The Geotechnical Properties of Sands with Varying Grading in a Stress-Controlled Ring Shear Tests

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ABSTRACT

A series of tests undertaken to investigate the undrained shear behavior of sands with different grading is presented. The tests were conducted by means of a new ring shear apparatus at different relative densities and normal stresses. Silica sands constituted to various uniformity coefficients were classified as well graded (WG), intermediately graded (ING), narrowly graded (NAG) and gap graded (GAG). Results of the investigation show that in medium and dense states the values of their peak strengths are ranked as WG> ING> NAG> GAG**.** However, their steady state strengths are in the order of NAG> ING>WG>GAG. From the viewpoint of public safety, the narrowly graded sample in medium and dense states has the lowest potential for large travel distances following a slope failure. In loose state however, the narrowly graded specimens readily undergo complete liquefaction, described in this paper as the reduction of steady strength to zero after failure. Although the steady state strengths of WG, ING graded and NAG specimens in loose state are very low, only the steady strengths of NAG and ING specimens have steady state strengths of zero. These results show that while higher uniformity coefficient may equate to or translate into higher peak and higher steady-state strength in soils at a given set of conditions, it might not when the set of conditions are altered; and that under certain circumstances, engineers might want to choose poorly graded soils over better graded ones.

KEYWORDS: Complete liquefaction, grading, peak strength, steady-state strength, ring shear tests.

INTRODUCTION

The subject of liquefaction and the shear behavior of cohesionless soils have been discussed by a great deal of researchers, including Casagrande (1936, 1976), Terzaghi and Peck (1948), Seed (1966, 1979, 1981), Castro (1969, 1975), Seed and Idriss (1971), Whiteman (1971), Ambraseys (1973), Gilbert (1976), Poulos (1981), Kutter (1982), Poulos *et al.* (1985), Eckersley (1985), It has been variously mentioned that the void ratio or relative density of the soil, the confining stress on the soil, the intensity and duration of ground shaking, are important factors determining the liquefaction susceptibility of a saturated soil. While highlighting the importance of such factors as confining stress, initial shear stress, particle angularity, grain structure or fabric, over-consolidation ratio, previous strain history, intensity and duration of ground shaking in predicting the liquefaction susceptibility of a soil mass, researchers have also provided evidences that indicate liquefaction is associated, primarily, with loosely deposited, poorly graded sands and silts (Seed 1979; Castro and Poulos 1977; Seed 1981; Sassa 1985; Vaid and Chern 1985; Vaid *et al.* 1990; Kramer and Seed 1988; Sassa and Wang 2003; Yoshimine *et al.* 1999), Gilbert and Marcuson (1988), Ishihara *et al.* (1990), Ishihara (1993), Sassa (2000) Sassa *et al.* (1996 and 2003). It is widely accepted that in loose state, poorly graded specimens are weaker than better graded specimens. But does such a finding remain true at all densities, including medium-dense and dense states; does better grading translate into higher shearing strength at all densities? This is one of the primary issues our paper sets out to investigate. This paper wants to find out whether or not a material slides or flow is closely tied to its nature, and to what extent grain size distribution influences the shear behavior of sandy soils in loose, medium and dense states. Attempts will be made to scrutinize the influence of grading on the peak and steady state strengths of soils categorized as narrowly graded, intermediately graded, well graded and gap graded. The concept of gap-graded soils has not been properly understood yet. Good knowledge of the underlying mechanism of failure in gap- graded soils may yield strong insight on liquefaction of soils. Researchers including Hutchinson and Townsend (1961), Kirkpatrick (1965), Koerner (1969), Lee and Fitton (1969), Ross *et al.* (1969), Kishida (1969), Wong *et al.* (1975), and Vaid *et al.* (1990), Kokusho *et al.* (2004) have conducted tests using soils of varying grading, but they have done so using some apparatuses which do not permit soils to be displaced for long distances. It stands to reason that if soils do not undergo sufficient displacement, making conclusions about their steady state strengths would be, at best, an inconclusive exercise. The ring shear apparatus, among other attributes, permits unlimited displacement of soils; and should be suitable for examining the post-failure behavior of granular materials.

APPARATUS AND MATERIAL

The results presented and discussed in this paper are from a new ring shear apparatus, hereafter referred to as DPRI-5, which is the fifth version of ring shear apparatuses available at Disaster Prevention Research Institute, Kyoto University, Japan (Fig. 1) is reinforced with devices capable of sustaining undrain loading throughout the duration of a test. The apparatus is structured to eliminate some difficulties commonly encountered while studying the mechanism of landslide motion, and sufficiently equipped to allow speed-, and stress- controlled tests; and the measurement of very large shear displacement. The schematic diagram of the ring shear apparatus is shown in Fig. 2. Details of the structure and efficiency of the apparatus have been discussed comprehensively by Sassa et al. (2002), and Wang and Sassa (2001).

Figure 1: Picture of the Ring Shear Apparatus

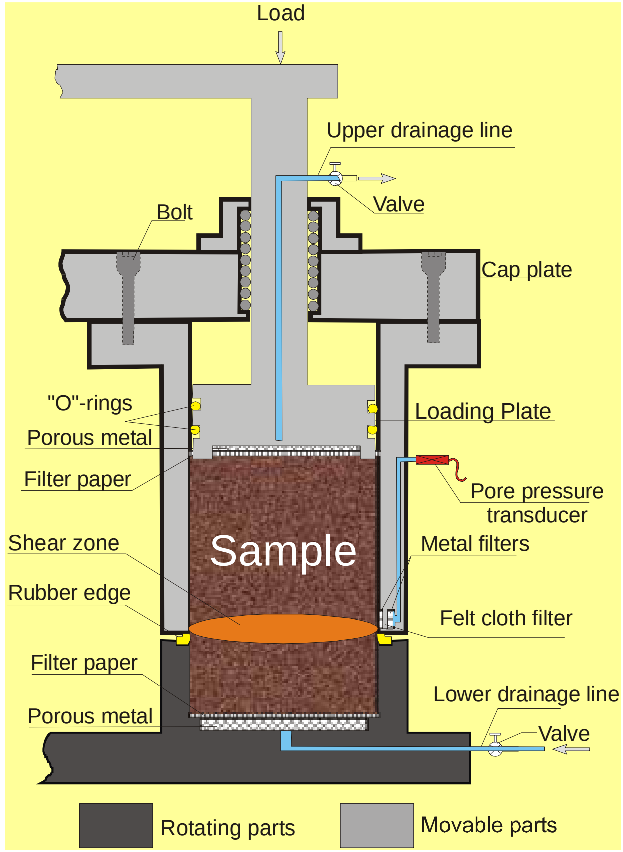

Figure 2: Schematic diagram of the ring shear apparatus

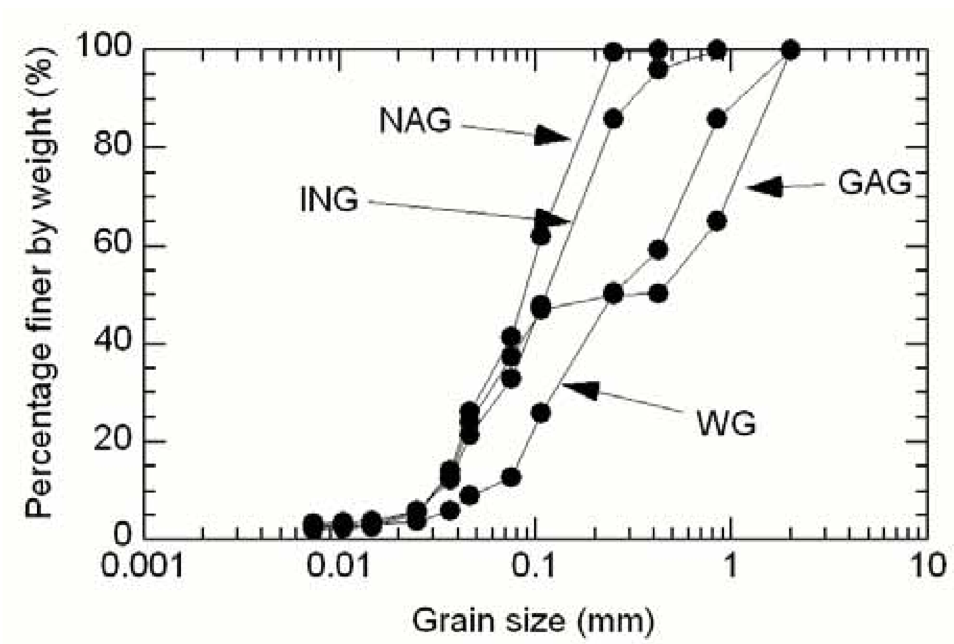

Figure 3: Grain size distribution curves of the specimens

SPECIMEN CHARATERISTICS

Industrial sand materials composed of sub-angular to angular quartz and small amount of feldspar were reconstituted to four uniformity coefficients – 3.3, 4.5, 9.0 and 17.5 – referred to as narrowly graded (NAG), intermediately graded (ING), well graded (WG) and gap graded (GAG) The grain size distribution curves are shown in Fig. 3, The authors are aware of the importance of reconstituting sands with different uniformity coefficients while keeping the mean particle size constant.

SAMPLE PREPARATION AND TESTING PROCEURE

Oven dried specimens having the desired uniformity coefficients were placed in the shear box by moist tamping method, a method chosen to decrease the possibility of particle segregation. Thereafter, test specimens were saturated with water. To achieve a BD value of at least 0.95, which was the minimum acceptable value used in this study, carbon dioxide was first introduced into the samples, at a slow rate, for at least one hour, after which, de-aired water was introduced, again, at a slow rate, to ensure adequate saturation. BD parameter – the ratio of change in pore pressure and change in normal stress ($\Delta u/\Delta\sigma$) over a specified period of time – was the standard parameter used in assessing the degree of saturation of the test samples (Sassa 1988). Obtaining the BD parameter involved a simple process of consolidating the samples at 49 kPa normal stress in drained condition, and increasing the normal stress to 98 kPa in undrained condition when a constant value of vertical displacement signaled the end of the consolidation process which lasts an average of 60 minutes. The resultant increase in pore pressure (from zero to a certain monitored value), Δu, divided by the corresponding increase in normal stress (from 49 kPa to 98 kPa), $\Delta\sigma$, is the BD parameter. Specimens were considered fully saturated if the BD was equal to or greater than 0.95. The objectives of the present research did not permit over-consolidation of specimens. In the light of this, all samples were normally consolidated and thereafter, shearing was performed by incremental loading of shear stress at the rate of 0.98 kPa/sec.

TEST RESULTS

The test results are summarized in Table 1. Two main behaviors were observed. Specimens in loose state responded to undrained shearing in a purely contractive manner while specimens in medium-dense to dense states responded in a dilative way.

Table 1: A summary of results of investigation the sand specimens.

Test No.	Material	Grading	*Dr* (%)	σ_i (kPa)	τ_p (kPa)	τ_{ss} (kPa)	I_B	Behavior	Test condition	B_D value
1	Silica	NAG	29.4	257	70	0	1.0	Purely contractive	Undrained	0.96
2	Silica	NAG	29.5	220	49	0	1.0	Purely contractive	Undrained	0.97
3	Silica	NAG	29.5	305	85	0	1.0	Purely contractive	Undrained	0.96
4	Silica	NAG	32.2	200	142	136		Large volume reduction	Drained	0.97
5	Silica	NAG	33.1	201	49	7	0.85	Purely contractive	Undrained	0.97
6	Silica	NAG	42.1	196	83	35	0.60	Contractive and dilative	Undrained	0.96
7	Silica	NAG	44.1	375	145	40	0.72	Contractive and dilative	Undrained	0.96
8	Silica	NAG	44.1	280	116	40	0.65	Contractive and dilative	Undrained	0.96
9	Silica	NAG	44.2	196	93	40	0.57	Contractive and dilative	Undrained	0.96
10	Silica	Silica	NAG	44.3	360	144	40	Contractive and dilative	Undrained	0.96
11	Silica	NAG	48.2	196	100	45	0.54	Contractive and dilative	Undrained	0.97
12	Silica	NAG	50.5	196	120	48	0.60	Contractive and dilative	Undrained	0.96

13	Silica	NAG	52.3	196	130	56	0.57	Contractive and dilative	Undrained	0.96
14	Silica	NAG	53.3	196	137	52	0.54	Contractive and dilative	Undrained	0.97
15	Silica	NAG	42.5	196	81	36	0.55	Contractive and dilative	Undrained	0.96
16	Silica	NAG	55.2	290	134	40	0.70	Contractive and dilative	Undrained	0.96
17	Silica	NAG	56.2	196	154	70	0.49	Contractive and dilative	Undrained	0.96
18	Silica	NAG	60.0	196	168	65	0.58	Contractive and dilative	Undrained	0.96
19	Silica	NAG	62.1	196	196	84	0.57	Contractive and dilative	Undrained	0.97
20	Silica	NAG	64.5	196	200	84	0.58	Contractive and dilative	Undrained	0.96
21	Silica	NAG	74.1	196	243	100	0.59	Contractive and dilative	Undrained	0.97
22	Silica	ING	29.1	262	67	0	1.0	Purely contractive	Undrained	0.97
23	Silica	ING	29.2	202	54	5	0.91	Purely contractive	Undrained	0.96
24	Silica	ING	31.5	290	97	10	0.90	Purely contractive	Undrained	0.97
25	Silica	ING	44.3	196	81	32	0.60	Contractive and dilative	Undrained	0.96
26	Silica	ING	44.3	280	127	32	0.75	Contractive and dilative	Undrained	0.96
27	Silica	ING	44.1	374	161	32	0.80	Contractive and dilative	Undrained	0.96
28	Silica	ING	46.2	196	89	34	0.62	Contractive and dilative	Undrained	0.96
29	Silica	ING	53.3	196	145	55	0.62	Contractive and dilative	Undrained	0.96
30	Silica	ING	53.5	290	140	42	0.70	Contractive and dilative	Undrained	0.96
31	Silica	ING	56.5	196	165	56	0.66	Contractive and dilative	Undrained	0.96
32	Silica	ING	57.0	196	176	50	0.71	Contractive and dilative	Undrained	0.96
33	Silica	ING	60.2	196	188	62	0.67	Contractive and dilative	Undrained	0.96
34	Silica	ING	62.2	196	199	70	0.64	Contractive and dilative	Undrained	0.97
35	Silica	ING	66.4	196	212	81	0.62	Contractive and dilative	Undrained	0.97
36	Silica	ING	66.5	290	235	70	0.70	Contractive and dilative	Undrained	0.97
37	Silica	ING	68.1	197	235	74	0.68	Contractive and dilative	Undrained	0.97
38	Silica	ING	74.3	196	246	86	0.65	Contractive and dilative	Undrained	0.96
39	Silica	ING	77.2	290	290	81	0.72	Contractive and dilative	Undrained	0.96
40	Silica	WG	29.5	250	80	13	0.84	Purely contractive	Undrained	0.96
41	Silica	WG	29.5	290	106	12	0.88	Purely contractive	Undrained	0.96
42	Silica	WG	29.5	196	60	12	0.80	Purely contractive	Undrained	0.96
43	Silica	WG	30.2	196	146	137		Large volume reduction	Drained	0.96
44	Silica	WG	39.5	105	47	13	0.72	Contractive and dilative	Undrained	0.97
45	Silica	WG	43.2	196	95	25	0.73	Contractive and dilative	Undrained	0.96
46	Silica	WG	44.1	235	122	23	0.81	Contractive and dilative	Undrained	0.96
47	Silica	WG	44.3	203	107	21	0.80	Contractive and dilative	Undrained	0.96
48	Silica	WG	44.3	366	198	23	0.88	Contractive and dilative	Undrained	0.96
49	Silica	WG	44.4	290	173	23	0.86	Contractive and dilative	Undrained	0.96
50	Silica	WG	45.1	196	115	30	0.74	Contractive and dilative	Undrained	0.97
51	Silica	WG	47.1	196	119	30	0.74	Contractive and dilative	Undrained	0.97
52	Silica	WG	48.5	196	128	31	0.75	Contractive and dilative	Undrained	0.96
53	Silica	WG	50.5	196	150	33	0.78	Contractive and dilative	Undrained	0.96
54	Silica	WG	53.2	200	166	35	0.79	Contractive and dilative	Undrained	0.96
55	Silica	WG	55.3	198	198	37	0.81	Contractive and dilative	Undrained	0.96
56	Silica	WG	57.1	196	209	29	0.81	Contractive and dilative	Undrained	0.96
57	Silica	WG	57.3	200	215	24	0.88	Contractive and dilative	Undrained	0.96
58	Silica	WG	64.1	198	230	57	0.75	Contractive and dilative	Undrained	0.96
59	Silica	WG	66.5	196	238	52	0.78	Contractive and dilative	Undrained	0.97
60	Silica	WG	70.2	196	257	54	0.79	Contractive and dilative	Undrained	0.96
61	Silica	WG	72.1	200	287	56	0.80	Contractive and dilative	Undrained	0.96
62	Silica	WG	58.2	196	217	35	0.83	Contractive and dilative	Undrained	0.96
63	Silica	GAG	42.1	196	81	17	0.79	Contractive and dilative	Undrained	0.96
64	Silica	GAG	42.1	196	81	17	0.79	Contractive and dilative	Undrained	0.96
65	Silica	GAG	57.2	196	104	13	0.87	Contractive and dilative	Undrained	0.96
66	Silica	GAG	60.1	196	125	23	0.78	Contractive and dilative	Undrained	0.96
67	Silica	GAG	66.5	196	186	20	0.89	Contractive and dilative	Undrained	0.96
68	Silica	GAG	69.1	229	193	36	0.81	Contractive and dilative	Undrained	0.97
69	Silica	GAG	74.1	196	196	22	0.90	Contractive and dilative	Undrained	0.97
70	Silica	GAG	70.3	200	207	29	0.86	Contractive and dilative	Undrained	0.97
71	Silica	GAG	83.3	200	242	35	0.86	Contractive and dilative	Undrained	0.97

PURELY CONTRACTIVE BEHAVIOR

Narrowly graded, intermediately graded and well-graded specimens in loose state all responded to undrained shearing in a purely contractive manner characterized by the attainment of peak strengths at small shear displacements and the rapid reduction of those strengths to very low values at steady state. This type of behavior has been referred to in this paper as mass-liquefaction (flow-liquefaction). Although all NAG, ING, and WG specimens in loose state exhibited purely contractive behaviors, there were still some obvious differences, arising from their differences in grading. These differences are expounded below. A summary of the soils behavior at different normal stresses but same relative density is shown in Fig. 4. Whereas all the narrowly graded specimens underwent complete liquefaction, no specimen of well graded specimens suffered complete liquefaction. A narrowly graded specimen, for example, consolidated at 220 kPa to a relative density of 29.5 % completely liquefied. It may be seen that soon after reaching a peak resistance of 49 kPa it underwent rapid loss of strength that saw its resistance decrease to about zero and remained so until shearing was terminated at 10 m shear displacement. By having its shear strength reduced to zero after failure, the specimen acquired a brittleness index of 1.0. The entire behaviour leading to zero or almost zero steady state strength has been termed complete liquefaction in this paper. A similar test on an intermediately graded specimen (ING) yielded a significantly different result (Fig. 4). A specimen consolidated at 202 kPa and having a relative density of 29.2 % was loaded by incremental addition of shear stress at the rate of 0.98 kPa/sec. It could be observed that after the specimen reached its peak strength of 54 kPa its shear resistance suffered severe and rapid reduction until the resistance became 5 kPa at around 10 mm and remained so until the test was terminated. For considerably losing its shear resistance, the specimen had a high brittleness index of 0.91. The behaviour of a well graded specimen consolidated at 196 kPa to a relative density of 29. 5% is presented in Fig. 4. It may be seen that after the specimen reached peak shear strength of 60 kPa, the sample appeared to have collapsed and underwent rapid loss of resistance until it attained its steady state strength of about 12 kPa where deformation continued without further changes to both shear resistance and effective stress. As a result, the specimen acquired a high brittleness index of 0.80.

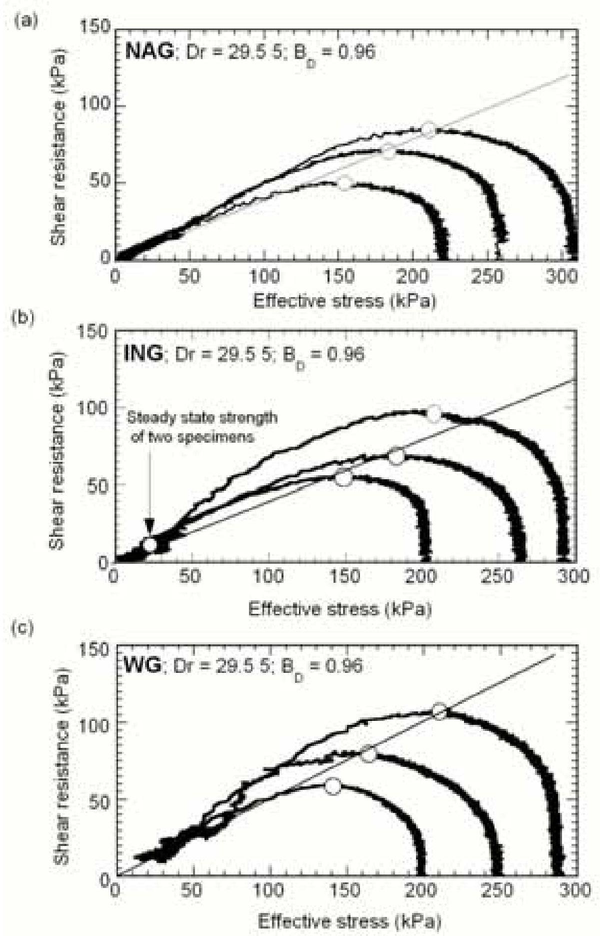

Figure 4: Summary of the behaviour of the specimens at different normal stresses but same relative density (a) NAG specimens (b) ING specimens (c) WG specimens

The sample is thought to have liquefied, but not completely, because of the high excess pore pressure built-up and the consequent low effective stress attained in the process.

Similar tests on loose specimens were carried out at different initial normal stresses. The brittleness index ($I_B = \dfrac{\tau_p - \tau_{ss}}{\tau_p}$) of the specimens plotted against normal stress is shown in Fig. 5 It may be seen from the figure that at every normal stress, the brittleness index of the narrowly graded specimens is unity. Among the specimens, the well graded specimens have the lowest brittleness index. Similarly, the plot of normalized peak and steady state strengths against uniformity coefficient (Fig. 6) show that the higher the uniformity coefficient, the higher the peak and steady state strengths. While the steady state strengths of the three narrowly graded specimens all plotted on zero, those of the well graded specimens plotted on values greater than zero. Of the three intermediately graded specimens, one liquefied completely and as a result its steady state strength plots on zero. Two others did not undergo complete liquefaction; Fig. 6 shows that their steady state strengths plot on values greater than zero.

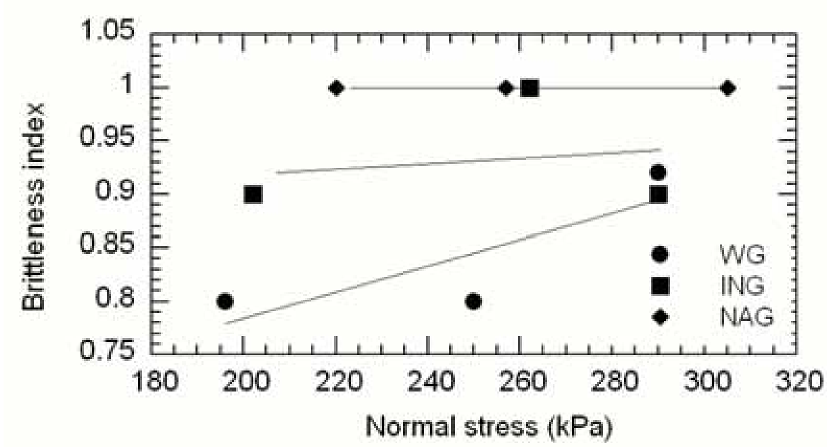

Figure 5: The brittleness index of the loose specimens plotted against normal stress

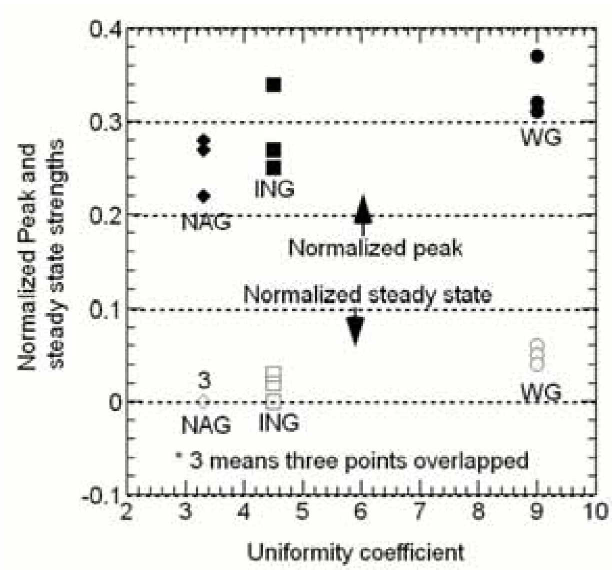

Figure 6: The relationship between uniformity coefficient and normalized strengths

VOLUME CHANGES IN DRAINED TESTS

To investigate the volume changes associated with these loose sands, two drained tests were performed. The results are summarized in Fig. 7. The narrowly graded specimen was consolidated at 200 kPa to D_r = 32.2 %, and Fig. 7 shows that soon after shearing commenced, the specimen first suffered considerable volume reduction until around 30 mm from which point the sample underwent less volume reduction until the test was terminated at 10 m. Volume reduction before the peak where the sliding surface is formed (Wafid *et al.*, 2004) will be caused by the collapse of metastable structure. The major cause of volume reduction after the peak might be grain crushing along the sliding surface, especially after a certain shearing point where the effect of collapse might not exist anymore. A similar test performed on a well graded specimen consolidated at 196 kPa to D_r = 30.2 % shows that soon after shearing commenced, the sample first suffered considerable volume reduction until about 30 mm from which point the sample underwent little volume reduction until the test was terminated at 10 m. It is understood that the volume reduction in the NAG specimen is slightly higher than that of the WG specimen, although it may be difficult to imply that that the greater volume reduction in NAG specimen is due to grading from only two drained tests.

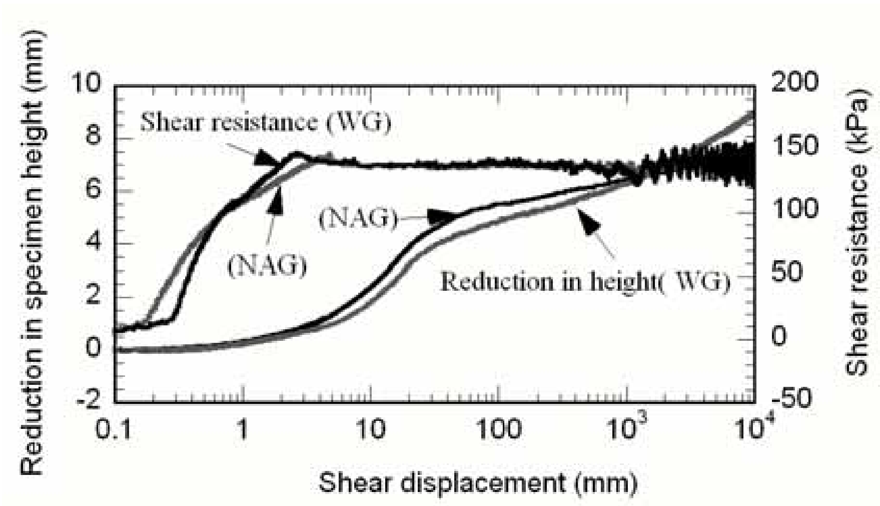

Figure 7: Drained shear behavior of WG and NAG specimens compared

INFLUENCE OF GRADING ON THE COLLAPSE LINE

One of the objectives of the present study was to investigate the sensitivity of the position and slope of the collapse line to changes in uniformity coefficient in hopes of adding to what is already known about conditions necessary for liquefaction. Although research on the mechanism(s) controlling the initiation of liquefaction failure in loose, saturated, sands has been going on for decades now, not much is known about the conditions necessary for liquefaction and the characteristics of liquefiable materials.

Attempts at identifying points in a stress space diagram which could trigger flow liquefaction in granular materials have led to a number of beneficial concepts including the collapse line and the critical stress ratio concepts. Sladen *et al.* (1985) while proposing the concept of collapse surface observed that the peak strengths of a soil consolidated to the same void ratio at different initial confining stresses lay on a straight line passing through the steady-state point. On the basis of some strong evidences, they reported that the position of the line was a function of only void ratio. While Ishihara (1993) and Sasitharan *et al.* (1993, 1994) have made similar observations, there are other works where the position of the collapse line has been represented differently. Vaid and Chern (1985) and Alarcon-Guzman, Leonards and Chemeau (1988) have reported that the critical stress ratio (CSR) line, a line which passes through the peak strengths of a soil at different initial confining stresses and which may be extrapolated through the origin of a given stress diagram was independent of initial void ratio. In the same vein, Vasquez-Herrera *et al.* (1988) and Negussey *et al.* (1988), Vaid *et al.* (1990), Konrad (1993), have not only reported the existence of the line which passes through peak strengths through the origin but have shown that any attempt by effective stress paths to cross this line could trigger flow liquefaction. A similar investigation by Mohamad and Dobry (1986) found that the stress ratio at which flow failure is triggered was independent of relative density and, as a consequence, that the critical stress ratio line or the collapse line extrapolates through the origin of a stress space diagram.

Figs. 4 *a*, *b*, and *c* show that while the slope of a line that tends to pass through the peaks to the origin of the shear resistance-effective stress space diagrams appear to be greater in the well graded specimens (Fig. 4 *c*) than in the Intermediately and narrowly graded specimens (Fig. 4 *a*, *b*), there is no strong indication that grading affects the position of the line. It may be seen that the position of the line is essentially the same in the well-graded, intermediately graded, and narrowly graded materials with all the materials characterized by a collapse line extrapolating through the origin through the peak points.

DILATIVE BEHAVIOR

SPECIMENS IN MEDIUM-DENSE STATE

Typical result in which dilative behaviour was observed in narrowly graded specimens is summarized in Fig.8. All the specimens were consolidated at different normal stresses to the same relative density of 44.2 %. It is noted that at the same relative density but different normal stresses, the steady state strengths were the same. Another point notable in this Figure is that the mechanical behaviour of a medium-dense narrowly graded specimen is clearly and significantly different from that of a narrowly graded specimen in loose state. Whereas narrowly graded specimens in loose state suffered complete liquefaction (because their steady state strengths were zero) the specimens in medium-dense state has relatively high steady state strength. For example, the specimen consolidated at 196 kPa to a relative density of 44.2 % attained peak resistance of 93 kPa after barely dilating, and then subsequently mobilized steady-state strength of about 40 kPa after failure. Soon after failure, precisely at 10 mm corresponding to 102 seconds, the pore pressure had risen to 136 kPa, which was about 68% of the total normal stress (Fig. 8c). The peak strength may be relatively low, as we shall soon see, but its steady state strength is certainly high. Why such a reversal of fortune in medium-dense state? Why would a sample type that is prone to complete liquefaction in loose state turn around to have high steady state strength in medium-dense state? It seems important to investigate this question capable ring shear apparatus that permits reliable measurement of pore pressure, shear resistance, and large shear displacement, by analyzing the pore pressure and effective stress features along the stress paths of four artificially constituted silica sand samples whose uniformity coefficients are different.

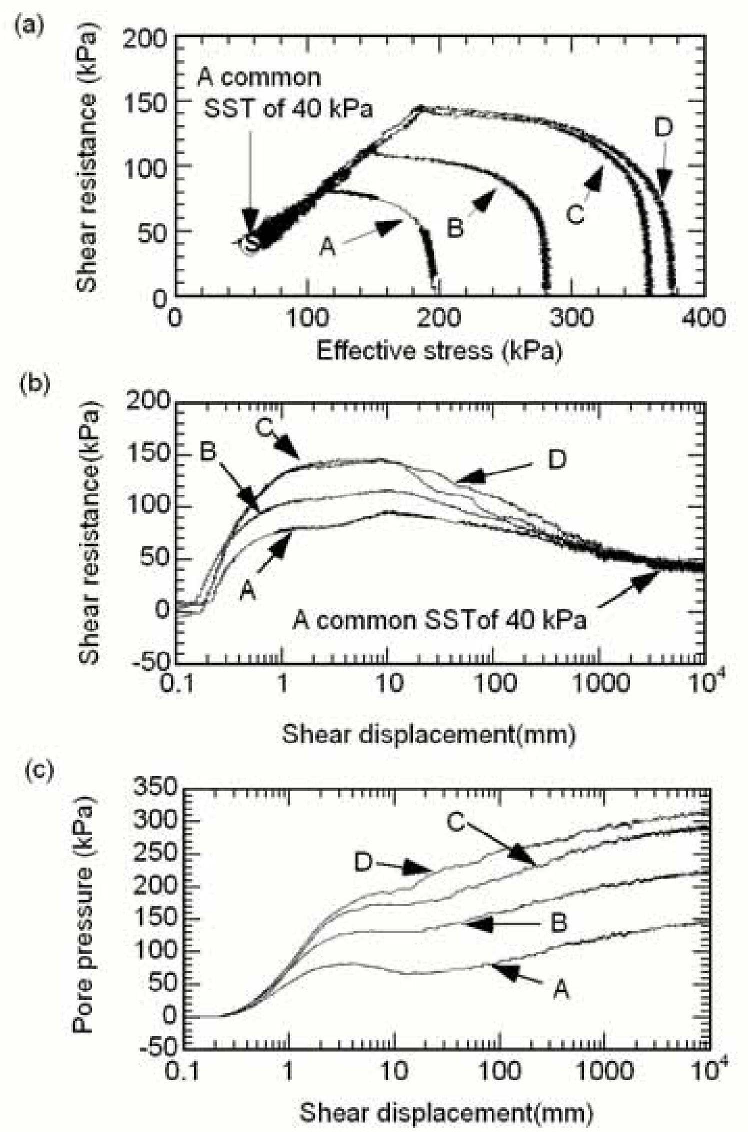

Figure 8: The undrained response of medium-dense NAG specimens at different normal stresses

The behaviour of medium-dense intermediately graded specimens at different normal stresses but same relative density is shown in Fig. 9. A specimen of the intermediately graded samples consolidated at 196 kPa and having a relative density of 44.3% clearly underscores the influence of grading on the shear behaviour of granular soils. It attained peak resistance of 81 kPa after dilation. Upon failure, its strength was reduced to a steady state value of about 32 kPa after 10 m of shear displacement. Fig. 9 shows that although the normal stresses are different, all the ING specimens reached the same steady state strengths of about 32 kPa. It may be noticed from Figs 8 and 9 that with the exception of the specimen at a normal stress of 196 kPa, the peak strengths of the intermediately graded specimens are higher than those of the narrowly graded ones. But, the steady state strengths of the intermediately graded specimens are lower than those of the narrowly graded ones. The results seem interesting because they have shown that the pre- and post-failure behaviour of soil samples can differ widely, and that predictions based solely on speculation without careful laboratory tests could dangerous.

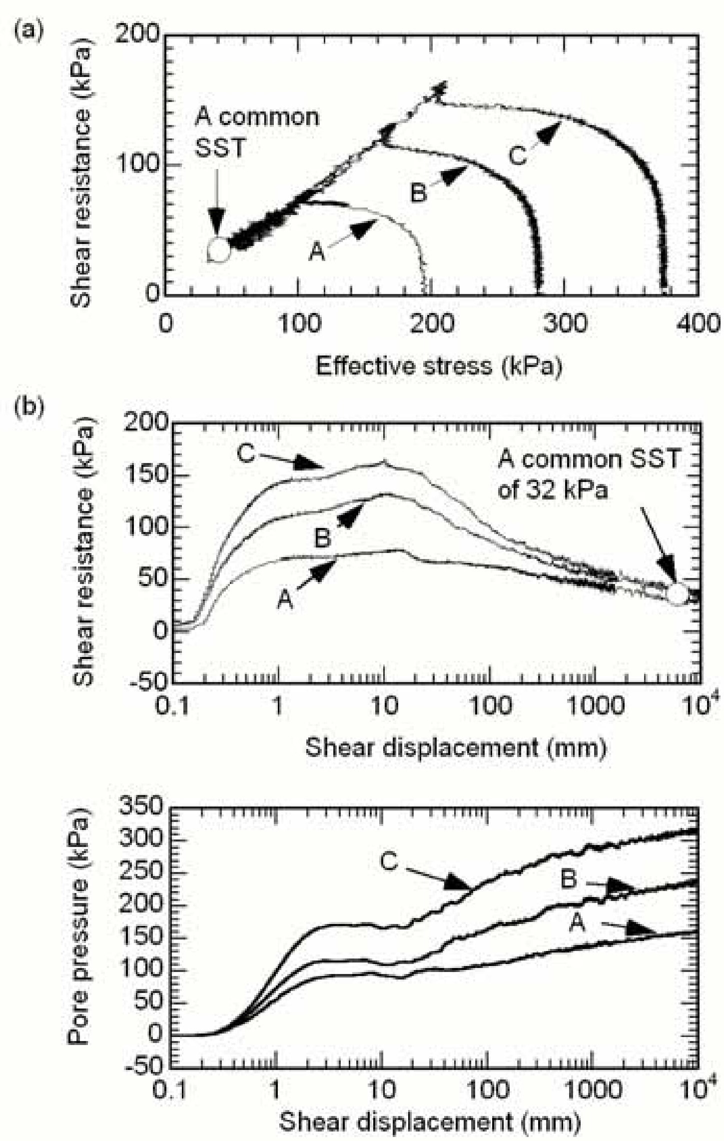

Figure 9: The undrained response of medium-dense ING specimens at different normal stresses

The results of tests on well graded specimens at different normal stresses but same relative density are summarized in Fig. 10. A well-graded specimen, under effective normal stress of 202 kPa, which had a relative density of 44.3 %, is shown to be very different from the NAG and ING specimens at the same test condition. Once the stress path reached point PT, it dilated and attained peak strength of 107 kPa at 11 mm (shear displacement), after which, the specimen failed and had its resistance quickly reduced to a residual value of 23 kPa after 10 m of displacement. The rapid reduction of shear resistance after failure observed in the specimen under consideration might have resulted from particle crushing at the shear zone, and the consequent high pore pressure increase. The rapidity of pore pressure increase was such that at about 5 m shear displacement corresponding to 130 seconds, the pore pressure had risen to 177 kPa, which was about 88.5% of the normal stress imposed on the specimen (Fig. 10 b, c). The mode of behaviour after failure whereby the resistance of the specimen was rapidly reduced to a small residual value is especially typical of the WG specimens containing larger particle sizes. It is noted that the WG at different normal stresses have different peak strengths (which are higher than those of both the ING and NAG specimens) but reach the same steady state strengths of about 23 kPa (which are lower than both the ING and NAG). It should be recalled that in loose state, well graded specimens had higher steady state strength. In medium-dense state however, the behaviour is reversed.

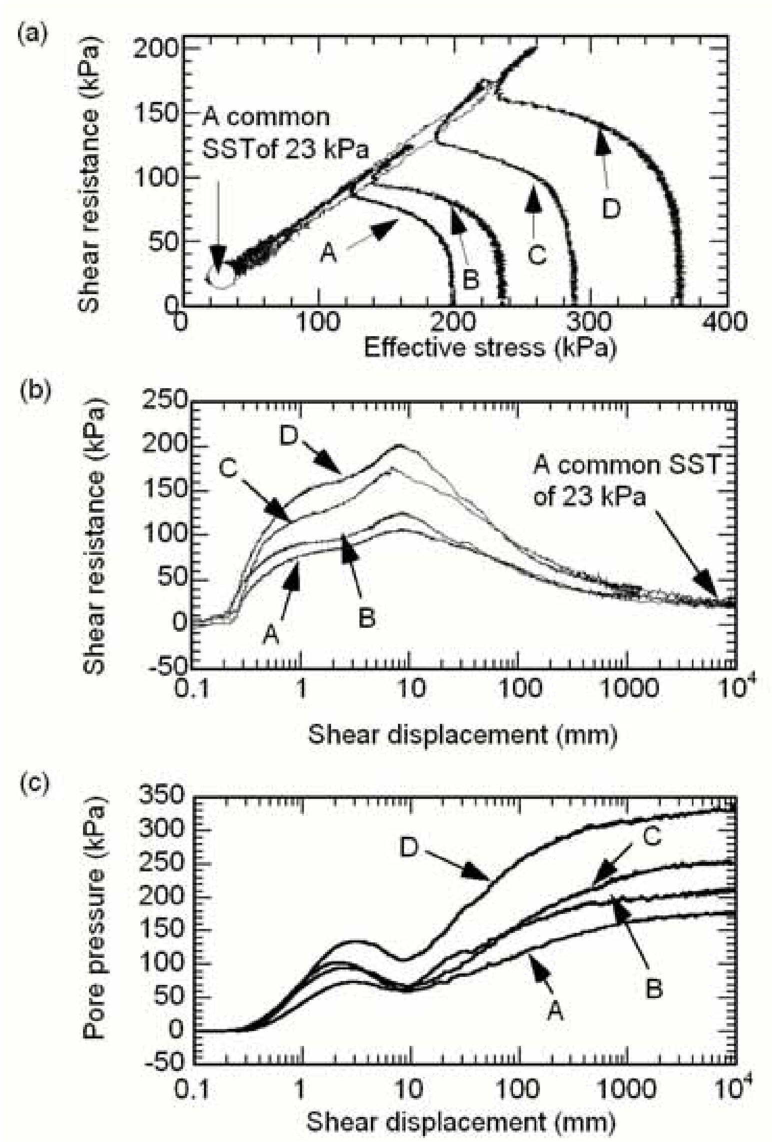

Figure 10: The undrained response of medium-dense WG specimens at different normal stresses

The result of a gap graded specimen is shown in Fig. 11. It may be observed from the Figure that even at a relative density of 57.2 %, the gap graded specimen has very low steady state strength.

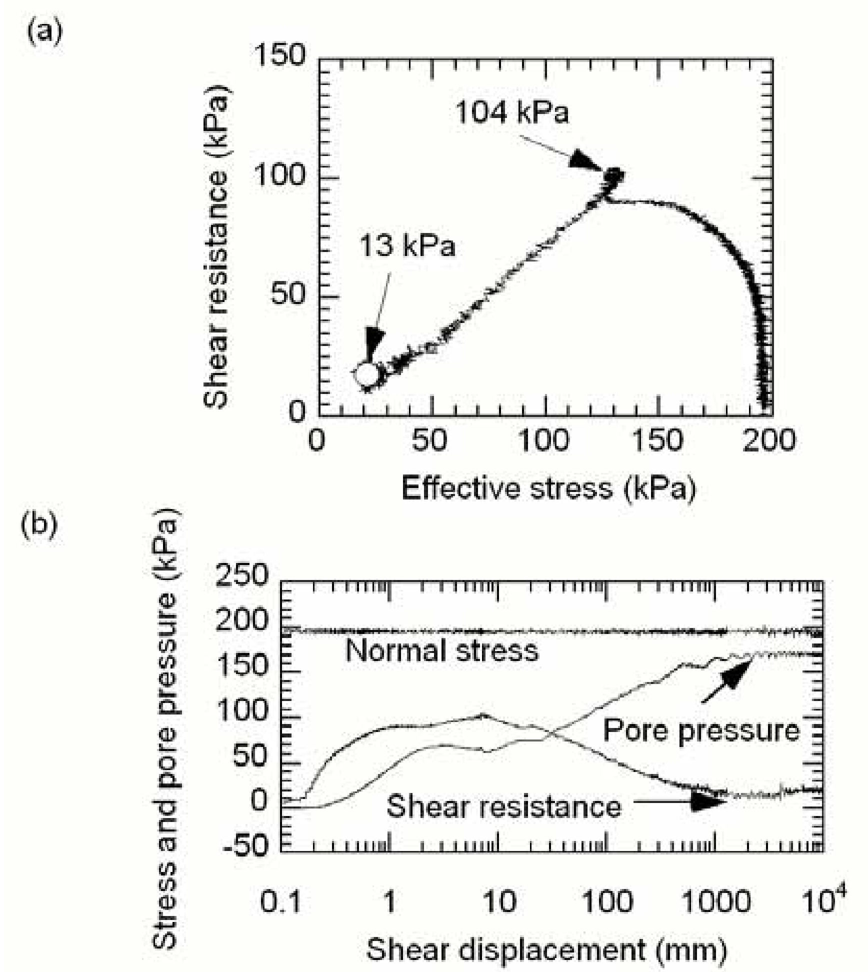

Figure 11: The undrained response of a medium-dense GAG specimen with a relative density of 57.2 %

SPECIMENS IN MEDIUM-DENSE STATE

The specimens in dense states appear to follow the same pattern of behaviour observed for medium-dense specimens with the specimens undergoing what has been termed a reverse behaviour in this paper. The results of the tests conducted on medium-dense and dense specimens are summarized in Fig. 12. The Figure, which contains only data acquired at an initial effective normal stress of about 200 kPa, shows that there are significant differences between their peaks in medium and dense states; the differences appear to widen as density increases. This may imply that the effect of grading on their peak strengths may be significant for only densities above a certain limit. The proximity of peak strengths of narrowly and intermediately graded specimens may find explanation in the closeness of their uniformity coefficients. While the peak strengths of dense intermediately graded specimens approach those of dense narrowly graded ones, the well-graded specimens at the same state move away. On the basis of the results presented in Fig. 12a it is evident that at the same relative density and effective normal stress, well-graded specimens have higher peak shear strengths than the other specimens.

In what may amount to a reversal of behavior, well graded specimens, which contain some quantity of relatively large particle sizes, can be observed in Fig. 12b to have lower steady state strength than the intermediately and narrowly graded which are composed of only finer sizes. Test results seem to indicate that the degree of crushing or breakage is heavily dependent on the size of particles of which the specimens are composed (Marsal, 1967; Lee and Farhoomand, 1967; Hardin, 1985; Lade and Yamamuro, 1996).

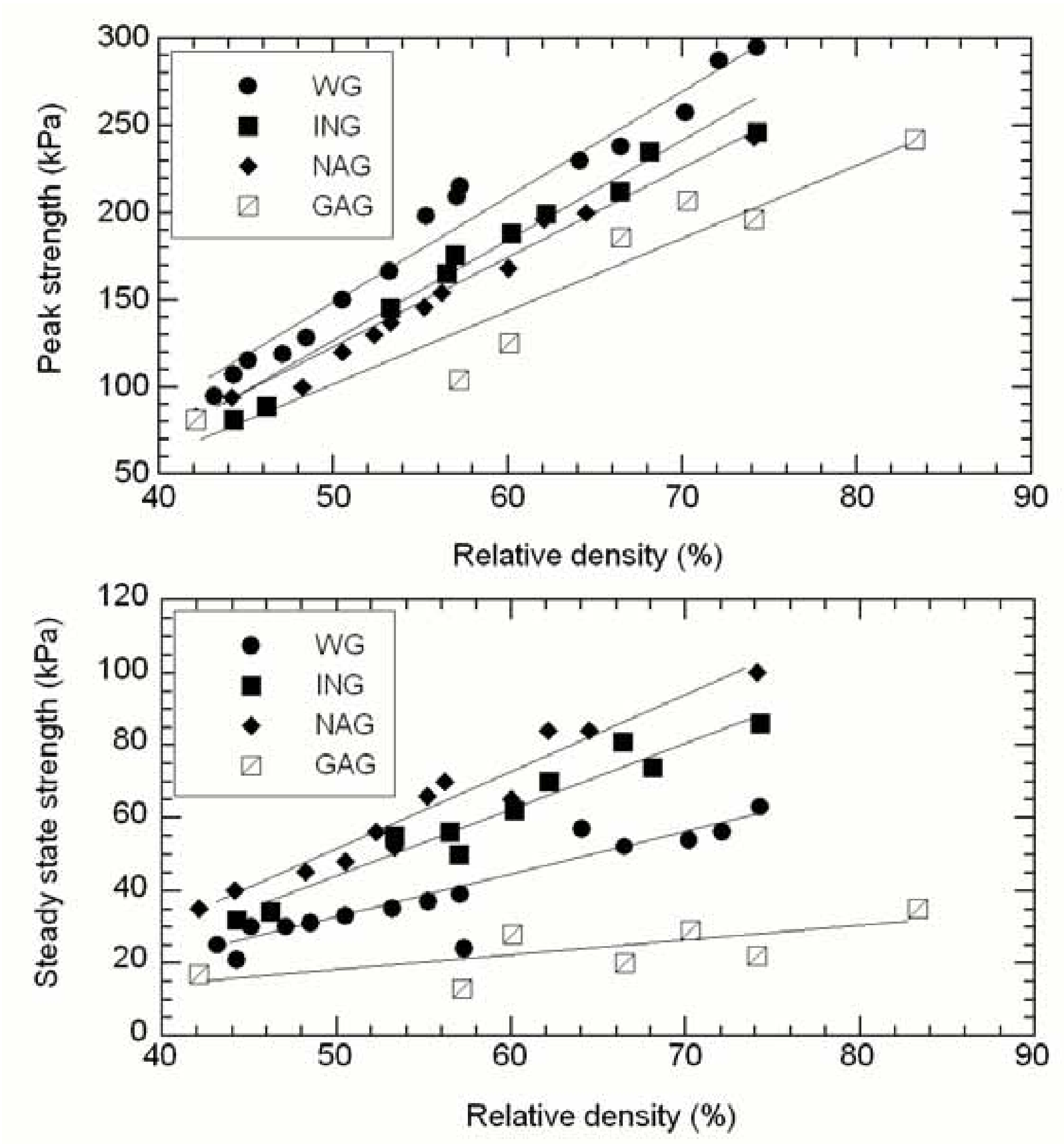

Figure 12: The relationship between relative density and shear strength

Analyses have shown that, at the same initial state, well-graded sands have higher peak strengths than poorly graded ones, the difference increasing as the sands become denser. In an interesting reversal however, it has been shown that in medium and dense states, the steady state strengths of well-graded materials are lower than those of the ING and NAG, the difference, again, increasing with relative density (Igwe et al., 2005) From the view point of public safety, this paper shows that the better graded soils in medium and dense states are more dangerous after failure because of their potential for large post-failure travel distances than the NAG, and ING specimens.. These results may aid engineers when they make decisions on what soils to use for civil engineering works; or scientists when they assess slope stability, investigate landslides, or plan prevention methods (Igwe *et al.*, 2004b; 2005; 2007; Wang *et al.*, 2002).

CONCLUSION

1. Well-graded specimens in medium-dense to dense states have higher values of peak strength than the rest of the specimens at the same condition, with the difference appearing to increase with relative density. The values are ranked as WG> ING> NAG> GAG**.** Better interlocking of particles at contacts achieved by mixing a wide range of particle sizes in the well-graded specimens is thought to be responsible for the higher peak strength values associated with the well-graded specimens.

2. The steady state strengths of specimens in loose state showing collapse behavior are very small or almost zero The steady state strengths of specimens in medium to dense states have greater values in proportion to the relative densities. The values are affected by grading of sands and ranked as NAG> ING> WG> GAG. It is noted that whereas the steady state strengths are affected by grading, the friction angle at steady state is not affected by grading as stated in conclusion 8 below.

3. It is noted that widely graded specimens have greater peak shear strengths than poorly graded specimens of NG and ING. On the contrary, the steady state strengths of well graded specimens are smaller than those of poorly graded specimens. It may be interpreted as fine particles can exist within big pores of coarse particles in well graded specimens. So the packing of grains in well graded specimens is dense, it will increase difficulty of overriding of grains in shearing. Accordingly the peak shear resistance will be increased. Difficulty of overriding is likely to result in greater possibility of grain crushing during shearing. It may generate high excess pore water pressure and result in lower steady state shear strengths.

The post-peak behaviors of these samples are important from the viewpoint of public safety. It is noted that although well-graded samples have higher peak strengths, their low steady-state strength may pose a serious safety concern because of their potential for long travel distances.

4. Gap graded specimens have the lowest peak and steady state strengths. The coarse and very fine grains in gap graded specimens may be packed in a way that permits the overriding of grains readily. The coarser grains are vulnerable to crushing, and as they crush or break, the finer particles tends to fall within the pores of coarse grains, making it denser but also resulting in a smaller steady state value due to high pore water pressure.

5. It is noted that specimens of a given grading at different initial normal stresses but having approximately the same relative density will all reach the same steady state strength**.**

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