Retaining walls damaged in the Chi-Chi earthquake

Y.S. Fang, Y.C. Yang, and T.J. Chen

Abstract: This paper investigates the failure of three gravity walls due to the 1999 Chi-Chi earthquake. Characteristics of the damaged walls were carefully recorded and backfill materials behind the damaged walls were collected and tested in the laboratory. Both the simplified analysis based on the Mononobe–Okabe method and the simplified dynamic analysis based on the Richards–Elms method were adopted. For the first case, the sliding of concrete wall blocks along the construction joint was observed. It was found that, during the earthquake, the frictional resistance at the untreated construction joint was not sufficient to resist the dynamic lateral thrust. For the second case, the retaining wall settled significantly and tilted about its toe. Seismic analysis of the wall indicated that, under the same horizontal acceleration, the factor of safety against bearing capacity failure was lower than that against overturning and sliding. A stability check against bearing capacity failure for the retaining wall should never be neglected. For the third case, the retaining wall built on top of the Che-Lung-Pu fault was severely damaged by the fault rupture. During the earthquake, the vertical displacement of the hanging wall uplifted the backfill, causing the wall to overturn. Horizontal displacement of the hanging wall caused the wall to slide and the soil in front of the toe to heave.

Key words: analysis, bearing capacity, case study, earthquake, failure, retaining wall.

Résumé : Cet article étudie la rupture de trois murs poids causées par le tremblement de terre de Chi-Chi en 1999. Les caractéristiques des murs endommagés ont été notées soigneusement et les matériaux de remblai à l'arrière des murs endommagés ont été prélevés et soumis à des essais en laboratoire. On a adopté l'analyse simplifiée basée sur la méthode de Mononobe–Okabe de même que l'analyse dynamique simplifiée basée sur la méthode Richards–Elms. Dans le premier cas, on a observé le glissement de blocs du mur de béton le long du joint de construction. On a trouvé que, durant le tremblement de terre, la résistance au frottement le long du joint de construction non traité n'était pas suffisante pour résister à la poussée dynamique latérale. Dans le deuxième cas, le mur de soutènement s'est affaissé de façon significative et s'est incliné autour de son pied. Une analyse séismique du mur a indiqué que, sous la même accélération horizontale, le coefficient de sécurité contre la rupture en capacité portante de la fondation était plus faible que contre le renversement et le glissement. Une vérification de la stabilité contre la rupture en capacité portante d'un mur de soutènement ne devrait jamais être négligée. Dans le troisième cas, le mur de soutènement construit sur la faille de Che-Lung-Pu a été endommagé considérablement par la rupture de la faille. Durant le tremblement de terre, le déplacement vertical du mur suspendu a soulevé le remblai, causant le renversement du mur. Le déplacement horizontal du mur suspendu a produit un glissement du mur, et le sol en avant du pied s'est soulevé.

Mots clés : analyse, capacité portante, étude de cas, tremblement de terre, rupture, mur de soutènement.

[Traduit par la Rédaction]

Introduction

On 21 September 1999 at 1:47 a.m., a disastrous earthquake with a magnitude of 7.3 on the Richter scale struck Taiwan. The Central Weather Bureau located the epicenter at 23.85°N, 120.82°E near the town of Chi-Chi (sometimes translated as Ji-Ji or Gi-Gi), as indicated in Fig. 1. The focal depth was about 8.0 km. The rupture of ground along the Che-Lung-Pu fault was about 105 km long. The frequency of shaking ranged from 1.0 to 4.0 Hz. The horizontal peak ground acceleration (PGA) measured at strong ground motion station TCU-065 was as high as 774 Gal (0.79*g*) as listed in Table 1. In this incident, 49 542 dwellings were totally damaged and 42 746 dwellings were partially damaged. Most unfortunately, 2432 people were killed, and 46 people are still missing. Immediately after the earthquake, the National Science Council of Taiwan mobilized more than 1200 scientists and engineers to assess the various kinds of damage due to the earthquake. The authors joined the investigation project and studied the failure of retaining structures due to the earthquake. For more information regarding the strong ground motion characteristics of the Chi-Chi earthquake, the reader is referred to Tsai and Huang (2000) and

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Fig. 1. Location of strong ground motion stations (TCU) and sites investigated.

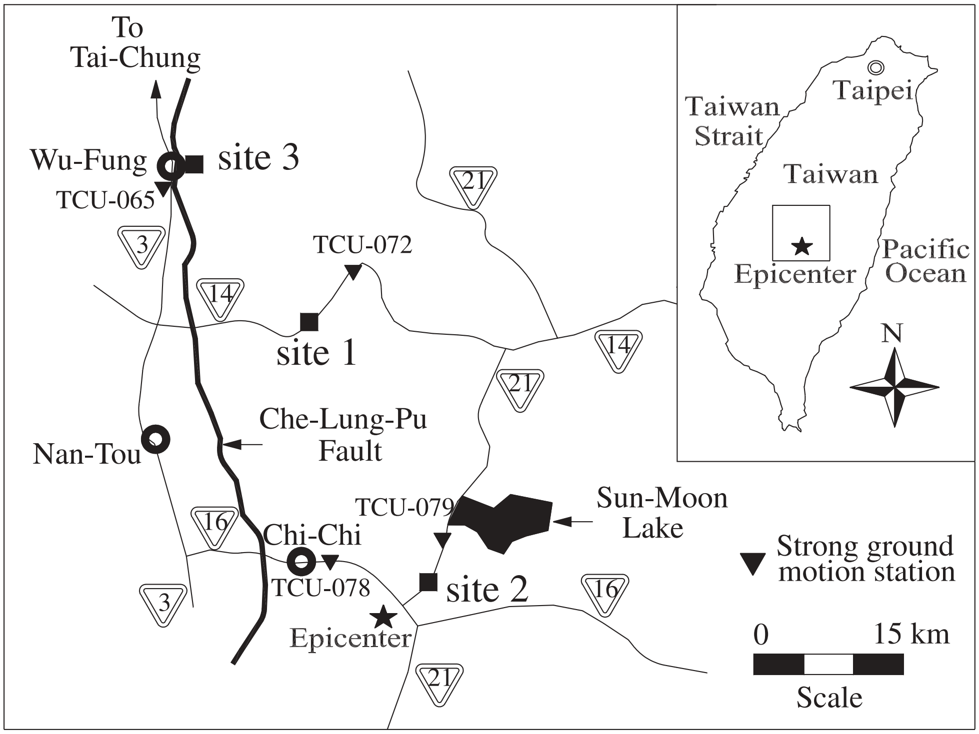


Table 1. Records of peak ground acceleration (cm/s^2) (Central Weather Bureau 1999).

		Horizontal	
Station	Vertical	East–west	North–south
TCU-065	258	563	774
TCU-072	275	371	465
TCU-078	171	302	440
TCU-079	384	417	580

Idriss and Abrahamson (2000). A dedicated issue containing 42 articles regarding the seismic aspects of the Chi-Chi earthquake was published as a Bulletin of the Seismological Society of America in October 2001.

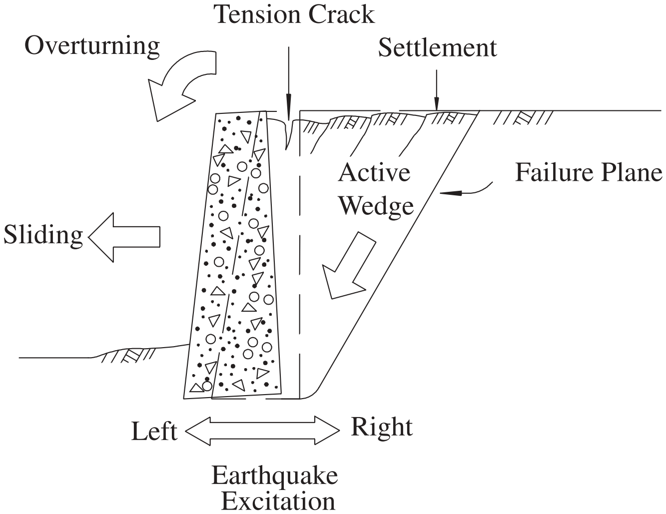
Ueng et al. (2001) reported the geotechnical hazards observed after the earthquake, including landslides, soil liquefaction, foundation failures, and ground movements. Fang and Chen (1999), Huang (2000), and Fang et al. (2001) recorded the failure of quay walls, masonry walls, gravity walls, modular-block retaining walls, and reinforced earth-retaining structures. This paper analyzes the seismic adequacy of three gravity retaining walls under earthquake excitation. Mononobe (1924) and Okabe (1924) proposed an analytical method (the Mononobe-Okabe method) to estimate the dynamic earth pressure against the wall under a seismic condition. Seed and Whitman (1970) described how to estimate the dynamic earth pressure based on the Mononobe–Okabe theory. Richards and Elms (1979) proposed a procedure (the Richards–Elms method) for the design of gravity retaining walls based on the limit-displacement concept.

In this paper, the authors introduce the behavior of retaining walls under seismic excitation. Three cases of gravity wall failure are reported. For the first case, sliding of concrete wall blocks along construction joints was observed. For the second case, the retaining wall settled significantly and tilted about its toe. For the third case, the gravity wall built on top of the Che-Lung-Pu fault was severely damaged. The research team carefully measured and recorded the characteristics of the damaged walls. Backfill materials behind the damaged walls were collected and taken to the soil mechanics laboratory. Experiments were conducted to determine the physical properties and strength parameters of the backfill. Using these parameters, an analysis was done to evaluate the seismic adequacy of the damaged walls. In this paper, the simplified analysis based on the Mononobe–Okabe (M–O) method and the simplified dynamic analysis based on the Richards–Elms (R–E) method were adopted. The intention of the paper is to document several case histories of damaged walls such that lessons can be learned from this failure.

Seismic behavior of gravity walls

Seed and Whitman (1970) reported outward movements of retaining walls and wing walls for the 1960 Chilean and 1964 Niigata earthquakes. The behavior of a gravity retaining wall under horizontal acceleration a_h can be explained with the help of Fig. 2. As the ground was shaken from left to right, the dynamic earth pressure and inertia force of the wall would act from right to left. If the resistance at the base of the wall was not sufficient to defy the dynamic thrust, active wall movement would occur. If the direction of ground shaking was reversed, however, the wall would be thrown from left to right. Due to the existence of the backfill, pas-

Fig. 2. Seismic behavior of gravity wall.



sive earth pressure would act between the wall and the soil, and only partial active wall movement could be recovered. If the wall moved outward a small amount due to each effective cycle of ground shaking, with increasing effective cycles, the wall would gradually move away from the backfill. As indicated in Fig. 2, with the outward movement of the wall, the active soil wedge would slide down along the failure plane, causing settlement and tension cracks in the backfill.

Site 1: sliding along construction joints

The damaged retaining wall was located along Taiwan Provincial Highway 14 and is labeled as site 1 in Fig. 1. The gravity wall was built to retain a steep excavation for highway construction. The wall was 4.7 m high, 2.0 m thick at the base, and 0.6 m thick at the top as illustrated in Fig. 3. The retaining structure was constructed by placing concrete in five pours. It should be mentioned that no special treatment had been provided at the construction joints of the wall. During the Chi-Chi earthquake, the upper parts of the wall gradually moved away from the backfill along the construction joints. Eventually, the top two blocks fell down to the side ditch of Highway 14 as indicated in Fig. 4. The damaged zone was approximately 20 m long. Based on field observations, it was suspected that the frictional resistance at the flat construction joint was not sufficient to resist the dynamic earth pressure and the inertial force of the upper part of the wall. It was clear that a more detailed analysis would be needed

Soil testing

After the earthquake, the authors joined the damage reconnaissance team organized by the National Science Council of Taiwan. Backfill materials behind the damaged walls were collected at the site and tested in the soil mechanics laboratory of the National Chiao Tung University. Physical property tests and standard Proctor compaction tests (ASTM 2000) were conducted. Physical properties for backfill obtained from sites 1, 2, and 3 are summarized in Table 2, and

Fig. 3. Profile of the damaged retaining wall at site 1.

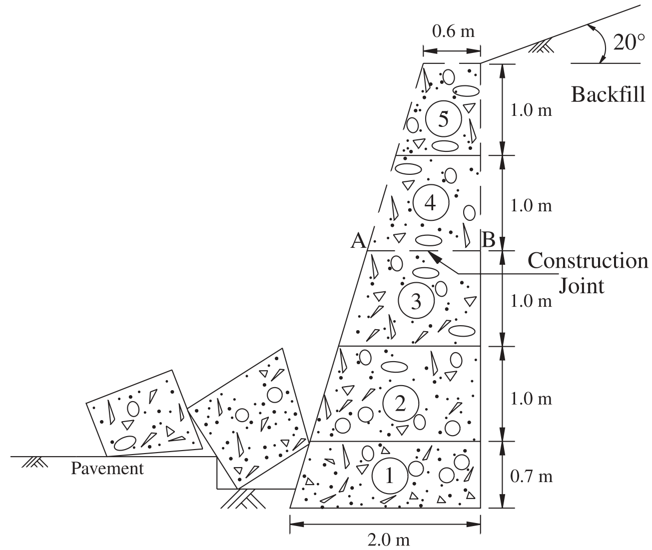


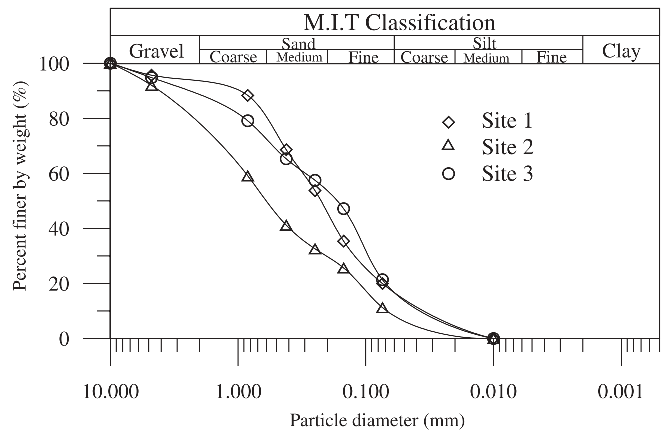
Fig. 4. Outward movement of the gravity wall along construction joint.

Table 2. Properties of backfill materials.

Site No.	Location	Unified classification of backfill	Specific gravity, G_s	Optimum moisture content (%)	Maximum dry unit weight, $\gamma_{d,max}$ (kN/m^3)	Cohesion, c (kN/m^2)	Internal friction angle, ϕ (°)
1	Provincial Highway 14, kilometre 38	SC–SM	2.51	12.2	17.6	0.8	35.7
2	Provincial Highway 21, kilometre 71	SC	2.51	24.9	15.6	1.7	31.0
3	Taiwan Cinema Culture Town	SP	2.62	11.3	18.1	1.1	38.8

gradation curves of the soil samples are illustrated in Fig. 5. In the field, the contractors are generally required to compact the soil to at least 90% of standard maximum dry density. To simulate the performance of the backfill in the field, the soil sample at its optimum moisture content was compacted in the laboratory with the standard Proctor method and then tested with the direct shear apparatus. Seed and Chan (1959) reported that samples compacted dry-of-optimum tend to be more rigid and stronger than samples compacted wet-of-optimum. From a practical point of view, in this study the soils collected from the sites were compacted at the optimum moisture content and then sheared to failure. The strength parameters obtained are listed in Table 2 and were used in the stability analyses of the retaining walls.

Fig. 5. Grain-size distribution of backfill materials at sites 1–3.



Seismic pressure based analysis

Kramer (1996) stated that gravity walls are customarily designed by one of the following two approaches: a seismic pressure based approach, or a permanent displacement based approach. Gravity walls have traditionally been designed on the basis of seismic earth pressure. The Mononobe–Okabe method is commonly used to estimate the dynamic earth pressure due to earthquakes. The total active thrust P_{AE} can be expressed as

$$[1] \qquad P_{AE} = \frac{1}{2}\gamma H^2 (1 - k_v) K_{AE}$$

where the dynamic active earth pressure coefficient K_{AE} is given by

$$[2] \qquad K_{AE} = \frac{\cos^2(\phi - \beta - \theta)}{\cos\theta \cos^2\beta \cos(\delta + \beta + \theta)\left[1 + \sqrt{\dfrac{\sin(\phi + \delta)\sin(\phi - \theta - i)}{\cos(\delta + \beta + \theta)\cos(i - \beta)}}\right]^2}$$

$$[3] \qquad \theta = \tan^{-1}\left(\frac{k_h}{1 - k_v}\right)$$

γ is the unit weight of soil, H is the height of the wall, ϕ is the internal friction angle of soil, δ is the wall friction angle, i is the slope of the ground surface behind the wall, β is the slope of the back of the wall to the vertical, k_h is the horizontal seismic coefficient (= a_h/g, where a_h and g are the horizontal acceleration and gravitational acceleration, respectively), and k_v is the vertical seismic coefficient (= a_v/g, where a_v is the vertical acceleration). Pseudo-static accelerations are generally considered smaller than anticipated peak accelerations. Whitman (1990) reported that values corresponding to one third to one half of the peak ground surface acceleration were commonly used with factors of safety (FSs) of 1.0–1.2. Under the combined effect of static and earthquake load, the NAVFAC DM-7.2 design manual of the U.S. Department of the Navy (1982) recommends that an FS between 1.1 and 1.2 is acceptable. It is reported that gravity walls designed by the traditional approach have generally performed quite well in earthquakes.

Seed and Whitman (1970) suggested that the total active thrust P_{AE} could be resolved into the static active thrust P_A and the dynamic increment ΔP_{AE} components:

[4] $$P_{AE} = P_A + \Delta P_{AE}$$

P_A acts at $H/3$ above the base of the wall, and ΔP_{AE} acts at $0.6H$ above the base of the wall. The magnitude and the point of application of ΔP_{AE} suggested by Seed and Whitman were adopted in this study for estimation of overturning moment of the wall. The static active thrust P_A is calculated using Coulomb theory with the cohesion of soil neglected.

The cross section of the collapsed wall at site 1 is shown in Fig. 3. It is clear in Figs. 3 and 4 that sliding failure occurred along the construction joint A–B. Therefore, stability analysis was carried out only for the upper two blocks of the wall. It is true that, due to the earthquake excitation, sliding failure could occur along any of the construction joints. To limit the scope of this study, however, the FSs were only determined for blocks 4 and 5 of the wall which actually failed. The coefficient of friction at joint A–B was assumed to be 0.5. It is obvious that the actual value of the friction coefficient depends on the roughness of the upper and lower surfaces in contact.

The wall friction angle δ between the backfill and the wall is commonly assumed to be between $\phi/2$ and $2\phi/3$. The study by Seed and Whitman (1970) indicated that, for $\delta = 0$ and $\delta = \phi/2$, the seismic active coefficients K_{AE} determined using the Mononobe–Okabe theory were nearly identical. It was concluded that values of K_{AE} computed for $\delta = \phi/2$ could be considered satisfactory for most conditions. In this study, it was assumed that $\delta = \phi/2$. Assuming the backfill had been compacted to a relative compaction of 90% before the earthquake, the unit weight of the compacted backfill would be 17.8 kN/m^3. For the gravity wall, the unit weight of concrete was assumed to be 23.6 kN/m^3.

To evaluate the stability of a retaining wall, the NAVFAC DM-7.2 design manual (U.S. Department of the Navy 1982) requires checks for (*i*) resistance against overturning, (*ii*) resistance against sliding, (*iii*) allowable pressures on the base, (*iv*) settlement, and (*v*) overall stability of the wall. For the wall damaged at site 1, it is obvious that the resistance against sliding would be the point of emphasis. The factor of safety against sliding ($FS_{sliding}$) can be expressed as

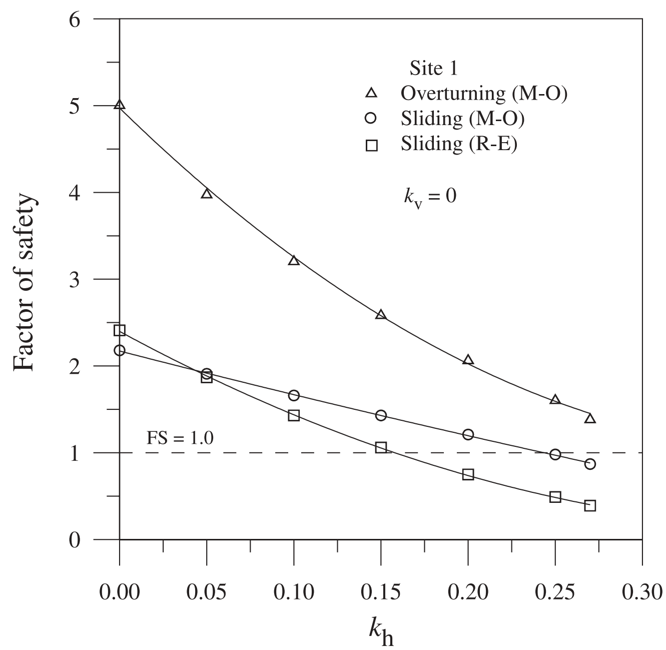
[5] $$FS_{sliding} = \frac{\Sigma F_R}{\Sigma F_D}$$

where ΣF_R is the sum of the horizontal resisting forces, and ΣF_D is the sum of the horizontal driving forces. The factor of safety against overturning ($FS_{overturning}$) about the toe can be expressed as

[6] $$FS_{overturning} = \frac{\Sigma M_R}{\Sigma M_O}$$

where ΣM_R is the sum of the resisting moments about the toe, and ΣM_O is the sum of the overturning moments about the toe. The computation of FS against sliding and overturning under earthquake excitation is introduced in most soil dynamics textbooks, such as those by Das (1993) and Kramer (1996).

Fig. 6. Factor of safety (FS) of wall at site 1 as a function of seismic coefficient.



The variation of FS of the wall (blocks 4 and 5) as a function of the horizontal seismic coefficient k_h is shown in Fig. 6. Since for most earthquakes the horizontal acceleration components are considerably greater than the vertical components, Seed and Whitman (1970) concluded that k_v can be neglected for practical purposes. In this study, the vertical ground acceleration was neglected. Under the static condition ($k_h = k_v = 0$), the FS against sliding obtained using the Mononobe–Okabe method was found to be 2.18. Das (1999) suggested that, under a static condition, the FS against sliding, overturning, and bearing capacity failure should be at least 1.5, 2.0, and 3.0, respectively. It was obvious that the gravity wall was safe from sliding without the earthquake excitation.

Figure 6 shows that the FS against sliding decreased with increasing horizontal ground acceleration. As the horizontal acceleration exceeds $0.24g$, the FS against sliding would be less than 1.0. This means the frictional resistance at the construction joint A–B would be insufficient to resist the dynamic soil thrust. In Table 1, the peak horizontal accelerations in east–west and north–south directions measured at station TCU-072 during the Chi-Chi earthquake were $0.38g$ and $0.47g$, respectively. These values are apparently greater than the yield acceleration of $0.24g$. It is possible that, as mentioned in the second section of this paper, the wall moved a small amount due to each effective cycle of ground shaking. After several effective cycles, the wall gradually moved away from the backfill and eventually fell into the side ditch. In Fig. 6, sliding failure is shown to be more likely to occur than overturning, since the FS against sliding is significantly lower than the FS against overturning for the same k_h value.

Permanent displacement-based analysis

This approach allows the designer to take into consideration the wall inertia effect and leads to the conclusion that there might be some lateral movement of a wall even for mild earthquakes. Design procedures based on Richards and Elms (1979) and Whitman and Liao (1985) procedures for estimation of permanent displacement are available. In the Richards and Elms analysis, the weight of the wall W_w required to prevent wall motion is expressed as

$$[7] \qquad W_w = \left[\frac{1}{2}\gamma H^2(1-k_v)\,K_{AE}\right]C_{IE}$$

The coefficient K_{AE} can be determined using eq. [2], and the wall inertia factor C_{IE} is given by

$$[8] \qquad C_{IE} = \frac{\cos(\beta+\delta) - \sin(\beta+\delta)\tan\phi_b}{(1-k_v)(\tan\phi_b - \tan\theta)}$$

where $\tan\phi_b$ is the friction coefficient at the base of the wall. The FS to take into account the effects of dynamic soil pressure and wall inertia is defined as

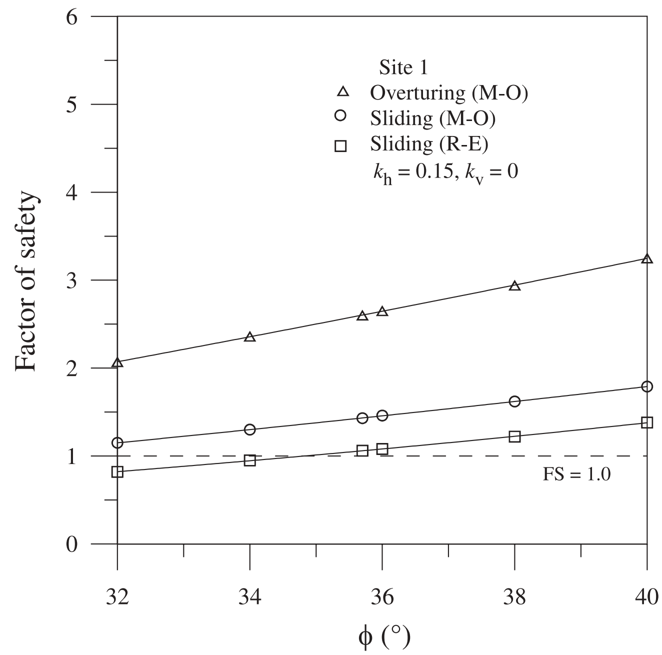
$$[9] \qquad \mathrm{FS} = \frac{W}{W_w}$$

where W is the weight of the wall for equilibrium in the static condition. For this method, an FS of 1.5 is generally required. The variation of FS against sliding based on the method proposed by Richards and Elms (1979) is also shown in Fig. 6. It is clear that as the horizontal ground acceleration exceeds 0.15g, the FS would be less than 1.0, which means that the frictional resistance at the construction joint will not be able to resist the horizontal soil thrust and wall inertia force. When the ground shaking is equal to or greater than the yield acceleration, permanent wall displacement will occur.

Figure 7 shows how the variation of soil property affects the results of analysis. Based on the laboratory test results, the internal friction of the backfill is assumed to be 35.7°. Under the horizontal shaking of 0.15g, the calculated FS against sliding (R–E method) is 1.06. If the ϕ angle of soil is reduced to 34.0°, however, the FS against sliding (R–E method) would drop to only 0.95. The analytical results are relatively sensitive to the variation of soil strength. It is possible that locally poor compaction or the presence of weaker soil triggered the 20 m long wall failure at site 1, not failure for the whole length of wall.

The Foundation Design Code of Taiwan adopted the Mononobe–Okabe method to determine the dynamic earth pressure under seismic loading. Central Taiwan, the most severely damaged area, was included in zone 2 of the seismic zoning chart of Taiwan. The design horizontal acceleration recommended for zone 2 is only 0.23g. On 21 September, however, the horizontal PGA measured at stations TCU-072 and TCU-065 was 0.47g and 0.79g, respectively. Under such a strong vibration, the retaining walls designed with a horizontal acceleration of 0.23g probably could not remain stable. About 2 months after the earthquake, the damaged wall at site 1 had been repaired. Shearing reinforcements were fabricated at the construction joint as indicated in Fig. 8. During construction, steel H-piles were driven to resist the potential movement of the steep slope. The installation of a shearing key would be another efficient method to prevent sliding failure at the construction joint.

Fig. 7. Variation of FS as a function of soil strength.



Site 2: bearing capacity failure under the toe

The damaged retaining wall was built near Sun-Moon Lake and is labeled as site 2 in Fig. 1. The gravity wall was built to retain the steep slope for the construction of Provincial Highway 21. After the Chi-Chi earthquake, the retaining wall settled significantly and tilted about its toe as shown in Fig. 9. Little lateral movement of the wall was observed. The active soil wedge behind the wall collapsed, however a large portion of the wall body remained undamaged. The beetle-nut plantation on the uphill slope remained stable during the earthquake.

The wall was 2.5 m high, 1.25 m thick at the base, and 0.5 m thick at the top, as shown in Fig. 10. Weep holes were fabricated in the wall 0.25 m apart horizontally. The damaged zone was about 40 m long. Figure 1 shows that site 2 is quite close to the epicenter of the Chi-Chi earthquake. It is possible that the unexpected strong ground motion is the main cause of the damage. Adequacy of the gravity wall under seismic excitation is discussed in the following section.

Seismic adequacy

The variation of FS for the gravity wall at site 2 as a function of the horizontal seismic coefficient k_h is shown in Fig. 11. It was assumed that the foundation soils had been compacted and had the same properties as the backfill. The resisting force against sliding at the base of the wall can be expressed as

$$[10] \qquad \sum F_R = \sum V \tan\left(\frac{2\phi}{3}\right) + B\left(\frac{2c}{3}\right)$$

Fig. 8. Shearing reinforcement at construction joint.

Fig. 9. Overturning of gravity wall at site 2.

where ΣV is the sum of the vertical forces, B is the width at the base of the wall, and c is the cohesion.

Under a static condition, the FS against sliding is only 1.15. This value is unacceptable for design, since it is less than the required FS of 1.5. In Fig. 11, the FS against sliding decreases with an increase in the horizontal acceleration. With the Mononobe–Okabe method, the FS against sliding would be less than 1.0 for a horizontal acceleration greater than 0.05g. With the Richards–Elms method, the wall would start to slide as the horizontal acceleration exceeds 0.025g. It is clear in Fig. 10 that the wall is relatively short and thick and should exhibit good resistance against overturning. In Fig. 11, at any k_h value, the FS against overturning is apparently higher than that against sliding. This analysis indicates that the wall should slide instead of overturn under seismic shaking. This finding is contrary to the wall behavior observed in the field.

Tatsuoka et al. (1998) reported that, under a seismic load, gravity and semigravity walls could display bearing capacity failure under the toe. As indicated in Fig. 12, the incremental seismic forces ΔP_{AE} and $k_h W$ would increase the contact pressure at the toe, q_{max}. If q_{max} exceeds the ultimate bearing

Fig. 10. Profile of damaged wall at site 2.

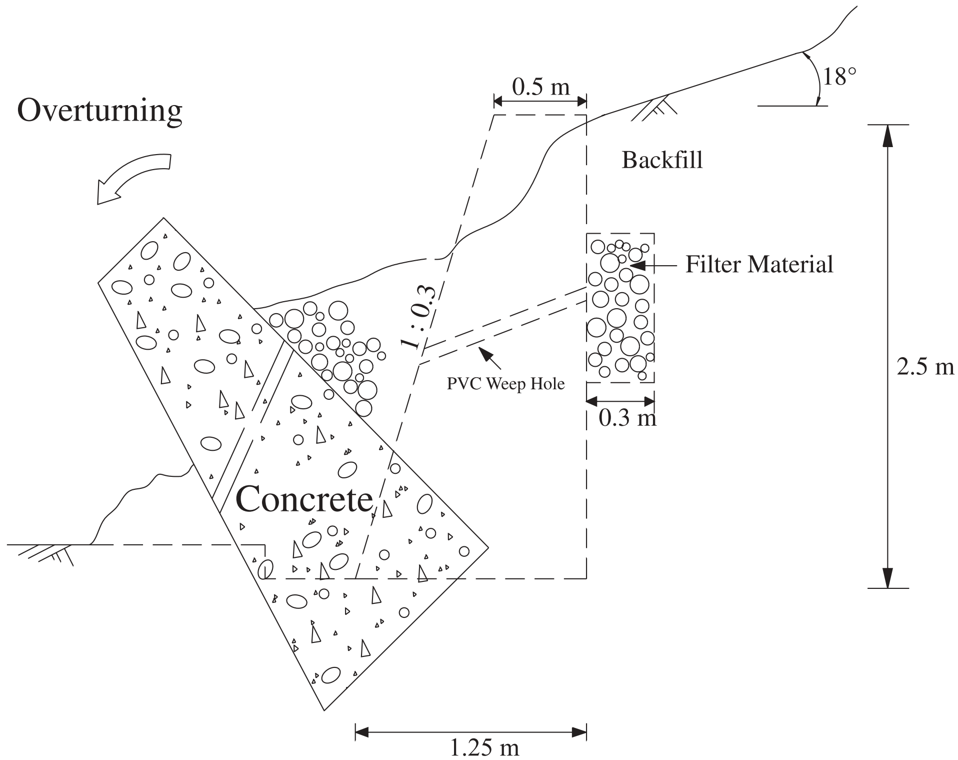


Fig. 11. Factor of safety of wall at site 2 as a function of seismic coefficient.

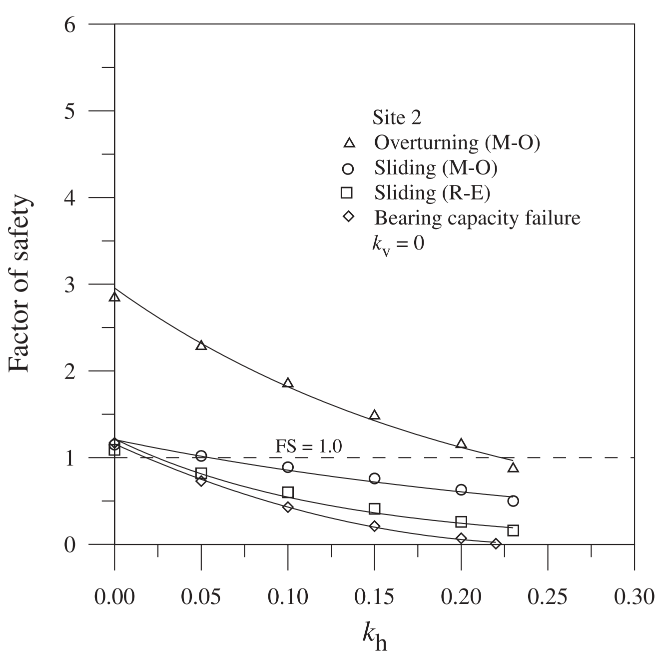
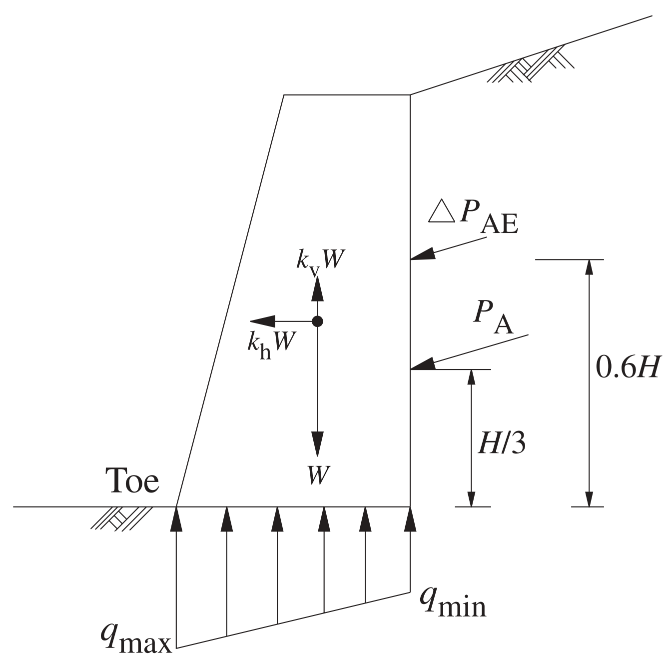


Fig. 12. Assumed contact pressure at the base of the wall.



capacity, q_{ult}, of the underlying soils, settlement would occur at the toe, causing the retaining wall to tilt. The FS against bearing capacity failure is defined as

$$[11] \qquad FS = \frac{q_{ult}}{q_{max}}$$

The ultimate bearing capacity of the foundation soil was determined with the following equation proposed by Meyerhof (1963):

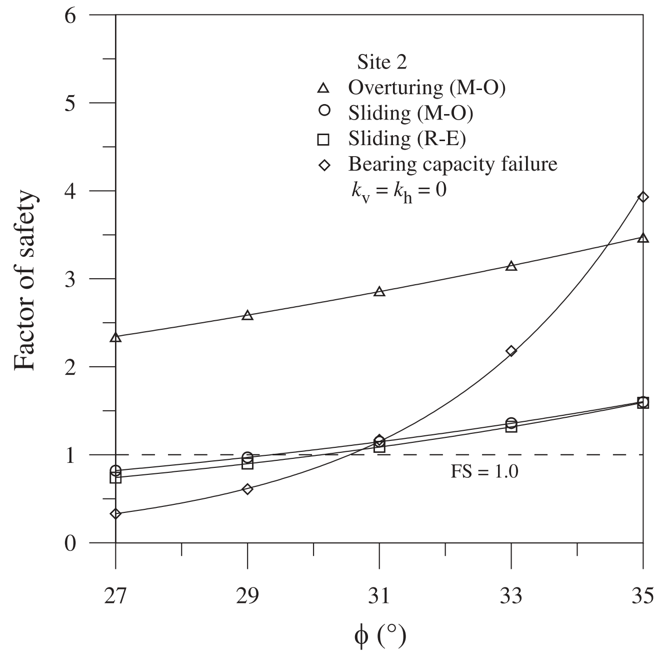
$$[12] \qquad q_{ult} = cN_c F_{cs} F_{cd} F_{ci} + qN_q F_{qs} F_{qd} F_{qi} + 0.5\gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i}$$

where N_c, N_q, and N_γ are bearing capacity factors; F_{cs}, F_{qs}, and $F_{\gamma s}$ are shape factors; F_{cd}, F_{qd}, and $F_{\gamma d}$ are depth factors; and F_{ci}, F_{qi}, and $F_{\gamma i}$ are load inclination factors. It should be noted that the resultant force acting at the base of the wall is not vertical, and the load inclination factors would significantly influence the estimated q_{ult} value. Unfortunately, due to limited funding, the strength parameters c_2 and ϕ_2 of the foundation soil were not available for this case. It was assumed that the properties of the soils under the wall were similar to those of the compacted backfill. In the analysis, the parameters γ_2 = 17.5 kN/m^3, ϕ_2 = 31°, and c_2 = 1.7 kN/m^2 were adopted for the foundation soil.

The variation of FS against bearing capacity failure is also indicated in Fig. 11. Under the same seismic coefficient, the FSs obtained are slightly lower than the FS against sliding. During the Chi-Chi earthquake, the horizontal PGA recorded at the nearby station TCU-079 was 0.59*g*. Under such a strong seismic vibration, the overturning wall movement was most probably triggered by insufficient bearing capacity under the toe. Based on this case study, it could be concluded that a stability check against bearing capacity failure for the gravity wall should never be neglected.

It should be noted in Fig. 11 that, even under static loading, the FSs against sliding and bearing failure are very low. Figure 13 shows how the variation of soil friction angle affects the analytical results prior to the earthquake. Based on the laboratory tests, the angle ϕ of the compacted backfill is assumed to be 31.0°. In Fig. 13, if ϕ is reduced to 29°, bearing failure would be more likely. However, the FS against bearing failure increases rapidly with an increase in ϕ. If ϕ is increased to 33°, the FS against bearing failure would be 2.18, and sliding becomes the possible mode of failure.

Fig. 13. Variation of FS as a function of internal friction angle, ϕ.



Site 3: retaining wall damaged by fault rupture

Figure 14 shows the failure of a gravity wall located at the Taiwan Cinema Culture Town (site 3) near the township of Wu-Fung. The wall was constructed right above the Che-Lung-Pu fault. During the earthquake, the fault rupture caused the wall to slide and overturn. Figure 14 shows that the power pole tilted along with the retaining wall. The body of the wall remained unbroken, however. Ground heave in front of the toe is quite obvious in Fig. 14. The backfill behind the wall cracked, and the buildings constructed on the backfill were severely damaged by the ground deformation and strong ground shaking.

The failure mechanism of the damaged wall is illustrated in Fig. 15. The gravity wall is 4.0 m high and 0.4 m thick at the top. The retaining wall was built on top of the Che-Lung-Pu fault. Figure 16 is an oblique aerial photograph showing the fault rupture cutting through the Taiwan Cinema Culture Town. Lee et al. (2000) investigated the surface fault rupture due to the Chi-Chi earthquake. It was reported that the vertical and horizontal displacements measured at Wu-Fung were 2.0 and 1.3 m, respectively. The maximum vertical fault rupture reported for the Chi-Chi earthquake was as much as 7.0 m. It is clear in Fig. 16 that the buildings constructed along the fault were severely damaged by the ground deformation and strong ground shaking. The vertical and horizontal PGA measured at station TCU-065 near site 3 were 0.26*g* and 0.79*g*, respectively. Figure 15 shows that during the earthquake the vertical displacement of the hanging wall uplifted the backfill, causing the wall to overturn. Horizontal displacement of the hanging wall pushed the wall, causing it to slide, and the soil in front of the toe, causing it to heave. It may be concluded that it is of critical importance for the designer to locate the active faults near the site and not to construct any structure on top or even near active fault zones.

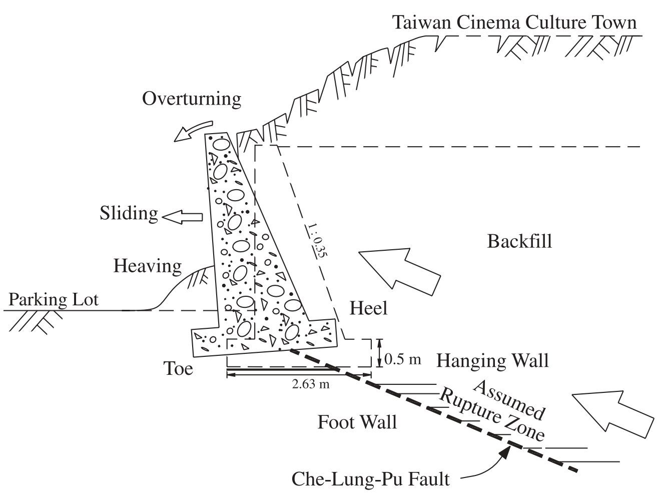
Figure 17 shows the variation of FS against sliding and overturning for the wall as a function of the horizontal seismic coefficient. It should be mentioned that the change of FS in Fig. 17 is solely due to the effect of ground shaking. The procedure of analysis is the same as that for the walls at sites 1 and 2. Under a static condition, the FSs against sliding and overturning are greater than the required values of 1.5 and 2.0, respectively. The wall should be safe from sliding and overturning without earthquake excitation. Based on the Richards–Elms method, as the horizontal acceleration exceeds 0.17*g*, the FS against sliding would be less than 1.0. In Fig. 17, the FS against overturning is apparently greater than the FS against sliding under the same seismic coefficient k_h. The wall should be safe from overturning up to a horizontal acceleration of 0.50*g*. It may be concluded that failure of the retaining wall was mainly caused by excessive fault displacement.

Conclusions

This paper has investigated the failure of retaining structures due to the 1999 Chi-Chi earthquake. Stability analysis was performed with both the Mononobe–Okabe and the

Fig. 14. Sliding and overturning of retaining wall at site 3.

Fig. 15. Schematic diagram of the wall damaged by fault rupture.



Richards–Elms methods. Based on the case study for three gravity walls, the following concluding remarks are made.

For the first case, the sliding of concrete wall blocks along the construction joints was observed. With the Mononobe–Okabe method, as the horizontal acceleration exceeded $0.24g$, the FS against sliding at the construction joint would be less than 1.0. With the Richards–Elms method, the minimum horizontal acceleration to initiate any sliding is $0.15g$. When the ground shaking is equal to or greater than the yield acceleration, permanent wall displacement will occur. During the Chi-Chi earthquake, the horizontal PGA measured in the east–west and north–south directions was $0.38g$ and $0.47g$, respectively. It would be reasonable to infer that the wall moved a small amount due to each effective cycle of ground shaking. After several effective cycles, the wall gradually moved away from the backfill and eventually fell into the side ditch. The installation of shearing keys or shearing reinforcements would be effective methods to prevent sliding at the construction joint.

For the second case, the retaining wall settled significantly and tilted about its toe. Tatsuoka et al. (1998) reported that, under a seismic load, gravity and semigravity walls could display bearing capacity failure under the toe. If the contact pressure under the toe, q_{max}, exceeds the ultimate bearing capacity, q_{ult}, of the underlying soils, settlement would occur at the toe, causing the retaining wall to tilt. Seismic analysis of the wall indicates that, under the same horizontal acceleration, the FS against bearing capacity failure was lower than the FS against overturning and sliding. The overturning wall failure was most probably triggered by the insufficient bearing capacity beneath the toe. It is concluded that a stability check against bearing capacity failure for the gravity wall should never be neglected.

For the third case, the retaining wall built on top of the Che-Lung-Pu fault was severely damaged by excessive fault displacement. During the earthquake, vertical displacement of the hanging wall uplifted the backfill, causing the wall to overturn. Horizontal displacement of the hanging wall caused the wall to slide and the soil in front of the toe to heave. The failure of the retaining wall was mainly due to the excessive fault rupture. It is of critical importance for the engineer not to construct any structure on top of or near an active fault.

Fig. 16. Che-Lung-Pu fault cut through Taiwan Cinema Culture Town (after Sino-Geotechnics Research and Development Foundation 1999).

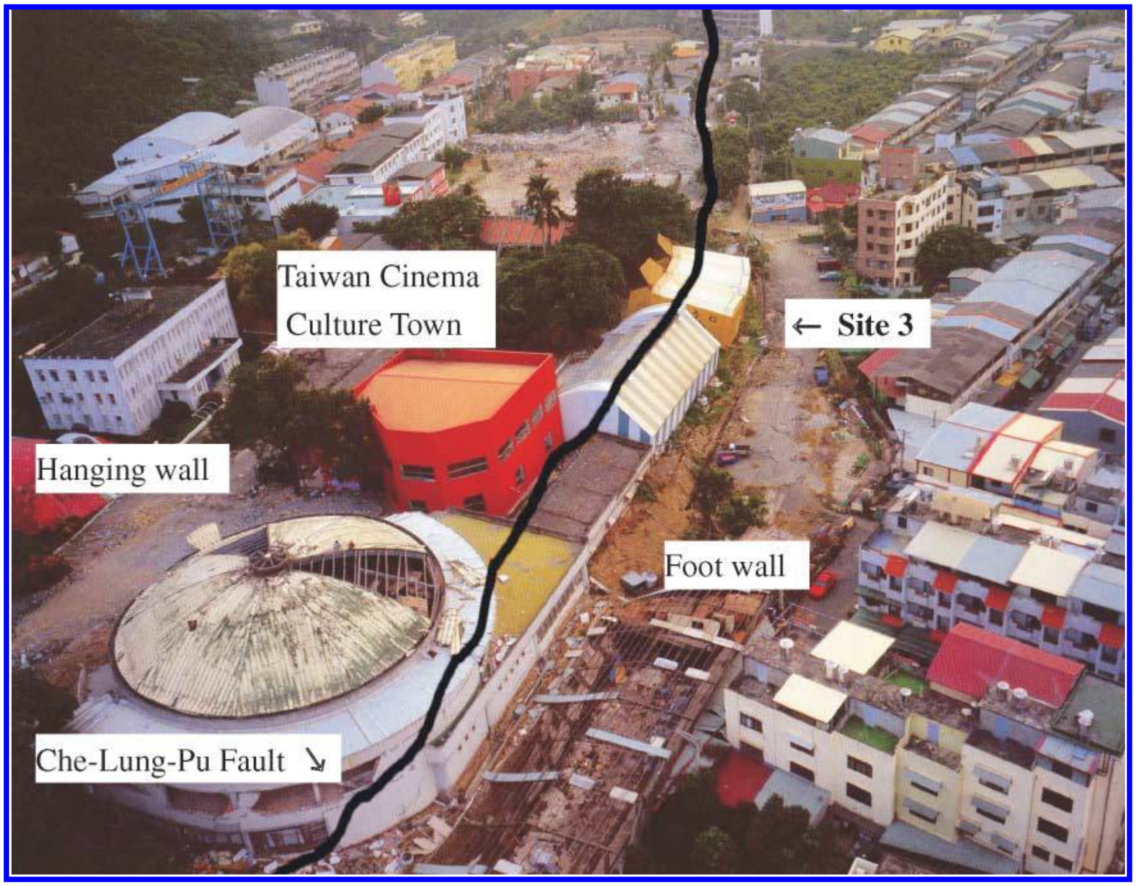
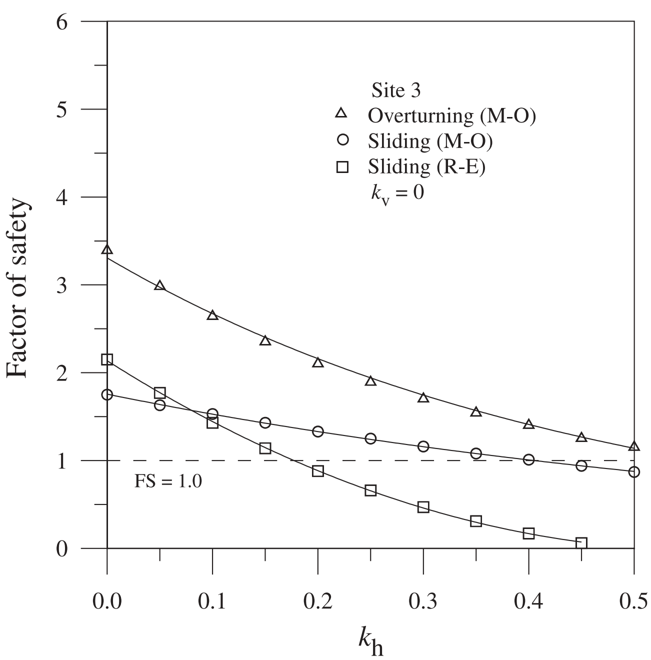


Fig. 17. Factor of safety of wall at site 3 as a function of seismic coefficient.



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