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# EFFECT OF THE INITIAL STATIC SHEAR STRESS ON THE CYCLIC RESISTANCE OF SANDS

**Gonzalo CORRAL [1], Ramón VERDUGO [2]**

## ABSTRACT

In this paper, the effect of the initial static shear stress on the cyclic resistance of a tailings sand is experimentally studied. The correction factor, $K_\alpha$, is examined. The tested tailings material is a cycloned sand containing 18% non-plastic fines. The experimental program consisted of a series of undrained cyclic triaxial tests conducted under an effective confining pressure of 1 ksc (98.1 kPa). Two different relative densities, 45% (loose case) and 75% (dense case), were studied. The magnitudes of induced initial static shear stress (i.e. before undrained cyclic load application) selected were 0.0, 0.1, 0.2, 0.4, 0.6, -0.2 and -0.4 ksc. This experimental study provides new evidence demonstrating that the initial static shear stress affects the cyclic resistance, especially in the case of looser soils. Particularly, it was observed that the correction factor, $K_\alpha$, reaches maximum values of 1.8 and 1.3, for the loose and dense case, respectively, when initial compression loading conditions are imposed. In contrast, for extension loading conditions, $K_\alpha$ becomes consistently lower than 1.0. Although these observations differ from many recently published data, they are in accordance with experimental results reported by Japanese authors.

Keywords: initial static shear stress; cyclic resistance; liquefaction; correction factor.

## INTRODUCTION

Most of the experimental studies into the cyclic resistance of sandy soils have been conducted using isotropically consolidated cyclic triaxial tests. In nature, however, an initial static shear stress usually exists before the application of an undrained cyclic load. Experimentally, it has been observed that the anisotropic consolidation affects the cyclic and liquefaction resistance, even though reported results are not definitive and sometimes are contradictory. In practice, the effect of the initial static shear stress is usually incorporated through the correction factor, $K_\alpha$, which modifies the cyclic resistance obtained from tests under isotropic consolidation.

To account for the magnitude of the initial shear stress, the parameter $\alpha$ has been defined as the ratio between the absolute value of the initial static shear stress, $|\tau_{st}|$, applied in the horizontal plane, and the effective vertical stress, $\sigma'_{vo}$ (Equation 1). Similarly, for the cyclic triaxial test, this ratio can be rewritten by evaluating the shear stress and the effective normal stress in a 45° plane (Equation 6). Generally, soil elements in slopes, embankments, dams, and close to buildings present static shear stress in the horizontal plane which can significantly affect the cyclic resistance. Additionally, the factor, $K_\alpha$, is defined as the ratio between the cyclic resistance ratio for a given value of $\alpha$ ($CRR_\alpha$), and the cyclic resistance ratio with non applied initial static shear stress ($CRR_{\alpha=0}$), Equation (2).

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$$\alpha = \frac{|\tau_{st}|}{\sigma_{vo}'} \tag{1}$$

$$K_\alpha = \frac{CRR_{\alpha \geq 0}}{CRR_{\alpha = 0}} \tag{2}$$

Seed (1983) proposed the correction factor $K_\alpha$ in order to correct the cyclic resistance of sands, for different values of the initial shear, or α. This correction factor suggested that the presence of initial static shear stress will always increase the cyclic resistance. Seven years later, the same correction factor was modified by Rollins & H. Seed (1990) and by R. Seed & Harder (1990), specifying different correlations for different relative densities.

The effect of the initial static shear stress on the cyclic resistance of sands has been experimentally studied by several researchers. For instance, for dense sands, Lee and Seed (1967), Vaid and Finn (1979), Vaid and Chern (1983), and Szerdy (1986) found that the cyclic resistance ratio (CRR) increases when α increases, implying a correction factor $K_\alpha$ higher than 1.0. However, Vaid and Chern (1985) concluded the opposite when the mean effective stress was equal to 16ksc. In contrast, for loose sands, Yoshimi and Oh-Oka (1975), Vaid and Chern (1983), and Szerdy (1986) concluded that by increasing α, the CRR decreases; although, Lee and Seed (1967) found that the CRR could increase when low effective mean stresses are present. Additionally, Boulanger at al. (1991) gathered data with those reported by Vaid and Finn (1979) (using cyclic simple shear tests) showing consistent results among them: after increasing α values from 0.0 to 0.3, $K_\alpha$ decreases (i.e. lower than 1.0) for loose to medium relative densities and increases for medium to dense states.

Nevertheless, Pillai (1991) stated that these correlations were just empirical, pointing out that all previous results appear to be conflictive and erroneously interpreted. He justifies this statement because researchers had based their studies only on relative densities and so they had failed to identify fundamental phenomena and parameters that govern these effects.

More recently, Yoshimine and Hosono (2001) and Hosono and Yoshimine (2004) showed that the initial state, either in compression or in extension, affects the cyclic resistance of sands in different ways. They concluded that the correction factor $K_\alpha$ was more important for loose than dense states.

In order to provide new information regarding the effect of the initial shear stress on the liquefaction resistance of sandy soils, a series of cyclic triaxial tests anisotropically consolidated were carried out. The results, together with the corresponding analysis, are presenting herein.

## MATERIAL TESTED AND SPECIMEN PREPARATION

The material tested corresponds to a tailings sand with 18% non-plastic fines. This material is derived from the wall Torito Dam; located at the "El Soldado" mine, Calera, V Region, Chile. Tables 1a and 1b present the parameters associated with the grain size and the results of specific gravity and maximum and minimum densities, respectively. The grain size distribution curve of the tested material is given in Fig. 1. This material is classified as a silty sand (SM) using the Unified Soil Classification System (USCS).

**Table 1. Torito Dam Tailings Sand: (a) Gradation Properties, and (b) Specific Gravity and Maximum and Minimum Densities**

(a)

| $d_{10}$ (mm) | $d_{30}$ (mm) | $d_{60}$ (mm) | $C_u$ | $C_c$ |
|---|---|---|---|---|
| 0.046 | 0.108 | 0.174 | 3.78 | 1.46 |

(b)

| $G_s$ | $\gamma_{dmax}$ (gr/cm$^3$) | $\gamma_{dmin}$ (gr/cm$^3$) | $e_{max}$ | $e_{min}$ |
|---|---|---|---|---|
| 2.75 | 1.773 | 1.243 | 1.212 | 0.551 |

The wet tamping method of sample preparation was adopted. For each sample, five equal pre-weighed-oven-dried portions of sand were mixed with water at 5% of water content. All specimens were cylindrical in shape, with 10 cm in high and 5 cm in diameter. To reduce radial friction between the caps and soil samples, both end caps (i.e. top and bottom) of the triaxial cell apparatus were lubricated and prepared using silicone lubricant and rubber sheets (Verdugo, 1992). Alternatively, the void ratio of the samples was evaluated at the end of the tests through the measurement of the water content of the sample (Verdugo, 1992). A back pressure of 1 ksc was applied which was sufficient to achieve a B-value greater than 0.95 and to assume that the sample was fully saturated.

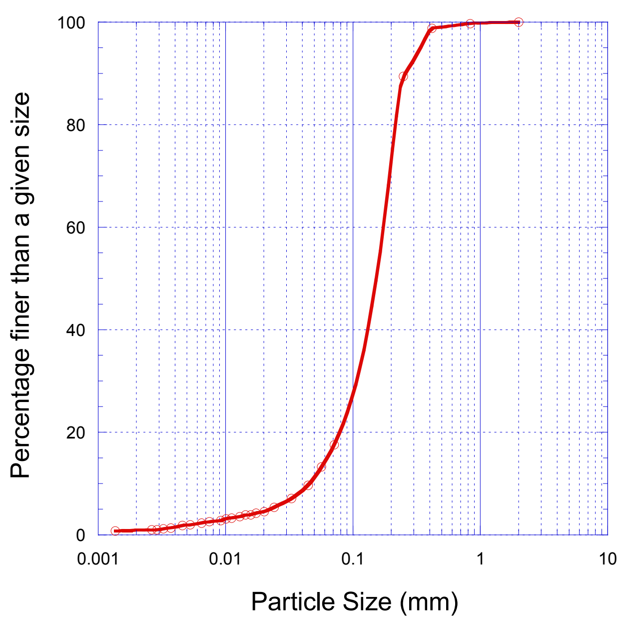


**Figure 1. Granulometric Analysis for Tailings Sand of the Wall Torito Dam**

## TEST RESULTS

### Undrained Static Triaxial Tests

A series of five static undrained triaxial tests (CIU) in compression were conducted. Samples were initially compacted at relative densities equal to 20, 30, 40, 50, and 70%. Every test was isotropically consolidated up to an effective mean pressure of 3 ksc. Figure 2 gives the results including the critical state line, assuming that the critical state condition is reached at 20% of axial strain. The stress-strain-strength relations show fully consistency and agreement with the range of relative densities tested as well as with the critical state soil mechanics theory.

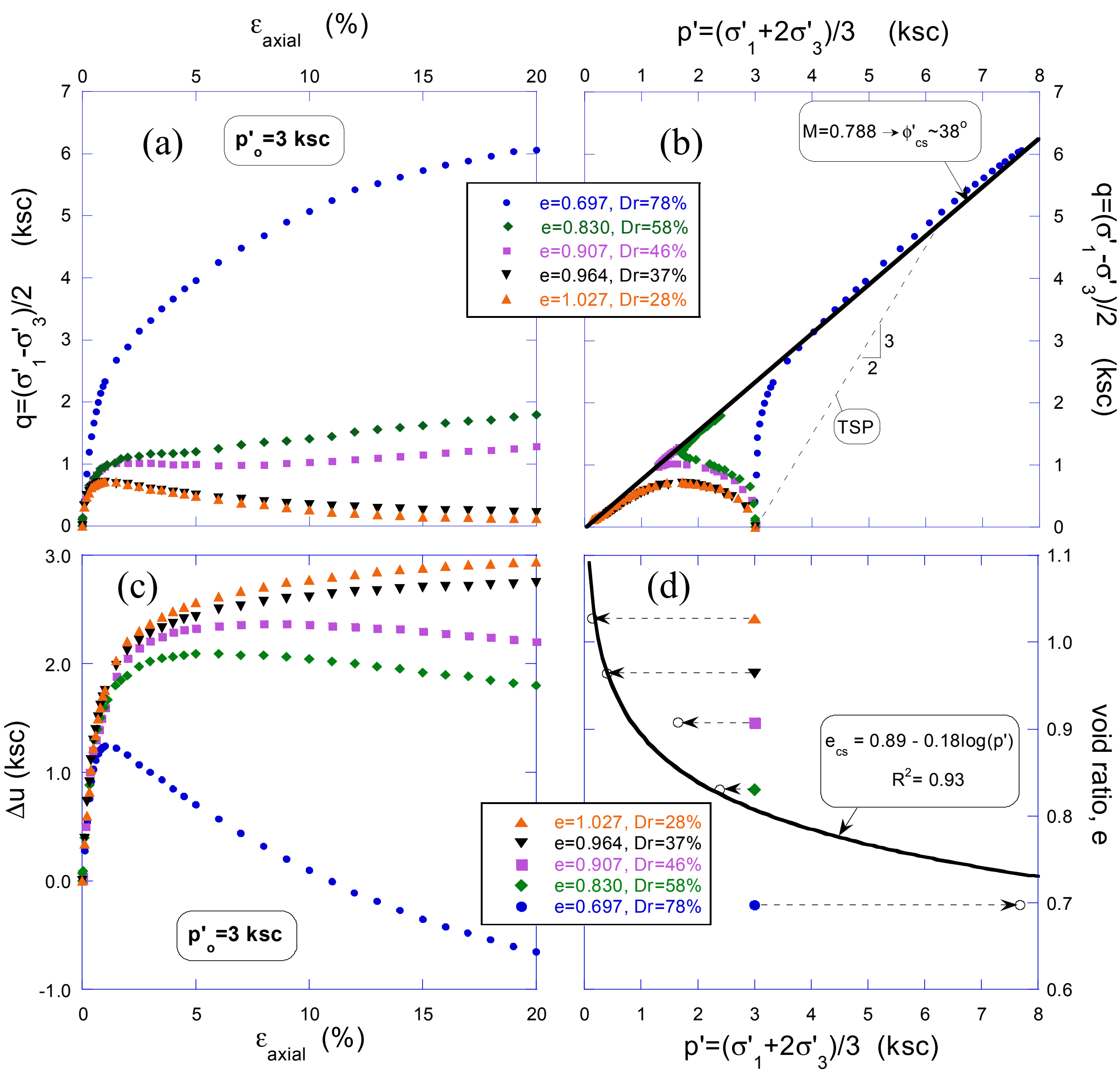


**Figure 2. Undrained Triaxial Tests in Compression: (a) Stress-Strain Curves, (b) Effective Stress Paths, (c) Pore Pressure Change and (d) Void Ratio vs. Mean Effective Stress**

**Undrained Cyclic Triaxial Tests**

To investigate the correction factor, $K_\alpha$, a series of 34 undrained cyclic triaxial tests were conducted, considering loose and dense cases. In each test, uniform cycles with a rate of 3 cycles per minute were applied. The direction of the first cycle was always applied to the compression side. 18 and 16 cyclic triaxial tests were performed with a relative density of 45% and 75%, respectively (obtained after consolidation). The samples were carefully consolidated both isotropically and anisotropically. For the anisotropic consolidation both compression and extension loading conditions were considered: following firstly an isotropic path up to 1ksc, and then a constant mean effective stress path. This consolidation path permitted to decouple the static shear stress with the mean effective pressure application.

Figure 3 shows the CRR versus the number of cycles N needed to achieve a 5% of double amplitude of axial strain, corresponding to the liquefaction criterion adopted for this study. The impact of the relative density and the initial shear stress magnitude can be readily distinguished. For both relative densities, the solid squares shown in Figure 3 represent the CRR under isotropic consolidation (i.e. $\tau_{st}$=0). From this figure, it can additionally be inferred that the use of the CRR under isotropic consolidations can be conservative in most initial loading conditions in compression, while its use would be highly risky in extension case.

A plot of CRR versus Kc or $\alpha$ (Figure 4) is a practical way of visualizing between reverse and non-reverse cyclic loading. In fact, through basic algebra it is possible to obtain the reverse frontier for initial compression and extension states:

$$K_c = \frac{\sigma'_{1c}}{\sigma'_{3c}} \geq 1 \tag{3}$$

$$\tau_{st/comp} = q_{st/comp} = \frac{\sigma'_{1c} - \sigma'_{3c}}{2} = \frac{K_c - 1}{2}\sigma'_{3c} \geq 0 \tag{4}$$

$$\tau_{st/ext} = q_{st/ext} = -\frac{\sigma'_{1c} - \sigma'_{3c}}{2} = \frac{-K_c + 1}{2}\sigma'_{3c} \leq 0 \tag{5}$$

$$\sigma'_{no} = \frac{\sigma'_{1c} + \sigma'_{3c}}{2} = \frac{K_c + 1}{2}\sigma'_{3c} \tag{6}$$

Re-writing Equation (1) for the triaxial test case, we obtain:

$$\alpha = \frac{|q_{st}|}{\sigma_{no}'} \geq 0 \tag{7}$$

Combining Equations (4), (5) and (6) into (7), $\alpha$ and $K_c$ can be mutually correlated as follows:

$$\alpha = \frac{K_c - 1}{K_c + 1} \tag{8}$$

$$K_c = \frac{\alpha + 1}{1 - \alpha} \tag{9}$$

Defining the CRR as follows:

$$CRR = \frac{|\tau_{cyc}|}{p'_o} \tag{10}$$

In addition, the initial mean effective stress for compression and extension can be expressed in terms of major and minor principal stresses at the end of each anisotropic consolidation side:

$$p'_{0/Comp} = \frac{1}{3}(\sigma'_{1c} + 2\sigma'_{3c}) \tag{11}$$

$$p'_{0/Ext} = \frac{1}{3}(2\sigma'_{1c} + \sigma'_{3c}) \tag{12}$$

At the reverse cyclic load limit, the following equivalence is achieved:

$$|\tau_{cyc}| = |q_{st}| \tag{13}$$

Thus, using Equations (10), (11), (12) and (13) we have that:

$$CRR_{Comp.Limit} = \frac{3}{2} \cdot \frac{(K_c - 1)}{(K_c + 2)} = \frac{3\alpha}{3 - \alpha} \tag{14}$$

$$CRR_{Ext.Limit} = \frac{3}{2} \cdot \frac{(K_c - 1)}{(2K_c + 1)} = \frac{3\alpha}{3 + \alpha} \tag{15}$$

These relations (Equations 14 and 15) between the CRR and $K_c$ or $\alpha$ represent the frontier for reverse (two-ways) and non-reverse (one-way) cyclic loads, which are shown in Figure 4a and 4b. It can be noted that each limit frontier (i.e. compression and extension case) has to be observed separately. All cyclic loads are reverse for the dense case (Dr=75%). However, for the loose case (Dr=45%) and in compression, there are two occasions of non-reverse cyclic loads (happening for $q_{st}$ values of 0.4 and 0.6 ksc), and, in extension, only one case corresponds to a non-reverse load (occurring when $q_{st}$ is equal to -0.2ksc).

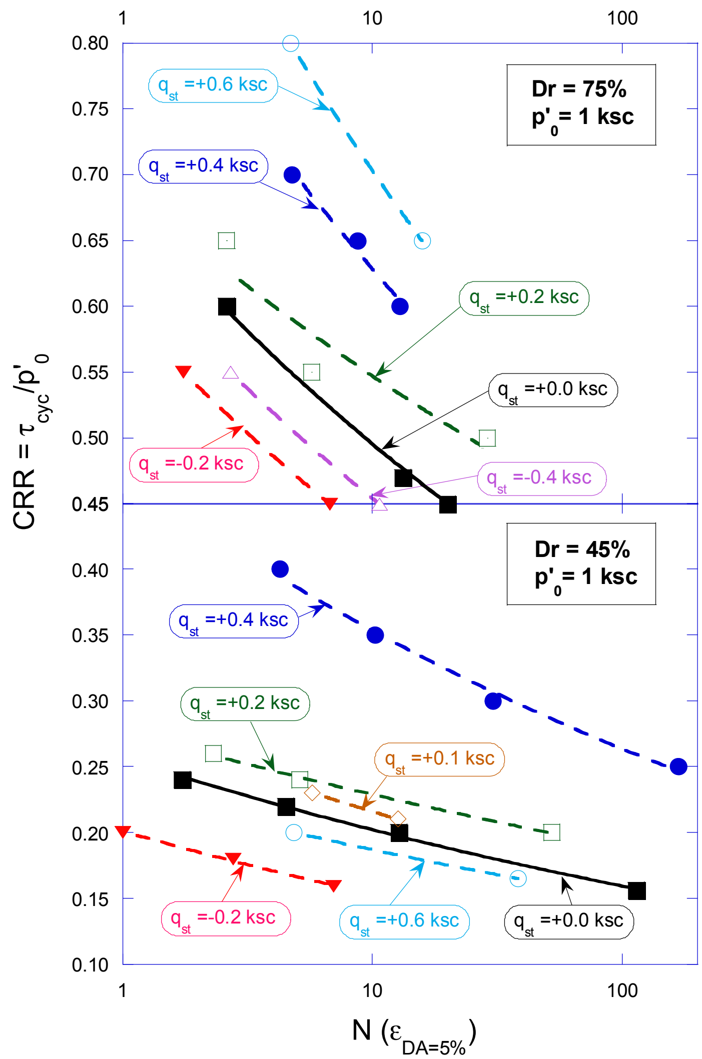


**Figure 3. Cyclic Resistance Ratio vs. Number of Cycles to reach 5% of Double Amplitude of $\varepsilon_{axial}$**

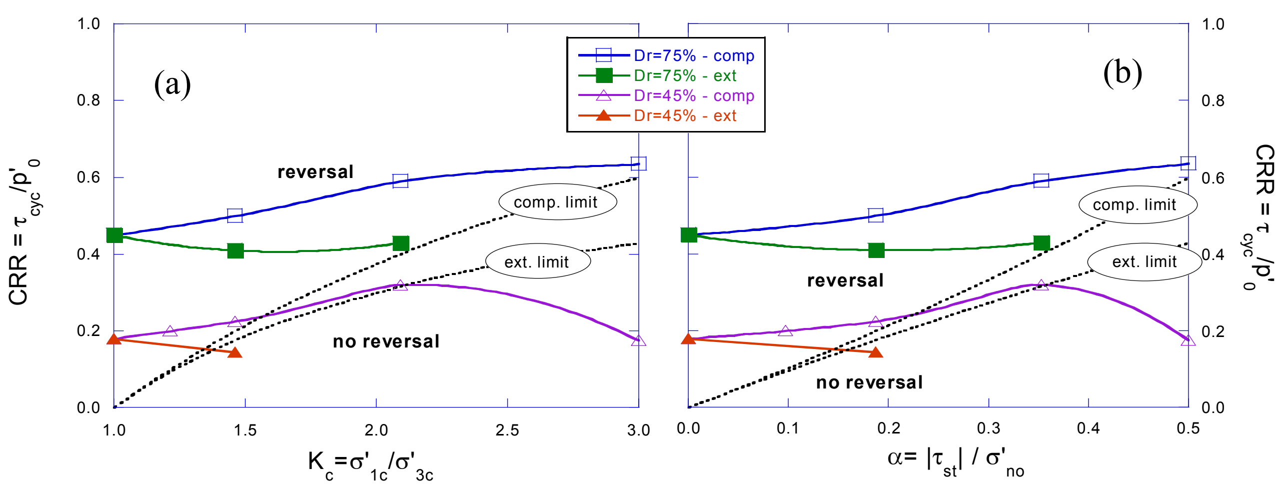


**Figure 4. Cyclic Resistance Ratios for N=20, showing Reverse Load Limits, vs. (a) $K_c$ and (b) $\alpha$**

Figure 5 shows the values of $K_\alpha$ obtained for the loose and dense cases, as well as for initial shear in compression and extension. For the loose case, it is possible to highlight the following: 1) in compression, $K_\alpha$ significantly increases up to $\alpha = 0.35$ ($q_{st}$=0.4ksc), and then goes down; 2) in extension, $K_\alpha$ notoriously decreases when $\alpha$ is equal to 0.18 ($q_{st}$= -0.2ksc), being the most adverse case. For the dense condition, the following can also be emphasized: 1) in compression, $K_\alpha$ is always increasing regardless the values of $\alpha$; and 2) in extension, $K_\alpha$ roughly decreases up to about $\alpha = 0.18$, and then increases for $\alpha$ = 0.35, but without exceeding the value of 1.0.

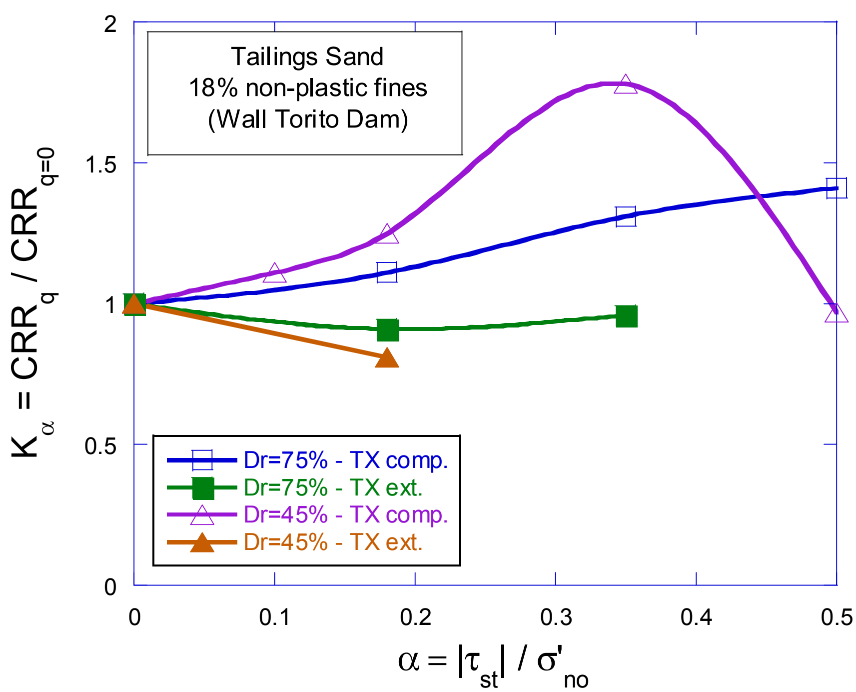


**Figure 5. $K_\alpha$ vs. $\alpha$ for Loose and Dense Cases for N=20 and when 5% of D.A. of $\varepsilon_{axial}$ is reached.**

These results are compared with data available in the literature: Rollins and H. Seed (1990) and R. Seed and Harder (1990), as shown in Figure 6a and 6b. Additionally, the results obtained in this investigation are also compared with more recent data reported by Hosono and Yoshimine (2004), Figure 6c. From these figures, it can be noted that both these current results and those reported by Hosono and Yoshimine (2004) imply that the initial shear stress on the cyclic resistance of sandy soils is much more relevant on looser than on denser states. However, the opposite can be observed using reported data by Rollins and Seed (1990) and Seed and Harder (1990).

It should be mentioned that data from Rollins and H. Seed (1990), probably the most used in geotechnical practice nowadays, correspond to suggested representative relationships (shown in Figure 6a) - data gathered from several authors. These average relationships came not only from different relative density ranges but also from different test apparatuses (e.g. triaxial, simple shear, and ring torsion tests) and different assumed liquefaction definitions. R. Seed and Harder (1990) obtained more data, and therefore they added those to the ones presented by Rollins and H. Seed (1990) (Figure 6b). It is very important to highlight that all triaxial test data used for estimating these suggested representative relationships of $K_\alpha$

versus α came only from compression triaxial initial states. Hosono and Yoshimine (2004) showed data from compression and extension triaxial initial states (Figure 6c). Additionally, they included some results from cyclic simple shear tests (not shown in this study) obtaining important differences among them.

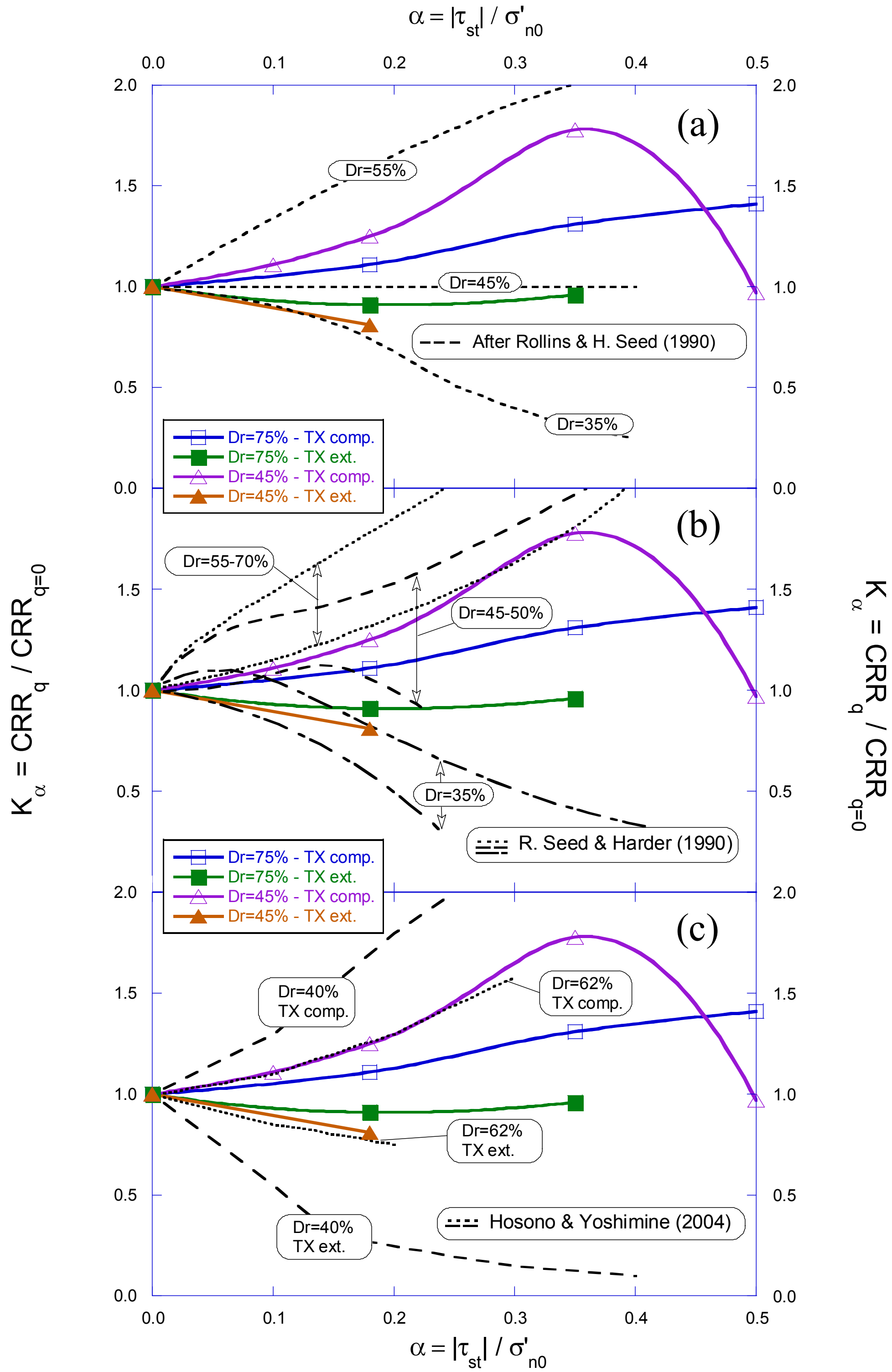


**Figure 6: Current Results Compared with (a) Rolling and H. Seed, 1990; (b) R. Seed and Harder, 1990; (c) Hosono and Yoshimine, 2004.**

## CONCLUSIONS

Based on the results of a series of undrained cyclic triaxial shear tests carried out on compacted loose and dense specimens of a tailings sand, the following conclusions are drawn:

- The presence of the initial static shear stress on the cyclic resistance is much more important for loose soils than it is for dense soils. This conclusion differs from the majority of reported studies, but agrees with that given by Hosono and Yoshimine (2004).
- From practical point of view, the value of $\alpha$ is not greater than 0.35. Based on that, for the compression case, the magnitude of $K_\alpha$ is higher than 1.0, reaching values of 1.8 and 1.3 for loose soils and dense soils, respectively.
- When initial shear stress is applied in extension it is possible to conclude that there is a negative effect on the cyclic resistance, so $K_\alpha$ becomes lower than 1.0.
- For initial shear stress in compression, the obtained results indicate that the cyclic resistance gradually increases as $\alpha$ increases. This trend is similar for both loose and dense soils.
- For the loose soil condition, for $\alpha$ = 0.18, 0.35 and 0.50, the mode of failure and the development of deformations are cumulative in the compression side. However, for the dense soil condition, all failure modes were with a cyclic deformation.
- For initial shear in extension, the development of deformation was only accumulative in extension.
- The use of the cyclic resistance ratio under isotropic consolidation is conservative. Nonetheless, for initial extension states, this situation is completely contrary.

Further research is needed to investigate different aspects that may affect the cyclic resistance correction factor. Some of them may include a possible coupling between $K_\alpha$ and $K_\sigma$ (correction factor by confining pressure level), different sandy soils, and the effect of decoupling different types of failures obtained mainly from cyclic triaxial shear tests.

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