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TECHNICAL NOTE

LARGE-SCALE CYCLIC SHEAR BIN TO EVALUATE METHODS FOR MITIGATING LIQUEFACTION HAZARD

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ABSTRACT

A new test facility for inducing liquefaction in a large mass of saturated sand by cyclic shear is described. The sand specimen is 4 m wide, 6 m long, and about 5 m deep, and the cyclic shear stresses are applied by rotating a pair of walls hinged at the bottom, thereby forcing the sand specimen to deform. Based on elastic stress analysis, the ratio between the cyclic shear stress and the vertical effective stress is nearly constant within the middle part of the specimen, simulating the probable condition in level ground during earthquakes. When liquefaction is induced, the behavior of the sand is similar to that which has been observed in the field during actual earthquakes, and the time histories of excess pore water pressure and shear deformation are also similar to those which have been observed in shaking table tests on saturated sand. Thus, the test facility can induce liquefaction in a large mass of sand at a fraction of the cost of shaking table tests. The large size and sturdy construction of the facility enables one to follow normal construction procedures in preparing realistic test specimens for evaluating the effectiveness of various methods to mitigate liquefaction hazard.

Key words : dynamic, horizontal load, liquefaction, pore pressure, repeated load, sand, special shear test, test equipment (**IGC** : E 8/D 7)

INTRODUCTION

For aseismic design of earth structures and structural foundations, the engineer is often faced with the problem of proposing some methods to mitigate possible damage due to soil liquefaction. Dynamic compaction methods such as sand conpaction piles usually provide the most practical solution from the standpoint of economy and reliability. Their use, however is often precluded because of noise and vibration, and the engi-

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neer is asked to find a less noisy alternative, e. g., gravel drains or bored piles. Because the effectiveness of such methods depends a great deal on the method of installation that cannot be properly simulated in a small-scale model test nor can it be predicted by an analysis, one must have a means to simulate seismic loading conditions in a full-scale or nearly full-scale test in which a realistic construction method can be employed. Such tests would require a soil specimen several meters deep and several hundred tons in weight. Carrying out shaking table tests on such a heavy specimen would be very expensive, and a less expensive alternative would be desirable.

The object of this note is to describe a new method for conducting cyclic loading tests on a large mass of soil for the primary purpose of evaluating methods to mitigate liquefaction hazard.

BASIC CONCEPT

A cyclic shear bin as shown in Fig. 1 was devised based on the assumption that the effect of horizontal ground motion on a level sand deposit could approximately be simulated in a rectangular block of sand if cyclic shear stresses are induced by rotating the hinged walls in unison. This does not guarantee, however, that the stresses are uniformly distributed over the length of the soil specimen, because the horizontal stresses vary from one side to the other and because the complementary shear stresses along the hinged walls cannot be controlled to reproduce the free-field values. Elastic stresses within the soil specimen were computed by the finite element method in order to assess their distribution. Plane strain conditions were assumed by ignoring side friction and change in distance between the side walls, and the hinged walls were assumed rigid and smooth. The physical and mechanical properties of the sand were assumed as follows :

Saturated unit weight of soil
······1.92 tf/m^3 (18.8 kN/m^3)

Young's modulus of soil
···$993\sqrt{\sigma_m'}$ kgf/cm^2 (98.1 kPa)
(σ_m' in kgf/cm^2 (98.1 kPa))

Poisson's ratio······0.48

Fig. 2 shows the finite element mesh and

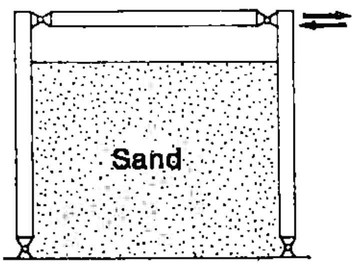

Fig. 1. Cyclic shear bin for liquefaction tests

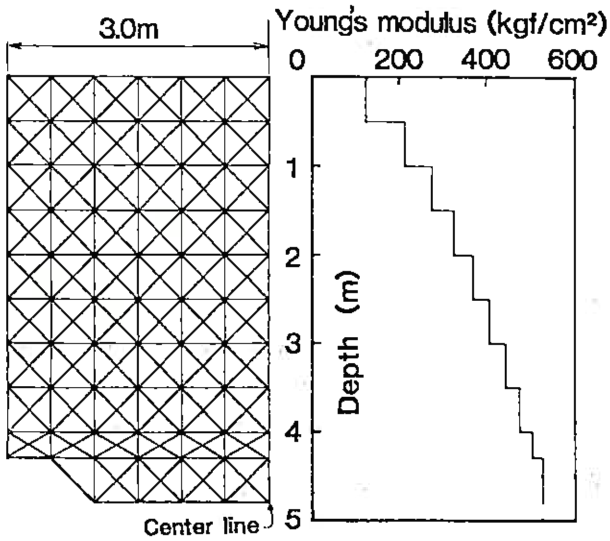

Fig. 2. Finite element mesh and Young's modulus of sand (1 kgf/cm^2=98.1 kPa)

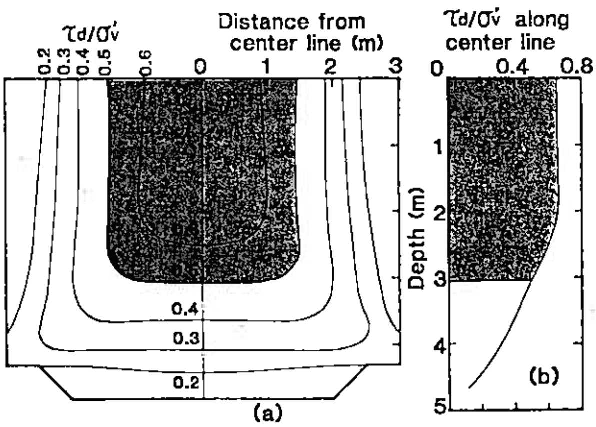

Fig. 3. Shear stress ratio of sand specimen caused by forced shear deformation

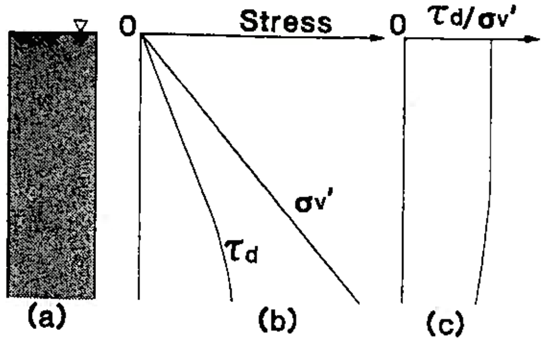

Fig. 4. Probable distribution of shear stress ratio in level sand deposit during an earthquake

the assumed values of the Young's modulus, and Fig. 3 shows the distribution of the ratio between the cyclic shear stress and the vertical effective stress, τ_d/σ_v'. It can be seen that within the shaded zone that covers an area 3 m long and 3 m deep, the shear stress ratio is relatively uniform, being between 0.5 and 0.66. As shown in Fig. 4, the uniformity of the shear stress ratio could be realized in a level deposit of saturated sand because both the cyclic shear stress and the vertical effective stress are nearly proportional to the depth below the ground surface, provided that the ground water table is at or above the ground surface.

EQUIPMENT AND PROCEDURE

The test facility consists of a test bin, a reaction wall, a sand storage bin, a vibratory sieve, a gantry crane, a compressor room, and a control room, as shown in Fig. 5. The test bin which is buried to the rim consists of a rectangular, reinforced concrete box, whose inside dimensions are 4.0 m wide, 8.0 m long and about 5 m deep, and a pair of end walls hinged at the bottom, as shown in Fig. 6. The middle portion of the bin is made deeper than the ends so that the results of soil sounding, e.g., the standard penetration test, are not influenced by the presence of the floor slab. In order to minimize friction between the side walls and the sand, the inside surfaces of the side walls are covered with polyethlene boards whose coefficient of friction is 0.09

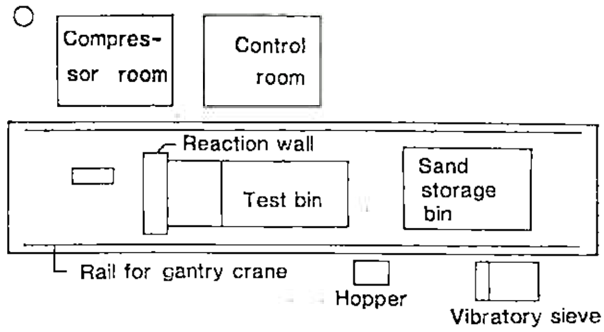

Fig. 5. Layout of test facility

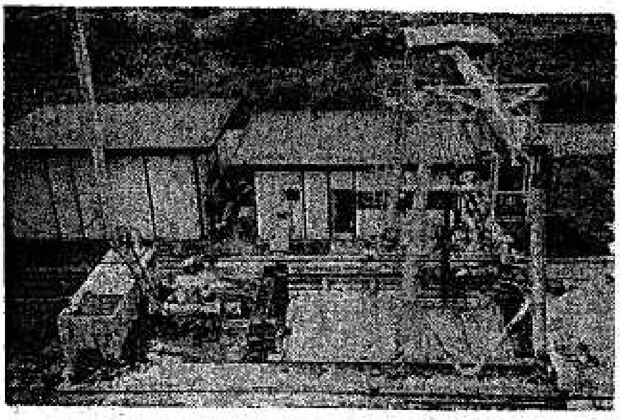

Fig. 6. Longitudinal cross-section of cyclic shear bin

according to the ASTM D 696 method. The end walls are constructed of steel H sections that are covered on the inside with 16-mm thick steel plates. The end walls are connected with a pair of steel pipes at the top, and a hydraulic actuator is installed at the same elevation as the connecting pipes in order to apply horizontal loads via a cross-beam. The specifications of the actuator are as follows :

Maximum cyclic load……35 tf (≒350 kN)

Maximum cyclic displacement

…±10 cm for 20 load cycles at 1 Hz

Maximum cyclic velocity …63 cm/s for 20 load cycles at 1 Hz

Frequency range…………0 to 10 Hz

Wave forms controlled by electro-hydraulic servo valves……Sinusoidal, rectangular, triangular, or irregular

The instrumentation shown in Fig. 7 consists of a displacement transducer and earth pressure cells installed on one of the end walls, pore pressure transducers and a pipe strain gage embedded in the sand, and transducers to measure vertical and horizontal displacements at the ground surface. The displacement transducers are of LVDT type, the earth pressure cells are of wire strain gage type with a diameter of 86 mm and a capacity of 2 kgf/cm^2 (≒200 kPa), the pore pressure transducers are of semiconductor type with a capacity of 2 kgf/cm^2 (≒200 kPa), and the pipe strain gage for measuring the deformation of the soil specimen consists of a 1.8-mm thick pvc pipe, 48 mm in diameter, and wire strain gages spaced 50 cm apart. In order to minimize compliance, the pressure transducers are intended to operate at a small fraction of their capacity, by sacrificing sensitivity. The electrical signals from the transducers are amplified and

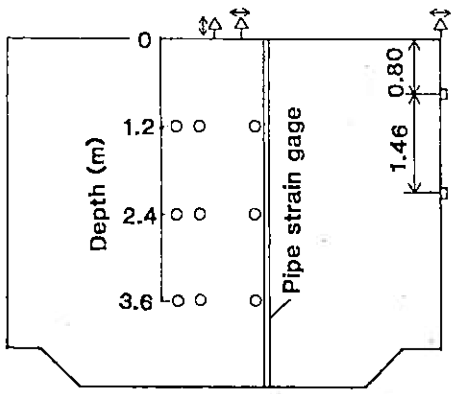

Fig. 7. Location of measuring instruments for a typical test

recorded on magnetic tapes while they are being monitored on a strip chart during the tests. Forty simultaneous measurements are possible.

The gantry crane is used to carry sand from the storage bin to the test bin with a hydraulic clamshell bucket of 0.6-m^3 capacity. In a typical test, constant-amplitude sinusoidal loads are applied in stages, starting with a small load of say 0.5 tf (≒5 kN). The succeeding loads are applied after the excess pore water pressure has dissipated.

Upon completion of a series of cyclic loading tests, the measuring instruments are removed and the sand is returned to the storage bin with the clamshell bucket. When gravel is used with the sand as in the case of tests involving gravel drains, the gravel and sand are separated with the vibratory sieve.

TYPICAL TEST RESULTS

A loose sand specimen was prepared by depositing silica sand No. 6 from Seto, Aichi Prefecture. The saturated sand in the storage bin was carried 0.3 m^3 at a time with the clamshell bucket to the test bin and gently placed in the test bin at 18 equally spaced locations, in such a way that the depth of water was about 50 cm at all times. The method was adopted after comparing six methods from the point of view of degree of saturation, sand density, workability, and economy. It took about two days to fill the test bin with the sand. The physical properties of the sand were as follows:

50% diameter ……………………0.29 mm
10% diameter ……………………0.16 mm
Uniformity coefficient ……………2.0
Specific gravity of soil particles…2.68
Maximum dry density ……………1.584 t/m^3
Minimum dry density ……………1.234 t/m^3

The maximum and minimum dry densities were determined by the JSSMFE Standard Method of Test for the Maximum and Minimum Densities, JSF Standard T 26-81 T (JSSMFE, 1979). The above-mentioned meth-

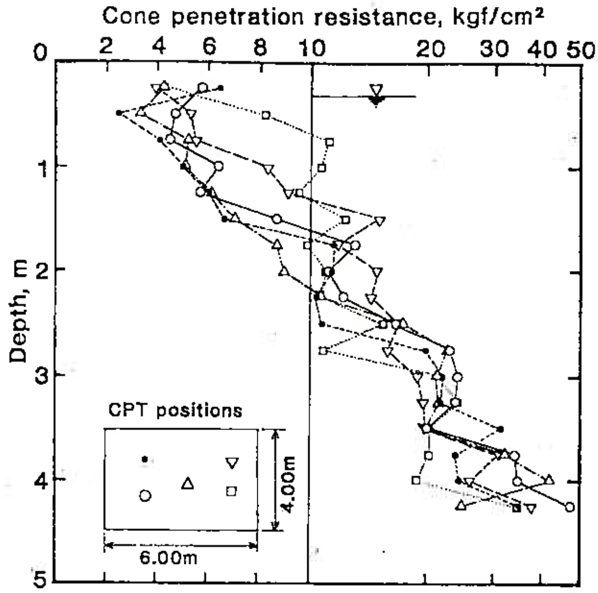

Fig. 8. **Typical results of static cone penetration tests (1 kgf/cm²=98.1 kPa)**

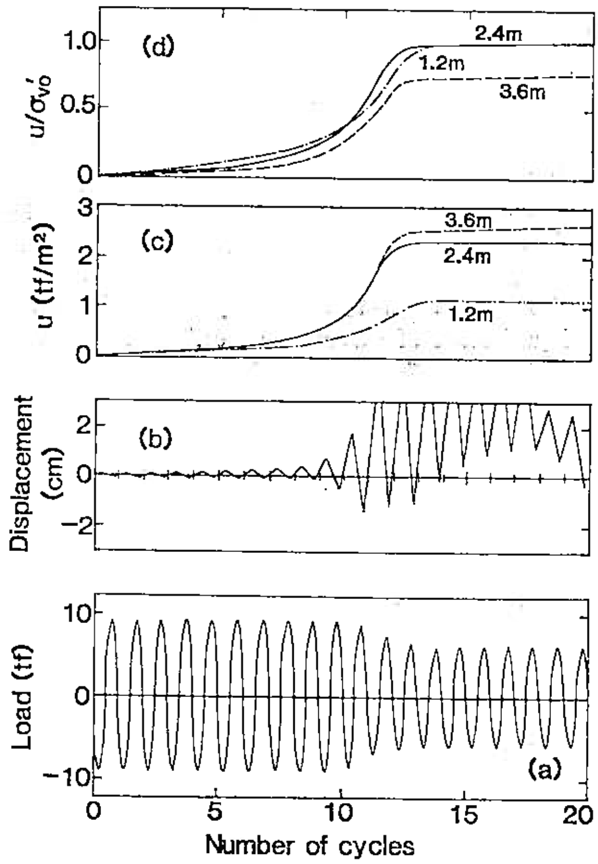

Fig. 9. **Time histories of cyclic load, horizontal displacement, and excess pore water pressure during loading (1 tf=9.81 kN, 1 tf/m²=9.81 kPa)**

od of preparing the sand deposit yielded a relative density of about 66% throughout the depth.

Of various methods of sounding for evaluating the properties of the sand specimen, the static cone penetration test was found most convenient. As shown in Fig. 8, the penetration resistance increases nearly linearly with depth.

Fig. 9 shows the time histories of the cyclic load, the horizontal displacement of the ground surface, and the excess pore water pressure during the application of 20 load cycles. After about 12 cycles when the displacement and the excess pore water pressure increased abruptly, the amplitude of the load dropped somewhat. As shown in Fig. 9(d), the pore pressure ratio reached 1.0 at the depths of 1.2 m and 2.4 m whereas it did not quite reach 1.0 at the depth of 3.6 m, probably because the shear stress ratio in the deeper zone was lower as suggested by Fig. 3.

Fig. 10(a) shows the time histories of the

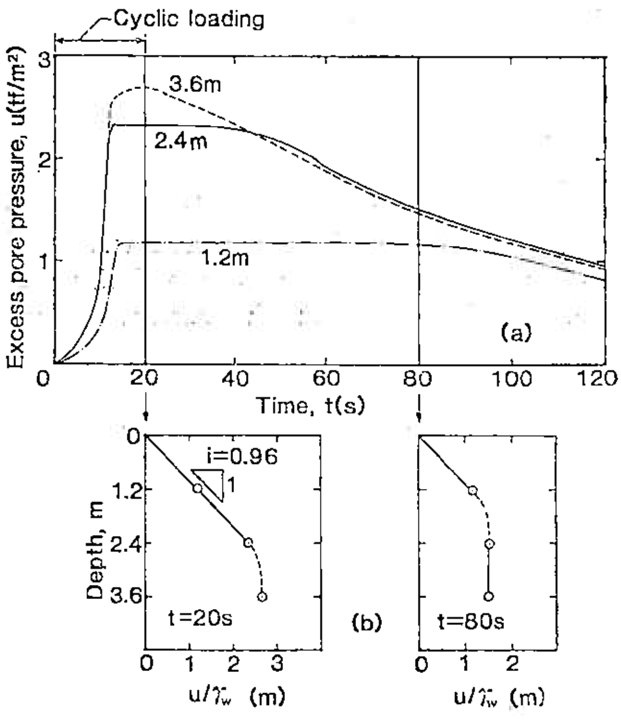

Fig. 10. **Time histories of excess pore water pressure during and after cyclic loading, and vertical distribution of excess pore water head at 20 s and 80 s (1 tf/m²=9.81 kPa)**

excess pore water pressure at the three depths for a period of 120 seconds, and Fig. 10(b) shows the distribution of the excess head, u/γ_w, at 20 s and 80 s. The upward hydraulic gradient of 0.96 as shown in Fig. 10(b) is a reasonable value for the critical hydraulic gradient in a sand deposit. The pore pressure response shown in Figs. 9 and 10 is consistent with that observed in shaking table tests of saturated sand deposits (e. g., Yoshimi, 1967; Tokimatsu and Yoshimi, 1978) in every detail except for the sand near the bottom that was not completely liquefied in the present test. The ground water that had begun to emerge on the surface towards the end of the period of cyclic loading continued to flow up vigorously for quite some time after the loading had ceased, and as a result the whole area was inundated. The phenomenon was identical to what had been observed in the field during and after actual earthquakes. Thus, it was evident that the cyclic loading bin could successfully simulate the essential features of soil liquefaction due to earthquakes.

CONCLUSIONS

The large-scale cyclic loading facility is capable of inducing liquefaction in a large mass of saturated sand at a fraction of the cost of shaking table tests. The large size and sturdy construction of the facility as well as the fact that the surface of the test specimen is nearly level with the surrounding ground should enable one to follow normal construction procedures in preparing realistic test specimens for evaluating the effectiveness of various methods to mitigate liquefaction hazard.

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