by Ohta & Goto (1976) for cohessionless soil and Mayne & Rix (1993) for cohesive soil. These equations are presented as (8a) and (8b) below. Other relationships are relationships are available as summarized in Kramer (1996).

$$G_{\max} = 20{,}000 \cdot (N_1)_{60}^{0.333} \cdot (\sigma'_m)^{0.5} \quad \text{(cohesionless soil)} \qquad (8a)$$

Where $G_{max}$ and $\sigma'_m$ (mean confining stress) are in pounds per square foot (20.88 psf = 1 kPa).

$$G_{\max} = 406 \cdot (q_c)^{0.695} \cdot e^{-1.130} \qquad \text{(cohesive soil)} \qquad (8b)$$

Where $G_{max}$ and $q_c$ (CPT tip resistance) are in kPa and e is the void ratio.

Since equation (6b) calculates $\gamma_{eff}$ in terms of $\gamma_{eff}$, iteration is required to converge upon a solution. This can be performed automatically with commonly used spreadsheets. For example, in Excel a circular reference is established and the iterations toggle is set on under Tools – Options – Calculations tab. The author has found that satisfactory convergence is achieved in equation (6b) with the maximum iterations set to 10,000 and the maximum change set to 0.0001.

Peak shear strain ($\gamma_{peak}$), which is typically more important than $\gamma_{eff}$ in the seismic design of tunnels, can be obtained by dividing $\gamma_{eff}$ by 0.65 as shown in equation (9). While $\gamma_{peak}$ could have been computed directly by eliminating the 0.65 term from equation (2), the 0.65 term was retained due to its familiar form in equation (2) and because $\gamma_{eff}$ may also be of interest to structural engineers evaluating tunnel liner response.

$$\gamma_{peak} = \frac{\gamma_{eff}}{0.65} \qquad (9)$$

Note that the peak shear strains should occur simultaneously over a depth range less than or equal to about ¼ of a wavelength. When estimating average shear strain applicable over the depth range of a tunnel, it is conservative to assume that the peak shear strain occurs simultaneously over a depth range greater than the quarter wavelength depth. The quarter wavelength depth can be estimated using the following equation which is based on wave propagation theory:

$$\frac{\lambda}{4} = \frac{\overline{V}_{s,\deg} \cdot T}{4} \qquad (9)$$

Where $\lambda$ is the mean wavelength, T is the mean or smoothed spectral predominant period (Rathje et al., 2004) of the ground motion and $\overline{V}_{s,\deg}$ is the average shear wave velocity of the softened profile with n sublayers:

$$\overline{V}_{s,\deg} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{V_{s,\deg,i}}} \qquad (10)$$

Where the degraded shear wave velocity of the $i^{th}$ layer is calculated based on the secant shear modulus ($G_i$) and soil density ($\rho_i$) for the $i^{th}$ layer:

$$\overline{V}_{s,\deg,i} = \sqrt{\frac{G_i}{\rho_i}} \qquad (11)$$

With the peak shear strains occurring simultaneously, a peak displacement profile can be constructed by multiplying peak shear strains by the layer thicknesses and cumulatively adding the displacements per layer over the soil column depth. A single design peak shear strain must be used in the simple closed form solutions for evaluating shear, thrust and moments in tunnel liners. It is recommended that the design peak shear strain be taken as the average peak shear strain within the tunnel horizon, which can be graphically evaluated using the peak displacement profile.

## EXAMPLE CALCULATION

An example calculation using the proposed simplified method is described next. Preliminary seismic evaluations for a 7-meter diameter bored tunnel with a precast, prestressed segmental concrete liner required estimation of peak shear strain for evaluation of earthquake induced ovaling. The site conditions consist of dense sand to silty sand and very stiff clays that extend to about 36 meters below the ground surface, underlain by fine sandstone. The PI of the sand to silty sand is 0 and the PI of the clays is 30. The unit weight for all layers was taken as 18 kN/$m^3$. Figure 4 presents the subsurface profile with CPT tip and sleeve resistance measurements, and down-hole shear wave velocity measurements. The groundwater table occurs at a depth of approximately 5 meters below ground surface. The tunnel is to be constructed with 14 meters of cover above its crown. The design earthquake has a magnitude of 7.0 and an $a_{max}$ of 0.30g.

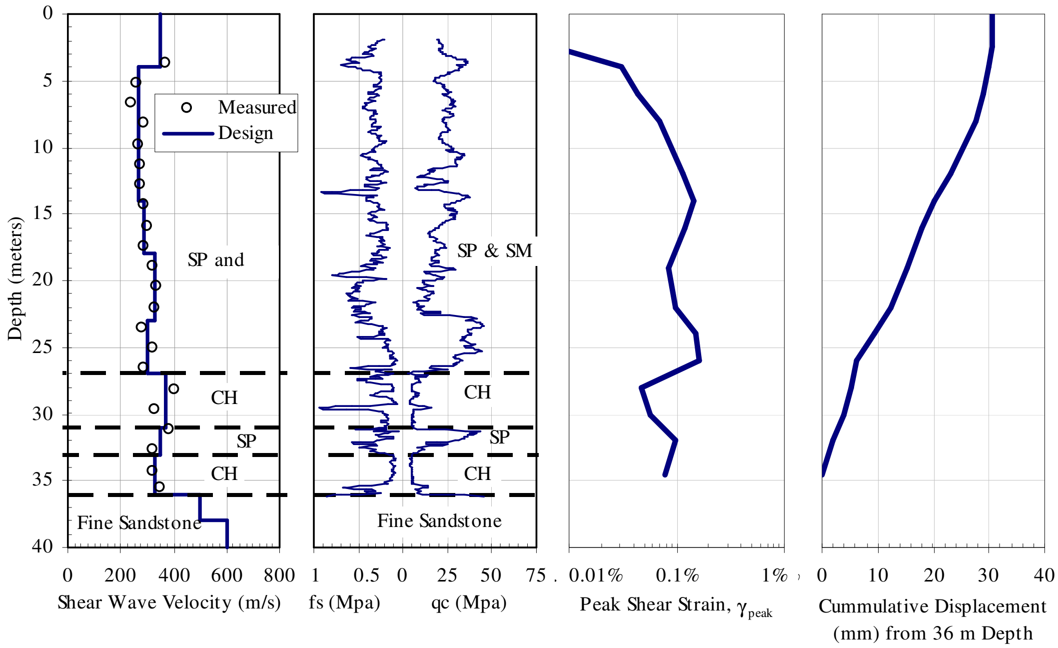


**Figure 4. Subsurface profile for example calculation**

Peak shear strain versus depth computed using the proposed method is presented in Figure 4. Figure 4 also presents a displacement profile relative to a depth of 36 m computed assuming the peak shear strains occur at the same time during in earthquake. For the sample calculation, a quarter wavelength depth of 30.5 m was calculated based a mean period is 0.6 seconds and average degraded shear wave velocity of 203 m/s. Thus, the assumption in this case that peak shear strains occur simultaneously over the soil profile depth of 36 m is slightly conservative. For a tunnel horizon depth range of 14 to 21 m, the average shear strain is about 0.11 percent.

## COMPARISON OF SIMPLIFIED METHOD TO SITE RESPONSE ANALYSIS METHOD

An equivalent linear site response analysis was performed using the program SHAKE2000 (Ordonez, 2000) with the same site conditions as the example calculation above. Stratigraphy, unit weight, and modulus reduction curves used in the site response analysis were the same as those used in the simplified analysis. A suite of 3 spectrally matched earthquake time histories were used in the analysis. The time histories were also slightly scaled such that the peak ground acceleration at the ground surface was 0.30 g, the same as that used in the simplified method calculation. The response spectra computed at the ground surface for the three time histories is shown for reference in Figure 5.

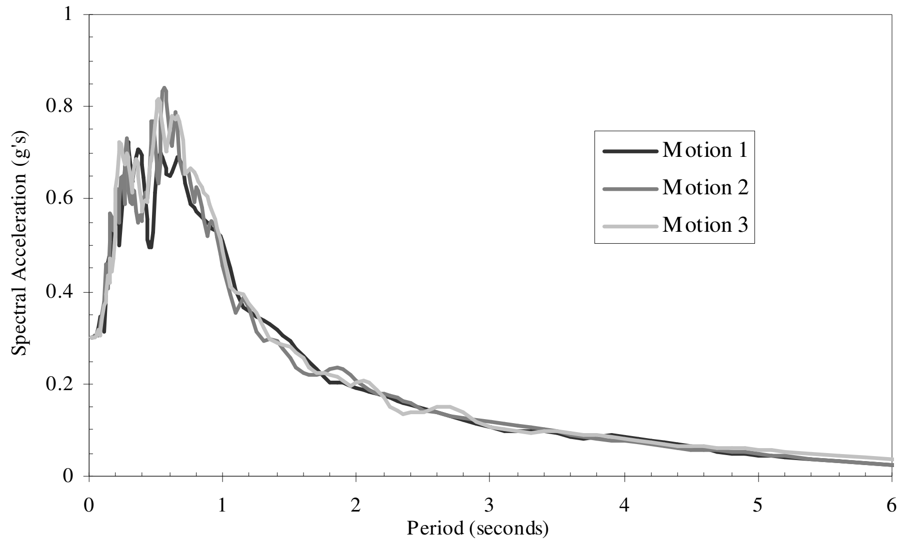


**Figure 5. Acceleration response spectra at the ground surface from the site response analyses**

Peak shear strains computed in the site response analyses are presented in Figure 6a along with the peak shear strains computed using the simplified method. The shear strains computed using the simplified method provided a good match with the shear strains computed in the site response analyses in terms of both magnitude and trend. The quality of the matching is similar for both the sand and clay layers in the profile.

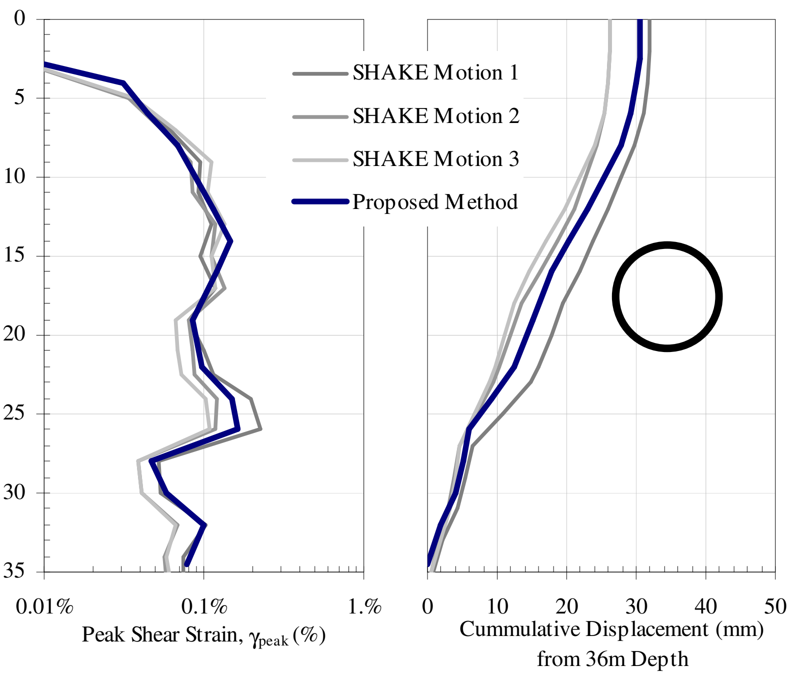


**Figure 6a. Peak shear strains** **Figure 6b. Displacement profiles**

Provided the height or diameter of the tunnel is less than the quarter wavelength depth described earlier, the average shear strain within the tunnel horizon can be computed from the slope of the cumulative displacement profile within the tunnel horizon. As shown in Figure 6b, the average shear strain for a tunnel horizon from a depth of 14 to 21 m is essentially the same for the simplified method and for the three profiles generated from the site response analyses. The SHAKE output confirmed that the peak shear strains throughout the 35 m profile occurred at essentially the same time during the earthquake ground motion.

## LIMITATIONS AND CAUTIONS

The simplified method for estimating shear strains from VPSW presented herein is subject to some of the same limitations that apply to equivalent linear site response analyses. The proposed method is not suitable for high levels of ground shaking (i.e., $a_{max}$ > ~0.4 or ~0.5g) or for soils such as soft to medium stiff clays where high levels of shear strain and yielding can occur. For these types of profiles it is recommended that other methods, such as fully nonlinear site response analyses be used. The proposed method is also not suitable for profiles that could involve generation of significant excess pore water pressure and liquefaction. Other methods, such as nonlinear effective-stress site response analysis that incorporate dynamic pore water pressure generation should be used for such cases.

While Idriss & Boulanger (2004) present equations for $r_d$ for depths in excess of 35 m, they indicate that the $r_d$ equations become less reliable below a depth of about 20 m. Therefore, caution should be exercised when estimating shear strain below a depth of about 20 m.

As with the seismic design of above ground structures, analytical techniques of varying levels and complexity are available, with the appropriate technique for a particular project depending upon code and regulatory requirements, and the size and importance of the project. For any given project, the designer should consider these factors and decide whether the proposed simplified method is suitable, or whether the more rigorous site response analysis is warranted. It is recommended that at a minimum, final design of important or critical tunnels should utilize shear strain data developed from site response analyses.

## CONCLUSIONS

A simplified method for estimating earthquake induced shear strains has been presented that is useful for evaluations of ovaling and racking in seismic tunnel design. The method permits relatively rapid estimation of shear strains from VPSW and is consistent with the level of effort exercised in simplified, closed form solutions that are commonly used to evaluate tunnel liner response to shear strains. Comparison of the proposed method with a more rigorous equivalent linear site response analysis shows good agreement between the shear strains predicted by the two methods.

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