Studies on Seismic Earth Pressure of Cohesive Backfill

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Abstruct

Mononobe–Okabe's equation has been proposed in 1924 and has been used for the aseismic design of the retaining wall, today.

But it is well known that the effect of cohesion is not considered in this equation.

Author measured a resultant force and a distribution of seismic earth pressure acted on a movable model wall, using a shaking table test. Sand used in the tests is slightly cohesive. It was found that the results agree with the theoretical calculation proposed by the author in 1960.

So that the crtical seismic coefficiets for the base sliding of the gravity type quay walls were obtained by using our calculation and it was shown that the seismic damage can be well explained with criterion.

Introduction

The seismic earth pressure of cohesive backfill, acting on the retaining wall, is not clear yet and it's calculation is not established.

Mononobe–Okabe's equation was proposed in 1924 and has been used for the aseismic design of the retaining wall, until today.

But it is well known that the effect of cohesion is not considered in Mononobe–Okabe's equation.

Recently, some equations for the seismic earth pressure of cohesive backfill, have been proposed but these equations are not verified experimentally.

The author has proposed the calculation of seismic earth pressure, assuming that the backfill is the elastic medium with the modulus increases linearly with the depth.

The experimental results could be examined by the calculation.

In ordinary experiment of seismic earth pressure, a comparatively large sand box set on the shaking table is filled by sand and is vibrated horizontally.

Then the earth pressure act on the model wall is measured by the pressure cells mounted on the wall surface or by the load cells which suppot the wall, horizontally and vertically.

In our previous experiment[7], the pressure cell's measurement has been only used. But the measured value obtained by this mean was led to noticeable error in case of cohesive soil.

It is assumed that the error is due to the compaction at filling sand into the box.

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Therefore, the measurement of the earth pressure was performed by the load cell in the present experiment.

In this method, the resultant of the earth pressure could be measured without noticeable error.

In this experiments, the wall was slowly rotated about toe untill top's deformation become ±2.0mm (positive sign indicates movement to passive side and negative sign indicates to active side).The relationships between the seismic earth pressure and the displacement of wall were obtained.

The relationships between the displacements of wall and measured values of load cells were shown as a hysteresis loop and then the charactors of the resultant of the seismic active earth pressure and of a height of the application point, etc. were obtained.

As above mentioned, these results agree with our theoretical solution.

So that the seismic stability of gravity type quay walls were examined in order to estimate our calculation of seismic earth pressure.

Sand and apparatus

The depth of a sand box used in this experiment is 50cm and therefore soil within high cohesion can not be used in this experiment because a critical depth for a cohesion becomes above 50cm.

Toyoura sand mixed Glycerin was used as a cohesive soil (called Glycerin sand).

A cohesion of Glycerin sand obtained by a triaxial test was 1. 18 kN and then a critical depth is about 20cm.

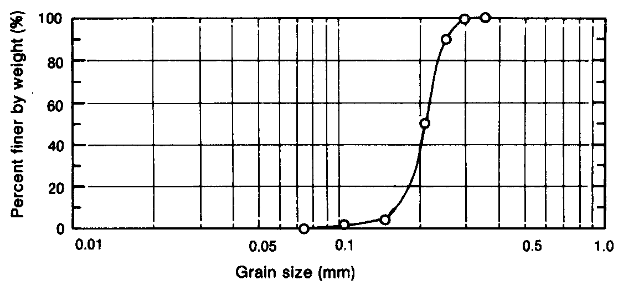

Fig. 1 Grain size distribution curve.

Table–1 Physical properties of sand.

specific gravity	G_S	2. 640
uniformity coefficient	U_C	1. 58
average grain diameter	D_{50}	0. 205mm
maximum grain diameter	D_{max}	0. 840mm

Table–2 Shearing properties of sands.

	sand	sand mixed with glycerin
cohesion (kP_a)	0	1.18
angle of internal friction (°)	37.4	31.0

Beside, in order to discussed experimental results, the experiments of dry Toyoura sand were carried out again at present.

A sand box used in this test is maken of aluminium plate and is 98cm in length, 50cm in width and 50cm in depth.

As shown in Fig. 2, both walls set at right angles to a shaking direction were

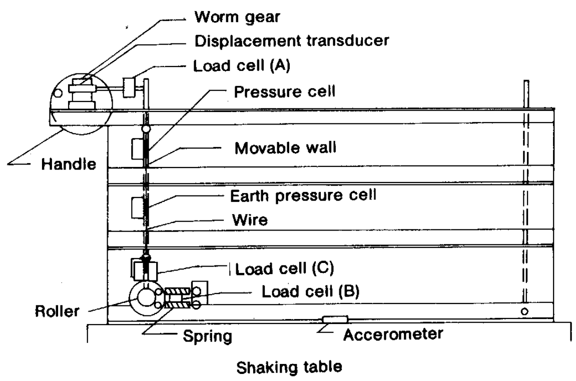

Fig. 2 Shaking sand box.

connected by hinges at toe with the box. One is supported by 5 load cells at both sides of the hinge and a top of the wall and the horizontal and the vertical resultant forces acted on the wall were measured. Also, three pressure cells were set on the wall surface at 10cm, 25cm and 40cm in respective depth and a distributions of earth pressure were measured, too.

This wall rotates slowly about toe by a rotation of handle.

The another wall sway to does not restrain a deformation of sand layer as possible, during a vibration.

These outputs of load cells and pressure cells were recorded on the penwrite recorder together with accerelation of the shaking table and displacement of the wall.

Glycerin sand was packed into a sand box, slightly compacting each of 7 equal layers by a wooden tamper. A mean unit weight of sand layer prepared by such procedure was 14.3kN/m^3. This value was obtained from three samples at third layer.

A mean unit weight of a dry sand filled in sand box was 13.4kN/m^3.

Experiment

After the filling sand into sand box, the shaking table is drived horizontally with 0.1 in seismic coefficient and 3Hz in frequency.

First, the wall is moved with rate of 0.01mm/s until a displacement of upper supporting point attain to −2.0mm.

Next, the wall is moved with same rate in the opposite direction until a displacement attain to +2.0mm and further, the wall return to original position.

At this time, the amplitude of shaking table was brought to zero and settlement of sand layer was measured. The increase of unit weight of sand layer was calculated from these settlement.

After the height of sand layer was reset to 50cm by the addition of sand, same experiments were repeated at cases of 0.2, 0.3 and 0.4 in respective seismic coefficient.

The static tests were carried out in advance of the dynamic tests.

Test results

Fig. 3 and Fig. 4 show the relationships between the resultant of the earth pressure and the displacement of the wall top. Fig. 3 is the static results and Fig. 4 is the dynamic results.

Fig. (a) and Fig. (b) in both fiqures show the results for the dry sand and the Glycerin sand respectively.

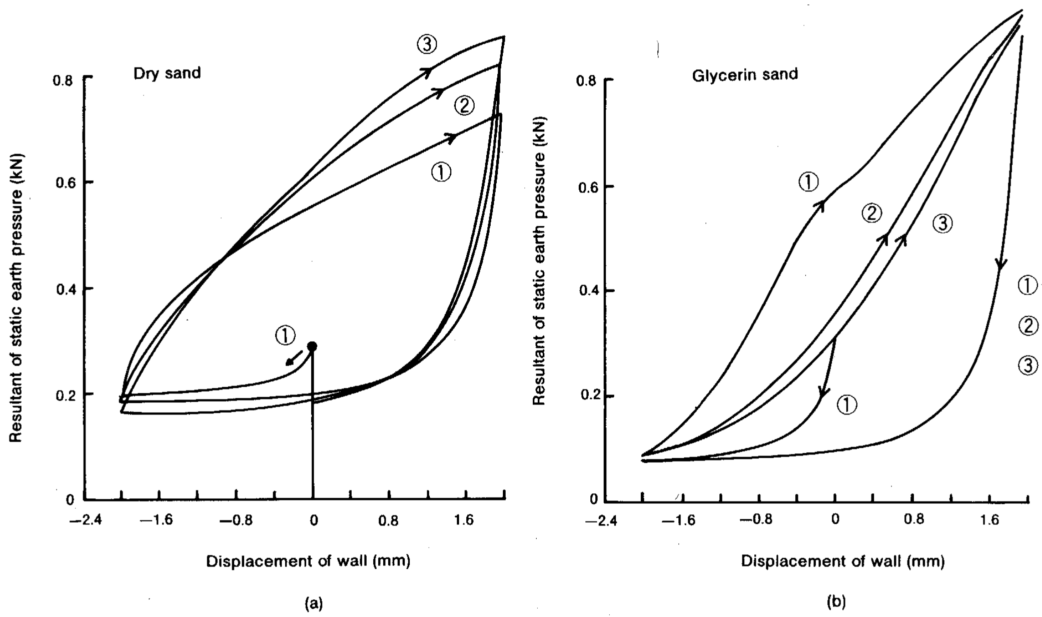

Fig. 3 Relationships between resultant of static earth pressure and displacement of wall top.

It is found in these hysteresis loops that the earth pressure fall into the active state when the displacement of the wall top increased above −0.08mm. Therefore, the experimental values of the active earth pressure were obtained at −2.0mm in the displacement of wall top.

Fig. 5 shows the relationships between the resultant force of the the seismic active earth pressure and the seismic coefficient. It can be found in Fig. 5 that the difference between both resultant forces is due to the cohesion and is independent of the seismic coefficient.

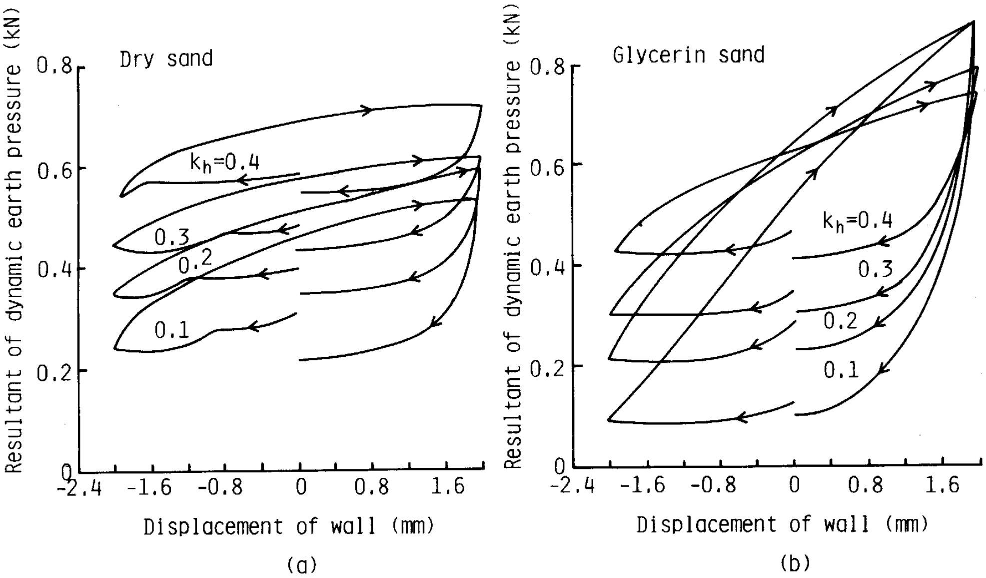

Fig. 4 Relationships between resultant of dynamic earth pressure and displacement of wall top.

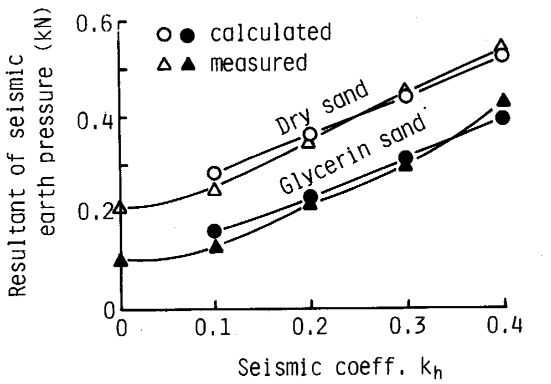

Fig. 5 Resultant of seismic active earth pressure.

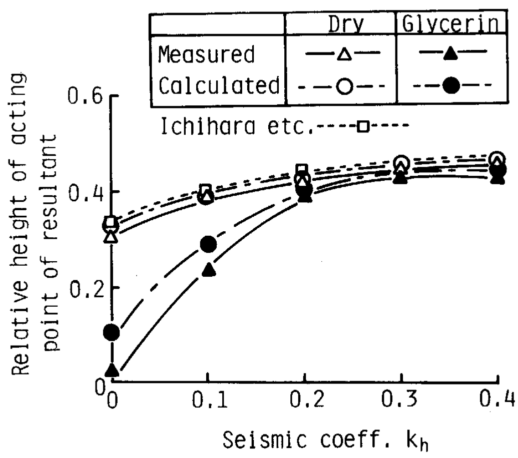

Fig. 6 Relative height of application point of resultant.

Fig. 6 shows the relationships between the height of application point of the resultant of the seismic active earth pressure and the seismic coefficient.

The height of application point increase gradually with a seismic coefficient. These same results have been obtained in other laboratory[8].

The consideration of experimental results

The authors notice that the difference between both resultants due to cohesion is constant regardless of the seismic coefficient increase. We considered that the seismic earth pressure is consist of the static earth pressure and the vibratory earth pressure. The former depends on a cohesion but it is independent of the seismic coefficent. On the contrary, the latter depends on the seismic coefficient but it is independent of the cohesion. In previous paper, the vibratory earth pressure has been calculated by the author, regarding the backfill as the elastic media whose constants increase linearly with the depth.

This calculated vibratory earth pressure for a model wall is shown in Fig. 7.

Acoording to our consideration, seismic active earth pressure can be calculated

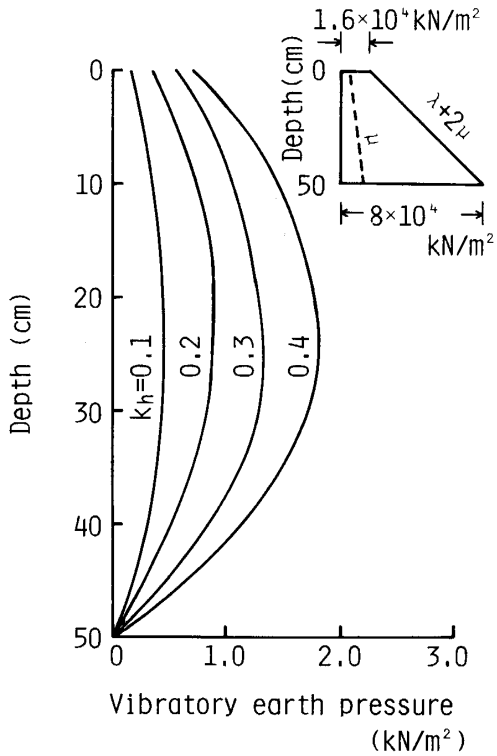

Fig. 7 Distribution of vibratory earth pressure.

by the addition the vibratory earth pressure to the static active earth pressure.

Fig. 8 and Fig. 9 show the calculated seismic active earth pressure distributions of the dry sand and the glycerin sand, respectively.

The resultant force and the height of application point of the seismic active earth pressure which were calculated from the distributions shown in Fig. 8 and Fig. 9, are shown in Fig. 5 and Fig. 6, to comparing with the measured values.

It is seen in Fig. 5 and Fig. 6 that the calculated values agree well with the measured values.

The static active earth pressure distributions shown in Fig. 8 and Fig. 9 were obtained from the static experimental results.

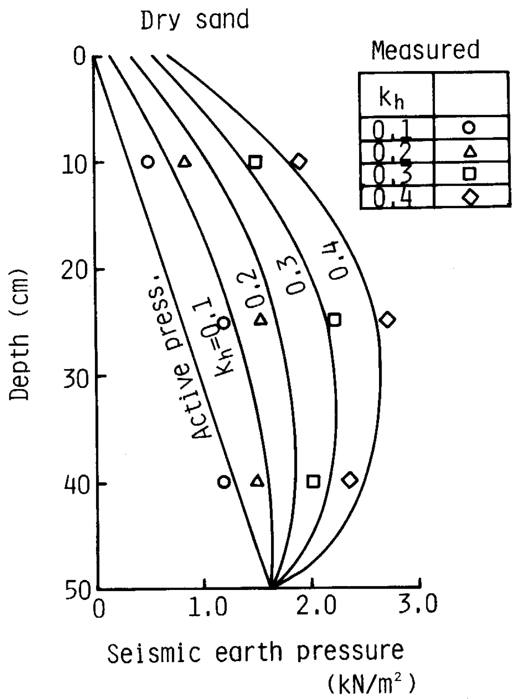

Fig. 8 Distribution of seismic active earth pressure for dry sand.

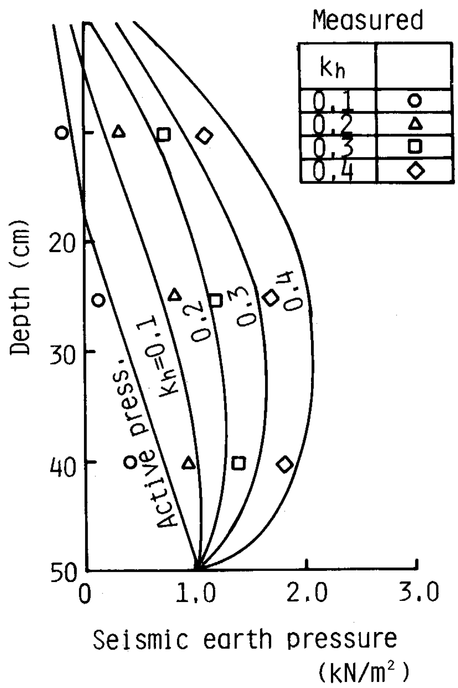

Fig. 9 Distribution of seismic active earth pressures for cohesive sand.

Examination of the seismic stability of the quay walls

As above mentioned, it is possible to estimate the seismic active earth pressure whether for a cohesionless soil or for a cohesion soil, by usinf ur calculation.

So that the seismic stability of the gravity type quay walls has been examined by the using our calculation. Fig. 10 is the diagram for calculating the vibratory earth pressure and the diagram has been proposed in 1960. The product of the ordinate value (ψ) and the seismic coefficient (k) become the resultant of th vibratory earth pressure and the height of the application point of the resultant is shown in the upper diagram. The critical seismic coefficient (k_{co}) for the base sliding of the gravity type quay wall was obtained as in Fig. 11 and it was compared with the seismic

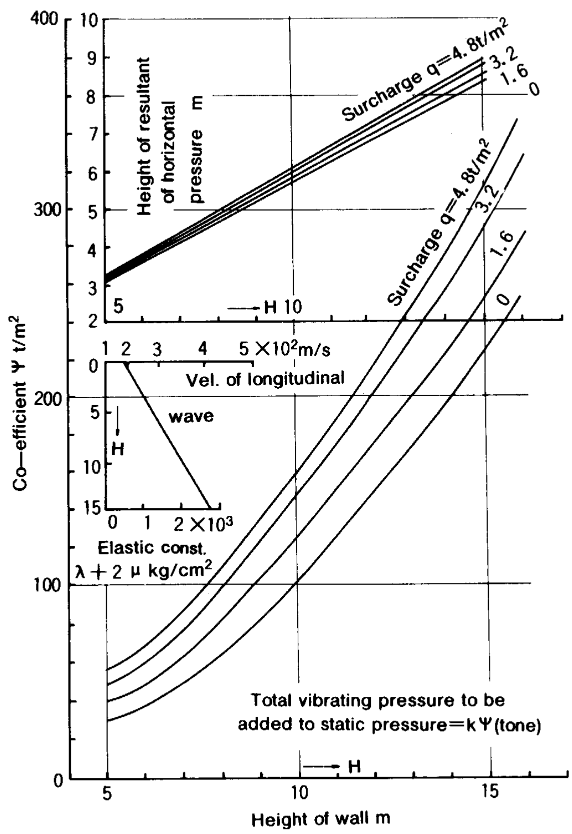

Fig. 10 Diagram for calculations of vibratory earth pressure and point of application of resultant force.

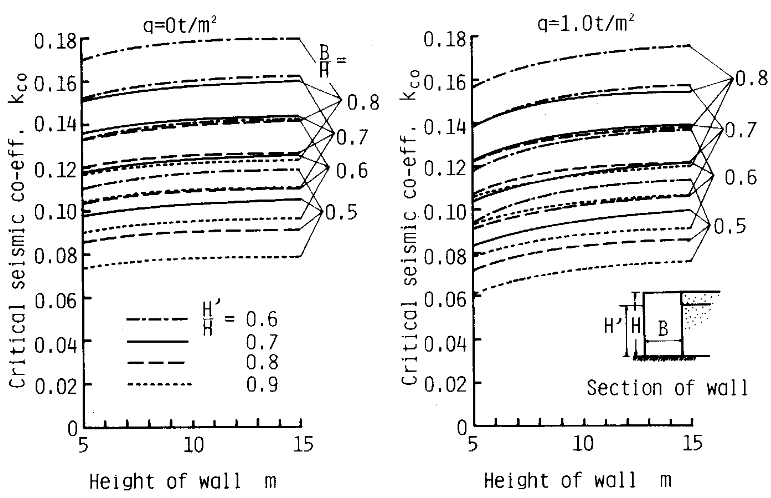

Fig. 11 Critical seismic coeff. of quay wall for base sliding.

coefficient (k) which the quay wll received during the past earthquake.

The results of the examination of the seismic stability are shown in Table. 3, where H is the total height, B is base width and H′ is the residual water depth, of the

Table–3 Examination of stability of damaged quay walls.

QW : quay wall, W : wharf, P : pier, C : caisson, B : block
◎ : serious damaged, ○ : damaged, △ : slight damaged, × : no damaged

Earth Quake	Harbor	Quay Wall	B	H	B/H	H'/H	k_{co}	k	k_{cm}	Damage
KANTO (1923)	Yokohama	Shinko W, No.1 QW (−6.1m B) (No.2)	4.20	7.16	0.59	0.62	0.134	0.20	0.114	○
		Shinko W, No.4 QW (−9.7m B)	4.30	8.79	0.49	0.64	0.116	0.20	0.096	○
KITA-IZU (1930)	Shimizu	Hinode W, A-QW (−10.1m C)	9.6	15.0	0.64	0.84	0.118	0.10	0.098	×
		Hinode W, B-QW (−8.5m C)	7.8	12.81	0.61	0.82	0.112	0.10	0.094	◎
		Hinode W, C-QW (−7.3m C)	7.0	10.80	0.65	0.78	0.108	0.10	0.092	×
		Rail way QW	7.0	11.3	0.62	0.73	0.128	0.10	0.107	×
SHIZU-OKA (1935)	Shimizu	Hinode W, A-QW (−10.1m C)	9.6	14.90	0.64	0.81	0.118	0.14	0.098	○
		Hinode W, B-QW (−8.5m C)	7.0	10.80	0.65	0.74	0.128	0.14	0.106	×
		Hinode W, C-QW (−7.3m C)	7.0	10.80	0.65	0.69	0.134	0.14	0.112	○
		Rail way QW (−7.3m C)	7.0	11.50	0.61	0.70	0.126	0.14	0.106	○
OJIKA PEN. (1939)	Hunakawa	QW (−7.3m B)	3.03	6.10	0.50	0.70	0.099	0.13	0.085	○
NANKAI (1946)	Uno	No.1 P, No.2 W (−9m B, C)	9.6	13.60	0.71	0.80	0.128	0.15	0.108	△ (pile)
		No.1 P, −5.2m QW (B)	6.51	9.60	0.68	0.73	0.137	0.15	0.117	△ (pile)

		No.2 P, −7.9m QW (C, B)	8.55	13.04	0.66	0.78	0.120	0.15	0.100	△
		No.2 P, −5.2m QW (C)	6.5	9.85	0.66	0.71	0.135	0.15	0.114	○
		No.2 P, No.2 W	8.55	11.8	0.72	0.67	0.146	0.15	0.123	○ (pile)
		No.2 P, No.1 W (C)	6.5	9.10	0.71	0.57	0.161	0.15	0.136	○ (pile)
	Saka-ide	West QW (−6.1m C)	5.45	10.00	0.55	0.81	0.100	0.14	0.084	◎
		Central W, No.3 QW(B)	7.7	13.40	0.57	0.80	0.104	0.14	0.087	○
	Koma-tsu-zima	Shinko QW (−6.4m C)	7.0	10.15	0.69	0.76	0.123	0.20	0.104	○
	Waka-yama	Nezumizi-ma QW (−6.4m B)	7.3	11.6	0.63	0.72	0.130	0.16	0.109	○
OFF TOKACHI (1952)	Ku-shiro	North W (−9.1m C)	8.3	12.62	0.66	0.85	0.098	0.20	0.084	○
		North W (−8.2m C)	7.42	11.71	0.63	0.84	0.109	0.20	0.091	○
		North W (−2.7m L−type B)	3.6	5.35	0.67	0.77	0.116	0.20	0.102	○
	Ura-kawa	No.2 W (−1.8m B)	3.0	5.50	0.55	0.63	0.123	0.22	0.106	○
	Toka-chi	W(−2.4m Cellar B)	3.0	5.30	0.57	0.66	0.114	0.22	0.099	×
OFF HYUGA (1961)	Uchi-umi	−4.7m QW (B)	5.1	8.90	0.57	0.66	0.117	0.20	0.099	△
	Abu-ratsu	-7.5m QW (C)	6.9	8.50	0.81	0.90	0.121	0.19	0.105	○
		−5.0m QW (B)	4.9	6.90	0.71	0.90	0.108	0.19	0.095	○
		−6.6m QW (C)	6.0	10.30	0.58	0.79	0.105	0.19	0.089	×
NIIGATA (1964)	Iwa-hune	W(−3m L−type B)	3.4	4.9	0.69	0.70	0.134	0.20	0.118	○
	Saka-ta	No.4 Sakata W (−3.8m B)	2.87	4.95	0.58	0.82	0.100	0.08	0.089	×
	Akita	10000t QW (−9m C)	9.0	11.6	0.78	0.84	0.139	0.10	0.118	×

OFF TOKACHI (1968)	Ura-kawa	No.3 W (L−Type B)	3.2	5.05	0.63	0.64	0.139	0.30	0.122	○
	Miya-ko	−7.3m QW (C)	7.5	10.85	0.69	0.77	0.124	0.12	0.104	×
		Desaki −4m W (C)	5.0	7.1	0.70	0.68	0.139	0.12	0.120	×
		Desaki −7.3 QW	7.5	10.85	0.69	0.67	0.141	0.10	0.118	×
	Hako-date	Central W, North −9.0m(C)	8.4	12.45	0.67	0.77	0.122	0.20	0.102	×
	Muro-ran	West No.1 W, −4.5m QW	5.0	7.5	0.67	0.60	0.152	0.22	0.130	△
		West No.1 W, −5.5m QW	5.7	8.5	0.67	0.65	0.135	0.22	0.116	○
		West No.2 W, −7.5m QW	7.5	10.50	0.71	0.71	0.144	0.22	0.122	○
	Aomo-ri	Hamamachi W, −5.5m QW(C)	4.5	8.0	0.56	0.69	0.114	0.24	0.107	○
	Ha-chi-nohe	Same, −5.0m QW	4.5	7.5	0.60	0.67	0.122	0.19	0.104	×
		Shirogane W, −9.0m QW	9.0	12.0	0.75	0.75	0.133	0.19	0.113	×
OFF NEMURO PEN. (1973)	Atsu-keshi	−4m QW (L−type B)	4.8	6.3	0.76	0.71	0.147	0.14	0.129	○
	Hana-saki	−5.5m QW (L-type B)	6.3	7.9	0.80	0.70	0.156	0.29	0.135	○
		−4.0m Central W(C)	3.5	6.1	0.57	0.63	0.113	0.29	0.099	△
	Nemu-ro	−5.5m QW (L−type B)	3.3	7.9	0.80	0.70	0.156	0.29	0.135	△
OFF IZU PEN. (1974)	Shi-moda	−4m W (B)	4.0	7.0	0.57	0.65	0.115	0.23	0.099	○
	Ina-tori	−4m W (B)	5.0	7.0	0.71	0.71	0.139	0.15	0.122	○
	Mera	−3m QW (B)	3.8	5.7	0.67	0.73	0.130	0.15	0.116	○

quay wall.

k_{cm} is the critical seismic coefficient, too and was obtained by the using Mononobe–Okabe's equation. k_{cm} is smaller than K_{co}.

Therefore the examination of the seismic stability of the qravity type quay wall was perfomed with k_{co}.

There show 51 quay walls in Table 3, the damage conditions for 37 quay walls of them are supported by the result of the examination.

Conclusions

The experimental studies were carried out, using the sand box set on the shaking table in order to investigate the effects of cohesion on seismic earth pressure.

Dry sand and Glycerin sand (mixed sand with glycerin) were used in this tests. Glycerin sand has a little cohesion.

The model wall is rotated about toe and the wall top somewhat moved by the rotation of handle.

The resultant of the seismic earth pressure was measured by 5 load cells which supported the model wall horizontally and vertically and the distribution of the seismic earth pressure was measured by 3 pressure cells set on the wall surface.

The authors remarked the following experimental results. The difference in the resultant of the seismic active earth pressure which caused by the cohesion is independent of the seismic coefficient.

Thereupon the experimental results, the resultant force and the height of application point of the seismic active earth pressure, were examined upon comparison with our theoretical solution.

It could be found from these examinations that our calculated values agree well with the experimental values.

Next, the critical seismic coefficients for the base sliding of the gravity type quay walls were obtained by using our calculation and it was shown that the seismic damage can be well explained with criterion.

Acknowledgement

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References

1) N. Mononobe "Earthquake Resistant Design of Civil Engineering Structures" (Riko, Tokyo, Revised Edition) (1952)

2) S. Okabe "General Theory on Earthpressure and Seismic Stability of Retaining walls and Dams", Journal of the Japan Society of Civl Engineers, **10**,1277–1327 (1924)

3) Japan Road Association "Specification for Highway Bridges, Part V, Earthquake Resistant De-

sign", 10−12 (1980) (in Japanese)
4) M. Ichihara, K. Yamada and G. Kajii "CALCULATION OF ACTIVE EARTH PRESSURE DURING EARTHQUAHE FOR COHESIVE SOILS POSSESSING INTERNAL FRICTION", Proc. of JSCE, No. 302, 89−101 (1980) (in Japanese)
5) H. Matuo and S. O–hara "ON THE EARTHQUAKEPROOF OF GRAVITY TYPE QUAYWALL", Proc. of JSCE,No. 70, 1−7 (1960) (in Japanese)
6) H. Matsuo and S. O − hara "LATERL EARTH PRESSURE AND STABILITY OF QUAY WALLS DURING EARTH−QUAKES", Proc. of 2nd WCEE, **1**, 165−181 (1960)
7) S. O − hara and T. Yamamoto "Experimental studies on the seismic active earth pressure of cohesive soil", JSSMFE, **30**,No. 4, 33−37 (inJapanese)
8) M Ichihara and H. Matsuzawa "Properties of Earth Pressure for Earthquake Resistant Design", Proc. of 3rd JEES, 129−136 (1970) (in Japanese)
9) S. Noda and T. Uwabe "Seismic Disasters of Gravity Quaywalls", Technical Note of the Port and Harbour Research Institute, No. 227 (1975) (in Japanese)