Damage to Masonry Retaining Walls During Niigataken-Chuetsu Earthquake

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ABSTRACT

Niigata Prefecture of Japan was hit by two large earthquakes in recent years 2004 and 2007. In particular during the 2004 Earthquake, large number of retaining walls which had supported road embankments in the mountainous area collapsed, resulting in cutting off road traffic and complete isolation of people in the neighborhood for a long time.

An extensive investigation of a number of damaged and undamaged road embankments in the mountainous area revealed that catastrophic failure of embankments constructed on sloping foundation soils were in many cases triggered by the damage of retaining walls and most of such damaged retaining walls failed in the mechanism of the bearing capacity failure of the foundation soil. In this study, laboratory tests on undisturbed sample obtained from the sites are conducted to identify strength profiles of foundation soils.

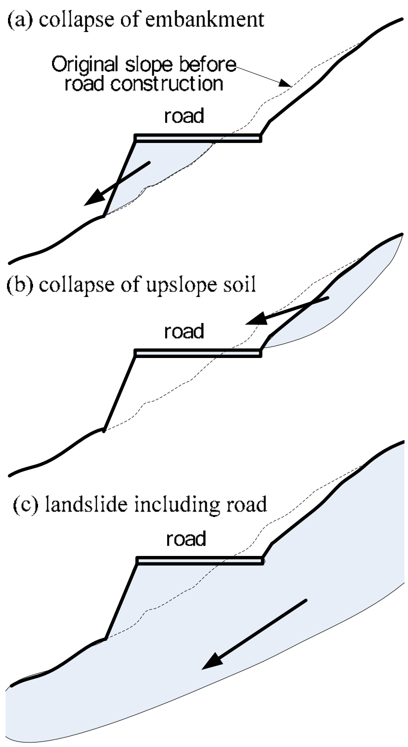
A simple, pseudo static method, to examine the seismic stability of existing retaining walls was developed, which evaluates a factor of safety for the bearing capacity failure of foundation on slope under combined loading. It was found that the factor of safety is an excellent index to sort out severely damaged walls from practically non-damaged walls. A practical method using in-situ portable dynamic cone penetration test is also proposed.

INTRODUCTION

Niigata Prefecture of Japan was hit by two large earthquakes in recent years 2004 and 2007. In particular during the 2004 Earthquake, large number of retaining walls which had supported road embankments in the mountainous area collapsed, resulting in cutting off road traffic for a long time. Many villages studded in the areas have been completely isolated due to cutting off the poorly developed road network and people had to be evacuated by helicopters.

In the current design practice, stability of road embankments and retaining walls under the action of strong earthquake ground motion is not examined since such earth structures are usually easy to restore in a short term, even if they are damaged [Japan Road Association, 1999a, 1999b]. However, this is not the scenario for road in mountainous area but plain area. Restoration works of collapsed road embankments and retaining walls in mountainous area are often extremely difficult and time consuming. A reason for this is that, for embankment supported by retaining walls on a slope, one of the typical failure mechanisms is the complete loss of roads with the embankment soil sliding down the slope. Such embankment failures are usually triggered by the cyclic softening including liquefaction of embankment soils [Matsuo et al., 2002] or collapse of retaining walls. In such cases, it is difficult to either reconstruct the road or make a detour. The other reason is accessibility to damaged locations. In cases of road collapses at more than one location, the embankments have to be fixed one by one because vehicles for the restoration work cannot access the collapsed locations without fixing the location before, while restoration works may proceed simultaneously at many locations for embankment in plain area. Thus, road embankments and retaining walls have to be earthquake-resistant if restoration works are difficult and any alternative route for emergency traffic is not available.

Based on an extensive investigation after the earthquake, Okamura and Matsuki [2007] classified typical seismic damage to road embankments in mountainous areas as illustrated in **Fig. 1**. The first mechanism is a failure of embankment itself, the second is a blockade by collapsed soil coming down from the upslope side, and the third is a large scale land slide including the road. Among these three mechanisms, damage due to the second mechanism occurs quite often but the amount of soil which blocks a road is limited and road can be reopened for traffic relatively in a short term. The soil may be removed or a road may be



temporally rebuilt on the soil. Damage by the third mechanism is extremely time consuming to fix but is rarely the case. The

Fig. 1 Typical mechanisms of damage to embankment in

mountainous area

Fig. 2 Photo of the sites F-1 and F-2 showing masonry retaining wall sliding down the slope due to bearing capacity failure

first mechanism includes embankment failure due to; (a) loss of strength of embankment soil by generated excess pore water pressure during an earthquake and (b) instability of retaining walls which support embankment on its valley (downslope) side. This is the extremely difficult type of embankment failure to restore in a short time and, in particular, failure caused by the retaining wall instability is often the case. Therefore, a simple and practical method is needed to find out seismically unstable retaining walls among existing walls.

In this study, embankment failure in mountainous area due to the instability of retaining walls is focused on. Damaged and undamaged retaining walls which supported road embankments were studied. A simple and practical method is developed which is able to examine the seismic stability of existing retaining walls.

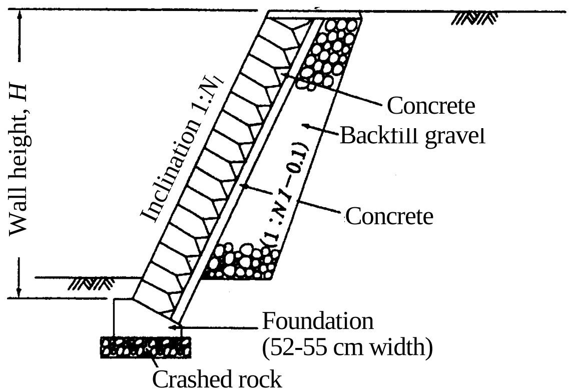
FIELD INVESTIGATION ON DAMAGED and UNDAMAGED RETAINING WALLS

A total of 12 masonry retaining walls damaged by the 2004 Niigataken-Chuetsu Earthquake was studied, which had supported road embankments in mountainous area. A typical damaged wall is shown in **Fig. 2**. Nine walls out of 12 were in the area of the JMA seismic intensity of 6-upper and the rest was in the area of 6-lower. The JMA seismic intensity of 6 corresponds roughly to maximum ground acceleration of 250 - 400 gals. At each location, undisturbed block samples were obtained and dynamic cone penetration tests using a portable device were conducted to assess strength profile of foundation soils. **Table 1** summarizes dimensions of the walls and displacements at the top of the walls caused by the earthquake.

Table 1 Summary of investigated walls

Location	JMA seismic intensity	Wall dimensions		Slope angle, β (dgree)	Displacement of wall (m)		Dyn. Cone blow count
		angle, θ (degree)	height, H (m)		Hori-zontal	Ver-tical	$N_{d,aved}$
B	6-upper	62	4.4	25	0.30	0.10	9
C		62	4.8	25	0.15	0.10	19
D-1		62	4.4	20	1.1	2.7	4
D-2		62	4.4	20	0.60	1.2	2.3
F-1		62	5.3	25	0.20	0.20	4.9
F-2		62	5.3	30	>5	-	2.4
P-1		63	4.1	25	>20	-	3.2
P-2		63	4.2	25	0	0	7
S		50	6.3	30	0.30	0.15	13
E	6-lower	69	3.7	22	0.03	0.08	4
h-1		61	4.0	79	0.35	0.80	12
h-2		66	3.5	27	0.15	0.20	2

In the design of masonry retaining walls for road embankment with a wall height lower than 5 m, the standard wall dimensions stipulated in the design manual [Public Works Research Institute, 2002] are usually employed in Japan. A typical standard cross section of wall is demonstrated in **Fig. 3**. The inclination of the wall, *N1*, which ranges between 1 : 0.3 and 1 : 0.5 corresponding to the angles 73 degree and 63 degree to the vertical, is determined according to the wall



height and type of the embankment soil, irrespective of foundation soil profile. It is rarely the case to assess the

Fig.3 Standard cross section of masonry retaining wall (after PWRI, 2000)

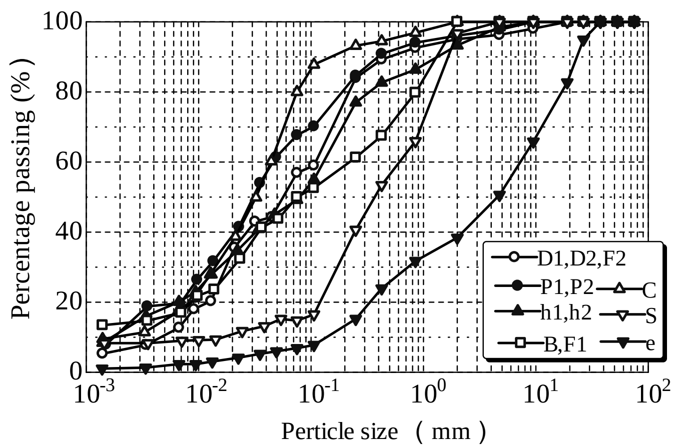


Fig. 4 Grain size distribution of samples

foundation soil conditions, therefore, stability of the walls against foundation failure widely varies between walls. The inclination angles of the studied walls were consistent with the design manual except for the wall "S" (see Table 1). For some walls the width of foundation, *B*, was directly measured and was 0.55 m. The blow count shown in the table, $N_{d,ave}$, is the average value for the depth between the foundation base and 0.6 m blow the base, i.e. to the depth of the foundation width from the base. Strength of the soil at that depth is considered to have dominant effect on the failure in the mechanism of the bearing capacity. Overall relationship between $N_{d,ave}$ value and wall deformation is that walls on the soil with lower $N_{d,ave}$ value tend to be susceptible to failure but there are walls on soil with relatively higher N_d value and vice versa..

LABORATORY TEST ON UNDISTURBED SAMPLE

Undisturbed samples were obtained at each site. **Figure 4** depicts grain size distribution of each sample. The soil was mostly silt with fraction of clay and sand, except for two locations, "S" and "e".

The samples were trimmed to triaxial specimens of some 50 mm diameter and 100mm high. For specimens containing gravel, obtained at the sites h-1 and h-2, gypsum was used to level the upper and bottom surfaces. Degree of saturation of all the specimens ranged from 30 % and 80 %. The specimen was tested at natural water content under effective confining pressures of 10 kPa and 50 kPa in the drained condition. Strength parameters obtained from the tests are given in **Table 2** together with N_d value of the soil where each sample was obtained. The N_d values are more or less the same as $N_{d,ave}$ values indicated in **Table 1**.

Table 2 Soil condition at investigated sites

Location	JMA seismic intensity	Triaxial test			PDCP at sampling location	Type of soil
		c_d (kPa)	ϕ' (deg.)	$q_u/2$ (kPa)	N_{dd}	
B	6-upper	25	0.30	0.10	9	sandy silt
C		25	0.15	0.10	19	sandy silt
D-1		20	1.1	2.7	4	sandy silt
D-2		20	0.60	1.2	2.3	sandy silt
F-1		25	0.20	0.20	4.9	sandy silt
F-2		30	>5	-	2.4	sandy silt
P-1		25	>20	-	3.2	sandy silt
P-2		25	0	0	7	sandy silt
S		30	0.30	0.15	13	clay mixed sand
E	6-lower	22	0.03	0.08	4	sandy gravel
h-1		79	0.35	0.80	12	clayey sand
h-2		27	0.15	0.20	2	clayey sand

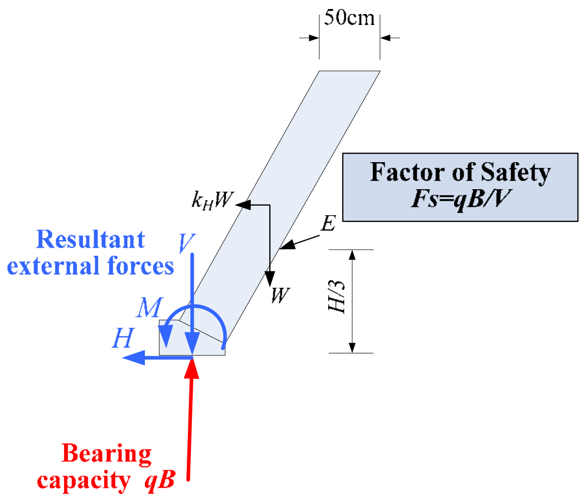
EVALUATION OF SEISMIC STABILITY OF WALLS

As previously mentioned, retaining walls in the mountainous area are in most cases resting on slope. Most of damaged walls in the area were failed in the mechanism of bearing capacity failure, i.e. sliding downward the slope. Forces acting on such a wall are schematically illustrated in **Fig. 5**. In this study, pseudo-static bearing capacity was calculated for each wall foundation, in which effects of seismic force on the wall was taken into account but on the sliding soil mass.

The bearing capacity, *q*, depends not only on strength parameters of foundation soil but also on load inclination and eccentricity. Resultant forces of self weight of the wall, *W*, active thrust, P_A , and inertia force on the wall, $k_H W$, were obtained as horizontal load *H*, vertical load *V* and moment with regard to the center of the foundation base, *M*. The bearing capacity is,

$$q = \left(cN_c + \gamma DN_q + \frac{1}{2}\gamma B' N_\gamma \right)\mu \tag{1}$$

where c and γ denote cohesion and unit weight of soil, respectively, N_c, N_q and N_γ are bearing capacity factors for inclined load with load inclination, H/V, B' ($=B-2M/V$) is effective foundation width under the action of the moment load, and μ is a coefficient to represent the effect of slope on the bearing capacity. The factors N_c, N_q and N_γ provided by Road Association [2002] and μ derived from the limit analysis



by Kusakabe [1985] were used in this study.

Fig. 5 Loads acting on a wall

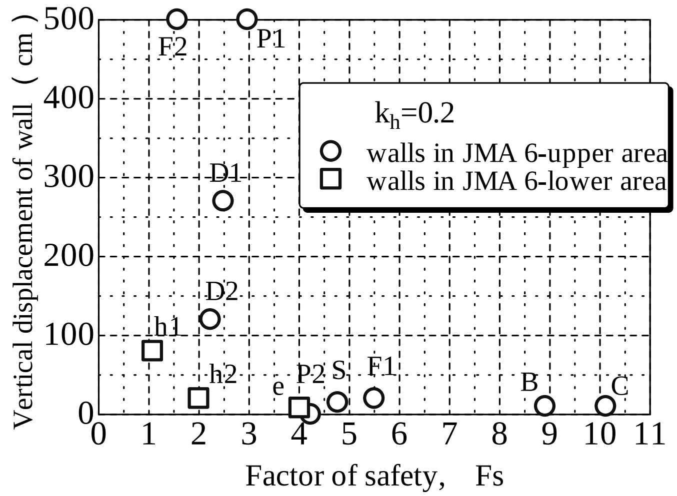
For all the studied walls, soil above the foundation was very soft with N_d value mostly lower than 2. Thus, resistance of the soil to lateral wall displacement is not taken into account but self weight was considered as overburden pressure in estimating the bearing capacity. Factor of safety against bearing capacity failure, Fs, is given as,

$$F_s = \frac{qB}{V} \tag{2}$$

in which V is the vertical component of the resultant force.

RELATIONSHIP BETWEEN DAMAGE AND F_s

Figure 6 indicates relationship between vertical displacement of the top of walls and the factor of safety. Note that the seismic coefficient k_H of 0.2 was invoked. It can be clearly seen that walls with significant settlement had lower factors of safety. For walls in the area subjected to stronger ground motion of JMA seismic intensity 6-upper, settlement is limited for walls with F_s higher than 4 and large settlement occurred for walls with F_s lower than 3. While for walls in the area of 6-lower, threshold value of F_s, below which the significant settlement occurred, is about 1.5 though the number of data point is limited. A reason for the different threshold value of F_s between walls in the two seismic intensity area is that the same value of k_H was used in the calculation. However, it is apparent that the factor of safety is an excellent index to sort out severely damaged walls from practically non-damaged



walls.

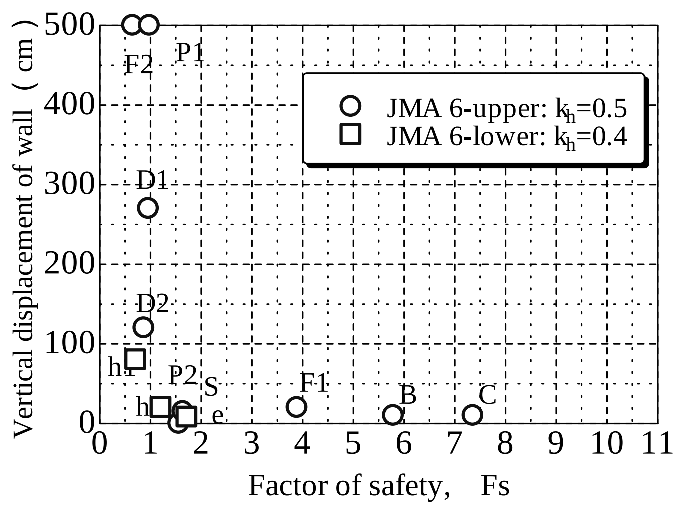


Fig.6 Relationship between vertical displacement of walls and factor of safety for cases of k_h = 0.2

Fig. 7 Relationship between vertical displacement of walls and factor of safety. k_h = 0.4 and 0.5 for areas with JMA seismic intensity 6-lower and 6-upper, respectively

In order to examine seismic stability of earth structures by pseudo-static approach, appropriate values of seismic coefficient, k_h, have been investigated. Noda et al. [1975] conducted back analysis of a number of damaged and non-damaged gravity type quay walls during past 12 earthquakes and suggested that $k_H = a_{max}/g$ for cases of moderate ground motion ($a_{max} < 0.2\,g$) and $k_H = (a_{max}/g)^{1/3}/3$ for strong ground motion ($a_{max} > 0.2\,g$) provide upper bound estimation of the seismic coefficient, in which a_{max} and g represent peak ground acceleration during the earthquake and gravitational acceleration, respectively. Tamoto et al. [1999] also analyzed damaged and non-damaged embankment using the method of arc and reported similar conclusion of $k_H = 0.8\ a_{max}/g$.

The seismic coefficients were determined so that the threshold value of F_s to be unity. The derived seismic coefficient was 0.5 for the area of 6-upper and 0.4 for 6-lower area, respectively. Relationship between vertical displacement of the top of walls and the factor of safety is shown in **Fig. 7**. It should be noted that these values are derived based on the use of the static bearing capacity and the coulomb active earth thrust where no inertia force on the sliding soil mass was taken into account. The seismic coefficient for masonry retaining walls in this study is higher than those reported for quay walls and embankment. The use of the static earth pressure and bearing capacity may be responsible for this. Provided that seismic effects on the soil mass behind the walls and foundation soil in the calculation of the active earth pressure and the bearing capacity, the seismic coefficients are expected to be lower than the above mentioned values.

A PRACTICAL METHOD TO EXAMINE SEISMIC STABILITY

In the previous section, it was confirmed that severely damaged walls were successfully sorted out from practically non-damaged walls with the use of strength parameters obtained from triaxial test on undisturbed samples. If we follow the same way in examining walls in practice, a problem arisen may be that how to determine the strength parameter to calculate the bearing capacity. In order to sort damage prone retaining walls from huge number of existing walls, practical method is necessary.

In this study, an attempt was made to estimate strength parameters based on the portable dynamic cone penetration test (PDCP). PDCP [JGS, 2004] is a test to measure blow counts to penetrate a 25 mm diameter cone 10 cm into foundation soil. A 5 kg hammer is fallen from the height of 50 cm repeatedly and blow counts (N_d value) are measured at every 10 cm penetration. The assets of this test include; (a) blow counts are measured at a short interval of penetration depth (every 10 cm), (b) a better mobility with the total weight of the testing devices including the hammer, rods and a cone being about 10 kg, and (c) quick execution of the test. A defect is that relationships between N_d value and strength parameters of soil are not well established.

N_d values at the studied site are listed in **Table 2** together with type of soils. Strength parameters are plotted against N_d value in **Fig. 8**. Sample was classified into two soil types, sand and clay, according to sand and fines fraction For sand samples drained friction angles obtained from triaxial tests are plotted in the Fig. 8(a), while for clay samples unconfined strengths, $q_u/2$, are presented in Fig. 8(b). N_d value in Fig. 8 was measured within 0.5 m from sampling location and at the same depth of the samples.

There are no well-established relationship between strength parameters and N_d , following two empirical relation were employed to correlate N_d value and friction angle,

$$N = 1.1 + 0.3N_d \quad \langle N > 4 \rangle \quad \text{(JGS [2004])} \tag{3}$$

$$N = 0.66N_d \quad \langle N \le 4 \rangle \quad \text{(JGS [2004])} \tag{4}$$

$$\phi' = 4.8 \ln N_1 + 21 \tag{5}$$

where N denotes SPT-N value and N_1 is normalized N value to account for overburden pressure [JRA, 2002]. The curve in Fig. 8(a) corresponds to the relation obtained using equations (3) to (5). On the other hand, the cohesion of clay samples increased with N_d and approximated by a parabola as shown in Fig. 8(b). It is apparent that number of data shown in Fig.8 is very limited to derive appropriate relationship, accumulation of data is needed.

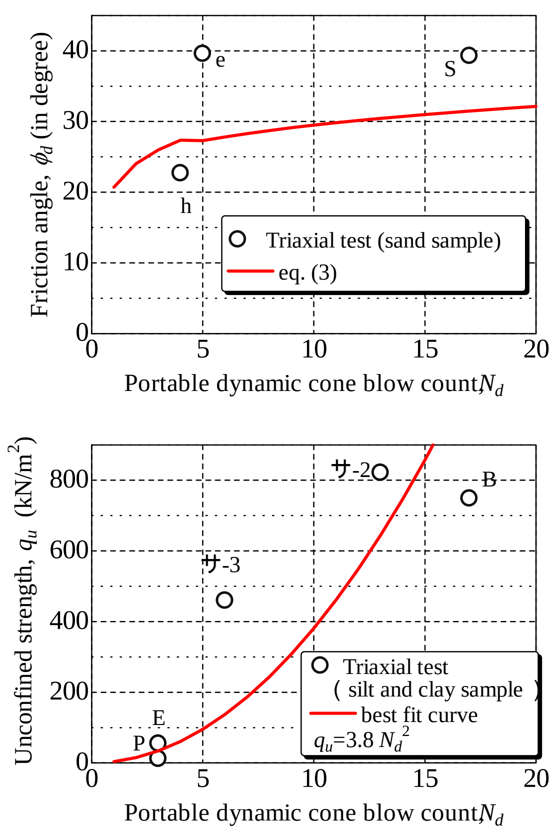
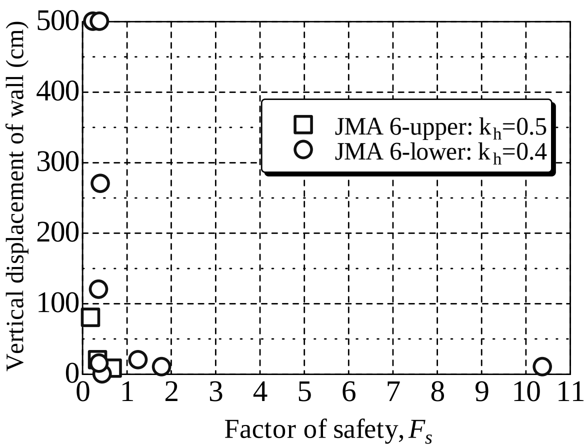


Fig. 8 Rrelationship between strength parameters and N_d value of samples

Again, factor of safety for all the 12 sites was calculated and shown in **Fig. 9**. In the calculation, two empirical relationship shown in Fig. 8 with assumptions of c=0 for sand and $\phi' = 0$ for clay were used in conjunction with observed $N_{d,ave}$ values at each site. The use of conservative strength parameters resulted in lower F_s. The factors of safety of four practically non-damaged walls out of seven are lower than unity. Authors selected walls which are studied in this paper to identify the seismic coefficient k_H and to examine the accuracy of the proposed method. For this purposes, selected wall were more or less damaged with lower seismic stability. **Figure 10** depicts the same relation as Fig. 9 but data reported by Okamura and Matsuki [2007] is also used. They investigated more than 20 walls including 10 non-damaged walls in the same area after the same earthquake. The severely damaged walls and non-damaged walls are clearly distinguished, with factors of safety of all severely damaged walls being lower than unity and those for most non- damaged walls being higher than unity.

Fig. 9 Relationship between vertical displacement of walls and factor of safety derived using N_d value.



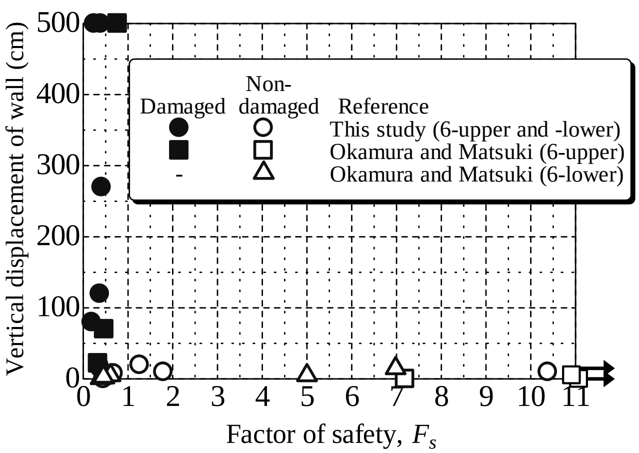


Fig. 10 Relationship between vertical displacement of walls and factor of safety derived using N_d value. Available data by Okamura and Matsuki [2007] is included.

CONCLUSIONS

An extensive investigation were made on damaged and undamaged retaining walls in the mountainous area where strong ground motions were observed during 2004 Niigataken-Chuetsu earthquake. Laboratory tests including triaxial compression tests on undisturbed sample obtained from the sites are conducted to identify strength profiles of foundation soils.

A simple, pseudo static method, to examine the seismic stability of existing retaining walls was developed, which evaluates a factor of safety for the bearing capacity failure of foundation on slope under combined loading.

It was found that the factor of safety is an excellent index to sort out severely damaged walls from practically non-damaged walls. The Seismic coefficient used to account for inertial force of a wall in the pseudo static method was found to be 0.4-0.5 for JMA seismic intensity of 6.

An attempt was made to use the portable dynamic cone penetration test for estimating in situ strength parameters. Accumulation of data is clearly needed to establish more reliable relationships, however, the proposed method is appeared to be effective examining the seismic stability of masonry retaining walls in sloping foundation soils.

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