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Pore water pressures and settlements due to cyclic footing loads on a saturated sand

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ABSTRACT: An eccentrically loaded footing arrangement has been used to model foundation elements subjected to cyclic moments such as might be experienced during an earthquake or during cyclic loading of a structure. A uniform sized silica flour, having an average grain size of 0.017 mm, was used in the test tank to provide similitude between the drainage and consolidation behaviour of the model footings and typical prototype foundation elements situated on a saturated sand. Pore water pressures beneath and around the footings were measured using hypodermic tubing and small compliance electric transducers. Footing displacements were monitored using displacement transducers. Cyclic loading triaxial tests were also conducted to show that the silica flour exhibited cyclic mobility phenomenon similar to those of a saturated fine sand.

Analyses of the model tests indicate that cyclic settlement of the foundation element developed when the excess pore water pressure in the material outside of the zone of influence of the footing approached about 50% of the total overburden pressure. The model tests indicated that the cyclic stress level had to exceed the initial footing stress in order for cyclic mobility to be initiated. The model test facility described in this paper provides an opportunity for direct observations of pore water pressures and settlements of foundation elements during cyclic loading. The potential for avoiding severe settlements by using soil improvement techniques such as chemical cementation or artificial drainage could be studied in the model test facility.

INTRODUCTION

Following a decade of triaxial and simple shear testing to determine the nature of liquefaction and cyclic mobility phenomenon in saturated sands (Seed and Lee, 1966; Castro, 1969), research in this area appears to have continued in two complimentary directions: (1) elemental testing, where uniform cyclic stresses are applied to a single undrained or partially drained element of saturated sand in order to examine in detail the factors controlling pore water pressures and deformtions (Castro and Poulos, 1977; Martin et al, 1975; Castro et al, 1982); (2) model testing, where larger volumes of saturated sand are subjected to cyclic boundary dsturbances under internal and/or boundary drainage conditions (Finn et al, 1970; DeAlba et al, 1976, Yoshimi and Tokimatsu, 1977). More recently, geotechnical centrifuges have been used for liquefaction studies (Rowe et al, 1977, Morris, 1981 ; Heidari and James, 1982; Whitman and Lambe, 1982).

The research presented in this paper is from model studies although some triaxial test data used for model material evaluation is included. These model studies differ from previous models in that the applied stresses were localized, by using cyclic footing loads, rather than generalized by subjecting the entire model to cyclic acceleration. The present models are designed to simulate foundation elements subject to a rocking motion due to either applied cyclic loads or ground motions.

MODEL DEVELOPMENT

The model studies reported by Yoshinmi and Tokimatsu (1977) and by Whitman and Lambe (1982) were concerned with pore water pressure and settlements beneath a model footing which rested on a bed of sand that was subjected to lateral accelerations. The former studies used a relatively loose sand while the latter used medium dense sand and was done in a centrifuge. The model studies reported in this paper used a cyclic rocking motion on a model foundation element supported by a relatively loose model sand. Internal dissipation of excess pore water pressures generated beneath the footing was a primary consideration in these model studies. A comparison of the results of different types of model studies is important in gaining insight into the soil-structure interaction associated with earthquake loadings.

Model studies were carried out in a sealed aluminum tank 0.30 m wide by 0.38 m long by 0.47 m deep, having a 50 mm thick clear plastic front as shown on Figure 1. Bottom drainage was provided through a drainage valve connecting with a 20 mm thickness of geotextile covering the bottom of the tank.

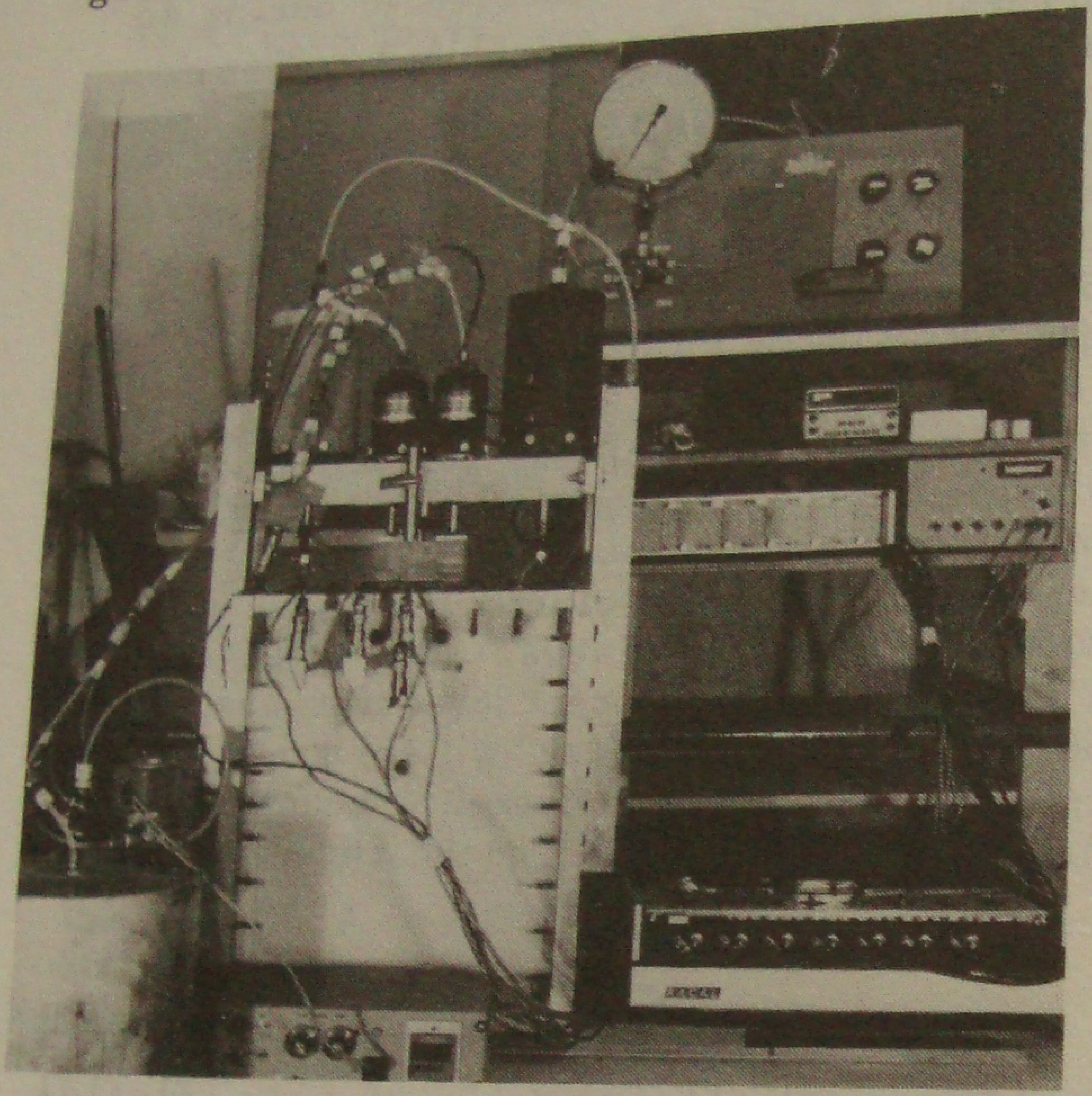

Figure 1 - Model test Apparatus

A relatively deep (four times the model footing width) model sand profile was created above the base drainage layer. The 120 mm square model footing was fitted into an aluminum top plate such that settlements and rotation of the footing could develop under eccentric cyclic loading. A thin rubber membrane was placed over the soil profile before the top plate and footing were positioned. This membrane prevented movement of soil or water into the spaces between the footing and top plate. Surcharge loads were applied to the top plate and footing so that the models represented footings at some depth below a slab. The surcharge loads were controlled by the air pressure in the two 100 cm² air cylinders located near the sides of the loading frame on Figure 1 while four smaller (25 cm² area) air cylinders were used to apply the initial footing stress as well as the eccentric cyclic loading. Pore water pressure transducers mounted on the front of the test tank seen on Figure 1 were connected by 1.0 mm O.D. stainless steel tubing to small saturated geotextile tips embedded in the model sand. Two direct current differential transformers (DCDT) were used to monitor the footing settlement along the centre of rotation. Data from the instruments were recorded on a Racal seven channel data acquisition tape recorder which can be seen on Figure 1. The air tank, 3-way solenoid valve and timer/counter seen in the lower left of Figure 1 were used to control the cyclic footing stresses.

MODEL MATERIAL

The white model sand seen in the test tank on Figure 1 is a commercially available silica flour which was decanted to produce an average grain size of 0.017 mm, a D_{10} value of 5 micron and a coefficient of uniformity of 4. Typical particles were found from scanning electron micrographs to be quite angular. The material is best classified as a non-plastic silt. Minimum and maximum void ratios for this material, by wet determinations, were found to be 0.41 and 1.19 respectively. Falling head permeability tests indicated a hydraulic conductivity of $k = 3.5 \times 10^{-7}$ m/s at $e = 0.80$. This void ratio, representing a relative density of 50%, was based on the average achieved in preparing the test models. Oedometer testing indicated a coefficient of volume change of $m_v = 1 \times 10^{-4}$ m²/kN for this material at a relative density between 45% and 55%. To provide similitude in the degree of internal drainage between a model and prototype, the dimensionless characteristic, $kt/m_v \gamma_w d^2$, where d is the drainage path length, should be similar. In typical uniform fine sand the values of m_v and k, while of different units, are often quite close to the same numerical value. Then, for the same loading frequency, the 0.12 m model footing represents, in terms of drainage times, a 2 m to 3 m prototype footing in a uniform fine saturated sand. The model sand, therefore, can be used to investigate the dissipation of pore water pressures under typical foundation elements on uniform fine saturated sands.

ELEMENTAL TESTING OF MODEL SAND

Static and repeated loading undrained triaxial tests were carried out on the model material at a void ratio close to 0.80. All tests were done under a 200 kPa back pressure. Static tests were carried out at an axial deformation rate of 0.15 mm/min. These samples reached a peak stress ratio, with the pore water pressure attaining values close to 40% of the confining pressure, at axial strains of less than 5%. This was followed by dilative shear which caused the effective stress path to follow the Mohr-Coulomb failure line at $c' = 0$, $\phi' = 35°$.

Repeated loading triaxial tests were carried out at a loading rate of 1 Hz. Failure was not attained for stress ratios, $\Delta\sigma_1/\sigma_c$ less than 0.5 and 400 loading cycles. Figure 2 shows peak pore water pressure buildup for tests that failed under $\Delta\sigma_1/\sigma_c$ = 0.5. Tests 1 and 3 had σ_c = 50 kPa while tests 2 and 4 had σ_c = 100 kPa. True liquefaction (u/σ_c = 1.0) did not occur in these samples but severe cumulative strains (cyclic mobility) developed. The main purpose of these tests was to show that the model material behaved like a sand and was susceptible to cylic mobility as a result of excess pore water pressure buildup and softening of the sample.

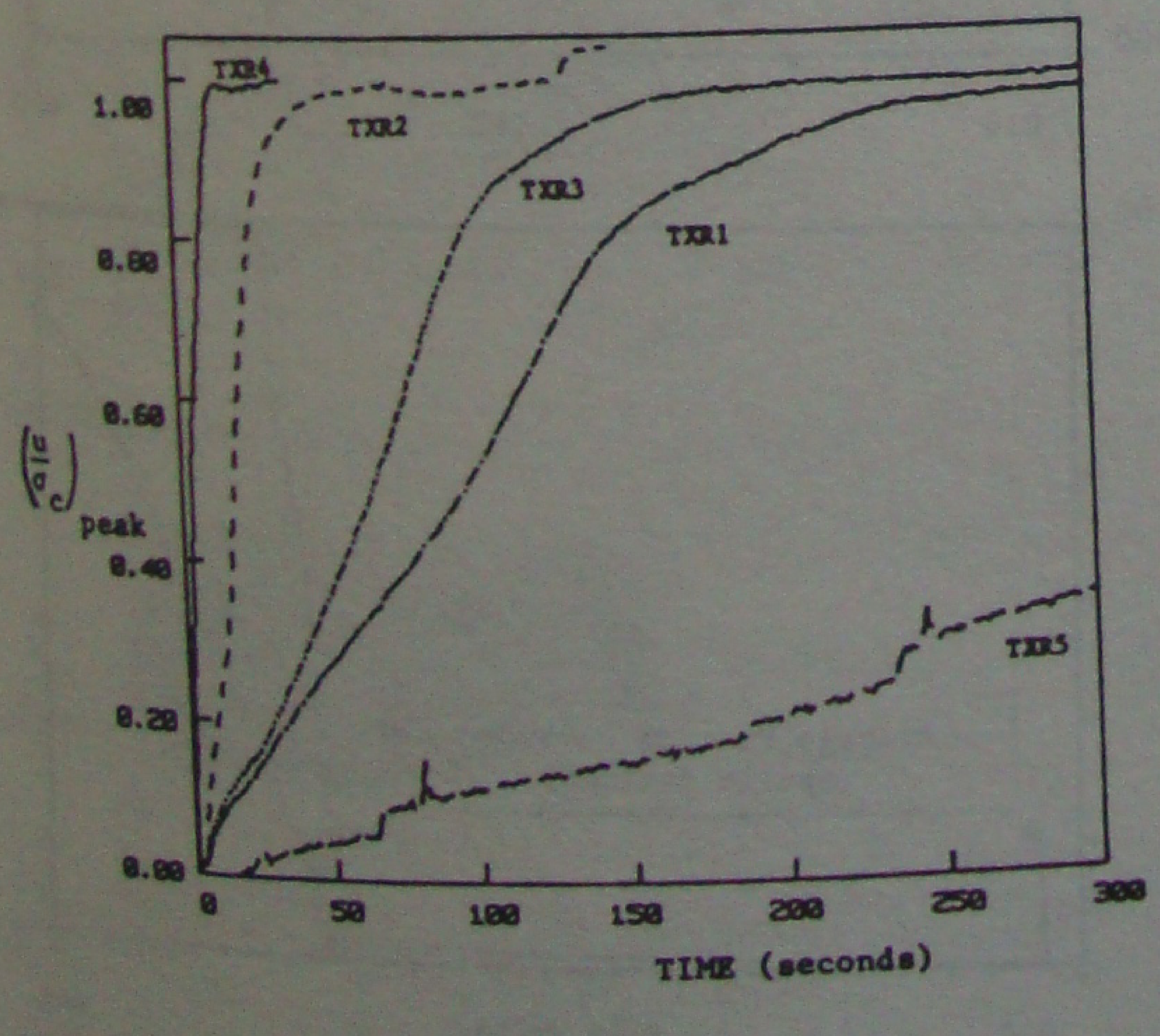

Figure 2 - Repeated Load Triaxial Data

MODEL TEST PROCEDURES

Models were prepared by mixing and pouring the model sand into the box at 37% water content (void ratio of unity). This material was allowed to settle overnight with the base drainage open. The membrane, top plate and footing were then placed on the surface and the material was consolidated under a uniform vertical confining pressure, σ_v. The footing pressure was increased to the initial value, σ_f, in small equilibrium increments and was then allowed to stabilize for 24 hours. The base drainage was then closed. Footing rocking was initiated by alternating the low and high pressures in the pairs of small air cylinders on each end of the footing. The cyclic stress range, $\Delta\sigma$, is taken to be the low and high footing stresses that the low and high pressures would create if applied in all four belloframs. Most of the tests were carried out using a cyclic stress range, the average of which equalled the initial footing pressure. Four pressure transducers were used to monitor pressures at the tips of probes placed on the model sand as the samples were prepared. Pressures were monitored at locations 36 mm and 90 mm below the footing centreline and at two other points at 36 mm depth - beneath the edge and 36 mm outside the edge of the footing. Two DCDT's were used to monitor the settlement along the centre of rotation of the footing. The recorded values were closely similar in all cases and the average value was taken to be the averge footing settlement, Δ , for the test. Further details on model test preparation and procedures are reported by Walker (1985).

MODEL TEST RESULTS

The results of model tests summarized in Table 1 fall into three categories: category A tests did not experience cyclic mobility, category B tests all exhibited cyclic mobility and category C tests exhibited initial sudden deformations followed by stabilization. The results of these tests are presented below.

Category A tests did not experience cyclic mobility even though the rocking of the

Table 1. Summary of model tests

Category	Test #	e	RD (%)	σ_v (kPa)	σ_f (kPa)	Cyclic Stress $\Delta\sigma$ (kPa)	$\Delta\sigma/\sigma_f$	σ_{max}/σ_f
A	A1	0.76	55	10	50	50–75	0.5	7.5
	A2	0.79	51	10	50	50–75	0.5	7.5
	A3	0.81	49	20	100	100–175	0.75	8.8
	A4	0.80	50	20	50	25–75	1.0	3.8
	A5	0.76	55	30	50	25–75	1.0	2.5
B	B1	0.80	50	10	50	50–75	0.5	1.5
	B2	0.79	51	10	50	50–100	1.0	2.0
	B3	0.82	47	10	100	50–150	1.0	1.5
	B4	0.81	49	30	50	25–75	1.0	1.5
	B5(1)	0.80	50	30	75	50–100	0.67	1.33
	B5(2)	0.80	50	30	75	25–125	1.33	1.67
	B6	0.79	51	10	50	25–75	1.0	1.5
C	C1	0.77	54	10	50	25–75	1.0	1.5
	C2	0.81	49	20	100	50–150	1.0	1.5

footing lasted over 500 cycles. Static footing penetration testing was conducted following the cyclic loading in an attempt to determine the footing bearing capacity. The entire time record for test A2 is shown on Figure 3. Pore water pressures are normalized with respect to the vertical confining stress, σ_v, and settlements are normalized with respect to the footing width, L. From Figure 3 it can be seen that the static loading resulted in a significant negative pore water pressure accompanying the rapid footing penetration under increasing stress. This was followed by a buildup of pore water pressure as deformation continued under the maximum applied footing stress. The maximum peak normalized pore water pressures ranged from 0.07 to 0.34 in category A tests and were generally only 25% higher directly beneath the footing than beneath the surcharge plate outside the footing influence area. In all but one of these tests the accumulated cyclic settlement was less than 5 mm ($\frac{\Delta}{L} < 0.05$) and the pore water pressures had stabilized within 100 cycles of stress application. It is believed that segregation during the preparation of samples in category A created a model soil profile where the hydraulic conductivity increased with depth and that internal

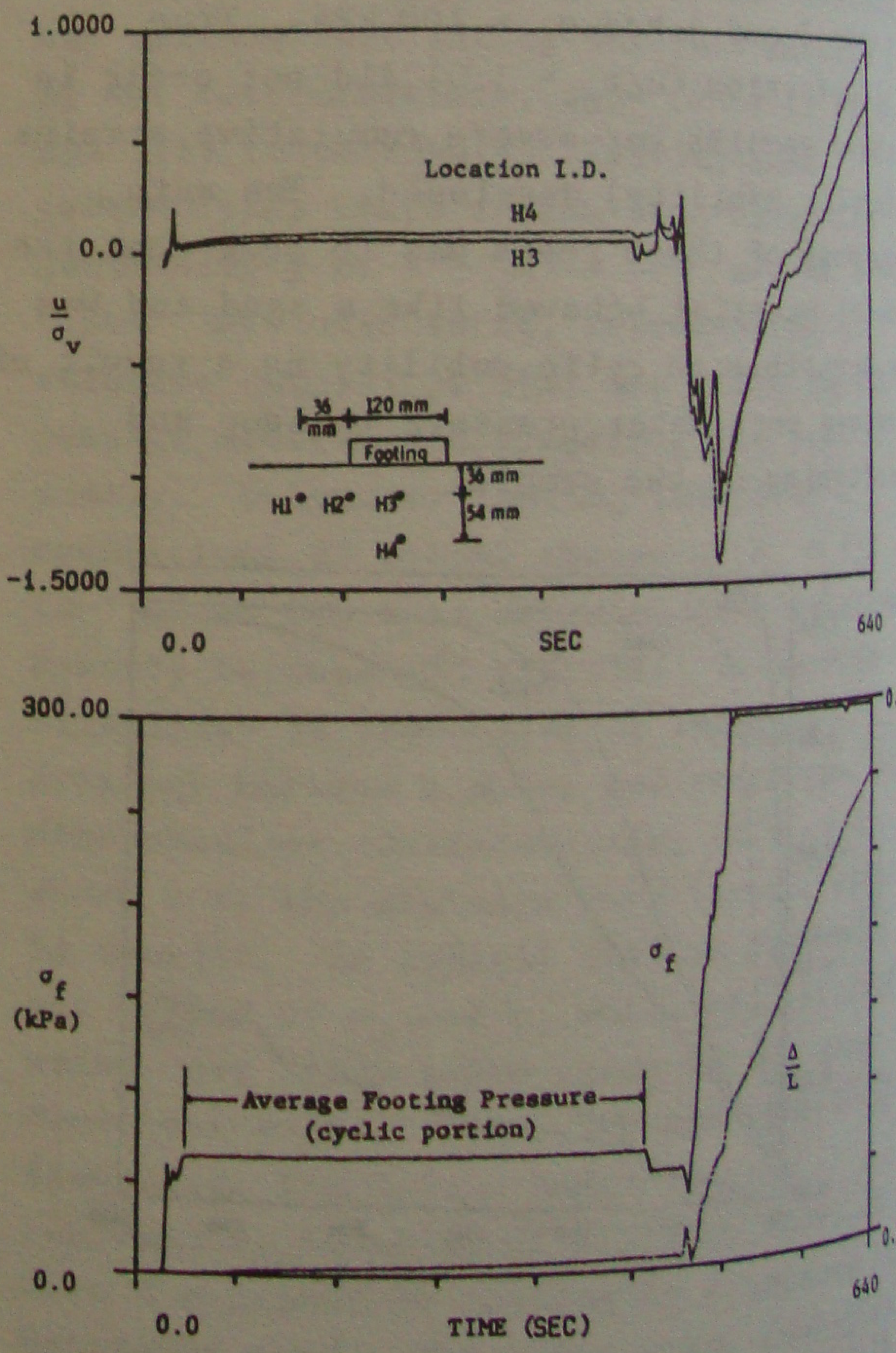

Figure 3 - Test A2 Time Record

drainage allowed excess pore water pressures to dissipate fairly quickly. Air entrainment is another possible explanation for the low excess pore water pressures.

New mixing and pouring procedures used for the category B tests (see Walker, 1985) resulted in a longer preparation time (each 10 mm pour layer being allowed to settle for about 30 minutes) but a much more uniform and better saturated model profile. No segregation could be seen through the clear plastic box front and miniature vane tests were not able to define any layering in the upper profile. Category B tests all underwent cyclic mobility. Figure 4 shows the entire time record for test B4. It is interesting to note that the pore pressures measured at locations outside the footing are almost identical with those of centreline locations. It appears, from the data on Figure 4, that cyclic mobility commenced after 15 seconds when u/σ_v was about 0.4. The apparent end of this settlement, after 21 cycles and at u/σ_v

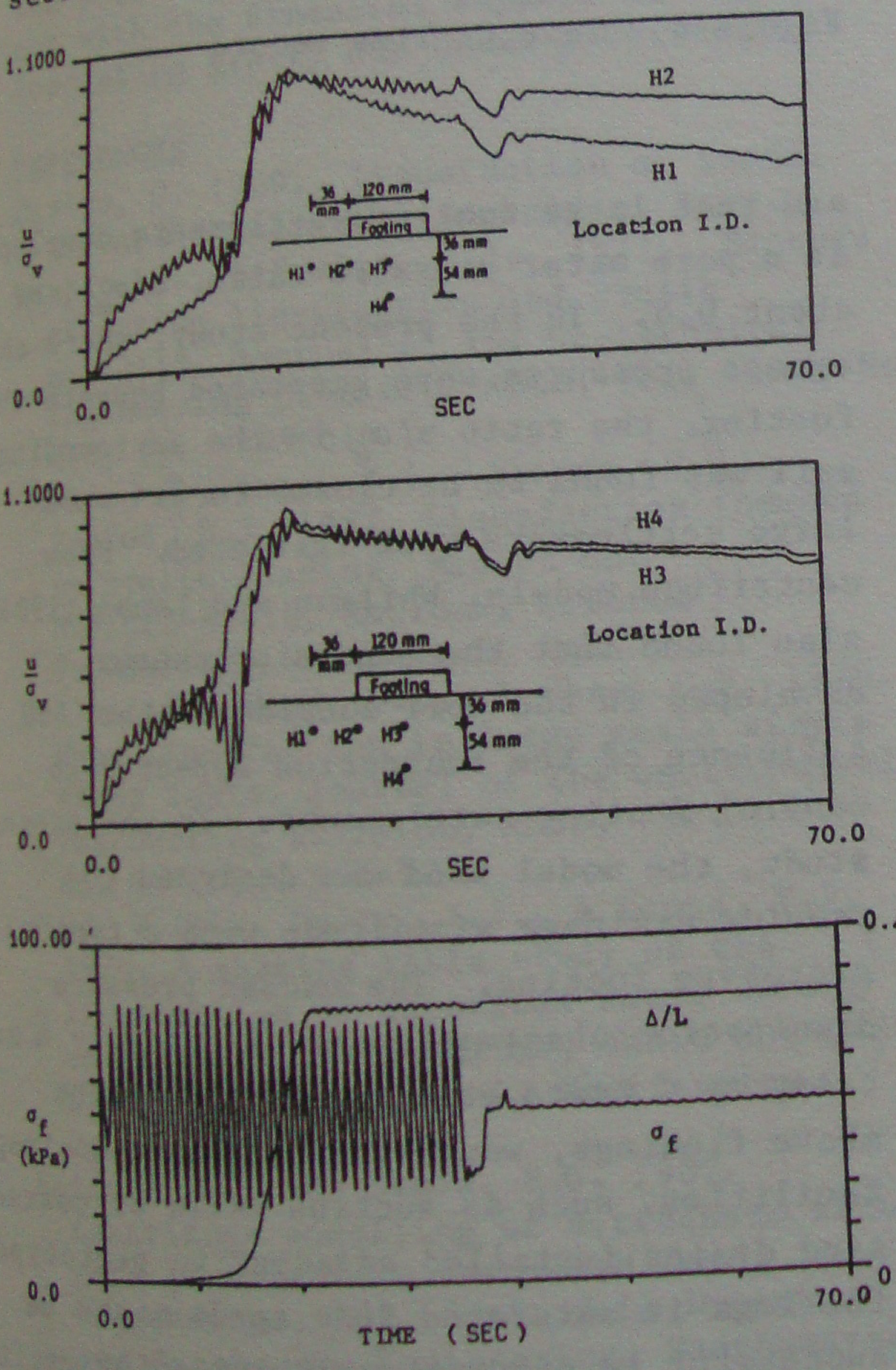

Figure 4 - Test B4 Time Record

close to unity, occurred because the 40 mm stroke limit on the air cylinder was attained. Test B4 is typical of the category B tests with the exceptions of test B1 which exhibited only moderate settlement and test B5, the time record for which is shown on Figure 5. In this test the initial cyclic footing stress range of 50 to 100 kPa was discontinued after 350 cycles because the excess pore water pressures were diminishing. Large deformations resulted within 30 loading cycles when the cycling was reinstated using a stress range of 25 to 125 kPa.

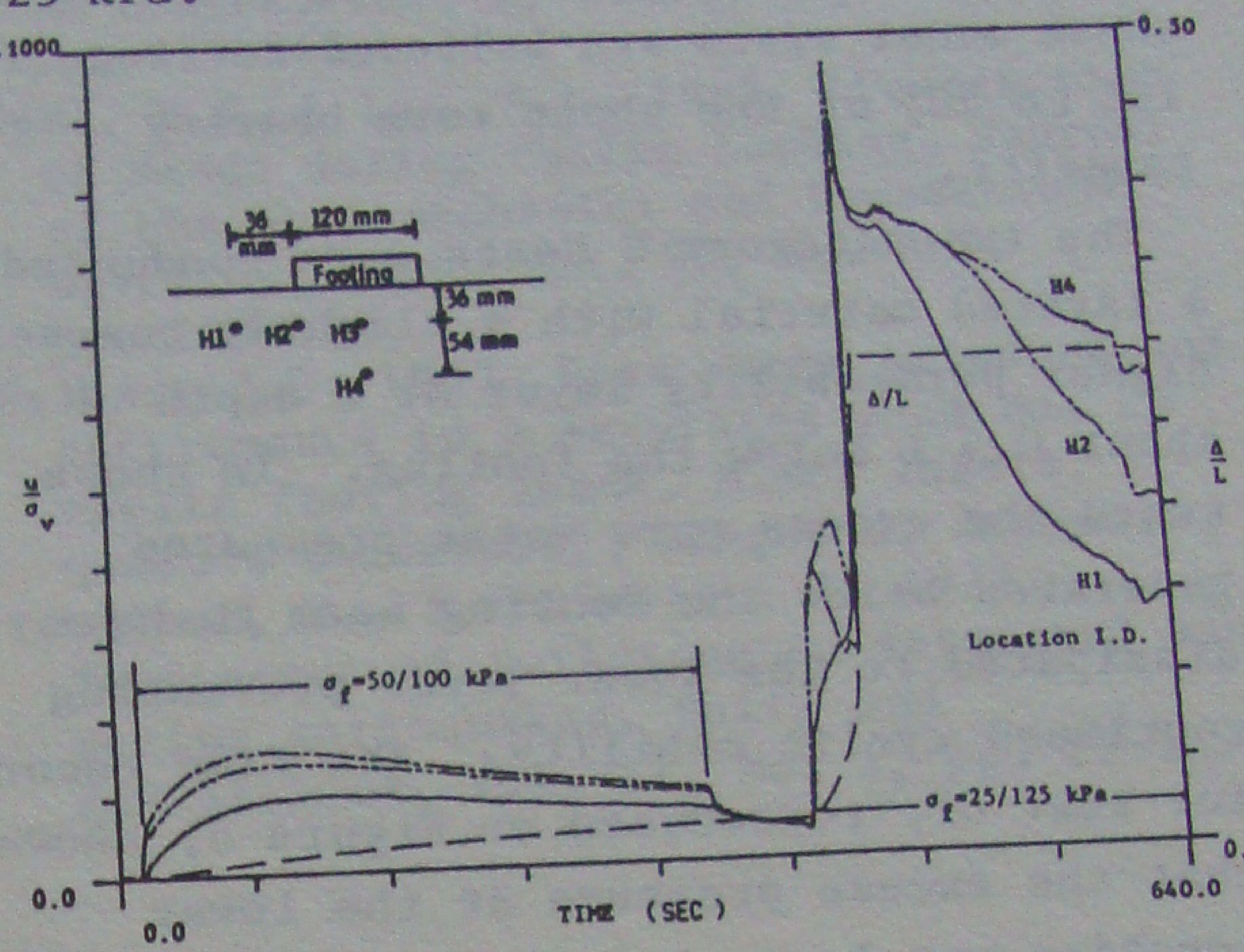

Figure 5 - Test B5 Time Record

Large cumulative deformations or cyclic mobility was initiated in the category B tests at peak pore water pressure ratios of 0.3 to 0.5. If this ratio was expressed in terms of the lateral confining pressure instead of σ_v, it would likely be close to twice the numerical value and, hence, comparable to the ratio, u/σ_c , of about 0.8 needed for triaxial cyclic mobility. The most surprising result is that the peak pore water pressures directly beneath the footing were not significantly higher than those in the soil surrounding the footing. In fact the highest ratio of u/σ_f was about 0.25 prior to cyclic mobility and 0.6 during cyclic mobility. The pressure at the deeper measurement location was consistently higher than directly below the footing. These observations indicate that lateral drainage developed relatively quickly, leading to high excess pore water pressures and cyclic mobility in the surrounding soil mass.

The behaviour observed in tests B1 and B5 when compared with the other category B tests would indicate that cyclic mobility is to be expected only if the cyclic stress range is equal to or greater than the initial footing pressure and where the maximum cyclic stress level is at least 1.5 times the initial footing pressure. In these loose sands the void ratio immediately below the footing is determined by the initial applied footing pressure and, hence, the short term undrained bearing capacity is a function of the initial applied footing pressure. The initial footing stresses in these model tests are believed to range from 15% to 30% of the short term bearing capacity.

The two category C tests were conducted on a layered material with a slightly coarser, higher permeability layer at a depth of about 0.5 L below the footing. In these tests the excess pore water pressures generated below the footing seem to have dissipated to the lower layer preventing continued cyclic mobility. The time record for test C1, reproduced on Figure 6, shows that the excess pressure at the lower location was less than at any other measurement point. A sudden settlement of the footing took place after 52 cycles of loading and the maximum excess pressure reached a value of about 22 kPa, nearly equal to the minimum cyclic pressure of 25 kPa. At this time the excess pressure outside the footing was about 0.5 σ_v but it quickly attained a value equal to σ_v. Then, equally quickly, the excess pressures dissipated and the footing settlement rate rapidly decreased. It appears that internal drainage caused the cyclic mobility to be arrested. The last 30 seconds of the data on Figure 6 are the results of a rapid static bearing capacity test on the settled footing.

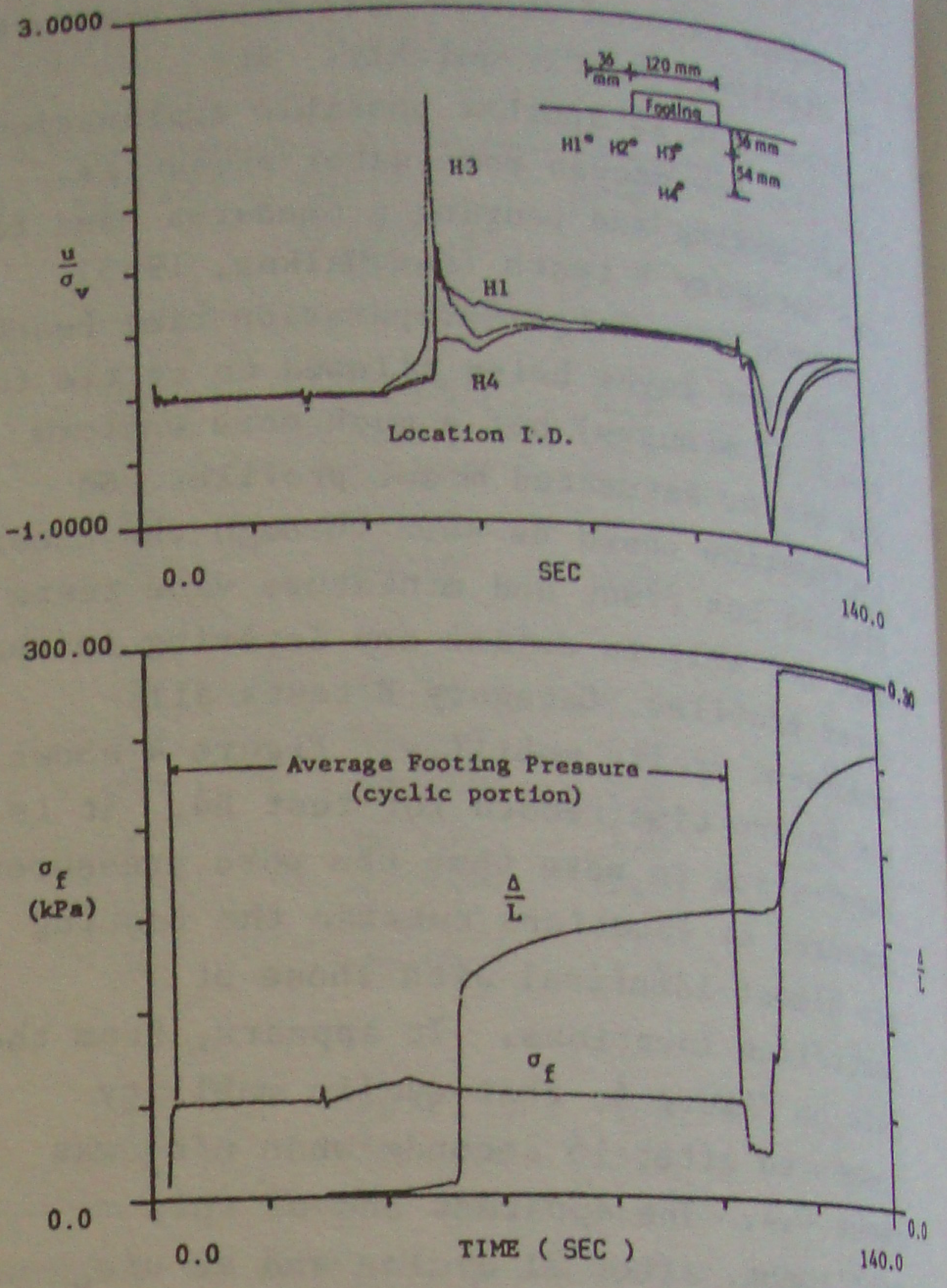

Figure 6 - Test C1 Time Record

DISCUSSION AND CONCLUSIONS

Yoshimi and Tokimatsu (1977) showed that the excess pore water pressures under a model footing resting on a bed of sand that was subjected to cyclic lateral accelerations were less than those in the surrounding soil and that large footing settlements developed at a pore water pressure ratio, u/σ_v, of about 0.6. In the present study, where the excess pressures were generated beneath the footing, the ratio u/σ_v in the surrounding soil was found to be closer to 0.4 when large settlements were initiated. From centrifuge models, Whitman and Lambe (1982) also found that the excess pressures developed in the soil outside of the influence of the foundation appeared to control footing settlements. In the present study, the model sand was designed to provide drainage similitude with a typical prototype footing. The excess pressure dissipation observations from category A and category C tests when combined with the above findings, would indicate that drainage facilities, such as suction wells or coarse sand drains installed adjacent to prototype footings in saturated fine sands might be successful in preventing or arresting cyclic mobility and large settlements under earthquake frequency loadings. This interpretation supports a similar suggestion by Mitchell and Dubin (1986). Other foundation treatments, such as chemical cementation, might also be studied -- the modelling techniques outlined in this paper would be useful in such studies.

The model studies show that cyclic mobility of sand surrounding a footing subjected to a cyclic rocking motion is to be expected if the peak excess water pressures in this sand attain values of about 40% of the overburden pressure. It would appear that the actual value of the overburden pressure is not a controlling factor but that, for footings with static factors of safety greater than three, cyclic mobility may develop if the cyclic stress range is greater than the initial footing pressure and the maximum cyclic stress is at least 1.5 times the initial footing pressure.

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