Effect of fines content on instability behavior of loose silt-sands containing gas bubbles

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ABSTRACT
Submarine and offshore slope failures are commonly encountered in silt-sand soils. Investigations on the cause of submarine slope failures have confirmed the presence of gas bubbles, especially during tidal variations have contributed to the liquefaction of silt-sands. However, lab-scale experiments to date have focused on pure sands, both saturated and gassy, and it is these results that are used to simulate field conditions. The research work presented in this paper aims at study the instability behavior of silt-sands containing gas bubbles under undrained monotonic loading. The study confirms loose silt-sands containing gas bubbles at different fines content experience strain softening behavior and posses unique instability curves.

RÉSUMÉ
Sous-marins et ruptures de pente offshore sont couramment rencontrées dans les sols de limon et de sable. Les enquêtes sur la cause des glissements de terrain sous-marins ont confirmé la présence de bulles de gaz, en particulier lors des variations de marée ont contribué à la liquéfaction de la vase-sables. Toutefois, des expériences de laboratoire échelle à ce jour ont porté sur les sables purs, à la fois saturé et gazeux, et ce sont ces résultats qui sont utilisés pour simuler les conditions de terrain. Les travaux de recherche présentés dans cet article vise à étudier le comportement d'instabilité de limon-sables contenant des bulles de gaz sous chargement monotone non drainée. L'étude confirme lâche limon-sables contenant des bulles de gaz à des amendes différentes souche expérience contenu comportement adoucissant et possède des courbes de l'instabilité unique.

1 INTRODUCTION

Instability of a soil mass containing occluded gas bubbles may occur due to a sudden drop of pore pressure under essentially constant total stress (for example, tidal drawdown). One example is the flow slides that have occurred in the Fraser River delta (Chillarige et al. 1997b). Christian et al. (1997) showed the residual pore pressures in these sediments during low-tide conditions leads to instability of the soil matrix and 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. Table 1 provides a list of other flow liquefaction failures related to submarine slope failures. Such failures may have occurred due to the presence of gas under low-tide conditions and the formation of unstable gas-soil-water matrix. Due to a sudden load application, the gassy soil mass could also undergo instantaneous liquefaction; the inability of the gassy/saturated soil element to sustain change of stresses results in large excess pore pressure and large stains are observed.

Leong et al. (2000) and Chu et al. (1993) stated instability as one of the failure mechanisms that lead to flow slides or collapse of granular soil slopes. Lade (1992) indicated that instability is not synonymous with failure, although both may lead to catastrophic events. Lade (1992) observed loose fine sand under undrained conditions becomes unstable even before the stress state reaches failure. Similar observations were made by Chu et al. (1993) - for medium loose to dense sand under strain controlled conditions and Leong et al. (2000) - for loose granular filling material under stress controlled conditions.

Hanzawa et al. (1979) investigated instability line behavior for saturated sands under monotonic undrained triaxial tests conditions. Analysis of these results showed a trend line representing peak shear strength points that passes through the origin. The tests were conducted for specimens with similar void ratio and different effective confining pressures. Similar conclusions have been made by Chu et al. (2002), Leong et al. (2000), and Kramer (1996).

The work by Haththotuwa and Grozic (2008, 2009) showed soil behaviour of saturated and gassy silt sands depends on silt content, drainage conditions and loading path and that these factors are crucial to understanding the resulting behaviour. Haththotuwa and Grozic (2010) showed the presence of gas in silt sand can have an important influence on soil behaviour.

Despite the wide occurrence of silt in submarine soils, liquefaction research is often performed on clean sands with the assumption that the instability behaviour of gassy silty sand (also referred to as silt sands containing gas bubbles) is similar to that of sand. This research work explains how silt sands containing small amounts of gas could experience instability and undergo flow liquefaction at various low silt contents. This study is conducted within the framework of critical state soil mechanics and the experimental program using mixtures of Ottawa sand and Penticton silt.

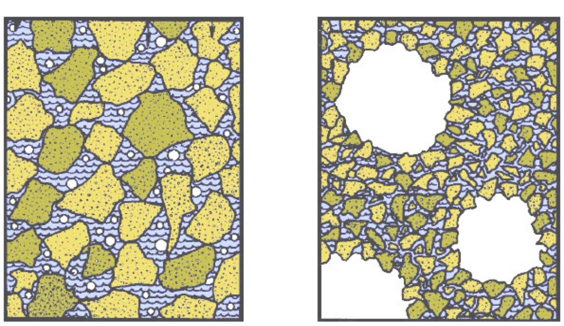
Table 1. Statically induced liquefaction case studies in silty soils (after Chillarige et al. 1997a)

Site	Type of failure	Predominant soil type	Reference
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
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

2 BACKGROUND

2.1 Gassy Soil

Gassy soil is a multi-phasic system which is composed of three phases namely, soil, water and air. These 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 pore water (Figure 1a) or large bubbles of gas in the matrix of a fully saturated soil (Figure 1b).



(a) (b)

Figure 1. Microstructure of gassy soil. (a) when bubble size smaller than pore space and (b) when bubble size is greater than soil particle size.

In this research, it was assumed that the soil contained small bubbles embedded in its matrix.

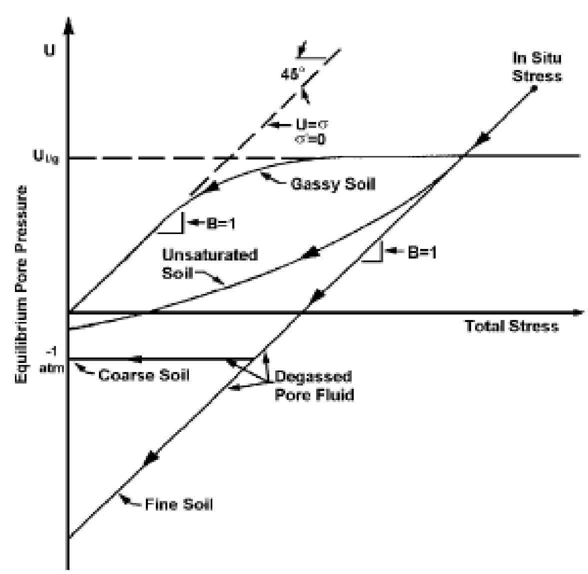


Figure 2. Undrained equilibrium behaviour of an element of soil on unloading: the effect of the amount of gas dissolved in the pore fluid (after Sobkowicz and Morgenstern 1984)

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 2). According to Figure 2, 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 effective stress is reduced to zero and u becomes equal to the total stress). 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 State Boundary Surface

Roscoe et al. (1958) showed the parameters - mean effective stress (p') and the deviator stress (q) can be uniquely related in the region between the critical state and normally consolidated state. When p' and q are linked with e, the curve generated on this three dimensional space are referred to as the critical state boundary surface (Figure 3a).

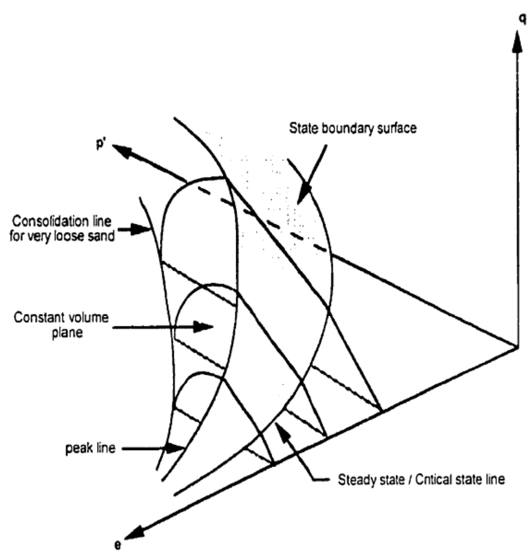


Figure 3a. State Boundary Surface for Loose Sand (after Sasitharan 1994)

The projection of state boundary surface on q-p' space is shown in Figure 3b. Studies by Sladen et al. (1985) showed loose sand undergoes liquefaction once it reaches a maximum peak strength value represented on the boundary surface and is followed by strain softening. Based on this study, Sladen et al. (1985) defined the collapse surface in effective mean normal stress-deviator-void ratio space by connecting the locus of peak failure strengths (also known as peak deviator stress) during undrained-loading response and the corresponding peak normal effective stresses by straight lines. Dry sand experiments conducted by Skopek (1994) showed dry sand will collapse when stress paths hits or crosses the collapse surface. The work by Sasitharan (1994) showed state boundary surface could be defined by post peak portion of undrained stress paths for very loose sands. Similar conclusions were made by Ishihara et al. (1991).

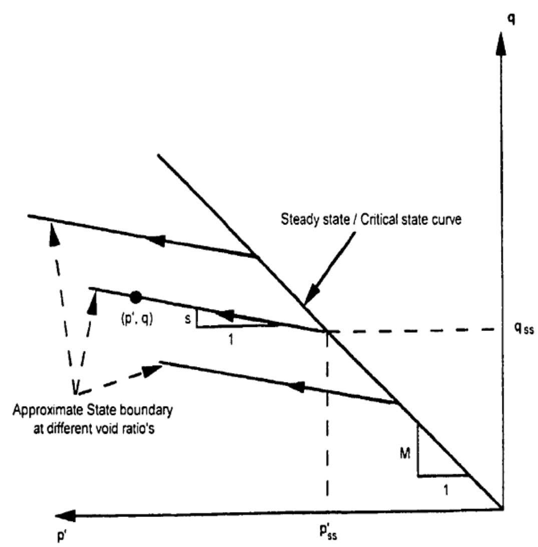


Figure 3b. Projection of state boundary surface on p' - q space for loose sand (after Sasitharan 1994).

2.3 Instability Line

Based on research work by Chu et al. (2002) and Leong et al. (2000) the instability line can be defined as the line that represents peak deviatoric stress points corresponded to undrained stress paths plotted on q-p' space. In addition, the projection of collapse surface on e-q-p' space (Section 2.2) on q-p' space is referred to as the instability line. Unique characteristic feature of the instability line is that it passes through the origin of q-p' plot for sand with no cohesion. Generally, the effective stress path is used for the instability line without any normalization.

2.4 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 4. 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 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

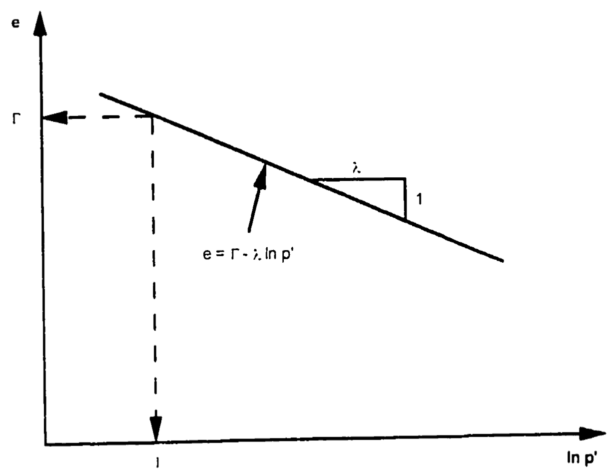


Figure 4. CSL in void ratio – effective mean normal stress space

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 on e-lnp' space.

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 (Haththotuwa and Grozic 2010)

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. Detailed analysis and discussion of Deviator stress vs. Mean normal effective stress (q-p'), Deviator stress vs Axial strain (q-Ea), and Excess pore pressure vs. Axial strain (Δu-Ea) plots for silt-sand mixtures are provided in Haththotuwa and Grozic (2010). In summary, effective stress analysis indicated the effective stress paths for silt-sand mixtures (Silt:Sand = 0% to 20%) with high degree of saturation (i.e. Sr >95%) plummet towards the origin of the q–p′ plane after reaching their respective peak deviatoric stress states, indicating strain softening and flow liquefaction behaviour. A significant drop of peak strength of the gassy silt sands compared to the gassy sand and higher peak strengths for higher effective consolidation pressures was observed.

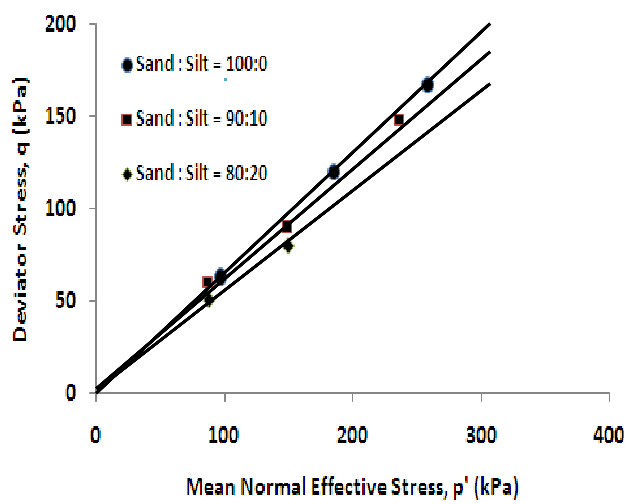


Figure 5. Instability analysis for gassy silt-sand mixtures

Analysis of instability lines (ILs) for gassy silt-sand mixtures are shown in Figure 5. Instability line (IL) analysis for gassy sand and gassy silt-sand mixtures indicates IL for sand posses a higher gradient with that of sand with fines. Most importantly, the inclusion of fines appears to rotate the IL about the origin of q-p' space clockwise direction. The analysis showed the shape of IL for all tests is a straight line with an intercept = 0. The critical state line on q-p' space, on the other hand, indicated inclusion of fines tempts to rotate CSL in anti-clockwise direction. (Figure 6)

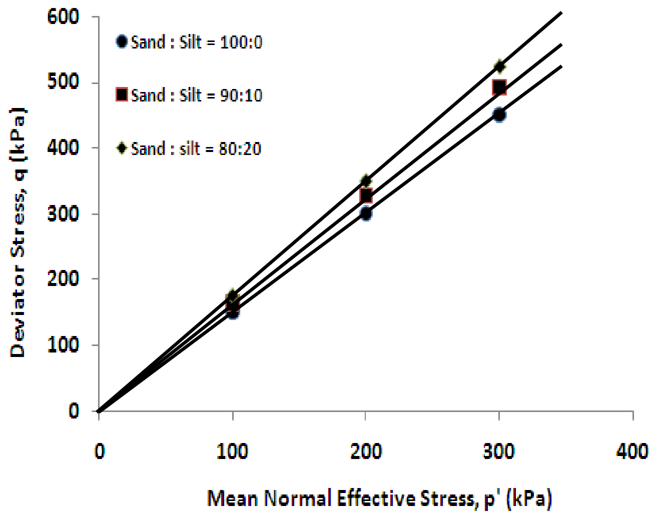


Figure 6. Critical State Line (CSL) behaviour on q-p' space

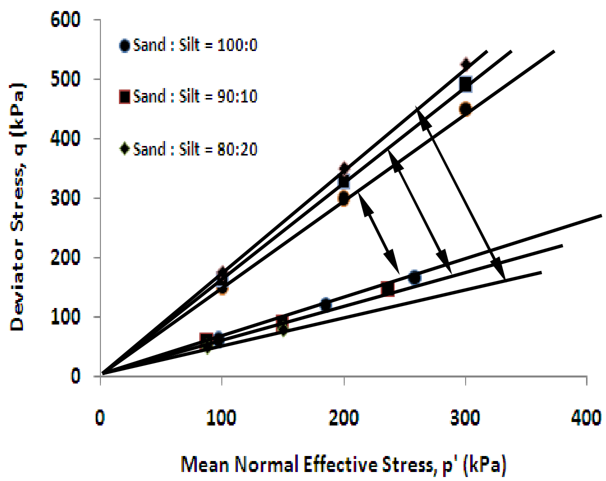


Figure 7. Instability zone analysis for gassy silt-sand mixtures. The zone between critical state line (CSL) and its corresponding instability line (IL) is referred to as instability zone which is marked by a two headed arrow [Note: the upper three lines represent critical state lines (CSLs) and bottom three lines represent instability lines. Gradient of CSL (M) and slope (s) of effective stress path on q-p' space are: 1.3,0.98 - pure sand; 1.66,0.98 - 10% Silt; 1.87,0.98 - 20% Silt. M and s values are determined as explained in Figure 3b]

Potential instability zone is defined as the zone between the critical state line and the instability line. The potential zones identified for the saturated and gassy silt-sand mixtures are presented in Figure 7. [Analysis is based on Haththotuwa and Grozic (2009, 2010)].

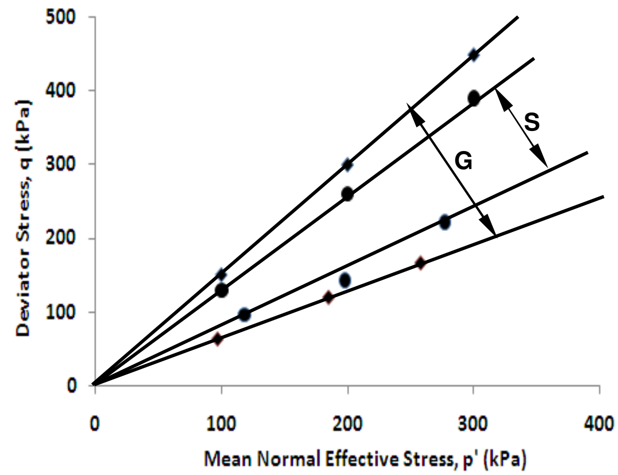


Figure 8. Instability zone analysis for gassy sand and saturated sand (FC = 0%) mixtures. [Note: G: Gassy sands; S: Saturated sands. Upper two lines represent CSLs and bottom two lines represent ILs]

Instability zone analysis for gassy sand and silt-sand mixtures indicated a smaller instability zone for gassy sands, compared to gassy sands with fines. Similar to saturated sand and silt-sand instability zone behaviour, instability analysis for gassy sand and silt-sands observed expansion of instability boundaries when fines content is increased. The comparison of instability zones for pure saturated sand and gassy sand are presented in Figure 8. The comparison of instability zones showed a larger instability zone for gassy sand and silt-sand soils, compared to saturated sand and silt-sand soils respectively. For example, Figure 8 shows how the effect of gas content on the instability zone for pure saturated sand and gassy sand (i.e Fine content = 0%). Similar behaviour (i.e. the expansion of instability zone due to gas effect) was observed for saturated and gassy silt-sand specimens with fine content in the range: 10% - 20% (Haththotuwa 2011).

5 CONCLUSIONS

This paper presents instability behaviour of gassy silt-sand mixtures with a tracer amount of plastic fines (10% and 20% of silt by weight), based on research work described in Haththotuwa and Grozic (2010) and Haththotuwa (2011).

Haththotuwa and Grozic (2010) confirmed the gassy loose silt-mixtures with a degree of saturation over 95% experience flow liquefaction and strain softening behaviour. The analysis of instability lines for gassy silt-sand mixtures indicated unique lines for each material mixture, where increased fine content tended to rotate the instability line above the origin in clock wise direction and critical state line in anti-clockwise direction. For the given test conditions (i.e. when confining pressure is kept high and constant for all tests), this study confirmed the instability line manifested as a straight line passing through the origin. The results confirmed the instability zone expanded when fine content was increased from 0% to 20%, given that the silt-sand mixtures very loose and the degree of saturation is above 95%. In addition, study confirmed the instability zone for saturated silt sands expands when low gas content (i.e. degree of saturation is greater than 95%) is introduced to the soil matrix

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