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CENTRIFUGE DYNAMIC MODELING TEST AND TWO-DIMENSIONAL LIQUEFACTION ANALYSIS FOR HUGE RIVER DIKE

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SUMMARY

This study compared a centrifuge dynamic modeling test result on a prototype embankment (a huge river dike) with two-dimensional liquefaction analysis. The modeling tests expressed a layered foundation ground with the Kyuedo-gawa river dike in Tokyo low land were performed, and done with unimproved ground and the cell type improved ground by using the deep mixing method. For the simulation, we used the relatively simple program combining the hysteresis type function model which was an extension of the nonlinear elasticity analysis method (the modified R-O model) with test systems leading to excess pore water pressure to evaluate the seismic resistance of earth-structure considering the problems of soil liquefaction. In this test results, maximum excess pour water pressure ratio at the soft sandy layer below the embankment tended to give rise to liquefaction, whether the cell type improved ground was used or not. But, displacements of the ground surface with the cell type improved ground were smaller than those with the unimproved ground. Especially, the vertical displacements of the berm were small. In this simulation, the data obtained with the program corresponded exactly to the pore pressures from experiments. However, the analytical displacements and accelerations were smaller than experimental results so that this program will need to be reexamined.

INTRODUCTION

Now, earthquake-proof projects of embankments (Kyuedo-gawa, Sumida-gawa and Naka-gawa river dike, etc.) are carried out in Tokyo low land, Japan. Tokyo Metropolitan Government is planning to change in haste from the river dike with the thin shaped revetment designed to protect against high flood tide to the huge river dike (refer to **Fig.1**). Particularly, as the major cause of severely damaged Yodo-gawa river dike in Osaka by 1995 Hyogoken-Numbu earthquake was liquefaction of the ground where the loose sand layer exists, it has been paid attention to Deep Mixing Method (refer to **Fig.2**). One of authors performed the embankment model experiments in 1g gravitational field for

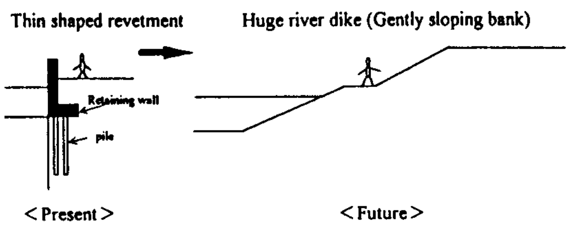

Fig. 1 Huge river dike

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confirming the effectiveness of the anti-liquefaction measure [Abe, 1995, 1996]. Many researchers [Kazama et al., 1983, Terashi et al., 1988] took the most effective Deep Mixing Method into consideration, also. As a result, the grid-shaped cell type improved ground was generally the most effective.

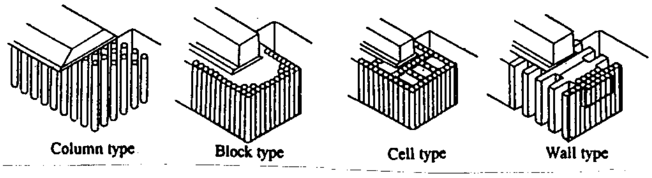

Fig. 2 Deep Mixing Method

This study compared the centrifuge dynamic modeling test results on the prototype embankment with two-dimensional direct non-linear liquefaction analysis based on the effective stress method. The modeling tests for a layered foundation ground with Kyuedo-gawa river dike were performed, and done with the cell type improved ground by Deep Mixing Method and unimproved ground.

CETRIFUGE DYNAMIC SIMULATOR

The centrifuge dynamic simulator owned by the Institute of Technical Research Laboratory, Takenaka Corporation is shown in **Fig.3**. The dimensions of the soil container (soil box made of rigid walls) with a shaking table were an internal length in the direction of vibration of 1000mm, width of 300mm, depth of 350mm [Suzuki et al., 1991]. The shaking table moved by rotation of the torque motor with a gearwheel when a high oil pressure in the accumulator released. A input motion was 1.8Hz sinusoidal wave, a horizontal acceleration was about 200gal (2m/s^2) and a centrifuge acceleration amplitude was set at 100g. **Fig.4** shows the input wave.

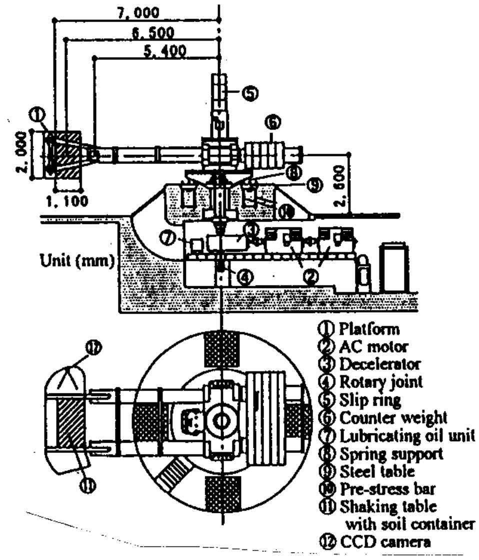

Fig.3 Centrifuge dynamic simulator

EXPERIMENTAL PARAMETER

Fig.5 shows the area of the investigation in Tokyo. **Fig.6** shows the soil profiles (Boring No.1 and Boring No.2) at the lower of Kyuedo-gawa river. The deposit consisted of, from the top, sandy layer, soft silt layer and bearing stratum. **Fig.7** shows the experimental model in the case of improved ground. The cell type improved ground made of Epoxy resin, which an area ratio of improvement was about 35%. **Fig.8** shows the monitoring system for the dynamic behavior of the experimental model in the case of improved ground. An embankment consisting of unsaturated Toyoura sand with Kaolin clay was heaped on the foundation ground which a sand layer consisted of saturated Toyoura sand (Dr (Relative density) =65%), a saturated soft silt layer was made of Nagoya bay silt (C=40□50kN/m^2) with Portland cement, a saturated bearing stratum was made of Toyoura sand (φ=37□, C=28kN/m^2) with Bentonite, and silicon oil was injected from the bottom of the soil container. The system consisting of twenty-one pore pressure transducers, six displacement transducers, eight displacement transducers in the earth, thirty-two accelerometers and five earth pressure transducers was prepared.

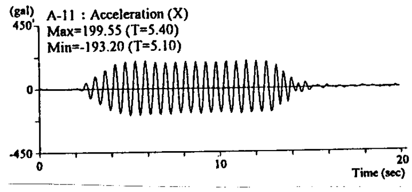

Fig.4 Input wave

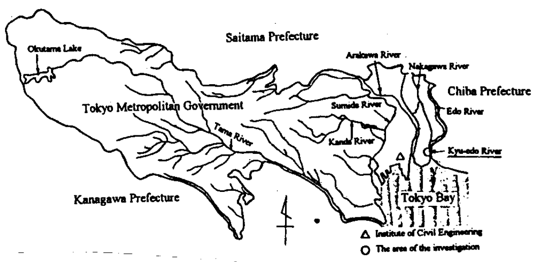

Fig.5. Area of the investigation

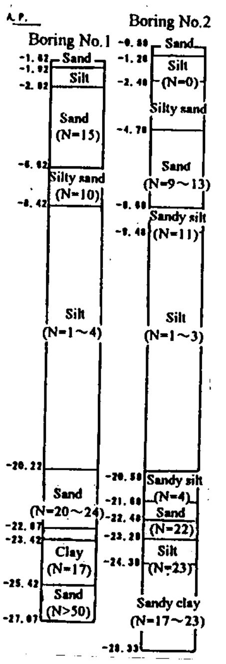

Fig.6 Soil profiles

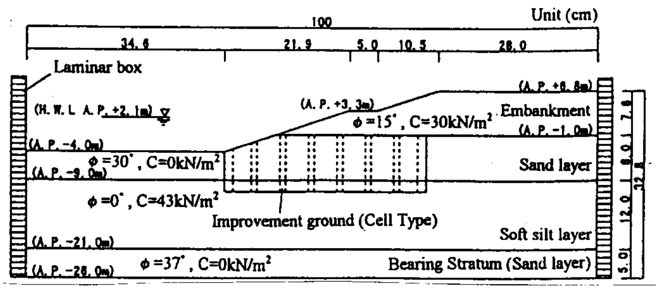

Fig. 7 Experimental model

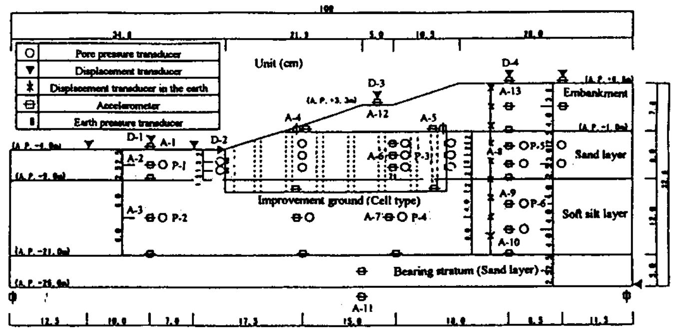

Fig.8 Monitoring system

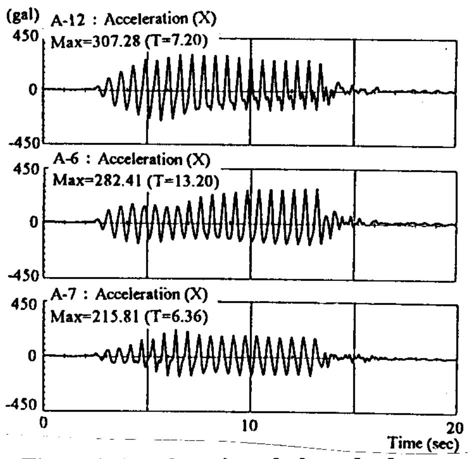

Fig.9(a) Accelerations below the berm (Unimproved ground)

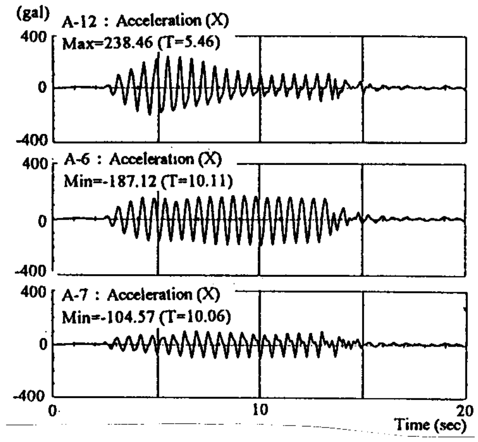

Fig.9(b) Accelerations below the berm (Improved ground)

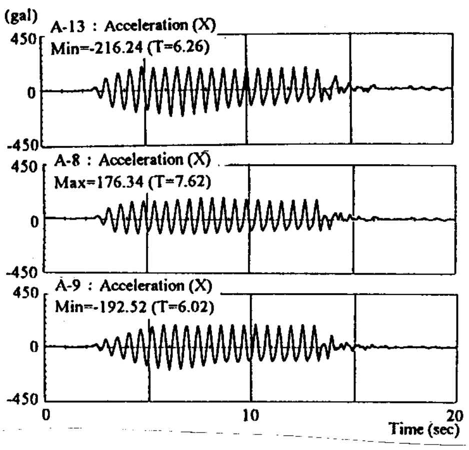

Fig.10(a) Accelerations below the embankment (Unimproved ground)

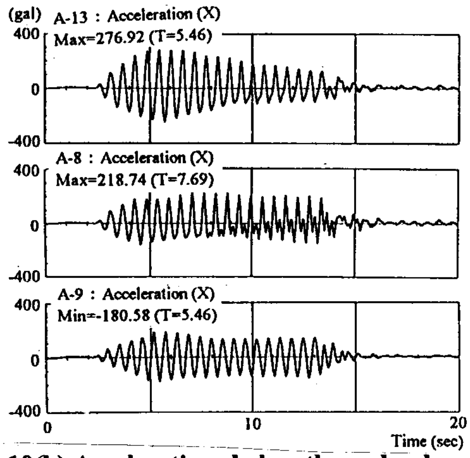

Fig.10(b) Accelerations below the embankment (Improved ground)

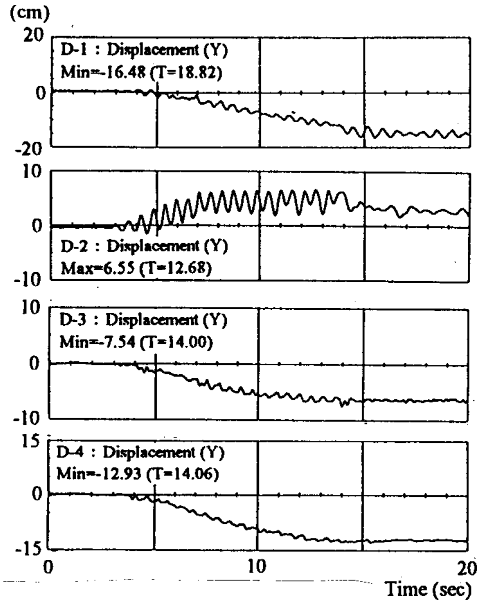

Fig.11(a) Vertical displacements (Unimproved ground)

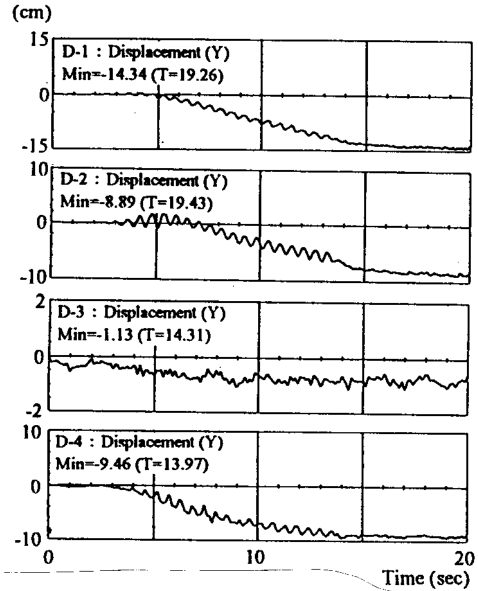

Fig.11(b) Vertical displacements (Improved ground)

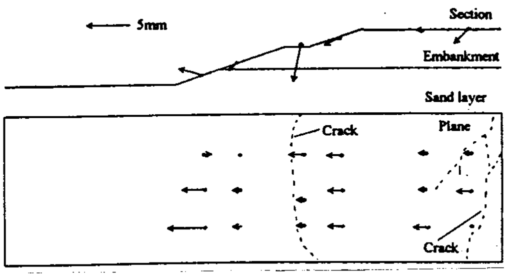

Fig.12(a) Loci of markers (Unimproved ground)

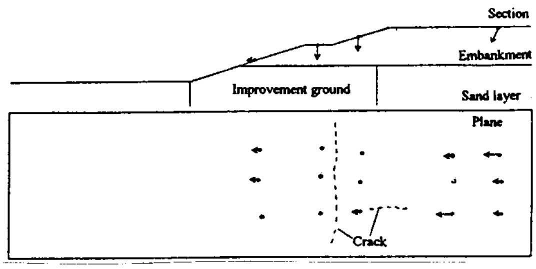

Fig.12(b) Loci of markers (Improved ground)

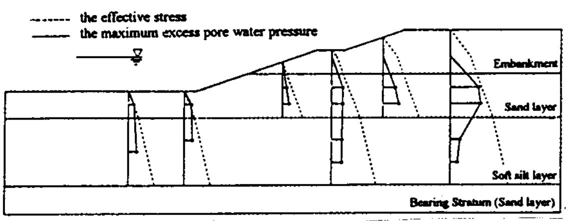

Fig.13(a) Excess pore water pressures (Unimproved ground)

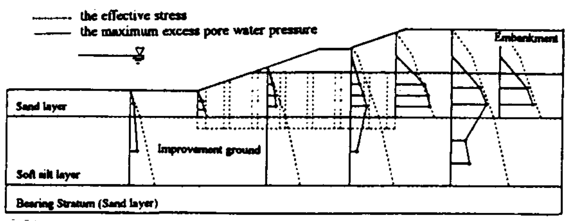

Fig.13(b) Excess pore water pressure (Improved ground)

EXPERIMENTAL RESULTS

Fig.9(a)(b) below the berm and **Fig.10(a)(b)** below the embankment show relationships between response accelerations and time histories in the case of the unimproved ground and improved ground. The accelerations of the unimproved ground were larger than those of the improved ground. Shape of acceleration waves which appears remarkably when liquefied (a period tended to be long generally), would not appear. **Fig.11(a)(b)** show relationships between vertical displacements and time histories in the case of the unimproved ground and improved ground. The vertical displacements of unimproved ground were larger than improved ground, entirely. Particularly, the vertical displacement at the berm (D-3) appeared remarkable. **Fig12(a)(b)** show the loci of markers after vibrating which set on ground surface in the cases of the unimproved ground and improved ground. The transformation of ground moved into the direction of riverbed. The cracks appeared around the berm. **Fig13(a)(b)** show distributions of the effective stress and excess pore water pressure in the cases of the unimproved ground and improved ground. Though it reached liquefaction in shallow sand layer below the riverbed, it did not reach liquefaction in sand layer below the embankment and inside cell type improved ground.

Table 1 Analytical parameters

Mat. No.	Material type	N-value	γ_t (kN/m³)	D_{50} (mm)	Fc (%)	$R_{L(20)}$	ν_B	ν_S
1	Embankment(1)	5	18.7	0.16	30	0.33	0.33	0.33
2	Embankment(2)	5	18.8	0.16	30	0.27	0.49	0.33
3	Sand	12	18.0	0.18	5	0.26	0.49	0.33
4	Silt	2	15.0	0.03	80	0.34	0.49	0.33
5	Bearing stratum (Sand)	25	19.0	0.17	10	0.30	0.49	0.33
6	Improved ground	-	19.4	-	-	-	0.16	0.16

Mat. No.	E (kN/m²)	G_o (kN/m²)	ϕ' (deg.)	ϕ_{CM} (deg.)	α'	β	h_{max} (%)	η	ξ
1	147000	31000	41	28	2.19	1.13	23	-5.5	2
2	148000	20000	40	28	2.19	1.13	23	-6.5	2
3	245000	27000	40	28	2.31	1.21	24	-7.0	1.5
4	66000	6000	38	28	1.89	0.92	20	-7.5	0.7
5	410000	33000	42	28	2.31	1.21	24	-6.0	2.5
6	240000	-	-	-	-	-	-	-	-

5. ANALYTICAL PARAMETER

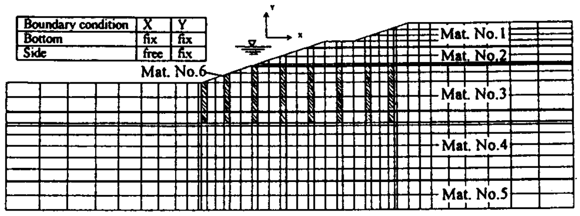

Fig.14 Finite element mesh

This program permitted seismic response analysis using direct integration method (Newmark-β) and was based on the effective stress method. It was a relatively simple program developed by combining the hysteresis type function model which was an extension of the nonlinear elasticity analysis method (the modified R-O model) with test systems leading to excess pore pressure [Abe, 1992]. The analytical parameters is shown in **Table 1**. $R_{L(20)}$ was liquefaction resistance factor at the number N of load cycle of 20. G_o was shear modulus under an effective mean stress of 9.8kN/m^2. ν_B and ν_S were Poisson's ratios (B meant volume elasticity term and S did shear term). φ' and φ_{CM} were internal friction angle and transformation angle, respectively. α' (=2^{β}) and βwere the R-O model's nonlinear parameters. h_{max} was maximum damping ratio. $\eta$was gradient of liquefaction resistance curve ($\log_{10}R_L$-$\log_{10}N$). $\xi$was an empirical parameter proposed by Seed. et al. [1976]. **Fig.14** shows finite element mesh in the case of the cell type improved ground by Deep Mixing Method. For expressing the cell type improved ground by two-dimensional analysis, finite elements corresponded to the improved ground defined by Mat. No.6 (an oblique line) were joined in beam elements.

ANALYTICAL RESULTS

In the case of the unimproved ground and improved ground, **Fig.15(a)(b)** show maximum acceleration distributions. **Fig.16(a)(b)** below the berm and **Fig.17(a)(b)** below the embankment show relationships between response accelerations and time histories, also. Analytical accelerations were smaller than experimental accelerations indicated in **Fig.9** and **Fig.10**, entirely. As good as the experiments, the accelerations of the unimproved ground by analysis were larger than those of the improved ground under the influence of the improved ground. In the case of the unimproved ground and improved ground, **Fig.18(a)(b)** show relationships between vertical displacements and time histories at ground surface, and **Fig.19(a)(b)** show transformation distributions at 20 second. Except for the vertical displacement under the slope (D-2), the vertical displacement could be restrained by the improved. But, the vertical displacements by analysis were small than those by

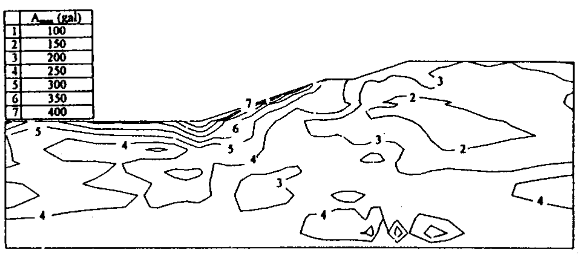

Fig.15(a) Maximum acceleration distribution by analysis (Unimproved ground)

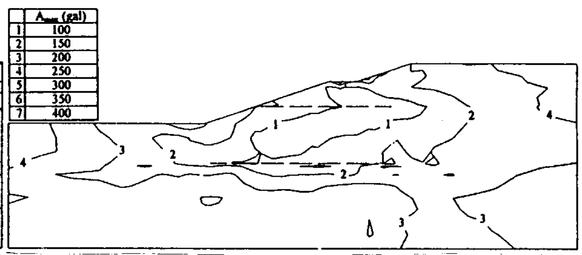

Fig.15(b) Maximum acceleration distribution by analysis (Improved ground)

experiments indicated in **Fig.11**. The heaving occurred in the case of the improved ground and whole transformation of ground by analysis tended to agree with the experimental tendency of D-2 indicated in **Fig11(b)**. **Fig.20(a)(b)** show maximum pore pressure ratios (U'_m/σ'_m: U'_m: mean pore water pressure, σ'_m: effective mean stress). Though it reached liquefaction in the shallow sand layer below the riverbed, it did not reach perfect liquefaction in sand layer below the embankment and inside cell type improved ground.

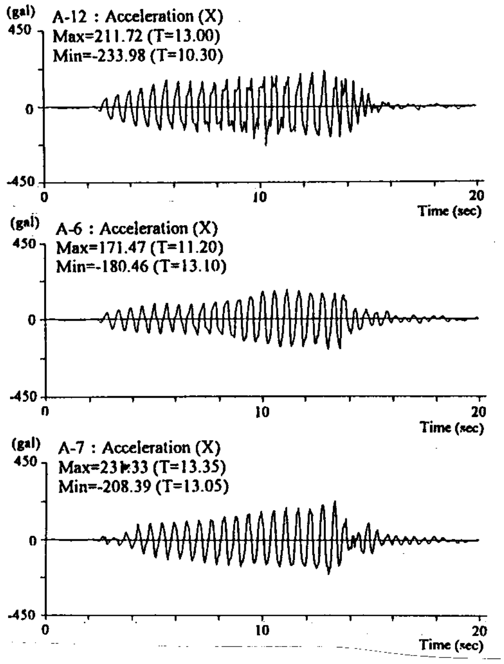

Fig.16(a) Accelerations below the berm by analysis (Unimproved ground)

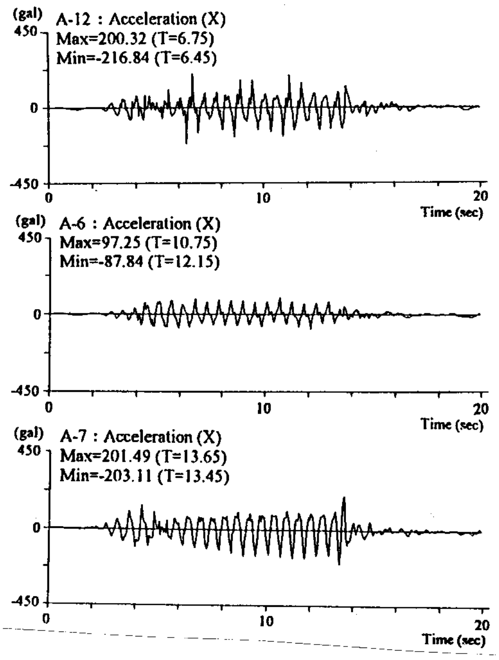

Fig.16(b) Accelerations below the berm by analysis (Improved ground)

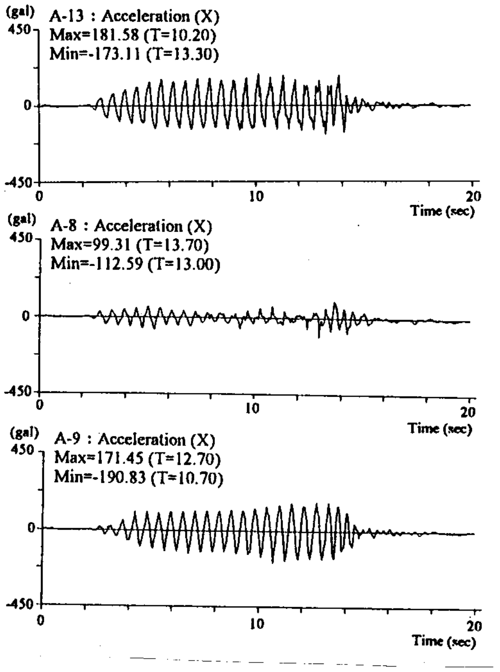

Fig.17(a) Accelerations below the embankment by analysis (Unimproved ground)

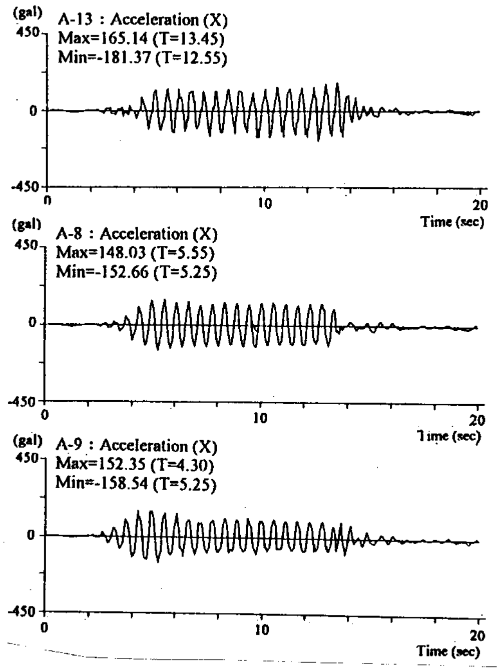

Fig.17(b) Accelerations below the embankment by analysis (Improved ground)

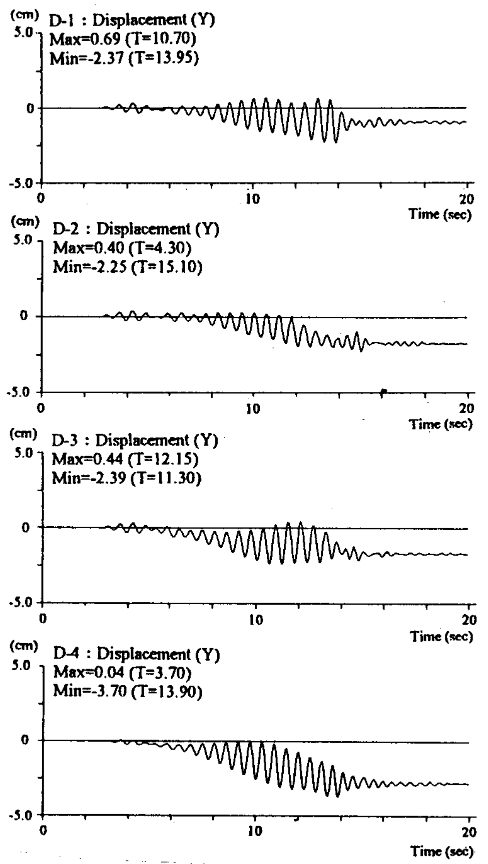

Fig.18(a) Vertical displacements by analysis by analysis (Unimproved ground)

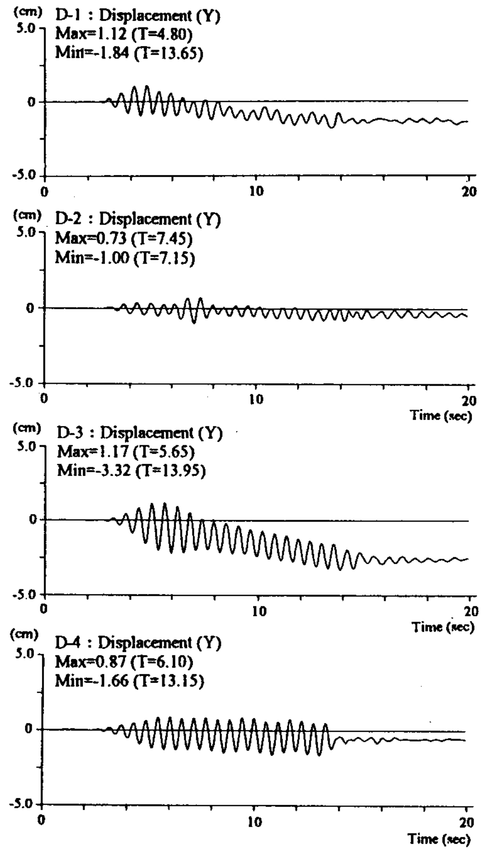

Fig.18(b) Vertical displacements by analysis by analysis (Improved ground)

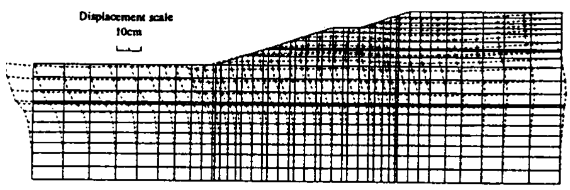

Fig.19(a) Transformation distribution by analysis (Unimproved ground)

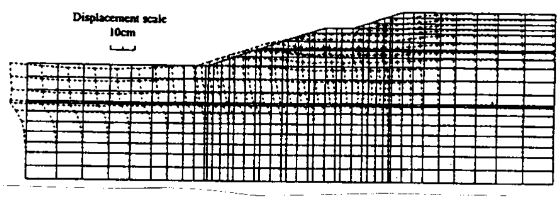

Fig.19(b) Transformation distribution by analysis (Improved ground)

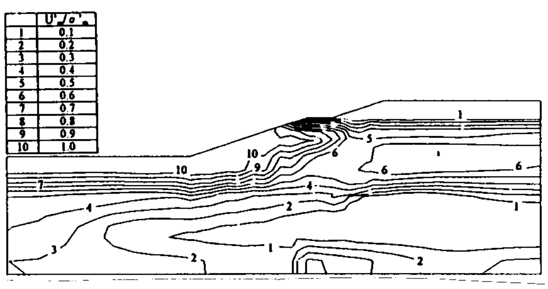

Fig.20(a) Maximum pore pressure ratio by analysis (Unimproved ground)

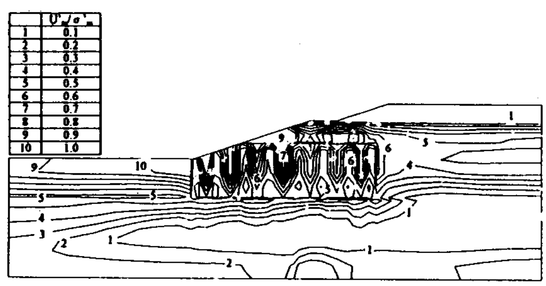

Fig.20(b) Maximum pore pressure ratio by analysis (Improved ground)

CONCLUSION

In this test results, it recognized the difference between unimproved ground and improved ground which it was possible to evaluate the seismic resistance of earth-structures with the cell type improved ground. Analytical displacements and accelerations were smaller than experimental results so that this program will need to be reexamined. However, as good as the experiment, the analysis by this program code could indicate that the improved ground with cell type was effective and explain the possibility of seismic response of earth-structures.

ACKNOWLEGEMENT

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