The Behaviour of Soils under Transient Loadings

Le Comportement des Sols sous des Charges Momentanées

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Summary

A hydraulic apparatus and special instrumentation were constructed to test triaxial soil samples. Failure was achieved in times as short as 0·001 second. Curves of compressive strength *versus* strain rate were determined for cohesive soils, dry sands, and saturated sands. Transient pore-water pressures were recorded during tests on saturated sands.

Another apparatus was constructed to study wave propagation. Soil samples, 2 in. in diameter and 32 in. long, were struck at one end by a ram. Results for a dry sand were compared with theoretical solutions for the wave propagation problem.

Other tests were devised to study creep and relaxation phenomena in dry sands, and to study the permeability of saturated sands to pressure gradients applied suddenly.

Sommaire

On a construit un appareil hydraulique et un appareillage spécial pour faire un essai triaxial sur des échantillons de sol. La rupture se produisait dans un intervalle aussi bref que 0·001 de seconde. On a établi des courbes montrant la relation entre la résistance à la compression et les vitesses de déformation pour des sols cohérents, des sables secs, et des sables saturés. On a enregistré les pressions passagères de l'eau interstitielle pendant des expériences sur sables saturés.

Un autre appareil fut construit pour étudier la transmission des chocs. Des échantillons de sols, de 5 centimètres de diamètre et longs de 81 centimètres, étaient frappés par un bélier à un bout. Les résultats pour un sable sec ont été comparés avec les solutions théoriques pour le problème de la transmission des chocs.

On a inventé d'autres essais pour étudier les déformations dans le temps de sables secs exposés à une pression constante, et pour étudier la perméabilité des sables saturés aux gradients de pression appliqués subitement.

Between 1951 and 1954 the Corps of Engineers, United States Army, sponsored research into the behaviour of soils under transient loadings. This work was performed by a research team at the Massachusetts Institute of Technology under the direction of the late Professor Donald W. Taylor. Testing techniques and significant results are summarized in this paper.

Transient Triaxial Tests

The purpose of these tests was to study the relationship between shearing strength and rapidity of loading. For brevity, this relationship will be called 'strain-rate effect'. The tests were very similar to those performed earlier at Harvard University (CASAGRANDE and SHANNON, 1948), but included much faster rates of loading. The definition of the phrase 'time of loading' is shown in Fig. 1.

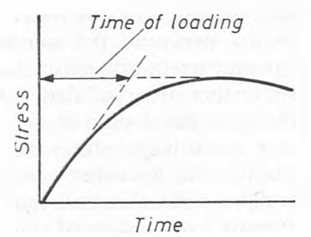

Fig. 1 Definition of time of loading
Définition d'intervalle de chargement

Loading apparatus—The apparatus was designed to meet the following criteria:

(1) Strain-control type of loading to be used.
(2) Very plastic and very stiff soils to be tested.
(3) Time of loading to be as short as 0·005 second.

A standard testing apparatus provided tests with a time of loading of approximately 5 minutes. By re-gearing this apparatus and employing a cinematograph camera record gauge readings, times of loading as short as 30 seconds could also be achieved. A new transient loading machine was designed to provide times of loading between 1 and 0·005 second. This machine utilized the basic hydraulic circuit shown in Fig. 2. Initially, the flow from a positive displacement pump passed through the throttle valve, whose function was to establish the desired initial pressure. By means of the control valve, the oil flow was quickly diverted to push against the loading piston. Since oil was supplied at a constant rate, the piston moved at a substantially constant speed. The piston velocity could be varied from test to test by controlling the discharge from the pump and by varying the area of the loading piston.

The actual machine was too complex to permit a detailed description, and it must suffice to mention a few major features.

(a) A variable displacement piston pump gave oil flows up

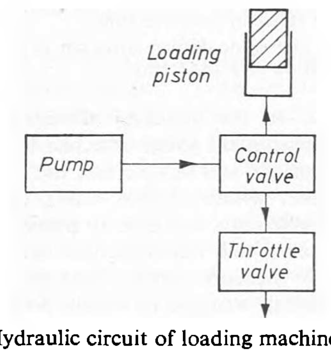

Fig. 2 Hydraulic circuit of loading machine
Circuit hydraulique de machine de chargement

to 40 gallons per minute, with pressures as high as 1000 lb./sq. in.

(b) The control valve was designed to divert the entire oil flow in less than 0·001 second.

(c) A secondary hydraulic circuit, using pressures up to 2000 lb./sq. in., actuated the control valve and auxiliary control devices.

(d) The various control devices were actuated in the proper sequence by electronic timing devices.

(e) The frame housing the control valve and soil test chamber was mounted upon springs to reduce shocks to the building. The frame plus mounting stood 8 ft. high.

The soil test chamber was similar to that used in a standard triaxial apparatus. Lateral pressures of 100 lb./sq. in. or less could be applied, and provision was made for consolidation of samples. Nearly all test samples were 1·4 in. in diameter and 3·5 in. long, although the test chamber could accommodate samples with twice these dimensions.

As expected, compressibility of oil and inertia of the loading piston modified the oversimplified circuit theory presented above, and caused cycles of velocity fluctuation. When the average piston velocity was less than 10 in. per sec., a substantially uniform velocity was obtained. Velocity fluctuations were serious with an average velocity of about 20 in. per sec. When the average velocity increased to 40 in. per sec., failure of the sample would occur during the first quarter-cycle of motion, and serious fluctuations were avoided. The last-mentioned velocity corresponded to a time of loading of 0·001 second. Since satisfactory tests could be obtained at a sufficient variety of loading times, no attempt was made to modify the apparatus so as to damp velocity fluctuations.

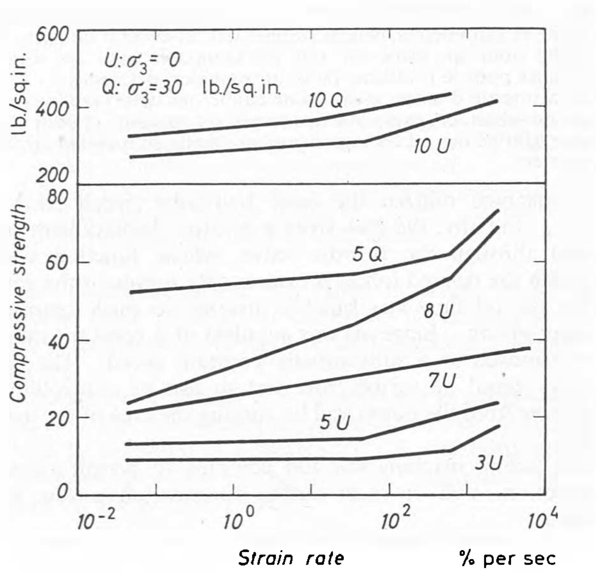

Fig. 3 Strain-rate effect in cohesive soils
Effet de la vitesse de déformation sur la rèsistance à la compression de sols cohérents

Instrumentation—In the majority of tests, instrumentation consisted of a displacement gauge attached to the piston and a load gauge between the soil sample and the fixed reaction. A special transformer, whose output was proportional to the position of a movable core, was used to measure displacements. A ring or a tube, on which were mounted wire resistance strain gauges, served to measure loads. Design of suitable load gauges was a constant struggle to obtain both adequate sensitivity and a sufficiently large natural frequency. The mass of the disc between load gauge and soil sample affected natural frequency, and hence this disc was made as light as possible. Accuracy of load measurements was a real problem when testing very soft soils.

The strain gauges mounted upon the load and displacement gauges were connected into appropriate electrical bridge circuits, whose output was displayed upon the screens of oscilloscopes. The screens were photographed using a camera with an open shutter, and calibration steps were imposed on the same photograph. Oscilloscopes with two independent beams were used, and hence both load-time and displacement-time curves were obtained on the same photograph. A Poloroid-Land camera was employed, giving complete test records within several minutes following completion of a test.

In a few tests an additional load gauge was placed between the loading piston and soil sample. It was necessary to subtract the inertia force of the load cap from the gauge reading to obtain the force upon the soil sample.

Tests on cohesive soils—Results from tests upon four cohesive soils are given in Fig. 3. The numbers on the curves refer to Table 1, which lists all cohesive soils tested in this programme and at Harvard University.

Table 1

Table 1. Cohesive soils used in transient tests
Sols cohérents employés dans les essais passagers

No.	*Description*	*Plasticity index*	*Plastic limit*	*Water content*	*Unconfined strength*[1]	*Strain-rate effect*[2]
		PI	*PL*	*w*	$(\sigma_1-\sigma_3)f$	*S–R*
1	*Normally consolidated sensitive ocean sediment, undisturbed*	63	49	92	0·3	2·0 (4)
2	*Very plastic clay, remoulded*	27	38	48	7	1·6 (6)
3	*Plastic clay loam, remoulded*	17	11	16	10	1·7 (6)
4	*Medium soft slightly sensitive clay, undisturbed*	24	26	27	10	2·0 (4)
5	*Slightly plastic clay loam, remoulded*	23	22	21	13	1·4 (3)
6	*Plastic clay, remoulded*	27	38	44	15	1·7 (6)
7	*Moderately sensitive silty clay undisturbed*	21	22	35	22	1·6 (3)
8	*Impervious compacted fill*	17	11	12	25	1·8 (4)
9	*Tough compacted fill*	41	21	26	35[5]	2·0 (3)
10	*Stiff dry clay, undisturbed*	23	30	20	250	1·3 (4)

Notes:—(1) Unconfined compressive strength in standard slow test.
(2) *True* strain-rate effect between strain rates of 0·03 and 1000 per cent per second.
(3) Determined from triaxial tests.
(4) Determined from unconfined tests at small strains.
(5) Strength with $\sigma_2 = 42$ lb./sq. in.
(6) Determined from unconfined tests on plastic soils.

It was observed that, when a soil was tested both with and without lateral confining pressure, the unconfined samples generally exhibited a greater strain-rate effect than did the confined samples. It was further observed that, when unconfined samples failed by splitting or developing shear planes, the peak resistance occurred at a much larger strain in rapid tests than was the case in slow tests. On the other hand, when samples were subjected to lateral pressures, the strain required to develop peak resistance was almost independent of time of loading.

Thus it appeared that two different time effects were present. The first effect can be likened to that encountered in an extremely viscous fluid, and is associated with continuous, plastic deformation of the soil sample. The second effect is associated with the formation of discontinuities such as shear planes and splits. This effect is analogous to that observed in some metals, where a definite time interval is required to initiate formation of a tensile rupture.

The following statements appear to be valid for almost all soils tested.

(i) When samples were subjected to a lateral pressure, the strain-rate effect was the result of viscous phenomena, and was the same regardless of the magnitude of strain.

(ii) The same situation prevailed in very plastic soils, regardless of confining pressure.

(iii) The same situation prevailed in all soils for strains much smaller than the strain required to develop peak resistance.

(iv) In unconfined samples of a non-plastic soil, the strain-rate effect for maximum stress was partly the result of a time-lag phenomenon.

It is felt that the time-lag phenomenon cannot be relied upon in possible practical applications of strain-rate effect data. Hence the following conclusion can be stated:

> Strain-rate effect should be evaluated from triaxial tests with confining pressure adequate to prevent splitting or shear plane development prior to occurrence of the peak stress.

Data obtained from such a test will present the *true* strain-rate effect, as contrasted to the *apparent* effect determined from unconfined tests. The available information regarding *true* strain-rate effect has been listed in the last column of Table 1.

Tests on dry sands—Three sands, ranging from uniform Ottawa standard sand to well graded sands with high interlocking, were tested. Very loose and very dense samples of each sand were used. All tests were performed at one atmosphere confining pressure.

No significant strain-rate effect could be detected. However, these tests did not disprove the existence of small strength

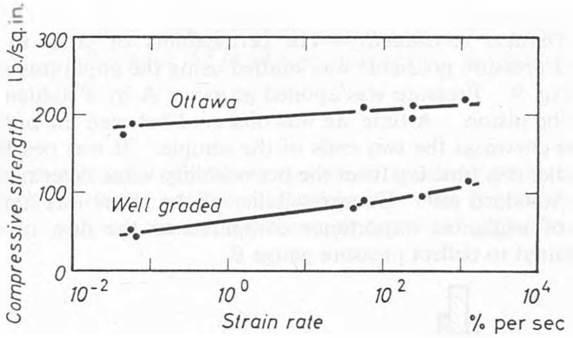

Fig. 4 Strain-rate effect in saturated sands
Effet de la vitesse de déformation sur la rèsistance à la compression de sables saturés

increases, owing to errors in instrumentation and inconsistencies in sample preparation. Hence, carefully performed creep and relaxation tests were used for further study. These tests led to the conclusion that the compressive strength of sands increases about 10 to 15 per cent between times of loading of several seconds and 0·05 second. It would appear that no further significant increase occurs as the time of loading decreases to 0·005 second.

Tests on moist sands—The conclusions for slightly moistened sands were the same as for dry sands.

Tests on saturated sands—Constant volume tests were performed upon samples of uniform Ottawa sand and upon samples of a well graded sand. The voids ratios for the two sands were 0·51 and 0·78 respectively, while the permeabilities were 0·07 and 0·008 cm/sec, respectively. The confining pressure was 60 lb./sq. in., with an initial pore-water pressure of 30 lb./sq. in. Samples were 2·8 in. in diameter and 6·5 in. long. Results of the tests are shown in Fig. 4.

The appearance of a strain-rate effect in saturated sands must result from differences between the pore water pressures in slow and rapid tests. When sand is tested at constant volume, pore water migrates to the central portion of the sample. As the speed of deformation increases, more energy must be expended to overcome resistance to the flow of water. Hence, the effort required to fail saturated samples increases as the rate of loading is increased.

Exploratory measurements of transient pore water pressures were made, using a diaphragm gauge located at one end of the sample. Typical results for the two sands are given in Fig. 5. With rapid loading conditions, this pressure gauge recorded pressures larger than those existing in the zone of greatest strains. This explains the apparent, but probably false, increase in the ratio $(\sigma_1 - \sigma_3)/\sigma_3$ throughout the range of applied strains. Despite this situation, several significant conclusions were obtained.

(1) In both sands, the pore water pressure at a given strain was smaller in rapid tests than in slow tests. This explains the sharper rise, in rapid tests, in the curve of deviator stress $(\sigma_1 - \sigma_3)$ *versus* strain.

(2) In Ottawa sand the pore pressure dropped to absolute zero prior to development of peak compressive strength. This was true in both slow and rapid tests. Thus there was little strain-rate effect when peak stresses were plotted.

(3) In the well graded sand, the pore pressure was smaller, when a failure condition developed, in rapid tests than in slow tests. Thus it was possible for this sand to exhibit the strain-rate effect shown in Fig. 4

A few transient tests with pore pressure measurements were performed on loose samples. There was no evidence of liquefaction.

Transient testing limitations—Two factors place a lower limit upon time of loading which can yield useful results. The first

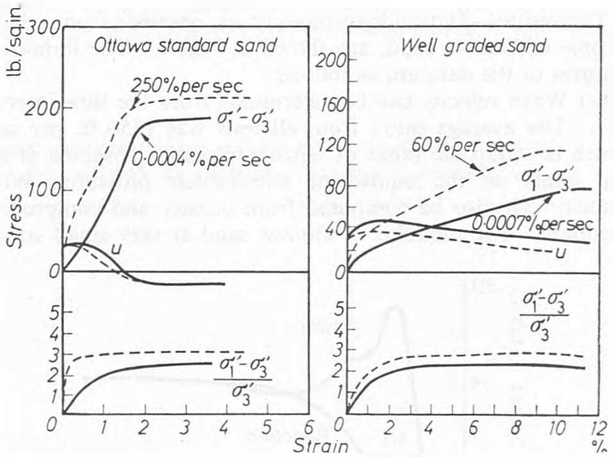

Fig. 5 Transient pore-water pressures
Pressions passagères de l'eau interstitielle

is the appearance of wave propagation phenomena, which results in non-uniform stresses. Time delays caused by wave propagation were observed in the most rapid tests where load gauges were placed at both ends of a sample. The second factor will be called 'lateral inertia effect'. Lateral strains must occur before failure can take place, and in very rapid tests inertia delays the development of lateral strains. Thus it is possible to develop, during very short periods of time, stresses far in excess of the true peak resistance. Both effects increase with increasing sample size, and both were found to be significant for times of loading of one millisecond and less.

Wave Propagation Tests

These tests were designed to permit study of the interrelationship between stress/strain characteristics and wave transmission phenomena. Testing was limited to dry sands.

Testing apparatus—Short-duration loads were applied by a 50 lb. ram striking against one end of a horizontal soil sample. Impact velocities of 100 in. per sec. and less were possible. Samples were 2 in. in diameter, and were as long as 32 in. so that wave transmission patterns would be accentuated. Confining pressure was applied by evacuating air from the interior of the sample.

Samples were supported by slings which permitted longitudinal strains but minimized lateral displacements. Samples were prepared in lengths of 7 or 13 in., and then placed together and membranes overlapped. Special techniques were developed to accomplish joining of samples, but pockets of slightly loose sand at the joints could not be eliminated entirely.

Instrumentation—Instrumentation was generally the same as was used for transient triaxial tests. However, a vastly improved load gauge was developed. This gauge, the details of which appear in Fig. 6, had a natural frequency in excess of

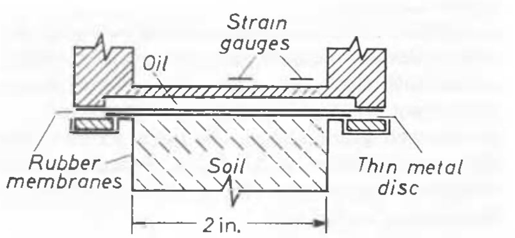

Fig. 6 Improved load gauge
Appareil perfectionné mesurant les charges

8000 cycles per sec. and excellent sensitivity. One such gauge was placed at each end of the soil sample.

Test results—Typical load/time curves, obtained from a dense sample of Ottawa sand, are shown in Fig. 7. The important features of the data are as follows:

(*a*) Wave velocity can be determined from the time interval Δt_1. The average result from all tests was 1350 ft. per sec., which is within the range of seismic velocities observed *in situ* for sands at the equivalent overburden pressure. Wave velocity can also be computed from density and compressive modulus. The modulus of Ottawa sand at very small strains

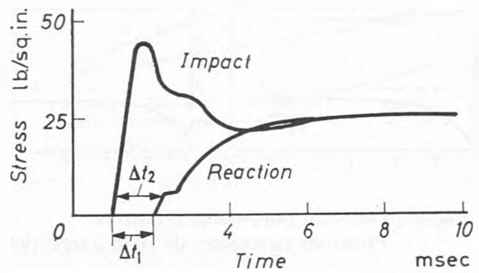

Fig. 7 Typical wave propagation test results
Résultats typiques d'essais de propagation de l'onde

is about 40,000 lb./sq. in. (Chen, 1948), and the computation gives substantially the observed velocity.

(*b*) Using the theory developed by von Karman and Duwez (1950), a curve of impact stress *versus* impact velocity can be computed from a stress/strain curve. Experimental determination of the impact stress was complicated, owing to the appearance of a very high initial stress caused by the 'lateral inertia effect'. Using the impact stress immediately following decay of the initial peak, the comparison shown in Fig. 8 was obtained.

(*c*) In a material with a non-linear stress/strain curve, it would be expected that the time interval Δt_2 at a stress of 5 per cent would be greater than the interval Δt_1. This trend was observed, and the magnitude of Δt_2 could be predicted from the average slope of the stress/strain curve between 0 and 5 lb./sq. in.

(*d*) Following the initial rise of stress at the reaction end, a stress plateau appeared. The magnitude of the stress at this plateau decreased as the sample length was increased. Such a decrease would be expected in a material possessing a strain-rate effect.

(*e*) The average duration of the stress plateau was twice the time interval Δt_1. Such a condition would be expected from the theory for wave reflections.

A complete theory for wave propagation through a non-linear material which also possesses a strain-rate effect is lacking. Furthermore, additional tests are needed to clarify and complete the observations discussed above. However, it was

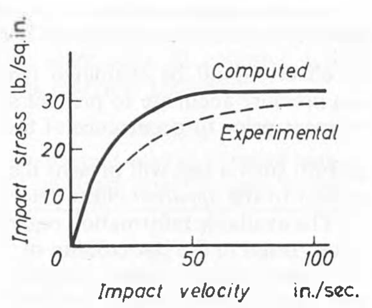

Fig. 8 Impact stress *versus* impact velocity
Tension d'impact ; vitesse d'impact

encouraging to note so many phenomena for which qualitative, and in some cases quantitative, predictions were possible.

Other Tests

Transient permeability—The permeability of sands to transient pressure gradients was studied using the apparatus shown in Fig. 9. Pressure was applied at gauge A by a sudden blow on the piston. A time lag was observed between the pressure/time curves at the two ends of the sample. It was possible to predict this time lag from the permeability value determined by the standard test. Compressibility of the water was found to be of negligible importance compared to the flow of water required to deflect pressure gauge B.

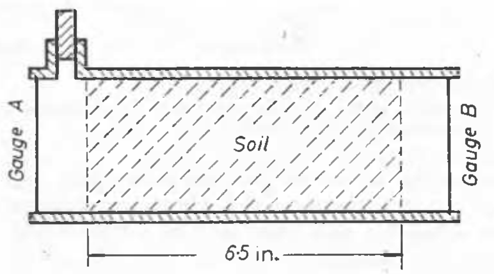

Fig. 9 Transient permeability test apparatus
Appareil pour mesurer la perméabilité passagère

Creep and relaxation tests—In these tests either a constant strain or a constant load was applied to a soil sample, and the decrease in stress or the increase of strain with time was observed. The strain or load was applied in about 0·25 second. Careful attention was given to instrumentation, rigidity of the test apparatus, and preparation of the soil sample. Tests were made on two dry sands, one uniform and one well graded, at one atmosphere confining pressure. Results showed a 10 to 15 per cent increase above the strength observed in slow triaxial tests, and indicated that both sands were characterized by a relaxation time of about 0·2 second.

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