Failure analysis of an 80m high geogrid reinforced wall in Taiwan

Une analyse de la rupture d'un mur reinforcé avec des géogrids au Taiwan

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ABSTRACT: During the magnitude 7.3 Chi-Chi earthquake that occurred in Taiwan on September 21st 1999, an 80 m high geogrid reinforced wall failed. The failure site is located about 20 km from the epicenter, and peak ground acceleration at the site is believed to have been in excess of 0.5g during the earthquake. The GRS wall used geogrids as reinforcement and the backfill was an on-site lateritic soil. Because geosynthetic reinforced soil (GRS) structures have generally performed well under seismic conditions, this failure offered a rare opportunity to investigate the ultimate seismic behavior of a full-scale GRS structure. A series of field observations and measurements were made in an attempt to establish the possible failure mechanisms of the wall. Laboratory experiments were conducted on the soils and geosynthetics to obtain appropriate material properties. The results of these studies and the postulated failure mechanisms provided some valuable lessons learned from the failure.

RÉSUMÉ: Pendant le M 7.3 Chi-Chi tremblement de terre qui a eu lieu au Taiwan le 21 septembre 1999, un mur haut de 80 m s'est écroulé. Le site de cette défaillance se trouve circa 20 km de l'épicentre, et on croit que l'accélération maximale au site était plus de 0.5 g pendant le tremblement de terre. Le mur a emploié les géogrids pour le renforcement, et le remblai était d'un sol latéritique régional. Parce que, en général, les structures renforcées fonctionnaient bien pendant des conditions sismiques, cet écroulement donnait une occasion rare pour étudier le fonctionnement sismique d'une structure intégrale. Une sèrie des observations et des mesures était fait pour établir les mécanismes possibles pour la rupture de cet mur. On a fait des expériences au laboratoire sur les sols et sur les géosynthétiques pour obtenir des propriétés appropriés au matériel. Les résultats de ces enquêtes et les

Geosynthetic reinforced soil (GRS) structures have generally enjoyed a reputation for being stable under adverse conditions, including seismic loading. However, a 40 to 50 m high GRS wall retaining an up to 80 m steep earth slope failed during the 1999 Chi-Chi earthquake. Prompted by the scarcity of failures of GRS structures as well as the size of the failed wall, the authors decided to analyze this wall in some detail.

1 SITE CONDITIONS AND ORIGINAL DESIGN OF THE GRS WALL

The failed wall was used as part of the entrance to the National Chi-Nan University (NCNU) located in Pu-Li Township, Tai-Chung County of Taiwan. The NCNU campus is located approximately 20 km northeast of Chi-Chi in central Taiwan, the epicenter of the 7.3 magnitude earthquake which occurred on September 21, 1999. The geologic formations in this region are of Pleistocene to Pliocene age, and the soil deposits within the depths of interest are lateritic in nature consisting of mostly gravel with clay infill.

The NCNU campus is built on a highland that rises approximately 80 m above a neighboring highway (Route 21). The edge of the highland immediately along the highway had an original slope of 28 to 30°. Earlier explorations performed by NCNU indicated that there was a 2-3 m thick clay layer with a dip angle of 30 to 35° towards the face of the slope. The exploration also indicated that the groundwater level was near or below the toe level of the slope. In order to create space for a campus entrance at the highway level and a divided roadway for access to the campus, the toes of the original slope was cut back by as much as 40 m, as shown in Figure 1. The cutting resulted in a very steep slope that required additional support to maintain its stability. So a GRS wall using geogrids as the reinforcing element was used to support the cut slope. The wall face was of the wrap-around type, with geotextile "sandbags" used inside the wrapped face to provide support during construction.

Details of the GRS wall as designed and constructed are shown in Figure 1. The vertical spacing of the geogrids was 1 m. The bottom of the GRS wall was 15 m above the toe of the slope. The height of the GRS wall (from bottom to top) varied from 40 to 50 m and the length of the wall was 250 m, running in north-south direction. Properties of the geogrids are provided in the following sections. The in situ lateritic gravel and clay soil was used as the backfill of the GRS wall. Unfortunately, no record is available as to the compaction specifications or field density tests on the backfill material. To the north of the GRS structure, the slope gradually transformed to its original shape, and a steel net was used for slope stabilization. The steel net reinforced slope did not fail during the earthquake.

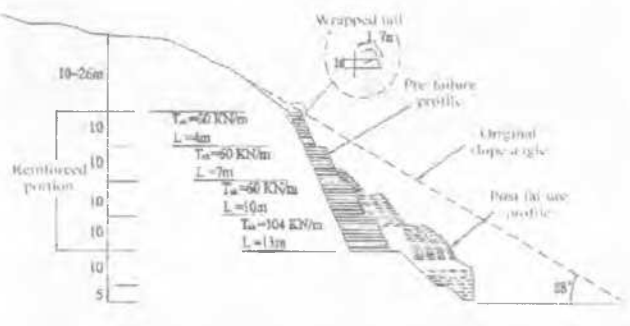

Figure 1. Cross sectional view of the original slope and the GRS wall.

2 FAILURES OF THE GRS WALL

Construction of the GRS wall started in 1994. A massive slope failure as shown in Figure 2 occurred near the top of the slope in 1995 after the original slope was cut to its design grade and just prior to the placement of the GRS wall. The failure zone was backfilled and the GRS wall completed in 1996. A second failure occurred in 1997 near the south end of the GRS wall. A drainage ditch that ran transversely from top to the bottom of the GRS wall slipped downward. A concrete grid with no anchors was constructed at the bottom of the drainage ditch to enhance local stability.

Figure 2. The 1995 slope failure.

The latest failure was triggered by the magnitude 7.3 Chi-Chi earthquake that occurred on September 21st, 1999. The failure site is located about 20 km from the epicenter, and peak ground acceleration at the site is estimated to have been in excess of 0.5 g during the earthquake. The failure was massive. A majority of the GRS wall slipped downward from its original position by as much as 10 to 13 m. Figure 3 shows a front view of the failed GRS wall and Figure 1 depicts a cross-sectional view of the failed GRS wall. According to our observations, the failure did not appear to have extended into the original earth slope behind the GRS wall. In other words, the slippage happened almost entirely within the GRS structure itself. During site visits following the Chi-Chi earthquake, field sand cone and nuclear soil density tests as well as total station surveys were performed

Figure 3. Front view of the failed GRS wall.

3 MATERIAL PROPERTIES

3.1 *Foundation and backfill soil*

According to field direct shear tests, the in situ lateritic gravel and clay mixture had a peak cohesion value (c) of 13 kPa and a drained friction angle (ϕ') of 49°. The residual values of c and ϕ' were 43 kPa and 38°, respectively (Genesis Group, 2000). Table 1 summarizes the dry densities (ρ_d) and water contents (w) of the backfill material recovered from the GRS and steel net reinforced sections, according to the field sand cone and nuclear density tests. Field density tests in the GRS section were performed at 0.5m below the failed surface, where the backfill was believed to be close to its original condition prior to failure.

Table 1. Backfill densities and water contents.

Test Section	ρ_d, Mg/m^3	w, %
Steel net (no failure)	1.6~1.7	12~13
GRS (failure zone)	1.45~1.5	12~13

3.2 *Geogrid reinforcement*

Five different types of geogrids were identified and retrieved from the failure site. "Fresh" samples were cut from the geogrid reinforcement embedded inside the intact portion of the reinforced slope by carefully removing the backfill soil on top of the grids in order to avoid taking samples damaged by the slope failure. Sampled grids were then sealed in a plastic bag and stored in a moisture and temperature controlled laboratory prior to testing. It should be noted, however, that installation damage were not evaluated in this study.

All five types of grids were made of polyester with a polymeric protection coating. Single rib strength tests (GRI:GG1), wide width tensile strength tests (ASTM 4595), and junction strength tests (GRI:GG2) were performed on the sampled geogrids in an effort to identify the strength properties of the reinforcement material. Table 2 shows the results of these tests. Figure 4 depicts typical load-displacement curves from the single rib tests. Some slow strain rate (5 mm/min) were also applied on single ribs to investigate the influence of the strain rate on the strength properties of the geogrids. It was found that the differences caused by the change of strain rates (50 mm/min versus 5 mm/min) were negligible.

Table 2. Results of the single rib strength tests.

	Number of Rib in 1m	Ultimate Tensile Strength, kN/m	Peak Strain, %	Tensile Strength at 5% Strain, kN/m
Grid 1	45	107.8	8.2	59.9
Grid 3	43	267.6	12.9	109.2
Grid 4	43	218.1	11.7	100.0
Grid 5	43	397.0	12.2	151.8
Grid 6	43	140.2	18.6	34.13

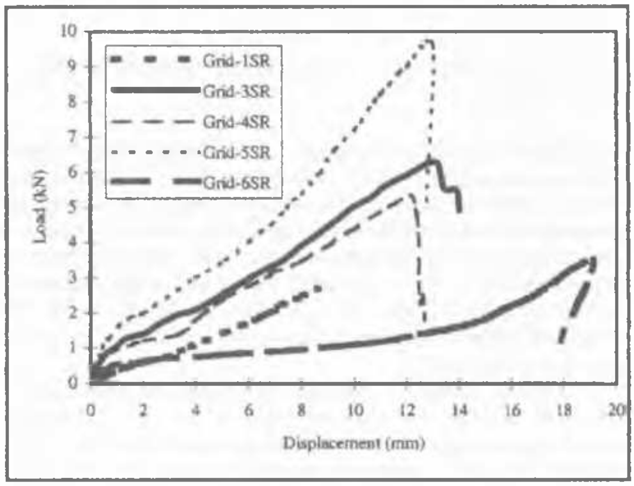

Figure 4. Load-displacement curves from the single rib strength tests.

Table 3 shows the results from wide width tensile strength and junction strength tests. Figure 5 shows typical load-displacement curves from the wide width tensile strength tests. These tests were not performed on samples of Grid 1 and Grid 6 because of the limited amount of grids retrieved from the site. As shown in Table 3, the ultimate strength obtained from wide width tensile strength tests are in a range of 81% (high strength material) to 87% (medium strength material) of those from the single rib tests.

As shown in Figure 1, the designed ultimate tensile strengths are in a range of 60kN/m to 104kN/m, or considerably less than the results of the tensile strength tests in tables 2 and 3. This indicates that the combined reduction factors for installation damage and chemical and biological degradation for all the reinforcement materials were between 3 and 4, which is quite common practice for GRS steep slope and wall designs in Taiwan.

Table 3 Results of the wide width tensile strength tests.

	Ultimate Tensile Strength, kN/m	Peak Strain, %	Tensile Strength at 5% Strain, kN/m	Junction Strength, (kgf)	Ultimate Strength Ratio,* (%)
Grid 1	--	--	--	19.9	--
Grid 3	234.4	8.4	148.0	94.2	87.6
Grid 4	289.4	8.1	139.8	82.2	86.9
Grid 5	323.0	4.7	263.2	79.9	81.4
Grid 6	--	--	--	33.5	--

*:Ultimate tensile strengths obtained from wide width tensile tests divided by those from single rib tests.

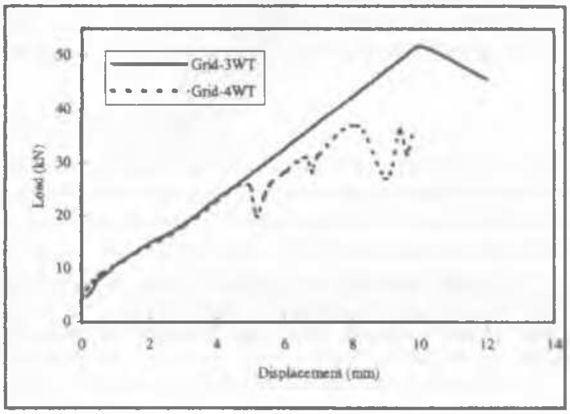

Figure 5. Typical load-displacement curves from the wide width tensile strength tests.

4 FAILURE ANALYSIS

4.1 *Observed types of failures*

Field observations of the wall indicate that failure occurred along the boundary between the GRS structure and the original slope surface. The failure surface had a fairly steep angle (Figure 6). This observation suggests that, under the strong ground motion that occurred during the Chi-Chi earthquake, the reinforcing design of the failed GRS wall was insufficient, i.e., the reinforcement length was too short and/or the vertical spacing of the reinforcement was probably too large.

Unfortunately, the original design calculations are unavailable, so it is impossible now to know what the original design concept for the wall was. However, from the lengths of the reinforcement shown in Figure 1, it appears that potential sliding of the wall was either overlooked or not considered to be of concern. Another possible explanation for the short reinforcement lengths is that the design followed steep slope rather than wall design procedures, although even a steep slope design would likely have required longer lengths than were actually utilized in the wall.

Three types of GRS system failures were identified in this case study:

1. Separation between the adjacent geogrid sheets,
2. "Decomposition" of the GRS system, and
3. Collapses of the wrapped face.

Figure 6. Side view of the failed GRS slope.

Figure 7 shows an example of the separation between two adjacent geogrid sheets. The slope face opened up during the earthquake and the apparently small overlap of the adjacent geogrid sheets failed to accommodate the large local deformations. The separation of the geogrid sheets in turn probably triggered progressively the other two types of failures. As a result of the reinforcement separation, the backfill soil fell out of the geogrid-sandbag face through the opening (Figure 8). The "tail" of the geogrid reinforcement then overturned and pulled out the portion that still remained in the backfill (Figures 8 and 9). The failure of the geogrid-sandbag face then led to the instability of the entire GRS slope.

Figure 7. Separation between the adjacent geogrid layers.

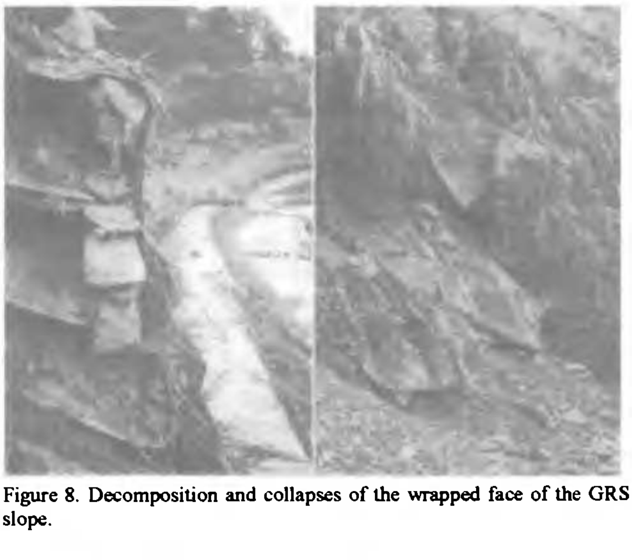

Figure 8. Decomposition and collapses of the wrapped face of the GRS slope.

Figure 9. Remaining reinforcement being pulled out from the backfill.

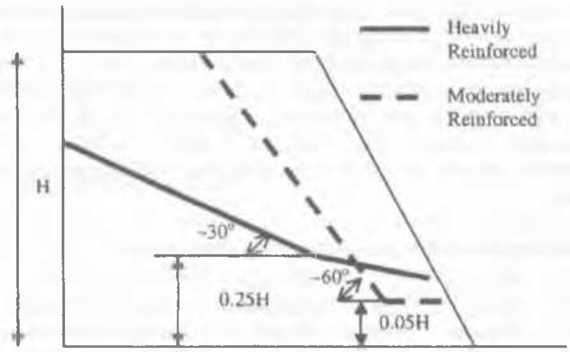

Figure 10. Failure modes for steep GRS slope under seismic loading conditions.

Another factor that probably contributed to the failure was the poor quality construction, particularly the low compacted densities, as indicated in Table 1.

4.2 *Failure modes of steep GRS slopes under seismic loading*

Results of shaking table tests performed at the University of Washington have indicated that the failure modes of model geosynthetic reinforced steep slopes under seismic loading involve bilinear failure surfaces such as those shown in Figure 10 (McElroy, 1997; Perez, 1999). Slopes designed with longer reinforcements and smaller vertical spacings ("heavily reinforced") have a failure surface that is generally flatter and extends more into the backfill than slopes designed with shorter reinforcement lengths and larger vertical spacings ("moderately reinforced"). These observations are consistent with the performance of the GRS wall at NCNU.

The mechanism of failure was investigated numerically using the finite element program, PLAXIS. A model of the reinforced slope, using tension-only geotextile elements surrounded by interface elements, was constructed and subjected to earthquake loading similar to that estimated to have occurred in the Chi-Chi earthquake. The computed permanent displacements, shown in Figure 11, indicate that the failure occurred primarily within the reinforced zone. This mechanism is consistent with the behavior observed in the field.

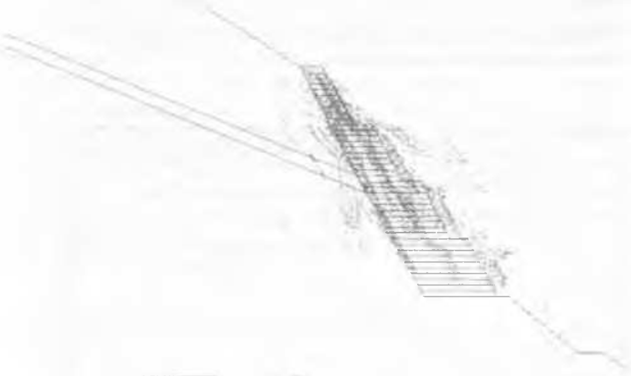

Figure 11. Schematic illustration of permanent displacements computed by PLAXIS. Displacements are highest in reinforced zone, indicating a deformation pattern consistent with that observed following the earthquake.

5 LESSONS LEARNED AND CONCLUSIONS

There were a number of valuable lessons learned from the failure of the GRS wall at NCNU.

The wall was under-reinforced; the embedded lengths of the geogrids were too short for a wall of this height. The vertical grid spacing of 1 m also probably contributed to the instability observed at the wall face. In seismically active areas, simple butt joints or small overlap of the sheets of reinforcement perpendicular to the face may not be sufficient to prevent separation and failure of the face. To further increase face stability, it may be necessary to tie or otherwise positively join the sheets together in the lateral direction.

Poor compaction of the backfill was found in the failed section as compared with the natural ground in the area and with the results of Proctor tests on the same material.

The mode of failure observed in the NCNU wall is similar in concept to those observed in model tests of steep GRS slopes conducted on the shaking table.

However, even with poor backfill compaction, an under-reinforced design, and when subjected to very strong ground shaking from the Chi-Chi earthquake, the failure of the wall was more of a slump than a catastrophic landslide. The failure was ductile rather than a brittle type failure.

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