



Yield Strength Approach to Evaluate the Triggering of Liquefaction for Sloping Ground

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ABSTRACT:

The yield strength ratio concept, $S_u(\text{yield})/\sigma'_{v0}$ (Olson & Stark, 2003) and the cyclic strength ratio CRR (Seed et al, 1984) remain the main approaches of assessing triggering of liquefaction resistance for sloping ground. The K_α correction factor (Seed, 1983) has been extensively studied for considering the effect of the initial static stress on the triggering of liquefaction of sloping ground. Based on a suite of liquefaction triggering analyses, these most-commonly used cyclic stress methods, Seed et al. (2003)/Cetin et al. (2004) and Idriss and Boulanger (2008), are compared to the yield strength ratio approach (Olson and Stark 2003; Olson et al. 2006; Mesri 2007; and Olson and Zitny 2012), and the efficacy of the yield strength ratio is demonstrated. For validation of the comparison, one Mw 7.0 Haiti (2010) earthquake flow failures cases (North River near Dechapelle), (Olson et al., 2010) was selected to estimate the triggering of liquefaction and post-liquefaction resistance.

INTRODUCTION

Methods to evaluate the triggering of liquefaction in sloping ground (i.e., ground subjected to a static driving shear stress) were first proposed nearly three decades ago. Because of the limited number of well-documented case histories of liquefaction in sloping ground, the majority of these methods were based on extending the widely-used cyclic stress approach (Seed and Idriss 1971; Whitman 1971) used to evaluate the triggering of liquefaction under level ground. Olson and Stark (2003), Olson et al. (2006), Mesri (2007), and Olson and Zitny (2012) proposed an alternate approach to evaluate sloping ground liquefaction using a yield strength ratio. The yield strength ratio, $s_u(\text{yield})/\sigma'_{v0}$ is a function of the static shear stress ratio [where $s_u(\text{yield})$ is the peak, or yield, shear strength mobilized under undrained conditions in contractive soils and σ'_{v0} is the prefailure vertical effective stress]. If liquefaction is triggered in sloping ground, a post-triggering stability analysis is performed using either the liquefied shear strength, $s_u(\text{liq})$, or the liquefied shear strength ratio, $s_u(\text{liq})/\sigma'_{v0}$ for the liquefied soils. Seed and Harder (1990) proposed a widely-used correlations for liquefied (or undrained residual) shear strength, while Terzaghi et al. (1996), Olson and Stark (2002), Mesri (2007), and Idriss and Boulanger (2008) have proposed correlations for $s_u(\text{liq})/\sigma'_{v0}$.

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The 12 January 2010 Mw 7.0 Haiti earthquake triggered extensive liquefaction failures (i.e. lateral spreads, bank slumps) along the Gulf of Gonave coastline and along rivers north of Port-au-Prince, causing considerable damage. Based on two reconnaissance missions after the earthquake (January/February 2010 and April 2010), Olson et al. (2010) documented eight liquefaction case histories. The geotechnical reconnaissance effort involved surface mapping, dynamic cone penetration tests (DCPT), hand auger borings, and laboratory index test.

In this study, the methods above to evaluate the triggering of liquefaction in sloping ground and post-triggering slope stability are compared using two liquefaction-induced slope failures triggered by the 2010 Haiti earthquake.

APPROACHES TO EVALUATE LIQUEFACTION OF SLOPING GROUND

The widely-used cyclic stress approach relates the seismic shear stress ratio, $\tau_{\text{seismic}}/\sigma'_{\text{vo}}$, required to trigger liquefaction (or cyclic resistance ratio, CRR) with the overburden stress-normalized standard penetration test (SPT) resistance, $(N_1)_{60}$. From its original form (i.e., Seed and Idriss 1971; Whitman 1971), numerous updates have been published as new field case histories become available. In addition, recent studies have proposed modifications to estimating $\tau_{\text{seismic}}/\sigma'_{\text{vo}}$, e.g., revisions to the magnitude scaling factor (MSF) and depth reduction factor (r_d), and have introduced probabilistic tools (e.g., Cetin et al. 2004; Idriss and Boulanger 2008). As this method was developed for level ground, additional corrections have been proposed to apply the approach to sloping ground, including a high overburden stress (K_σ ; Youd et al. 2001; Seed et al. 2003; Idriss and Boulanger 2008) and static shear stresses correction (K_α ; Seed 1983; Rollins and Seed 1990; Seed and Harder 1990; Harder and Boulanger 1997; and Boulanger 2003). Using this approach, the sloping-ground liquefaction resistance, or yield strength ratio, can be estimated as:

$$(1) \quad \frac{s_u(\text{yield})}{\sigma'_{\text{vo}}} = \frac{\tau_{\text{static}}}{\sigma'_{\text{vo}}} + K_\alpha K_\sigma \text{CRR}$$

As an alternative, Olson and Stark (2003) back-analyzed 10 static loading- and deformation-induced liquefaction flow failures case histories to evaluate the yield strength ratio [$s_u(\text{yield})/\sigma'_{\text{vo}}$] mobilized at the time of failure. Olson et al. (2006) collected a large database of published isotropically- and anisotropically-consolidated undrained triaxial compression tests, as well as direct simple shear and rotational shear tests with a monotonically-applied drained shear stress prior to undrained loading. Based on these laboratory data, Olson et al. (2006) proposed a family of yield strength ratio correlations related to the static shear stress ratio, $\tau_{\text{static}}/\sigma'_{\text{vo}}$. Recently, Olson and Zitny (2012) performed a suite of ring shear tests with a monotonically-applied drained shear stress prior to constant volume loading. Combining these data with those from Olson et al (2006), Olson and Zitny (2012) proposed the following relationship among $(N_1)_{60}$, $s_u(\text{yield})/\sigma'_{\text{vo}}$, and $\tau_{\text{static}}/\sigma'_{\text{vo}}$ (as well as a similar relationship based on cone penetration test tip resistance).

$$(2) \quad \frac{s_u(\text{yield})}{\sigma'_{vo}} = \frac{\tau_{\text{static}}}{\sigma'_{vo}} + \left(\frac{0.002}{\frac{\tau_{\text{static}}}{\sigma'_{vo}}} \right) (N_1)_{60}$$

Terzaghi et al. (1996) and Mesri (2007) approximated the level-ground liquefaction resistance relationship [defined as $s_u(\text{yield})/\sigma'_{vo}$ for level ground] proposed by Seed et al. (1985) as a linear function of SPT blow count up to $(N_1)_{60} = 20$. Incorporating the K_α correction for sloping ground proposed by Rollins and Seed (1990), Terzaghi et al. (1996)/Mesri (2007) proposed the following relationship among $(N_1)_{60}$, $s_u(\text{yield})/\sigma'_{vo}$, and $\tau_{\text{static}}/\sigma'_{vo}$.

$$(3) \quad \frac{s_u(\text{yield})}{\sigma'_{vo}} = \frac{\tau_{\text{static}}}{\sigma'_{vo}} + (0.011)(1 - 2 \frac{\tau_{\text{static}}}{\sigma'_{vo}})(N_1)_{60}$$

SLOPE FAILURE TRIGGERED BY 2010 HAITI EARTHQUAKE

Case 1. Slope failure near Dechapelle

Case 1 involved the failure of a 7-m high riverbank. Figure 1 presents the pre-failure geometry, approximate phreatic surface, and dynamic cone penetration test resistance, $(N_1)_{60,DCPT}$ profiles, reproduced from Olson et al. (2010). The soil consists chiefly of a fairly homogeneous brown and gray, fine- to medium- grained sand with trace non-plastic silt ($D_{50} \sim 0.3$ mm), although zones of stiff fine-grained soil were encountered above the watertable. As discussed by Olson et al. (2010), below the watertable, $(N_1)_{60,DCPT}$ in the sand are on the order of 7 to 15, suggesting that this layer liquefied during the earthquake.

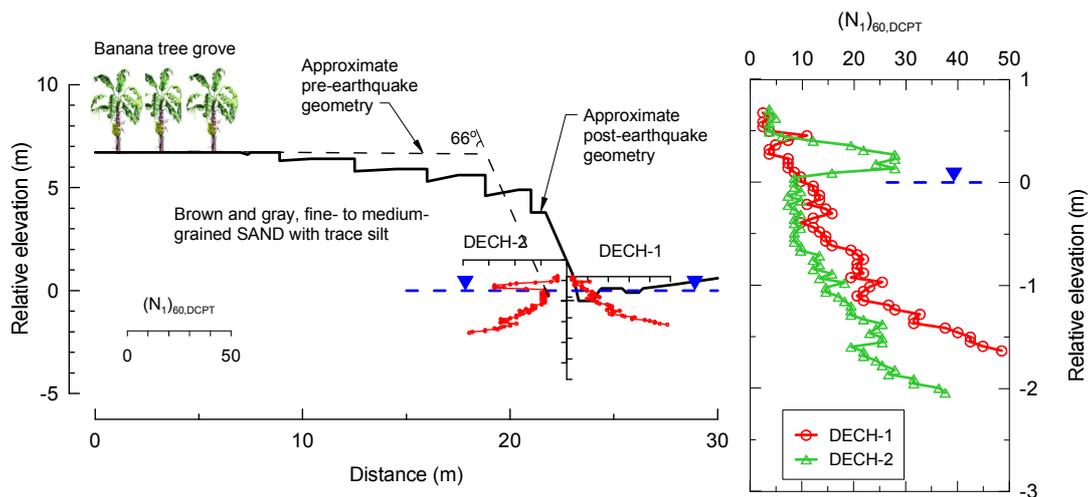


Figure 1 - Profile of slope failure at North River near Dechapelle (Case 1) and $(N_1)_{60}$ profile (from Olson et al. 2010)

COMPARISON OF SLOPING GROUND TRIGGERING ANALYSES

The authors performed liquefaction triggering analyses for the the slope failure described above. The approaches are as follows.

Method 1. Level-ground cyclic stress method with K_α and K_σ corrections using a probability of liquefaction of 20% (Seed et al. 2003; Cetin et al. 2004; Harder and Boulanger 1997).

Method 2. Level-ground cyclic stress method with K_α and K_σ corrections (Idriss and Boulanger 2008).

Method 3. Yield strength ratio method (Olson and Stark 2003; Olson et al. 2006; Olson and Zitny 2012).

Method 4. Yield strength ratio method (Terzaghi et al. 1996; Mesri 2007).

Triggering analyses were performed for the slip surfaces shown in Figure 2 for Case 1. Strengths associated with non-liquefiable soils were selected as the fully mobilized drained or undrained estimated from SPT-based empirical correlations. Table 1 summarizes the parameters for the triggering analyses.

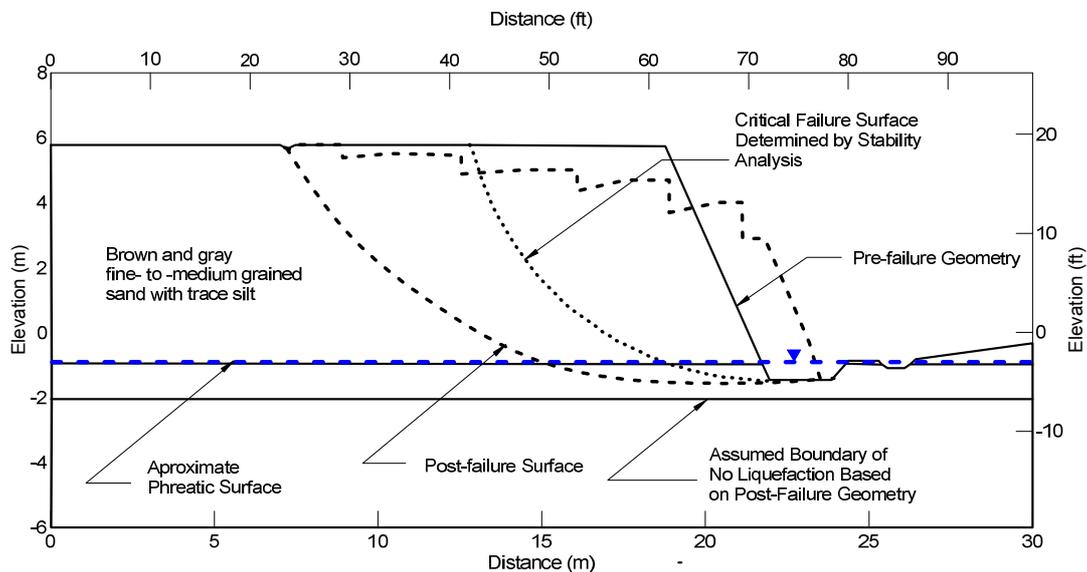


Figure 2 - Failure surfaces analyzed for Case 1.

Table 1. Average parameters for analysis of slope failures (seismological parameters from Olson et al. 2010).



Case	$(N_1)_{60}$	FC (%)	D_{50} (mm)	$\tau_{static}/\sigma'_{vo}$	σ'_{vo} (kPa)	M_w	pga (g)
1	10	< 5	0.3	0.38	56	7	0.17

For Case 1, Methods (1), (3), and (4) predict that liquefaction would be triggered along the entire sliding surface (Fig. 3), while Method (2) predicts that liquefaction is not triggered, but the factors of safety against liquefaction are not much above unity.

Table 2. Factors of safety against liquefaction (FS_{liq}) using average input parameters from Table 1.

Case	FS_{liq}			
	(1)	(2)	(3)	(4)
1	0.90 – 1.00	1.08 – 1.21	0.83 – 0.93	0.77 – 0.86

COMPARISON OF POST-TRIGGERING STABILITY ANALYSES

The authors performed post-triggering analyses for the slope failure described above as each case had several segments of the critical slip surface with $FS_{liq} < 1$. The approaches are as follows.

Method 1. Liquefied segments were assigned liquefied shear strengths (residual undrained shear strength) from Seed and Harder (1990). The lowerbound, 33rd percentile, and average of the Seed and Harder (1990) were considered. Non-liquefied segments within the estimated liquefiable layer were assigned a fraction of the drained strength (Seed and Harder 1990). For $FS_{liq} \leq 1.1$, the soil was assigned the residual strength. For $FS_{liq} \geq 1.4$, the soil was assigned 75% of the drained shear strength. For $1.1 \leq FS_{liq} \leq 1.4$, the soil strength was interpolated between the residual strength and 75% of the drained strength.

Method 2. Liquefied segments were assigned liquefied shear strengths based on the liquefied shear strength ratio correlations from Idriss and Boulanger (2008). The correlations considering some void redistribution (upperbound) and no void redistribution (lowerbound) were considered. No guidance was provided for soils in the liquefied zone that were not predicted to liquefy. Therefore, the authors employed the approach in Method (1) for these segments.

Method 3. Liquefied segments were assigned liquefied shear strengths based on the average liquefied shear strength ratio correlations from Olson and Stark (2002). Nonliquefied segments within the liquefiable zone were assigned their yield shear strength ratios (Olson and Stark 2003).

Method 4. Liquefied segments were assigned liquefied shear strengths based on the liquefied shear strength ratio correlations from Mesri (2007). Nonliquefied segments within the liquefiable zone were assigned their mobilized yield shear strength ratios (Mesri 2007).



For Case 1, Methods (3) and (4) yielded similar factors of safety against slope stability (FS_{Flow}) of approximately 0.885 and 0.88, respectively. Method (1) yielded a wider range of FS_{Flow} , with $FS_{Flow} = 0.85, 0.92,$ and 0.96 for the lowerbound, 33rd percentile, and average residual strength correlations, respectively. Only Method (2) predicts FS_{Flow} greater than unity, with $FS_{Flow} = 1.03$ and 1.17 for the lowerbound and upperbound correlations, respectively.

CONCLUSIONS AND RECOMMENDATIONS

This study tested four methods to evaluate triggering of liquefaction and post triggering stability using one flow failures where the seismic/dynamic demand induced liquefaction of the sandy soils below the water table. From the analysis, it is verified that Olson and Zitny (2012), Mesri (2007) and Seed et al. (2003), successfully predict liquefaction triggering in each case, being Mesri (2007) more conservative than Olson and Zitny (2012) and Seed et al. (2003). Idriss and Boulanger (2008) appeared to be the least conservative method as it does not predict the triggering of liquefaction for all variations between potential failure surfaces and penetration resistance in both cases.

Based on post-triggering stability analyses, Olson and Stark (2002) and Mesri (2007) successfully predicts the flow failure in each case, being Mesri (2007) the less conservative since it includes the effect of the initial static shear stress in its liquefied correlation. Seed and Harder (1990) does not predict the flow failure for both cases and turned out to give the most unconservative result for the flow failure case associated with the lowest prefailure vertical stress (23kPa) by a factor of two, considering its lower bound recommendation. Idriss and Boulanger (2008) does not predict the flow failure for each analyzed case. However, it seems to predict reasonably well the flow failure for the case with the highest prefailure vertical stress (56kPa). On the other hand, Idriss and Boulanger (2008) Sr/σ'_{vc} lower and upper bound for considering the significance of void redistribution, was found to be critical in the estimation of the flow failure factor of safety for $(N_1)_{60} \geq 14$, as its flow failure prediction gives values of factors of safety less and more than one, depending if the Sr/σ'_{vc} lower or upper bound is used, respectively.

From the study, it is observed that:

- (1) Olson and Zitny (2012), Mesri (2007) and Seed et al. (2003) approaches provide better estimates to evaluate the triggering of liquefaction.
- (2) Olson and Stark (2002) and Mesri (2007) approaches provide better estimates to evaluate the post-triggering stability. Seed and Harder (1990) should not be used for projects involving relatively low confining stress (i.e. $\sigma'_o < 50\text{kPa}$), and its use for higher confining stresses should be limited to its lower bound recommendation. Analogously, Idriss and Boulanger (2008) residual shear strengths are not conservative for moderate confining stress (i.e. $\sigma'_o > 50\text{kPa}$) and its used for lower confining stresses should be limited to its lower bound.



Table 3. Factors of safety against flow failure

Case	Failure Surface	Method																				
		Seed & Harder (1990)									Olson & Stark (2002)			Mesri (2007)			Idriss & Boulanger (2008)					
		Lower Boundary			1/3			Average									Lower Boundary			Upper Boundary		
		(N ₁) ₆₀			(N ₁) ₆₀			(N ₁) ₆₀									(N ₁) ₆₀			(N ₁) ₆₀		
7	10	13	7	10	13	7	10	13	7	10	13	7	10	13	7	10	13	7	10	13		
1	Circular	0.81	0.85	0.94	0.86	0.92	1.02	0.89	0.96	1.07	0.84	0.85	0.86	0.87	0.88	0.89	0.84	1.03	1.17	-	1.17	1.18



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