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Liquefaction flow slide at horizontal ground

Coulée par liquéfaction en terrain horizontal

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ABSTRACT
A liquefaction flow slide is an instability in loosely packed saturated sand or silt after liquefaction under monotonic loading. The conditions for such a flow slide are discussed. The relevance of these conditions is illustrated by application to slope failure. The conditions may also be present at horizontal ground under special circumstances, as discussed for three examples.

RÉSUMÉ
Une coulée par liquéfaction est une rupture en sable ou silt saturé meuble après liquéfaction sous changement du chargement monotone. Les conditions pour une telle coulée sont discutées. L'importance de ces conditions est illustrée par application à rupture de pente. Elles peuvent être là en terrain horizontal sous circonstances particulières, comme discutées pour trois examples.

1 INTRODUCTION

Loosely packed saturated sand may be sensitive to liquefaction. The same holds for silt or other non-cohesive soils. For simplicity, however, only 'sand' will be mentioned in this paper. The most common type of liquefaction in loose sand or silt is due to cyclic loading. Liquefaction due to monotonic loading, however, is frequently observed as well. The corresponding instability is usually called a 'liquefaction flow slide'. Liquefaction flow slides are a well-known phenomenon for deep under water slopes in loosely packed sand.

One of the essential conditions for such slope failure is a slope of sufficient steepness, as found by Silvis et al. (1995). Consequently no liquefaction flow slides would be possible in case of horizontal ground. However, a more general description of the conditions, as presented by Stoutjesdijk et al. (1998) makes clear that liquefaction flow slides are principally also possible in special situations with horizontal ground.

These general conditions are described with the help of the results of undrained triaxial tests on loose sand. The commonly known conditions for a liquefaction flow slide in an under water slope will be derived from these general conditions. Subsequently the conditions will be derived for situations at horizontal ground. Three of such situations will be discussed:

- a person standing on quick sand
- a loosely packed, saturated sand body surrounded by very soft clay and peat
- the front of a shield tunnel bored in loose, saturated sand.

2 GENERAL CONDITIONS FOR LIQUEFACTION DUE TO MONOTONIC LOADING

Liquefaction due to monotonic loading in a mass of loose sand develops in two stages:

1. a gradual change in loading resulting, through more or less drained soil response, into a critical distribution of effective stresses
2. a quick, often small, change in loading resulting in a sudden liquefaction of some soil elements through more or less undrained response immediately followed by such a redistribution of stresses in the soil mass that a large part of the soil mass liquefies.

The liquefaction is followed by the actual flow slide, the stages of which will not be discussed here. Stage 1 may take hours, days, years, centuries or even longer; stage 2 may take some minutes, a few seconds or even less.

The sensitivity to liquefaction due to monotonic loading can be tested in a triaxial cell by simulating the first stage by drained loading and the second stage by undrained loading, as explained by Lindenberg and Koning (1981). An example of the result of such a 'wet critical density test' is shown in Figure 1.

The stress paths are presented in the left graph. The drained part is shown by the line from the origin to point A. The total stress path of the undrained part is shown by the vertical line starting in point A and the (unique) effective stress path of the undrained part by the line A-B-C-D-E

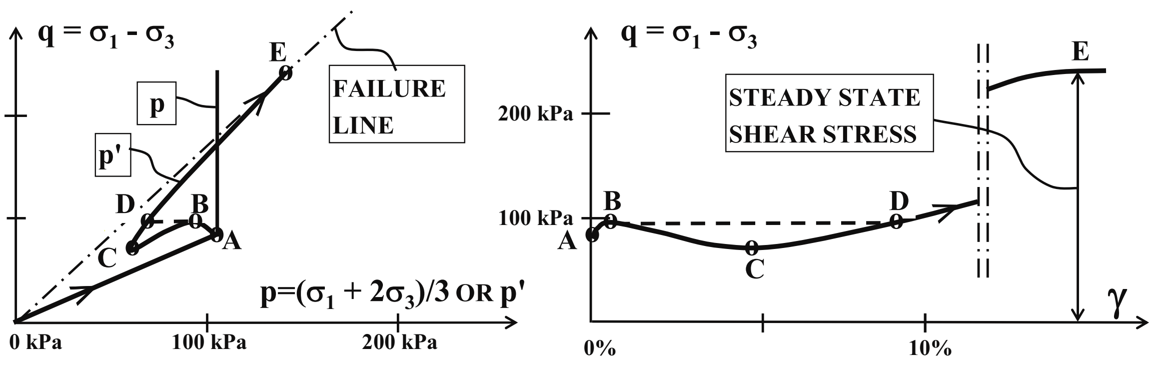

Figure 1 Typical result of wet critical density test on loose sand

The excess pore pressure, u, in this last part is found as the difference between the mean total stress, p, and the mean effective stress, p': u = p – p'. The excess pore pressure increases between A and C from zero to a large positive value. Point C corresponds to the 'characteristic state' or the point of 'phase transformation', where, in drained loading, contraction turns into dilation. In the dilative part of the curve, from C to E, the excess pore pressure decreases and becomes even negative. The curve ends in point E where it meets the failure line.

The corresponding shear strain, γ, is shown in the right graph of Figure 1. It is seen that much deformation occurs as soon as point B is passed. Point E in the left graph corresponds to a horizontal line in the right graph, because it represents the 'steady state' or 'critical state' where deformation continues at constant speed, constant deviator stress and constant pore pressure, thus, constant mean effective stress.

The undrained part is found by a strain-controlled test procedure. Point B represents an intermediate maximum of the shear strength. The presence of such a point is essential for liquefaction as can be understood by considering the stress path and the deformation curve resulting in case of a stress controlled test. The shear stress is gradually increased, starting from point A in the undrained part of the test. A large increase in pore pressure and a large shear deformation occur in a fraction of a second as soon as point B is reached. Point B is followed immediately by point D, as indicated by the dashed line.

The large shear deformation is seen in the triaxial cell by the sudden drop of the plunger and the sudden widening of the sand sample, while vertical stress and cell pressure remain constant (Figure 2).

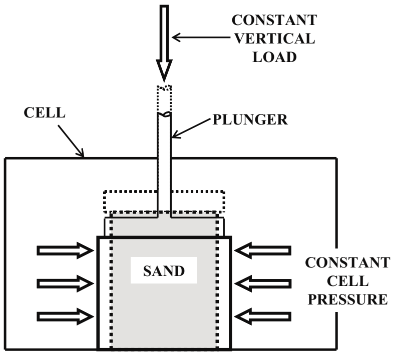

Figure 2 Sudden drop of plunger and sudden widening of sample in stress controlled wet critical density test on loose sand

Point B represents a 'metastable' state of the sample in case the sample is free to deform without change in shear load. Any disturbance will cause instability, i.e. an immediate collapse or large deformation. This is characteristic for liquefaction due to monotonic loading.

If the test is repeated with samples at other densities, results like those presented in Figure 3 are found. In medium dense sand the stress path shows less reduction in mean effective stress, thus less increase in excess pore pressure, than the loose sand stress path. Most remarkable, however, is the absence of any metastability point B. No sudden increase in pore pressure and corresponding large deformation will take place. Consequently, the medium dense sand is not liquefiable. At least this is the case when the undrained test part starts at the initial mean stress indicated by point A. The medium dense sample may become liquefiable, when the mean effective stress of point A is much higher, e.g. p'(A) = 300 kPa. The density at which a metastability point is just present is called the 'wet critical density' (Lindenberg and Koning, 1981). The larger the initial mean stress, the higher the wet critical density.

Very loose sand has a metastability point, like the loose sand. Its behaviour differs, however, because the excess pore pressure is so large that practically no effective stress remains: the sample is 'completely' liquefied, whereas the loose sample shows just 'partial' liquefaction. Another remarkable difference is that no recovery in strength takes place after the very loose sand reached its maximum pore pressure. The steady state (point E) coincides with the state of phase transformation (point C). The sample collapses completely after it reached point B in a stress controlled test.

Now the conditions for liquefaction due to monotonic loading of a sample in a triaxial test can be formulated as follows:

- Undrained loading of a saturated sample starting from an initial stress state reached by drained loading.
- Sand loose enough and initial mean stress high enough to have a metastability point.
- The actual stress state is at (or near to) the metastability point.
- Further loading is stress controlled: the sample is free to deform while the external load is kept constant
- The deviator stress is increased (slightly).

Generalised for large masses of sand:

I. ***saturated, undrained***, i.e. response of soil mass to disturbance (condition 5) is practically undrained.
II. ***loose enough***, i.e. enough sand elements loose enough to have metastability points in their undrained stress paths at their initial mean stresses.
III. ***shear load high enough***, i.e. the initial stress state or the stress state after undrained loading is at the metastability point or near to it for many sand elements.
IV. ***shear deformation possible at constant shear load.*** i.e. sand mass is free to deform and such deformation does not cause change in external load
V. ***quick, small disturbance***, i.e. a sudden (small) change in loading.

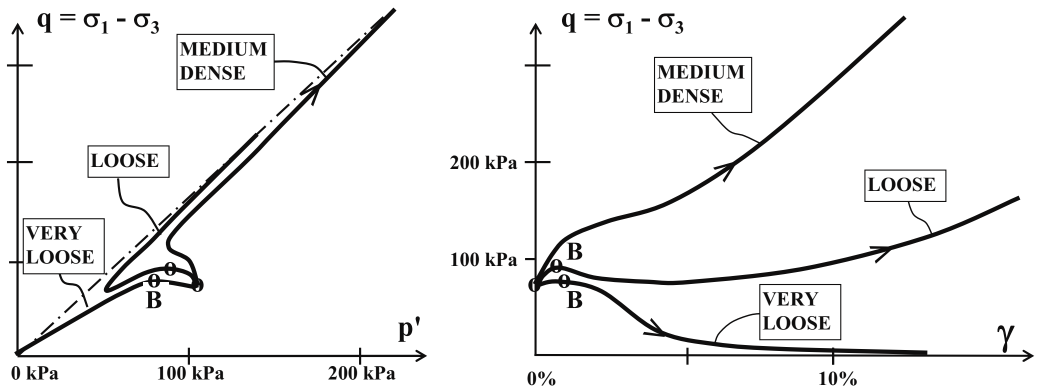

Figure 3 Influence of density on results of wet critical density test

3 LIQUEFACTION FLOW SLIDE IN UNDER WATER SLOPE

The fundamentals of liquefaction flow slides in under water slopes are described by Stoutjesdijk et al. (1998). The conditions I, II, III and V can be transformed for such flow slides to the following conditions:

I. SATURATED, UNDRAINED, like above.
II. SLOPE HIGH ENOUGH & sand LOOSE ENOUGH to have enough sand elements with metastability points.
III. SLOPE STEEP ENOUGH to have the initial stress state at the metastability point or near to it for many sand elements.
V. QUICK, SMALL DISTURBANCE, i.e. a vibration due to pile driving, a quick water level descent through a passing ship or sudden, local erosion.

Conditions IV is automatically met, because sliding down of the upper sand layers along the slope may continue over a long distance, while the shear load, referred to in condition III, does not change.

4 QUICK SAND AT HORIZONTAL GROUND: LOCAL LIQUEFACTION

Condition III is not met in case of zero slope angle. Consider homogeneous loose, saturated sand at horizontal ground not loaded by anything else than its own weight. Than, condition III may be met if the K_0 is small enough. However, as soon as any disturbance causes the sand to start shearing and generate any excess pore pressure, the horizontal soil stress in the vertical planes would immediately increase, causing a decrease in deviator stress. Each sand element is horizontally constrained by the neigbouring sand elements and behaves like in an oedometer test, where no liquefaction can be generated.

The situation is different as soon as a person is present on top of the horizontal ground, causing a local load (Figure 4). The local shear load caused by the person remains present during his sinking down. Thus, condition IV is met locally and local liquefaction is possible.

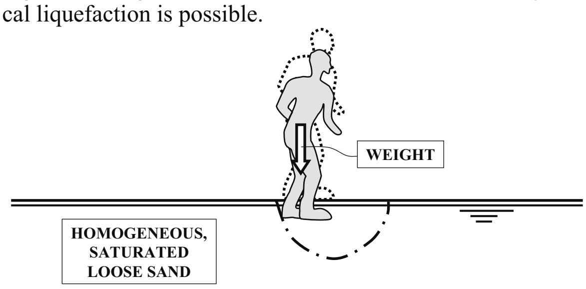

Figure 4 Quick sand at horizontal ground: liquefaction only local

5 LIQUEFACTION FLOW SLIDE IN SAND BODY SURROUNDED BY VERY SOFT CLAY AND PEAT

A loosely packed sand body surrounded by very soft clay and peat was found to be present in the foundation of a railway in Rotterdam, when plans were made to have a tunnel bored underneath the existing railway as described by Korff (2003) and Pachen et al. (2005). Tests made clear that the deepest part of the sand body behaved like the loose sand in Figure 1 in a wet critical density test. The history of the man-made sand body made likely that the initial stress state would be between point A and B, i.e. very near to the metastability point.

The question was raised whether the sand body could liquefy (partly), like sand in a stress controlled triaxial test. Compare the deformations sketched in the Figures 5 and 2. The horizontal support of the sand body differs in both cases. The supporting pressure by the cell fluid in the triaxial test is constant, whereas the clay and peat will resist any widening of the sand body and cause an increase in supporting pressure, although this resistance may remain small due to the softness of the soil.

Finite Element modelling was performed to find out if in this case the soft layers would support the loose sand to prevent a liquefaction flow slide. Finite element of liquefaction due to monotonic loading is no straight forward business. Modelling the loose sand during the stage of quick, small load changes as an incompressible, one-phase material with a shear stress - shear strain relationship according to A-B-C-D-E in Figure 1, seemed attractive at first sight. The part B-C of this curve, however, would implicate a negative stiffness, which easily brings about numerical instability.

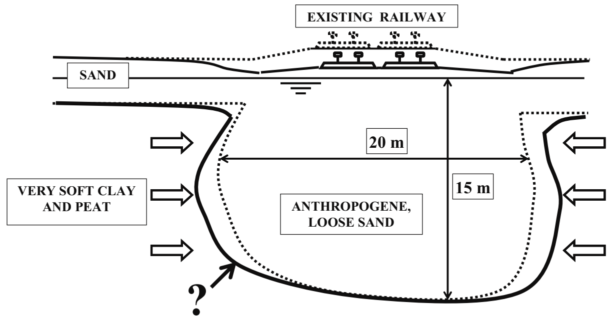

Figure 5 Liquefaction in loose sand body surrounded by soft soil

Such a numerical model would anyhow stop before (partial) liquefaction occurs, if the situation is such that sand elements come near to a metastable state. Liquefaction goes along with sudden large deformations and a sudden large change in stresses. Compare the changes from point B to point D in the stress controlled triaxial test illustrated in Figure 1. Indeed, liquefaction due to monotonic loading is a highly instable physical process, which cannot be modelled directly without numerical instability.

Nevertheless, finite element modelling was found helpful, even for the stage of the quick and small change in loading, to quantify the margin to (partial) liquefaction by stepwise decreasing the strength of the sand similar to a ϕ-c-reduction in 'normal' soils. Here the stepwise ϕ-c-reduction was replaced with the stepwise increase of the excess pore pressure generated in the sand. The properties of the two-phase sand material were modified as follows:

- contraction, dilation and stiffness to isotropic loading of the skeleton were not modelled directly; instead the material was supposed to be incompressible
- the excess pore pressure which would result from contraction, dilation and stiffness to isotropic loading, was introduced as a boundary condition; it was assumed to be equal in all sand elements, given the very fast propagation of any excess pore pressure in saturated sand
- the shear stress-shear strain relationship and other properties were modelled as usual.

The other soil types were modelled as undrained materials. The quick and small change in loading was assumed to be infinite small and was neglected. The excess pore pressure in the loose sand was increased stepwise and the corresponding changes in stress distribution and deformations were calculated by the finite element model. Shear strains of $\gamma > 10$ % were found in a large number of sand elements from the finite element calculations when an excess pore pressure was introduced of $\Delta u = 0.5 \cdot \sigma_{v0}$, where σ_{v0} is the initial vertical stress. This partial liquefaction would also result in a sudden deformation of more than 0.1 m at the top of the sand, which would be an unacceptable settlement of the railway track.

The thus calculated shear deformations in the loose sand were compared to those derived from the triaxial tests (part B-C in Figure 1) for the same increase in excess pore pressure. If the shear deformations, γ, found in the finite element model would have been found smaller than those in the triaxial test, then no liquefaction could occur and the introduced excess pore pressure would have been unrealistic. However, the triaxial tests for the loosest sand tested in the laboratory resulted in $\gamma \approx 3\%$ for an increase in excess pore pressure of $\Delta u = 0.2 \cdot \sigma_{v0}$. The maximum excess pressure for the actual condition of the sand was assessed at a safe 50% of the initial vertical stress. The shear deformation at that value would be $\gamma \approx 5\%$, which is smaller than the shear strain found in the FE-calculations. Apparently, the surrounding peat and clay are not stiff enough to prevent liquefaction and large deformations are to be expected.

Increasing the effective stresses by adding an overburden on the clay and peat layers and lowering the ground water table in the sand layer resulted in shear deformations of about 1-2% at 50% excess pore pressure. Thus no liquefaction is expected to occur with those measures.

6 LIQUEFACTION FLOW SLIDE AT TUNNEL FRONT IN LOOSE SAND

Problems with liquefied sand at tunnel fronts during shield tunnel boring operations were reported in the past several times, e.g. (Darling 1993). The question whether this may have been caused by metastability and subsequent liquefaction is discussed here for the situation of a slurry shield. .

Figure 6 illustrates the initial horizontal soil pressure for a value of $K_0 \approx 0.4$ and the soil pressure that would result if the sand would liquefy completely. Partial liquefaction would result in a pressure distribution in between both lines. The pressure distribution of the front supporting slurry is indicated as well. The pressure is chosen such that it is equal to the initial horizontal soil pressure in the middle of the front. A higher or lower pressure could be chosen, but it will never be equal to the initial horizontal pressure found at all heights, given the limitations for the unit weight of the slurry. The inequality of horizontal soil pressure and slurry pressure increases as soon as some excess pore pressures occur in the sand, especially at the lower end of the front.

Considering the general conditions presented at the end of Chapter 2, it may be concluded that conditions I, II and V will be met if the tunneling is done in loose, (medium) fine sand below the phreatic level. The 'undrained' condition is nearly approximated at least at a distance of more than 0.1 m from the boring front for the relatively quick load changes occurring during boring.

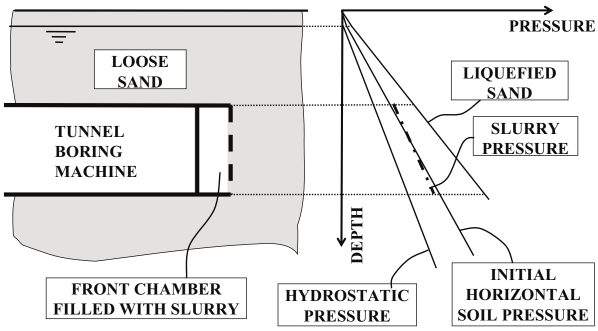

Figure 6 Tunnel boring front in loose saturated sand

No doubt the shear load for the soil very near to the boring front is high enough to meet condition III, given the increase in shear load until failure which is essential for the boring process. However, some negative excess pore pressures may also be generated in this soil just before failure as indicated for point E of Figure 1, and the cutting knives of the Tunnel Boring Machine may cause such a change for no more than just a thin layer of soil.

Whether condition III is also met for a larger part of the soil in front of the boring front is less certain. It probably depends on the value of the slurry pressure. It is most likely met in front of the lower part of the boring front, if the slurry pressure is kept at a value corresponding to $K_0 \approx 0.4$ or 0.5 at the middle of the boring front, as indicated in Figure 6. Then, the pressure corresponds to $K_0 < 0.4$ in the lower part, which brings the stress state very near to the meta-stability point. See point B in Figure 1.

Metastability in the soil in front of the lower part of the boring front may be prevented by increase of the slurry pressure. However, this may bring about metastability or instability in the soil in front of the higher part of the front, if the diameter of the tunnel is large with respect to the depth below the soil surface.

Condition IV is certainly met if the slurry is the main way to support the boring front. The slurry pressure remains constant, when the excess pore pressure in the sand and the associated horizontal soil pressure increase. The conditions for the sand in front of the lower part of the boring front are comparable to those for the sand in a stress controlled triaxial test (Figure 2): the deforming soil may freely flow into the front chamber. This yields an immediate increase in shear stress for sand at a larger distance from the front, probably sufficient to meet condition III, and room for deformation of the sand without decrease in shear stress.

7 CONCLUSIONS

A liquefaction flow slide is the result of liquefaction due to monotonic loading. In this process the excess pore pressure in loose, saturated sand suddenly jumps to a high value and the effective shear strength of the sand suddenly reduces significantly. A metastable state of the sand is an essential condition for liquefaction due to monotonic loading. In such state, liquefaction may occur after a small disturbance.

The general conditions I – V, presented at the end of Chapter 2 must be met for a liquefaction flow slide to occur. These conditions may occur at horizontal ground in special circumstances. Direct numerical modelling of this process does not seem possible. However, numerical modelling in which the sand properties are modified and the excess pore pressures are increased stepwise may be helpful to quantify the margin to metastability.

Liquefaction flow slides may occur at the front of a tunnel boring machine in loose, saturated sand, if slurry is the main front supporting medium, especially if the tunnel diameter is relatively large with respect to the depth below the soil surface.

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