Behaviour of loose silty sands

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ABSTRACT
Numerous soil failures have been documented in silty sands, particularly in coastal and offshore soils. This paper draws upon critical state framework to characterize the effect of silt on the response of loose silt-sand mixtures. The result of triaxial test using Ottawa sand and Penticton silt mixtures are presented and analyses indicates that the steady state line is particularly sensitive to silt contents.

RÉSUMÉ
De nombreuses ruptures de sol ont été documentées pour les sables vaseux, en particulier pour les zones côtières et au large. Ce papier s'appuie sur le concept d'état critique des sols afin de caractériser l'effet du limon sur la réponse des mélanges limon-sable meubles. Les résultats de tests de cisaillement triaxial utilisant du sable d'Ottawa et des mélanges limoneux de Penticton sont présentés. Les analyses indiquent que l'état d'équilibre dépend particulièrement de la teneur en limon.

1 INTRODUCTION

The presence of silt can have an important role on soil behaviour. When soils contains silt, referred to as silty soils, the silt content combined with the drainage conditions and loading path are crucial to understanding the resulting behaviour.

Near and offshore coastal areas, marine deltaic sediments are usually a mix of silt and sands. These areas are prone to submarine flow slides and indeed numerous soil failures have occurred in silty sands as described by Yamamura and Lade (1999 - Table 1) and Chillarige et al. (1997 - Table 2). Despite the wide occurrence of silt in these soils, liquefaction research is often performed on clean sands with the assumption that silty sand behaviour is similar to that of sand. The aim of the research presented here is to evaluate the undrained behaviour of plastic silty sands within the framework provided by critical-state soil mechanics.

In order to capture the critical state behaviour of silty sands, an experimental program has been carried out using mixtures Ottawa sand and Penticton Silt. The results of the experimental program, described in this paper are the background requirements for further testing on gassy silt-sand mixtures.

2 BACKGROUND

2.1 Silty Sand

As explained in the inter grain concept proposed by Thevanayagam et al. (2002), non plastic silts can be categorized into two groups as sand dominated behavior and fines dominated behavior. The fines within a sand–silt mixture are regarded as voids when the fines content is low because the fines do not participate in the resistance of shearing (sand dominated behavior – Figure 1). The sand particle can be considered equivalent to a void when the fines content is high, because the sand grains may not contribute to the shearing resistance (fines dominated behavior – Figure 2).

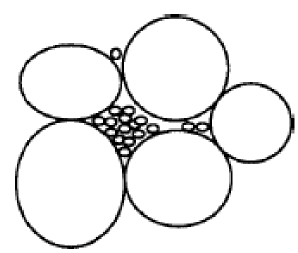

Figure 1. Sand dominated behavior (Yang et al. 2006)

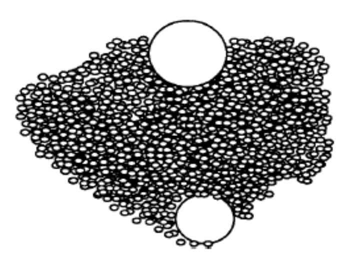

Figure 2. Fine dominated behavior (Yang et al. 2006)

Table 1. Statically induced liquefaction case studies in silty soils (Yamamura and Lade 1998; 1999)

Site	Type of failure	Predominant soil type	Reference
Trondheim harbor, 1888	Natural submarine slope	Silty sand	Andresen and Bjerrum 1968
Orkdalsfjord, 1930	Natural submarine slope	Non-plastic silt	Andresen and Bjerrum 1968
Helsinki Harbor, 1936	Natural submarine slope	Silty sand	Andresen and Bjerrum 1968
Fort Peck Dam, 1938	Hydraulic fill earth dam	Silty sand	Turnbull and Mansur 1973
Finnvika, 1940	Man-made submerged fill	Fine sand and silts	Bjerrum 1971
Follafjorden, 1952	Natural submarine slope	Fine sand and silts	Bjerrum 1971
Trondheim Harbor, 1950	Natural submarine slope	Silty sand	Bjerrum 1971
Nerlerk Berm, 1982	Man-made submerged fill	Silty sand	Sladen et al. 1985
Puget Sound, 1985	Submarine slope	Silty sand	Kraft et al. 1993
Fraser River, 1970-1986	Natural submarine slope	Silty sand	McKenna et al. 1992
Lade, Trondheim,1990	Natural submarine slope	Silty sand	Emdal and Janbu 1996

Table 2. Statically induced liquefaction case studies in silty soils (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

2.2 Critical State Line (CSL)

The critical state of a soil refers to a condition where the soil is continuously deforming at a constant volume, a constant normal effective stress, a constant shear stress, and a constant velocity (Poulos 1981). Been and Jefferies (1985) introduced a critical state framework for sand by introducing the state parameter, ψ. This parameter is a measure of how far the current void ratio is from the void ratio at critical state given by $\psi = e_{CS} - e$; where e_{CS} is the void ratio at critical state. Based on critical state data of void ratio and mean stress, Been and Jefferies (1985) proposed the following equation for CSL of sand: $e_{CS} = \Gamma - \lambda \ln(p')$ (Figure 3). Experimental observations show that the critical state line changes from low slope to higher slope; and from low stress level to higher stress level due to grain crushing effects at higher stress levels (Sasitharan 1994).

Poulos et al. (1985) carried out monotonic triaxial experiments on silty sand and showed that the slope of the CSL is affected by soil gradation and grain angularity. Minor changes in gradation could result in significant changes in the slope of the CSL. Similarly, Olson et al.

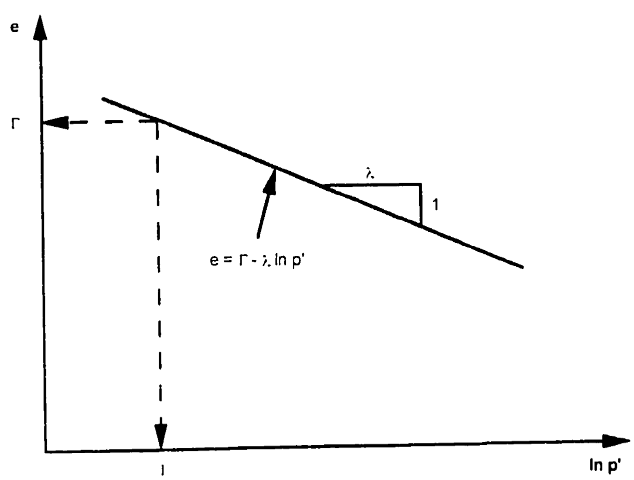

Figure 3. Projection of CSL in void ratio – effective mean normal stress space

(2000) stated that the grain angularity may affect the slope of the CSL more significantly than the fines content. Also, based on drained experiments carried out on silty sand, Yang et al. (2006) showed that fines content has influence on the position of CSL, but no influence on the slope of the CSLs.

The available research results can be divided into two groups, depending on the positions of the CSLs of sand–silt mixtures. In group 1, the slopes of CSLs change with fines content (Been and Jefferies 1985 – Figure 4), whereas in group 2, the CSLs are observed more or less parallel at certain stress levels (Zlatovic and Ishihara 1995; Yang et al. 2006 – Figure 5).

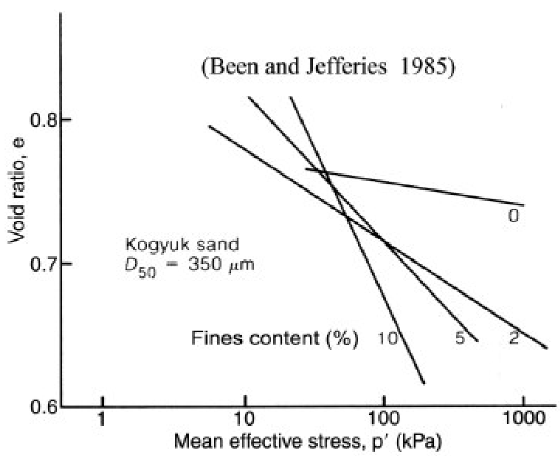

Figure 4. Critical state lines for sands with fines contents ranging from 0% to 10%. Note: D_{50}, is the mean grain size of the coarser grains. (After Been and Jefferies 1985)

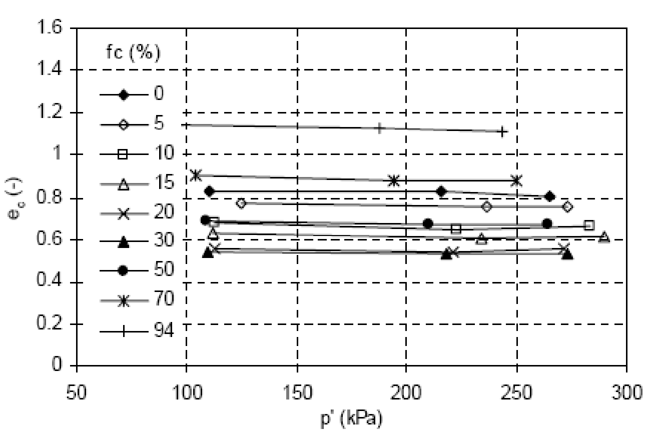

Figure 5. Critical state lines for sand with fines content from 0 to 100% (Yang et al. 2006)

The slopes of the CSLs of sand–silt mixtures are also controlled by silt plasticity. In literature, it is noted that Bouckovalas et al. (2003), Zlatovic and Ishihara (1995), Thevanayagam et al. (2002; 1997) and Yang et al. (2006) used nonplastic silts in their experiments. However, plasticity information has been not supported with silts used by Naeini and Baziar (2004) and Been and Jefferies (1985).

Bobei et al. (2009) carried out a research to investigate the static liquefaction behaviour of sand with a small amount of fines. Based on results, and similar to results obtained by Yang et al. (2006), 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, a round to sub-rounded quartz, and Penticton silt were used in the experimental program. Ottawa sand has a specific gravity of 2.65 and is 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 is noted that Ottawa sand has 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 is modified from an unsaturated stress path triaxial system. A double walled cell construction enables precise specimen volume change measurements. The cell pressure capabilities are 2000 kPa, higher than a conventional system. The system is servo controlled and capable of stress path or cyclic testing.

3.1.3 Specimen Preparation and Testing Procedures

Reconstituted specimens (100% Ottawa sand; 90% Ottawa Sand and 10% Silt; 80% Ottawa Sand and 20% Silt) were prepared using the moist tamping method. This technique consists of placing moist soil layers into a mold 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 is introduced to the specimen bottom port and collected from the specimen top. To ensure complete saturation, 2 to 3 times the pore volume of water is allowed to pass through the specimen. Cell and back pressures were then slowly increased to 800 kPa and 750 kPa, respectively. At this point, a B - test was performed to check saturation; B-values of 0.98 and greater were obtained. After the B-test, consolidation was induced by increasing the pressures 850 kPa, 950kPa, or 1050kPa, while maintaining a constant back pressure of 750 kPa.

Consolidation took approximately one day after which pore and cell pressures were dropped to (ramping down stage) 400kPa, 500kPa, or 600kPa and 700 kPa, while maintaining a constant effective stress. The objective of the ramping down is to reproduce the same testing procedures that will be used for gassy soil testing. All valves to and from the specimen were then closed, thus 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 differently. Pore pressure, cell pressure, and axial and volumetric deformations were measured.

The research work presented in this paper includes only triaxial tests on saturated sand and silty sands. Future research work will involve triaxial tests on gassy sand and gassy silty sand under same stress path conditions explained above.

4 LABORATORY RESULTS AND DISCUSSION

A summary of experimental results is presented in Table 3. The test results of isotropically consolidated undrained tests for saturated Ottawa sand are shown in Figure 6, with shearing commencing from a mean normal effective stress (p') in the range of 100 to 300 kPa.

The effective stress paths for saturated Ottawa sand (Figure 6) plummet towards the origin of the q–p' plane after reaching their respective peak deviatoric stress states, thus indicating strain softening and flow liquefaction behaviour. Using the same test conditions, effective stress path results for silty sand (Silt:Sand = 20:80 and 10:90) are presented in Figures 7 and 8.

Table 3. Summary of Test Results.

Sample No.	**Silt content (%)**	**Effective Stress (kPa)**	**Void Ratio (end of test)**	**P'_{peak} (kPa)**	**q_{peak} (kPa)**	**$P'_{steady\ state}$ (kPa)**	**$q_{steady\ state}$ (kPa)**
S1	0 (pure sand)	103	0.720	100	115	21	36
S2		205	0.717	205	138	2	6
S3		304	0.694	277	222	26	38
C11	10	100	0.702	84	55	3	3
C12		205	0.654	150	115	14	25
C13		305	0.673	220	169	34	55
C21	20	109	0.632	71	44	3	11
C22		205	0.602	143	93	3	11
C23		305	0.603	194	120	5	18

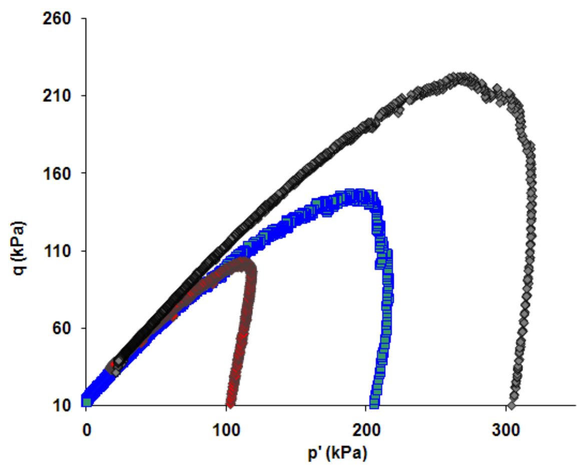

Figure 6. Effective stress paths (q–p') for saturated Ottawa sand

Analysis of Figures 6 to 8 shows a significant drop of peak strength of the silty sand compared to the clean sand. The strength reduction is the greatest at the lowest initial confining pressure of 100 kPa, with similar percentage strength reductions observed for the 10% and 20% silt contents, when compared with pure sand. The drop of peak strength could be due to the high compressibility of silt sands, compared to pure Ottawa sand. It is also noted that the shape of the effective stress path becomes more rounded when silt is present within the specimen.

The deviator stress versus axial strain, q–ε_a, curves of C11 to C13 and C21 to C23 are shown in Figures 9 and 10, respectively. All the stress–strain curves initially show a similar response: 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.

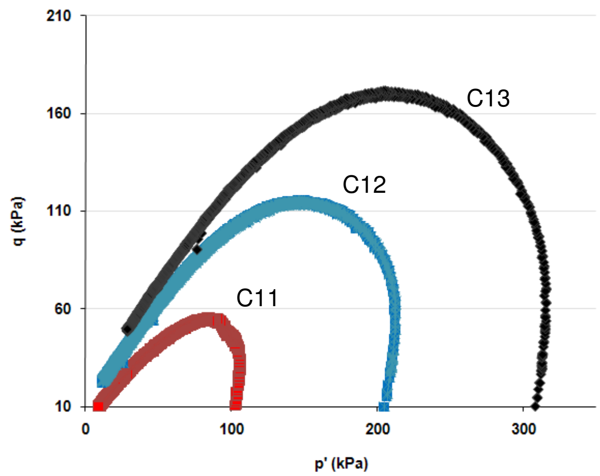

Figure 7. Effective stress paths for silty sand (90%- Sand; 10%- Silt).

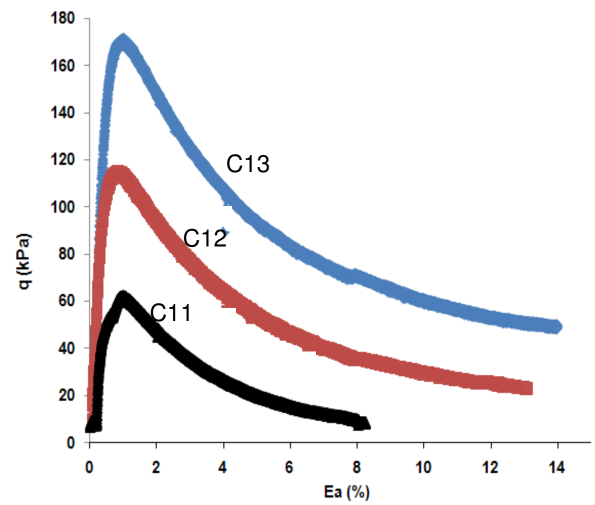

Figure 9. Stress strain curves (q-ε_a) for silty sand (90%- Sand; 10% - Silt)

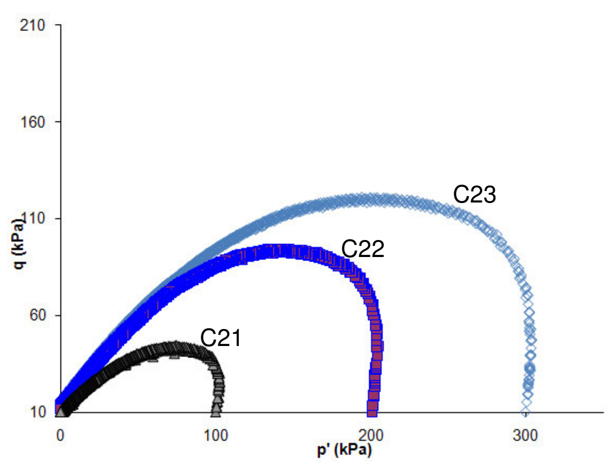

Figure 8. Effective stress paths for silty sand (80%- Sand; 20%- Silt).

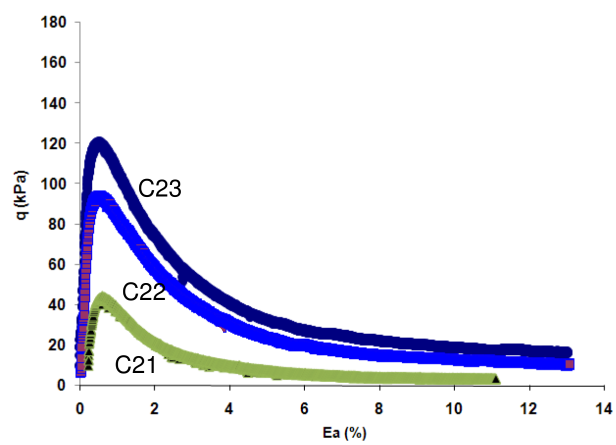

Figure 10. Stress strain plots (q-ε_a) for silty sand (80%- Sand; 20% - Silt).

The pore-water pressure of 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 11. Similar behaviour with high pore water pressure values were observed on tests C11 to C33.

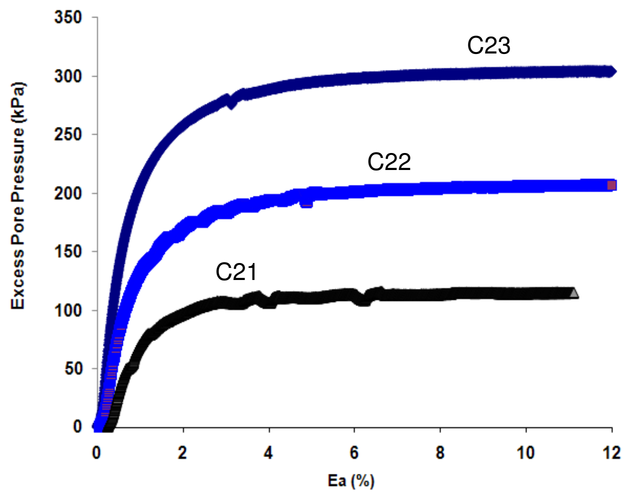

Figure 11. Pore pressure versus axial strain plot for silty sand (80%- Sand; 20% - Silt)

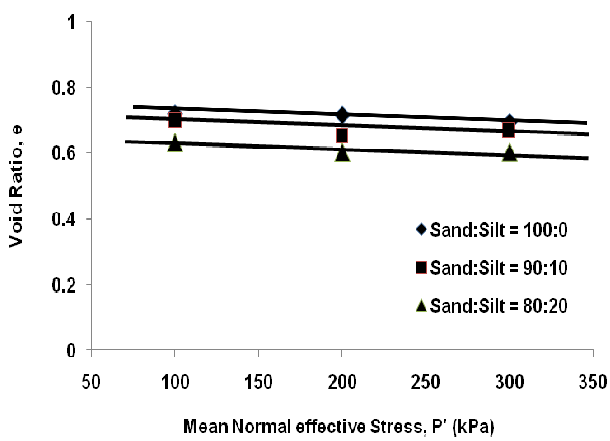

Figure 12. Void ratio versus mean effective stress plot

The void ratio versus mean effective stress for the end of test conditions for the saturated Ottawa sand and saturated silty sand tests for 10% silt and 20% silt by weight are plotted in Figure 12. This chart enables a detailed examination of the uniqueness of the critical state and steady state for sand and sand with fines. According to Figure 12, for pure sand, a linear trend line is observed. Based on published literature, e-p' curve should represent a single straight line, except when p' drops below 100kPa. (Poulos et al. 1985; Been et al. 1991; Ishihara 1993, Chu 1995, Verdugo and Ishihara 1996; Riemer and Seed 1997; Li and Wang 1998; Yoshimine and Ishihara 1998). The results of this research indicate the trend line obtained for pure sand possess a shape similar to sand with fines. More specifically, 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 in 10% and 20% by weight respectively.

Further analysis of data shows trend lines on e-p' space (Figure 12) can be represented by a unique critical state line (Figure 13), based on intergrain state concept explained by Thevanayagam et al. (2002). Although Thevanayagam's concept is applicable for non-plastic silts, the results of tests indicate silts with less than 10% clay content show similar behaviour. Results confirm the fine content has an influence on the position of the critical state line but without any influence on the slope of the line and that when corrected for fines content, the CSL collapses back to a unique line.

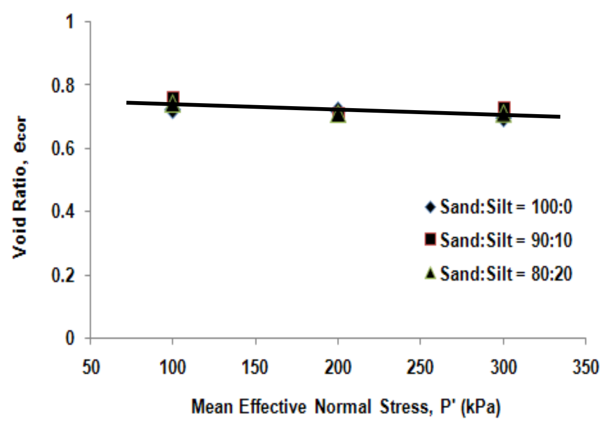

Figure 13. e_{cor} - p' plot (e_{cor} is the modified void ratio, based on intergrain state concept)

5 CONCLUSIONS

This paper presents sets of experimental results on the behaviour of sand with a small amount of plastic fines (10% and 20% of silt by weight). The equivalence and uniqueness of critical state and steady state lines for void ratio versus mean effective stress for the end of test conditions for the saturated Ottawa sand and saturated silty sand tests for 10% silt and 20% silt by weight are investigated; thus enabling a detailed examination of the uniqueness of the critical state and steady state for sand and sand with fines. For pure sand, a linear trend line is observed. The results of this research indicate the trend line obtained for pure sand possess a shape similar to sand with fines. More specifically, 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 in 10% and 20% by weight respectively. Using the intergrain state concept of correcting the void ratio for fines content, it was confirmed that the three observed critical state lines collapse to a unique line.

The results also confirm silty sand (Silts: 10% and 20% by weight) is more susceptible to liquefaction, compared to pure sand.

Further research on the behaviour of silty sands is planned, where the effect of gas content on the behaviour, specifically the slope and position of the critical state line will be examined.

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