SEISMIC STABILITY ANALYSIS OF AN EMBANKMENT

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SUMMARY

This paper discribes a large scale slope failure of an embankment in a housing complex during the Miyagiken-oki Earthquake of 1978. The 25m high embankment constructed by filling a small valley on a hillside failed by the earthquake and total soil mass of $80{,}000m^3$ flew down. In order to investigate the causes of the failure, laboratory tests were conducted on disturbed samples from the site and two dimensional finite element model was considered. A simple method to evaluate the seismic stability of an embankment slope was proposed, in which the nonlinearity in stress strain relationship and pore pressure generation by dynamic load are taken into account. It is concluded from the results that the slope failure was caused by seismic force and by the reduction in shear resistance due topore pressure generated by the earthquake.

INTRODUCTION

The Miyagiken-oki Earthquake of June 12, 1978 was a magnitude 7.4 shock, which caused approximately 1,000 million dollars damage to public and private structures. Especially many houses and public utilities in housing areas of Sendai City were damaged by the earthquake. A large scale slope failure of a high embankment took place in housing complex also in Shiroishi City, Miyagi Prefecture. Total soil mass of $80{,}000m^3$ flew down from the embankment which had been constructed by filling a small valley on a hill with volcanic ash soils. Since the embankment suffered damages not only by the earthquake but also by strong rainfall in 1976, one year after the completion of construction, it is required to investigate the reasons of the slope instability.

The housing complex, Kotobukiyama, locates 2km in the east of Shiroish City, Miyagi prefecture. The housing area was constructed on a hill of 80 to 100m over sea level by cutting the hill of pumiciuos sand stone and by filling the small valley with the excavated materials and volcanic ash soils. The construction began in 1972 and finished in 1975. Before the construction benchcut was made on the natural slopes and a line of drain pipe was laid. The compaction of fill materials was not executed during the work, but 10t bulldozers ran on the fill to convey

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excavated soils. The embankment collapsed once by strong rainfalls in 1976. Site exploration was carried out after the disaster and it was pointed out that the slope was not enough stable if seismic force might act on it since the ground water level was relatively high in the embankment. Thereafter no effective countermeasure was executed to stabilize the slope untill it was destroyed by the earthquake. There found spring waters on slip surface of embankment just after the earthquake and the residual soils which flew downstreams were in extreamly muddy state.

DYNAMIC TESTS ON SOIL

In order to determine static and dynamic strength of the fill material, disturbed sample of maximum grain size of 5mm were prepared because fill materials contain so much gravel that undisturbed samples for triaxial tests could not be expected. Static triaxial test was carried out in unconsolidated undrained state on specimen, 50mm in diameter 125 mm in height, which had been prepared by compaction of disturbed sample to unit weight of $1.8t/m^3$ at water content of 28.3%. Consolidated drained test was also conducted to determine the material constants in a yield function proposed in this paper for elastic-plastic analysis of the embankment. For dynamic test, specimens were prepared in 1.7 and $1.8t/m^3$ unit weight respectively. Fig.1 shows typical results of the dynamic test. It is clearly seen in the figure that pore pressure increases gradually as number of cycles of axial loading increases. Axial strain increases also gradually at first, but it suddenly increases as pore pressure reaches to a certain value. This behaviour is similar to well known phenomena of liquefaction of saturated sand. Unfortunately distinct point of initial liquefaction and complete liquefaction could not be found in the test. If liquefaction can be defined at the point when the axial strain reaches 15 per cent, then we can get stress conditions which might cause liquefaction in a given number of cycles of loading as shown in Fig.2. With the reference to the test results on sands equation to predict the rise in pore pressure can be expressed as

$$\Delta U_N = 0.58\, N^{-0.17}(1 - U_{N-1})^{2.4}(\tau_N/\sigma_{N-1})^{2.9} \qquad (1)$$

where N is number of loading cycles
and U_N is the pore pressure at Nth cycle

It is well recognized from this figure that dinamic strength would be considerably affected by unit weight of specimen. If the material had not sufficiently been compacted and unit weight of the soil was smaller than $1.7t/m^3$, liquefaction may possibly occur in embankment. On the other hand in case of $\gamma_t=1.9\ t/m^3$, only slight increase in pore pressure was seen and sudden increase in axial strain could not find out.

For the purpose to evaluate the change in elastic constants in relation to the stress state of the material, the shear modulus was measured by resonant column method. The resonant column equipment was built especially for cylindrical specimen, but it was already proved that the test results by the apparatus were almost same as ones by hollow cylinder tests. Fig.3 shows the shear strain dependency in the shear modulus of the material. The relation is expressed neither by Hardin-Drnevich model nor by Ramberg-Osgood model, but good approximation can be obtained by the equation

$$G = a(\log \gamma + b)\sigma_o^{\,c} \tag{2}$$

where a, b, and c are material constants.
From the test results the constants are given as a=-2600, b=0.82 and c=0.6. The equation is used for the range of strain from 10^{-6} to 10^{-3} and limiting values of G are given for the strains beside this range.

ELASTIC-PLASTIC ANALYSIS OF SLOPE

The method to analyse the seismic response of embankment is pseudo-dynamic approach using the elastic-plastic stress strain relationship proposed by Hirai and Yanagisawa.(1982) Assumed that the plastic potential of the material is given by the equation

$$f = J_2 + \alpha I_1^{\,2} + \gamma I_1 = 0 \tag{3}$$

where J_2 : second invariant of deviatoric stress
I_1 : first invariant of stress
: material constant
: hardening parameter.

Although Eq.3 is the form similar to the yield function given by Burland, the hardening parameter γ is different from the one. The rate of work-hardenind parameter can be expressed in the form

$$\dot{\gamma} = \phi_1 T_i^{\ i} \dot{\varepsilon}_j^{\ j\,(p)} + \phi_2 T^{ij} \dot{\varepsilon}_{ij}^{\ (p)} \tag{4}$$

where ϕ_1 and ϕ_2 were material constants and $\dot{\varepsilon}_{ij}^{\ (p)}$ are plastic strain rates. Triaxial tests are carried out for remolded samples to determine the material constants mentioned above, and the constants are assumed as follows; α=0.052, ϕ_1=25.1, ϕ_2=8.38.

Finite element approach for elastic-plastic analysis proposed by Yamada et al (1968) was used to estimate the response of embankment to seismic loading. The original program is not relevant to soils, whose plastic behaviour depends on hydrostatic pressure, and the procedure in this program is considered to be helpful to the elastic-plastic analysis of soils. The significant modification of the program was made on the yield function and work hardening parameters. Assumed that the soil mass is subjected the increment of external load Δp, increase in stresses in all elastic element can be expressed as ΔT^{ij}. Let the initial condition of stresses in elastic state be $T^{ij}(0)$, then we can assume a stress state in a element in such a way that the stresses satisfy the yield condition

$$f(T^{ij}(0) + r\,\Delta T^{ij}) = 0 \tag{5}$$

where r is a positive constant. From the definition mentioned above, if the minimum of r is designated by r_{min}, $r_{min}\Delta p$ is the load increment possible failure plane was calculated for each element. The direction of potential failure line can be obtained from the angle between major principal stress direction and shear failure plane, which makes the angle of $45^\circ-\phi/2$ to the principale stress direction. It can be clearly seen from Fig.8 that there are many potential slip circles in the embankment. But we can imagine the circular yield zone which is estimated from the series of yielded elements in the embankment in Fig.7. If we consider potential slip surfaces in Fig.8 together with the circular yield zone in Fig.7, we can determine a few possible circular slip surfaces in the embankment. Considering the actual shape of slope after failure, the most possible slip surface can be estimated.

Time histories of response acceleration and response displacement are important quantities for structural design. As an example of time history of slope, displacement and acceleration of the top of the slope are shown in Fig.9. Stress ratio τ/σ in an element near the top of the slope and input wave are also shown in the same figure. The maximum acceleration at the top of the slope was calculated to be 380 gals, and magnification factor of the point is about 2.1 since the maximum acceleration of input wave was modified to 184 gals as mentioned before. The natural period of the embankment is estimated to be 0.4 second. It is well known that the stress ratio τ/σ, which is defined by the ratio. of dynamic shear stress and hydrostatic stress, is closely related to the liquefaction strength. Assumed that the embankment vibrates with the natural frequency of the first mode, it may be considered that the stress ratio varies principally with the period of 0.4 sec, which can be understood from the waves shown in Fig.9. If the duration of main ground motion of an earthquake is assumed, then number of loading cycles can be obtained. From the stress ratio and numbers of loading cycles, pore pressure rise in an element can be calculated by Eq.1. In case of 10 seconds duration the pore pressure ratios, which are defined by the ratio of pore pressure to the hydrostatic stress in an element, are calculated by using Eq.1 and the relation between the pore pressure ratio and number of loading cycles proposed by Seed et al (1976). Fig.10 shows the distribution of pore pressure ratio in the embankment obtained by the calculation. It is recognized from this figure that not a small amount of pore pressure is generated in the lower part of the slope. The pore pressure can significantly affect the stability of the slope. Fig.11 shows the residual displacement which is obtained by the pseudo-dynamic analysis in relation to the effective stress. The embankment is calculated to be deformed by 90cm at the top of the slope, which would be enough to recognize that the embankment is collapsed by the seismic force.

CONCLUSIONS

High embankment in a housing complex in Shiroish City, Miyagi Prefecture was collapsed during earthquake, and the causes of the slope failure was considered by a seudo-dynamic procedure. From the results obtained by the calculation, following conclusions can be made.

(1) Pseudo-dynamic procedure is developed by considering the elastic-plastic constitutive equation for stress strain relationship.

(2) Dynamic properties of fill material are investigated and empirical equations to estimate pore water pressure and the shear modulus of the material are obtained.
(3) Potential circular slip surface in the embankment was estimated from the potential failure lines in elements and from the distribution of yielded elements and the maximum acceleration.
(4) Calculating the pore pressure in elements, effective stress analysis was carried out and a large amount of residual displacement was obtained by the calculation, which suggests the failure of the slope.

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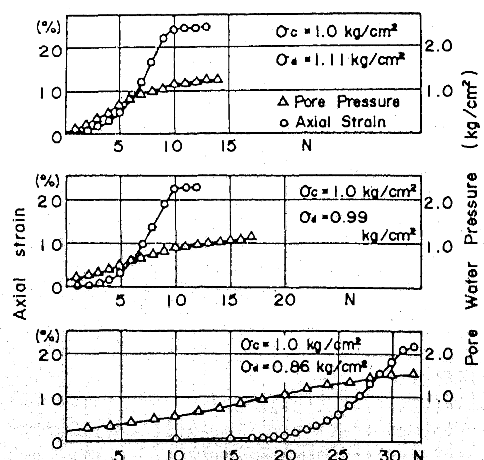

Fig. 1 Dynamic Triaxial Tests on Fill Materials

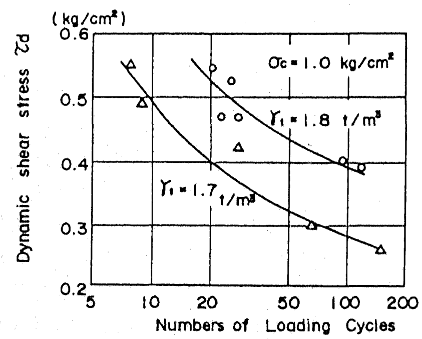

Fig. 2 Stress Condition causing Liquefaction

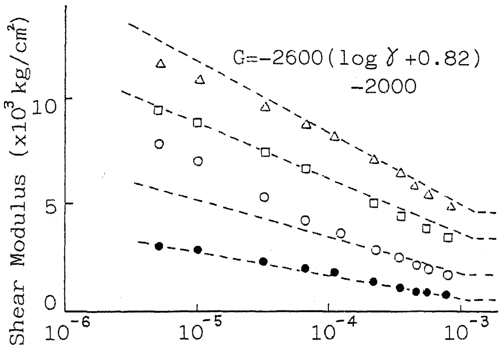

Fig.3 Shear Modulus of Fill Material

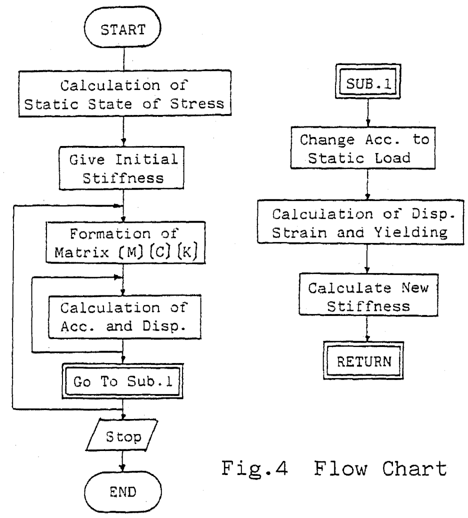

Fig.4 Flow Chart

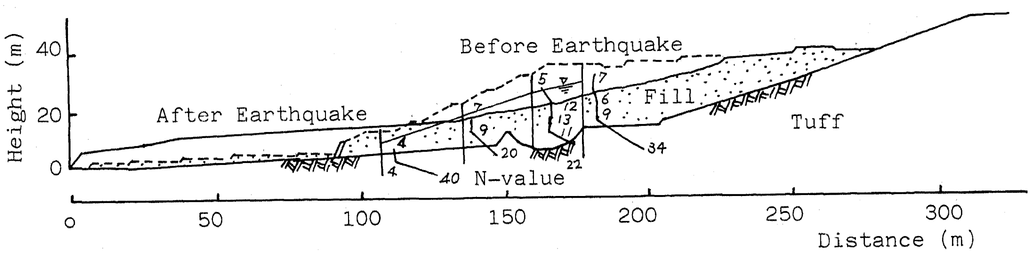

Fig.5 Cross Section of Collapsed Slope

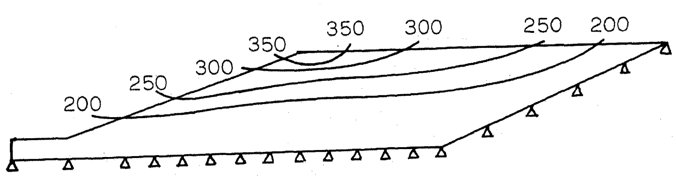

Fig.6 Distribution of Maximum Acceleration

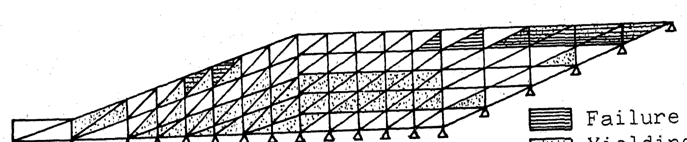

Fig.7 Yielding and Failure in Elements

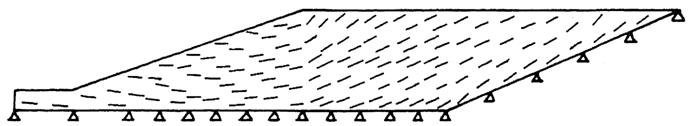

Fig.8 Potential Failure Lines in Embankment

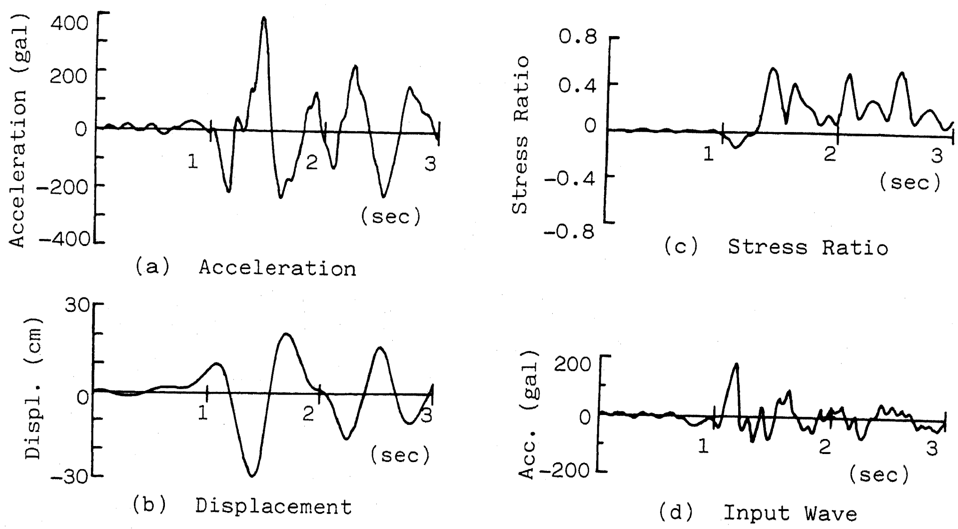

Fig.9 Response at the Top of the Slope

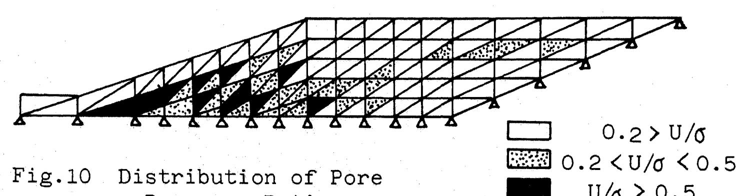

Fig.10 Distribution of Pore Pressure Ratio

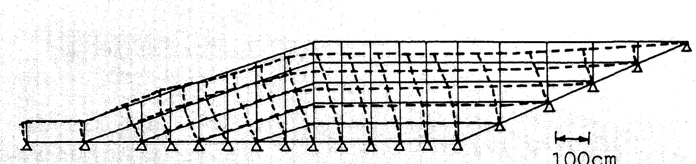

Fig.11 Residual Displacement