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Time effects, cycles of tension – Compression of model piles

Influence du temps, cycles de traction – Compression de pieux modèles

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ABSTRACT: Scale models of piles in sand were used to study long duration and cycles behavior under static loading. Stress conservation was carried out by Hydraulic Gradient simulation (scale 1:75) and by 2 DIM rods model (1:7.4). Impact vibration tests carried on the Hydraulic Gradient models were compared to the static ones. Image analysis was used in the 2 DIM “sand”.

RESUME: Des modèles réduits de pieux dans un sable ont été utilisés pour étudier le comportement à longue durée et dans des cycles sous chargement statique. La conservation des contraintes a été assurée par une simulation par Gradient Hydraulique (échelle 1:75) et dans le modèle d'un sable 2 DIM composé de cylindres (1:7.4). Des tests de Vibration - Impact ont été exécutés dans la simulation par Gradient Hydraulique et comparés aux essais statiques. Une analyse d'images a été utilisée dans le “sable” à 2 DIM.

1 INTRODUCTION

The paper presents results of scale model tests on piles in both real and simulated sands. Succinctly a review of most of the problems is presented. Also the same kind of problems exist for other structures: on the one hand design is carried out by simplistic formulas based on such parameters as capacity, toe resistance, lateral resistance. On the other hand quite heavy calculations are carried out which have little impact on the design and which are “validated” by fitting of some force - displacement curves. Validation of the design has been carried out sporadically by expensive static loading tests. Those tests are limited by the capacity of the loading installation, not that of the pile. The time effects which appeared could neither be studied in detail nor accounted for. Thus, for some time now it has been clear that the subject could only be studied by model tests which conserve the stresses: The Hydraulic Gradient method and centrifugation. As will be shown the pile's resistance is strongly influenced by the loading history in a way that suggests a partial dependence on local grain - arrangements, which cannot be dealt with by the continuum methods of soil mechanics. As the soil varies all the time the notion of capacity is meaningless as well as the dynamic methods for its “prediction”. A direct demonstration is given that the “toe” and “lateral” resistances are one and the same so that the whole pile should be excited also in dynamic tests. Using the two dimensional “rod - sands” it has become possible to analyze video films and extract the displacement and deformation fields, which any computer algorithm must from now on give correctly. Also the micro motion of the grains can be followed.

2 METHODOLOGY

Conservation of stresses requires that all the volume forces be increased by the factor of length reduction. That implies the conservation of the particle velocity and the simulation of gravity, either by Hydraulic Gradient or centrifugation. Centrifugation requires robotics (Zelikson et Al, 1994). The Hydraulic Gradient method is by far more suitable and less expensive for piles.

The Hydraulic Gradient method used a highly siliceous fine sand mediumly compacted. It was placed in a large

“permeameter” and a saturated vertical percolation was installed between a free surface water pressed by air from above to a bottom filter. The dissipation of the water - head produced a simulated specific weight corresponding to length reductions by 75. The diameter of the soil was 0.8 m [60 m in situ] and its depth 0.4 m [30 m]. The model pile was a close ended steel tube of 1.6 cm [1.2 m] diameter which was tested at penetrations of 0.1 to 0.3 m [7.5 to 23 m]. It was loaded vertically by a double action hydraulic jack connected to an electric pump (Zelikson, 1969). The rods of the 2 DIM “sand” were brass hexagonal cylinders (diameters 6 mm, 10 mm) and 3 mm circular ones. The hexagonal rods were machined to circular sections of a slightly reduced diameter on both ends so that the contact length was halved and the apparent density doubled. The hexagonal form prevented grain rotations and the granular ensemble resembled sand more realistically. The artificial density (relative to 1.6 g / cm³) corresponded to a length reduction by 7.4. The angle of internal friction was 30°. The picture presented to a camera in front was that of separate circular bright targets. The “soil” was 1.0 m [7.4 m] deep and 1.2 m [9.0 m] large. Two “piles” 4 cm large [30 cm] at 0.3 m [2.2 m] outer spacing were held together at the head and at the toe and the structure pushed into the bottom “soil” which rose 20 cm between the piles, giving the semblance of a single hollow-tube pile. The head was loaded by a double action jack connected to a servo - pump. The forces were both vertical and at 33° to the vertical. The work was carried out at the Laboratoire de Mécanique des Solides, Ecole Polytechnique, Palaiseau, France.

3 EVOLUTION OF THE REVERSIBILITY RANGE: TIME EFFECTS (HYDRAULIC GRADIENT).

During construction the pile undergoes loadings which cause residual stresses and grain arrangements in the soil. A “virginal test” is carried out on a pile where those changes of the soil are negligible.

Fig. 1 depicts the force (F) displacement (U) graph of a common loading test. The loading was as virginal as could be as the model pile was installed in the soft sand prior to the application of the gradient. A virginal capacity $C_v \cong 1500 \text{ N}$ [8.4 MN] at $U / d \cong 4\%$ ($d = \text{diameter}$) was followed by cycles of increasing load maxima. At these cycles the lowest force was above zero.

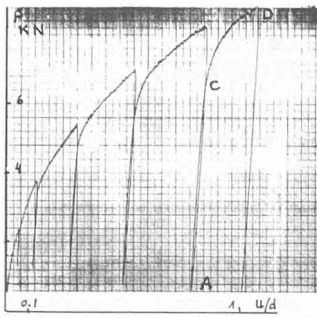


Figure 1. Compression loading test

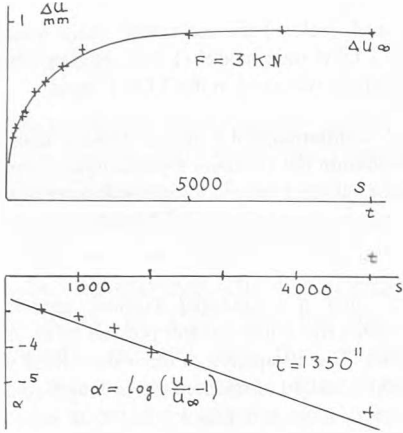


Figure 2. Delayed settlement

At each cycle the reversibility range expanded, while the rigidity remained essentially constant. The last hypothetical loading test ACD has a capacity $C \cong 7000 \text{ N}$ [45 MN]. The total residual settlement was about one diameter, which could have been produced at the toe by loading the soil against the pile either by jacks (i.e. underpinning) or by grouting, (a relevant case history from a bridge in Argentina was reported at the San Francisco conference). Underpinning capacity is recognized by the codes. Is the 5th cycle different than the 1st one? Above the "yield point" time effects appear (Fig. 2). The load was rapidly raised to about double the capacity then held constant. From that instant the additional displacement is depicted as a function of time. A simple Kelvin Visco - Elastic model gives:

$$U/U_x = (1 - \exp(-t/\tau)) \text{ or } (U_x - u)/U_x = \text{EXP}(-t/\tau)$$

According to Fig 2 the characteristic time τ is 23 min. Also U_x / U was 1.5. The reloading curve ACD of Fig. 1 has an Elastic part AC ($C \gg C_0$), which can be explained by the formation upon the preceding unloading of a field of residual stresses (Zelikson, 1969). This however does not account for the time effect. Loading tests using screw - jacks operating at a constant speed showed that the additional rigidity was independent of the speed. It is natural to conjecture that re-

TABLE I
(cycling started after 4000 S, at $U_x \cong 6.33 \text{ mm}$, $F_x = 2200 \text{ N}$).

F;N	300	1600	300	1600	300	1600	300
U;mm	5.63	6.12	5.67	6.15	5.70	6.19	5.74
F;N	1600	300	1600	300	1600	300	1600
U;mm	6.30	5.82	6.34	5.90	6.42	6.00	6.48
F;N	300	1600	1200	320	1200	320	1200
U;mm	6.02	6.52	6.40	6.08	6.40	6.08	6.40

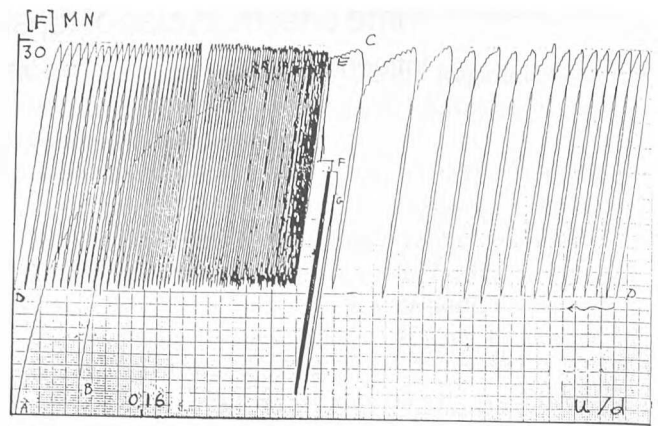


Figure 3. Cycles in compression

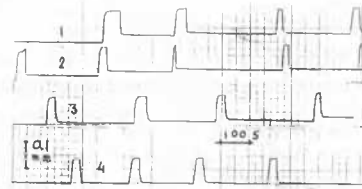


Figure 4. Delayed settlement accelerated by pulses

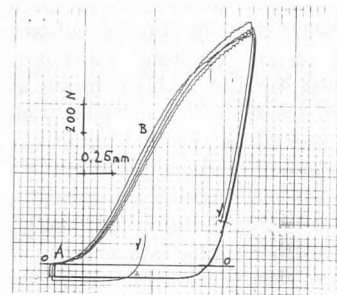


Figure 5. Cycles of tension - compression

arrangement of the grains takes place, which takes time. Putting the results of figures 1;2 together one is inclined to attach to each F_x a U_x , and thus construct the "final" force displacement curve. However, according to the tests' results, later cycling up to F_x then produced further residual displacements (Table I). According to table I additional permanent settlements were obtained even at $F_{max} = 0.7 F_x$, while stability existed for $F_{max} = 0.4 F_x$. Fig. 3 depicts similar results, where cycling from point C onwards produced diminishing residual settlement per cycle but not stability up to $U = 1.7 U(C)$. Fig. 3 depicts a yet unstable cycle up to point F, and a stable one up to point G (at about twice the initial capacity).

Fig. 4 depicts the interplay between cycling and delayed settlements: at $\approx 500s$ intervals a square pulse of force was added, causing additional residual settlements and accelerating the final stability.

Based on these and many other results it can be stated that up to several times the virginal capacity any force can be made the upper limit of stable compression cycles, and thus be taken as a capacity. The corresponding settlement is very long to define and might not be unique. When the stresses become zero in a zone of soil the preferential grain arrangement is obliterated. This is the reason for the big softening which follows some loading in tension. However, it is interesting theoretically to note that when cycling is carried out between fixed limits, one in tension and one in compression then after few cycles the plastic

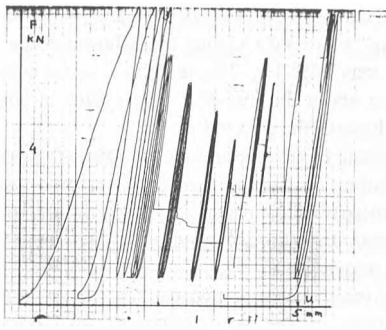


Figure 6. Compression cycles following tension

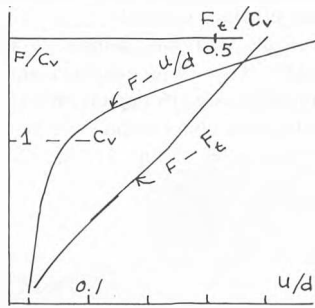


Figure 7 Toe resistance related to total force

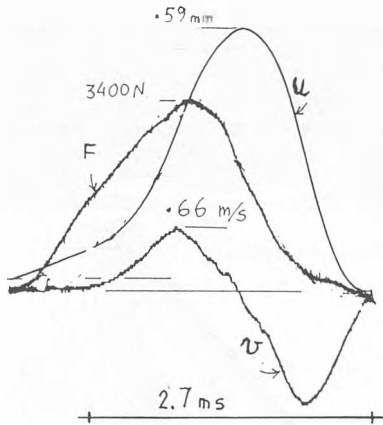


Figure 8 Signals of impact

displacements in one leg are fully compensated by those of the other one leading to stability. In Fig. 5 the closing of the "fissures" is in the part AB of the curves. In Fig. 6 is depicted the rapid rehabilitation of the rigidity in compression following a loading in tension, at a price of an additional settlement of about 0.15 d. Barring exceptional cases (e.g. earthquakes), the loads vary between close limits in which case according to Fig. 6 there is stability. Much evidence is present (e.g. Zelikson, 1969) to show that the variations in the resistance is not caused by density changes. However in a denser sand the "work hardening" is greater.

4. DYNAMIC PREDICTION OF STATIC TESTS (HYDRAULIC GRADIENT)

By the preceding chapter it is clear that all that is related to the slow time effects, mainly grain re-arrangements cannot possibly be predicted by impact loading. Methods using pile driving utilize short impacts which scan the pile in order to discover the

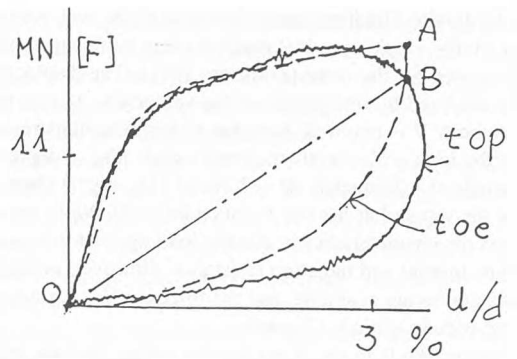


Figure 9 Impact force vs. displacement

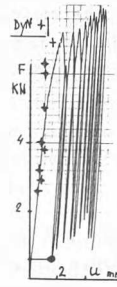


Figure 10

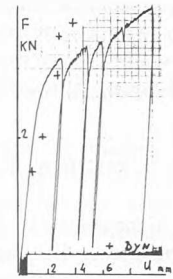


Figure 11

Force vs. displacement from impact & static tests

resistance at each point separately. In this connection it is pertinent to discuss some old and new results of static tests (e.g. Zelikson, 1986). In a pile instrumented by straingages it is possible to measure the resistance at different stations. When this is drawn versus the total force F , a very important fact appears: the resistance is proportional to F not only in the Elastic range, which is normal, but through the plastic range, well above the capacity, and also in cycles.

An example is given in Fig. 7 by the straight line of "toe resistance vs. F ". If the yield points corresponding to piles at different toe depths are compared, then in compression the capacity is proportional to γh , and in tension to γh^2 (γ = volume weight). Clearly in tension the tangential stress is proportional to the horizontal pressure at rest. The result in compression should be taken together with the fact that the instrumented piles show that at 3 diameters from the toe there exist 70 - 80% of the total resistance. This part of the pile is the "tip" where the stresses are proportional to the vertical earth pressure at this depth i.e. γh (with a large factor). Accordingly the horizontal pressures on the pile and the tangential stresses which follow are very much higher than the case "at rest". It follows that the whole tip, and preferably the whole pile should be excited "in phase" and not just scanned. From the results of the previous chapter it is inferred that each impact stands by itself (no time for re-arrangement), that a series of impacts is under the influence of the grain arrangement of a preceding static loading and so "predicts" the next to come one. Also an "over-rigidity" is to be encountered above the capacity. All this was verified by the tests.

The name "impact vibration" was given (Zelikson, 1991) to tests where the frequency is so low that the whole pile vibrates in phase. The best mode of excitation is that of a mass falling on a soft "cushion" (rubber, coil springs etc.). The model tests indicate for the ram a weight of about 5% of the maximum static load. An industrial method was proposed by Gonin (1984) and is now copied around the world.

In the Hydraulic Gradient methods the velocity was measured directly by the Hopkinson Bar method using straingages, which were also used for the force (Zelikson, 1991). The displacement U was calculated by integration of the velocity V . Conservation of the velocity V is required from that of the dynamic stresses. It follows the time scale equals the length scale. Fig. 8 depicts the time signals at a frequency of 37 Hz [5 Hz]. Fig. 9 shows the force at the top and at the toe as functions of U . While the toe's graph has the characteristics of a static loading the top's graph is blurred by inertial and radiation resistance. However, in situ only the top's graphs are available, and the forces at $V \cong 0$ are used to construct pseudo-static $F(U)$ graphs.

Those points like B in fig. 9 are hard to define, and are used to construct pseudo static force displacement graphs (fig. 10 - 11). It is seen that the dynamic results become stiffer in the "plastic" range and "over-shoot" the static curves, and could not give detailed information about this range. Also the curve of one blow (fig.9) do not follow the static curve (Zelikson, 1991). However, the line OB can be used to calculate the Elastic rigidity as well as the quasi static graphs.

5 CYCLES IN THE RODS' MODEL

The length of the contact line between the rods was 3 cm. The two pile structure can be taken either as one with a gross base area of 30 x 3 cm or as two separate piles with a net area (4 x 3 cm) x 2. Fig. 12 depicts two cycles of tension - compression. It resembles Fig. 5 of the pile, although the concave part AB is now smaller which indicates a lesser degree of grain re-arrangements.

The maximum force of about 6000 N divided by the net area gives an average pressure of $p = 2.5$ Mpa. The pile's area was 2 cm² and the maximum force of 3000 N in Fig. 6 gives

$p = 15$ Mpa. The corresponding depths are [5 m] and [14 m] respectively. Fig. 13 depicts cycles of different force intervals. The frequency was 1/20 Hz. The tension - compression cycles became stable at about the 20th one. The cycles in compression are similar to those of the pile in Fig. 6.

The restricted space of the paper does not allow the presentation of a larger number of similar curves of the two simulations. Taking this similarity to be the case, the possibility of studying the arrangements and the deformations in the rods model by video becomes relevant.

Cycling of an inclined force approaches the general case of foundation systems in that the number of load parameters is three, all functions of time (the force's amplitude, inclination and eccentricity). Also the zones of large deformation and stresses are quite separated in tension and compression so that the mutual interaction of which is small.

Fig. 14 depicts a cycle and half which is already stable (inclination angle 33°). Fig. 15 depicts ten slow symmetric cycles (period 5 minutes). Fig. 16 depicts twenty quick cycles (period 2 seconds), the maximum compression being 3 times the maximum tension. The cycles of Fig. 14, 15, 16, have similar

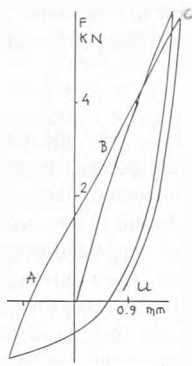


Figure 12 Rods' model - two cycles tension - compression

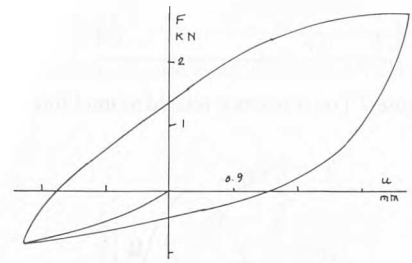


Figure 14 Tension - compression at 33° to the vertical

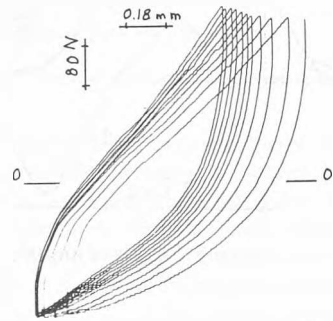


Figure 15 Symmetric cycles; period 5 minutes (force at 33°)

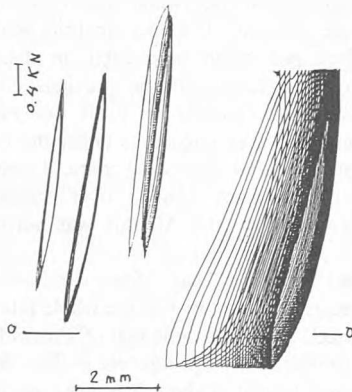


Figure 13 Rods' model cycles tension - compression

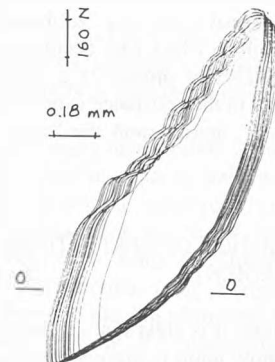


Figure 16 Tension - compression cycles: period 2 seconds (force at 33°)

graphs. Stability exists at the end in all the cases. The short period of fig. 16 had no influence relative to the much longer one of Fig. 14.

6 IMAGE ANALYSIS

In 3 DIM models, when the "soil" is simulated by glass grains, an oil with a suitable refraction index makes the model transparent. One of the pioneers of the method is Allersma (1987, 1991). The thickness where transparency exists depends on the uniformity of the glass. Using Pyrex glass, tests have shown that beyond 6 - 8 cm, a steel ball disappears in the "fog". Hopefully, when grains made of the highly pure glass used in communication cables are available transparent models of about 50 cm diameter would be constructed. The Hydraulic Gradient models would then use the same oil for percolation. For sands the method of "sand - castle" was introduced by the author in 1986 for the centrifuge of LCPC in Nantes (Zelikson

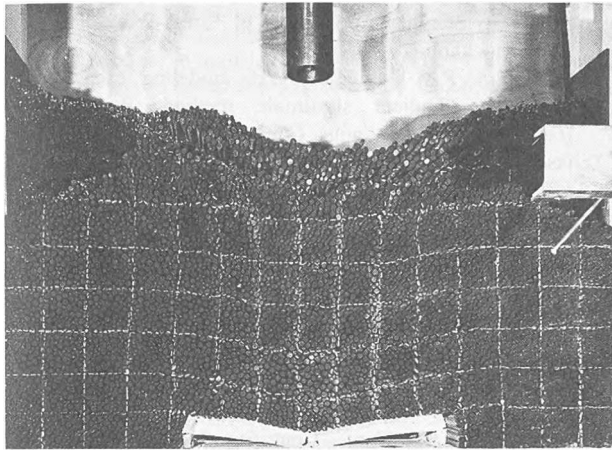


Figure 17 Deformation of a grid rods model (collapse of roof by blast).

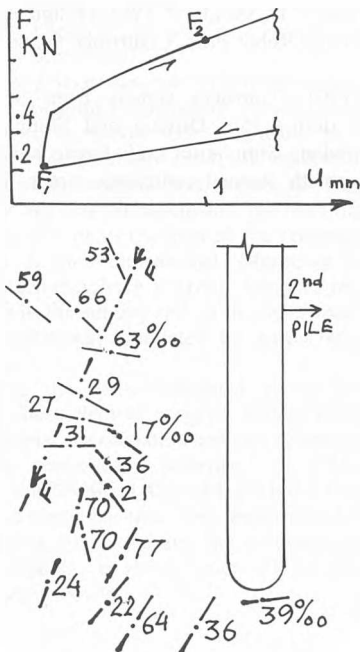


Figure 18 Field of maximum contraction (two piles in the rods' model)

1989, 1991). According to the method thin layers of colored sand are introduced during the construction of the model. After the test the sand is drained but not dried. The moist sand is then sliced showing the deformed pattern of the colored lines. Using a fine sand, the resolution is rather good.

The rods' model was introduced for the purpose of visual inspection. If ordinary cylinders are used than a grid of chalk can very efficiently be used. Fig. 17, which depicts the collapse of a cavity caused by a shock tube blast is an example.

However, at present visual methods are judged by their amenability to image processing. The basic problem of processing is the identification of the same "mark" on successive clichés. The simplest method uses markers, which are good both for 2 DIM and 3 DIM. In the present case they were extensively used in the rods' model. During the test a video film is prepared. At desired time intervals the computer picks clichés. The markers are separated from the background by filters. For that to be successful, the background must be much darker. After trials the best method was found to be the use of paints that glow in the dark for some time after the room is darkened, during the time the film is taken.

Accordingly all the markers and only the markers were picked out. The center of "gravity" of the lighted pixels is then calculated. Using successive clichés the next position of each marker is identified and the displacement calculated. The next step is triangulation. The quick Hull Algorithm for Convex Hulls of C.B. Barber, D.P. Dobkin and H. Hudhanpaa was used. Any 4 points give 2 triangles in 2 different ways, and some triangles are too "flat". The angle between two edges was calculated using the ratio of the vector product of the edges to the scalar product. The unsuitable triangles were deleted. The gradient of displacement increments was calculated by linear interpolation and assigned to the gravity center of the triangle. Either Green's deformation or the linear one was calculated, and then the principal axes and values. Fig. 18 as an example depicts the largest contractions and their axes near the toe of the piles. The force was at 33°. The displacements between F_1 and F_2 were used. The image was composed by 512 pixels. Therefore large zooming was necessary for the second method which followed the motion of each rod in a restricted zone (Zelikson, 1993).

In preparation the cylinders of 3 mm diameters were painted black. The laithed rods do not touch each other. Therefore they appear as bright circular disks on a dark back-ground. The computer programmes menu (installed on a Macintosh) includes a threshold filter which enhances the picture. Using "pencil" "brush" etc the disks which are seen to be in contact are separated. This operation reduces in a negligible way the "masses". The programme then sieves the particles by their area. In the present tests there are only two sizes. Finally it calculates the center of gravity in pixel units, and numbers the particles

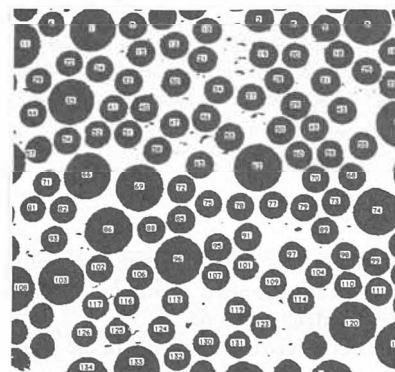


Figure 19 Computer printout of each grain's identification for center's calculation.

TABLE 2 Motion of the rods

Length unit=pixel, 1 pixel≈0.16mm, window: 233 by 189

Initial position			Final position			U _x	U _y
No.	x	y	No.	x	y		
60	283.00	291.00	60	238.00	294.00	0	3
61	367.00	285.00	61	367.00	288.00	0	3
62	471.00	282.00	62	471.00	285.00	0	3
64	609.00	271.00	64	609.00	274.00	0	3
65	701.00	262.00	65	701.00	265.00	0	3
66	153.00	260.00	66	152.00	263.00	-1	3
67	406.00	260.00	67	406.00	263.00	0	3
68	218.00	257.00	68	222.00	256.00	3	-1
69	110.00	251.00	70	109.00	254.00	-2	3
70	313.00	245.00	71	313.00	248.00	0	3
71	479.00	225.00	72	479.00	229.00	0	4
72	655.00	237.00	73	655.00	240.00	0	3
74	351.00	227.00	75	351.00	230.00	0	3
76	142.00	219.00	76	144.00	221.00	2	2
77	203.00	219.00	77	203.00	222.00	0	3
78	95.00	213.00	78	94.00	216.00	-1	3
79	545.00	199.00	79	546.00	202.00	1	3
80	620.00	211.00	81	620.00	214.00	0	3

according to a lexicographic criterion. As seen from figure 19, all the hexagonal rods are accounted for. The "window" was 8 cm large, covered by 500 pixels, i.e. one pixel equivalent to 0.16 mm. Table 2 lists part of the results. The problem of correspondance can be examined as follows: if the displacement is small enough relative to the particle size than the centers of corresponding disks will differ only slightly. In fact in table 2 in most cases the first two digits in both x and y are the same, and the others differ only by 1. This can be the basis an automated routine. The lexicographic ordering gives sometimes consecutive numbers to far away disks. As table 2 shows this is not frequent. Also the sieving filter will drop out clusters of disks in apparent contract. However, the programmes that have been used all contain some interaction usually in the filtering phase. Also errors of pairing have been seen. This indicates that the use of some human intelligence is not altogether undesirable. The average displacement is $(U_x) = 0.08$; $(U_y) = 2.97$ pixels. The standards deviations corespond to 0.13 and 0.05 mm respectively, indicating a precision of about 0.1 mm (or 0.5 pixel).

The second method provides the displacements at the centers of clusters of rods, and then follows the procedure of the first method. As the frame is 512 x 512 pixels, a cumbersome method of camera displacements has to be used, with the ensuing errors. Blocks of several cameras were also used. It stands to reason that in the near future cameras of a much greater number of pixels would be available, and all these technical difficulties disappear. The paper strives to show that in many ways the rods' behavior is similar to that of real sands, especially as concerning work hardening. Therefore any computational model should correctly predict the field of displacements which is available by image processing.

7 CONCLUSIONS

1. The dependence of the static resistance on residual stresses and on grain re-arrangements according to the load history makes it very difficult to define.
2. For the factors of safety the whole life of the structure should be considered.
3. The use is in doubt of calculations based on continuum

mechanics, of parameters such as capacity and of its "prediction" by dynamic tests.

4. Simple static tests and impact vibration tests by Gonin's method seem equally suitable for validation of the design provided large enough drop-masses are used.
5. Static tests and impact vibration tests are not interchangeable.
6. Simple methods of execution and interpretation of the impact vibration tests should be defined by the codes.
7. The methods of image analysis are expected to become major tools for the understanding of the pile - soil interaction, even in the modest environment of the rods' model.

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