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# Stability Analysis of the Geosynthetic-Reinforced Modular Block Walls Damaged During the Chi-Chi Earthquake

Ching-Chuan Huang
*National Cheng-Kung University, Taiwan*

Fumio Tatsuoka
*University of Tokyo, Japan*

# STABILITY ANALYSIS OF THE GEOSYNTHETIC-REINFORCED MODULAR BLOCK WALLS DAMAGED DURING THE CHI-CHI EARTHQUAKE

**Huang, Ching-Chuan**
National Cheng Kung University
Tainan, Taiwan. R. O. C

**Tatsuoka, Fumio**
University of Tokyo,
Tokyo, Japan

## ABSTRACT

Psuedo-static stability analysis based on "Coulomb's one-wedge" and "two-wedge" methods was performed for two geosynthetic-reinforced modular block walls which were either collapsed or lightly damaged during the 1999 Taiwan Chi-Chi earthquake . It was shown that two-wedge failure mechanism is a dominant one for the walls investigated. Difference in the seismic behavior of these walls was partially explained based on the psuedo-static analysis. It was also shown that seismic stability of the reinforced wall depends largely on the connection strength between the facing and the geogrid.

**KEYWORDS**
Earthquake, Geosynthetic-reinforced soil wall, Modular blocks, Stability analysis, Limit equilibrium, Failure, Site investigation

## INTRODUCTION

The Chi-Chi earthquake occurred at 1:47AM on September 21, 1999 in central Taiwan. A fracture of the Chelunpu Fault that was about 100km long in N-S direction and 30km wide in E-W direction was the cause of this major earthquake. A scarp of the ground surface formed by the strike movement of the fault is schematically shown in Fig. 1. Magnitudes of $M_L$=7.3 and $M_w$=7.8 were reported. In addition, maximum peak ground accelerations of 989gal (W-E), 749gal (N-S), and 519gal (U-D), were measured. Severe damage to the near-fault buildings, residential houses, highway embankments, soil retaining structures, bridges was observed. In the present study, results from site investigations and psuedo-static analysis of some near-fault geosynthetic - reinforced modular block walls that were either severely damaged or lightly damaged during the earthquake are reported. The investigated sites are shown in Fig. 1.

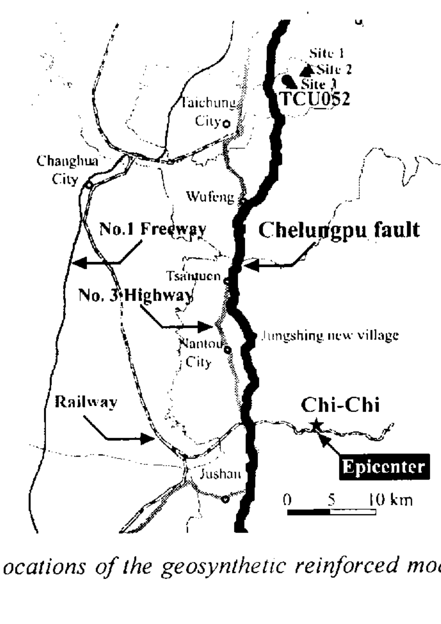


*Fig. 1 Locations of the geosynthetic reinforced modular block walls.*

## SITE INVESTIGATIONS

Sites 1 and 2 loacate about 5km east from the scarp of the Chelunpu Fault, as shown in Plate 1 and Fig. 1. Similar failure modes as shown in Figs. 2 (a) and 2 (b) were observed at these sites. The modular block walls at sites 1 and 2 were about 80m-long and 3m-high. A part of the facing of wall (about 10m-long) collapsed as shown in Fig. 2(a). Adjacent to the collapsed sections, facing buckled at levels between 1/3 and 1/2 of the total wall height from the bottom of wall (Fig. 2b). Ruptures at the juctions of the geogrids (nitted Polyester nets with about 0.3kN/junction) were found behind the collapsed facing. It was observed that the positions of the ruptured junctions coincide with those for FRP rods which were used for connecting the geogrids to the facing blocks. Plate 2 and Fig. 3 show a lightly damaged modular block wall (Site 3) which locates about 2km from site 1. Site investigations revealed the following:

(1) Vertical spacings of reinforcements were 80cm at site 1; 60cm at site 3.
(2) No significant difference in the SPT N values between sites 1 and 3. Despite the above, the N values for the backfill of site 1 is generally larger than those at site 3 (Huang, 2000a). This fact may infer that the soil strength was not the major factor for the failure at site 1. The SPT N values and result of direct shear tests (see Table 2) all reveal that both sites were poorly-compected.
(3) The peak ground accelerations measured at a seismographic station near sites 1-3 are shown in Table 1. The longitudinal direction of the walls at sites 1-3 are approximately in E-W and are facing towards the south. Therefore, the peak horizontal ground acceleration which induce the largest inertial force of the wall was probably around 439gal (Max. N-acceleration).
(4) This area was considered as a less intensive seismic area. A max. horizontal ground acceleration $a_{max}$=0.23g was considered in the bridge design (Ministry of Transport, 1995), and a psuedo-static seismic coefficient $k_h$=0.115 was used in the earth pressure-related design. The small value of $k_h$ may not be a major factor accounted for this failure, because a 7.5m-high cantilever RC wall at the opposite side of the highway embankment was almost intact.

*Plate 1 Collapsed geosynthetic-reinforced modular block wall.*

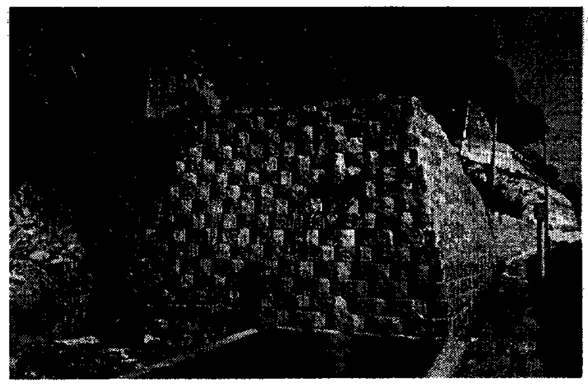

*Plate2 Lightly damaged geosynthetic- reinforced modular block wall.*

Table 1 Peak ground accelerations measured at a station near to sites 1-3.

| Station | Max. U-D (gal) | Max. N-S (gal) | Max. E-W (gal) |
|---|---|---|---|
| TCU 052 | Max. U: 194<br>Max. D: 181 | Max. N: 439<br>Max. S: 353 | Max. E: 349<br>Max. W: 306 |

STABILITY ANALYSIS

The following four types of analysis were performed:

(1) two-wedge method considering the stability of facing.
(2) two-wedge method without considering the facing.
(3) Coulomb's one-wedge method considering the stability of facing.
(4) Coulomb's one-wedge method without considering the facing.

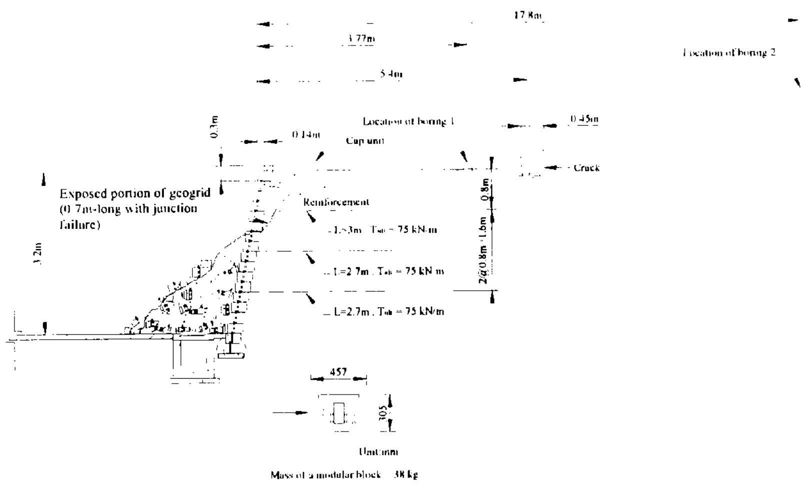


*Fig. 2(a) Cross section of the collapsed modular block wall (site 1).*

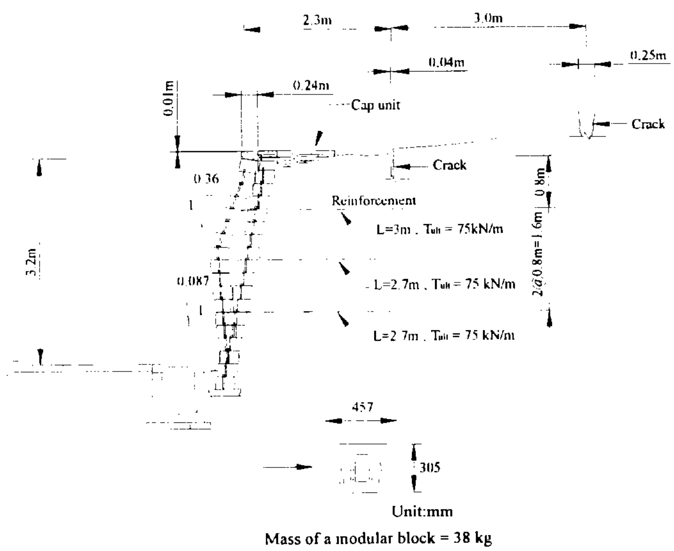


*Fig. 2(b) Cross section of the modular block wall at the verge of collapse (site 1)*

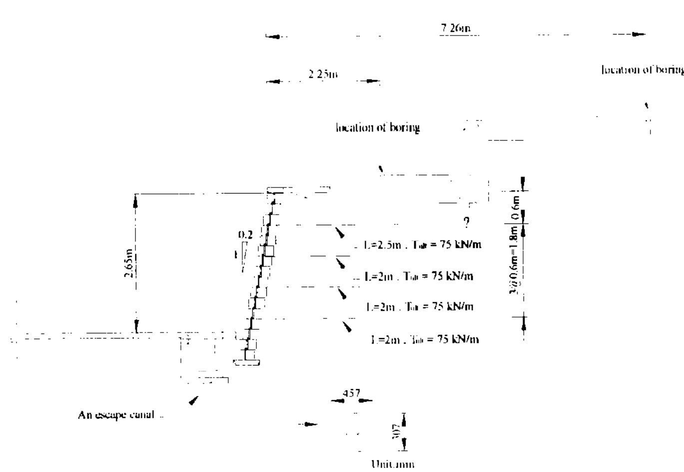


*Fig. 3 Cross section of the intact modular block wall (site3)*

In the analyses (1) and (3), three types of reinforcement-facing block connection strength were employed: (a) junction strength of reinforcement, (b) junction strength + block friction, and (c) tensile strength of reinforcement. For type (a) and (b), twice of the junction strength (i.e.,0.6kN/pin) was used to simulate the pin-to-geogrid junction strength. Type (b) connection takes into account the junction strength of the geogrid as well as the normal contact force between the modular blocks (i.e., Junction strength + block friction). This type of connection may present a higher bound of possible connection strength in the investigated sites. For type (c) connection, the connection strength between the facing and reinforcement was assumed to be the tensile strength of the geogrid. This may simulates a facing-reinforcement connection used in the Reinforced Railway embankment with Rigid facing (RRR) method (Tatsuoka, et al., 1996). In the RRR method, the wrap-around geogrid facing is embedded in a cast-in-place full height concrete (or reinforced concrete) panel. Therefore, the facing-reinforcement connection force could be as large as the tensile strength of the reinforcement. This type of facing-reinforcement connection has demonstrated excellent seismic resisting capability during a major earthquake occurred in 1995 in Japan (Tatsuoka et al. 1998). In the analysis, the block-block friction angle was assumed to be 30°. The block-backfill, and block-foundation friction angles were assumed equal to $\phi/2$.

The reinforcement tensile force used in the analysis was selected from the minimum value of the following three mechanisms:

1. pull-out strength for the reinforcement embedded at right side of the potential failure surface.
2. pull-out strength from the reinforcement embedded at left side of reinforcement + connection strength between block and reinforcement (when the stability of facing was not considered the latter term was omitted)
3. ultimate tensile strength of reinforcement.

Table 2 Strength parameters used in the analyses

| | Site 1 | Site 3 |
|---|---|---|
| c (kN/m$^2$) | 2.0 | 0.0 |
| $\phi$ (°) | 33.8° | 34.4° |

The values of soil strength (c and $\phi$) used in the analyses were obtained in a series of direct shear tests for the on-site soil remolded to the same field density. In these tests, small shear boxes (100mm*100mm) were used. The values of c and $\phi$ used in the analyses were summarized in Table 2. Further tests using large specimens are undergoing.

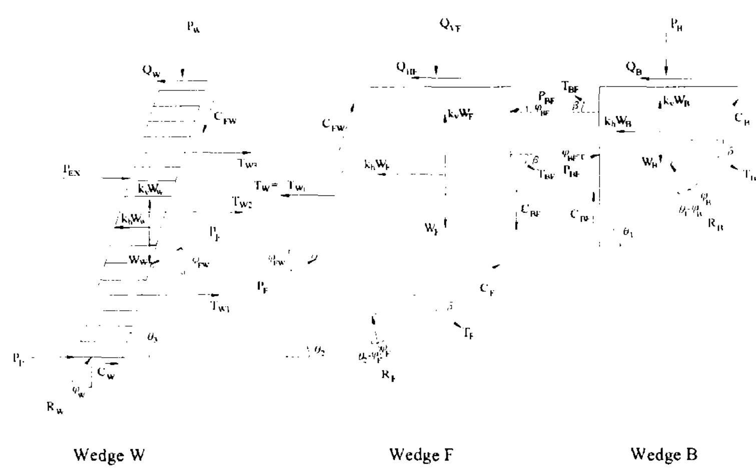


*Fig. 4 Schematic figure of the limit equilibrium method used in the present study.*

Fig. 5(a) shows a comparison on the $F_s$ vs. $k_h$ relationships between types (1) and (2) analyses for site 1. It is seen that for two-wedge analysis, the values of $k_{hcr}$ ( the value of $k_h$ when $F_s$=1.0) largely depends on the values of input connection strength. It is noted that in the case of two-wedge method considering the stability of facing (i.e., type (1) analysis), the seismic earth pressure is a major factor that dominates the stability of facing. Therefore, to attain a global stability for the soil-wall system, a small connection strength for facing requires larger mobilized soil strength in the backfill. A max. value of $k_{hcr}$ ($\cong$0.7) was attained when the connection strength equal to the tensile strength of reinforcement. A min. value of $k_{hcr}$ ($\cong$0.292) was attained when only junction strength of the reinforcement was used. The curve of "No facing element" constitutes a lower bound for all $F_s$ vs. $k_h$ relationships. For $F_s$<1.0 conditions, only the curve of "No facing element" was available because the facing failed at $F_s$=1.0. Fig. 5(b) shows a otherwise similar calculation with Fig. 5(a) except that Coulomb's one-wedge method is used. A conclusion similar to that for Fig. 5(a) can be drawn except that all values of $k_{hcr}$ are larger than those shown in Fig. 5(a). This means that Coulomb's triangular wedge is not a critical failure mechanism in this case. Similar calculations have been performed on site 3. The results are shown in Figs. 6(a) –6(b). Similar tends as seen in Figs. 5(a) and 5(b) can be obtained. It is seen in Fig. 5(a) and 6(a) that the values of $k_{hcr}$ for site 3 are generally larger than those for site 1 by about 10-12%. This may only partially explain their different performances. The following questions remain:

1. Influence from the gravity retaining wall behind the reinforced zone at site 3 is unclear. (The investigation on the geometry of the gravity wall is now undergoing)
2. It is conceivable that cohesion may exist at site 3 because similar on-site soils were used in the construction of these walls.
3. The facing of wall at site 3 may collapse under buckling mode induced by the settlement of backfill. The stress concentration at geogrid-backfill connection imposed by the deformation of the backfill is beyond the scope of limit equilibrium analysis.
4. No test has been performed on the FRP rods (15mm$\phi$, 200 mm long) that were used to connect the blocks and the reinforcement. The strength as well as the length of the rods may influence the shear resistance of the stacked block.

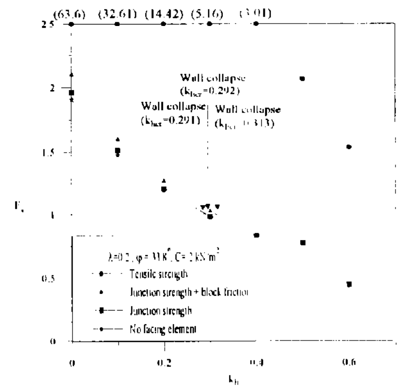


*Fig. 5(a) Calculated $F_s$ vs. $k_h$ relationships for site 1 using "two-wedge" method.*

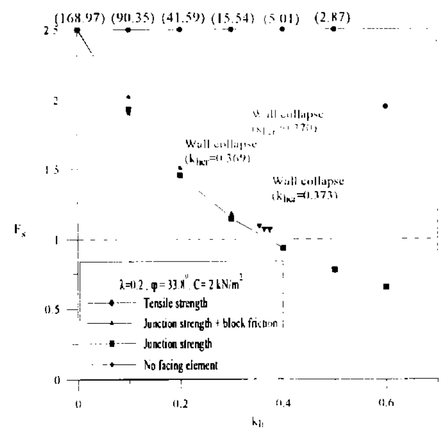


*Fig. 5(b) Calculated $F_s$ vs. $k_h$ relationships for site 1 using "Coulomb's one-wedge" method.*

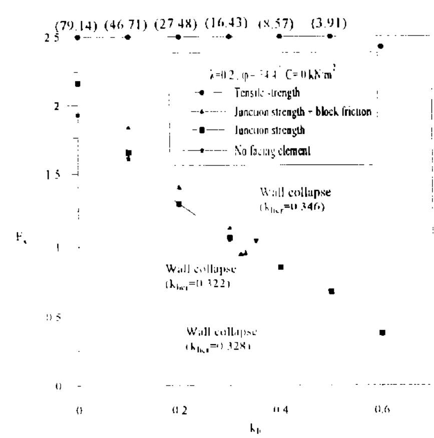


*Fig. 6(a) Calculated $F_s$ vs. $k_h$ relationships for site 3 using "two-wedge" method.*

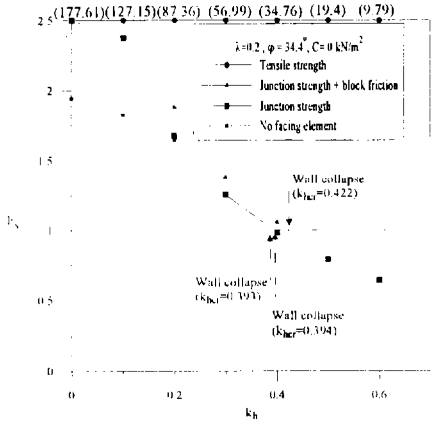


*Fig. 6(b) Calculated $F_s$ vs. $k_h$ relationships for site 3 using "Coulomb's one-wedge" method.*

## CONCULSIONS

Site investigations and psuedo-static analysis were performed on two similar geosynthetic-reinforced modular block walls that behaved very differently in the 1999 Taiwan Chi-Chi earthquake. The following conclusions can be drawn tentatively:

1. The two-wedge failure mechanism, rather than Coulomb's triangular failure wedge, dominates in the collapsed geosynthetic-reinforced modular block wall.
2. The calculated values of $k_{hcr}$ for the lightly damaged site were 10 to 12% higher than that for the collapsed one based on the two-wedge analysis. Different seismic behaviors for the two sites were only partially explained. This indicates that further investigations on the boundary conditions and soil tests for these two sites are necessary.
3. The values of $k_{hcr}$ can be increased significantly by introducing large connection force between the facing and the reinforcement. This inferred that the structural facing element and soil-facing connecting system used in the RRR method is effective in the seismic stability of reinforced wall from the point of view of psuedo-static analysis.

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