

EVALUATION OF SEISMIC FAILURE POTENTIAL OF EXISTING RETAINING WALLS USING PORTABLE CONE

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ABSTRACT

During the 2004 Niigataken-Chuetsu Earthquake, large number of retaining walls which had supported road embankments in the mountainous area collapsed. This resulted in cutting off road traffic and complete isolation of people in the neighborhood for a long time. The authors studied a number of damaged and undamaged road embankments in the mountainous area where strong ground motions were recorded during the 2004 Niigataken-Chuetsu Earthquake. It was revealed that catastrophic failure of embankments constructed on sloping foundation soils were in many cases triggered by 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 a simple 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 in conjunction with soil strength parameters obtained by dynamic cone penetration tests using a portable testing device. A good correlation was found between the factor of safety of the surveyed walls derived from the method and the observed deformation of the walls, confirming the effectiveness of the method to identify seismically unstable walls out of existing walls.

Keywords: Retaining wall, earthquake, road embankment, bearing capacity

INTRODUCTION

The 2004 Niigataken-Chuetsu Earthquake damaged many houses and civil engineering structures. A large number of road in mountainous area was also severely damaged. 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. The roads leading to those isolated villages were mostly consisted of embankments constructed on slopes or cut slopes.

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, b). 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

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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.

Figure 1 indicates typical seismic damage to road embankments in mountainous areas. 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 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 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. A total of 30 damaged and undamaged retaining walls which supported road embankments were studied and a simple and practical method was developed which is able to examine the seismic stability of existing retaining walls.

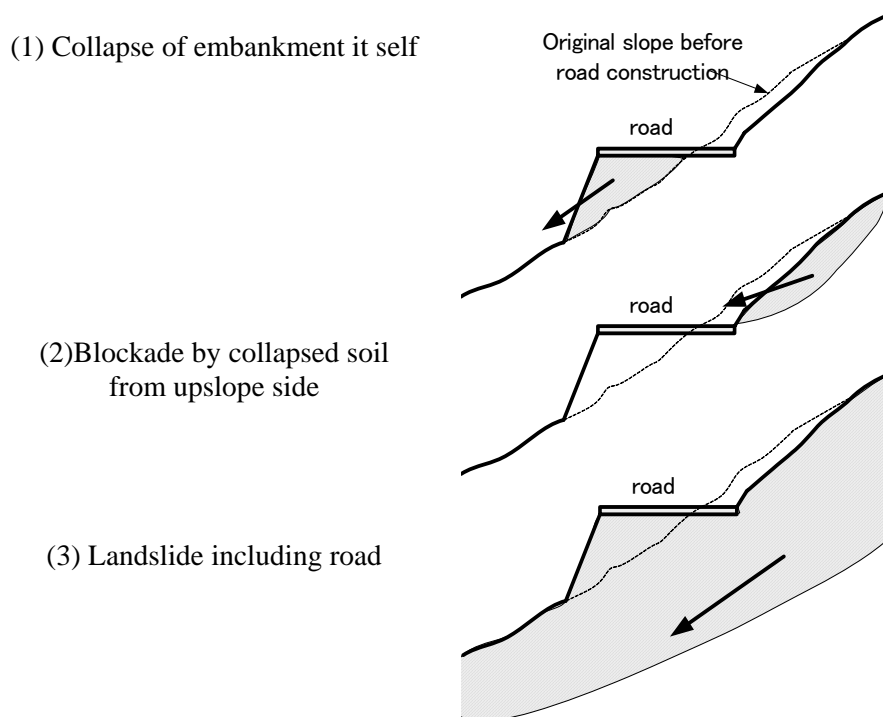


Figure 1. Typical mechanisms of damage to embankments in mountainous area

STUDY ON DAMAGED AND UNDAMAGED RETAINING WALLS

Locations and type of studied walls

A total of 30 retaining walls, most of them were damaged by the 2004 Niigataken-Chuetsu Earthquake, were studied which had supported road embankments in mountainous area. Locations of the walls are shown in **Figure 2**. 18 walls out of 30 were in the area of the JMA seismic intensity of 6-upper and 12 walls were in the area of 6-lower. At each location, dynamic cone penetration tests using a portable device were conducted to assess strength profile of foundation soils. Test results were converted into strength parameters to estimate the bearing capacity of the foundation soils as described later in this paper. The depth of the cone penetration was approximately 3 m (6 times the foundation width) below the base of the foundation of retaining walls. Type and dimensions of the walls as well as displacement caused by the earthquake are summarized in **Table 1**. 28 masonry walls and two gravity type walls were studied. **Figure 3** depicts type of retaining walls constructed by the Ministry of Construction during the period from 1994 to 1998 (Public Works Research Institute, 2002). This clearly indicates that most of the retaining walls constructed in this country are of masonry or gravity type. In particular, regarding walls constructed on slopes, the number of gravity type walls is small since the bearing capacity of foundation soil may not be sufficient. Indeed, most of the walls in the surveyed area, either damaged or undamaged, studied or not studied, were of masonry type.

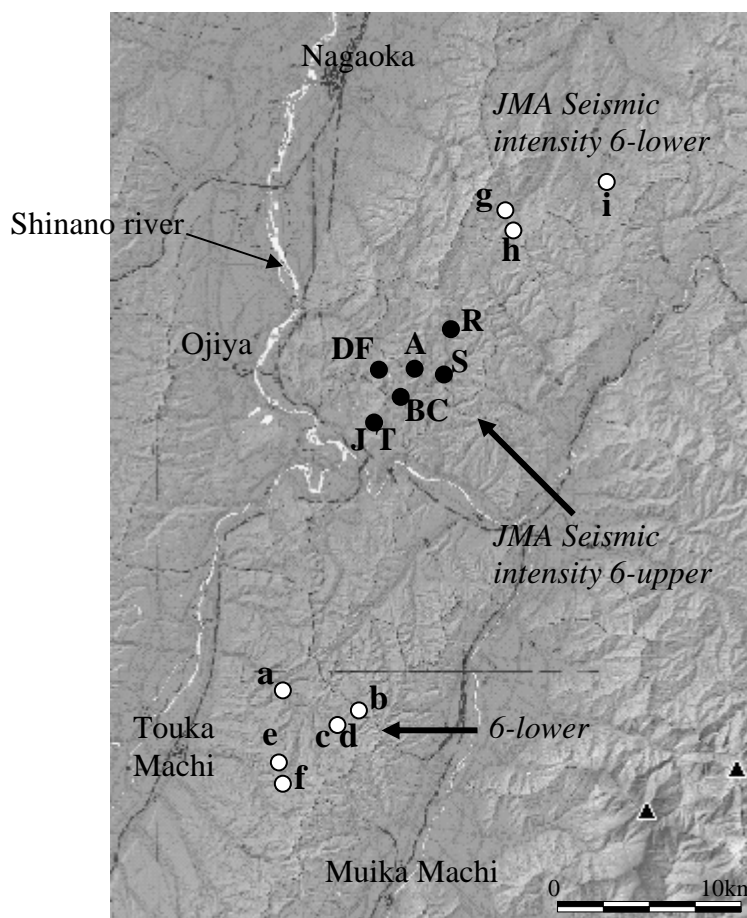


Figure 2. Locations of studied wall

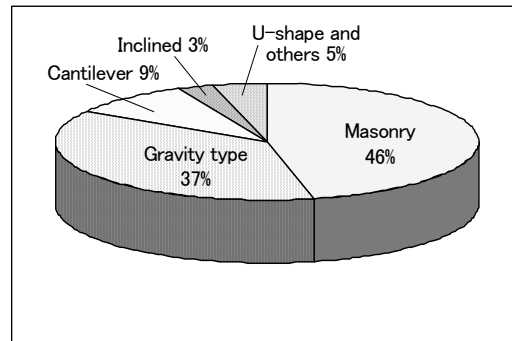


Figure 3. Type of retaining walls constructed in Japan during 1994-1998 (PWRI, 2000)

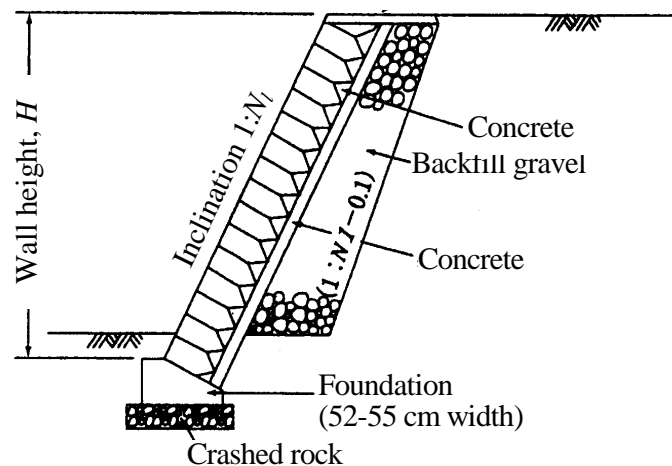


Figure 4. Standard cross section of masonry retaining wall (after PWRI, 2000)

In the design of masonry walls and gravity type walls 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 without ascertaining stability. A typical standard masonry wall is demonstrated in **Figure 4**. The inclination of the wall, N , which ranges between 1 : 0.3 and 1 : 0.5, 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 foundation soil condition, therefore, stability of the walls against bearing capacity failure widely varies between walls.

At the sites, K, N and P (see **Table 1** and **Figure 2**), a pair of a damaged wall and an adjacent undamaged wall located within a 40 m distance were studied. For such a pair of walls, conditions including seismic ground motion, embankment soil, and type and dimensions of walls are essentially identical and thus the significant differences in damage of the walls are considered to be induced by the different foundation soil condition.

Damage mechanism

For an example of damaged walls, a photograph and a cross section of walls at the site F are depicted in **Figures 5** and **6**. The locations of the wall before and after the earthquake in **Figure 5** were inferred from the wall inclination of the adjacent undamaged walls, the width of crack at the embankment shoulder and the settlement of guardrails. It can be seen in **Figure 6**, for the heavily damaged section of 40 m long including the location F-1, the walls slid down the slope and about two third of road width was lost. A clear slip surface passing through the original position of the wall foundation was observed, indicating that bearing capacity failure of foundation soil dominated the mechanism. Although there were only some minor cracks and bumps appeared on the road surface at

F-2, horizontal and vertical displacement of the wall foundation were 50 cm and 20 cm, respectively, which were large enough to cause bearing capacity failure for the 55 cm-wide foundation base.

Once the bearing capacity failure of foundation soil occurs, the bearing resistance drops from the peak resistance to the residual one. The steeper the slope angle of foundation soil, the more apparent the degradation of bearing resistance may be. Also, the degradation is more marked if the foundation subjects not only to vertical load but to horizontal and moment loads as well (Okamura et al., 2002). These facts suggest that a retaining wall on a slope is susceptible to a catastrophic failure.

Table 1 Type and height of walls, slope angle and type of foundation soil and estimated displacement due to the earthquake

Site	Type of wall	Wall height (m)	Disp. at top of walls, h: horizontal, v: vertical (m)	Slope angle of soil (°)	Type of foundation soil	Seismic Intensity in JMA scale
A	Masonry	5.1	No disp.	45	Sandy soil	6 upper
B	"	4.0	h:0.3, v:0.1	25	"	
C	"	4.5	h:0.15, v:0.1	25	"	
D-1	"	4.0	h:1.1, v:2.7	20	"	
D-2	"	4.0	h:0.6, v:1.2	20	"	
F-1	"	4.9	v:5m or larger*	25	"	
F-2	"	5.0	h:0.1, v:0.7	30	"	
J	"	4.6	h:-0.05, v:0.2	45	"	
K-1	"	4.4	No disp.	40	"	
K-2	"	1.9	h:0.1, v:0.2	30	"	
M	"	5.2	No disp.	35	Clayey soil	
N-1	"	4.3	v:30m or larger*	40	"	
N-2	"	4.3	No disp.	40	Sandy soil	
P-1	"	3.8	v:30m or larger*	25	"	
P-2	"	3.9	No disp.	25	"	
R	"	5.8	h:0.1, v:0.2	33	"	
S	"	4.8	h:0.3, v:1.0	30	"	
T	Gravity	3.5	v:30m or larger*	35	"	
a	Masonry	5.0	h:0.05, v:0.02	27	Sandy soil	6 lower
b-1	"	3.3	H:0, v:0.02	27	"	
b-2	"	2.3	h:0.05, v:0.05	50	"	
c	Gravity	3.3	No disp.	45	"	
d	Masonry	2.9	h:0.04, v:0.06	55	"	
e	"	3.3	h:0.03, v:0.08	22	"	
f-1	"	2.5	h:0.13, v:0.15	34	"	
f-2	"	4.1	h:0.10, v:0.5	34	"	
g	"	3.3	v:30m or larger*	35	"	
h-1	"	3.6	h:0.35, v:0.80	79	"	
h-2	"	3.1	h:0.15, v:0.20	27	"	
i	"	3.5	h:0.30, v:0.08	34	"	

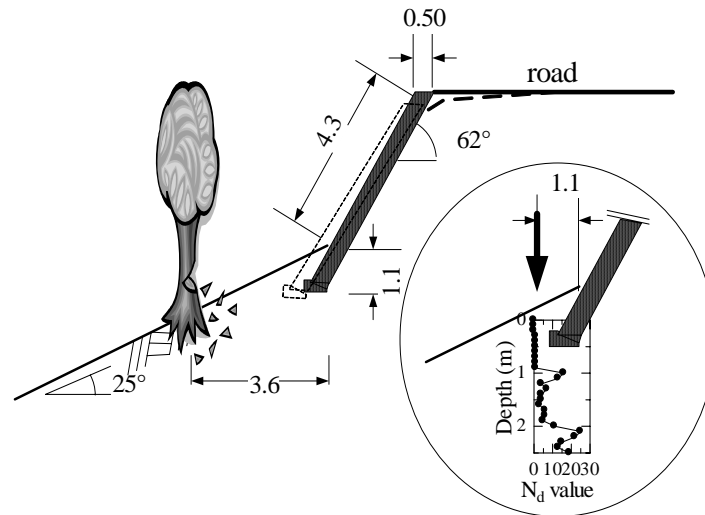


Figure 5. Cross section of road and foundation soil at F-2 and variation of blow count with depth. The arrow in the inset figure indicates the location of dynamic cone penetration test

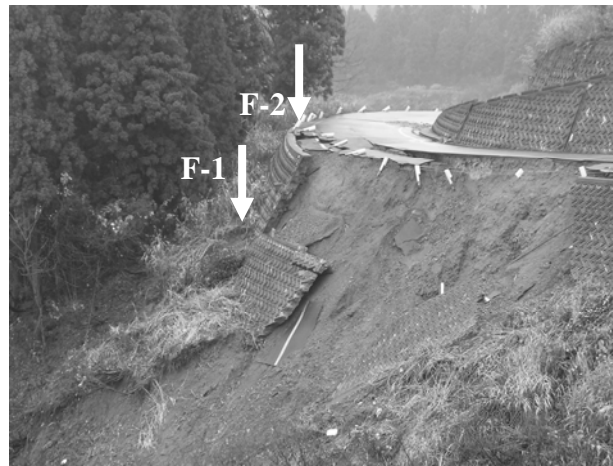


Figure 6. Photo of the sites F-1 and F-2. The arrows indicate locations of the dynamic cone penetration test

It can be seen in **Figure 6** that the embankment at F-1 was stable even though the wall displaced largely. This was also the case in some other sites. This alludes that the wall foundations caused the bearing capacity failure due mainly to the action of the earthquake-induced inertial force of the wall without significant increase in the earth pressure from the embankment soil during the earthquake.

Some embankments were still stable after the retaining wall slid down, due to the apparent cohesion of unsaturated soils.

It was found that the road failure caused by the retaining wall failure was sporadic; lengths of the failure section at a site were mostly less than 20 m. This fact indicates that in order to identify walls with less seismic stability, walls have to be examined at a relatively short interval along roads. In this study, an attempt was made to assess the seismic stability of wall against bearing capacity failure based on the dynamic cone penetration test. A portable device for the test is suitable for test in a mountainous area. The test was done in a short time, typically in half an hour at each location.

It should be noted here that there were cracked and/or broken masonry retaining walls without settlement of wall foundation. However, this type of failure rarely results in severe damage to road surface, and thus, is out of the scope of this study.

EVALUATION OF SEISMIC STABILITY OF RETAINING WALLS

Soil strength assessment by dynamic cone penetration test

Dynamic cone penetration (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) empirical relationships between N_d value and SPT N -value are available, (c) a better mobility with the total weight of the testing devices including the hammer, rods and a cone being about 10 kg, and (d) quick execution of the test. As an example of the test result, the profile of N_d value at F-2 is shown in the inset figure of **Figure 6**. Soil below the wall foundation was soft with N_d value less than 5. As listed in **Table 1**, the type of soils just below the foundations of wall was sandy soil except for the sites M and N-1.

Evaluation of stability against bearing capacity failure

Estimation of soil strength

The strength parameters, c and ϕ , of foundation soils are derived from the dynamic penetration test results as follows. Soil at the depth of 60 cm from the base of wall foundation (approximately the same as the base width) was considered and the averaged N_d value was used as a representative blow count. The depth is considered to be a typical failure zone under inclined and eccentric loads. It was reported that the relationship between N_d value and SPT N -value for sandy soil may be $N_d = (1 \sim 2)N$. In this study the following formula (JGS, 2004) is utilized to convert N_d value into N -value.

$$N = 0.67N_d \quad (1)$$

The angle of internal friction was derived using the relationship stipulated in the design code for road bridge (JRA, 2002) as,

$$\phi \text{ (in degree)} = 4.8 \ln N_n + 21 \quad (2)$$

where N_n is the normalized blow count for the effective vertical stress, σ_v , of 100 kPa ($N_n = 170N/(\sigma_v + 70)$).

For clayey soils, a relationship between N_d value and the tip resistance of cone penetration tests, q_c , has been proposed,

$$q_c = N_d / 15 \text{ (in MPa)} \quad (3)$$

The undrained strength of clay, c_u , is estimated from equation (4) assuming the cone factor N_{kt} as 12.

$$c_u = \frac{q_c - \sigma_v}{N_{kt}} \quad (4)$$

Loads acting on retaining wall

Figure 7 indicates loads acting on a retaining wall including the self weight (W), the seismic inertial force of the wall ($k_H W$), the earth thrust from the backfill (E) as well as the subgrade reaction. Vertical load due to the vertical ground motion is not taken into account. In the calculation of the earth thrust, the strength parameters for embankment soils are assumed based on the design code of road earth works (JRA, 1999b) as $c = 0$ and $\phi = 35^\circ$. Seismic effect is not considered in the calculation of the earth pressure in the same way as the design code. The seismic coefficient k_H of 0.2 which was also stipulated in the design code as a value for strong earth quake is invoked. The

resultant vertical, horizontal and moment loads regarding the center of the base were calculated for each wall.

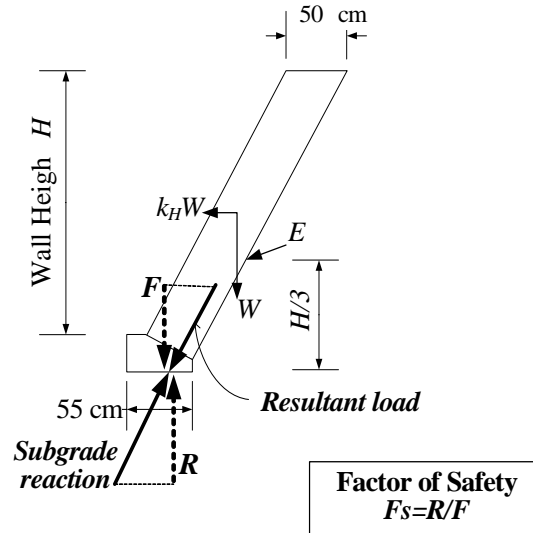


Figure 7. Loads and subgrade reaction acting on a wall

Bearing capacity

The bearing capacities of the foundation of walls should be estimated with effects of the eccentricity and inclination of the load as well as the sloping foundation soil taken into consideration. In practice, the effective width of foundation is widely used to account for the effect of the load eccentricity.

Also, practical charts of the bearing capacity factors for various load inclination angles have been developed based on the theory of plasticity (JRA, 2002). This enables us to calculate the bearing capacity of the wall foundations under the eccentric and inclined load. Also, the ratio of the bearing capacity of vertically loaded foundation near a slope to that on a level ground surface have been summarized in a simple chart by Kusakabe (1985).

It should be noted here that a practical method to account for effects of the eccentricity and inclination of loads have not been developed for foundations on a slope. Recently, Sugano et al. (2005) conducted a series of bearing capacity tests of foundation on a slope under combined loads and concluded that the effect of load eccentricity on the bearing capacity can be properly evaluated by the effective width in the same way as level ground surface. On the other hand, degradation ratio of bearing capacity of a foundation on a slope due to load inclination may be more significant than on a level ground surface. Apparently, there is a room to investigate the bearing capacity of slopes under combined loads. However, this is not the point to discuss in the present paper. In this study, the bearing capacity of wall foundations, R , is simply estimated as,

$$R = R' \cdot \mu = \left(\frac{1}{2} \gamma B_e N_\gamma + \gamma D_f N_q \right) B_e \mu \quad (5)$$

where R' is the bearing capacity of foundations on a level ground surface under combined loading, μ is the degradation ratio due to the effects of sloping ground surface, γ is the unit weight of soil (18 kN/m³ was invoked), B_e and D_f are the effective width and the embedment depth of foundation, and N_γ and N_q are the bearing capacity factor under inclined loading for the level ground condition.

The foundation base at any site was embedded to a depth ranging between 0.5 m and 1.2 m. Since the soil above the foundation base in most sites was soft with N_d value less than 2, the passive resistance of the soil above the base level was ignored in the calculation of loads acting on the walls. However,

in the calculation of contribution on the bearing capacity the weight of the soil was taken into account as an overburden pressure.

Factor of safety

Factor of safety of walls at each site was obtained as a ratio of the bearing capacity, R , to the resultant vertical load, F . The observed vertical settlement at the top of walls is plotted against the factor of safety, F_s , in **Figure 8**. Note that the arrows in the figure indicate the walls with settlement larger than 5 m. There is an excellent correlation between settlement and the factor of safety in the figure, indicating that the factor of safety derived as shown above is a good index of seismic stability of retaining walls. Two curves in the figure correspond to upper bound of walls in the area of seismic intensity of 6-upper and 6-lower. The factor of safety for walls in the area of 6-lower is apparently lower than those in the area of 6-upper. The critical values of F_s below which the wall may cause severe damage are approximately 0.6 and 0.4 for seismic intensities of 6-upper and 6-lower, respectively. This difference in F_s is attributed to the fact that the same seismic coefficient $k_H = 0.2$ was used for all walls.

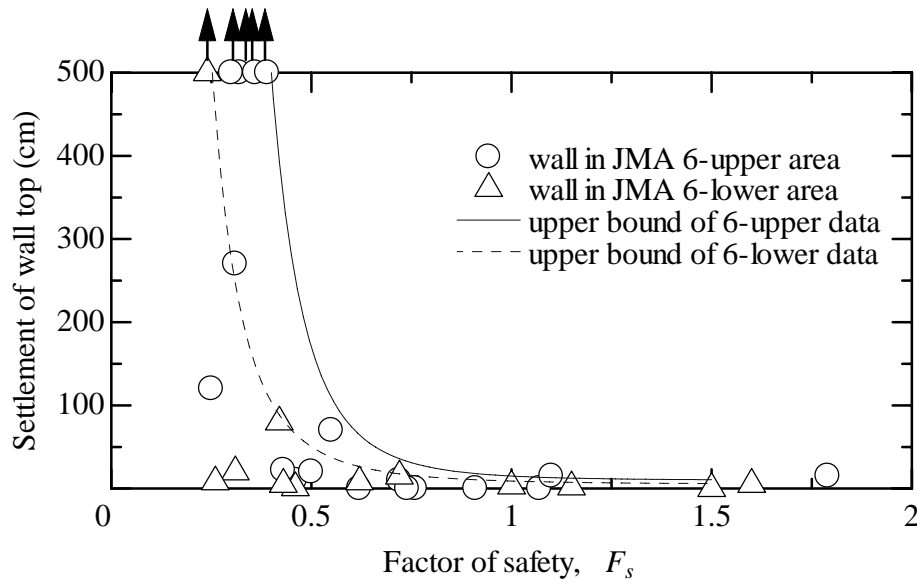


Figure 8. Relationship between observed settlement and estimated factor of safety

CONCLUSIONS

This paper deals with seismic stability of retaining walls which support road embankments in a mountainous area. The extensive survey of damaged and undamaged retaining walls after the Niigataken-Chuetsu Earthquake revealed that retaining walls founded on sloping foundation soils are brittle; once a foundation soil causes bearing capacity failure, retaining walls slid down the slope and a large part of the embankment was lost. A total of 30 damaged and undamaged retaining walls which supported road embankments were studied in the mountainous area where strong ground motions were recorded. A simple and practical method was developed to examine the seismic stability of existing retaining walls. The dynamic cone penetration test is employed to assess the strength parameters of foundation soil, which can be done using portable devices in a short time. The factor of safety for the bearing capacity failure was calculated for the studied walls. There is an excellent correlation between observed settlement and the factor of safety, indicating that the factor of safety is a good index of seismic stability of retaining walls.

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