CYCLIC SHEAR STRENGTH OF UNDISTURBED AND REMOULDED MARINE CLAYS

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ABSTRACT

Undisturbed clay samples were taken from alluvial deposits at seven different urban coastal locations in Japan. At several of the locations samples were taken from different depths in order to sample different sedimentary environments. The samples were then triaxially tested in both undisturbed and remoulded states. A testing programme was devised to separately examine the influence of plasticity index, aging factors and overconsolidation ratio on the undrained cyclic strength of clays. The cyclic data for normally consolidated samples was used to define the cyclic strength of the clays as an intrinsic function of the plasticity index I_p . This was modified to take into account the initial vertical yield stress ratio and degree of aging, and then further modified for the effect of geological OCR. A further series of tests was carried out on samples which were anisotropically consolidated by being subjected to an initial drained shear stress for 24 hours. The cyclic strength of the anisotropically consolidated clays at low initial drained shear stress ratios increased with increasing plasticity. This reversed as the drained shear stress ratio increased. It was possible to define a simple function for the cyclic strength normalised by the isotropic cyclic strength against the initial drained shear stress ratio. This function was shown to be dependent on plasticity index I_p but independent of stress history.

Key words: aging effect, clay, cyclic strength, cyclic triaxial compression test, initial shear stress, intrinsic strength, overconsolidated, undisturbed sample (IGC: D7)

INTRODUCTION

The shortage of land around the large conurbations in Japan has resulted in numerous coastal engineering schemes including coastal defences, break waters, harbours and reclaimed land developments which are often located on soft marine clay deposits. Additionally many cities and urban areas are located on alluvial clay deposits. Thus a knowledge of the properties of these soils particularly under wave and earthquake loading conditions is essential. The major concern in seismic design has usually been related to the problems of liquefaction of sands, while clays have always been considered stable materials under earthquake conditions. However there are frequently recorded instances of earthquake initiated failures in clay deposits. For example in the 1964 Alaska earthquake, a major slope failure occurred in quick clay; in the 1985 Mexico earthquake, buildings founded on clay suffered severe damage (Mendoza, 1985) and in the 1964 Niigata, 1978 Miyagi-ken Oki (Sasaki et al., 1980)

and the 1983 Nihonkai Chubu earthquakes, road and river embankments founded on clay deposits underwent deep seated slope failures. Although relatively clear guidelines exist on the liquefaction behaviour of sands, a similar knowledge base for clays under cyclic loading needs to be developed, particularly for in-situ undisturbed clays.

The characteristics of undisturbed or in-situ clays differ from those for remoulded soils due to the influence of aging effects. Bjerrum (1967) was one of the earliest researchers to investigate aging effects in clays. He maintained that aging occurred as a result of secondary consolidation. Ladd and Foott (1974) developed the SHANSEP method in an attempt to overcome sampling disturbance. Samples were compressed to some vertical consolidation pressure beyond their in-situ state and then unloaded to the in-situ overconsolidation ratio. Mesri (1975) however, pointed out that this method was self-defeating since it ignored the natural soil structure formed by the sedimentation process and secondary con-

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solidation, arguing that by removing aging effects the undrained strength of a clay was underestimated. Hanzawa (1983) tested many different undisturbed clay samples and used the term "aged clay". He suggested that an intrinsic sedimentation line existed where the soils undrained shear strength was linearly related to the overburden pressure. He then suggested that this basic strength was increased by a combination of cementation and secondary consolidation and decreased as a result of leaching. These combined effects over a period of time produced an aged clay. Burland (1990) showed that the properties of a natural clay differ from the intrinsic properties of a remoulded material due to the influence of soil fabric and bonding. He also showed that the undrained strength of a normally consolidated sediment is a function of the in-situ effective stresses and of the soil structure, and not the moisture content as is suggested by critical state theory. However, similarly to Hanzawa (1983), he suggested that the intrinsic properties of remoulded soils form a basis for the interpretation of undisturbed behaviour.

For the purpose of this research programme therefore, over 100 undisturbed clay samples were taken from alluvial deposits at seven different urban coastal locations in Japan using a thin walled sampler. At several of the locations samples were taken from many different depths in order to sample different sedimentary environments. The samples were then tested in both undisturbed and remoulded states. Hyodo et al. (1994a; 1996) have already shown that the properties of these clays depend not only on characteristics such as their plasticity but also their sedimentary origins, fabric and applied stress path. A testing programme was devised to separately examine the influence of plasticity index, aging factors and over-consolidation ratio on the undrained cyclic strength of clays.

CYCLIC FAILURE OF CLAYS

Cyclic failure in clay deposits due to earthquakes or wave loading usually occurs in the foundation zone below civil engineering structures where there is an imposed initial drained shear stress. This form of failure can be distinguished from the liquefaction type of failure occurring in sands and will be called cyclic shear failure. Hyodo et al. (1994b) have shown that the cyclic strength of clays decreases with increasing initial drained shear stress. This can be contrasted with the behaviour of sand where the cyclic shear strength increases with an initial drained shear stress. This is because in a clay an initial drained shear stress moves the state path closer to the critical state and resulting instability, while in a sand it is moved towards a point of phase transformation after which the sand hardens under undrained cyclic loading.

In-situ sand is likely to liquefy due to cyclic horizontal shearing without any initial shear stress. Clays however fail along a circular shear plane beneath a structure which also imposes an initial shear stress. This distinction between the dynamic failure of sands and clays is il-

Fig. 1. Distinction between the dynamic failure of sands and clays

lustrated in Fig. 1. The transmission of horizontal shear waves in a liquefied sand is likely to be heavily damped and there are therefore smaller inertia forces applied to any structure. Nonetheless, because of the liquefied state of the ground, instability and failure still occur. In the case of a clay deposit, the inertia forces from the structure will be very large, such as those witnessed in the Mexico earthquake in 1985 (Mendoza et al., 1988). These forces in turn will impose cyclic shear stresses along a failure plane beneath the structure, causing instability but with a different failure mechanism to that for sand.

MATERIAL PROPERTIES AND EXPERIMENTAL METHODS

Samples

Eight kinds of marine clay were collected. Samples of Hiroshima A and B clays were obtained from Ujina and Itsukaichi respectively in Hiroshima bay. Ariake A and B clays were from the mouth of the Shirakawa river in Kumamoto and Higashiyoka in Saga prefecture on the coast of the Ariake Sea. Yokohama clay was taken from area MM21 in Yokohama harbour, Oita clay from Oita harbour, Nagasaki clay from Nagasaki harbour and Arakawa clay from the a riverbank in Tokyo's Arakawa

Table 1. Physical properties of undisturbed clays

Name	Series No.	Depth G.L(m)	ρ_s	w_L (%)	w_p (%)	I_p	F.C. (%)	C.C. (%)
Hiroshima-A	1-1	3.50 ~ 4.30	2.670	44.1	19.7	24.4	64.3	16.0
	2-3	21.50 ~ 22.50	2.655	44.9	17.8	27.1	39.0	18.0
	2-1	3.50 ~ 4.30	2.639	—	—	N.P.	38.3	10.0
	2-2	12.50 ~ 13.30	2.607	94.1	37.6	56.5	98.9	45.0
	UH-10	9.50 ~ 10.30	2.640	77.4	32.3	45.1	98.7	45.0
Hiroshima-B	DN	18.00 ~ 18.80	2.532	124.2	51.4	72.8	98.0	48.0
	DM01	18.00 ~ 18.80	2.505	99.1	37.7	61.4	99.0	52.0
	DM04	18.00 ~ 18.80	2.521	99.0	40.3	58.7	95.0	40.3
Ariake-A	AN01	17.00 ~ 17.85	2.664	72.0	32.1	39.9	98.4	62.5
	AN3	16.65 ~ 17.55	2.557	107.0	46.4	60.6	—	—
	AN6	21.55 ~ 22.45	2.581	98.4	47.0	51.4	—	—
	AN04	25.00 ~ 25.80	2.612	99.8	40.4	59.4	98.5	73.2
	AN8	25.80 ~ 26.60	2.624	141.5	59.6	81.9	—	—
Ariake-B	AT9	11.90 ~ 12.70	2.628	68.5	34.1	34.4	68.0	42.5
	AT12	14.60 ~ 15.40	2.625	45.9	27.3	18.6	97.8	53.5
	AT16	16.20 ~ 17.00	2.616	75.0	34.8	40.2	80.5	47.5
	AT18	18.60 ~ 19.40	2.570	70.0	33.7	36.3	100.0	68.2
	AT11	13.30 ~ 14.10	2.583	93.6	36.8	56.8	92.2	53.7
	AT15	15.70 ~ 16.50	2.538	60.5	33.5	27.0	98.1	69.9
	A24	7.50 ~ 8.30	2.660	100.3	40.8	59.5	—	—
	A27	11.00 ~ 11.80	2.688	43.8	—	N.P.	—	—
	A211	18.00 ~ 18.80	2.695	67.5	31.3	36.2	—	—
	55	7.50 ~ 8.30	2.616	40.6	—	N.P.	52.9	8.2
Oita	ON	3.00 ~ 3.90	2.604	78.0	37.3	40.7	—	—
Nagasaki	NC	5.00 ~ 5.80	2.667	72.5	31.5	41.0	78.5	30.0
Yokohama	MM	30.90 ~ 31.70	2.564	126.7	48.2	78.5	99.0	59.5

Table 2. Physical properties of remoulded clays

Name	Series No.	Depth G.L(m)	ρ_s	w_L (%)	w_p (%)	I_p	F.C. (%)	C.C. (%)
Hiroshima-A	PS	—	2.613	62.1	28.6	33.5	84.5	36.5
Hiroshima-B	HN	—	2.532	124.2	51.4	72.8	98.0	59.0
Ariake-A	AC	—	2.652	108.8	42.3	66.5	98.5	73.5
	RAN	—	2.653	86.8	44.2	42.6	93.5	59.0
Ariake-B	RAT	—	2.585	67.5	32.7	34.8	83.3	43.0
Arakawa	AR	—	2.659	49.8	32.0	17.8	96.0	38.0

district.

Their physical properties are summarised in Tables 1 and 2 for undisturbed and remoulded samples respectively. In Table 1 it can be seen that a wide variety of clays has been tested with widely varying fines contents and plasticity indices. In each case clays were sampled by a fixed piston sampler from the sea bed, except for Arakawa clay which was taken from below a river embankment. Non plastic clays in the table were those sampled by the same procedure as the other plastic clays and with some liquid limit but no plastic limit. Fines contents and clay contents of each soil are shown in the tables.

The remoulded soil in each case was reconstituted from a mixture of tested and untested undisturbed samples. The remoulded soil properties therefore tended to

have single average values compared with the variation in the undisturbed samples. Reconstituted samples were prepared over a period of two weeks by reconsolidating a slurry mixed to twice the liquid limit.

Vertical Yield Stress Ratio

The undisturbed clays will have undergone different degrees of creep and secondary consolidation, sedimentation and aging histories along with possible cementation due, for example, to biological activity as was observed by Hight (1994) for Bothkenner clay, or in the case of Japanese marine clays, chemical bonding due to calcification (Fukue et al., 1996). In this process the micro-fabric of these clays will have developed an increased resistance to compression such that when an aged clay is loaded,

(a)

(b)

(c)

(d)

(e)

Fig. 2. Variation of overburden pressure and vertical yield stress with depth: (a) Hiroshima-A clay, Borehole 1, (b) Hiroshima-A clay, Borehole 2, (c) Hiroshima-A clay, Borehole 5, (d) Hiroshima-B clay, Borehole 6 and (e) Ariake-A clay

the structural resistance breaks down at a critical pressure, which following Burland (1990) will be called the 'vertical yield stress', σ'_{vy} . Similarly the associated apparent overconsolidation ratio observed by Hanzawa, (1983) and others, will be termed the 'vertical yield stress ratio' to distinguish it from the geological overconsolidation ratio. Prior to cyclic triaxial tests, strain controlled oedometer tests with a strain rate of 0.05%/min were performed on all undisturbed samples in order to investigate the yield stress and consolidation characteristics. Strain control was used because it was easier to distinguish the yield point than under stress controlled conditions. The vertical yield stress ratio was then calculated from a consideration of the in-situ effective overburden stress and the oedometer vertical yield stress determined from the strain controlled tests.

Figures 2(a)–(e) show the distribution of vertical yield stress σ'_{vy} and overburden pressure σ'_{v0} with depth, and consequently the variation of vertical yield stress ratio $\sigma'_{vy}/\sigma'_{v0}$ for the Hiroshima A and B clay and the Ariake A sites. In these figures at shallow depths down to 5 m, the vertical yield stress ratio is as much as 3.5 due partly to weathering and dessication. It can be argued that the data in these figures lies outside the yield stress ratio range of 1 to 2 normally expected for Japanese marine clays. This may be because the clay has actually been overconsolidated in-situ and is not merely aged. However at a depth of 10 m or more the vertical yield stress ratio approaches the geological OCR of 1.0. Similar distributions were observed for the other clays.

Triaxial Testing

Triaxial Specimens with initial dimensions of 50 mm diameter and 100 mm height were trimmed from blocks of clay. These were initially isotropically consolidated for 24 hours. In the case of samples with an initial drained shear stress, this was applied in steps over 24 hrs until the prescribed stress was achieved. Samples were then tested under cyclic two-way loading using a pneumatic actuator at a frequency of 0.02 Hz. Although earthquake frequencies are often closer to 1 or 2 Hz it was considered to be more important to allow an even dissipation of pore water pressures throughout the samples by testing at a lower frequency.

A test programme was devised to examine the effects of aging, initial undrained shear strength and overconsolidation ratio on the cyclic strength of the clays. To determine the effects of sedimentation and aging, tests were first carried out on isotropically consolidated undisturbed samples and compared with data from remoulded samples. A further series of tests was carried out on remoulded samples in order to determine the effects of an initial drained shear stress on the cyclic strength of these clays. Additionally cyclic tests with an initial drained shear stress were carried out on undisturbed Ariake B samples. Finally testing was carried out on remoulded Hiroshima B clay at different OCR's to determine the effect of this on the cyclic strength.

Figure 3 is a schematic illustration of the $e-\log \sigma'_v$

Fig. 3. Schematic diagram of $e-\log \sigma'_v$ showing different relative magnitudes of effective confining pressure σ'_c : (a) Normally consolidated, (b) With vertical yield stress ratio and (c) Overconsolidated

curves for undisturbed clay. The values of isotropic consolidation pressure σ'_c chosen for testing are shown relative to the vertical yield stress σ'_{vy} . In the case of isotropically consolidated tests the value of the consolidation cell pressure σ'_c is assumed equal to σ'_v for the one dimensional oedometer test. It is acknowledged that the stress conditions are only comparable if the K_0 value is unity, but in order to carry out symmetrical two way loading, isotropic consolidation was needed and it was felt therefore that some compromise was necessary. For overconsolidated clays K_0 is likely to be between 0.5 and 1. In the interests of developing an initial approach to the evaluation of in-situ cyclic shear strength it is felt therefore that

this approximation is valid at this stage.

Considering the consolidation pressure σ'_c in each of the figures, samples in Fig. 3(a) have been taken beyond the vertical yield stress onto the normal consolidation curve as might occur under a large structure. In Fig. 3(b) samples which might be defined as normally consolidated in geological terms have not been loaded past the yield point σ_{vy} and still have a *residual yield stress ratio*. In Fig. 3(c) the soil is overconsolidated as σ'_c is less than the in-situ overburden pressure.

Figure 4 shows a typical relationship between the cyclic double amplitude plastic strain and the peak mobilised effective stress ratio $\eta_p = q_p/p'$ during a cyclic triaxial test, where q_p is the maximum deviator stress and p' is the mean normal effective stress. This figure is for isotropically consolidated Oita clay, but similar figures have been produced by Hyodo et al. (1994b) for other isotropically consolidated clays. As a test progresses p' decreases and thus η_p increases along with the cyclic strains. The maximum effective stress ratio η_p usually coincides with a double amplitude axial strain $\varepsilon_{DA} = 10\%$ where it can be seen that cyclic shear failure is imminent. This strain has therefore been chosen as the failure criterion for cyclic loading.

Figure 5 shows the number of cycles N required to achieve $\varepsilon_{DA} = 10\%$ for a given cyclic deviator stress ratio $\sigma_d/2\sigma'_c$. This figure shows the results for undisturbed Ariake A clay for tests carried out at three different consolidation pressures σ'_c of 70, 130 and 200 kPa. This clay had an estimated in-situ overburden pressure $\sigma'_{v0} = 80$ kPa and an oedometer vertical yield stress $\sigma'_{vy} = 170$ kPa. In the case of the consolidation pressure $\sigma'_c = 70$ kPa the soil may be considered to be overconsolidated as in Fig. 3(c). For a pressure $\sigma'_c = 130$ kPa the sample was considered to lie between the initial in-situ state with an overburden pressure $\sigma'_{v0} = 80$ kPa and the yield point $\sigma'_{vy} = 170$ kPa as in Fig. 3(b), and therefore still had a residual yield stress ratio $\sigma'_{vy}/\sigma'_c > 1$ such that the sample aging effects had not been completely erased. Finally, samples consolidated to 200 kPa were considered to be normally consolidated past the yield point as in Fig. 3(a). It can be seen that the normalised cyclic strength decreased as the clay became more destructured and the aging effects were removed. Thus the sample with the least cyclic strength was the normally consolidated soil. This result will be reconfirmed for the other clays with various plasticity in the following session. Jamiolkowski et al. (1985) carrying out CKU and Tsuchida et al. (1991) carrying out CIU tests on undisturbed clays both observed a related phenomenon when they showed that the aging effect and inherent vertical yield stress ratio in undisturbed clays caused a brittle type of failure and that this effect was gradually removed as the effective confining pressure was increased.

UNDRAINED CYCLIC BEHAVIOUR OF CLAYS

Effect of Plasticity Index on Cyclic Behaviour

Figures 6(a)–(d) show typical cyclic stress paths and

Fig. 4. Variation of double amplitude axial strain with mobilised peak effective stress ratio

Fig. 5. Variation of cyclic strength with effective consolidation pressure

related cyclic stress-strain curves for normally consolidated clays of varying plasticity index from N.P. to $I_p = 72.8$. Examination of the stress path for the non-plastic Hiroshima A clay sample reveals a behaviour similar to that for sands, with the stress path cycling through zero effective stress. As the plasticity increases, the behaviour changes and although the effective stresses still decrease, at $I_p = 27$ the stress path does not go through the zero effective stress state and instead moves towards a state of cyclic mobility. As the plasticity increases further, the locus of the cyclic stress path becomes vertical and p' becomes almost constant although still non-zero. The corresponding stress-strain curves for the non-plastic clay show it to be weaker in extension than in compression. In the case of the more plastic clays a more even development of strain was seen with compressive axial strains increasing to about 5%. It can also be observed that the cyclic stiffness of the clays increases with increas-

Fig. 6. Cyclic stress path and relationship between deviator stress and axial strain for normally consolidated clays of varying plasticity index: (a) Undisturbed Hiroshima-A clay, $I_p=N.P.$, (b) Undisturbed Ariake-B clay, $I_p=27.0$

ing plasticity.

Intrinsic Cyclic Strength Ratio

Figure 7 compares the cyclic strength for undisturbed and disturbed samples of Hiroshima B clay. The curve shown is for the normally consolidated clay, that is the clay has been consolidated past the vertical yield stress and the effects of aging and sedimentation have been removed for the undisturbed clay. It can be seen that both the remoulded and undisturbed samples lie on the same curve and have the same cyclic strength. The cyclic data for the normally consolidated clays therefore will be used to define an intrinsic cyclic strength ratio which will then be modified for aging effects, plasticity and overconsolidation ratio (OCR). Figure 8 shows the normalised cyclic strength for a double amplitude strain $\varepsilon_{DA}=10\%$, for nor-

mally consolidated samples of varying plasticity index I_p . It can be seen that as the plasticity index increases so does the cyclic strength. In each case the clays were undisturbed except for the Arakawa clay, which was remoulded.

The cyclic deviator stress ratio $\sigma_d/2\sigma_c'$ to cause failure after 20 cycles defined as $R_{L(N=20)}$ is defined as the cyclic strength ratio. The data in Fig. 8 and data for remoulded samples from Fig. 7 together with the other data for remoulded Hiroshima-A clay and remoulded Ariake-A, B clays has been redrawn in Fig. 9 as a function of the plasticity index. Since all the clays were normally consolidated, both undisturbed and remoulded samples lay on the same line. A unique relationship was obtained such that:

Fig. 6. Cyclic stress path and relationship between deviator stress and axial strain for normally consolidated clays of varying plasticity index: (c) Undisturbed Oita clay, $I_p=40.2$ and (d) Undisturbed Hiroshima-B clay, $I_p=72.8$

Fig. 7. Comparison of cyclic strength for undisturbed and remoulded normally consolidated clays

Fig. 8. Effect of plasticity index on cyclic strength curves for normally consolidated clays

Fig. 9. Relationship between plasticity index and cyclic strength at 20 cycles for normally consolidated clays

Fig. 11. Cyclic strength of clays at 20 cycles for $\sigma'_{vy} > \sigma'_c > \sigma'_{v0}$

Fig. 10. Deviation of cyclic strength data R_L from function R_1 ($\sigma'_c > \sigma'_{vy}$)

$$R_1 = 0.0007 I_p + 0.25. \quad (1)$$

This will be called the intrinsic cyclic strength ratio, R_1 . Figure 10 illustrates the fact that the deviation of the data from the computed line is very small.

Effect of Aging on the Intrinsic Cyclic Strength

In Fig. 11 the data for undisturbed samples with a residual yield stress ratio i.e. $\sigma'_{vy} > \sigma'_c > \sigma'_{v0}$ is compared with the line drawn using Eq. (1). In this case the aging effects

Fig. 12. Deviation of cyclic strength data R_L from function R_1 ($\sigma'_{vy} > \sigma'_c > \sigma'_{v0}$)

remaining in the undisturbed clay can be seen quite clearly such that most of the samples lie above the computed line. In this case the fit is not quite so good as can be seen in Fig. 12 where the deviation of the data is illustrated. This deviation of the aged cyclic strength, R_L , from the intrinsic cyclic strength ratio R_1 , will be defined as R_2 and has been plotted against the initial yield stress ratio $\sigma'_{vy}/\sigma'_{v0}$ for the undisturbed clays with a residual stress ratio in Fig. 13. It was apparent from this diagram that the deviation tended to be greater for the samples with a greater initial yield stress ratio. An aging parameter α was therefore introduced which is a measure of the residual yield stress ratio for the clays, such that:

$$\alpha = \frac{\sigma'_{vy} - \sigma'_c}{\sigma'_{vy} - \sigma'_{v0}}. \quad (2)$$

The aging parameter α only applies to soils with an initial vertical yield stress ratio i.e. $\sigma'_{vy} > \sigma'_c > \sigma'_{v0}$. If σ'_c is equal

Fig. 13. Cyclic strength correction factor for aging

Fig. 14. Normalised function for aging correction factor R_2

to the in-situ overburden stress σ'_{v0} then the strength is identical to the in-situ aged stress, while if σ'_c becomes equal to σ'_{vy} the aging effect is destroyed. Lines representing values for α between 0.1 and 1.0 were subsequently drawn on Fig. 13 and can be compared with the actual data points. This parameter has been introduced as a further normalising parameter in Fig. 14 and it can be seen that the data then forms a unique line such that:

$$R_2 = 0.044 \left(\frac{\sigma'_{vy}}{\sigma'_{v0}} \right) \left(\frac{\sigma'_{vy} - \sigma'_c}{\sigma'_{vy} - \sigma'_{v0}} \right). \quad (3)$$

The lines on Fig. 13 were those calculated by Eq. (3). Figure 15 shows the effect of introducing the aging factor by comparing the function $R_1 + R_2$ with the data for the cyclic strength $R_{L(N=20)}$. The scatter is much reduced com-

Fig. 15. Deviation of cyclic strength data R_L from corrected function $R_1 + R_2$ ($\sigma'_{vy} > \sigma'_c > \sigma'_{v0}$) Fig. 16. Deviation of cyclic strength data R_L from corrected function $R_1 + R_2$ ($\sigma'_c < \sigma'_{v0}$)

pared with Fig. 12 and the derived values appear to fit the data well.

Effect of OCR on the Intrinsic Cyclic Strength

Following the resolution of the aging effect, data for overconsolidated remoulded and aged overconsolidated undisturbed clays was plotted in Fig. 16. In this case R_2 was applied to the undisturbed sample data but in the case of the remoulded soil, since there was no vertical yield stress σ'_{vy} due to aging, only R_1 was applied. Once again a large scatter was obtained. The deviation

Fig. 17. Cyclic strength function R_3 corrected for aging and OCR Fig. 18. Deviation of cyclic strength data R_L from corrected function $R_1 + R_2 + R_3$ (all data)

$(R_L - R_1 - R_2)$ this time was plotted in Fig. 17 against the overconsolidation ratio σ'_{v0}/σ'_c . To the left of the line for $\sigma'_{v0}/\sigma'_c = 1$ lay the samples with a residual yield stress ratio. As the overconsolidation increased so did the deviation of the cyclic strength ratio. A line was fitted to this data such that:

$$R_3 = (R_L - R_1 - R_2) = 0.17 \left(\frac{\sigma'_{v0}}{\sigma'_c} - 1 \right). \quad (4)$$

Application of this final factor for overconsolidated clays produced a reasonable fit to the data as shown in Fig. 18.

Cyclic Strength Relations

The cyclic strength has been determined as an intrinsic function of the plasticity index I_p modified for the vertical yield stress ratio $\sigma'_{vy}/\sigma'_{v0}$ or “aging” effect and geological OCR. For isotropically consolidated remoulded or undisturbed clays ($\sigma'_c > \sigma'_{vy}$):

$$R_L = R_1 = 0.0007I_p + 0.25. \quad (5)$$

In the case of an undisturbed aged clay where $\sigma'_{v0} < \sigma'_c < \sigma'_{vy}$, then:

$$R_L = R_1 + R_2 = 0.0007I_p + 0.25 + 0.044 \left(\frac{\sigma'_{vy}}{\sigma'_{v0}} \right) \left(\frac{\sigma'_{vy} - \sigma'_c}{\sigma'_{vy} - \sigma'_{v0}} \right). \quad (6)$$

Finally in the case of an overconsolidated clay there are two equations, one for undisturbed and the other for remoulded clays. For undisturbed clays:

$$R_L = R_1 + R_2 + R_3 = 0.0007I_p + 0.25 + 0.044 \left(\frac{\sigma'_{vy}}{\sigma'_{v0}} \right) \times \left(\frac{\sigma'_{vy} - \sigma'_c}{\sigma'_{vy} - \sigma'_{v0}} \right) + 0.17 \left(\frac{\sigma'_{v0}}{\sigma'_c} - 1 \right). \quad (7)$$

and for remoulded clays:

$$R_L = R_1 + R_3 = 0.0007I_p + 0.25 + 0.17 \left(\frac{\sigma'_{v0}}{\sigma'_c} - 1 \right). \quad (8)$$

UNDRAINED CYCLIC STRENGTH FOR ANISOTROPICALLY CONSOLIDATED CLAY

In the above section a formula has been developed for the cyclic strength of isotropically consolidated marine clays with varying plasticities and aging effects. However cyclic failure in plastic clays will tend to occur beneath structures where the stress conditions are anisotropic. Undrained cyclic shear tests were carried out on Ariake B, Arakawa, and Hiroshima A and B clays which had been initially isotropically consolidated for 24 hrs and then subjected to a prescribed initial drained shear stress for a further 24 hrs. Figures 19 and 20 show the cyclic strength curves for the plastic Hiroshima B clay, $I_p = 72.8$, and the low plasticity Arakawa clay, $I_p = 17.8$. In the cases of high initial drained shear stress, principal stress reversal did not occur during cyclic loading and failure occurred not as a result of high amplitude shear strains but as a result of increasing peak axial strains. It was therefore necessary to define the cyclic failure criterion as a peak axial strain of 10% or a double amplitude strain of 10% depending on which occurred first, which was itself dependent on the degree of principal stress reversal under cyclic loading.

Fig. 19. Effect of initial drained shear stress on cyclic strength of a high plasticity remoulded clay

Fig. 20. Effect of initial drained shear stress on cyclic strength of a low plasticity remoulded clay

In the case of the low plasticity Arakawa clay, Fig. 20, the increase in initial drained shear stress caused a slight increase in cyclic strength. However in the case of the plastic Hiroshima B clay this trend was reversed such that the cyclic strength decreased markedly as the initial drained shear stress ratio increased. The cyclic deviator stress ratio to cause failure after 20 cycles, has been plotted against the initial drained shear stress ratio in Fig. 21 for the full range of clays. In this figure it can be seen that for this standard definition of cyclic strength in the case of cyclic stress reversal, the cyclic strength increases with plasticity and for the higher plasticity clays, decreases with initial drained shear stress ratio. The decrease is particularly marked for the highly plastic Hiroshima B clay such that on the non-reversal side of this figure the cyclic strength decreases with increasing plasticity. This contrasts with the behaviour of sands such as that shown for Toyoura sand (Hyodo et al., 1991), where the strength increases consistently with increasing initial shear stress ratio. It can be seen that for clays, the cyclic strength defined in terms of the peak axial strain decreases at higher

Fig. 21. Variation of cyclic strength with initial drained shear stress and plasticity

Fig. 22. Variation of cyclic strength normalised with respect to isotropic cyclic strength

initial drained shear stress ratios while for sands, liquefaction is more critical under low initial shear stress ratios. As has already been mentioned, cyclic failure of clays will tend to occur under anisotropic stress conditions beneath structures. As the weight of structures increases so the initial drained shear stresses will increase with a resulting decrease in cyclic strength.

Finally we have normalised the data for all the clays shown in Fig. 21 by dividing by the cyclic strength ratio $R_{L(N=20)}=R_{ISO}$ for an initial drained shear stress ratio $\sigma_s/2\sigma'_c=0$; that is by the values of the intercepts for each curve on the vertical axis. This normalised data is presented in Fig. 22. This figure clearly demonstrates the effect of the initial drained effective stress ratio, particularly in

Fig. 23. Variation of parameter 'a' with plasticity index I_p

Fig. 24. Variation of cyclic strength with initial drained shear stress and overconsolidation ratio for a high plasticity clay

relation to the plasticity of the clay. A general equation for these curves could be written as:

$$\frac{R_{(N=20)}}{R_{ISO}} = a \left(\frac{\sigma_s}{2\sigma'_c} \right)^2 + 1 \quad (9)$$

where a varies with plasticity index I_p . Values of the parameter a have been plotted against I_p in Fig. 23. A straight line has been drawn through the points such that:

$$a = -0.032I_p + 0.725. \quad (10)$$

Using this relationship, lines have been plotted in Fig. 22 for clays of different plasticities. It can be seen that for $I_p=33$ or more there is a reasonably good fit to the data. For lower plasticity clays, the data is more scattered. In general, however, the normalised cyclic strength decreases with initial drained shear stress ratio and increasing plasticity.

Cyclic triaxial tests were carried out on Hiroshima B

Fig. 25. Variation of cyclic strength normalised with respect to isotropic cyclic strength for varying overconsolidation ratio

clay at various overconsolidation ratios from 1 to 4 with different initial drained shear stress ratios $\sigma_s/2\sigma'_c$. In Fig. 24 it can be seen that the cyclic strength appears to increase with OCR. However if each of the curves in Fig. 24 is normalised by the cyclic strength for the isotropically consolidated clay R_{ISO} then the data falls onto a single line shown in Fig. 25. The data in Fig. 24 includes that for normally consolidated Hiroshima B clay already shown in Fig. 21. Thus the normalising parameter can be applied regardless of the stress history of the soil. The shape of the curve is dependent on the plasticity index and not on the OCR.

CONCLUSIONS

Although relatively clear guidelines exist on the liquefaction behaviour of sands, a similar knowledge base for clays under cyclic loading needs to be developed, particularly for in-situ undisturbed clays. Over 100 undisturbed marine clay samples were therefore taken from alluvial deposits at many different depths in seven different urban coastal locations in Japan. The samples were then tested in both undisturbed and remoulded states. A testing programme was devised to separately examine the influence of plasticity index, aging factors and overconsolidation ratio on the undrained cyclic strength of clays.

The undisturbed samples from a variety of locations were shown to have a pre-existing vertical yield stress ratio as a result of aging. Samples were therefore isotropically consolidated to three different stress levels; a) to an overconsolidated stress state less than the in-situ overburden pressure; b) to a stress state between the in-situ overburden pressure and the vertical yield stress, defined as having an initial vertical yield stress ratio; and c) to a normal consolidation stress greater than the initial vertical yield stress level.

The cyclic data for the normally consolidated samples

was used empirically to define the cyclic strength of the clays as an intrinsic function of the plasticity index I_p . This was then modified taking into account the vertical yield stress ratio $\sigma'_{vy}/\sigma'_{v0}$, degree of aging and geological OCR. In the case of overconsolidated clays, separate equations were formulated for undisturbed and remoulded clays. The former contain terms relating to the in-situ vertical yield stress ratio and an aging factor α which measures the degree of aging remaining in a soil after consolidation. Normalised cyclic strengths were lowest for normally consolidated clays where the aging effects had been removed and increased with increasing aging factor and initial vertical yield stress ratio.

Examination of the stress path for the non-plastic clay revealed a behaviour similar to that for sands, with the stress path cycling through zero effective stress. As the plasticity increased, the behaviour changed towards a state of cyclic mobility. As the plasticity increased further the locus of the cyclic stress path became vertical and p' became almost constant, although still non-zero.

The corresponding stress-strain curves for the non-plastic clay showed it to be weaker in extension than in compression. In the case of the more plastic clays, a more even development of strain was seen with compressive axial strains increasing to about 5%. It was also observed that the cyclic stiffness of the clays increased with increasing plasticity.

The cyclic strength of anisotropically consolidated clays at low initial drained shear stress ratios increased with increasing plasticity. This reversed as the drained shear stress ratio increased. It was possible to define a simple function for the cyclic strength normalised by the isotropic cyclic strength R_{ISO} against the initial drained shear stress ratio. This function was shown to be dependent on plasticity index I_p but independent of stress history.

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