The undrained response of silt sands containing gas bubbles

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ABSTRACT
Silt sands containing gas are commonly found in coastal and offshore environments. Although many offshore failures have been documented in silt sands, the prediction of such soil behaviour still contains considerable uncertainties. The purpose of this research is the determination of whether loose silt sand containing small amounts of gas can strain soften and undergo flow liquefaction. The critical state framework is used to differentiate between the effects of silt and gas bubbles on the undrained response of loose gassy silt / sand mixtures. The results of triaxial testing using Ottawa sand and Penticton silt mixtures with low gas content are presented. Analyses indicate that loose gassy silt sands can strain soften and experience flow liquefaction and that the steady state line is particularly sensitive to silt content.

RÉSUMÉ
Les sables limoneux contenant des gaz sont souvent trouvés dans des environnements côtiers et offshore. Bien que de nombreuses ruptures de sol sous-marin aient été documentées pour des sables limoneux, la prédiction de telles ruptures contient toujours de nombreuses incertitudes. Le but de cette recherche est de déterminer si les sables limoneux meubles contenant de petites quantités de gaz peuvent perdre leur cohésion et se liquéfier. Le cadre de l'état critique est utilisé pour différencier les effets des bulles de gaz et limon dans la réponse non drainée des mélanges meubles de sable et limon. Les résultats des tests triaxiaux utilisant des sables d'Ottawa et des mélanges de limons de Penticton avec de faibles quantités de gaz sont présentés. Les analyses indiquent que les sables limoneux gazeux meubles peuvent perdre leur cohésion et se liquéfier, et que la ligne d'état critique est particulièrement sensible à la teneur en limon.

1 INTRODUCTION

The presence of gas in silt sand can have an important influence on soil behaviour. Previous work by Haththotuwa and Grozic (2009) showed that soil behaviour of saturated silt sands depends on silt content, drainage conditions and loading path and that these factors are crucial to understanding the resulting behaviour.

This research work explains how silt sands containing small amounts of gas could strain soften and undergo flow liquefaction. Submarine slope failures are directly associated with flow liquefaction in silt sands, and numerous case studies of submarine slope failures can be found in literature. One example is flow slides that have occurred in the Fraser River delta (Chillarige et al. 1997b). Christian et al. (1997) showed the residual pore pressures in sediments during low-tide conditions could lead to the triggering of flow liquefaction failures of the Fraser River delta. They concluded that the reduction in effective stress leading to flow liquefaction in Fraser River delta is largely caused by the presence of gas (methane) dissolved in pore water causing residual pore pressures. Some of the other flow liquefaction failures related to submarine slope failures are listed in Table 1. Such failures may have occurred due to the formation of a unstable gas-soil-water matrix under low-tide conditions.

Despite the wide occurrence of silt in submarine soils, liquefaction research is often performed on clean sands with the assumption that the behaviour of gassy silty sand (also referred to as silt sands containing gas bubbles) is similar to that of sand. The aim of this research is the evaluation of the undrained behaviour of plastic gassy silt sands within the framework provided by critical state soil mechanics.

In order to capture the critical state behaviour of gassy silt sands, an experimental program has been carried out using mixtures of Ottawa sand and Penticton silt.

2 BACKGROUND

2.1 Gassy Silt Sand

Gassy silty soil is a multiphasic system that is composed of three phases, namely soil (silt and sand), water and air. Gassy soils contain a relatively large amount of gas dissolved in the pore fluid, compared to unsaturated soils (Sobkowicz and Morgenstern 1984). Generally, gassy soils are found with a large number of small bubbles embedded in the pore water or with large bubbles of gas in the matrix of a fully saturated soil. In this research, it was assumed that the soil matrix contained small bubbles embedded in its matrix.

When there is no gas phase, the above soil matrix transforms into a two-phase system that consists of soil (sand and silt) and water. These soils, also known as silt sands, can be explained by the intergrain concept proposed by Thevanayagam et al. (2002). The concept states that non-plastic silt behaviour can be categorized into two groups: sand dominated behaviour and fines dominated behaviour. The fines within a sand/silt mixture are regarded as voids when the fines content is low (fines do not participate in the resistance of shearing). When the soil matrix contains more silt, a sand particle can be considered equivalent to a void (sand grains may not contribute to the shearing resistance).

Table 1. Statically induced liquefaction case studies in silty soils (after Chillarige et al. 1997a)

Site	Type of failure	Predominant soil type	Reference
The Netherlands	Low tides	Loose fine sand	Koppejan et al. 1948
Magdalena River delta,1935	Rapid sedimentation	Sand and silt	Menard 1964; Morgenstern 1967
Helsinki Harbour, 1935	Rapid filling	Sand and silt	Andresen and Bjerrum 1967
Follafjord slides, 1952	Dumping of dredged soils	Loose fine sand, silt	Terzaghi 1956; Bjerrum 1971
Orkdalsfjord, 1930	Low tides	Loose fine sand, silt	Terzaghi 1956; Andresen and Bjerrum 1967
Finnivaka slide, 1940	Low tides	Loose fine sand, silt	Bjerrum 1971
Hommelvika, 1942	Low tides	Loose fine sand	Bjerrum 1971
Trondheim, 1888	Low tides	Loose fine sand, silt	Terzaghi 1956; Bjerrum 1971
Scripps Canyon, 1959, 1960	Free gas and storm waves	Sand	Dill 1964; Morgenstern 1967
Puget Sound, 1985	Low tides	Loose sand	Kraft et al. 1992
Skagway, Alaska, 1994	Low tides of 4 m	Loose silty sand, silt	N.R. Morgenstern, unpublished data, 1995
Howe Sound, 1955	Low tides	Fine sand and gravel	Terzaghi 1956
Kitimat Fjord, 1975	Low tides of 6 m	Loose silty sand	Morrison 1984
Nerlerk sand berms, 1983	Fill placement	Loose sand	Sladen et al. 1985
Fraser River delta, 1985,	Low tides of 5 m	Loose fine sand silt	McKenna and Luternauer, 1987

Sobkowicz (1982) examined gassy soil response under undrained unloading conditions and illustrated its differing behaviour, as compared to unsaturated soils and saturated course and fine grained soils (Figure 1). According to Figure 1, for saturated and gassy soils above the liquid/gas saturation pressure ($u_{l/g}$), changes in pore fluid pressure (u) remain constant in response to decreases in total stress. However, when u reaches $u_{l/g}$, gas begins to exsolve, and u remains almost constant for further changes in total stress. As the effective stress becomes small, soil compressibility is increased and pore fluid pressure rapidly decreases. Eventually, u becomes equal to the total stress (σ), and the effective stress (σ') is reduced to zero. At this point, B=1 and the change in pore pressure is equal to the change in total stress.

Grozic et al. (1999) studied the behaviour of loose gassy sand under monotonic consolidated undrained conditions and concluded that such soils can strain soften and experience flow liquefaction.

2.2 Critical State Line

Poulos (1981) explained that the critical state of a soil can be described as a condition where the soil is continuously deforming at a constant volume, constant normal effective stress, constant shear stress and constant velocity. Been and Jefferies (1985) proposed the equation of $e_{CS} = \Gamma - \lambda \ln(p')$ for the critical state line (CSL) of sand, based on critical state data of void ratio and mean stress, as shown in Figure 2. Research work by Sasitharan (1994) showed the CSL changes from a low slope to a higher slope and from a low stress level to a higher stress level, due to grain crushing effects at higher stress levels.

Monotonic triaxial silt sand experiments carried out by Poulos et al. (1985) showed the slope of the CSL is

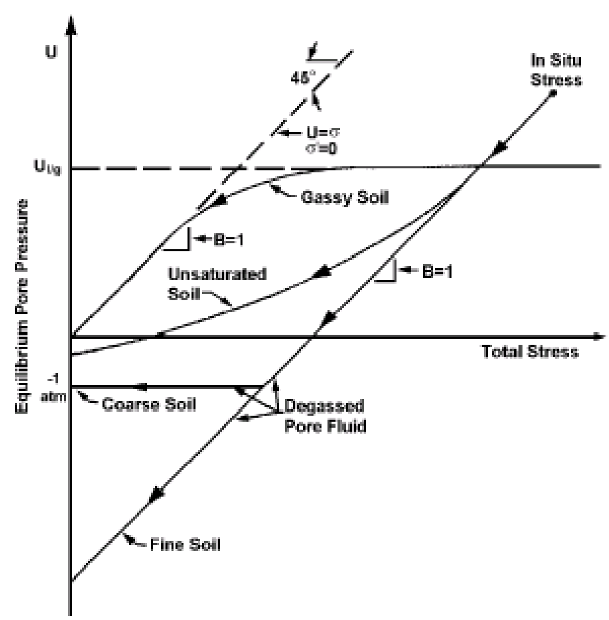

Figure 1. Undrained equilibrium behaviour of an element of soil on unloading: the effect of the amount of gas dissolved in the pore fluid (after Sobkowicz and Morgenstern 1984)

affected by soil gradation and grain angularity. For example, a significant change in the slope can be expected when minor changes are made in soil gradation. A similar conclusion was made by Olson et al. (2001), who stated that the grain angularity may affect the slope of the CSL more significantly than the fines content. However, Yang et al. (2006) showed that the position of

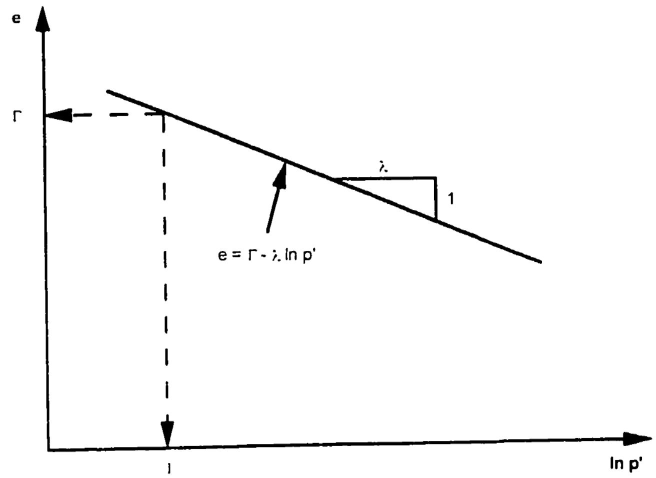

Figure 2. CSL in void ratio – effective mean normal stress space

the CSL of silt/sand mixtures is sensitive to the silt content, but not to the slope.

The slope of CSL is also governed by the plasticity of silt/sand mixtures. Yang et al. (2006), Bouckovalas et al. (2003), Thevanayagam et al. (2002) and Zlatovic and Ishihara (1995) used non-plastic silts in their experiments. However, silt plasticity information was not supported with silts used by Naeini and Baziar (2004) and Been and Jefferies (1985).

For silt/sand mixtures, depending on the positions of the CSLs, two groups can be identified: group 1 believes that the slopes of the CSLs change with fines content (Been and Jefferies 1985, 1991); and, group 2 has concluded that the CSLs are more or less parallel (Zlatovic and Ishihara 1995; Yang et al. 2006).

Similar to results obtained by Yang et al. (2006) and Bobei et al. (2009) indicated the presence of fines in the parent sand matrix was found to have the effect of shifting the position of critical state line downwards.

3 LABORATORY PROGRAM

3.1 Experimental Setup

3.1.1 Materials

Reconstituted Ottawa sand, which is a round to sub-rounded quartz, and Penticton silt were used in the experimental program. Ottawa sand had a specific gravity of 2.65 and was graded in accordance to ASTM C-778 standards. The minimum and maximum void ratios (0.81 and 0.51) were determined using ASTM D2049 standards. Based on the particle distribution curve, it can be noted that Ottawa sand had a uniform distribution with a mean grain size of 0.35 mm (i.e. D_{50}). Penticton silt contained less than 10% of clay and reported a specific gravity of 2.70, based on hydrometer test results.

3.1.2 Testing Apparatus

The triaxial system used for the tests was modified from an unsaturated stress path triaxial system. A double walled cell construction enabled precise specimen volume change measurements. The cell pressure capabilities were 2,000 kPa, which is higher than a conventional system. The system was servo controlled and capable of stress path or cyclic testing. A specialized circulation system enabled the replacement of the pore fluids under high back pressures.

3.1.3 Specimen Preparation and Testing Procedures

Reconstituted specimens (100% Ottawa sand; 90% Ottawa sand and 10% Pendicton silt; and, 80% Ottawa sand and 20% Pendicton silt) were prepared using the moist tamping method. This technique consists of placing moist soil layers into a mould and tamping each layer with a specified force.

Following assembly within the triaxial apparatus, carbon dioxide was percolated through the sample for a period of 20 to 30 minutes. Next, de-aired distilled water was introduced to the bottom of the specimen and collected from the top of the specimen. To ensure complete saturation, 2 to 3 times the pore volume of water was allowed to pass through the specimen. Cell and back pressures were then slowly increased to 800 kPa and 750 kPa, respectively. At this point, compression (P) wave velocity measurements, using ultrasonic apparatus, were done to check saturation; and, P wave velocity values in the range of 1750 – 1800 m/s were obtained.

After the P wave test, consolidation was induced by increasing the pressures to 850 kPa, 950 kPa, or 1050 kPa, while maintaining a constant back pressure of 750 kPa. Consolidation took approximately one day. The specimen pore water was then replaced with carbon dioxide saturated water (approximately three times the volume of voids was used) by circulating the gas dissolved water through the specimen under a pore pressure of 750 kPa with a driving head of 0.5 m.

In order to produce free gas bubbles, pore and cell pressures in this ramping down stage were dropped to 400 kPa, 500 kPa or 600 kPa and 700 kPa, respectively, while maintaining a constant effective stress. The objective of the ramping down was the reproduction of the same testing procedures that would be used for silt-sand soil testing.

All valves to and from the specimen were then closed, thereby creating an undrained boundary condition; and, shearing was commenced under strain controlled conditions. An axial strain rate of 0.2% per minute was used, except where noted. Pore pressure, cell pressure and axial and volumetric deformations were measured.

Table 2. Summary of test results.

Sample No.	Silt content (%)	$P'_{initial}$ (kPa)	e_{final}	P'_{peak} (kPa)	q_{peak} (kPa)	$P'_{steady\ state}$ (kPa)	$q_{steady\ state}$ (kPa)	$S_{r,initial}$ (%)
S1		105	0.898	97	63	1	12	98.9
S2	0	204	0.857	185	120	7	16	98.2
S3		301	0.819	258	167	8	14	98.8
C11		114	0.842	87	60	6	15	96.3
C12	10	204	0.806	149	90	7	14	96.1
C13		304	0.757	236	148	11	21	96.3
C21		109	0.702	88	51	4	13	95.6
C22	20	203	0.655	150	76	12	20	95.5
C23		307	0.605	359	408	363	416	84.2

4 LABORATORY RESULTS AND DISCUSSION

A summary of experimental results is presented in Table 2. The test results of isotropically consolidated undrained tests for gassy Ottawa sand are shown in Figure 3, with shearing commencing from a mean normal effective stress (p') in the range of 100 to 300 kPa.

The effective stress paths for gassy Ottawa sand (Figure 3) plummet towards the origin of the *q–p′* plane (where *q* is the deviatoric stress and *p'* is the mean normal effective stress) after reaching their respective peak deviatoric stress states, indicating strain softening and flow liquefaction behaviour. Using the same test conditions, effective stress path results for gassy silt sands (silt:sand = 20:80 and 10:90) are presented in Figures 4 and 5.

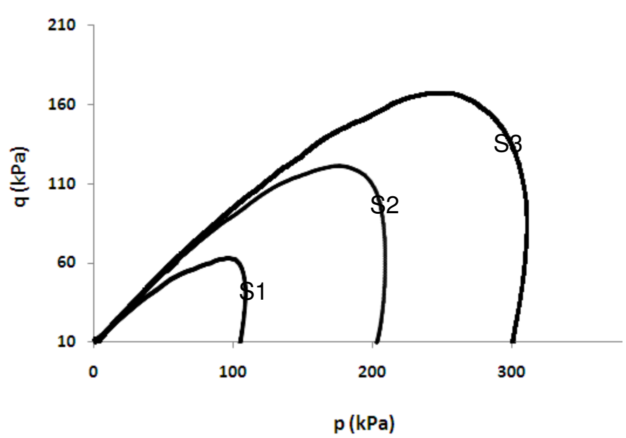

Figure 3. Effective stress paths (*q–p′*) for gassy Ottawa sand

Analysis of Figures 3 to 5 shows a significant drop of peak strength of the gassy silt sands compared to the gassy sand. The strength reduction is the greatest at the lowest initial confining pressure of 100 kPa, with similar percentage strength reductions observed for silt contents of 10% and 20%, compared with pure sand. The drop of peak strength may be due to the high compressibility of gassy silt sands, compared to gassy Ottawa sand.

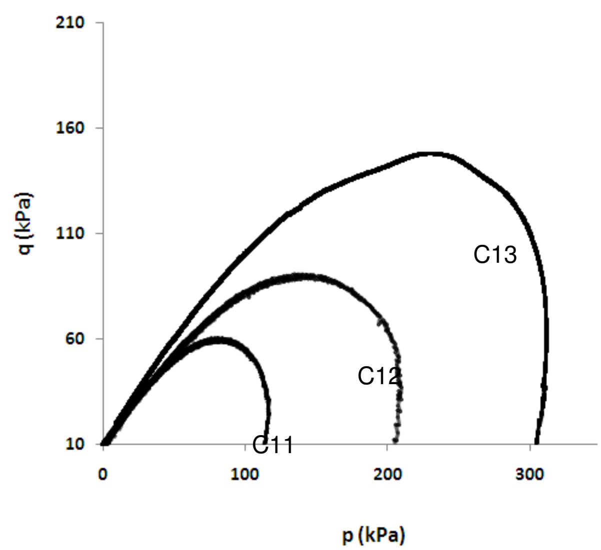

Figure 4. Effective stress paths for gassy silty sand (90% sand and 10% silt)

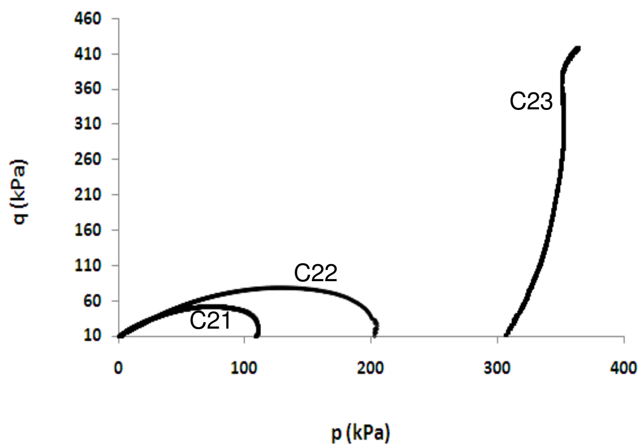

Figure 5. Effective stress paths for silty sand (80% sand and 20% silt)

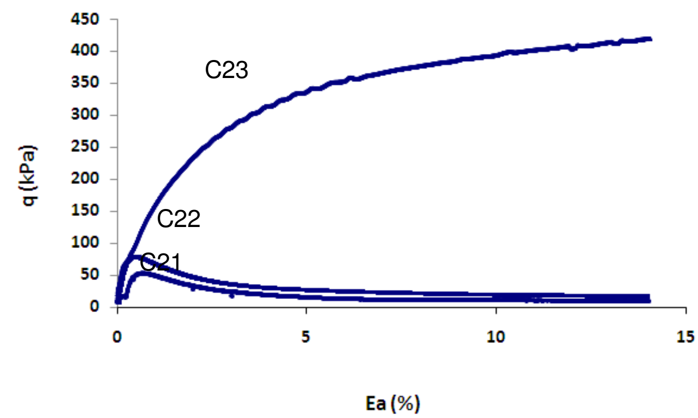

Figure 7. Stress strain plots (q-ε_a) for silty sand (80% sand and 20% silt)

The deviator stress versus axial strain (q–ε_a) curves of C11 to C13 and C21 to C23 are shown in Figures 6 and 7, respectively. Other than C23, all other stress–strain curves initially show a similar response, which is a sharp increase in q to a peak at $\varepsilon_q \leq 1.0\%$. This is followed by a rapid strain softening to a minimum value, q_{min} in all the tests. These curves illustrate the effect of initial consolidation stress, clearly indicating higher peak strengths for higher effective consolidation pressures. The C23 curve illustrates strain hardening behaviour; therefore, a distinctive peak on the curve could not be recognised.

The pore water pressure of the C21 to C23 tests initially increased to approximately the effective confining pressure and then remained constant with shearing after strain softening to q_{min} (after 10%), as shown in Figure 8. Similar behaviour with high pore water pressure values were observed on tests C11 to C33.

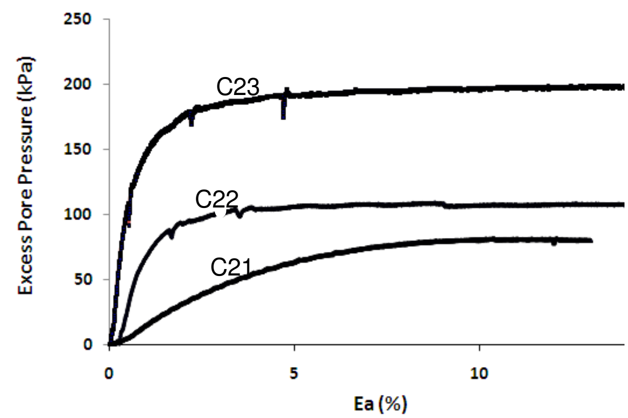

Figure 8. Pore pressure versus axial strain plot for silty sand (80% sand and 20% silt)

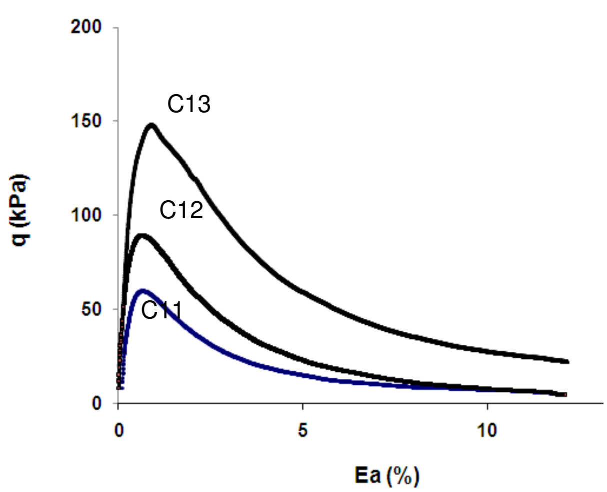

Figure 6. Stress strain curves (q-ε_a) for silty sand (90% sand and 10% silt)

The void ratio versus mean effective stress for the end of test conditions for gassy saturated Ottawa sand and gassy silty sand tests for 10% and 20% of silt by weight are plotted in Figure 9. This chart enables a detailed examination of the uniqueness of the critical and steady states for gassy sand and gassy sand with fines. According to Figure 9, for gassy sand, a linear trend is observed. The results of this research indicate the trend line obtained for gassy sand possesses a shape similar to gassy sand with fines. More specifically, the inclusion of fines (10% and 20% of silt by weight) appears to shift the critical state line downwards (i.e. without any significant changes in shape).

Further analysis of the data shows that trend lines on the e-p' space (Figure 9) can be represented by a unique critical state line (Figure 10), based on the intergrain concept explained by Thevanayagam et al. (1997, 2002). Although Thevanayagam's concept is applicable for non-plastic saturated silts, the results of this research indicate silts with less than 10% clay content and low gas content (i.e. degree of saturation is greater than 95%) show similar behaviour. Results confirm the fines content has an influence on the position of the critical state line, but no impact on the slope of the line; and, when corrected for fines content, the CSL collapses back to a unique line.

Figure 11 shows the CSLs for gassy silty sands and silty sands under similar test conditions. Results confirm gassy bubbles containing in gassy silty sands has an influence on the position and slope of the CSL with that of silty sands (Haththotuwa and Grozic, 2009).

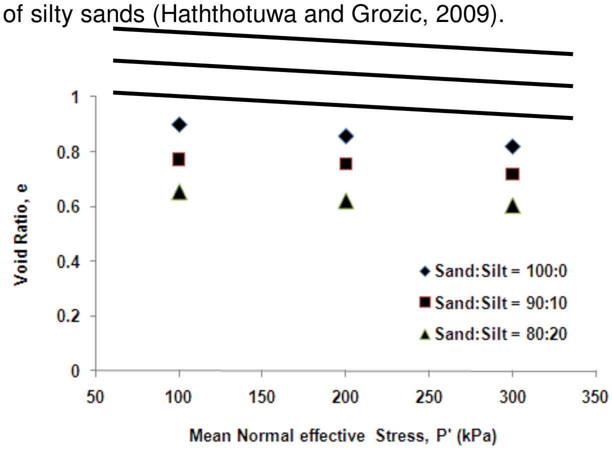

Figure 9. Void ratio versus mean effective stress plot

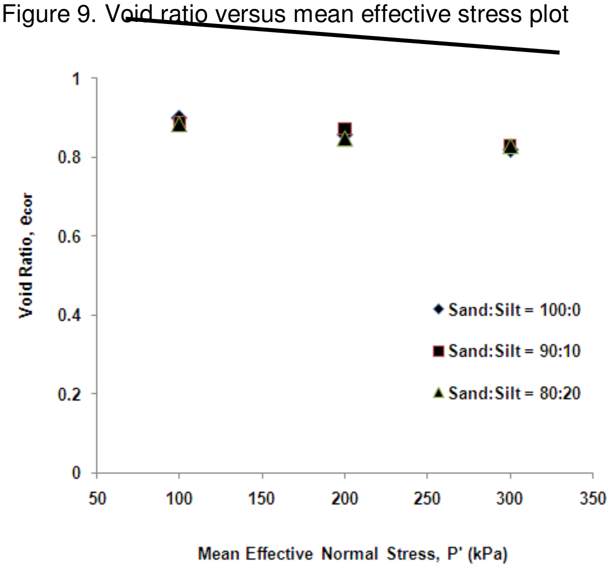

Figure 10. e_{cor} - p' plot (e_{cor} is the modified void ratio, based on the intergrain concept)

5 CONCLUSIONS

This paper presents experimental results on the behaviour of gassy sand with a small amount of plastic fines (10% and 20% of silt by weight). The equivalence and uniqueness of the critical state and steady state lines for the void ratio versus mean effective stress at the end of the test conditions for the gassy Ottawa sand and gassy silty sand tests at 10% and 20% of silt by weight are investigated. This enables a detailed examination of

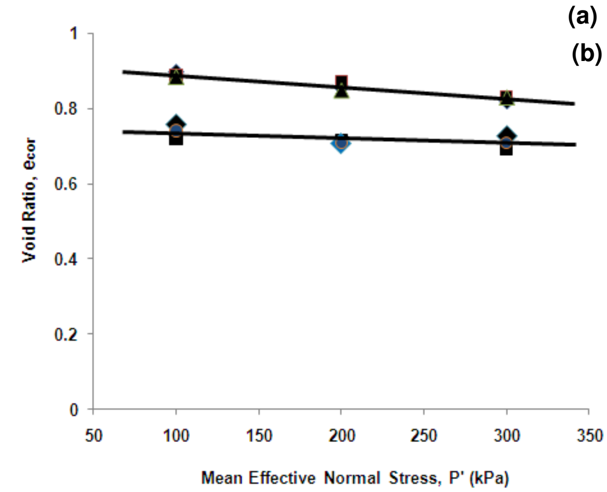

Figure 11. e_{cor} - p' plot for: (a) Gassy silty sands; (b) Silty sands (Haththotuwa and Grozic, 2009)

the uniqueness of the critical and steady states for gassy sand and gassy silty sands. For gassy sand, a linear trend is observed.

The results of this research indicate the trend line obtained for gassy sand possesses a shape similar to gassy sand with fines. More specifically, at a high degree of saturation, the inclusion of fines appears to shift the critical state line downwards (i.e. without any significant changes in shape) when silt is added to sand at 10% and 20% by weight.

The results confirm that the three observed critical state lines collapse to a unique line; therefore, the intergrain concept is applicable to silt sands with a high degree of saturation with low clay content. The results also confirm gassy silty sand (10% and 20% silt by weight) is more susceptible to liquefaction compared to gassy sand.

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