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Simplified procedure to estimate ground settlement from seismic compression in compacted soils

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ABSTRACT: Seismic compression is defined as the accrual of contractive volumetric strains in unsaturated soil during strong shaking from earthquakes. We describe a simplified procedure for estimating ground displacements from seismic compression in compacted fill. The procedure has three steps: (1) estimation of shear strain amplitude within the fill soil mass from the peak acceleration at the ground surface and other seismological and site parameters; (2) estimation of volumetric strains within the fill mass based on compaction conditions in the fill, the shear strain amplitude, and the equivalent number of uniform strain cycles; and (3) integration of volumetric strains across the fill section to estimate settlement. The framework of the present procedure is similar to that of Tokimatsu and Seed (1987), which is strictly applicable only to clean sands. We update this widely used procedure to incorporate relatively recent material models for clean sands, and to extend the formulation to allow analysis of non-plastic silty sands and low-plasticity clays. The procedure is implemented for three field case history sites with measured settlements, and is found to generally provide reasonable, first-order estimates of ground settlements given the simplifying assumptions associated with this approximate method of analysis.

1 INTRODUCTION

Ground deformations in compacted fill slopes from seismic compression have been well documented in the literature (e.g. Pyke et al., 1975; Stewart et al., 2001), and are recognized as representing a significant hazard with respect to collateral loss during future earthquakes. Accordingly, the estimation of ground settlements from seismic compression is becoming a common component of geotechnical seismic design practice for hillside areas in the mountainous areas of southern California.

The current state of practice for estimating the seismic compression of unsaturated compacted fill soils consists of the methodology presented by Tokimatsu and Seed (1987), which is strictly applicable only to clean sands. Our objective in this paper is to update this widely used procedure to incorporate relatively recent test data for compacted sandy soils, and to extend the formulation to allow analysis of recently compacted fill soils containing significant fines. Partial motivation for this update comes from laboratory testing by Stewart et al. (2002), which has shown that clean sands can experience up to ten times more vertical strain than soils with fines compacted to a comparable density. Consequently, current methods for estimating seismic compression may be overly conservative, and may not be applicable to soils containing fines. In this paper, we present a brief summary of the Tokimatsu and Seed (1987) methodology; discuss advances since 1987 that provide an opportunity to improve the analysis procedure; outline a new analysis procedure similar in format to the 1987 procedure but incorporating recent advances; and apply the procedure to three field case history sites.

2 EXISTING STATE-OF-PRACTICE FOR SEISMIC COMPRESSION ANALYSIS

The original Tokimatsu and Seed (1987) analysis procedure is based on a simplified representation of the distribution of shear stress with depth in a one-dimensional soil column. If the soil column above a soil element at depth *h* behaves as a rigid body, and the surface peak horizontal acceleration is *PHA*, then the mass of soil above *h* would impose a maximum shear stress of:

$$t_{rigid,\max} = \frac{PHA}{g} \cdot s_0 \tag{1}$$

where g = the acceleration due to gravity and s_o = total overburden pressure at depth h.

Soil flexibility reduces the shear stress to values less than $t_{rigid,\ max}$, as a result of vertical incoherence of ground motion. Seed and Idriss (1971) developed a simplified technique to estimate earthquake induced cycle shear stresses at depth. They multiplied $t_{rigid,max}$ by a stress reduction factor, r_d (which is the ratio of the actual shear stress at depth vs. the theoretical "rigid body" shear stress). A factor of 0.65 is then applied to reduce the peak cyclic shear stress, t_{max}, to the effective cyclic stress, t_{eff}, as:

$$t_{eff} = 0.65 \cdot \frac{PHA}{g} \cdot s_0 \cdot r_d \tag{2}$$

Effective shear strain, g_{eff}, is estimated from t_{eff} using the effective shear modulus (G_{eff}), as follows:

$$g_{eff} = \frac{t_{eff}}{G_{eff}} = \frac{t_{eff}}{G_{\max}\left(\dfrac{G_{eff}}{G_{\max}}\right)} \tag{3}$$

where G_{max} = small strain shear modulus. Combining Eqs. 2 and 3 leads to:

$$g_{eff}\,\frac{G_{eff}}{G_{\max}} = \frac{0.65 \cdot PHA \cdot s_o \cdot r_d}{g \cdot G_{\max}} \tag{4}$$

The product g_{eff} (G_{eff}/G_{max}) in Eq. 4 can be readily translated to a shear strain amplitude g_{eff} using published models for soil modulus reduction with increasing shear strain (i.e. models relating g_{eff} to G_{eff}/G_{max}). Tokimatsu and Seed (1987) recommended using the modulus reduction curves of Iwasaki et al. (1978), which depend on effective stress.

Having estimated g_{eff} with the above procedure, volumetric strains at 15 cycles of shaking [$(e_v)_{N=15}$] are estimated using an appropriate volumetric strain material model (these models relate $(e_v)_{N=15}$ to g_{eff}, and depend on the compaction condition of the fill soil). Tokimatsu and Seed (1987) utilized the volumetric strain material model of Silver and Seed (1971), which are derived from laboratory simple shear testing of clean sands. In that work, compaction condition for sands is represented by relative density (D_r).

The values of $(e_v)_{N=15}$ are adjusted to the volumetric strain (e_v) for the actual number of strain cycles (N) using the factor $C_N = e_v/(e_v)_{N=15}$. Tokimatsu and Seed (1987) recommended using C_N relations for clean sand derived from testing by Silver and Seed (1971). Parameter N is a ground motion intensity measure (like *PHA*), and Tokimatsu and Seed (1987) recommended that it be estimated using an empirical relationship between magnitude (m) and N proposed by Seed et al. (1975).

The N-adjusted volumetric strain e_v is multiplied by two to account for multi-directional shaking effects per the recommendations of Pyke et al. (1975). Hence, the final estimate of volumetric strain at a point is represented by $2 \times C_N \times (e_v)_{N=15}$. These volumetric strains are then integrated over the depth of the soil column to calculate settlement.

3 ADVANCES SINCE 1987

Several features of the original Tokimatsu and Seed (1987) procedure for seismic compression analysis can be updated. These include: (1) new relations for r_d developed by Seed et al. (2001); (2) new relations for *N* by Liu et al. (2001); (3) new models for modulus reduction which incorporate the effects of effective stress, soil plasticity, and other factors, by Darendeli and Stokoe (2001); and (4) new material models relating shear strain to volumetric strain for sandy soils and soils with fines by Stewart et al. (2002). The following sections present the rationale for making these changes and synthesize the critical features of the new model components.

3.1 Stress Reduction Factors (r_d)

The original stress reduction factors recommended by Tokimatsu and Seed (1987) are the factors by Seed and Idriss (1971) that have been widely used for soil liquefaction applications. These factors, shown by the solid lines in Figure 1, are based on the results of a limited number of ground response analyses. Seed et al. (2001) found these factors to be biased (generally high) based on a relatively extensive parametric study involving 2153 combinations of site profiles and input motions. Profiles of r_d from that study are also shown in Figure 1. Using these profiles, Seed et al. (2001) regressed r_d against *PHA*, depth (*z*), magnitude (*m*), and average soil shear wave velocity in the upper 12 m (V_{s-12}), and recommended the following relationship for the median of r_d:

$$\textbf{z < 20 m}:\quad r_d = \frac{\left[1 + a_1 / a_2(z)\right]}{\left[1 + a_1 / a_3\right]} \tag{5a}$$

$$\textbf{z > 20 m}:\quad r_d = \frac{\left[1 + a_1 / a_2(z = 20)\right]}{\left[1 + a_1 / a_3\right]} - 0.0046(z - 20) \tag{5b}$$

where, $a_1 = -23.013 - 2.949 \cdot PHA / g + 0.999 \cdot m + 0.0053 \cdot V_{s-12}$

$$a_2(z) = 16.258 - 0.201 \cdot e^{0.341\left(z + 0.0785 \cdot V_{s-12} + 7.586\right)}$$

$a_2(z=20)$ is $a_2(z)$ with *z* set to 20 m

$$a_3(z) = 16.258 + 0.201 \cdot e^{0.341\left(0.0785 \cdot V_{s-12} + 7.586\right)}$$

In all of the above, depth (*z*) is in meters and velocity (V_{s-12}) is in m/s.

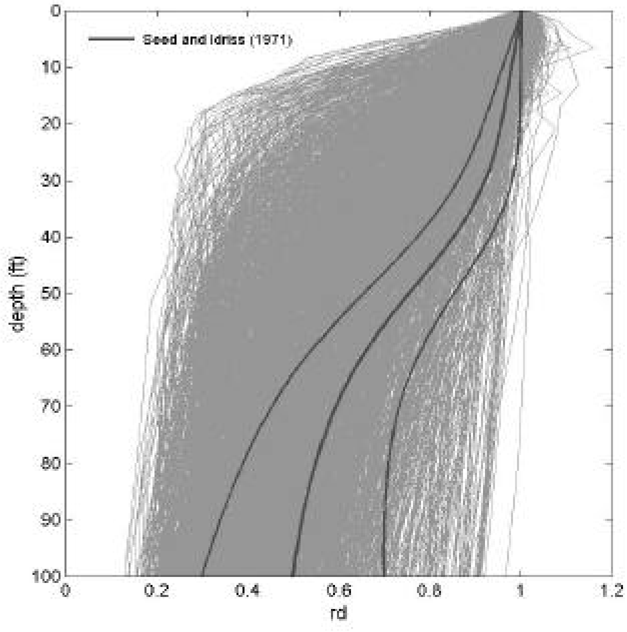

Figure 1. Stress reduction factors recommended by Seed and Idriss (1971) along with ground response analysis results of Seed et al. (2001)

3.2 Equivalent Number of Uniform Strain Cycles (*N*)

Using a strong motion data set for tectonically active regions, Liu et al. (2001) developed empirical regression equations to evaluate the equivalent number of uniform stress cycles of earthquake shaking as a function of magnitude (*m*), site-source distance (*r*), site condition (*S*=0 for rock, *S*=1 for soil), and near-fault rupture directivity effects. The *N* values were derived based on weighting factors specific to the problem of soil liquefaction triggering. However, one of the sets of weighting factors used by Liu et al. was found by Stewart et al. (2002) to be appropriate for evaluation of equivalent number of uniform *strain* cycles (*N*) for the seismic compression of sand. Predictions by that model for non near-fault conditions are shown in Figure 2, and the median model predictions are given by,

$$\ln(N) = \ln\left[\frac{\left(\dfrac{\exp\left(b_1 + b_2\left(m - m^*\right)\right)}{10^{1.5m+16.05}}\right)^{-\frac{1}{3}}}{4.9 \cdot 10^6 \, b} + Sc_1 + rc_2\right] \tag{6}$$

where *r* is in km, $b_1 = 1.53$, $b_2 = 1.51$, $c_1 = 0.75$, $c_2 = 0.095$, $b = 3.2$, and $m^* = 5.8$.

This model is more consistent with data than the model of Seed et al. (1975), which depends only on magnitude and is tailored to the problem of soil liquefaction. Figure 2 shows significant differences between these models, particularly at large distances.

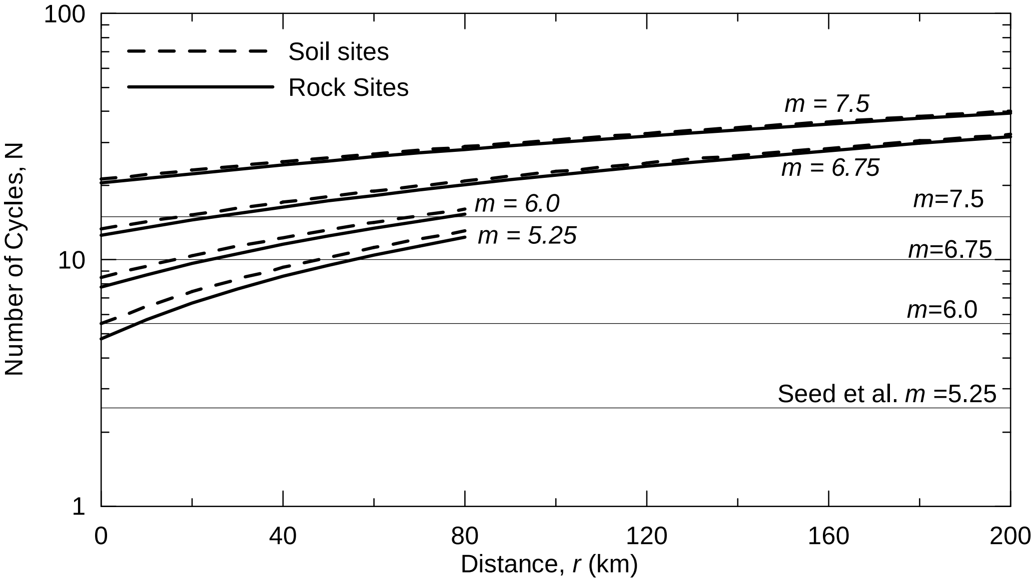

Figure 2. Variation of median values of *N* with distance and magnitude from Liu et al. (2001) along with recommendations of Seed et al. (1975).

3.3 Modulus Reduction Curves

Modulus reduction curves have a critical role in the analysis procedure, as they are used to estimate shear strains per Eq. 4. Tokimatsu and Seed (1987) recommended the use of modulus reduction curves for clean uniform sands by Iwasaki et al. (1978), which depend on effective stress. The model for modulus reduction by Darendeli and Stokoe (2001) is based on a much larger suite of test results, and incorporates effects of effective stress (*s¢*), soil plasticity (as represented by plasticity index, PI), and overconsolidation ratio (OCR). Figure 3 shows a family of modulus reduction curves (based on the D&S model) for varying PI and *s¢* (the effects of OCR are generally small, and the plots in Figure 3 apply for OCR = 1, which is generally appropriate for fills at *z* >3-6 m, Duncan et al., 1991). Note that the plots in Figure 3 are formatted to directly estimate shear strain, *g* from the product $g^{x}(G/G_{max})$.

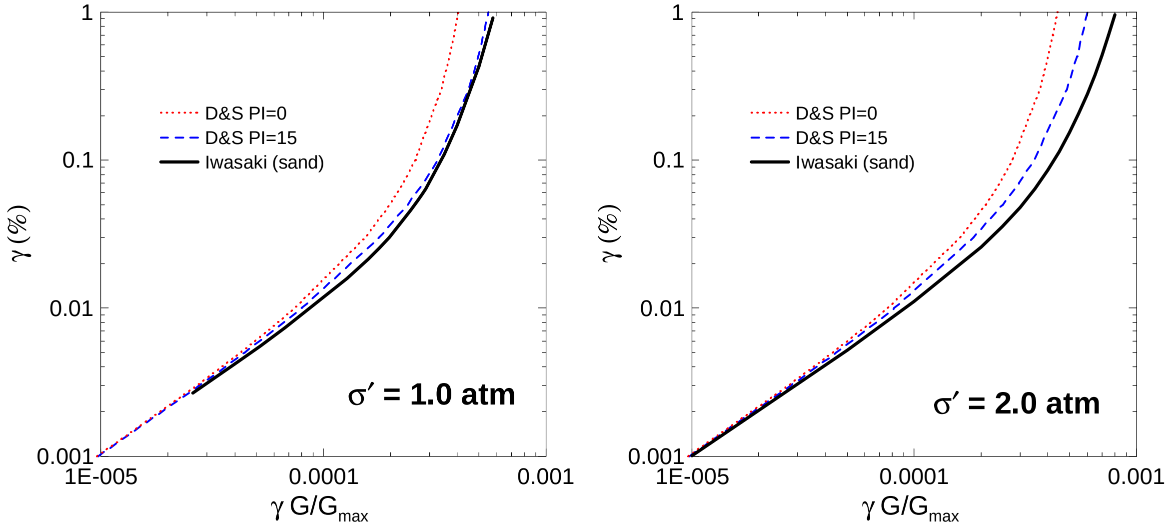

Figure 3. Modulus reduction curves from Iwasaki (1978) and Darendeli and Stokoe (2001) re-expressed in format for estimation of shear strain amplitude, showing effects of effective overburden stress and soil plasticity

The D&S results show that nonlinearity decreases with increasing soil plasticity (as shown by shifting of the curves in Figure 3 to the right as PI increases). At relatively large confining pressures (*s¢* > 0.5 atm), the D&S curves for both PI = 0 and 15 are more nonlinear than those by Iwasaki, which can affect significantly the computed shear strains as shown subsequently in Section 5.

Pradel (1998) developed a fit to the Iwasaki curves shown in Figure 3 using the following equation:

$$g = \frac{1+a\cdot e^{b\cdot R}}{1+a} R\cdot 100 \;\; \text{(in \%)} \tag{7}$$

where *R* is the product computed in Eq. 4. We use this same regression equation for the D&S curves, and find *a* and *b* to be soil-type dependant as follows:

PI ≈ 15: $a = 0.194\cdot\left(s\,'/p_a\right)^{0.265}$ $b = 7490\cdot\left(s\,'/p_a\right)^{-0.418}$

PI ≈ 0: $a = 0.199\cdot\left(s\,'/p_a\right)^{0.231}$ $b = 10850\cdot\left(s\,'/p_a\right)^{-0.410}$

where p_a = 101.3 kPa. Shear strains for PI between 0 and 15 can be interpolated using Eq. 7.

3.4 **Material Models for Volumetric Strain**

A volumetric strain material model is defined as a relationship between (1) cyclic shear strain amplitude, g_c, and $(e_v)_{N=15}$ and (2) C_N and *N*. Tokimatsu and Seed (1987) recommended the use of volumetric strain material models that were derived from cyclic simple shear testing of clean sands by Silver and Seed (1971). Recent simple shear testing at UCLA has re-examined these relationships for clean sand and has developed models for several fill soils containing fines.

The UCLA test results on clean sands are synthesized in the left frame of Figure 4 at D_r = 60%, and are also compared to the test results of Silver and Seed (1971). The recent testing was performed on 16 sands spanning a range of sand compositional factors (gradation, grain size, grain angularity). No trends in $(e_v)_{N=15}$ were found relative to these compositional factors, although the collective results provide insight into the variability associated with volumetric strain material models for sand. The right side of Figure 4 shows median ± two standard deviation results for C_N based on the testing of the 16 sands – once again, no trends with compositional factors was found. Also shown is the recommended curve by Silver and Seed (1971), which predicts less settlement for earthquakes with a small number of cycles (*N* < 15) and more settlement for *N* > 15.

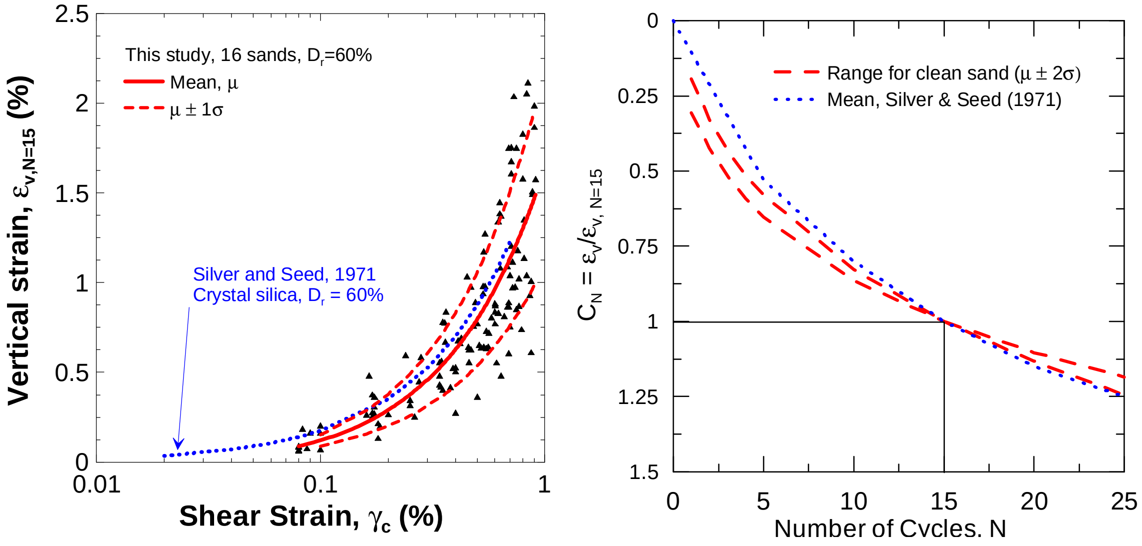

Figure 4. Volumetric strain material models based on simple shear testing of clean sands

Testing of fill soils with fines has been performed on four specimens from two field sites that were investigated in detail by Stewart et al. (2002). The results provide insight into the effects of essentially non-plastic fines (PI = 2) and low-plasticity fines (PI = 15) on the seismic compression of soil with large fines content (approximately 50%). Curve fits to these test results are compared to clean sand results in Figure 5, and show that

1. For the same Modified Proctor relative compaction (*RC*), soils with non-plastic fines experience less seismic compression than clean sands for a common set of baseline conditions, but these two materials behave similarly in the sense that *RC* is the principal construction-related factor affecting seismic compression (i.e. degree-of-saturation, *S*, is not important), and
2. Seismic compression in soils with low-plasticity fines decreases not only with increasing *RC*, but also for moderate *RC*s decreases with increasing as-compacted degree-of-saturation (*S*). At low *S*, volumetric strains from seismic compression are comparable to those for sand (at a common *RC*), whereas at high *S* the strains are approximately one-quarter of those for sand.

The C_N curves for these materials are generally similar to the lower-bound C_N curve shown in Figure 4 (i.e. the curve showing the most rapid degradation with *N*). The results shown in Figure 5 for these materials may not be applicable to other fill soils.

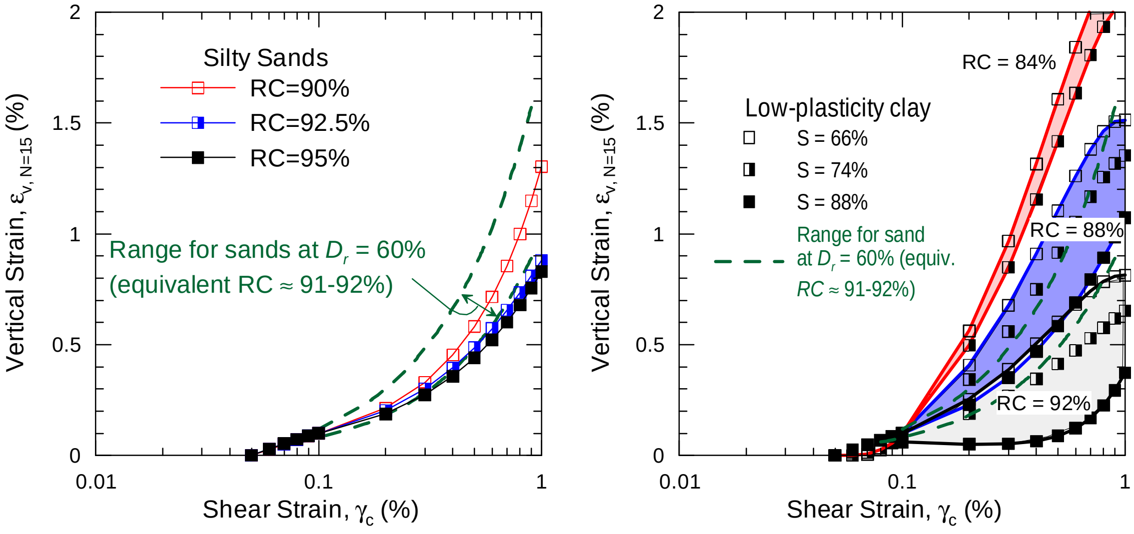

Figure 5. Volumetric strain material models based on simple shear testing of soils with fines

4 RECOMMENDED ANALYSIS PROCEDURE

The procedure has three general steps: (1) estimation of shear strain amplitude within the fill soil mass from the peak acceleration at the ground surface and other seismological and site parameters; (2) estimation of volumetric strains within the fill mass based on compaction conditions in the fill, the shear strain amplitude, and the equivalent number of uniform strain cycles; and (3) integration of volumetric strains across the fill section to estimate settlement. Details on the first two steps follow:

1(a). Estimate *PHA* and *N* using appropriate seismic hazard analyses. The *PHA* value should apply for the ground condition at the surface of the fill. This will generally require accounting for ground response effects either through site-specific analysis (preferred) or application of an amplification factor. Amplification factor models applicable to the shallow soil over rock configuration of hillside fills are unavailable, but investigations of several specific sites by Stewart et al. (2002) have generally found PHA amplifications of about 1.5 to 2.0 for typical design levels of shaking (i.e. *PHA* on rock ≈ 0.3-0.7 g).

1(b). Measure or estimate shear wave velocity in fill soils, and estimate the maximum shear modulus as $G_{max} = V_s^2\rho$, where ρ = mass density of soil.

1(c). Estimate stress reduction factors (r_d) as function of depth using the relation in Eqs. 5.

1(d). Estimate the variation of shear strain amplitude (γ_{eff}) with depth using Eq. 4 and Figure 3.

2(a). Estimate $(\varepsilon_v)_{N=15}$ based on γ_{eff} and soil compaction condition using appropriate material models for volumetric strain (Section 3.4).

2(b). Estimate C_N and calculate $\varepsilon_v = C_N \times (\varepsilon_v)_{N=15}$. Multiply by two to account for multi-directional shaking effects.

5 COMPARISON TO CASE HISTORIES

In this section, we compare predictions from the above analysis procedure to observed settlements at three sites where such settlements are reliably known from survey measurements. One of the case studies is a fill blanket at the Jensen Filtration Plant shaken by the 1971 San Fernando, California earthquake (Pyke et al., 1975), and the other two (denoted Sites A and B) are canyon fills in Santa Clarita shaken by the Northridge, California earthquake (Stewart et al., 2002). It should be emphasized that these case histories do not provide a sufficient data set against which to calibrate the analysis procedure. We present these comparisons merely to illustrate the general performance of the model for typical design-basis levels of shaking in seismically active regions.

At the Jensen site, the clayey sand fill is up to 17 m thick and overlies 1.5-6 m of alluvium. The water table is located in the alluvium, which liquefied during the earthquake causing lateral spreading. Estimated peak accelerations at the site are 0.5-0.6 g. Observed settlements along a survey baseline were 12.7 cm, although some of these settlements can be attributed to lateral spreading. Pyke et al. (1975) estimated the settlements from seismic compression to be approximately 9 to 10 cm.

At Santa Clarita Site A, the sandy clay fill is up to 24 m thick and overlies shallow alluvium and rock. Modified Proctor relative compactions of fill are ~88%, and the fill was generally compacted dry of optimum. Peak accelerations at the site have been estimated as 0.5-0.7 g. Settlements as large as 22 cm occurred in a building at the site. Site B has silty sand fill varying from 15 to 30 m thick overlying rock. Modified Proctor relative compactions of near-surface fill soils were approximately 92%, and approximately 95% at depth. Peak accelerations at the site have been estimated as approximately 0.8 to 1.2 g. Observed settlements of the fill ranged from about 2 to 6 cm.

For Santa Clarita Sites A-B, the variability of input parameters V_s and *PHA* was estimated and integrated into the analysis using a logic tree approach. Weighted means and standard deviations were calculated from the distributions of calculated settlements. Best estimate soil properties were used for the Jensen site. We account for matric suction effects on σ' in our analysis of γ. Matric suction does not affect total stress σ_o, but is estimated to add ~1 atm to σ' based on typical soil-water characteristic curves (Fredlund, 1993). As shown in Table 1, calculated settlements are reasonably consistent with observation for Jensen and Site A, but not for Site B. The problem at Site B appears to be the very strong levels of shaking (*PHA* > 1 g), for which Figure 3 provides unrealistically large estimates of *g*. Site-specific ground response analysis of Site B provides unbiased estimates (Stewart et al., 2002), and appear to be needed to reliably estimate *g* for very strong shaking.

Table 1. Summary of calculated (median ± one standard deviation) and observed settlements

Site	Recommended Procedure (cm)	Observation (cm)
Jensen	8.6	9-10
Site A	6.9 +/- 4.7	6
Site B	14.5 +/- 8.4	5.6

6 CONCLUSIONS

In this paper we present an analysis procedure to estimate settlements from seismic compression. The procedure de-couples the calculations of shear strain and volumetric strain, and utilizes recent research results on stress reduction factors (r_d), soil modulus reduction curves, and soil volumetric strain models. The results are generally found to compare favorably to observation, although problems in shear strain estimation are encountered for very strong levels of shaking (*PHA* values over ~1 g).

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