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Assessment of an alternative implementation of the Dobry et al. cyclic strain procedure for evaluating liquefaction triggering

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ABSTRACT: Despite its fundamental basis and many positive attributes, the cyclic strain approach has not been embraced by practice for evaluating liquefaction triggering. One reason for this may be the need to perform cyclic laboratory tests to develop a relationship among excess pore water pressure, cyclic strain amplitude, and number of applied strain cycles. Herein an alternative implementation of the strain-based procedure is proposed that circumvents this requirement. To assess the efficacy of this alternative implementation, Standard Penetration Test field liquefaction case histories are evaluated. The results are compared with both field observations and with predictions from a stress-based procedure. It was found that the strain-based approach yields overly conservative predictions. Also, a potentially fatal limitation of the strain-based procedure is that it ignores the decrease in soil stiffness due to excess pore pressure when representing the earthquake loading in terms of shear strain amplitude and number of equivalent cycles.

1 INTRODUCTION

Despite the popularity of the simplified stress-based liquefaction triggering evaluation procedures (e.g., Boulanger et al. 2012), multiple studies have shown that excess pore water pressure better correlates to cyclic strain than to cyclic stress (e.g., Figure 1) (e.g., Martin et al. 1975; Dobry et al. 1982). The reason for this is the relative movement of soil particles, which is requisite for excess pore water pressure generation, better relates to the induced strain. As a result, Dobry et al. (1982) proposed a strain-based liquefaction triggering evaluation procedure. Although the Dobry et al. (1982) procedure generally received a positive reception by liquefaction researchers, it has failed to be adopted into practice. One reason for this is likely the requirement to perform strain-controlled cyclic laboratory tests on undisturbed and/or reconstituted specimens. This is in contrast to the simplified stress-based procedures wherein in-situ test metrics are the primary parameters used to evaluate liquefaction potential, with laboratory index tests and grain size distribution analyses having supporting roles if their performance is deemed necessary (e.g., use of measured fines content, FC, versus apparent FC in conjunction with the Cone Penetration Test, CPT, stress-based simplified procedure).

Herein an alternative implementation of the Dobry et al. (1982) strain-based procedure is proposed which circumvents the need for performing strain-controlled cyclic laboratory tests. Per this procedure, a strain-based numerical excess pore pressure generation model is used in lieu of developing analogous relationships from laboratory tests. The soil parameters required to implement the procedure for clean sands (i.e., fines content, FC, ≤ 5%) include: relative density (Dr), secant shear modulus (G), and shear modulus degradation curve (i.e., G/G_{max} vs. γ_c, where G_{max} is the small-strain shear modulus and γ_c is the amplitude of the cyclic shear strain); note that focus herein is on soils that are susceptible to liquefaction (i.e., non-plastic soils) and thus Plasticity Index (PI) is not needed (e.g., Green & Ziotopoulou 2015). These required parameters are not too onerous and can be estimated using simple relationships or conservative assumptions.

To assess the efficacy of the proposed variant of the Dobry et al. (1982) strain-based procedure, 116 clean sand liquefaction/no-liquefaction case histories compiled by Boulanger et al. (2012) are evaluated. Accordingly, the efficacy of the strain-based procedure can be assessed

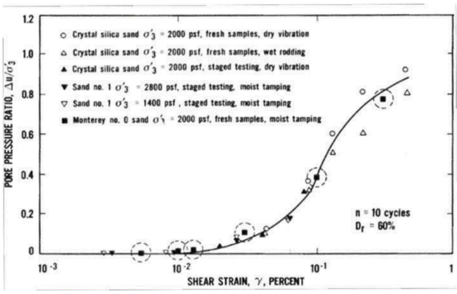

Figure 1. Porewater pressure ratios in cyclic triaxial strain-controlled tests, after ten loading cycles, as a function of cyclic shear strain, for various normally consolidated saturated sand specimens. The specimens were prepared to Dr ≈ 60% using various techniques and confined at various initial effective confining stresses. (Dobry et al. 1982)

both in an absolute sense (i.e., with respect to field observations) and in a relative sense (i.e., relative to the efficacy of the Boulanger et al. 2012 stress-based procedure).

The following sections present the background information related to the cyclic strain approach as originally proposed by Dobry et al. (1982) and the alternative implementation proposed herein. Next, the case histories are analyzed and the results from the analyses are presented. This is followed by a discussion of the analysis results and possible reasons for the observed trends.

2 STRAIN-BASED PROCEDURE

2.1 *Dobry et al. (1982) procedure*

Early studies showed that volumetric strain in a given soil subjected to cyclic loading under drained conditions almost uniquely correlates with γ_c, rather than the applied cyclic stress (γ_c) (e.g., Silver & Seed 1971). 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 and γ'_{vo} is the initial vertical effective stress) in a given saturated soil subjected to cyclic loading under undrained conditions almost uniquely correlates with the amplitude of the applied γ_c, rather than the applied γ_c (e.g., Martin et al. 1975). Building on these findings, Dobry et al. (1982) proposed a strain-based approach for evaluating liquefaction triggering potential, as an alternative to the stress-based approach.

Starting with the simplified equation to compute γ_c (e.g., Whitman 1971; Seed & Idriss 1971), Dobry et al. (1982) proposed a simplified equation to compute γ_c:

$$\gamma_c = \frac{\tau_c}{G} = 0.65\left(\frac{a_{max}}{g}\right)\frac{\sigma_v r_d}{G_{max}(G/G_{max})_{\gamma_c}} \tag{1}$$

where G = secant shear modulus of the soil; a_{max} = the peak horizontal acceleration at the surface of the soil profile; g = acceleration due to gravity; σ_v = total vertical stress at given depth in the soil profile; r_d = depth-stress reduction factor that accounts for the non-rigid response of the soil profile; G_{max} = small-strain ($\gamma_c \le 10^{-4}$%) secant shear modulus of the soil; and $(G/G_{max})_{\gamma c}$ = normalized secant shear modulus reduction ratio of the soil corresponding to γ_c. Dobry et al. (1982) found that there is a limiting value of γ_c, below which no excess pore water pressures develop, regardless of the number of applied load cycles (n_{eq}); they referred to this limiting value of γ_c as the threshold volumetric shear strain (γ_{tv}). Dobry et al. (1982) found that for normally consolidated clean and silty sands $\gamma_{tv} \approx 0.01$%.

The strain-based liquefaction triggering evaluation procedure proposed by Dobry et al. (1982) consists of three steps:

Step 1. Determination of γ_c and n_{eq}: γ_c is calculated using Eq. 1 and n_{eq} can be obtained from established correlations with earthquake parameters.

Step 2. Comparison between γ_c and γ_{tv}:

(a) If $\gamma_c \le \gamma_{tv}$, neither pore pressure buildup nor liquefaction will occur and the evaluation ends here.

(b) If $\gamma_c > \gamma_{tv}$, the values of γ_c and n_{eq} are used in conjunction with experimental curves developed from strain-controlled cyclic tests performed on undisturbed and/or reconstituted samples prepared to the same Dr as the soil in-situ to estimate r_u at the end of earthquake shaking.

Step 3. Determination whether liquefaction triggered: The value of r_u estimated in Step 3 is used to decide if the site will experience initial liquefaction ($r_u \approx 1.0$) or not ($r_u < 1.0$).

2.2 *Alternative implementation of Dobry et al. (1982) procedure*

The alternative implementation of the Dobry et al. (1982) strain-based procedure circumvents the need for performing strain-controlled cyclic laboratory tests (Step 2b), which is viewed by the authors as being, historically, the primary impediment of the procedure for use in practice. Per the alternative implementation of the procedure, the use of a strain-based numerical excess pore pressure generation model is proposed in lieu of developing analogous relationships from laboratory tests. The alternative implementation of the procedure is outlined below.

2.2.1 *Determination of γ_c and n_{eq}*

Step 1 of the Dobry et al. (1982) strain-based procedure is to determine γ_c and n_{eq}, which represent the amplitude and duration of the applied earthquake loading. In Eq. 1, the stiffness of the soil is represented by $G_{max}\cdot(G/G_{max})_{\gamma c}$, where G_{max} can be computed from the small-strain shear wave velocity (V_s) and relationships for G/G_{max} as a function of γ_c have been proposed by several investigators; in this study the relationship proposed by Ishibashi & Zhang (1993) is used. These relationships often include predictive variables such as mean effective confining stress (σ'_{mo}), PI, overconsolidation ratio (OCR), etc. Because the G/G_{max} relationships are expressed as a function of γ_c, an iterative procedure is required to determine $(G/G_{max})_{\gamma c}$ and, hence, γ_c using Eq. 1. In implementing the iteration algorithm to compute γ_c, a maximum cap of 3% was imposed. This was done because it is doubtful that strains larger than this are induced in-situ solely as a result of earthquake shaking and the validity of the shear modulus degradation curves become questionable at larger strains. Also, in using Eq. 1 to compute γ_c in this study, the r_d relationship proposed by Lasley et al. (2016) for active, shallow crustal tectonic regimes (e.g., western United States: WUS) was employed.

Per Step 1 of the Dobry et al. (1982) procedure, n_{eq} of the earthquake loading is also required, which Dobry et al. (1982) states is a function of earthquake magnitude. At the time of the writing of Dobry et al. (1982), a few relationships for equivalent number of stress cycles ($n_{eq\gamma}$) had been developed (e.g., Seed et al. 1975), but the authors are not aware of any equivalent number of strain cycles ($n_{eq\gamma}$) relationships existing at that time. Accordingly, Dobry et al. (1982) likely assumed that $n_{eq\tau}$ and $n_{eq\gamma}$ were equivalent, which is not necessarily the case (e.g., Green &Terri 2005). Even today, few relationships have been developed for $n_{eq\gamma}$ (e.g., Green & Lee 2006; Lee & Green 2017), and these relationships were developed for evaluating seismic compression in dry or partially saturated soils, not for evaluating liquefaction in saturated soils.

Despite its questionable applicability for use in a strain-based liquefaction evaluation procedure, the $n_{eq\gamma}$ relationship proposed by Lasley et al. (2017) is used in this study. The reason for selecting this relationship is because it was more rigorously developed than other existing $n_{eq\gamma}$ and $n_{eq\gamma}$ relationships and none of the alternative relationships are any more applicable for use in the strain-based liquefaction procedure than the Lasley et al. (2017) relationship. Additionally, the relationship accounts for the combined influence of both horizontal components of motion, while most competing relationships only consider the influence of a single horizontal component of motion.

2.2.2 *Threshold shear strain*

Step 2a of the Dobry et al. (1982) strain-based procedure determines whether $\gamma_c < \gamma_{tv}$. As stated previously, for normally consolidated clean and silty sands $\gamma_{tv} \approx 0.01\%$. This strain value was determined experimentally where the results of cyclic testing showed that when $\gamma_c \le$

0.01% no excess pore water pressures were generated, even when subjected to a large number of cycles. In addition, Dobry et al. (1982) presented analytical results, using a simple cubic array of quartz spheres, where the calculations also showed $\gamma_{tv} \approx 0.01\%$.

2.2.3 *Excess pore pressure generation for* $\gamma_c > \gamma_{tv}$

Step 2b of the Dobry et al. (1982) strain-based procedure considers the scenario when $\gamma_c > \gamma_{tv}$. For this scenario, excess pore pressures will develop in the soil, and the magnitude of the generated excess pore pressures need to be estimated to determine whether liquefaction will be triggered (Step 3). In lieu of developing a relationship between r_u and γ_c from laboratory tests, herein it is proposed that a strain-based numerical pore pressure generation model be used for this purpose. Specifically, the model proposed by Byrne (1991) is used herein. Byrne (1991) simplified the Martin et al. (1975) strain-based pore pressure generation model, which relates the increment of volumetric strain that would occur under drained conditions in a soil having a given Dr when subjected to a half cycle of loading of amplitude γ_c to the increment in excess pore water pressure that would have been generated in the soil under undrained conditions. The model is given as:

$$(\Delta_v)_{1/2cycle} = 0.5 \cdot (\gamma_c - \gamma_{tv}) \cdot C_1 \cdot e^{\left\{-C_2\frac{\varepsilon_v}{(\gamma_c - \gamma_{tv})}\right\}} \tag{2a}$$

where: $(\Delta\varepsilon_v)_{1/2\ cycle}$ is the increment in volumetric strain resulting from the i^{th} half cycle of loading; ε_v is the accumulated volumetric strain at the end of the (i-1) half cycle of loading; γ_c is the amplitude of the induced shear strain in the soil subjected to the i^{th} half cycle of loading; and C_1 and C_2 are calibration coefficients. Although not consistent with the value of γ_{tv} recommended Byrne (1991) (i.e., γ_{tv} = 0.00005 m/m), γ_{tv} = 0.01% was used in implementing the model in this study to be consistent with the strain-based liquefaction evaluation procedure proposed by Dobry et al. (1982), as discussed above.

Byrne (1991) showed that C_1 and C_2 are correlated and are a function of D_r, or penetration resistance, and proposed the following expressions for the coefficients for clean sands:

$$C_1 = 8.7 \cdot \left(N_{1,60}\right)^{-1.25} \tag{2b}$$

$$C_2 = 0.4/C_1 \tag{2c}$$

where $N_{1,60}$ is the Standard Penetration Test (SPT) blow count normalized for 60% hammer energy and 1 atm of vertical effective confining stress. The volumetric strain that occurs in a given soil sample subjected to a half cycle of loading under drained conditions is related to the excess pore pressure that would be generated in the soil sample under undrained conditions, $(\Delta u)_{1/2\ cycle}$, via the constrained modulus (M):

$$(\Delta u)_{1/2cycle} = M \cdot (\Delta\epsilon_v)_{1/2cycle} \tag{2d}$$

Byrne (1991) recommends the following relationship to estimate M:

$$M = K_m P_a \left(\frac{\sigma'_v}{P_a}\right)^m \tag{2e}$$

where σ'_v is the effective stress at the end of the (i-1) half cycle of loading; P_a is atmospheric pressure; and K_m and m are calibration coefficients. Byrne (1991) found that $K_m \approx 1600$ and $m \approx 0.5$ give moduli that are in good agreement with values reported by Martin et al. (1975) as well as results of liquefaction tests.

The excess pore water pressure, and corresponding r_u, that is generated in the soil sample after $n_{eq\gamma}$ cycles of loading (or $2 \times n_{eq\gamma}$ half cycles of loading) are given as:

$$\Delta u = \sum_1^{2\cdot n_{eq\gamma}} (\Delta u)_{1/2\ cycle} \tag{2f}$$

$$r_u = \frac{\Delta u}{\sigma'_{vo}} \leq 1.0 \tag{2g}$$

2.2.4 *Assessing whether liquefaction triggers*

The final step in the Dobry et al. (1982) strain-based procedure (i.e., Step 3) is to evaluate whether liquefaction is triggered. Dobry et al. (1982) defined liquefaction as $r_u = 1$; therefore, a value of r_u computed from Step 2b that is less than 1 implies that liquefaction is not triggered. However, depending on the density of the soil, $r_u < 1$ can still result in damage to nearby infrastructure. Accordingly, as discussed subsequently, defining liquefaction by an $r_u \le 1.0$ is considered appropriate. Note that r_u is unknown for the vast majority, if not all, the field case histories used to develop the stress-based liquefaction procedures; for these case histories, surficial manifestations, not r_u, were used to infer whether liquefaction was triggered or not.

3 ANALYSIS OF FIELD CASE HISTORIES

The efficacy of the alternative implementation of the Dobry et al. (1982) strain-based procedure is assessed by analyzing clean sand SPT field case histories compiled by Boulanger et al. (2012). The Boulanger et al. (2012) SPT database is composed of 230 case histories, where 115 were catalogued as "Liquefaction" cases, 112 were catalogued as "No Liquefaction" cases, and three cases were considered "Marginal." However, due to the uncertainty in the ability of the Byrne (1991) model to predict excess pore water pressure generation in silty sands and silts, only a subset of the SPT case histories that have a FC ≤ 5% was analyzed in this study. This subset consisted of 116 cases: 62 Liquefaction cases and 54 No Liquefaction cases.

3.1 *Parameter estimation*

The Boulanger et al. (2012) database include information about a_{max}, σ'_{vo}, σ_v, and depth of the critical layer for each case history. To implement the alternative form of the Dobry et al. (1982) strain-based procedure to evaluate the SPT case histories, the following additional parameters need to be estimated: small-strain shear wave velocity (V_s), total unit weight (γ_t) of the critical layer, and Dr of the critical layer.

To estimate V_s for each of the SPT case histories, the relationship proposed by Wair et al. (2012) for all Holocene aged soils was used:

$$V_s = 26.0 \cdot {N_{60}}^{0.215} \cdot {\sigma'_{vo}}^{0.275} \tag{3}$$

where N_{60} is the SPT blow count normalized for 60% hammer energy, V_s is in m/sec, and σ'_{vo} is in kPa. Values of γ_t between 16 and 20.5 kN/m^3 were assumed to match the reported values of σ'_{vo} and σ_v listed in the database. Eq. 4 was then used to compute G_{max}:

$$G_{max} = {V_S}^2 \cdot \frac{\gamma_t}{g} \tag{4}$$

The following relationship presented by Idriss & Boulanger (2008) for sands was used to obtain Dr from corrected SPT N-value (i.e., $N_{1,60}$):

$$D_r = \sqrt{\frac{N_{1,60}}{46}} \tag{5}$$

where D_r is constrained between 30% and 90%.

3.2 *Results*

The results from the analysis of the field case histories are listed in Table 1. In evaluating the results, the strict definition of liquefaction (i.e., $r_u = 1$) was relaxed some to a more pragmatic value of $r_u = 0.95$, where this value can potentially lead to surficial liquefaction manifestations. The results in Table 1 are expressed in terms of True Positive, True Negative, False Positive, and False Negative, which are defined as:

- True Positive: liquefaction is predicted and was observed (i.e., it was a "Liquefaction" case).

Table 1. Prediction accuracy

Prediction	Procedure Accuracy	
	Strain-Based	Stress-Based
True Positive	52%	52%
True Negative	25%	38%
Accurate Predictions	77%	90%
False Positive	21%	8%
False Negative	2%	2%
Incorrect Predictions	23%	10%

- True Negative: liquefaction is not predicted and was not observed (i.e., it was a "No Liquefaction" case).
- False Positive: liquefaction is predicted but was not observed (i.e., it was a "No Liquefaction" case).
- False Negative: liquefaction is not predicted but was observed (i.e., it was a "Liquefaction" case).

Accordingly, True Positives and True Negatives are accurate predictions, False Positive is an inaccurate and overly conservative prediction, and False Negative is an inaccurate and unconservative prediction.

In addition to the alternative implementation of the Dobry et al. (1982) strain-based procedure, the case histories were also analyzed using the Boulanger et al. (2012) SPT stress-based procedure. The results from these analyses are also listed in Table 1.

4 DISCUSSION

As may be observed from the results presented in Table 1, both the strain- and stress-based procedure yield far more accurate predictions than incorrect predictions (i.e., 77% and 90% vs. 23% and 10%). However, in comparing the efficacy of the two procedures, the stress-based procedure is superior to the strain-based procedure (i.e., 90% vs. 77% accuracy rate), with both procedures yielding same percentage of unconservative mispredictions (i.e., 2%) and the strain-based procedure yielding a significantly larger percentage of over-conservative mispredictions (i.e., 21% vs. 8%).

However, it should be noted that the case history database used to assess the efficacy of the stress-based procedure is a subset of the one used to develop the cyclic resistance ratio (CRR) curve inherent to Boulanger et al. (2012) stress-based procedure (i.e., the stress-based procedure was in essence "calibrated" using the case histories analyzed, along with others), while this is not the case for the strain-based procedure. From this perspective the prediction statistics listed in Table 1 are inherently biased in favor of the stress-based procedure. Nevertheless, other possible reasons that the strain-based procedure performed inferiorly to the stress-based procedure are discussed in the following.

4.1 *Cyclic shear strains and number of equivalent cycles*

The "simplified" procedure to compute γ_c proposed by Dobry et al. (1982) (i.e., Eq. 1) inherently assumes that $G_{max}\cdot(G/G_{max})_{\gamma c}$ is uninfluenced by the softening of the soil due to the generation of excess pore water pressures. However, this is known not to be the case when $\gamma_c > \gamma_{tv}$. Dobry et al. (1982) allude to this, stating that "…some additional research is needed to develop definite rules for computing γ_c." The authors actually view this as an inherent and potentially fatal limitation of the strain-based procedure. The representation of chaotic earthquake ground motions in an "equivalently damaging" and simplified form requires the specification of the simplified motion's amplitude (e.g., γ_c) and duration (e.g., $n_{eq\gamma}$). If it is assumed that $G_{max}\cdot(G/G_{max})_{\gamma c}$ is uninfluenced by the generation of excess pore water pressures in computing γ_c, then the softening of the soil due to excess pore pressure generation needs to be accounted for in computing $n_{eq\gamma}$. However, assuming that $n_{eq\gamma}$ is equivalent to $n_{eq\gamma}$, where the latter is computed using a "total stress" approach (e.g., Seed et al. 1975; Green and Terri 2005) does not satisfy this need. Furthermore, to the authors' knowledge, no existing $n_{eq\gamma}$ relationship accounts for the softening effects of the soil due to excess pore water, nor has any framework been proposed in literature on how to compute $n_{eq\gamma}$ that accounts for the softening effects of the soil due to excess pore water.

As an alternative to requiring $n_{eq\gamma}$ to account for the softening effects due to excess pore water pressure, γ_c could be computed for each half cycle of loading using Eq. 1, wherein the effective confining stress used to compute $G_{max}\cdot(G/G_{max})_{\gamma c}$, and hence γ_c, is updated to account for excess pore pressure generation each half cycle of loading. This would require that relationships such as that shown in Figure 1 be generated for $n_{eq}\gamma = 0.5$ cycles. This is certainly feasible, but in essence, this is what the Byrne (1991) strain-based pore pressure generation model does, with the Byrne (1991) model being more versatile than the Dobry et al. (1982) strain-based liquefaction evaluation procedure.

4.2 *Uncertainties*

The analyses performed in this study are inherently deterministic (i.e., liquefaction triggering is evaluated via alternative implementation of the strain-based procedure using best estimates of excess water pressure ratios and the deterministic version of the SPT simplified stress-based procedure). As a result, to assess whether the epistemic uncertainty associated with certain aspects of the alternative implementation of the strain-based procedure has a significant influence on the results presented above, parametric analyses were performed using the Vucetic & Dobry (1986) strain-based pore pressure generation model (versus the Byrne 1991 model), using the Darendeli (2001) secant shear modulus degradation relationship (versus the Ishibashi & Zhang 1993 relationship), and using a maximum cap of 1% on the computed γ_c (versus 3%) via the secant shear modulus degradation relationship. Of these, the choice of the strain-based excess pore water pressure generation model has the biggest influence on the computed results (Rodirguez-Arriaga & Green 2018), with the Byrne (1991) model resulting in a higher percentage of accurate, as well as overly conservative, predictions than when the Vucetic & Dobry (1986) model is used.

Finally, having measurements of both V_s and penetration resistance for sites being evaluated would alleviate the need for Eq. 3 and would likely improve the accuracy of the strain-based approach because there would be less reliance of correlations to estimate needed parameters. However, it is unknown whether this would significantly improve the efficacy of the procedure to make it competitive with the stress-based procedure.

5 CONCLUSIONS

The existence of a volumetric threshold shear strain, below which there is no development of excess pore pressures, and the unique relationship between excess pore pressures and cyclic shear strain, make compelling arguments for adopting a strain-based approach for evaluating liquefaction potential. Herein an alternative implementation of the Dobry et al. (1982) cyclic strain approach is assessed by evaluating liquefaction triggering using clean sand SPT case histories. Toward this end, γ_c was computed using the Dobry et al. (1982) procedure in conjunction with shear modulus degradation curves by Ishibashi & Zhang (1993).

For cases where $\gamma_c > 0.01\%$ (i.e., the threshold shear strain), excess pore pressures are predicted to develop and it becomes necessary to quantify these pore pressures to evaluate liquefaction potential. This was accomplished by implementing the pore pressure generation model by Byrne (1991) and the correlation by Lasley et al. (2017) to estimate n_{eq}. In comparing the efficacies of the strain-based and stress-based procedures, it was observed that the stress-based procedure yielded more accurate predictions than the strain-based procedure, with the strain-based procedure yielding as significantly higher percentage of False Positive (i.e., conservative) mispredictions. Additionally, the efficacy of the strain-based procedure significantly decreased when the Vucetic & Dobry (1986) strain-based excess pore pressure model was used in lieu of the Byrne (1991) model. However, it should be noted that the database used to assess the efficacy of the stress-based procedure is a subset of the that used to develop the CRR curve inherent to the procedure, while this is not the case for the strain-based procedure. From this perspective the comparison of the efficacies is inherently biased in favor of the stress-based procedure. Nevertheless, one likely reason for the lack of accuracy in the strain-based procedure's predictions is the inherent and potentially fatal limitation of the procedure ignoring the softening of the soil stiffness due to excess pore pressure when representing the earthquake loading in terms of γ_c and $n_{eq\gamma}$. This shortcoming relates to both the original variant of the Dobry et al. (1982) strain-based procedure, as well as the alternative implementation of the procedure proposed herein.

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