A MODEL TO PREDICT THE UNDRAINED BEHAVIOUR OF LOOSE GASSY SAND

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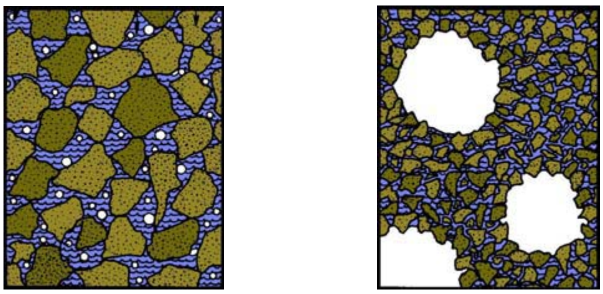
ABSTRACT:
Submarine slopes in deltaic environments often consist of loose sand containing gas bubbles within the soil voids. It has been speculated that the presence of gas bubbles is one of the causes of submarine slope instability problems. The presence of gas bubbles will affect the slope stability in three ways: (1) evolving gas bubbles generate excess pore pressure and cause the soil to be in a state of under consolidation, (2) even during undrained loading, a condition of partial drainage will prevail due to the presence of occluded bubbles, (3) pore fluid compressibility is increased due to the presence of free gas. A strain-softening model for loose sand has been chosen and modified to account for the partial drainage, under consolidation and the compressibility effect due to presence of gas bubbles, through idealising gassy soil as saturated soil with compressible pore fluid. Model predictions indicate that both degree of saturation (or gas content) and degree of under consolidation (or excess pore pressure) influence the undrained shear strength of gassy soil.

RESUME:
Les pentes sous-marines en les environs deltas souvent consistent de la sable délié qui contient des boules de gaz entre les vides du sol. La présence des boules de gaz est peut-être un cause des problèmes d'instabilité des pentes sous-marines. La présence de ces boules va affecter la stabilité des pentes par trois manières: (1) boules de gaz changeantes produisent des pressions interstitielles en excès et cause le sol d'être dans un état de consolidation. (2) même dans le chargement non-drainé, un condition partiellement drainé va être maintenir au cause de la présence des boules bouchées, (3) la compressibilité des fluides interstitiels est augmenté par la présence du gaz libre. Un modèle de amollissement des déformations pour la sable délié était choisi et modifié pour rendre compte du drainage partiel, sous-consolidation et les éffets de la compressibilité selon la présence des boules de gaz, par idéalistion du sol gazeux comme un sol avec les fluides interstitiels compressible. Les prédictions du modèle indiquent que le degré de la saturation (ou le contenu du gaz) et le degré de sous-consolidation (ou la pression interstitielle en excès) influencent la résistance au cisaillement non-drainé du sol gazeux.

1. INTRODUCTION

Gas found in marine sediments and in the oil sands and their associated tailings ponds has been reported to alter the pore pressure response and liquefaction potential of these soils. Gas, mostly methane, exists in these environments as bubbles either in occluded form within pore voids (Type C soil, see Figure 1a) or as large cavities spanning soil skeleton (Type D soil, see Figure 1b). While Type C soil is common in deltaic environment (eg. Chillarige et al. 1997) and oil sand tailings pond (Fourie et al. 2001) due to biogenic activity, Type D soil exists predominantly in fine grained soils, most commonly found in deep sea environments.

Though the presence of gas bubbles has been reported and suggested as one of the reason for flow liquefaction failure of submarine slopes, its exact role on liquefaction failure is not yet clearly understood. The presence of occluded gas bubbles in deltaic slopes has been attributed to triggering of flow failure in Fraser River Delta, British Columbia (Chillarige et al. 1997), Missippi River Delta, California (Whelan et al. 1977) and Klamath River Delta, California (Field 1991). In this environment, organic rich sediments and an anaerobic environment favour bacterial activity. Decomposition of gas hydrate due to post global warming and sea floor lowering that results in gassy soil in deep sea bed has been linked to several deep marine slope failures like Cape Fear land slide, North Carolina (Carpenter 1981), Storregga submarine slump, Norwegian continental margin (Bugge et al. 1987) and Humboklt submarine slide (Field and Barber 1991).



(a) (b)

Figure 1. Gassy soil models (a) In coarse grained soil in occluded form and (b) in fine grained soil spanning soil skeleton (Wheeler 1989). Pietruszczak and Pande (1996) referred to this as Type (C) and Type (D) soil, respectively.

Bubble nucleation and growth introduces excess pore pressure in the pore fluid and increases the compressibility of the soil matrix. Numerous researchers have found excess pore pressure in marine environments, which are believed to be associated with gas bubbles (eg. Erig and Kirby 1997; Lee et al. 1991; Field 1991). This excess pore pressure may be due to a high sedimentation rate compared to the rate of consolidation in deltaic environments (Terzaghi 1956) or due to bubble growth that exerts pressure in surrounding soil (Lee et al. 1991; Field 1991) or due to impervious gas hydrate layer that hinders dissipation of excess pore pressure generated upon sea level change (Kayen and Lee 1991). Coleman et al. (1991) defined this type of soil as being in a 'state of under consolidation'. The presence of gas bubbles will also increase the compressibility of the pore fluid, creating a partial drainage condition, even though fluid does not enter or exit the system. Partial drainage results in volume change as well as pore pressure development.

Most of the earlier works on the behaviour of gassy soil focused on the compressibility of gassy soil. Dessault (1979) explained this phenomenon for gassy soil using double spring analogy (Figure 2). The behaviour of pore gas, both free gas and dissolved gas in accordance with Boyle's law and Henry's law, has been shown to affect the compressibility of gas-water mixture (Dessault 1979; Schuurman 1992). Sobkowikz (1982) expressed the change in pore fluid pressure in a quadratic form, assuming equilibrium behaviour and pore gas pressure equal to pore water pressure (Equation 4).

Several numerical models to predict the undrained behaviour of gassy soil have been carried out by various researchers. Wheeler (1988) analysed Type D soil as gas cavities surrounded by saturated soil matrix and provided an upper and lower limit for undrained shear strength. Pietruszczak and Pande (1996) modified an elasto-plastic deviatoric hardening model to predict undrained behaviour of medium dense gassy sand, neglecting the solubility effect of gas. The model employed the specific surface area of soil or average pore size as a parameter and hence predicted different undrained response for fine and coarse sand for the same degree of saturation.

Atigh and Byrne (2003) modified an elasto-plastic model to incorporate the effect of gassy soil by considering gassy soil response in undrained condition as equivalent to saturated soil response in partially drained condition. Accordingly, the undrained loading of gassy sample that results in contractive volumetric strain was considered as outflow state in partially drained condition. However, the elasto-plastic model did not predict strain softening behaviour which has been reported in submarine slopes such as in Fraser River Delta (Chillarige et al. 1987).

Grozic (1999) carried out an extensive experimental work on very loose gassy Ottawa sand. Samples were prepared by moist tamping technique resulting in void ratio higher than the maximum void ratio obtained from ASTM (American Society for Testing of Materials) method. In situ sampling

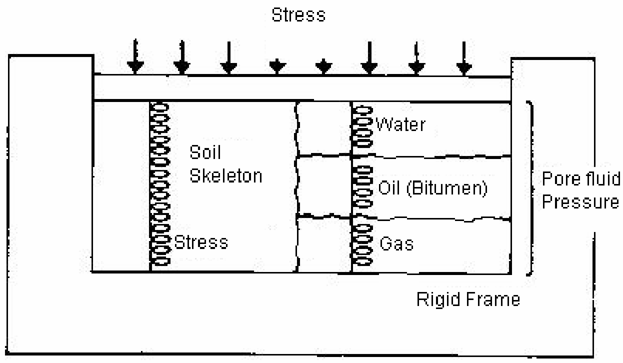


Figure 2. Double spring analogy proposed by Dessault (1979) for soil with compressible pore fluid.

during the Canadian Liquefaction Experiment Project (CANLEX) noted in situ void ratios higher than the maximum void ratio obtained from ASTM method and thus consistent with samples prepared using the moist tamping technique. This is of particular interest in this study, as submarine deltaic environment consists of loose sand (Chillarige et al. 1997). Grozic (1999) concluded that loose gassy sand with degree of saturation greater than 80% shows a strain softening response for undrained loading, which is an essential criterion for flow liquefaction failure to occur. Grozic (1999) modified a strain softening model developed for loose sand by Imam (1999) to incorporate effect of gas bubbles. Though it captured the general trend, deviation of predicted response from experiment at initial stages of loading was noted.

The proposed model described in this paper incorporates the effects of compressibility and excess pore pressure by coupling volume change due to partial drainage with the model developed by Imam (1999) for saturated loose sand.

2. THEORETICAL BACKGROUND

Two types of approaches are possible for analysis of gassy soil. One is to consider gassy soil as 'spherical cavities or gas voids surrounded by saturated soil matrix' (Wheeler 1988a) and the other way is to consider it as 'saturated soil matrix with compressible pore fluid' (Thomas 1989). While the first approach is appropriate for analysis of fine-grained soil with gas bubbles, the second approach will be appropriate for analysis of granular soils with occluded bubbles within voids. The first approach considers gassy soil behaviour in terms of cavity expansion/contraction and bubble flooding (Wheeler 1988b). For gassy sand with occluded bubbles contained within the voids, the second approach is considered more appropriate. Fredlund and Rahardjo (1993) argued that for degree of saturation higher than 85%, where free gas exists in occluded form, Terzaghi's principle of effective stress can be applied; in other words, surface tension effect can be neglected. Erig and Kirby (1997) adopted the same approach to analyse gassy soil in their early work.

A strain softening model proposed by Imam (1999) for loose sand has been chosen as the base model to incorporate effect of gas bubbles with degree of saturation greater than 85%. Gassy soil has been idealised as saturated soil with compressible pore fluid. A brief description of Imam's model for loose sand is presented in the following section.

2.1 Imam's model

Based on Lade's (1992) observation that the peak point of undrained effective stress path (P-UESP) of loose sand is very close to the peak point of yield surface (P-YS), Imam (1999) proposed the following expression for yield surface of an isotropically consolidated sand:

$$\eta^2 = 5M_p^2\left[1-\left(\frac{p}{p_c}\right)^2\right]^{\frac{1}{2}} \qquad [1]$$

where,
$\eta = p/q, p = (\sigma_1 + 2\sigma_3)/3$ and $q = (\sigma_1 - \sigma_3)$ in triaxial space. M_p is the stress ratio at peak point of UESP and p_c is the isotropic consolidation pressure for current state. M_p and p_c dictates shape and size hardening respectively. M_p is a function of mobilised friction angle and p_c can be related to plastic shear modulus.

Rowe's stress-dilatancy model modified for triaxial stress space by Wood (1990) has been used to find plastic shear strain increments (Equation 3).

$$\frac{d\varepsilon_q}{d\varepsilon_p} = \frac{9(M-\eta)}{3+9M-2M\eta} \qquad [2]$$

Where $d\varepsilon_p = d\varepsilon_1 + 2d\varepsilon_3$ and $d\varepsilon_q = 2/3(d\varepsilon_1 - d\varepsilon_3)$; $d\varepsilon_1$ and $d\varepsilon_3$ are major and minor strain increments. M is the stress ratio at phase transformation (from contractive tendency to dilative tendency). Evidently, Imam's model employs a non-associated flow rule for loose sand. A critical state framework for sand, proposed by Been and Jefferies (1985) has been utilised in this model by introducing state parameter, ψ, expressed as the difference between current void ratio and critical void ratio or steady state void ratio ($\psi = e - e_{SS}$). This parameter tells how far the current void ratio is from the void ratio at constant volume shearing or at steady state. The model assumes no shape hardening (i.e., M_p remains constant throughout shearing) and size hardening is controlled by plastic shear modulus.

Figure 3 shows the shape of the yield surface of loose Ottawa sand for isotropic consolidation pressure of 300 kPa and void ratio of 0.9, in triaxial compression predicted from

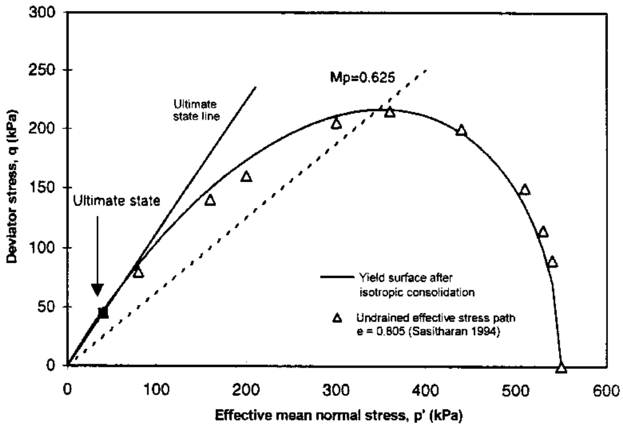


Figure 3. Shape of the yield surface compared with undrained effective stress path (UESP) of Ottawa sand for isotropic consolidation pressure of 550 kPa and void ratio of 0.805 (Imam 1999).

Imam's model. It agrees with experimental observations and other proposed models in that, at low stress level, yield surface is an ascending 'cap' surface and once the peak stress ratio (M_p) is reached, it start to decline, as denoted by 'front portion' in other models.

2.2 Gassy soil modification

The effect of occluded gas bubbles on undrained response of loose sand can be summarised as:

(a) Increasing the compressibility of pore fluid (Dessault 1979)

(b) Generating excess pore pressure and thereby causing the sediment to be in a state of under-consolidation (Coleman et al. 1991).

The combined effect of (a) and (b) is to create a partial drainage condition. Pore pressure build up and effective stress change upon undrained loading are function of the relative compressibilities of pore fluid and soil skeleton (Bishop 1984). The compressibility of the gas-water mixture can be expressed as:

$$\beta_f = \frac{(1 - S + SH)}{p_0} + \beta_W S \qquad [3]$$

As the compressibility of gas-water mixture is increased, pore pressure response and volume change occurs during undrained loading, creating a 'partial drainage' condition. Rad et al. (1994) suggested partial drainage and hence 'less intense' pore pressure reduction during dilative tendency of dense sand as the reason for reduced undrained shear strength of dense gassy sand. On the other hand, partial drainage can be expected to cause an increase in the shear strength of loose sand due to less intense pore pressure increase during contractive tendency of loose sand (Erig and Kirby, 1997).

Partial drainage can be modelled by incorporating the pore pressure change expressed by Sobkowikz (1982), which has been modified for effective stress increment ($\Delta\sigma_1$) as the independent variable, as:

$$A\Delta u^2 + B\Delta u + C = 0 \qquad [4]$$

where,

$$A = nS\beta_W$$
$$B = n\left(p_0 S\beta_L + 1 - S + SH\right) - \beta_T \Delta\sigma' \quad \text{and}$$
$$C = -p_0 \Delta\sigma' \beta_T$$

and the volume change with respect to total volume, expressed by Hilf (1948) and modified by Grozic (1999) as:

$$\frac{\Delta V_V}{V_0} = \left(\frac{\Delta p_0}{p_0 + \Delta p_0}\right)(1 - S + HS)n \cdot \qquad [5]$$

In addition to increasing the compressibility of pore fluid, the presence of gas bubbles also shifts the steady state line (SSL) up in e-log (p) space (Grozic 2000). This will be discussed in the latter part of this paper.

3. RESULTS

The above model has been analysed for loose gassy Ottawa sand with carbon dioxide as the guest gas and compared with the experimental result reported by Grozic (2000).

Table 1. Model parameters used in this study for Ottawa sand (Grozic 1999).

Parameter Type	Parameter	Value
Peak state	k_p	1.8
	e_μ	0.8
	ϕ_{CV}	32
Stress-dilatancy	k_{PT}	1
Failure	k_f	0.75
Compression	β	0.02
Elastic	G_r	500
	K_r	1000
	n	0.55

Table 1 shows the model parameters used in this study. In addition to these material parameters, degree of saturation, initial void ratio, initial mean effective stress and initial pore pressure should be inputted for this model.

Figure 4 shows the undrained response of loose sand for different initial degrees of saturation, but for the same initial pore pressure of 470 kPa. This figure illustrates the deviation of the gassy soil response from the saturated sand at initial stages of shearing and their subsequent convergence to same response at the ultimate state. This initial deviation can be attributed to differing compressibility values that result in varying volumetric strain. Kvalstad et al. (2003) observed the same trend in both Laponite and gassy clay samples. By analysing this result, it could be concluded that the presence of occluded gas bubbles within voids will increase the undrained shear strength of loose sand. Earlier, Erig and Kirby (1977) similarly concluded that gas-laden sediments would have the effect of stabilizing the marine slopes.

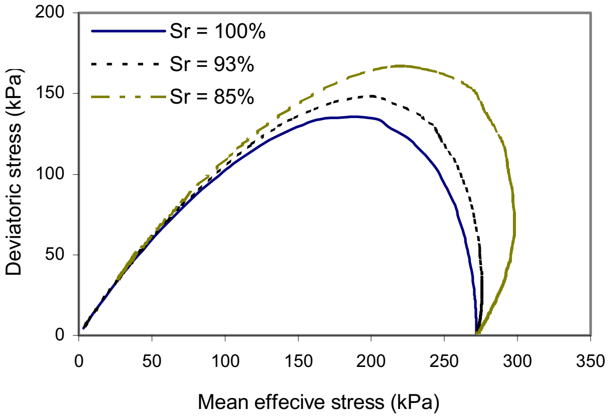


Figure 4. Predicted undrained response of Loose Ottawa sand for pore pressure of 470 kPa and varying degree of saturation.

However, care should be taken in drawing this conclusion, as excess pore pressure also influences the behaviour of gassy soil. Figure 5 shows the predicted undrained behaviour of three gassy soils with 90% initial degrees of saturation, but for different initial pore pressures. A reduction in undrained shear strength can be observed for higher initial pore pressure because the compressibility of the liquid-gas mixture is reduced, since compressibility of gas-water mixture is inversely proportional to absolute gas pressure (Equation 3).

Experimental result reported by Grozic (1999) have been chosen to validate the model performance. Researchers have applied various methods to mimic the field condition of bubble growth: for example, Zeolite technique (Wheeler 1998), culturing selective micro-organisms (Sills & Gonzalez 2000) and by reducing back pressure of gas saturated pore

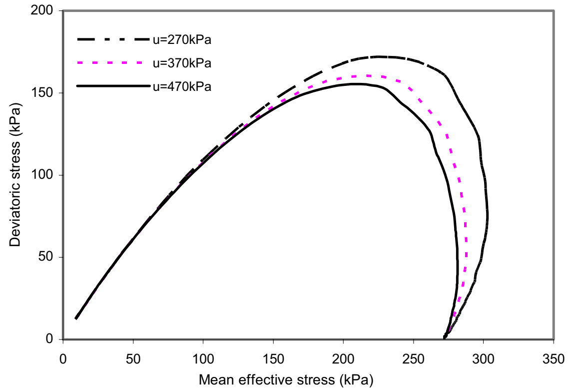


Figure 5. Predicted undrained response of loose Ottawa sand for initial degree of saturation of 90% and varying initial pore pressure level.

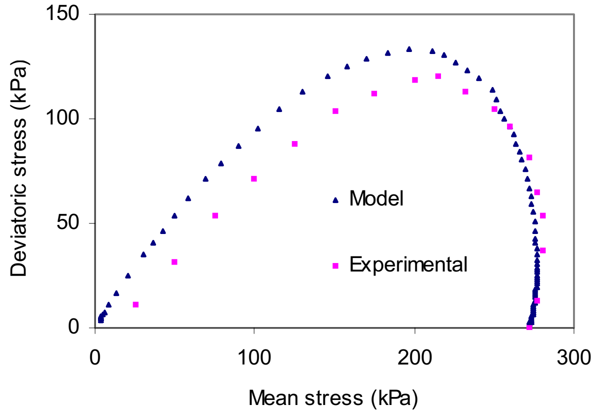
fluid (Rad et al. 1994). Grozic (1999) prepared loose gassy sand by reducing backpressure of the sample below liquid-gas saturation pressure of Carbon Dioxide - water mixture, thereby allowing gas bubbles to form within the soil matrix. In this case, excess pore pressure won't be introduced since bubble growth occurs under 'drained' condition. In the field, bubble growth and excess pore pressure are inter related, causing sediments to be under consolidated.

Figure 6 illustrates the model prediction compared with experimental result reported by Grozic (1999). The trend of an increase in mean normal effective stress, $p^{'}$ with deviatoric stress, q at small strain level, as observed by Grozic (1999), has been captured from this model (Figure 6a) by coupling volumetric strain introduced due to compression and diffusion/exsolution of free gas.

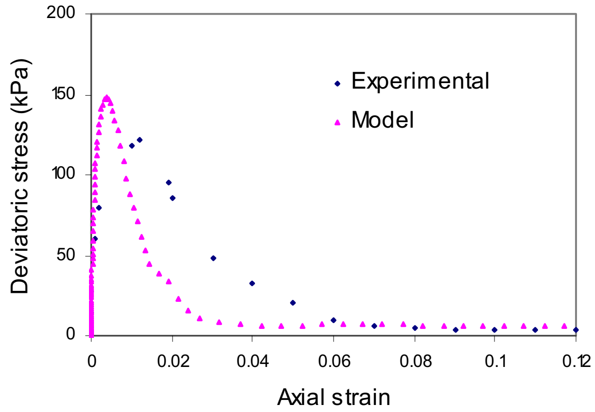
Change in elastic volumetric strain dictates mean effective stress increment according to the equation,

$$dp = K.d\varepsilon_v^e \qquad [6]$$

where K is the elastic bulk modulus and $d\varepsilon_v^e$ is the elastic volumetric strain. Since, $d\varepsilon_v^e = d\varepsilon_v - d\varepsilon_v^p$ and presence of gas bubbles introduces total volumetric strain (which can be approximated from Equation 5), change in elastic volumetric strain will be positive at early stages of shearing which results in increase in mean effective stress. There will be a stage where plastic volumetric strain (governed by Rowe's stress-dilatancy relationship) overcomes the total volumetric strain due to gas compression and diffusion. At this stage, mean effective stress will start to fall.



(a)



(b)

Figure 6. Comparison of experimental result reported by Grozic (1999) with model prediction for type C gassy soil for consolidation pressure of 272kPa; void ratio = 0.85 and S_r = 91%, (a) Deviatoric stress vs mean effective stress curve and (b) Stress-strain curve.

Figure 6b compares model prediction with experimental result in stress-strain space. A lower axial strain to reach peak strength has been predicted during the loading of gassy samples. This is because the model assumes only volumetric strain due to presence of gas bubbles. In other words, bubbles remain spherical after deformation. But, Thomas (1987) noted that gas bubbles deform to an ellipsoidal shape upon axial loading.

4. DISCUSSION

Imam (2000) has given a micro level explanation for the shape of the yield surface of sand. Accordingly, at small strain level ($\eta < M_p$), slippage at inter-particle contacts and breakage of grains results in semi-elliptical yield surface and at large strain level ($\eta > M_p$), gross slippage along a slip plane occurs and hence deformation is influenced by soil dilatancy.

Both yielding behaviour (discussed above) and stress-dilatancy are entirely due to behaviour of particle contact and not influenced by pore fluid. Thomas (1989) suggested that occluded gas bubbles can be idealised by a saturated soil with a compressible pore fluid. However, the effect of gas exsolution and/or presence of gas bubbles on the material parameters should be considered.

Grozic (1999) observed that presence of gas bubble shifts the steady state line above and to the right of steady state line for saturated soil. This supports Sills et al. (1991)'s observation that for any gas content, *the void ratio of a saturated soil matrix surrounding the bubble cavity is a function of the operative stress* (note that Sills et al. (1991) used the term 'operative stress' defined as the difference between total stress and pore water pressure, instead of effective stress for gassy soil). The Increase in steady state void ratio for a given mean effective stress is simply due to presence of gas bubbles (see Figure 7).

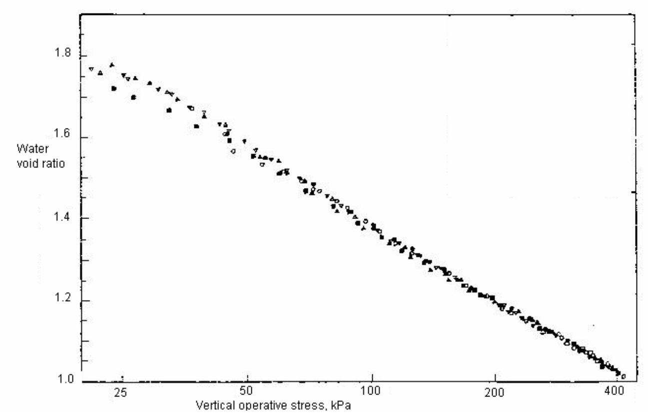


Figure 7. Water void ratio plotted against vertical operative stress for a 1-D consolidation test of cohesive gassy soil, prepared using Zeolite technique (Sills et al. 1991). Degree of saturation of specimens ranged from 86.7% to 100%.

5. CONCLUSION

For gassy soil containing occluded bubbles within voids, analysis can be performed by idealising it as saturated soil with compressible pore fluid. In analysing undrained behaviour, both degree of saturation and excess pore pressure level should be taken in to account. In the field, low degrees of saturation (or high gas content) might correspond to more bubble nucleation and growth, and hence more excess pore pressure. Gas content and excess pore pressure have opposing influences on the undrained strength with increase in gas content increases the undrained shear strength and increase in excess pore pressure reduces the undrained shear strength. This model can be used to predict the liquefaction potential of loose gassy sand by incorporating both the effects of degree of saturation and excess pore pressure, provided the degree of saturation is above 85%. By coupling these two effects, a more realistic response of the behaviour of gassy soil is obtained.

6. ACKNOWLEDGEMENT

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7. NOTATION

H	Henry's law constant
n	Porosity
p_0	Absolute gas pressure.
S	Degree of saturation
$\beta_f, \beta_W, \beta_T$	Compressibility of gas-water mixture, water and soil skeleton respectively.

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