APPROXIMATE PREDICTION OF SOIL DEFORMATION UNDER DRAINED-REPEATED LOADING

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ABSTRACT

Resulting from application of irregularly repeated loading over a long term period, an appreciable plastic displacement has often been observed in grounds and earth structures. In the present paper, a method for predicting this kind of plastic strain of soils under repeated loading is pursued using triaxial and modified oedometer tests. In the first half part of the paper, time-dependent deformation characteristics were examined through the repeated triaxial tests on a compacted sandy soil. Test results show that the deformation of soil due to repeated loading is fairly attributed to the plastic strain. Except at the beginning and in the vicinity of the failure region of number of load cycles, the ratio of plastic strain to total strain is extremely increased in the logarithm of number of load cycles. In the latter part, specially devised oedometer tests and triaxial creep tests under repeated loading were performed on various kinds of remolded and undisturbed cohesive soils. Arranging these test results, an approximate law was found out for the plastic strain performance. This law seems to be independent of the kind of soils and the loading conditions, and, therefore, to be common with the deformation of compacted sandy soil previously used. A method for predicting the converged strain for soil specimens under repeated loading is proposed making use of this simplified law. Since the converged strain by an extrapolation is particularly governed by the load increment ratio and the frequency of repeated loading, the diagram relating the converged strain to the load increment ratio as a parameter of frequency is suggested to be most practically useful in the prediction of settlement induced by repeated loading.

Key words : clay, compression, consolidation, consolidation test, creep, deformation, repeated load, secondary compression, settlement, triaxial compression test, (IGC : D 5/D 6/D 7/E 2/E 8)

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INTRODUCTION

Soft subgrades of the highway, railway and runway sometimes suffer from an unpredictable settlement and failure due to repeated loading resulting from the repetition of wheel loads. The same situation should be in the marine clays under foundations of off-shore structures subjected to a wind or a wave action. Yamanouchi and Yasuhara (1975) presented a case history in which the low bank highway on the soft ground exhibited the abnormal settlement after opening to traffic. At the same time, they proved the effectiveness of stabilizing the surface of soft subgrades with the quicklime by in-situ vehicle running tests over a low embankment highway (Yamanouchi et al., 1978).

On the basis of the judgement that this kind of settlement has been induced by consolidation under repeated loading, repeated oedometer tests have been performed on several remolded and undisturbed cohesive soils to make clear the mechanism of repeated consolidation (Yamanouchi et al., 1969, 1975; Yasuhara et al., 1980).

Some studies of soil deformation owing to application of repeated load have been carried out. A majority of this research have been conducted on various materials in connection with design problems related to pavement subgrades. Seed and his associates have significantly contributed to this research (for instance, Seed and Chan, 1955, 1958). The studies which belong to the same category as that by Seed et al. were attempted by Kawakami and Ogawa (1965), Yamanouchi and Aoto (1969), Luo (1973) and others.

Among them, a unique proposal was pursued by Yamanouchi and Aoto for prediction of strain performance and fatigue failure during repeated loading. So far, the authors infer that there must be only a possibility that the procedure proposed by Yamanouchi and Aoto will be made use for predicting strain performance and settlement behavior of clay even under repeated consolidation.

In this paper, following a simplified law which was discovered in a series of repeated triaxial compression tests on a compacted sandy soil, a method was introduced for predicting the converged strain of saturated cohesive soils in repeated oedometer tests and drained repeated triaxial compression tests. In addition, by arranging the results of repeated loading tests in laboratory, the diagrams of the load increment ratio and the converged strain plots are presented for the approximate prediction of settlement of soils under repeated loading.

A LAW FOR PREDICTING STRAIN UNDER REPEATED LOADING

Yamanouchi and Aoto, as previously stated, performed a family of repeated triaxial compression tests on a compacted sandy soil called "*Masa* soil" (decomposed granite). Index properties of this soil and testing conditions are summarized in Tables 1 and 2. They mainly investigated deformation characteristics and its time-dependency of the compacted sandy soil under repeated loading.

Generally speaking, deformation of soil during repeated loading is supposed to be contributed by unrecoverable and recoverable strain components. Fig. 1 illustrates the relation among each displacement or strain. From this figure, the total displacement, δ_t , at an arbitrary number of load cycles is composed of both recoverable displacement, δ^e , and unrecoverable one, δ^p . Then, we have

Table 1. Index properties of sandy soil (*masa* soil)

Specific gravity G_s	2.64
Optimum moisture content $w_{opt}(\%)$	15.0
Maximum dry density $\rho_d(t/m^3)$	1.74
Plasticity index I_p	N. P.

Table 2. Testing conditions of repeated triaxial test on a compacted sandy soil

Cyclic wave	Rectangular
Period of repeated loading (sec.)	4.0
Specimen height (cm)	8.15~8.25
Specimen diameter (cm)	3.5

Fig. 1. Modeling of strain-time behavior generated by repeated loading

$$\delta_t = \delta^e + \delta^p \quad (1-a)$$

It is emphasized at Fig. 1 that even the recoverable displacement as well as the unrecoverable displacement is time-dependent under repeated loading. Therefore, as matter of course, the recoverable displacement is not in intrinsic coincidence with the "elastic" displacement.

A typical axial strain-number of load cycles relation in repeated triaxial tests on the Masa soil is shown in Fig. 2(a). In the figure, ξ

Fig. 2(a). A typical axial strain-number of load cycles relation in repeated triaxial compression tests on a compacted sandy soil

Fig. 2(b). Typical strain ratio-number of load cycles relation in repeated triaxial compression tests on a compacted sandy soil

is defined as the ratio of the unrecoverable displacement to the total displacement.

It is seen from Fig. 2(b) that the linear relation between the strain ratio and log of number of load cycles holds good except in the vicinity of the failure region. The linearity in ξ -log N relations might depend on the physical properties, the initial structure of soil particles, the intensity of repeated loads and such repeated loading conditions as frequency and number of load cycles.

As can be seen in Fig. 2(b), the value of ξ slightly deviates from a linearity of ξ -log N relation, as well as in the vicinity of the failure region, in the region where number of load cycles is comparatively small.

On the other hand, specimens of *Masa* soil used may fail at the almost constant value (0.75 through 0.90) of ξ which will rely not only upon the loading conditions but also upon the kind of soils.

The gradient of the linear portion of ξ -log N relation is given by

$$\tan \theta = (\xi_n - \xi_0) / \log(N_n / N_0) \quad (2)$$

where N_n , N_0 ; the given and the first number of load cycles, respectively, and ξ_n , ξ_0 ; strain ratios corresponding to N_n and N_0 , respectively. Therefore, the value ξ_n necessary for the prediction of the number

Fig. 3. Value of θ correlated to the initial void ratio, e_i .

of load cycles at failure of soil specimens should be determined by Eq. (2) if the value of θ could be related to an index property which is conventionally used.

This value of θ for the *Masa* soil used herein is correlated to the initial void ratio, e_i , as shown in Fig. 3. The average water content of every specimen is 10%. From Fig. 3, we can ensure a good linearity between θ and $\log e_i$ at each intensity of repeated loads.

It is first postulated that the values of θ , ξ_0 and δ^p_1 (unrecoverable displacement at the first number of load cycles) are already given.

As mentioned before, the total displacement, $(\delta_t)_n$, at a given cycle number of soil specimen under repeated loading consists of unrecoverable and recoverable displacements, $(\delta^p)_n$ and $(\delta^e)_n$, respectively. Then, we have

$$(\delta_t)_n = (\delta^p)_n + (\delta^e)_n \quad (1-b)$$

Since the recoverable displacement is extremely small compared with the accumulated unrecoverable one, we can approximately assume

$$(\delta^e)_n \doteq (\delta^e)_1 \quad (3)$$

where $(\delta^e)_1$: the recoverable displacement at the first cycle number. Hence, Eq. (2) is rewritten as

$$(\delta_t)_n \doteq (\delta^e)_1 + (\delta^p)_n \quad (4)$$

Therefore, the displacement ratio ξ_n is given by

$$\xi_n = \frac{(\delta^p)_n}{(\delta_t)_n} = \frac{(\delta^p)_n}{(\delta^p)_n + (\delta^e)_n} \quad (5)$$

Incorporating Eq. (3) into Eq. (5), we obtain

$$\xi_n = \frac{(\delta^p)_n}{(\delta^p)_n + (\delta^e)_1} \quad (5)$$

Arranging Eq. (6), we get

$$(\delta^p)_n = \xi_n \cdot \frac{(\delta^e)_1}{1 - \xi_n} \quad (7)$$

Accordingly, the total displacement at the n -th cycle number of repeated loading can be written as

$$\begin{aligned} (\delta_t)_n &= (\delta^p)_n + (\delta^e)_n = \frac{(\delta^e)_1}{1 - \xi_n} \\ &= \frac{(\delta^e)_1}{1 - \{\tan \theta \cdot \log(N_n/N_0) + \xi_0\}} \end{aligned} \quad (8)$$

We notice in Fig. 2(b) shown before that there exists the number of load cycles in

Fig. 4. Key sketch for determination of experimental constants, ξ_0 and θ .

ξ - $\log N$ relation where the value ξ deviates from the linearity versus $\log N$ and then the specimen fails in every test as shown in Fig. 2 (b). This tendency implies that the failure zone is generated in specimens at a given number of load cycles. In other words, Eq. (8) is not available to both the beginning and failure region. Subsequently, the life of a specimen to the fatigue failure will be deduced if values of ξ and θ may be determined by the procedure shown in a key sketch of Fig. 4.

A METHOD FOR PREDICTING STRAIN PERFORMANCE

There should be a number of the factors in determining dynamic deformation characteristics of saturated cohesive soils. Particularly, the plasticity index and the sensitivity as physical properties, and the repeated load intensity, the drainage condition and the frequency of repeated loading as testing conditions are counted to be most important. Testing conditions of repeated oedometer tests and repeated triaxial tests which have been performed on undisturbed cohesive soils are summarized in Tables 3 and 4. Both kinds of tests follow the procedure shown in Fig. 5.

Index properties of clays used in the present paper contained in Table 3 and Table 4 include liquid limit (w_L) and plasticity index (I_p). Fig. 6 shows that most of soils classify to highly plastic clays and silts.

The outlines of the testing equipments

Table 3. Testing conditions for repeated oedometer test on saturated cohesive soil

Soil	Test No.	σ_v (kPa)	$\Delta\sigma_r$ (kPa)	$\frac{\Delta\sigma_r}{\sigma_v}$	f (Hz)	N (Cycles)	N_f (Cycles)	ε_f (%)	$\{w_L(\%) \}$ $\{I_p\}$	h_0 (cm)
Ariake clay (RO-1, D)	1	98.0	9.8	0.1	0.017	7.0×10^3	2.5×10^2	0.35	$\{115 \}$ $\{58\}$	2.0
	2		29.4	0.3			1.0×10^4	1.77		
	3		39.2	0.4			1.0×10^5	3.6		
	4		49.0	0.5			"	5.0		
	5		58.8	0.6			"	5.2		
	6		98.0	1.0			"	7.4		
Organic soil (RO-2, D)	7	98.0	39.2	0.4	0.167	3.0×10^4	2.5×10^5	3.2	$\{153 \}$ $\{66\}$	2.0
	8				0.056	"	2.2×10^5	6.9		
	9				0.033	2.0×10^4	1.4×10^4	7.2		
	10				0.022	"	5.3×10^5	7.7		
Peat (RO-3, U)	11	8.8	39.2	4.44	0.028	5.3×10^3	3.0×10^5	30.6	$\{219 \}$ $\{142 \}$ ($L_{ig}=75$)	2.0
	12	9.8	12.7	1.30			9.0×10^4	7.1		
	13	10.8	2.9	0.273			2.3×10^5	3.8		
Ariake clay (RO-4, U)	14	33.3	13.7	0.412	1.0	1.73×10^5	1.0×10^5	5.6	$\{117 \}$ $\{70\}$	2.0
	15	47.0	11.8	0.250			"	1.6		
	16	66.0	9.8	0.147			"	1.3		
	17	35.2	14.7	0.417	0.5	8.64×10^4	1.0×10^5	5.0		
	18	39.2	14.7	0.375			"	4.05		
Ariake clay (RO-5, U)	19	27.4	16.7	0.607	2.0	3.64×10^5	1.0×10^5	3.6	$\{150 \}$ $\{92\}$	2.0
	20	29.4	16.7	0.567			4.0×10^5	2.44		
Kumamoto clay (RO-6, U)	21	39.1	12.3	0.316	0.033	2.88×10^3	1.0×10^5	1.4	$\{85 \}$ $\{60\}$	2.0
	22	42.0	12.1	0.287			8.0×10^5	0.40		
	23	45.0	11.7	0.259			1.0×10^5	1.22		
	24	47.8	11.4	0.238			2.0×10^5	0.64		
Tamashima silt (RO-7, U)	25	35.8	9.8	0.274	0.1	8.64×10^3	6.0×10^5	1.45	$\{51 \}$ $\{26\}$	2.0
	26	51.5	4.9	0.095	0.1	8.64×10^3	"	0.73		
	27	35.8	9.8	0.274	1.0	8.64×10^4	"	1.46		
	28	51.5	4.9	0.095	1.0	"	"	2.00		
	29	35.8	9.8	0.274	10	8.64×10^5	"	0.95		
	30	51.5	4.9	0.095	10	"	"	0.47		
Onga clay (RO-8, U)	31	73.5	9.8	0.133	0.5	8.64×10^4	5.0×10^5	1.01	$\{54 \}$ $\{32\}$	2.0
	32		14.7	0.200			5.0×10^6	1.78		
	33		19.6	0.267			1.0×10^7	2.08		
	34		29.4	0.400			1.0×10^8	3.65		
	35	86.2	14.7	0.170	0.5	8.64×10^4	1.0×10^7	5.67	$\{154 \}$ $\{80\}$	2.0
	36		29.4	0.341			"	6.57		
	37	108.8	14.7	0.135	0.5	8.64×10^4	4.0×10^6	0.66	$\{45 \}$ $\{25\}$	2.0
	38		19.6	0.180			4.0×10^6	0.70		
	39		29.4	0.270			1.0×10^7	1.64		
	40	73.5	29.4	0.400	0.5	1.30×10^5	1.0×10^8	4.67	$\{63 \}$ $\{23\}$	1.0

are illustrated in Fig. 7 and Fig. 8. A repeated oedometer test apparatus is devised in order to apply the repeated loads to the specimens periodically by the air pressure. The repeated load is of the shape of the approximately sinusoidal wave as illustrated in Fig. 5. The intensity of the repeated load was estimated by summing up the stress due to distribution of the wheel load and the load increment due to vibration of the embankment itself.

Fig. 9 illustrates an example of stress combination at the field under traveling vehicles.

The typical relations between axial strain and log time in repeated oedometer tests are plotted in Fig. 10(a) on a disturbed clay sample and in Fig. 11 on an undisturbed clay sample. For comparison, results of ordinary oedometer tests are shown in Fig. 10(b). Each curve in Fig. 10(a) has the inflection points while we can recognize no inflection point

Table 4. Testing conditions for repeated triaxial compression test on saturated cohesive soil

(1) Soil (U or D)	(2) σ_1 (kPa)	(3) σ_3 (kPa)	(4) $\Delta\sigma_r$ (kPa)	(5) $\frac{\Delta\sigma_r}{\sigma_{m0}}$	(6) f (Hz)	(7) N (Cycles)	(8) N_f (Cycles)	(9) ε_f (%)	(10) $\{w_L(\%)$ $\{I_p$	(11) Drainage condition
Ariake clay (RT-1, D)	98.0	98.0	19.6 29.4 39.2 49.0 58.8 68.6	0.2 0.3 0.4 0.5 0.6 0.7	0.017	1.0×10^4	5.0×10^4 3.5×10^4 1.3×10^5 7.0×10^4 2.5×10^5 1.0×10^6	0.90 1.35 2.40 4.10 4.90 5.95	$\{115$ $\{58$	Drained
Ariake clay (RT-2, D)			19.6 29.4 49.0 58.8	0.2 0.3 0.5 0.6	0.017	1.0×10^4	7.8×10^4 5.6×10^4 1.6×10^5 3.0×10^5	0.5 1.15 2.40 5.20	$\{115$ $\{58$	Undrained
Ariake clay (RT-3, U)	21.6 27.4 32.3 48.0 21.6 35.3	21.6 27.4 32.3 48.0 21.6 35.3	20.6 19.6 6.9 2.9 10.8 9.8	0.955 0.714 0.212 0.061 0.500 0.278	1.0	5.0×10^5 " " " 2.5×10^5 "	— 4.3×10^6 — — — 3.0×10^7	— 6.67 — — — 2.87	$\{115$ $\{58$	Drained
Ariake clay (RT-4, U)	51.5 64.7 51.0 51.0	25.5 32.3 25.5 25.5	11.8 9.8 11.8 11.8	0.346 0.227 0.346 0.346	1.0 1.0 0.1 2.0	8.64×10^4 " 8.64×10^3 4.32×10^4	6.0×10^5 " 6.0×10^4 3.0×10^5	1.74 1.45 1.05 1.07	$\{117$ $\{70$	Drained
Ariake clay (RT-5, U)	51.0 64.7	25.5 32.3	11.8 9.8	0.346 0.227	1.0 1.0	8.64×10^5 "	6.0×10^5 "	1.35 1.28	$\{117$ $\{70$	Undrained
Kumamoto clay (RT-6, U)	39.1 42.0 45.0 47.8	19.6 21.0 22.5 23.9	12.3 12.1 11.7 11.4	0.473 0.430 0.389 0.357	0.033	2.88×10^3	1.0×10^5 8.0×10^4 3.0×10^4 2.0×10^5	0.70 0.19 0.21 0.47	$\{85$ $\{60$	Drained
Okayama silt (RT-7, U)	58.8 58.9	29.4 29.4	19.6 49.0	0.500 1.250	1.0	8.64×10^4	3.0×10^7 6.0×10^{10}	0.19 3.80	$\{35$ $\{15$	Drained
Tamashima-1 (RT-8, U)	35.8 51.5	35.8 51.5	9.8 4.9	0.274 0.095	10	8.64×10^5	6.0×10^6	1.15 0.33	$\{37$ $\{17$	Drained
Tamashima-2 (RT-9, U)	58.8	29.4	49.0 49.0 19.6 19.6 49.9 49.6 19.6 19.6	1.25 1.25 0.50 0.545 1.25 1.25 0.50 0.50	0.1 3.0 0.1 3.0 0.1 3.0 0.1 3.0	8.64×10^3 2.59×10^5 7.52×10^3 6.48×10^4 8.64×10^3 7.92×10^4 6.30×10^3 7.02×10^4	1.8×10^7 1.8×10^8 6.0×10^5 9.1×10^5 3.0×10^5 1.8×10^5 6.0×10^5 1.8×10^6	2.55 5.50 3.60 4.66 10.50 12.00 2.04 3.80	$\{43$ $\{20$ $\{60$ $\{36$ $\{58$ $\{35$ $\{68$ $\{41$	Drained
Tamashima-3 (RT-10, U)	98.0 49.0 (OCR=1.82) 75.3 49.0 (OCR=1.36)	49.0 29.4 36.8 29.4	19.6 19.6 49.0 19.6 49.0	0.545 0.545 1.362 0.954 0.545 0.545 1.362	0.1 3.0 0.1 3.0 0.1 3.0 0.1	8.23×10^3 2.48×10^5 8.64×10^3 2.59×10^5 8.64×10^3 " 7.20×10^3	4.08×10^5 3.64×10^7 6.0×10^4 5.45×10^6 2.28×10^5 2.91×10^6 3.0×10^6	0.45 0.60 3.80 1.80 0.60 0.58 8.90	$\{67$ $\{36$ $\{63$ $\{37$ $\{67$ $\{39$	Drained
Okayama silt (RT-11, U)	78.4 58.8 (OCR=1.21)	39.2 35.3	49.0	1.136	1.0	8.64×10^4	6.0×10^7	1.80	$\{34$ $\{15$	Drained

(U : Undisturbed, D : Disturbed)

Fig. 5. Testing procedures of repeated oedometer and triaxial tests

Fig. 6. Plasticity chart of clays used for repeated oedometer and triaxial tests

on each curve in Fig. 11. To grasp the secondary compression characteristics of an undisturbed clay under repeated loading, the axial strain is measured over one and a half months in a series of tests.

A comparison between the ordinary and the repeated oedometer tests on the same undisturbed clay sample is shown in Fig. 12. As can be seen in this figure, it must take a long duration to find the transition point from primary consolidation to secondary compression, especially in the case of a test with a small load increment ratio.

In general, it has been admitted that deformation of soils under repeated loading consists of the unrecoverable strain, ε^n , and

Fig. 7. Outline of repeated oedometer testing equipment

Fig. 8. Outline of repeated triaxial testing equipment

the recoverable one, ε^e , as mentioned before. Hence, even in repeated oedometer tests, we can assume

LEGEND

 $\Delta\sigma_{r1}$: Repeated stress increment due to wheel load $\Delta\sigma_{r2}$: Repeated stress increment due to vibration of embankment $\Delta\sigma_v$: Vertical stress increment due to embankment load

Fig. 9. Stresses acting on a soil element beneath the center of embankment

Fig. 10. Representative vertical strain versus log time relation of a disturbed clay (Ariake clay)

$$\varepsilon = \varepsilon^e + \varepsilon^p \quad (9)$$

As deformation falls into the elasto-plastic region, the plastic strain will considerably contribute to the total strain.

Fig.13 shows the variations of the re-

Fig. 11. Representative vertical strain versus log time relation of an undisturbed clay

Fig. 12. A comparison of axial strain versus log time relation between ordinary and repeated oedometer tests on an undisturbed clay (Onga clay)

Fig. 13. Variations of recoverable strain in repeated oedometer tests

coverable strain with the elapsed time which is calculated from the test results on Ariake clay. It can be seen from curves in Fig.13 that in most cases the recoverable strain is inclined to have the almost constant value with the elapsed time, and this constant

Fig. 14. Recoverable strain in repeated oedometer tests

value seems to be linearly proportional to the increased intensity of repeated loads as shown in Fig. 14. On the contrary, as mentioned before, the total strain does not readily converge to the final strain during the ordinary period of measurement in conventional repeated oedometer tests. This means that accumulation of the total strain must be almost ascribed to the unrecoverable strain. That is, accompanied by accumulation of the strain in repeated consolidation tests, the ratio of the recoverable and unrecoverable strains to the total strain finally lead to

$$\left. \begin{aligned} \lim_{t \rightarrow \infty} \left(\frac{\varepsilon^e}{\varepsilon} \right)_t &= 0 \\ \lim_{t \rightarrow \infty} \left(\frac{\varepsilon^p}{\varepsilon} \right)_t &= 1.0 \\ \lim_{t \rightarrow \infty} (\varepsilon)_t &= \varepsilon_f \end{aligned} \right\} \quad (10)$$

Consequently, by construction of $\varepsilon^p/\varepsilon$ versus $\log t$ relations and by extrapolation of N_f or t_f corresponding to $(\varepsilon^p/\varepsilon)=1.0$, the final strain (ε_f) of each test will be predicted. Figs. 15 and 16 which were obtained from test series of RO-1 and RO-8 suggest the validity of this practical method. The final strain was determined by extrapolation of the linear portion of each curve on Figs. 15 and 16. The procedure of the method, which was above-mentioned, for predicting the final strain of soil specimen during repeated consolidation is schematically summarized in Fig. 17. This is also available for soil defor-

Fig. 15. Variation of strain ratio with elapsed time in repeated oedometer test on a disturbed clay

Fig. 16. Variation of strain ratio with elapsed time in repeated oedometer test on an undisturbed clay

Fig. 17. Method for predicting soil deformation under repeated loading

mation in repeated triaxial compression tests as will be stated later. Then, finally, the converged strains estimated by this extrapolation are plotted to the load increment ratio in Fig. 18, together with results of the other series of tests.

It is clarified from this figure that the final strain is mainly governed by the load increment ratio and is appreciably less af-

Fig. 18. Summarized plots on final strain and load increment ratio in repeated oedometer test

affected by the frequency of repeated loading and the other factors. However, it should be kept in mind that the frequency and intensity of repeated loads employed in repeated oedometer tests (see Table 3) are not extended over wide ranges. Accordingly, provided that we could construct the more complete $\epsilon_f - \Delta\sigma_r/\sigma_v$ relation which will cover wide ranges of test conditions, it would be more profitable from the engineering point of view.

Repeated Triaxial Compression Tests

When a clay sample is loaded at a constant principal stress ratio in repeated triaxial compression tests, the sample will demonstrate the similar behavior to that in drained creep or anisotropic consolidation tests. Prior to application of cyclic loading, as well as in conventional triaxial compression tests, a specimen is isotropically or anisotropically pre-consolidated for one through four days. The intensity of pre-consolidation is kept for $\sigma_3 = \sigma_1$ as isotropic pre-consolidation and for $\sigma_3 = K\sigma_1$ (K ; principal stress ratio) as anisotropic pre-consolidation, respectively.

Fig. 19. Relation between axial strain and volumetric strain in drained-repeated creep test on an undisturbed clay

Major principal stresses as preconsolidation pressure in each test are listed in Table 4. An increment load is cycled during a constant period under the constant frequency. In most cases, drain from the specimen is permitted through the filter paper surrounding the specimen.

Fig. 19 shows the typical relation between the volumetric strain, ϵ_v , and the axial strain, ϵ_1 on an undisturbed clay sample in repeated triaxial drained creep tests. In much the same way as the relation in drained creep tests under sustained loading (Yasuhara, 1979), ϵ_1 versus ϵ_v relations are divided into three processes; the first one with the non-volumetric change (so called instant deformation), the second one where shear deformation is more predominant than the volumetric change and the third one where the volumetric change follows very rapidly overtaking the shear deformation. Fig. 20 is a key sketch for understanding of these processes.

The typical relations between axial strain and log time are plotted in Fig. 21. The symbol $\uparrow$ in each figure indicates the inflection point shown in curves of Fig. 19. This figure shows a good linear progress of the axial strain with log of time. On the contrary, however, in the case of the other sample with the higher water content as shown in Fig. 22, the linearity of axial strain versus log time

Fig. 20. Schematic relation between volumetric strain and shear strain in tri-axial drained creep and anisotropic consolidation tests

Fig. 21. Variation with axial strain of elapsed time in repeated drained creep test on an undisturbed clay (Tamashima silt) with the low water content

Fig. 22. Variation with axial strain of elapsed time in repeated drained creep test on an undisturbed clay with the high water content

relation does not hold good within these observed times. In such cases, the measurement of strains should be continued over a long time period to appreciate the distinct transition point of strain versus number of load cycles relation.

In order to look into the influence of stress history on deformation characteristics in drained creep tests under cyclic loading, repeated triaxial tests (RT-10, RT-11) are run on undisturbed clays under over-consolidation due to rebound. In Fig. 19 and Fig. 22 are shown examples of the ε_i versus ε_p relation and the axial strain versus log of time relation, respectively, on the over-consolidated clay.

The same method as described for repeated oedometer tests is applied to prediction of the strain in repeated triaxial drained creep tests on clay. One of the $\varepsilon^p/\varepsilon_i$ versus $\log t$ relations is shown in Fig. 23. The possibility of the predicted method of the converged strain is perceived from a family of curves to detect the final strain. Thus, we get the other $\Delta\sigma_r/\sigma_{mo}$ value ($\Delta\sigma_r$; the intensity of incremental repeated load, σ_{mo} ; initial mean principal stress) and the final strain plots summarized for the approximate prediction of the strain of the soil specimen in repeated triaxial creep tests (see Fig. 24).

In Fig. 24, also plotted are the results of the other series of tests which have been performed so far on cohesive soils. In comparison of this figure with Fig. 18 obtained from repeated oedometer tests, there is a

Fig. 23. Relation between strain ratio and elapsed time in drained-repeated creep test on an undisturbed clay

Fig. 24. Summarized plots of final strain-load increment ratio in repeated tri-axial compression tests

little more scattering between $\Delta\sigma_r/\sigma_{m0}$ and ε_f in triaxial tests than in oedometer tests. In general, the plots of the over-consolidated specimens are located below the plots of the normally-consolidated ones. Also, there is the another tendency of the larger converged strain with higher frequency in Fig. 24. Judging from these characteristics, it can be emphasized that both the frequency of repeated loading and the stress history of clay used are the most important factors to govern the location of the ε_f versus $\Delta\sigma_r/\sigma_{m0}$ relation.

CONCLUSIONS

A simplified law is found out from results of repeated triaxial tests on a compacted sandy soil called *Masa*. Then, to make sure of availability of this law to the other kinds of soils and the broader ranges of testing conditions, a number of series of oedometer and triaxial creep tests were carried out on saturated cohesive soils under repeated loading. Test results were conceived to utilize the settlement prediction such as the clay soil subgrades of low embankment highways, the off-shore structures on the clay sea bed and so on.

As pointed out before by the authors (Yamanouchi and Yasuhara, 1977), saturated clays under repeated loading with the drained or the partially drained conditions showed the deformation performance similar to that under sustained loading. Based on the fact that deformation of soils under repeated loading is more consolidative than that under sustained loading, an experimental law for predicting the axial strain of specimens in laboratory was discovered from a consideration of results of repeated oedometer and repeated triaxial compression tests on saturated cohesive soils. According to this finding, a method was proposed for predicting the axial strain of clay specimens under repeated loading. It was suggested from this method that the predicted final strain was primarily controlled by the load increment ratio and the frequency of repeated loading, except for an extremely soft soil such as peat. The more useful ε_f versus $\Delta\sigma_r/\sigma_{m0}$ plot will be obtained when the test data with every kind of testing conditions is hereafter accumulated.

The method for settlement prediction described in the present paper can be applied not only to settlement owing to traffic loads but also to the other case histories where the settlement is caused by cyclic or transient load including, for instance, wind and wave actions at off-shore structures.

NOTATION

- e = void ratio
- e_i = initial void ratio
- N_n = a given cycle number
- N_0 = cycle number at the beginning of the linear portion on ξ -log N
- N = cycle number in repeated triaxial or oedometer test
- t = time or elapsed time
- δ^e = recoverable displacement
- δ^p = unrecoverable displacement
- δ_t = total displacement
- ε_1 = axial strain
- ε_v = volumetric strain
- ε^e = recoverable strain
- ε^p = unrecoverable strain
- ε_f = final axial strain
- θ = gradient of the linear portion on ξ -log N

relation

ξ_1 =strain ratio at the first cycle number

ξ_n =strain ratio at the n -th cycle number

σ_v =vertical stress

$\Delta\sigma_r$ =incremental repeated stress

σ_{mo} =initial mean effective principal stress

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