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Analyses of a near-fault geosynthetic-reinforced modular block wall damaged during the 1999 Chi-Chi earthquake

Ching-Chuan Huang
Professor, Department of Civil Engineering, Cheng Kung University, Taiwan

Li-Hwei Chou & Yu-Hung Chen
Research students, ditto.

ABSTRACT: A near-fault geosynthetic-reinforced modular block wall (RMBW) damaged during the 1999 Chi-Chi earthquake (M_L=7.3) is analyzed using a new 'three-wedge' method. The new method calculates the stability of the facing and soil wedges interactively. As a result, a safety margin against shear failure on the potential failure surface is the only output. Therefore, irrelevant safety criteria can be eliminated. The new method gives not only an accurate estimation on the contribution of block facing to the seismic stability of RMBW's but also accurate failure patterns under seismic conditions. A new method for calculating seismic displacement of RMBW's based on the 'three-wedge' mechanism and the 'displacement diagram' is developed. It is demonstrated that this method gives more realistic displacements of RMBW's than any other existing pseudo-static methods.

1 INTRODUCTION

The Chi-Chi earthquake (M_L=7.3) occurred at 1:47AM, 1999 in central Taiwan. Rupture of the Chelungpu fault (about 100km-long and 30km-wide) was reported to be the cause of this earthquake. Intensive damage to the near-fault highway facilities, such as: bridges, embankments, and soil retaining structures were reported (Huang, 2000a). In the severely shaken area, no failure of geosynthetic-reinforced soil retaining walls was observed except an inadequately designed and constructed wrap-around reinforced slope and two dry stacked modular block reinforced walls. The present study investigated one of the failure sites of the modular block reinforced wall located about 4km east from the surface scarp of the ruptured fault (Site 1 in Figure 1).

Geosynthetic-reinforced modular block walls have been increasingly used in Taiwan. However, no authorized design guideline for modular block reinforced walls is available. The investigated RMBW is currently used in a highway-widening project for a secondary earth retaining purpose. At Site 1, a 3.2m-high RMBW collapsed, see Figure 2. Two typical sections of the collapsed walls are shown in Figures 3(a) and 3(b). See Huang (2000a, 2000b) for other RMBW sites.

Post-failure investigations at site 1 showed that:

(1) At the section where the wall was on the verge of collapse (Figure 2, Figure 3b), the wall buckled and /or bulged between the lower 1/3 and 1/2 of the wall height and caused large openings between the blocks. These openings in turn disabled the function of the FRP rods (15mm diameter and 200mm long) which were used as shear keys between the stacked blocks.

(2) Failures at the junction of the geogrids were found behind the collapsed facing. The locations of the junction failures coincided with those for

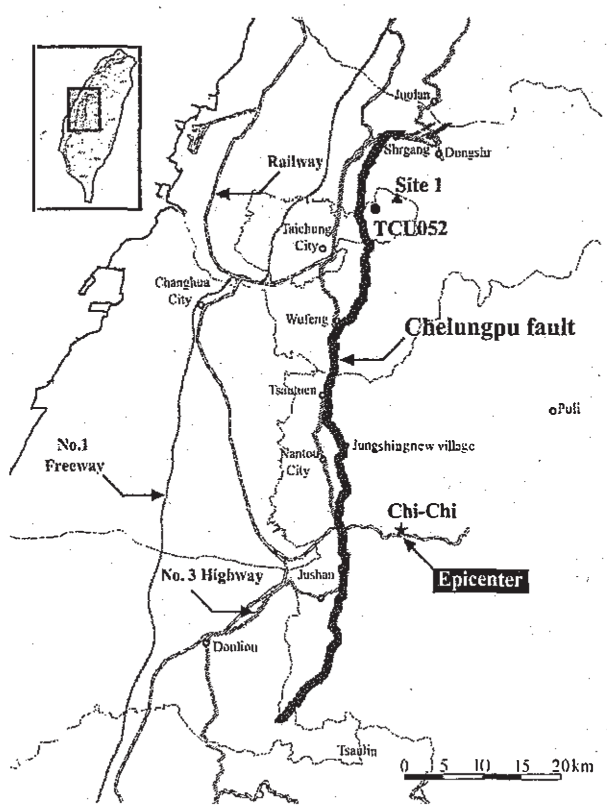

Figure 1. Location of the investigated site.

Figure 2. The geosynthetic-reinforced modular block wall damaged during the 1995 Chi-Chi earthquake.

the (FRP) rods used to connect the blocks. Ultimate shear strength of FRP rods is 48000 kPa according to ASTM D4475. Most of the rods found at the site were intact. The knitted polyester net, or PET geogrid, had ultimate tensile strength of 75kN/m and junction strength of 0.3kN/junction.

(3) In-lab direct shear tests were performed on the soil samples remolded under the in-situ density and natural water content. Strength parameters obtained from 60mm-diameter, 35mm-thick samples are c=0, φ =30.4° , those from 200mm-diameter, 100mm-thick samples are c=0, φ =29.2° .

A preliminary investigation on the failure mechanism of this wall was performed using the seismic design guideline proposed by NCMA (1998). Safety factors against various failure modes obtained from the analysis are summarized in Table 1. In this calculation, the experimental value of connecting force at block-backfill interface, T_{w_i}, and a possible range of the block-block friction angle, μ, were used (see Table 3). Table 1 indicates that:

(1) For k_h=0 , namely, static condition, F_s for toppling and connections are 0.62 and 0.65-0.94, respectively. However, none of these failure modes were observed in the field. Over-estimation on the lateral earth pressure may account for this disparity.

(2) For k_h=0.22 condition ($\fallingdotseq 0.5 \cdot a_{max}/g$, see Figure 11a), in addition to the under-estimated F_s for toppling and connections of reinforcement, as discussed in (1), a critical value of F_s=0.99 was obtained for pull-out failure. However, pull-out of the reinforcement has not been observed (see Figures 3a and 3b).

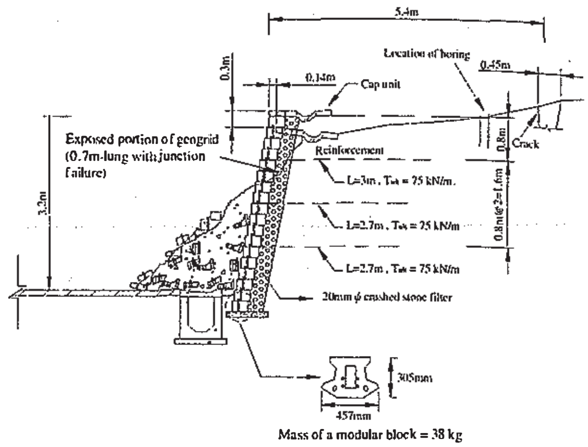

Figure 3(a). A cross section of the geosynthetic-reinforced modular block wall damaged during the 1999 Chi-Chi earthquake.

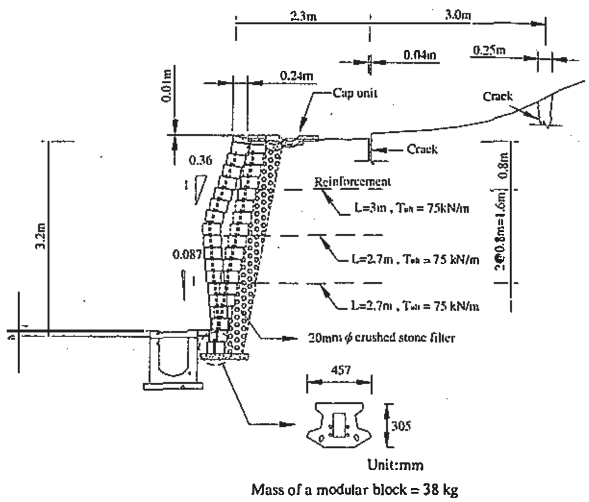

Figure 3(b). A cross section of RMBW on the verge of collapse in the Chi-Chi earthquake.

(3) For k_h=0.22 condition, F_s for interface shear and internal sliding are 11.61-15.29 and 2.48-3.24, respectively. These values are the highest ones, among all values of F_s. Unfortunately, the failure shown in Figure 2 and Figure 3(b) inferred that internal sliding and interface shear failures may occur to some extent. The over-estimated F_s against these failures may be due to the inappropriate failure mechanisms used. In the design guideline, horizontal failure surfaces along the re-

Table 1. Safety factors calculated using the NCMA design guidelines.

	External stability analyses		Internal stability analyses			Facing stability analyses		
k_h	Base sliding	Overturning	Over-stress of reinf.	Pull-out	Internal sliding	Interface shear	Toppling	Commections
0.00	3.55	6.93	7.83	1.85	7.59-9.90	19.39-25.53	0.62	0.65-0.94*
0.22	1.30	1.81	5.00	0.99	2.48-3.24*	11.61-15.29*	0.06	0.43-0.61*

* Ranges shown in this table indicate the effect of block-block friction angle, μ =30°-45°.

inforcement layer were used to evaluate the seismic stability of RMBW's.

In addition, buckling and/or bulging at the lower 1/3-1/2 of the modular block wall cannot be analyzed using the existing methods (e.g., Cai and Bathurst, 1996, Ling and Leshchinsky, 1998) in which only horizontal displacement along the base of the reinforced zone is taken into account. An inconsistency between the seismic design method and displacement calculation methods is that the displacement calculations are performed using the base sliding failure mode which may not be the dominant failure mode (see F_s for base sliding in Table 1). It is also noted that the 'three-wedge' mechanism as shown in Figures 4 and 7 is more appropriate for describing the observed failure patterns. Consequently, the present study focuses on the following:

(1) To develop a straightforward 'three-wedge' method for pseudo-static analysis of the RMBW, in which, only a safety factor against shear failure along the critical surface is the final result. This may eliminate the use of too many, sometimes, irrelevant safety criteria, as used in the current design guidelines.

(2) To perform a new deformation analysis, based on the 'three-wedge' mechanism, to describe the failure pattern observed in the 1999 Chi-Chi earthquake, and also to avoid the inconsistency discussed above.

2 PSEUDO-STATIC ANALYSIS

A schematic figure of the three-wedge method used in the present study is shown in Figure 4. The so-called "two-wedge" or "bi-linear failure line" method is a simplified case of the 'three-wedge' model shown in Figure 4. When the stability of facing is considered in the analysis, the connecting reinforcement force (T_{wi}) at the facing-backfill interface, and the block-block shear resistance may influence the result of the analysis. Therefore, four types of facing and connecting conditions were used in the present study. They are summarized in Table 2 and Figure 5. In which,

Type 1: Stability of facing is not considered. This type of analysis is equivalent to the conventional 'two-wedge' analysis. The mobilized tensile force at the failure surface, T_i, is equal to the smaller one of T_{Pb} and T_{Pf}, (T_{Pb}: the pull-out resistance for the back of the potential slip surface; T_{Pf}: the pull-out resistance for the front of potential slip surface)

Type 2: Stability of facing is calculated interactively with the stability of the two soil wedges. T_i = min. of $\{T_{Pb}, T_{Pf}+T_c, T_{tensille}\}$, and T_{wi} = min. of $\{T_c, T_{Pf}+T_{Pb}, T_{tensile}\}$, $T_{tensile}$: tensile strength of geogrid, T_c: junction strength of geogrid at FRP rod-geogrid connection.

Type 3: Stability of facing is calculated interactively with the stability of the two soil wedges. T_i = min. of $\{T_{Pb}, T_{Pf}+T_b+T_c, T_{tensille}\}$, and T_{wi} = min. of $\{T_b+T_c, T_{Pf}+T_{Pb}, T_{tensile}\}$.

Type 4: Stability of facing is calculated interactively with the stability of the two soil wedges. In addition to similar calculations of T_i and T_{wi} as used for type 3, shear resistance of the FRP rods was added to the block-block shear resistance.

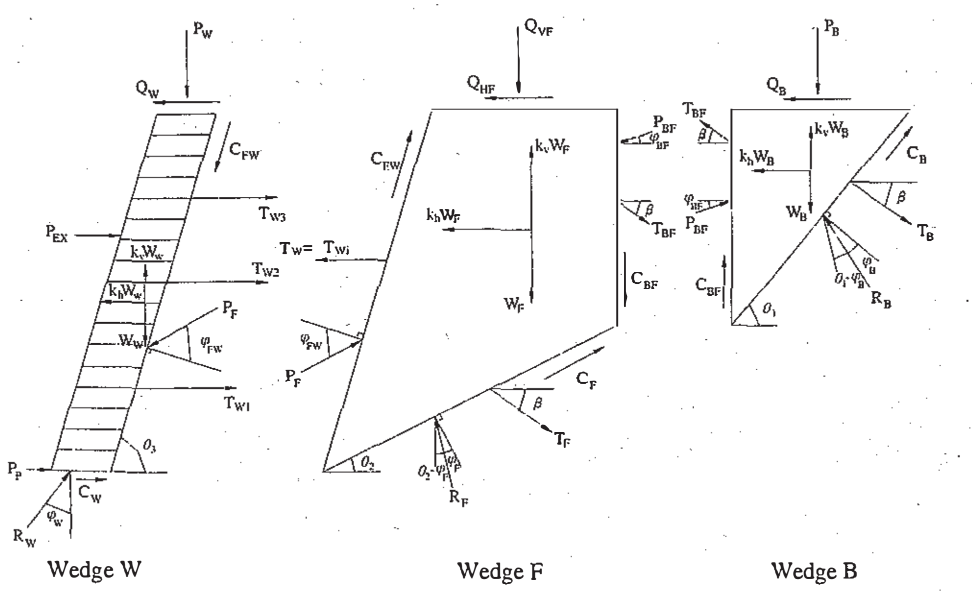

Figure 4. Force equilibrium system in the 'three-wedge' method.

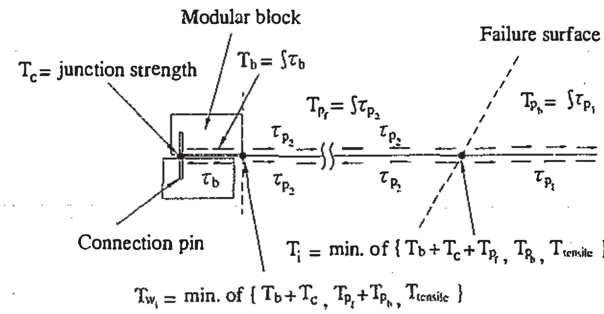

Figure 5. Schematic figure of the pull-out and connecting strength of the geogrid.

Table 2. Facing and connection types considered in the pseudo-static analysis.

	Type 1	Type 2	Type 3	Type 4
Facing stability	No	Yes	Yes	Yes
Geogrid juction strength	No	Yes	Yes	Yes
Block-Block friction	No	No	Yes	Yes
Shear strength of FRP rods*	No	No	No	Yes

* The shear strength of FRP rods was converted to an equivalent cohesion of 48.3 kN/m at block-block interface in the stability analysis.

Table 3. Upper and lower bound values of block-block interface strength parameters used in the present study.

	Cohesion, c (kPa)	Friction angle, μ (°)
Upper bound	0	45
Lower bound	0	30

For types 3 and 4, lower bound and upper bound values of the block-block interface friction angle, μ, were used (Table 3). In the present analysis, λ =0.2 was used ($\lambda = k_h/k_v$, k_h, k_v: horizontal and vertical ground acceleration, respectively) because the main pulse composed from the N-S and U-D ground accelerations (seismograph TCU052) showed that $\lambda \doteqdot 0.2$. The F_s vs. k_h relationships using four types of facing and connection are shown in Figure 6. The following points can be seen:

(1) The 'two-wedge' method rendered a largely under-estimated value of k_{hcr} (k_{hcr} is the value of k_h when F_s=1.0). This infers that facing elements perform a positive role to the seismic stability of RMBW.

(2) The values of k_{hcr} increase with increasing facing-reinforcement connecting force, T_{w_i}.

(3) The values of k_{hcr} increase with the increase of shear resistance at the block-block interface.

(4) The post-failure curves for types 2, 3 and 4 all joined the post-failure curve for type 1 (two- wedge) because beyond the critical failure condition (F_s=1.0), the facing loses its soil-retaining function.

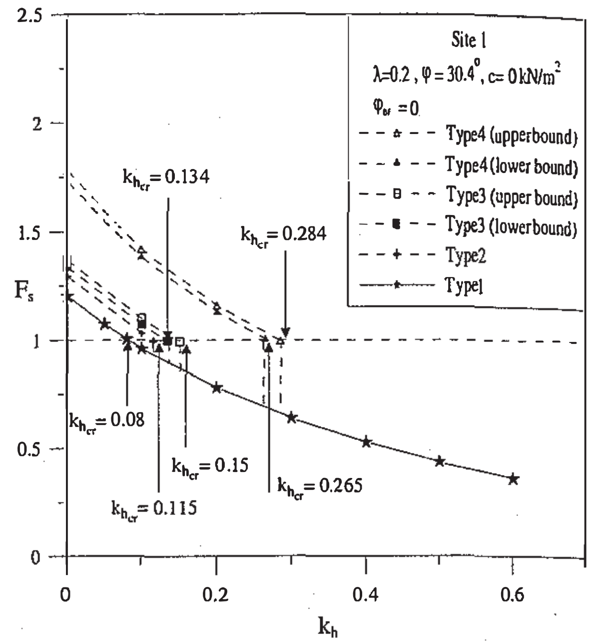

Figure 6. Results of analysis using various types of facing and connection.

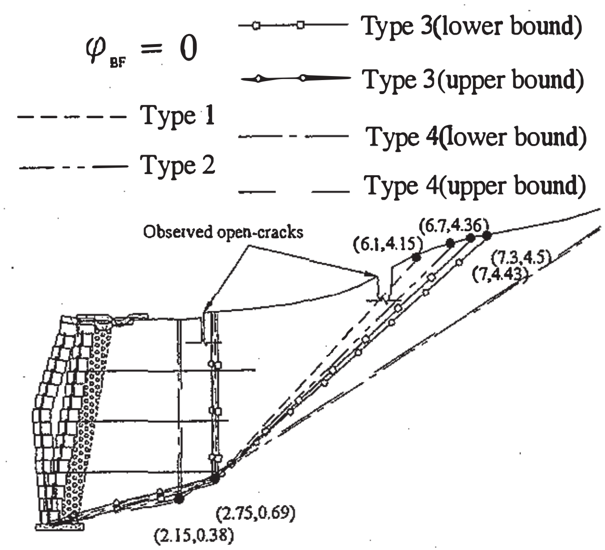

Figure 7. Failure lines calculated using the three-wedge method.

The value of $\varphi_{BF} = 0$ (φ_{BF} : friction angle at the interface of blocks B and F) is used throughout the present study because of a mainly-open crack close to the back of the reinforced zone, see Figure 7. This figure also shows the failure patterns that were calculated using various types of facing. The failure lines obtained from types 2 and 3 come close to the failure pattern observed in the field. The predicted failure lines, using facing type 4, deviated considerably from the observed ones. For the following deformation analysis of Geosynthetic-reinforced modular block walls, only facing type 3 is used.

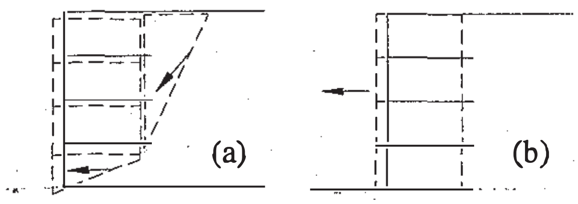

Figure 8(a). Displacement mode considered in the present study.
Figure 8(b). Displacement mode considered in the existing methods.

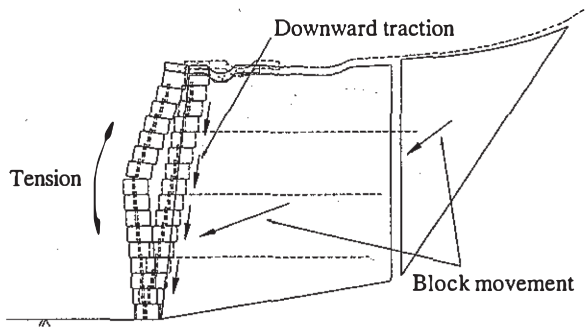

Figure 9. A possible mechanism for seismic displacement of facing and soil wedges.

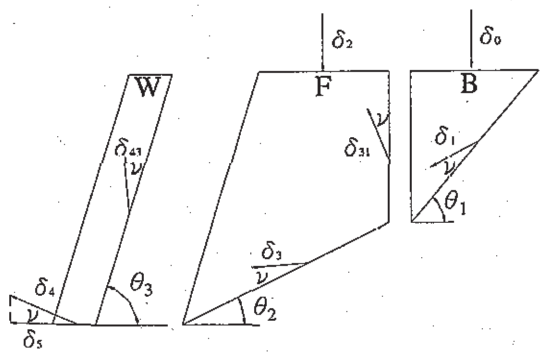

Figure 10(a). Schematic figure for the seismic displacement of geosynthetic-reinforced modular block walls.

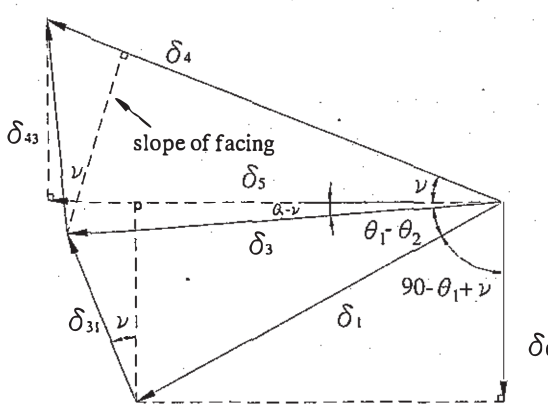

Figure 10(b). Displacement diagram for geosynthetic - reinforced modular block walls.

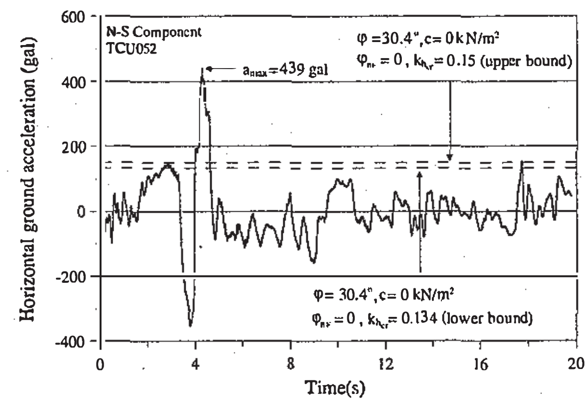

Figure 11(a). Ground acceleration record and constant values of k_{hcr} used in the displacement calculation.

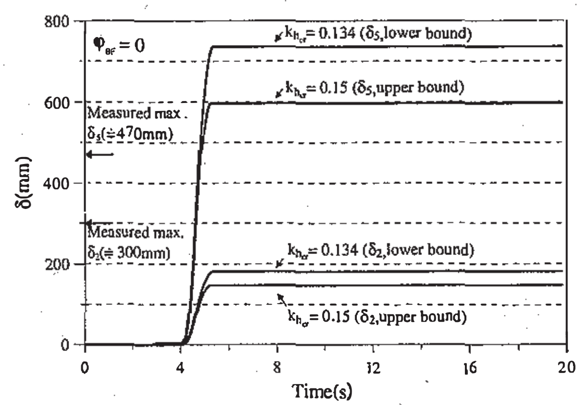

Figure 11(b). Calculated displacement of facing and backfill soil using the three-wedge failure mechanism.

3 DISPLACEMENT ANALYSIS

Figures 8 (a) and 8(b) compares the different failure mechanisms used in the three-wedge and the existing methods (e.g. Cai and Bathurst, 1996, Ling and Leshchinsky, 1998) for calculating seismic displacement of RMBW. The new method calculates not only the horizontal displacement but also the vertical displacement of the reinforced zone. Figure 9 schematically shows that the buckling and/or bulging of the stacked block facing may be primarily due to the traction force induced by the settlement of the soil behind the facing. Therefore, the calculation of vertical settlement of soil wedge behind the wall may facilitate the seismic design of RMBW's. Figures 10(a) and 10(b) show the failure mechanism and associated displacement diagram used in the displacement analysis for the failed Geosynthetic-reinforced modular block wall at site 1. The displacement analysis is based on the 'sliding block' concept, proposed by Newmark (1965) and the 'allowable displacement field' which is used in the limit analysis (e.g., Atkinson, 1981). The new method proposed herewith calculates not only the horizontal displacement but also the vertical displacement for all components of RMBW's. Figure 11(a) shows a major portion of the ground acceleration and the calculated values of k_{hcr} ,based on facing type 3. Figure 11(b) shows the calculated horizontal displacements for the facing (δ_5) and the vertical displacements of soil wedge 'F' (δ_2). In the present study, the angle of dilatancy $\nu=0^\circ$ is assumed. The calculated horizontal displacement (δ_5) for the facing was 300-700mm. The calculated vertical displacement (δ_2) for block 'F' were 50-200mm. These calculated results are comparable with the measured ones (see Figure 11b). Larger measured value of δ_2 than the calculated ones may be due to the compression of soil wedge during the earthquake. It is noted, however, the present calculation was based on a constant k_{hcr} using peak value of φ. Further displacement calculations using other possible post-failure values of k_{hcr} , e.g., k_{hcr} based on residual value of φ, or k_{hcr} based on two-wedge mechanism, as shown in Figure 12, should be performed in the near future.

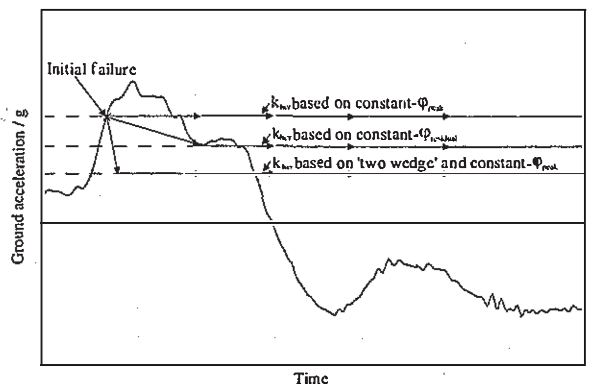

Figure 12. Possible variations of k_{hcr} in the post-failure deformation calculation.

4 CONCLUSIONS

Stability and displacement analyses were performed on an RMBW damaged during the 1999 Chi-Chi earthquake (M_L=7.3). Analysis of the damaged RMBW, following the seismic design guideline proposed by NCMA (1998), indicated that the failure mode resulted from the use of the design guideline deviated largely from the observed one. A new 'three-wedge' analysis method is developed. A major advantage of this method is to analyze the stability of the facing structure and the backfill soil wedges in an interactive way, resulting in a safety factor relating to the shear strength along the potential failure surface. This can eliminate safety criteria that may be irrelevant to the seismic stability of RMBW. For site 1, the new method results in safety evaluations and failure mechanisms comparable with those observed in the post-earthquake site investigations. A new method for displacement calculation of RMBW based on the 'three-wedge' mechanism and the 'displacement diagram' is developed. The new method gives more realistic and accurate displacements of RMBW's than any other existing 'sliding block' methods.

5 ACKNOWLEDGEMENT

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REFERENCES

Atkinson, J.H. 1981 Foundations and slopes, an introduction to applications of critical state soil mechanics, McGraw-Hill, London, Ch.4, 6.

Cai, Z. and Bathurst, R.J. 1996 Seismic-induced permanent displacement of Geosynthetic-reinforced segmental retaining walls Can. Geothech. J., Vol. 33, pp. 937-955.

Huang, C.C. 2000a Investigations of soil retaining structures damaged during the Chi-Chi (Taiwan) earthquake, Journal of the Chinese Institute of Engineers, Vol. 23, No.4, pp. 417-428.

Huang, C.C. 2000b Investigations of the reinforced block facing walls damaged during the Chi-Chi earthquake, Geosynthetic Technical Information, Japan Chapter of the International Geosynthetics Society, Vol. 16, pp. 40-45. (in Japanese)

Ling, H.I. and Leshchinsky, D. 1998 Effects of vertical acceleration on seismic design of Geosynthetic-reinforced structures Geotechnique, Vol. 48, No.3, pp. 347-373.

National Concrete Masonry Association 1998 Segmental retaining walls- Seismic design manual 1st edition, authored by Bathurst, R.J.

Newmark, N. M. 1965 Effect of earthquakes on dams and embankments Geotechnique, Vol. 15, No. 2, pp. 139-159.