Accounting for the influence of intrinsic soil properties and state variables on liquefaction triggering

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ABSTRACT

Current simplified liquefaction triggering models are limited in their ability to capture the effects of intrinsic soil properties (grain size, mineralogy, grain shape, etc.) and soil state variables (stress state, void ratio, fabric, etc.). This paper presents an overview of a recent study by the authors that proposes the use of the K_γ factor to overcome these limitations. The K_γ factor can be incorporated in developing penetration test, stress-based simplified liquefaction triggering models in place of the currently used K_σ factors. However, K_γ is conceptually very different from K_σ. While most K_σ relationships have largely been empirically based and relate to the soil's cyclic resistance to liquefaction, K_γ is based on equating the shear strain induced in a given soil at a given initial stress state and subjected to a given shear stress to the shear strain induced when the soil is confined at a reference initial stress state, all else being equal. Accordingly, K_γ incorporates the positive attributes of the small-strain shear wave velocity (V_S), stress-based simplified procedure and the cyclic strain procedure into penetration test, stress-based simplified liquefaction triggering models. Numerically, K_γ and K_σ are similar for young, normally consolidated sandy soils when the factor of safety (FS) against liquefaction triggering is close to one, but may differ significantly for other scenarios and/or conditions.

1 INTRODUCTION

The semi-empirical, stress-based simplified procedure, originally proposed by Whitman (1971), and subsequently and independently by Seed and Idriss (1971), is the most commonly used approach for evaluating liquefaction triggering worldwide. Whitman (1971) and Seed (1979) recognized the significance of intrinsic soil properties and soil state variables on liquefaction triggering in liquefiable soils. In this context, intrinsic soil properties include the mineralogy, size, shape, surface characteristics, and gradation of the soil particles, and soil state variables include particle arrangement and packing (i.e. fabric and relative density), cementation, and stress state (e.g. Salgado et al. 1997). Whitman (1971) and Seed (1979) assumed that penetration resistance is similarly influenced by these factors as the soil's resistance to liquefaction triggering is (i.e. Cyclic Resistance Ratio: CRR), such that correlations between normalized penetration resistance and CRR sufficiently account for the influence of intrinsic soil properties and soil state variables on liquefaction triggering. However, this has been shown not to be completely the case.

Subsequent studies have shown that small-strain shear wave velocity (V_S) is also a function of many of the intrinsic soil properties and soil state variables that influence liquefaction triggering (e.g. Tokimatsu et al. 1986; Tokimatsu and Uchida 1990), although the sensitivity of V_S to some of these properties/variables has been questioned (e.g. Verdugo 2016). Nevertheless, several stress-based correlations have been developed relating normalized V_S (i.e. V_{S1}) to CRR (e.g. Andrus et al. 2004), similar to the correlations relating normalized Standard Penetration Test (SPT) blow count ($N_{1,60cs}$) or normalized Cone Penetration Test (CPT) tip resistance (q_{c1Ncs}) to CRR.

Despite the popularity of the stress-based procedures, multiple studies have shown that excess pore water pressure development better correlates to cyclic shear strain than to cyclic stress (e.g. Martin et al. 1975; Dobry et al. 1982). The reason for this is that the relative movement of soil particles during shear, which is necessary for breaking down the soil skeleton and the generation of excess pore water pressures, relates to the induced strain, regardless of the amplitude of the stress applied to the soil. In this vein, Dobry et al. (1982) proposed the cyclic strain procedure as an alternative to stress-based approaches to evaluate liquefaction triggering. The Dobry et al. (1982) strain-based procedure highlights the role of soil shear stiffness (e.g. V_S) in evaluating liquefaction triggering when loading is quantified in terms of shear stress, τ, because V_S (or correspondingly, small strain shear modulus, G_{max}) is the link to the induced strain.

Presented herein is a summary of the study by Green et al. (2022) that proposes an approach for better accounting for the influence of intrinsic soil properties and soil state variables on both the loading and the soil's cyclic resistance to liquefaction by incorporating V_S into stress-based penetration-resistance triggering models. This is done through the newly-proposed K_γ factor, which would replace the K_σ factor in future-developed stress-based simplified models. K_γ is based on equating the shear strain induced in given soil at a given initial stress state and subjected to a given shear stress to the shear strain induced when the soil is confined at a reference initial stress state, all else being equal. In the following, summaries of relevant studies are presented on liquefaction triggering, where loading is quantified in terms of either shear stress or shear strain. Based on the findings from these studies, the conceptual basis for K_γ is then presented. This is followed by a limited validation of the K_γ-concept. The conceptual differences between K_γ vs. K_σ are then briefly discussed.

2 INFLUENCE OF INTRINSIC SOIL PROPERTIES AND SOIL STATE VARIABLES ON LIQUEFACTION TRIGGERING

2.1 Stress-based studies

Several studies have examined the influence of soil fabric on liquefaction resistance. One study performed by Tokimatsu and Uchida (1990) entailed samples of Niigata sand prepared using three different approaches: air pluviation (PA), air pluviation and then subjected to a small-strain seismic history (SH), and air pluviation and then subjected to an overconsolidation history (OC). In addition to varying the sample preparation technique, Tokimatsu and Uchida (1990) also varied the relative density (D_r) of the samples, but the intrinsic soil properties of the soil used and all other soil state variables of the samples were held constant. The D_r of the samples ranged from 48% to 100% and the samples were confined at an initial isotropic effective stress (σ'_o) of 98 kPa (~1 atm). The samples were subjected to stress-controlled cyclic triaxial loading and liquefaction was defined as 5% double-amplitude axial strain. As shown in Figure 1a, the relationship between cyclic resistance to liquefaction and D_r is dependent on the method used to prepare the samples. However, in addition to D_r, Tokimatsu and Uchida (1990) also measured the small-strain (i.e., ~10^{-5}) shear modulus (G_{max}) of the samples using a load transducer and two pairs of gap sensors installed inside the triaxial cell. And, as shown in Figure 1b, there is a very strong correlation between the cyclic resistance to liquefaction and G_{max} for the sand that is independent of the sample preparation technique.

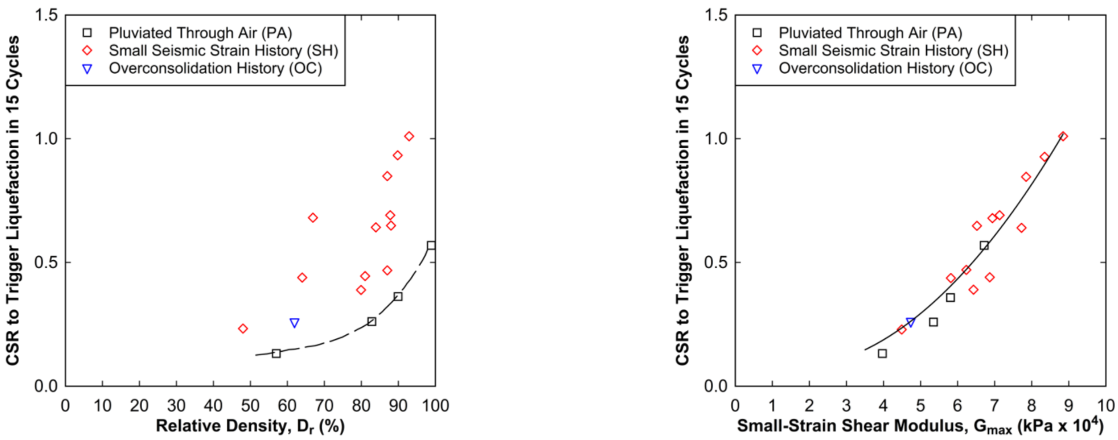


Figure 1: Influence of sample preparation technique on cyclic resistance to liquefaction of Niigata sand, σ'_o = 98 kPa, when samples are characterized in terms of (a) D_r and (b) G_{max}. (Data from Tokimatsu and Uchida 1990)

As significant as the correlation shown in Figure 1b is, the correlation between the cyclic resistance to liquefaction of a soil and the soil's stiffness is strongly dependent on the intrinsic properties of the soil (e.g. Verdugo 2016). This is illustrated in Figure 2, which shows the correlation between the cyclic resistance to liquefaction and small-strain stiffness of the soil for different sands and silts. Figure 2 shows cyclic triaxial test data for Niigata and Toyoura sands prepared using the same three techniques used by Tokimatsu and Uchida (1990): PA, SH, and OC (Tokimatsu et al. 1986). While there is a very strong correlation between G_{max} and cyclic resistance to liquefaction, regardless of the sample preparation technique used, the correlations are unique for each sand due to differences in the intrinsic properties of the sands.

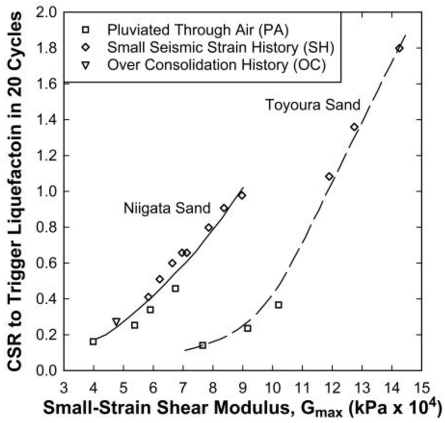


Figure 2: Correlation between cyclic resistance to liquefaction and small-strain stiffness of Niigata and Toyoura sands with soil stiffness quantified in terms of G_{max} for samples prepared using three different techniques, σ'_o = 98 kPa (Data from Tokimatsu et al. 1986)

These trends are not altogether surprising because soil stiffness (e.g. V_S or G_{max}) is a function of the soil's void ratio (e) (e.g. Hardin and Richart 1963), not a function of the soil's D_r per se (Alarcon-Guzman et al.

1989). [Note: V_S and G_{max} are both metrics of shear stiffness and are related through the mass density (ρ_t) of the soil: $G_{max} = \rho_t \cdot V_S^2$.] However, the contractive/dilative tendencies of a soil at small to intermediate strains, which controls the liquefaction response of the soil at a given initial stress state, is more influenced by D_r than it is by the void ratio (e.g. Seed 1979). While D_r and void ratio for a given soil are directly related, one can have different soils that have the same void ratio and initial stress state, and hence have approximately the same G_{max} or V_S, but that have vastly different D_r, and thus, have vastly different cyclic resistances to liquefaction. In short, both D_r and G_{max} (or V_S) are influential parameters in liquefaction triggering.

The initial stress state of a soil also has significant influence on the cyclic resistance to liquefaction of the soil. Seed and Lee (1966) is one of the earliest laboratory studies to systematically examine this. Towards this end, they applied the same cyclic shear stress, τ (= $\sigma_d/2$, where σ_d is the deviatoric stress in triaxial loading), to three sets of Sacramento River sand samples prepared using the same technique and to the same D_r, but confined at different σ'_o: 50, 75, and 100 kPa (or 0.5, 0.75, and 1.0 atm). They found that the higher the σ'_o, the greater the number of stress cycles required to trigger liquefaction. They state that this trend was of special interest because it seemingly contradicted what was expected based on critical state theory (i.e. soil having a given D_r becomes more contractive as effective confining stress increases and thus, expectedly, less resistant to liquefaction triggering). However, in their follow-up study, Lee and Seed (1967) empirically observed that the resistance to liquefaction increased approximately linearly with σ'_o, implying that τ should be normalized by σ'_o. τ/σ'_o is now widely referred to as Cyclic Stress Ratio (CSR), and the CSR required to trigger liquefaction in a specified number of cycles (e.g. 15 cycles) is now widely referred to as Cyclic Resistance Ratio (CRR).

Additional laboratory studies showed, however, that the increase in cyclic resistance to liquefaction is not exactly linear with increasing σ'_o, resulting in the introduction of an additional normalization factor, K_σ, which Seed (1983) defined as:

$$K_\sigma = \frac{CSR_{\sigma'_o}}{CSR_{\sigma'_o = 1\ atm}} \left(or = \frac{CSR_{\sigma'_{vo}}}{CSR_{\sigma'_{vo} = 1\ atm}} \right) \tag{1}$$

where $CSR_{\sigma'o}$ and $CSR_{\sigma'o=1\ atm}$ are the CSR required to trigger liquefaction in a given number of cycles (e.g. 15) in similar samples confined at an initial effective stress of σ'_o and at ~100 kPa (1 atm), respectively. Also, while $CSR_{\sigma'o}$ and $CSR_{\sigma'o=1\ atm}$ are appropriate for representing the loading imposed on isotropically consolidated cyclic triaxial samples, $CSR_{\sigma'vo}$ and $CSR_{\sigma'vo=1\ atm}$ are used to represent the loading imposed on soil in-situ where σ'_{vo} is the initial vertical effective stress acting on the soil at a given depth.

2.2 Strain-based studies

Early studies showed that volumetric strain in a given soil subjected to a given number of loading cycles under drained conditions almost uniquely correlates to the amplitude of the applied cyclic shear strain, γ, rather than the applied τ (or CSR). The corollary of this finding is that the excess pore pressure ratio (r_u: $r_u = \Delta u/\sigma'_{vo}$, where Δu is the excess pore water pressure) in a given saturated soil subjected to cyclic loading under undrained conditions almost uniquely correlates to the amplitude of the applied γ, rather than the applied τ (or CSR) (e.g. Martin et al. 1975). Building on these findings, Dobry et al. (1982) proposed a strain-based approach for evaluating liquefaction triggering as an alternative to the stress-based approach. The procedure entails quantifying the amplitude of the ground shaking in terms of γ and the duration of the shaking in terms of the number of equivalent strain cycles ($n_{eq\gamma}$) and correlating these to r_u, where $r_u \approx 1$ signifies liquefaction triggering.

As shown in Figure 3, one very attractive attribute of quantifying loading in terms of γ and $n_{eq\gamma}$ is that their relationship to r_u is relatively independent of the intrinsic properties of the soil and soil state variables. In this figure, r_u is shown as a function of γ and $n_{eq\gamma} = 10$ for different soil samples having a range of intrinsic soil properties and soil state variables. And, while there is some scatter in the data, the correlation is strong. However, as detailed by Rodriguez-Arriaga and Green (2018), the shortfall of the procedure is correlating γ and $n_{eq\gamma}$ to earthquake ground motions parameters. Aside from the issues related to the implementation of the Dobry et al. (1982) strain-based procedure for evaluating liquefaction triggering, the correlation between γ and $n_{eq\gamma}$ to r_u is very significant and is integral to the proposed K_γ factor, as discussed next.

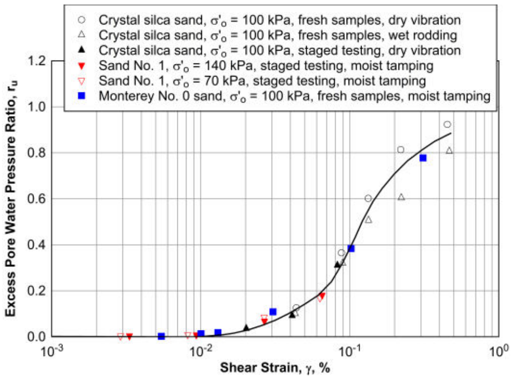


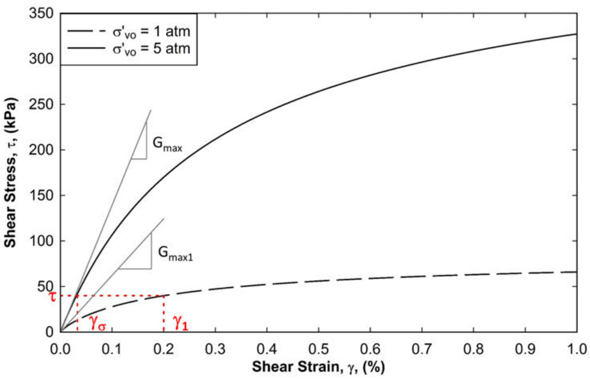
Figure 3: Excess pore water pressure ratio, r_u, for different sands having D_r of 60% after 10 cycles of strain-controlled triaxial loading (Data from Dobry et al. 1982)

3 CONCEPTUAL BASIS FOR THE K_γ FACTOR

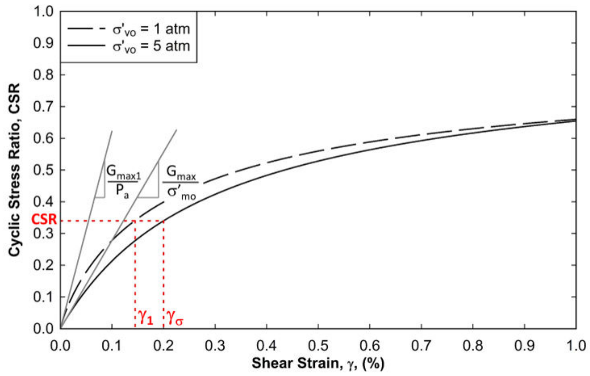
The findings from Dobry et al. (1982) that there is a unique relationship between γ and $n_{eq\gamma}$, and excess pore water pressure generation for a wide range of soils can be used to explain the trends observed by Seed and Lee (1966) and Seed (1983) on the influence of initial stress state on liquefaction triggering. Conceptually this is shown in Figure 4 using τ - γ curves modelled by the shear modulus reduction curve relationship proposed by Ishibashi and Zhang (1993) [IZ93] for cohesionless soil (i.e., Plasticity Index, PI, equal to zero). The IZ93 curves are empirically based, derived from numerous tests performed on multiple soils having a range of intrinsic soil properties and soil state variables.

Figure 4a shows that when a given amplitude τ is imposed on similar soil samples confined at different effective stresses, the shear strain induced in the sample confined at the higher confining stress is much less than that induced in the sample confined at ~100 kPa (1 atm) (i.e., $\gamma_\sigma < \gamma_1$). As a result, the sample confined at the higher confining stress will require more cycles of loading to liquefy (i.e. an apparent higher resistance to liquefaction, despite having higher contractive tendencies at larger strains commensurate with critical state), which is consistent with observations made by Seed and Lee (1966). However, when the amplitude of the load quantified in terms of CSR is imposed on the two samples (Figure 4b), the shear strain induced in the sample confined at the higher confining stress is slightly greater than that induced in the sample confined at ~100 kPa (1 atm) (i.e. $\gamma_\sigma > \gamma_1$). As a result, the sample confined at the higher confining stress will require slightly fewer cycles of loading to liquefy (e.g. Seed 1983). Hence, the additional need to normalize CSR by K_σ is so that when the loading is quantified in terms of CSR/K_σ, $\gamma_\sigma \approx \gamma_1$ and thus, both samples liquefying in approximately the same number of cycles (Figure 4c).

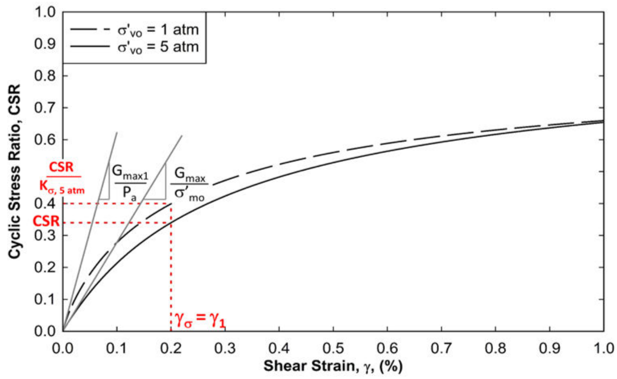
The concept of normalizing τ by σ'_{vo} and the further normalization by K_σ such that the resulting γ induced in a soil confined at the reference condition of $\sigma'_{vo} \approx 100$ kPa (1 atm) is equal to that induced in the same soil confined at σ'_{vo} forms the basis for the proposed K_γ factor. Note: To distinguish the proposed approach from past, largely empirically based approaches, hence forth, the "K_σ values" computed by equating induced shear strains per the approach proposed herein are referred to as K_γ (i.e. computed per Figure 4c), while K_σ is reserved for values computed per Eq. (1) from cyclic laboratory test data.



(a)



(b)



(c)

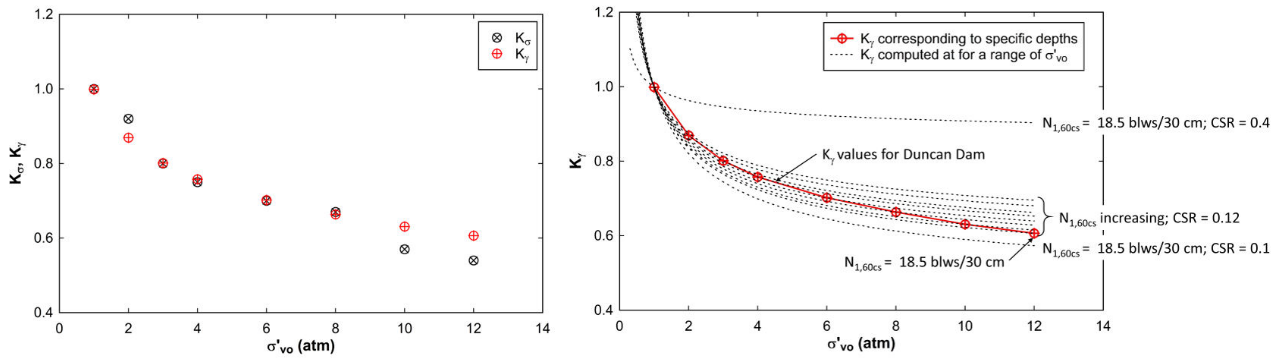
Figure 4: Shear stress–shear strain (τ - γ) response of soil for various conditions: (a) Soil having the same density, but confined at different σ'_{vo} and subjected to cyclic loading of amplitude τ; (b) Same conditions as described in (a) but the amplitude of the cyclic loading is quantified in terms of CSR (i.e., τ/σ'_{vo}); and (c) Same conditions as described in (a) and (b) but the amplitude of the cyclic loading is quantified in terms of CSR/K_σ (or CSR/K_γ)

4 VALIDATION

Pillai and Byrne (1994) performed a detailed liquefaction hazard study of Duncan Dam in British Columbia, Canada. The study entailed performing cyclic triaxial tests and cyclic simple shear tests on undisturbed samples obtained by frozen sampling of the foundation soils of Duncan Dam. The samples were confined at σ'_o and σ'_{vo} up to 1200 kPa (~12 atm), corresponding to different depths of interest below the dam. The K_σ values for the stresses considered are plotted in Figure 5a. To compute K_γ, G_{max} for the samples had to be estimated. Towards this end, V_S was estimated from the normalized SPT N-value ($N_{1,60cs}$) using a slightly modified version of the equation proposed by Ulmer et al. (2020):

$$V_S = 61.89 \cdot \left(N_{1,60cs}\right)^{0.4} \cdot \left(\frac{\sigma'_{mo}}{P_a}\right)^{0.25} \tag{2}$$

where V_S is in m/s; $N_{1,60cs}$ is in blows/30 cm; σ'_{mo} is the initial mean effective confining stress; and P_a is atmospheric pressure in the same units as σ'_{mo}. The modification was made by calibrating the V_S-$N_{1,60cs}$ correlation such that the proposed approach for computing K_γ yielded a similar value to the laboratory determined value for K_σ at σ'_{vo} = 600 kPa (~6 atm), although the K_σ value for any σ'_{vo} could have been used. Again, this modification was slight and results in V_S values that are well within the range of values predicted using $N_{1,60cs}$ via other published correlations. As may be observed from Figure 5a, there is exceptionally good agreement between the K_σ laboratory values and the K_γ values. This comparison serves to validate, to some degree, the K_γ-concept, even though V_S for the samples is estimated and not measured directly.



(a) (b)

Figure 5: (a) K_σ values based on laboratory data for undisturbed samples from Duncan Dam and K_γ values corresponding to specific depths (σ'_{vo} and $N_{1,60cs}$) (data from Pillai and Byrne 1994); and (b) K_γ computed for $N_{1,60cs}$ = 18.5 blws/30 cm but varying CSR for $N_{1,60cs}$ = 18.5 blws/30 cm but varying CSR

5 DISCUSSION AND CONCLUSIONS

Because K_γ is based on equating induced shear strains, it is a function of the small-strain soil stiffness (e.g. V_S or G_{max}), the non-linear stress-strain response of the soil from small to large strains (e.g. via IZ93 shear modulus reduction curve), and the imposed shear stress loading (e.g. CSR). The dependency of K_γ on CSR is in contrast to the laboratory data-based K_σ relationships that inherently assume CSR = CRR (i.e. FS against liquefaction triggering is equal to one). The significance of this can be seen in Figure 5b wherein the K_γ values corresponding to specific CSR and $N_{1,60cs}$ values are plotted; these are the same K_γ values that are plotted in Figure 5a. However, in addition to the K_γ values for specific depths, K_γ curves are computed and plotted for these same CSR and $N_{1,60cs}$ values for a range of overburden pressures. Specific to the K_γ values for Duncan Dam, the $N_{1,60cs}$ increases with each depth, but the CSR values corresponding to a FS = 1 are the same (i.e. CSR = 0.12) (Pillai and Byrne 1994). However, if we consider, for example, $N_{1,60cs}$ = 18.5 blws/30 cm, which is the value of $N_{1,60cs}$ at σ'_{vo} = 1200 kPa (~12 atm), and CSR = 0.1 and 0.4 (i.e. FS > 1 and FS < 1, respectively), the K_γ curves are very different. The reason for this is that the shear strain induced in the soil is a function of the imposed CSR and that the contractive/dilative tendencies of the soil vary significantly as a function of shear strain up to critical state.

The proposed K_γ factor should be used in place of K_σ in future-developed simplified triggering models. The K_γ relationship proposed herein is not intended to be used in conjunction with existing simplified liquefaction evaluation procedures to assess liquefaction potential. This is because the CRR curves for existing triggering procedures are inherently based on K_σ relationships used in the development of those CRR curves. As a result, the K_γ relationships proposed herein can be used to analyse liquefaction case histories to develop new cyclic resistance ratio curves and/or to quantify the differences in K_σ vs. K_γ to assess bias in K_σ relationships proposed by others, etc.

6 REFERENCES

Alarcon-Guzman A, Chameau JL, Leonards GA and Frost JD (1989). “Shear modulus and cyclic undrained behavior of sands”. *Soils and Foundations*, **29**(4): 105-119.

Andrus RD, Stokoe II KH and Juang H (2004). “Guide for shear-wave-based liquefaction potential evaluation”. *Earthquake Spectra*, **20**(2): 285-308.

Dobry R, Ladd R, Yokel F, Chung R and Powell D (1982). "Prediction of pore water pressure buildup and liquefaction of sands during earthquakes by the cyclic strain method". *NBS Building Science Series 138*, National Bureau of Standards, Washington, DC.

Green RA, Bradshaw A and Baxter CDP (2022). "Accounting for intrinsic soil properties and state variables on liquefaction triggering in simplified procedures". *Journal of Geotechnical and Geoenvironmental Engineering*, **148**(7): 04022056.

Hardin BO and Richart Jr FE (1963). "Elastic wave velocities in granular soils". *Journal of the Soil Mechanics and Foundations Division*, ASCE, **89**: 33-65.

Ishibashi I. and Zhang X. (1993). "Unified dynamic shear moduli and damping ratios of sand and clay". *Soils and Foundations*, **33**(1): 182–191.

Lee KL and Seed HB (1967). "Cyclic stress conditions causing liquefaction of sands". *Journal of the Soil Mechanics and Foundations Division*, **93**(SM1): 47-70.

Martin GR, Finn WDL and Seed HB (1975). "Fundamentals of liquefaction under cyclic loading". *Journal of the Geotechnical Engineering Division*, **101**(GT5): 423–483.

Pillai VS and Byrne PM (1994). "Effect of overburden pressure on liquefaction resistance of sand". *Canadian Geotechnical Journal*, **31**(1): 53–60.

Rodriguez-Arriaga E and Green RA (2018). "Assessment of the cyclic strain approach for evaluating liquefaction triggering". *Soil Dynamics and Earthquake Engineering*, **113**: 202-214.

Salgado R, Boulanger RW and Mitchell JK (1997). "Lateral stress effects on CPT liquefaction resistance correlations". *Journal of Geotechnical and Geoenvironmental Engineering*, **123**(8): 726-735.

Seed HB (1979). "Soil liquefaction and cyclic mobility evaluation for level ground during earthquakes". *Journal of the Geotechnical Engineering Division*, ASCE, **105**(GT2): 201–255.

Seed HB (1983). "Earthquake resistant design of earth dams". *Proc. Symposium on Seismic Design of Embankments and Caverns*, Pennsylvania, ASCE, NY: 41–64.

Seed HB and Idriss IM (1971). "Simplified procedure for evaluating soil liquefaction potential". *Journal of the Soil Mechanics Foundation Division*, **97**(9): 1249–1273.

Seed HB and Lee KL (1966). "Liquefaction of saturated sands during cyclic loading". *Journal of the Soil Mechanics and Foundations Division*, **92**(SM6): 105-134.

Tokimatsu K and Uchida A (1990). "Correlation between liquefaction resistance and shear wave velocity". *Soils and Foundations*, **30**(2): 33-42.

Tokimatsu K, Yamazaki T and Yoshimi Y (1986). "Soil liquefaction evaluations by elastic shear moduli". *Soils and Foundations*, **26**(1): 25-35.

Ulmer KJ, Green RA and Rodriguez-Marek A (2020). "A consistent correlation between Vs, SPT, and CPT metrics for use in liquefaction evaluation procedures". *Geo-Congress 2020: Geotechnical Earthquake Engineering and Special Topics*, ASCE Geotechnical Special Publication (GSP) 318: 132-140.

Verdugo R (2016). "Experimental and conceptual evidence about the limitations of shear wave velocity to predict liquefaction". *Soil Dynamics and Earthquake Engineering*, **91**: 160-174.

Whitman RV (1971). "Resistance of soil to liquefaction and settlement". *Soils and Foundations*, **11**(4): 59-68.