Fluidization Response of Sediment Bed to Rapidly Falling Water Surface

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Abstract: A strain-based criterion for sediment fluidization under transient pressure loading is presented. The criterion predicts that fluidization can be spontaneous, as opposed to incremental, and that it is triggered by a lowering—rather than a buildup—of the pore pressure. The criterion is examined and verified experimentally. A dam break is simulated in a laboratory flume, and a sediment bed is included in the half of the flume initially containing water. Both visual observation and extensive pressure measurements within the bed indicate the occurrence of a massive fluidization failure throughout the entire depth and length of the bed. The fluidization failure is shown to occur on an extremely short timescale. The results from several successive runs, allowing the bed to consolidate overnight between runs, demonstrate the tendency of a bed to repeatedly re-fluidize.

INTRODUCTION

It is well known that a granular bed can sometimes lose some of its strength and start to flow when subjected to transient fluid loading. The phenomenon has attracted interest from many disciplines due to its broad and important applications. In geophysics and hydrology the interest is focused on the various modes of aqueous and eolian sediment transport processes. In geotechnical engineering the emphasis is on soil liquefaction and soil failure modes. Fluidization beds are also used in a wide range of chemical processing applications, wastewater-treatment plants, and various powder technologies. Specifically, fluidization refers to the condition in which the contact stresses between particles in a granular medium reduce to zero. The net effect is that the granular material loses its shear strength and behaves much like a fluid. In geotechnical engineering applications, when this phenomenon is observed in soil, it is frequently referred to as full liquefaction. In the present analysis, this definition is adopted and used interchangeably with the term fluidization.

Sediment transport theories, as well as most soil liquefaction studies, focus on shear stress as the mobilizing agent [e.g., Sleath (1984); Mitchell (1993)]. On the other hand, industrial fluidization-bed applications rely on an imposed vertical pressure gradient across the bed to maintain fluidization [see Davidson et al. (1985) for a review]. Some recent, yet limited, wave flume studies, such as those of Clukey et al. (1983) and Foda and Tzang (1994), have demonstrated that soil liquefaction can indeed be achieved by a predominantly pressure loading from water waves.

The objective of the present paper is to re-examine the role of pressure in triggering sediment fluidization, particularly in the context of sediment transport and soil failure. Unlike most fluidization-bed theories where a steady-state pressure loading is assumed, we will concentrate on the case of transient pressure loading.

First, a simple conceptual model is presented. The one-dimensional (1D) balances of the two phases: the granular skeleton and the pore fluid, are considered. The analysis is not dependent on the particular form of the constitutive law that governs the elastic stress-stress relation for the skeleton, and in this sense is quite general. For fast enough surface loading, the bed response is shown to be primarily undrained, with solid and fluid moving in unison, expanding under tension and contracting under compression. This is combined with the underlying static strains to arrive at the total strain state inside the bed. A strain-based criterion for fluidization is then proposed. It associates fluidization with the condition of zero or tensile skeleton strain. One interesting outcome of the analysis is that our criterion is shown to be dependent on the magnitude of the pressure loading, not on the gradient of the pore pressure, as commonly assumed in earlier fluidization theories [e.g., Davidson et al. (1985); Sleath (1984)].

The model is then examined experimentally by applying transient pressure loading on a sediment bed. To facilitate interpretation it was decided to implement a pressure loading, which has a very simple time history (even simpler than a monochromatic wave), preferably monotonic. In the experiment a sediment bed is placed in a long laboratory channel under a layer of water, which is confined to one-half of the flume by a movable gate. Hydrodynamic loading on the sediment bed is initiated by suddenly opening the gate, allowing the water to flow into the dry half of the flume and simulating a classical dam break flow [e.g., Henderson (1966)]. The release of the gate will result in a rapid draw down of the water surface above the sediment bed, i.e., a quick monotonic decrease in water pressure at the water-sediment interface.

An array of pressure transducers is placed inside the sediment bed to measure the pore pressure response due to the quick monotonic decrease in mud line water pressure. The measurements confirm that a massive fluidization failure takes place inside the bed under the applied "negative" dynamic-pressure loading. Important observations are that the fluidization takes place on an extremely short timescale and has a vertical extent, which is on the order of the depth of the bed. After the initial trial, experimental runs were executed on the same bed, allowing the bed to consolidate overnight between runs. Similar massive failures were recorded for each ensuing run.

CRITERION FOR SPONTANEOUS FLUIDIZATION

We restrict attention here to just the initial triggering mechanism for fluidization, not to what happens afterwards. We consider a 1D uniform soil column, with the seafloor located at $x = 0$. The governing conservation equations for the two phases include either the solid skeleton and the pore water,

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drawn from the general theory of mixtures (Bowen 1976), or the generalized Biot's (1956) equations

$$n\rho_w u_t = -np_x - \frac{n^2}{k}(u - v) \tag{1}$$

$$(1 - n)\rho_s v_t = \sigma_x - (1 - n)p_x + \frac{n^2}{k}(u - v) \tag{2}$$

$$n(u - v)_x + v_x = -\frac{n}{\beta}p_t \tag{3}$$

where u = pore-water vertical velocity; v = solid skeleton vertical velocity; p = excess (above hydrostatic) pore pressure; and σ = excess (above static) solid, or "effective," normal stress in the vertical direction. The physical parameters are soil porosity n, water density ρ_w, solid density ρ_s, bulk modulus of water β, and permeability of the porous skeleton k. For closure a fourth equation on the constitutive stress-stain relation for the solid skeleton would be needed. For our present purposes we assume the loose solid skeleton to behave elastically, with a Young's modulus of elasticity E that varies with depth x below the mud line in some general manner [e.g., Mitchell (1993).].

First, we consider the role of drainage in the response of the soil column to a rather rapid surface loading. It is expected that drainage will be confined to a thin boundary layer, next to the seafloor, and that the thickness δ of this boundary layer according to the general formulation of Mei and Foda (1981) be given by

$$\delta = \sqrt{(t_o k E)} \tag{4}$$

where t_o = timescale of the surface loading. If we assume E to be proportional to the static overburden (Mitchell 1993), i.e., $E \sim \sigma_o \sim (1 - n)(\rho_s - \rho_w)g\delta$, we further get by substituting into (4) that

$$\delta = (1 - n)(\rho_s - \rho_w)g t_o k \tag{5}$$

For fine sand or finer, permeability k is 10^{-7} m^3 s/kg or lower; and for $t_o \sim 10$ s, we get $\delta \sim 1$ cm or smaller—a very thin drainage boundary layer. Below the boundary layer, we may ignore seepage and assume $u \approx v$. Therefore, from (3) we get

$$\epsilon_d = \frac{\partial X}{\partial x} = -\frac{n}{\beta}p \tag{6}$$

where $v = \partial X/\partial t$; and X = solid displacement in the bed. Notice that the solid strain ϵ_d is proportional to the pore pressure, not to the pressure gradient $\partial p/\partial x$.

The total strain in the solid skeleton would then be given by adding the preceding dynamic strain (6) to the static strain, caused by the bed's own weight. The static strain is a classical Hertz contact problem. Let two soil grains at depth x be pressed against each other by the weight of the soil column above (the overburden). The resulting static strain ϵ_s is obtained by solving the contact problem between the two grains [see, e.g., Timoshenko (1956)] to get

$$\epsilon_s = -0.813\{[\pi(1 - n)(\rho_s - \rho_w)gx]/E_{grain}\}^{2/3} \tag{7}$$

where E_{grain} = elasticity of the grains (e.g., the elasticity of quartz), which is orders of magnitude higher than the elasticity E of the loose granular skeleton.

If the total strain at some location x and some time t happens to be tensile or zero, then the soil grains, at such location and time, are considered in a state of momentary suspension, or momentary fluidization, i.e.

$$\epsilon_d + \epsilon_s \geq 0 \quad \text{for fluidization} \tag{8}$$

It is clear from (6) that for fluidization the imposed dynamic pressure has to be negative. This will yield tensile strain, which will act to neutralize the compressive static (overburden) strain.

We should note that the aforementioned condition and its implications are drastically different from commonly adopted fluidization conditions in the literature [see, e.g., Sleath (1984); Davidson et al. (1985)]. Earlier fluidization studies placed a rather restrictive condition for bed fluidization, namely that the pore pressure gradient should equalize the buoyant specific weight of the solid skeleton, i.e.,

$$\frac{\partial p}{\partial x} = -(1 - n)(\pi_s - \rho_w)g \tag{9}$$

Strictly speaking, this condition is valid only if we assume a steady-state condition in the bed. This is normally applied when considering industrial fluidization-bed systems, where a constant upward seepage flow is forced through a granular bed for a variety of chemical processing applications [e.g., Davidson et al. (1985)]. Under transient loading, the situation may be very different due to inertial effect. In spite of that, the previous steady-state condition has been "loosely" used to examine fluidization under transient wave loading [e.g., Sleath (1984)]. Because water waves are not likely to generate large enough pore pressure gradients inside the underlying porous bed, it has been generally concluded that fluidization [as defined by (9)] is not likely.

On the other hand, it is commonly assumed in the soil mechanics literature that the phenomenon of "soil liquefaction," which is triggered by the vanishing of the soil's "effective" or solid stress, is forced by a gradual buildup of the surrounding pore pressure. Usually, several cycles of wave loading are required for pore pressure to build up incrementally to the critical condition. Our criterion (8), which also identifies the vanishing of the skeleton's solid strain, is different in two fundamental ways: (1) it is associated with a lowering (not a buildup) of the pore pressure; and (2) it predicts spontaneous fluidization rather than incremental.

When fluidization takes place and solid particles become suspended within the pore fluid, a buildup in the pore pressure may then follow. At this stage the pore water will have to support the submerged weight of the suspended particles, resulting in an increase in the pore pressure. In other words, we argue here that the buildup in pore pressure is a result, not a cause, of the observed fluidization process.

Comparing (8) and (9) we observe that our criterion is a much weaker condition than that for the steady-state flow. A momentary fluidization of sediment, according to (8), is therefore a very likely possibility under typical water wave loadings. However, before attempting to examine the implications for water waves, we wish here to examine and verify the preceding criterion under a simpler transient loading condition.

EXPERIMENTAL SETUP AND PROCEDURE

Experiments were conducted in a 6.1 × 1.2 × 0.38-m flume as detailed in Fig. 1. A wooden false floor was constructed so as to create a sediment bed 1.8 m in length, 0.3 m in depth, and 0.38 m in width. A gate of machined aluminum was used to divide the flume in half. As illustrated in Fig. 2, the gate was hinged at the top of the flume and fit against 1.9 × 1.9-

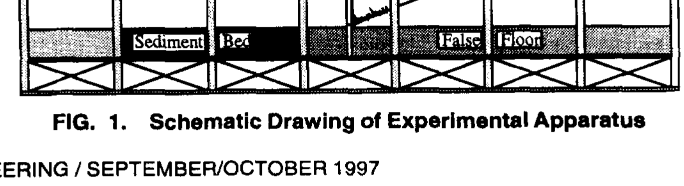

FIG. 1. Schematic Drawing of Experimental Apparatus

cm strips of Plexiglas, which were attached to the inside of the flume. The gate was latched at the bottom by means of a rectangular aluminum tab, and rubber automotive stripping was used at the seams to provide a watertight seal. The task of opening the gate was facilitated by attaching a series of four heavy duty springs to the bottom of the gate. Stainless steel cable of 0.32 cm diameter was used to connect these springs to a ratcheting winch located at the far end of the flume. Finally, a set of spring loaded hinges was used to prevent the gate from falling back into the water once it had been opened.

Pressure measurements were obtained by placing an array of six transducers (model number BP-500GRS27516, Kyowa Electronic Instrument Co., Ltd.) in the bed. As shown in Fig. 3, the transducers were placed at various depths at a distance of 0.3 m from either end of the bed. The diaphragms of the transducers were covered by caps of very fine mesh. This prevented any sediment from coming into contact with the diaphragms and ensured that the measurements obtained would be due solely to fluid pore pressure. The data were conditioned by a Daytronic Corporation model 9178A strain gauge conditioner and then processed by an IBM 486 personal computer. The acquisition was performed by a LVDT & Variable Reluctance Sensor Interface Card (UPC601-L), and an Easy Sense data acquisition software package, both from the Validyne Engineering Corporation. The soil used in the experiments was a commercially available silt of negligible plasticity. The mean grain size was 50 μm, and the grain-size distribution is shown in Fig. 4. The mixed bed had a porosity of 0.51, and the solid density of the soil was 2,610 kg/m^3 (Tzang 1992).

The first step in conducting a series of tests was to lay the sediment bed. Wooden forms were used to partition the bed from the rest of the flume while approximately 0.46 m of water were added. An air-powered, handheld mixer was then used to thoroughly mix the sediment and water into a thick slurry. The transducers were then placed in position and the bed allowed to consolidate overnight. After removing the forms the next day, the gate was closed and the half of the flume containing the bed was slowly filled with water to a depth of 0.69 m above the surface of the bed. The winch was then used to tension the springs and, finally, an overhead crane was used to pull out the tab securing the gate. After completion of a run, the water in the flume was slowly drained and bed allowed to consolidate overnight before the following run.

The ability of the experimental apparatus to simulate a theoretically instantaneous dam break was found to be quite remarkable. In a recent dam break investigation by Ramsden and Raichlen (1990), the dividing gate had been withdrawn vertically, normal to the direction of the bore's propagation, with the result that the advancing front was somewhat impeded. In the current investigation the gate rotated out of the way of the advancing front. The fact that the gate was removed in the same direction as the front's propagation yielded a much more effective ''instantaneous'' failure. Fig. 5 presents a quantitative evaluation of the performance of the gate. Data on the

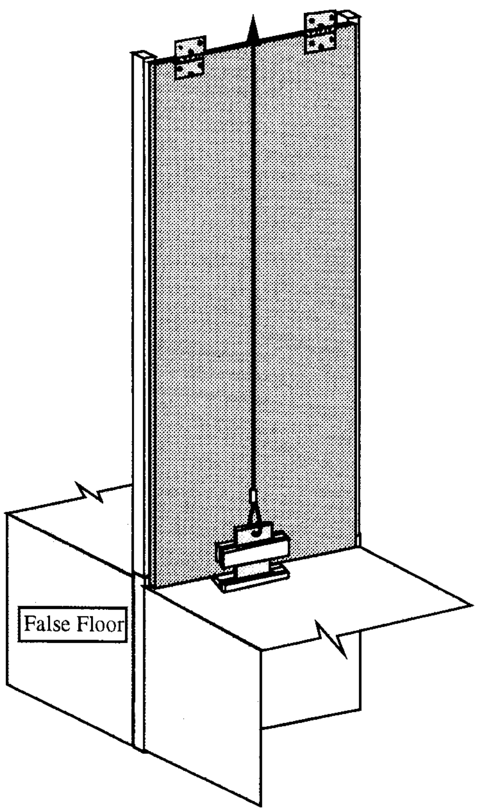

FIG. 2. Details of Gate Mechanism

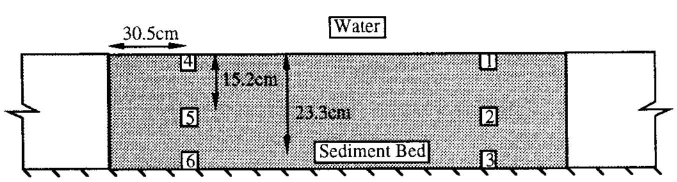

FIG. 3. Location of Pressure Transducers within Sediment Bed

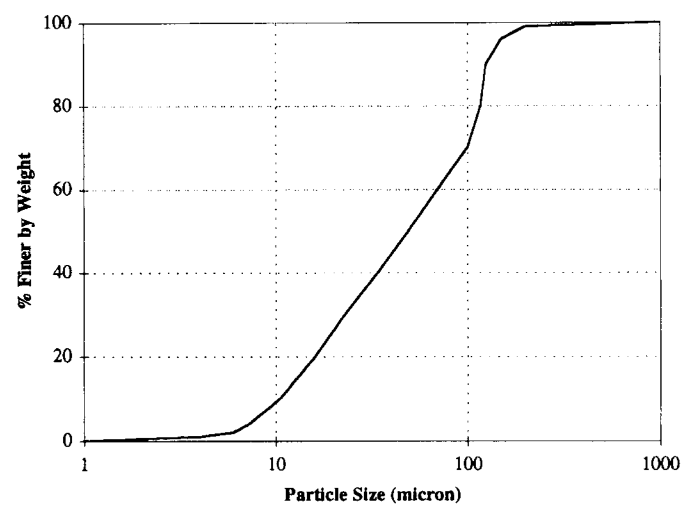

FIG. 4. Grain-Size Distribution

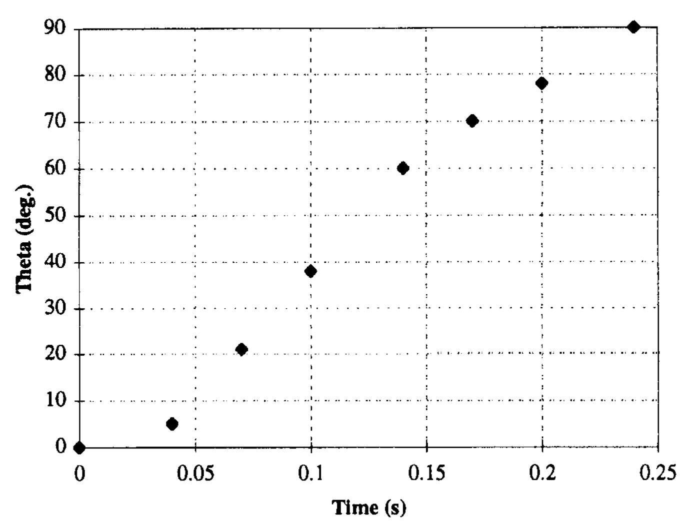

FIG. 5. Time History of Gate Angular Displacement from Vertical

gate's position were obtained through use of a video camera, operating at 30 frames per second. In less than 0.25 s, the gate was found to have rotated completely to the horizontal.

PORE PRESSURE RESPONSE

The transducers at locations 1 and 4 were at the mud line to record mud line pressure loading, T1 and T4, respectively, and also to serve as a reference for pressure measurements within the bed. Figs. 6(a) and 6(b) show sample measurements from a representative run. Fig. 6(a) shows pressures from the transducers column closest to the gate, and pressures further away are shown in Fig. 6(b). Only the pore pressures T3 and T6, taken at the bottom of the tank (locations 3 and 6) are shown. The intermediate pressures (locations 2 and 5 in Fig. 3) show transitional behaviors, and are therefore omitted for clarity of presentation. Also shown are the pressure differences T3 - T1 in Fig. 6(a), and T6 - T4 in Fig. 6(b). Note that immediately after the opening of the gate [just before time t = 15 s in Figs. 6(a) and 6(b)], high frequency oscillations are seen in the raw pressure records, but clearly absent in the pressure-difference curves. These oscillations are associated with vibration induced by the release of the gate and are damped out quickly. In the following discussion, we disregard these oscillations and focus on pressure response to the dam break.

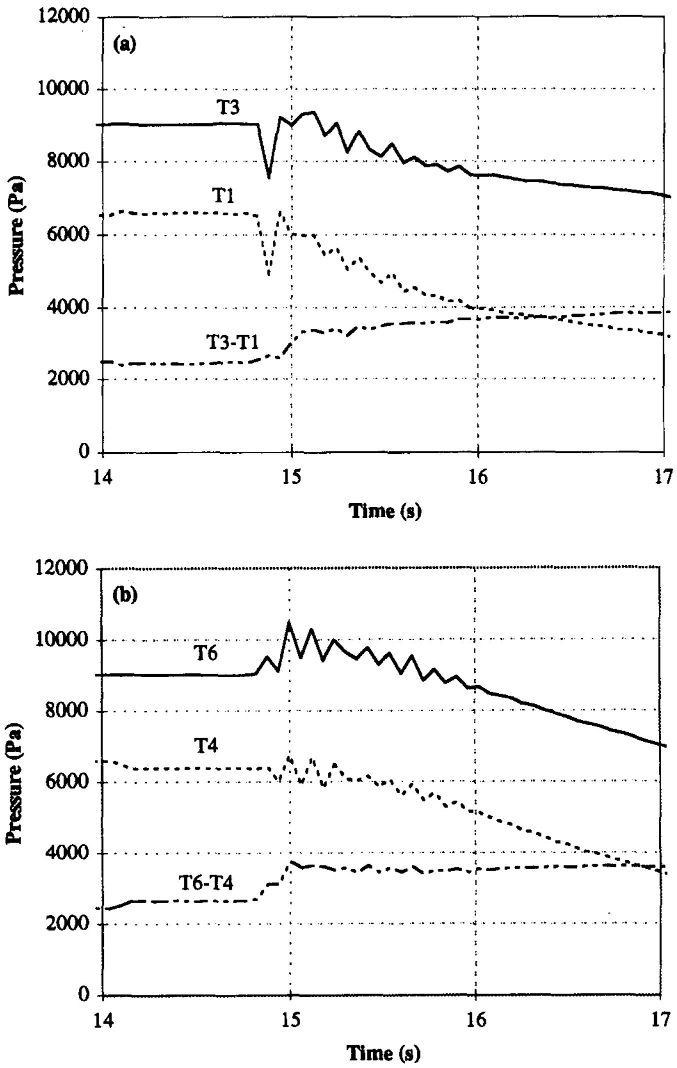

FIG. 6. Pressure Differences: (a) Mud line Pressure T1 at Location 1, Pore Pressure 30 cm below at Location 3 (T3), and Pressure Differences T3 - T1; (b) Mud line Pressure T4 at Location 4, Pore Pressure 30 cm below at Locations 6 (T6), and Pressure Difference T6 - T4

As the gate opens, water drains out and the water surface falls down causing a rapid decrease in pressure at the mud line as seen in T1 and T4. The pore pressure records T3 and T6 clearly show a spontaneous response, with hardly any lag behind the mud line loading. Furthermore, it is quite remarkable that at each location, there is an initial buildup in pore pressure—above hydrostatic value—in response to the lowering of the mud line pressure. The above-hydrostatic increase is more prominent in the T6 record than in T3. This pressure increase is explained as follows. First, surface pressure is communicated instantly (or, more precisely, at the speed of sound) to the continuously connected pore space below, with negligible seepage, or drainage relaxation effect. The applied pressure would also be acting on the almost undrained solid skeleton, causing it to expand. This results in the fluidization of part of the bed, as explained in a previous section. The weight of the solid particles suspended within the fluidized zone would be quickly transmitted to the pore pressure below, causing the observed spontaneous pressure rise shown in Figs. 6(a) and 6(b). The difference in pressure rises between T3 and T6 can easily be explained by looking at the overlying mud line pressures, T1 and T6, respectively. Being closer to the gate, the mud line pressure T1 drops down at a faster rate than in T6, taken further away from the gate. Fluidization-induced pressure changes are to be added to these boundary pressures to produce the internal pressures, thus making the net pressure rise—above hydrostatic—in T3 less pronounced than in T6. Clearly, the fluidization-induced pressure buildup is more directly displayed by plotting the pressure differences T3 - T1 and T6 - T4. In fact, as shown in Figs. 6(a) and 6(b), these two pressure difference curves are quite similar. This implies similar fluidization failures, even though the time histories of the applied mud line pressures at the two locations are different. These observations and the general behavior of the bed are quite consistent with our spontaneous fluidization model. Further, by substitution reasonable values for the relevant material properties, an estimate of the fluidization depth can be calculated from (8). For example, assuming $\beta = 10^8 - 10^9$ N/m^2, $E_{grain} = 10^{11}$ N/m^2, ρ_s = 2,600 kg/m^3, and n = 0.5 in (6) and (7) yields a fluidization depth $x_f \sim 0.1-1.0$ m, i.e., the size of our sediment box or larger.

The degree of fluidization at a depth d inside the sediment box may be represented in terms of the following nondimensional fluidization parameter $N(d)$:

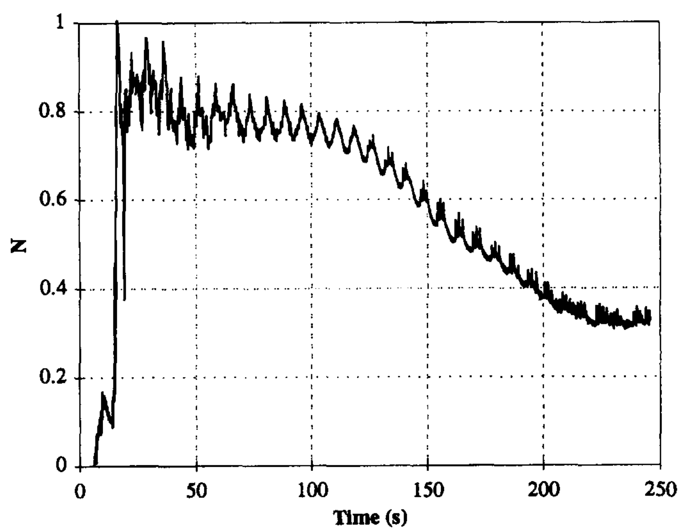

FIG. 7. Fluidization Percentage N at Location 3 (Extended Record)

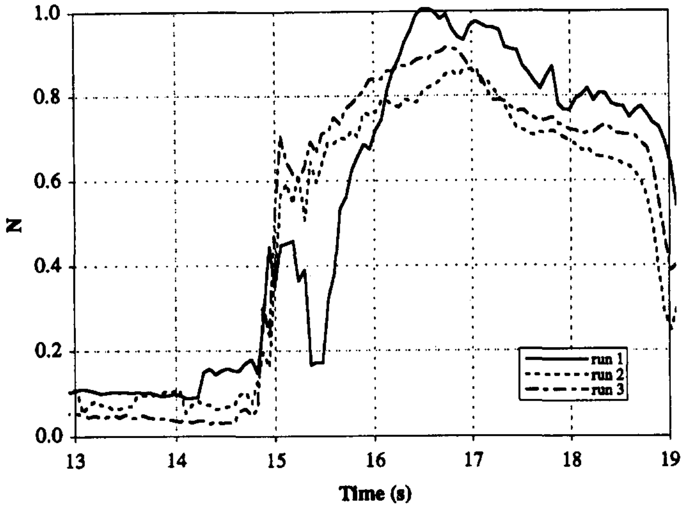

FIG. 8. Details of *N* at Location 3 during Initial Stages after Gate Release

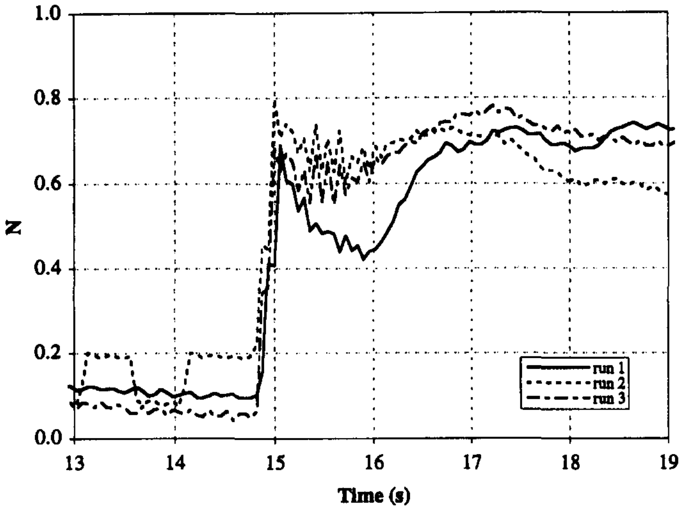

FIG. 9. Fluidization Percentage *N* at Location 6

$$N(d) = \frac{\Delta p}{\Delta p_{max}} = \frac{p(d) - p_m - \rho_w gd}{(1 - n)(\rho_s - \rho_w)gd} \quad (10)$$

where $p(d)$ = pore pressure at depth d; and p_m = mud line pressure above the point of measurement. $N = 0$ corresponds to the initial hydrostatic condition. $N > 0$ implies in our experiment a load transfer from the solid skeleton to the pore fluid. N approaching unity would indicate the soil above is approaching full fluidization failure, with a large portion of the submerged weight of the soil column above [the denominator in (10)] being transferred to the pore pressure. Clearly, this quasi-static interpretation would be violated at the very initial stages, after the gate release, where appreciable vertical momentum is very likely within the bed (see section under "criterion for spontaneous fluidization"). The bed, however, is expected to quickly reach a quasi-static equilibrium within a few seconds after that, where the aforementioned parameter N would be a reasonable measure of the bed's fluidization.

Fig. 7 details the fluidization percentage at location 3 for a duration of about 4 min after the release of the gate. First, the dramatic increase seen at about 15 s is due to the opening of the gate. This is followed over the next several minutes by N slowly decreasing as the sediment in the bed slowly settled. The oscillations in N are associated with the sloshing of water in the flume. Next, attention is turned to the short-scale events immediately following the release of the gate. The records for location 3 for three consecutive runs are shown in Fig. 8. Maximum values of N from 0.75 to 1.0 were attained very rapidly. Good repeatability between runs is seen. This seems to suggest that fluidization neither appreciably strengthened nor weakened the bed to subsequent failures. Finally, Fig. 9 shows the data from location 6. Again, similar behavior is observed of a sharp rise in N, approaching unity in a matter of a second, then slowly decaying over the following few minutes. Notice, however, that the maximum value of N at location 6 is slightly lower than the corresponding value at location 3. One possible explanation is that the magnitude of maximum mud line pressure drop above 3, being closer to the gate, was larger than that above 6.

CONCLUSION

Results are presented that detail the pore pressure response inside a silty bed when subjected to a sudden negative pressure loading. This loading was facilitated through the simulation of a dam break flow in the laboratory. It was found that fluidization throughout the vertical extent of the bed took place on a very short timescale, on the order of one second. A new strain-based fluidization criterion is proposed to explain the experimental observation, predicting spontaneous fluidization due to tensile strains. Successive runs were performed on the same bed to investigate the effects of refluidization and consolidation. The striking similarity of pressure data from one run to the next indicated that this cyclic fluidization and consolidation did not appreciably stabilize the bed.

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