Cycle-to-Cycle Behavior in a Volumetric Strain Material Model

Comportement de cycle à cycle dans un modèle de matériau de contrainte volumétrique

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ABSTRACT: The methods for estimating ground settlements from seismic compression have been expanding from the simplified procedure to methods incorporating ground response analyses and unsaturated soil mechanics. One component of the simplified procedure is the volumetric strain material model. A particular volumetric strain material model that has recently gained traction is the so-called "clean sand model" which significantly expands on the original volumetric sand material model presented in the original simplified procedure. The clean sand model consists of multiple components, one of which models cycle-to-cycle behavior. This component basically demonstrates the cycle-to-cycle behavior of sands in seismic compression is independent of compositional and environmental factors. This investigation demonstrates, through the use of advanced laboratory testing, that cycle-to-cycle behavior can be dependent on the effective strain amplitude and overburden pressure. These results suggest a slight difference in settlements from seismic compression.

RÉSUMÉ : Les méthodes pour estimer les règlements au sol à partir de la compression sismique ont été étendues de la procédure simplifiée aux méthodes incorporant l'analyse de la réponse au sol et la mécanique des sols non saturés. Un composant de la procédure simplifiée est le modèle de matériau de déformation volumétrique. Un modèle particulier de matériau volumétrique de contrainte qui a récemment acquis une traction est le «modèle de sable propre» qui se dilate considérablement sur le modèle de matériau de sable volumétrique original présenté dans la procédure simplifiée originale. Le modèle de sable propre se compose de plusieurs composants, dont un modèle de cycle à cycle de comportement. Cette composante démontre fondamentalement le comportement cycle-cycle des sables dans la compression sismique est indépendante des facteurs compositionnels et environnementaux. Cette étude démontre, à l'aide de tests de laboratoire avancés, que le comportement cycle-cycle peut dépendre de l'amplitude de contrainte efficace et de la surcharge. Ces résultats suggèrent une légère différence dans les colonies par rapport à la compression sismique.

KEYWORDS: seismic compression, settlement, soil dynamics.

1 INTRODUCTION

During and after large earthquakes, ground failure can take on many forms. Ground failure phenomena generally range from liquefaction to cyclic softening to seismic compression, depending on a variety of conditions. Table 1 identifies some of the ground failure types that have been observed in past earthquakes. The table simplifies a complex phenomenon down to the dependence on saturation conditions and soil material type. In this table, Sandy soils are those soils that exhibit behavior similar to what would be observed in soils with no to very low plasticity while clayey soils would be those soils that exhibit behavior similar to what would be observed in soils with some plasticity. Of course the rows and columns in the table are not strict delineations as settlement from seismic compression has been observed in clayey soils and liquefaction can occur when sand is not fully saturated.

Table 1. General ground failure types due to strong earthquake shaking.

Type\Condition	Saturated	Unsaturated
Sandy soils	Liquefaction, Lateral spreading, Reconsolidation	Seismic compression
Clayey soils	Cyclic softening, Reconsolidation	Cyclic softening

Needless to say, ground failure can have devastating effects on critical infrastructure when not accounted for. Figure 1 shows reconnaissance photos of ground settlement as a result of a major earthquake near a nuclear power plant taken by Sakai et al. (2009). This nuclear power plant campus was shut down for over a year with numerous observations of structural and ground failures. Since the groundwater table is artificially lowered near nuclear power plants, this type of ground failure is most likely settlement from seismic compression.

Figure 1. Example evidence of settlement around nuclear power plant infrastructure (taken and modified from Sakai et al. 2009).

The state-of-practice for analyzing settlements from seismic compression is the simplified procedure proposed by Tokimatsu and Seed (1987). This procedure is an extension and compilation of work by Silver and Seed (1971). One major component in the simplified procedure is the volumetric strain material model, VSMM, derived from laboratory tests based on sand. The VSMM basically estimates strains that would result from an M 7.5 earthquake and then scales it accordingly to different magnitude earthquakes. The magnitude scaling portion of the VSMM is how the model handles cycle-to-cycle behavior. Advanced test data suggest the traditional approach to account for cycle-to-cycle behavior can be improved.

Following this introduction, a brief description of the simplified procedure, along with an updated VSMM, will be presented. Some observations will be shown on recent test data regarding the cycle-to-cycle behavior and how it affects the VSMM. Simple seismic compression calculations incorporating this interpretation of cycle-to-cycle behavior are then demonstrated to showcase the differences.

2 VOLUMETRIC STRAIN MATERIAL MODEL

The simplified procedure provides a simplified representation of the distribution of shear strains in a soil column. These shear strains are then used as input into a decoupled VSMM that estimates volumetric strains for an M 7.5 earthquake. Another component of the VSMM takes these volumetric strains and scales them to other earthquake magnitudes for a resultant volumetric strain. The basis for this scaling was that the effects of an M 7.5 earthquake could be represented by 15 constant amplitude shear strain cycles (Seed et al. 1975). This scaling is shown in Figure 2 for the simplified procedure.

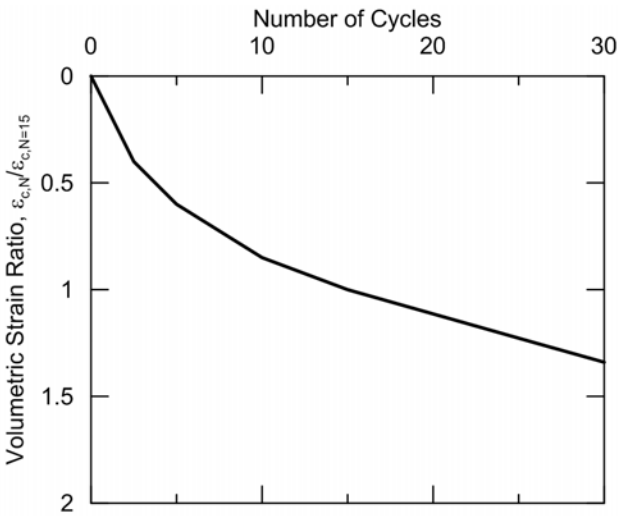

Figure 2. Relationship between volumetric strain ratio and number of cycles for the simplified procedure (reproduced from Tokimatsu and Seed 1987).

Work by Duku et al. (2008) and Yee et al. (2014) have separated the VSMM from the simplified procedure into two primary parts, as shown in Figure 3. Figure 3a shows the impact of varying shear strains on the volumetric strain for tests performed at 15 cycles. In Figure 3a, FC = fines content; S = saturation; σ_v' = effective vertical stress; D_R = relative density; and RC = relative compaction. Figure 3b is then used to scale the volumetric strains obtained from Figure 3a based on the different number of cycles.. These two parts can be summarized with the following equations, respectively,

$$(\varepsilon_V)_{N=15} = a^*(\gamma_c - \gamma_{tv})^b \tag{1}$$

$$C_N = (\varepsilon_V)_N / (\varepsilon_V)_{N=15} = R\ln(N) + c \tag{2}$$

Where $(\varepsilon_v)_{N=15}$ = the volumetric strain after 15 cycles of shear straining; γ_c = equivalent constant shear strain amplitude; γ_{tv} = threshold shear strain; N = number of cycles; $(\varepsilon_v)_N$ = the volumetric strain after N cycles of shear straining; and a^*, b, and R = regression model parameters. Combining the relationships in both parts allows the engineer to estimate settlements from seismic compression.

$$(\varepsilon_V)_N = (\varepsilon_V)_{N=15} \times C_N \tag{3}$$

One feature of the VSMM by Duku et al. (2008) is that the R parameter as shown in Eq. (2), is considered constant for clean sands, with parameter R = 0.29 (median value). If the VSMM portion of the simplified procedure (Tokimatsu and Seed 1987) that considers cycle-to-cycle behavior is also similarly parameterized, then parameter R = 0.35. Figure 3b shows how parameter R visually fits sample test data.

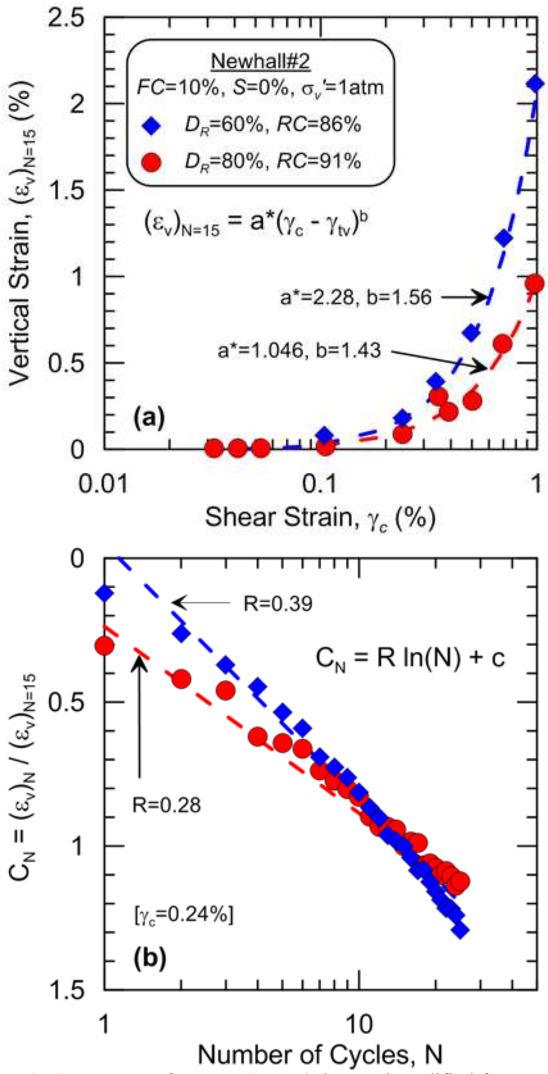

Figure 3. Components from a VSMM (taken and modified from Yee et al. 2014). (a) Vertical strain vs. shear strain and (b) normalized vertical strain vs. number of cycles.

3 LABORATORY TESTING

A site investigation was conducted near a nuclear power plant site that experienced widespread seismic compression (Yee et al. 2013). Bulk samples from the site indicated the soil to be sand with about 4% fines content and essentially two levels of saturation; approximately 6% and 18%. Reconstituted specimens were prepared from bulk samples for cyclic simple shear testing. The simple shear device used was the digitally controlled simple shear device specifically designed to measure volume change due to shear straining (Duku et al. 2007). Specimens were prepared to relative densities of approximately 40% and 65% after being consolidated under vertical stresses of 50, 100, 200, or 400 kPa. The effect of fines is surmised to be negligible given the low fines content of 4%, Each specimen was sheared at constant displacement amplitudes at a frequency $f = 1$ Hz for about 25 cycles. Figure 4 shows an example of the results from simple shear testing. Figure 4a shows the cyclic shear strain cycles applied to the sample sandy specimen and Figure 4b shows the resultant vertical strain from cyclic shear straining.

Vertical strains were calculated and then normalized to 15 cycles to estimate C_N. The parameter R was then tuned to fit the following regression model: $C_N = R \ln(N) + c$, where parameter $c = 1 - R \ln(15)$. Figure 5 shows a plot of parameter R against constant shear strain amplitude loading when relative density is varied while Figure 6 shows a plot of parameter R against constant shear strain amplitude loading when saturation is varied. These figures suggest that even for relatively clean sands, the cycle-to-cycle behavior is not constant in log space. It also shows a significant increase in parameter R as shear strains approach the threshold shear strain, which was estimated to be about 0.03-0.04%. However, interestingly enough, parameter R at or below the threshold shear strain appears to approach 0 as opposed to 1. Moreover, some data suggest negative parameter R values, which would suggest dilation, although compliance seems to be the more likely explanation.

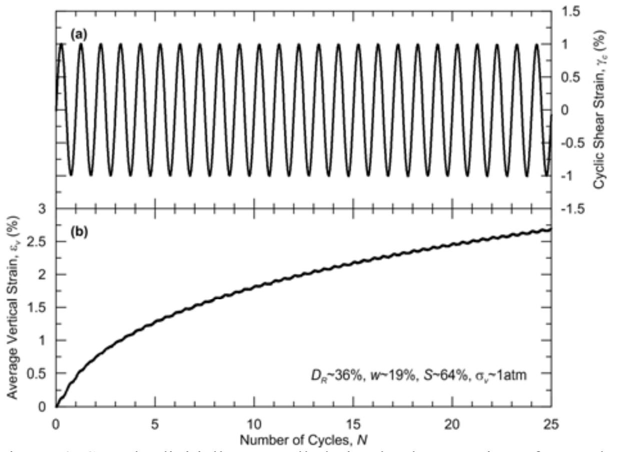

Figure 4. Sample digitially controlled simple shear testing of a sandy material. (a) Cyclic shear strain cycles and (b) average vertical strain vs. number of cycles.

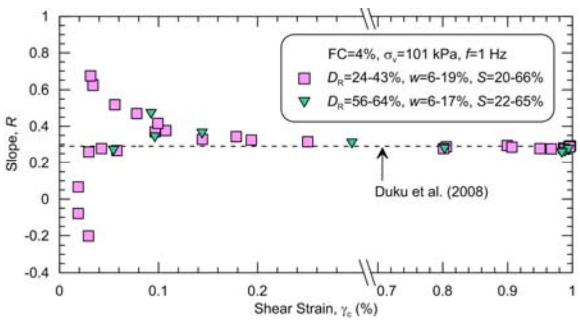

Figure 5. Variation of parameter R against shear strain in a VSMM considering changes in relative density.

4 EXAMPLE COMPUTATION

Tokimatsu and Seed (1987) provide an example taken from Seed and Silver (1972) to demonstrate the efficacy of the simplified procedure. That same example is used here to showcase the potential differences in settlement calculations when updated cycle-to-cycle behavior is considered.

The scenario is a 15 m sand deposit with a relative density of 45% subjected to a maximum surface acceleration of 0.45 g. The researchers divided the soil deposit into 6 layers with a relative density of 45% as shown in Table 2. The simplified procedure essentially correlated the M 6.6 San Fernando earthquake as being equivalent to 9 constant amplitude shear strain cycles. To account for potential material specific behavior of the crystal silica sand, the value taken from Figure 4 is increased by 0.06 (= 0.35-0.29).

Settlement calculations estimate settlement to be about 8.36 cm, which is close to the 8.56 cm of settlement computed using the VSMM in the simplified procedure. Interestingly, if the increase of 0.06 for parameter R is not taken into account, then the computed settlement from seismic compression would be about 8.68 cm, slightly more than the results from the simplified method. Seed and Silver (1972) reported settlements from the 1971 San Fernando earthquake to be in the range of 0.10 to 0.15 m.

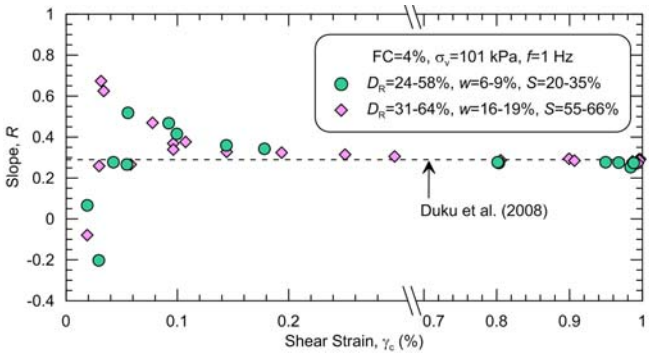

Figure 6. Variation of parameter R against shear strain in a VSMM considering changes in saturation.

Table 2. Sample computation of settlement for dry sand deposit.

Layer	Thickness (m)	γ_{eff} (%)	$2\times\varepsilon_{v,M=6.6}$ (%)	Settlement (cm)	$2\times\varepsilon_{v,M=6.6,\,update}$ (%)	Settlement (cm)
1	1.5	0.05	0.22	0.33	0.19	0.29
2	1.5	0.08	0.36	0.56	0.33	0.50
3	3	0.12	0.56	1.70	0.55	1.66
4	3	0.14	0.64	1.96	0.64	1.92
5	3	0.15	0.72	2.18	0.73	2.18
6	3	0.13	0.60	1.83	0.60	1.81
Total				8.56		8.36

5 DISCUSSION

Although the specimens tesed were not clean sands, the 4% fines content is low and should have very little influence on behavior. Additional and varying environmental factors were not considered at this point. The constant parameter R value of 0.29 is consistent with the behavior of the tested matieral at large shear strains. If this value was used, then settlements from seismic compression in the previously described scenario would be approximately 9 cm, which is not too different from the originally calculated value.

It is important to note that the behavior is different from the previous assumption and dependent on shear strain demand. The increase in parameter R as shear strains decrease does not appear to have significant impact on settlements in the previous scenario. Cases where this behavior is important would be earthquake scenarios that generate relatively large shear strain demands, this would cause parameter R to lower and thus increase parameter C_N to increase settlements. This indirectly suggests previous seismic compression calculations may have been overestimated, if all factors were constant.

6 CONCLUSION

A feature of modern volumetric strain material models used in estimating settlements from seismic compression was re-evaluated. These models state a critical parameter is constant for clean sands and slightly decreasing with shear strain amplitude for sands with low plasticity fines. Simple shear testing on a clean natural sand suggests the decrease in this parameter with shear strain amplitude is more pronounced than what was observed in sands with low plasticity fines. Additionally, behaviour near and below the threshold shear strain do not follow commonly accepted theories on soil behaviour, which might be attributed to compliance issues at small strains.

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8 REFERENCES

Duku, P.M., Stewart, J.P., Whang, D.H., and Venugopal, R. 2007. Digitally controlled simple shear apparatus for dynamic soil testing. *Geotech. Testing J.*, ASTM, 30(5), 368-377.

Duku, P.M., Stewart, J.P., Whang, D.H., and Yee, E. 2008. Volumetric strains of clean sands subject to cyclic loads. *J. Geotech. Geoenviron. Engrg.* ASCE, 134(8), 1073-1085.

Sakai, T., Suehiro, T., Tani, T., and Sato, H. 2009. Geotechnical performance of the Kasiwazaki-Kariwa Nuclear Power Station caused by the 2007 Niigataken Chuetsu-oki earthquake. Earthquake Geotechnical Case Histories for Performance-Based Design, CRC Press, Edited by Kokusho, T., pp. 1-29.

Seed, H.B. and Silver, M.L. 1972. Settlement of dry sands during earthquakes. *J. Soil Mech. Found. Div.* ASCE, 98(4), 381-397.

Seed, H.B., Mori, L., and Chan, C.K. 1975. Influence of seismic history on the liquefaction characteristics of sands. Report EERC 75-25, Earthquake Engineering Research Center, University of California, Berkeley.

Silver, M.L. and Seed, H.B. 1971. Volume changes in sands during cyclic loading. *J. Soil Mech. Found. Div.* ASCE, 97(9), 1171-1182.

Tokimatsu, K. and Seed, H.B. 1987. Evaluation of settlements in sand due to earthquake shaking. *J. Geotch. Engrg.* ASCE, 124(6), 479-491.

Yee, E., and Duku, P.M., and Stewart, J.P. 2014. Cyclic volumetric strain behavior of sands with fines of low plasticity. *J. Geotech. Geoenviron. Engrg.* ASCE, 140(4), 04013042.

Yee, E., Stewart, J.P., and Tokimatsu, K. 2013. Elastic and large-strain nonlinear seismic site response from analysis of vertical array recordings. *J. Geotech. Geoenviron. Engrg.* ASCE, 139(10), 1789–1801.