APPENDIX A. PERFORMANCE OF REINFORCED SOIL STRUCTURES DURING EARTHQUAKES

Past earthquakes have provided numerous case studies of reinforced soil wall performance under dynamic loading. These cases have expanded our knowledge and increased the confidence in this type of retaining systems. In general reinforced soil structures have performed well in earthquakes. Numerous cases have been reported where reinforced soil structure performance in major earthquakes have been documented (Chen et al. 2000; Collin et al. 1992; Eliahu and Watt 1991; Frankenberger et al. 1996; Fukuda and Tajiri 1994; Huang 2000; Kobayashi et al. 1996; Kramer et al. 2001; Kutter et al. 1990; Ling et al. 2001; Nishimura et al. 1996; Sandri 1994, 1997; Sitar et al. 1997; Stewart et al. 1994a; Stewart et al. 1994b; Tatsuoka et al. 1995, 1996a; Tatsuoka et al. 1996b; Tatsuoka et al. 1998; White and Holtz 1996).

LOMA PRIETA EARTHQUAKE, CALIFORNIA, 1989

Numerous reinforced soil walls and slopes were located within the affected area of the 1989 Loma Prieta Earthquake (Collin et al. 1992; Eliahu and Watt 1991; Kutter et al. 1990). The earthquake had a magnitude 7.1 and peak ground accelerations up to 0.6g were measured.

Several geosynthetic reinforced structures were identified and investigated. Of those structures reported the highest wall and slope were 5.5 meters and 24.5 meters respectively. These structures were built with geogrid reinforcements. Reinforcements were wrapped-around as wall facing in most of the structures. In some of the structures segmental masonry blocks were used as facing elements. No direct measurements were available at any of these sites except one. No signs of damage or cracking were observed. One of the reinforced slopes (La Honda Slope) had an inclinometer and it was possible to estimate the earthquake induced lateral displacements. Cross section of this slope is shown in Figure A-1. Inclinometer measurements were made before the earthquake to monitor the slope as shown in Figure A-2. Comparison of pre- and post-earthquake inclinometer measurements indicate that top of the slope deformed about 2 cm laterally, corresponding to 0.2% of the slope height.

Loma Prieta Earthquake was particularly important to demonstrate the seismic performance of geogrid reinforced soil structures, because it was the first major seismic event since the use of structural geogrids initiated in the early 1980s.

Numerous walls built with Reinforced Earth technology (i.e. steel strips and concrete facing panels) were also investigated for their performance in the Loma Prieta Earthquake (Reinforced Earth Co. 1998). Twenty Reinforced Earth walls at 9 sites, the highest one being 20 meters, were inspected. No evidence of damage was identified in any of these structures.

Thirty-four reinforced soil structures were reported (Kutter et al. 1990). These included a variety of construction technologies from steel strips reinforcements, tire-anchor timber walls to soil nailing. Significant damage was observed at 4.5 meter high tire-anchor timber wall. However, no details are available from any of these cases. Therefore, it is not possible to identify if any of these cases are of the ones reported in the other studies described above.

Eight soil nailed excavations were identified within the affected area. Tallest of these structures was 9.8 meters and all of these structures performed very well, showing no signs of distress (Felio et al. 1990; Vucetic et al. 1998).

NORTHRIDGE EARTHQUAKE, CALIFORNIA, 1994

Northridge Earthquake was a magnitude 6.7 event with peak ground accelerations reaching 0.9g at locations close to the source of energy release. Northridge Earthquake was an important event due to its relatively high vertical acceleration levels (Stewart et al. 1994a). Numerous cases of reinforced soil structure performance were documented (Bathurst and Cai 1995; Sandri 1994, 1997; Stewart et al. 1994a; Stewart et al. 1994b; White and Holtz 1996). In general reinforced soil structures exhibited excellent performance.

Of those structures reported the highest wall and slope were 11.6 meters and 24.5 meters respectively. These structures were built with geogrid reinforcements and segmental masonry blocks facing elements. Several of the walls showed some signs of distress (i.e. settlement and longitudinal tension cracks along the retained fill, slight out-of-plane bulging). It is noteworthy to mention that some of those structures were shaken with acceleration levels almost twice the values they were designed against (Sandri 1997).

There were 23 Reinforced Earth structures built with steel strips and concrete facing panels (Frankenberger et al. 1996; Sitar et al. 1997). Tallest of these structures was 17 meters. The walls performed well. Some panels separated causing spillage of material and some minor cracking was observed. Only one 16 meter wall experienced some problems, wall face bulging 46 centimeters, 3% of the wall height (Sitar et al. 1997).

HYOGOKEN-NANBU EARTHQUAKE (KOBE), JAPAN, 1995

This was a magnitude 6.9 earthquake that caused significant damage to Kobe and surroundings. Several case studies of reinforced soil structure performance are reported by researchers from Japan (Kobayashi et al. 1996; Nishimura et al. 1996; Tatsuoka et al. 1995, 1996a; Tatsuoka et al. 1996b; Tatsuoka et al. 1998).

Performances of four geogrid reinforced walls with heights between 3 to 8 meters are reported. These walls support the railway embankment and also serve as bridge abutment at several locations. These structures were composed of a monolithic cast-in-place facing and relatively short reinforcements. (Tatsuoka et al. 1995, 1996a; Tatsuoka et al. 1996b; Tatsuoka et al. 1998). These walls were designed with the pseudo-static method using a seismic horizontal coefficient of 0.2. Peak ground accelerations at these sites reached up to 0.8g. Several nearby conventional retaining walls and houses suffered heavy damage. Three of the walls performed very well and exhibited no visual damage. The cross section of one of the walls along Japan Railway Tokaido Line is shown in Figure A-3. There is an electric supply frame located behind the wall and the 2.2 meter wide circular foundation of this frame embedded deep behind the retaining structure. A detailed description of this geogrid-reinforced retaining structure including construction details, monitoring data during and after construction is given in Kanazawa et al. (1992). Additionally, full-scale experimental studies performed to investigate the effect of the embedded foundation are reported in Tamura et al. (1992). A plan view and cross section of other geogrid-reinforced retaining walls along the same railroad line are given in Figure A-4. These are utilized as bridge abutments. Geogrid-reinforced soil structures at this site at Amagasaki, Japan and at two other sites performed very well. However, the fourth wall (Tanata wall) experienced some damage. The wall moved about 30 cm horizontally as seen in Figure A-5. The wall also suffered facing panels spalling and cracking. This wall was built with relatively short reinforcements.

In addition to the above geogrid-reinforced walls, ten other structures were identified (Nishimura et al. 1996). Heights of these walls ranged between 4 to 11 meters. These structured performed very well with no signs of damage except two of the walls where some minor cracks and facing separation were observed.

Mechanically stabilized earth walls built with steel strips (i.e. Reinforced Earth) also performed well. Kobayashi et al. (1996) reports 124 Reinforced Earth structures within 40 km of the earthquake source. Twenty-one of these were within the area of severe shaking. Most of these structures performed very well. However, some signs of damage were observed at three of these walls. One of walls that was built as part of the approach embankment is shown in Figure A-6.

The wall was 9 meters high at its maximum and was designed with a seismic coefficient of 0.15. Earthquake caused the top of the wall move outward about 15 cm. Similar damage patterns were observed at 2 other Reinforced Earth walls. Tatsuoka et al. (1996a) indicates 66 of these structures were identified within 70 km of the earthquake source. Obviously Reinforced Earth wall inventory from both reports overlap (Kobayashi et al. 1996; Tatsuoka et al. 1996a) and it is not possible to identify cases that are not reported in detail. Tatsuoka et al. (1996a) describes the same three walls that suffered damage and asserts the similar damage assessments.

Seven slopes/excavations stabilized by soil nailing were reported (Tatsuoka et al. 1995). No noticeable deformation or signs of damage were observed at these structures except at one excavation supported by soil nailing. Soil nailing was used to support a 14-meter deep excavation. The excavation was at a late stage at the time up to the earthquake and was backfilled leaving a free face of about 4 meters. Comparison of before and after earthquake inclinometer measurements indicate that top of the wall deformed about 3 millimeters.

CHI-CHI EARTHQUAKE, TAIWAN, 1999

This was a magnitude 7.6 event and caused significant damage to structures in urban areas. Several studies have been performed on the performance of reinforced soil structures during this earthquake (Chen et al. 2000; Huang 2000; Ling et al. 2001). Geosynthetic reinforced walls with modular facing blocks are commonly used in Taiwan and this earthquake provided the first opportunity to test the performance of this type of MSE structures. Several failures have been reported where mechanically stabilized embankments and reinforced soil slopes experienced significant damage and total collapse.

Four sites with modular-block geosynthetic-reinforced retaining walls were identified. (Huang 2000; Ling et al. 2001). At one of these sites modular-lock geosynthetic-reinforced retaining walls were utilized to retain excavated slopes for a housing development project. Large cracks and settlements were observed along the slopes indicating global stability problems with the slopes in the area. Several reinforced concrete retaining walls and unreinforced modular-block walls at the site collapsed and/or got damaged. The height of the reinforced soil structures at the site varied from location to location, reaching 5 meters at its highest. Parts of these modular-block geosynthetic-reinforced soil structures experienced damage. At one location blocks near the top of the walls displaced outward and at another section the blocks fell apart. It was observed that the walls at this site were built using geogrid reinforcements and good quality backfill.

At another site along Ta Kung Roadway 129 a section of a geosynthetic-reinforced wall with modular block facing collapsed and other sections suffered heavy damage. This wall was built with geogrid reinforcements and silty sand backfill. The wall was 3.4 meters at the collapsed section. A cross section of the collapse is shown in Figure A-7. In addition to the section that collapsed, other sections of the wall also suffered significant damage where blocks moved out-of-alignment and separated causing backfill material to spill out. The largest bulging displacement was observed at about 1.6 meters from the bottom of the wall (about 1/2 to 1/3 of the wall height). Geogrid reinforcements were observed to be torn at location of connection pins. The cross section of the wall at this part of the wall and a sketch of the observed deformations are shown in Figure A-8. A 7.5-meter high conventional reinforced concrete retaining wall supports the other side of the highway embankment and suffered only minor damage. This RC retaining wall was designed by the same organization that built the reinforced soil structure. Both structures were built around the same period of time (Huang 2000). Peak ground accelerations during the earthquake were in excess of 0.4g in the area whereas seismic provisions require a seismic coefficient k_h=0.115 for this area.

Two short walls located near a stadium suffered some damage. One of the walls was 2 meters high and facing blocks separated. It is noted that this was due to the movement of the foundation of lamp-posts located behind the wall. The other wall was 3 meters high and collapsed. This failure was attributed to short reinforcements. Longitudinal cracks were observed behind the wall.

At the fourth site a reinforced soil structure suffered some damage. This is a geosynthetic-reinforced retaining wall composed of three stacked walls. The height of the wall is not mentioned in the paper, however from the given photograph it appears the wall is composed of 33 modular blocks making it about 6.5 meters high (given each block is 20 cm). Part of the wall that was not reinforced collapsed.

A 40-meter high reinforced slope collapsed during the earthquake. Geogrids were used as reinforcements and wrapped around as facing. On-site silty clay was used as backfill material (Huang 2000; Ling et al. 2001). The total height of the slope is 80 meters, 40 meters of that composed of 4 reinforced sections each 10 meters high as shown in Figure A-9. This slope failed in 1994 immediately following completion and large deformations were observed in 1996 following repair. In addition to this collapse-repair sequence, parts of the unreinforced slope beneath the reinforced slope were strengthened with reinforced concrete frame and tie-back anchors. All these complexities make it a difficult case to assess the mechanism that lead to collapse during the earthquake.

This slope was designed using a pseudo-static seismic coefficient k_h=0.15 and a pore pressure coefficient r_u=0. Slope stability analyses using these values in the design phase yielded a Factor of Safety, FS=1.1. Stability analyses performed after the earthquake, using the same parameters resulted in FS=1.0 (Huang 2000). In any case this slope exhibited stability problems immediately following construction and conventional stability analyses yield a marginal factor of safety. Studies have been performed on this case study (Chen et al. 2000; Holtz et al. 2001). However, limitations on the pre-earthquake conditions prevent these studies to be conclusive about the collapse.

In another case, a 35 meter high reinforced slope performed well. Geogrids were used as reinforcements and wrapped around as facing (Ling et al. 2001). Detailed description of this reinforced slope are given in Chou et al. (1994).

NISQUALLY EARTHQUAKE, WASHINGTON, 2000

A mechanically-stabilized earth wall supporting a hotel parking lot collapsed following the earthquake (Kramer et al. 2001). This structure was built with geogrid reinforcements and modular-block concrete. Parts of this wall experienced problems following construction and were repaired. It is not clear though if the failed section was actually part of the repaired segments or not. It is also possible that soft foundation soils contributed to this collapse.

Extended Stay Hotel MSE Wall- Seattle: (Wall Failure; Geosynthetic-Reinforced MSE Wall; Weak Foundation)

The Extended Stay Wall in Tumwater, WA is a geosynthetic reinforced MSE structure with concrete block facing (Kramer et al. 2001). The site is approximately 27 km from the epicenter of the 2001 Nisqually Earthquake and the peak ground accelerations in the area reached about 0.15g. The wall is 4.5 m high with 3.5 m-long geogrid reinforcement. The wall supports a backfill slope with a 2:1 grade. Sections of the wall collapsed during the earthquake. It is highly probable that the poor subsoil conditions were the major factors that caused the collapse.

Construction reports indicate that problems with the foundation conditions were noticed prior to and during construction (Geo-Group Northwest, 2000). Some segments of the wall were demolished, soft foundation soils removed and rebuilt. It is not clear which segments suffered heavy damage during the earthquake and if the repair efforts were effective in earthquake performance.

Costco MSE Wall – Seattle: (Large Movements; Geosynthetic-Reinforced MSE Wall; Inadequate Base Width)

The Costco MSE Wall near Tacoma, WA is a geosynthetic-reinforced MSE structure with concrete block facing that retains the parking lot area of the Costco Department Store. The site is approximately 30 km from the epicenter and the peak ground accelerations in the area reached about 0.07g during the event.

The wall is about 5.5 meters high at the maximum section and the reinforcement length at this section is 2.8 meters, giving a width-to-height ratio of 0.50 (Geo-Engineers 1999). At this section, the wall retains a level backfill. A sloping backfill (H:V 2:1) is retained at some sections. Design drawings indicate that the width-to-height ratio at these locations is typically 0.70. These width-to-height ratios lower than those generally recommended (0.70). Geogrid bars were used as reinforcing elements and were placed a 1.0 to 1.5-m vertical spacings.

The wall remained intact after the earthquake. Lateral displacements due to shaking resulted in a wedge-type sliding behind the wall. This movement produced large cracks at the crest of the sloping backfill and retained soil with horizontal offsets of up to 25 cm and vertical offsets in the range of 2 to 3 cm. The exact displacement pattern of the structure that led to this pattern is not known. Typically in such cases the wall may move laterally as a rigid block or it may deform undergoing internal straining, tilting etc. Some out-of-plane bulging was observed at lower elevations of the wall at some sections.

This is an important case because it demonstrates how these structures can undergo large displacements before actually collapsing. This is consistent with our reconnaissance observations and recent dynamic analysis results of the Arifiye RE wall associated with the 1999 Kocaeli (Turkey) Earthquake. This wall should also provide some insight into the effects of sloping backfill, geosynthetic structural elements (as opposed to more rigid steel strips, etc.), and width-to-height ratio.

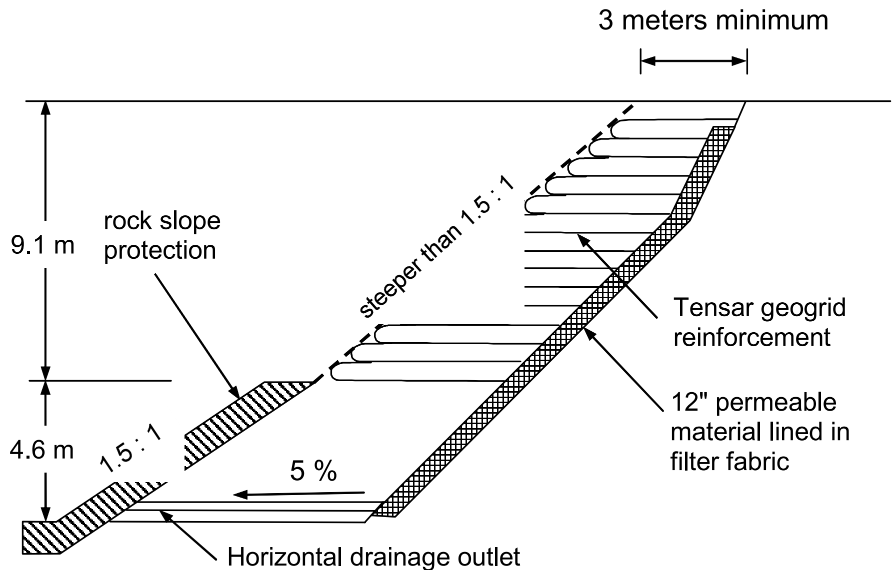

Figure A-1. Cross section of La Honda reinforced slope (Collin et al. 1992)

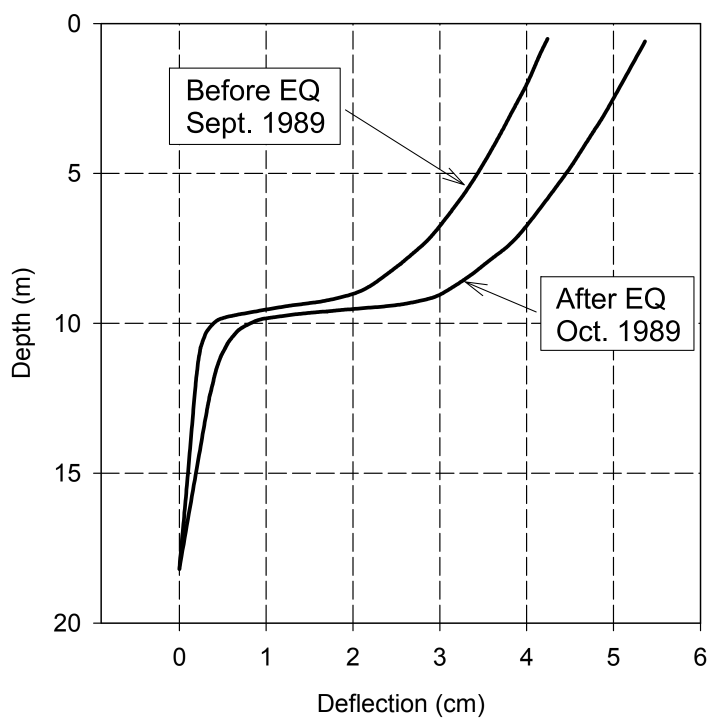

Figure A-2. Inclinometer data showing before and after earthquake measurements (Collin et al. 1992)

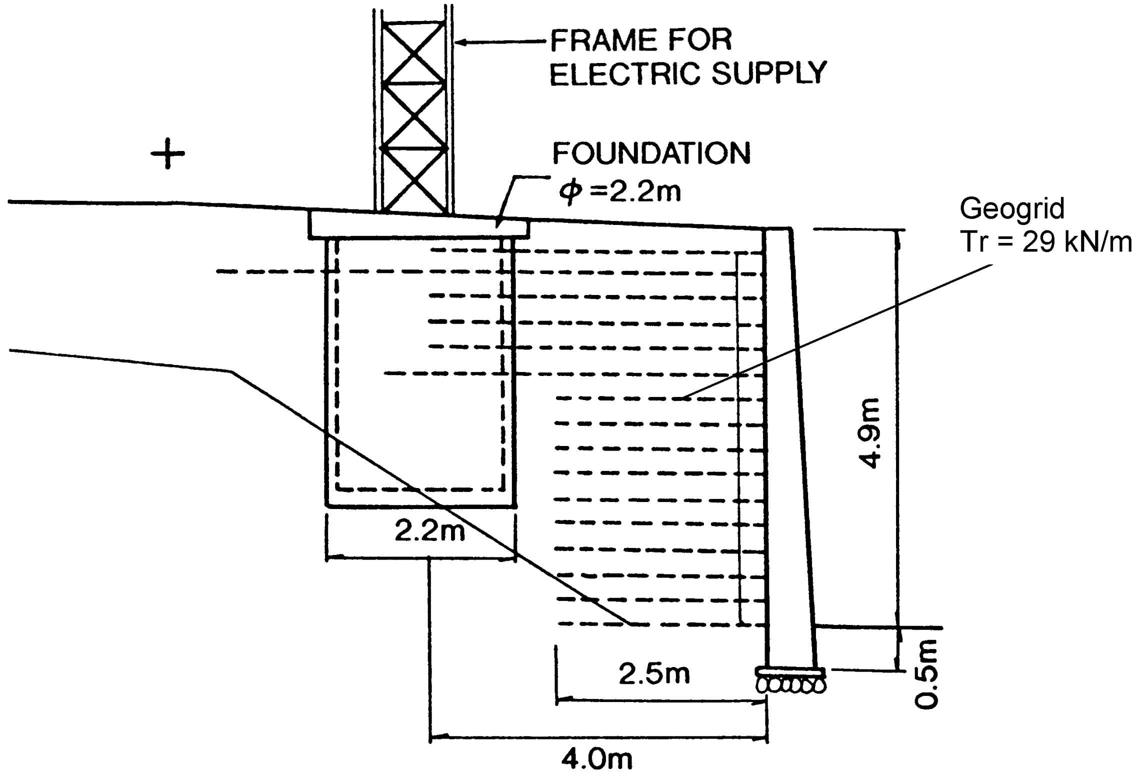

Figure A-3. Cross-section of a geogrid-reinforced wall along the railway (Tatsuoka et al. 1996a)

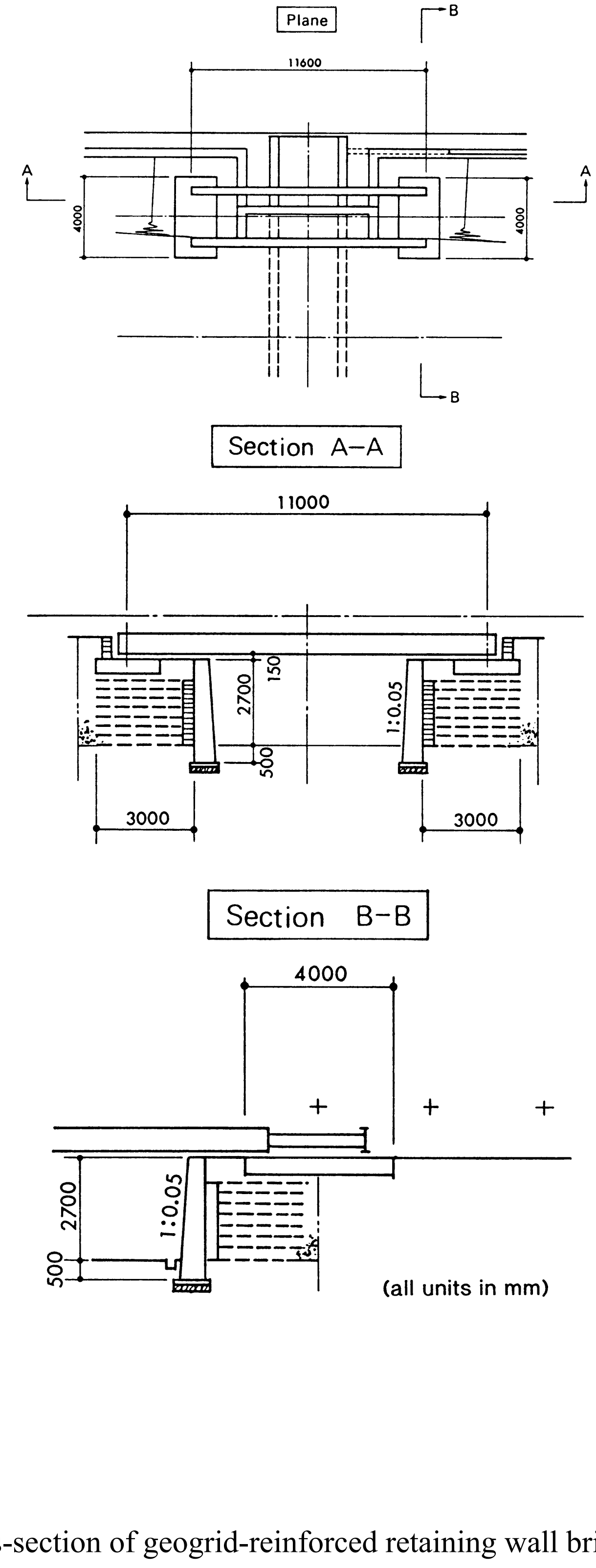

Figure A-4. Plan and cross-section of geogrid-reinforced retaining wall bridge abutments (Kanazawa et al. 1992)

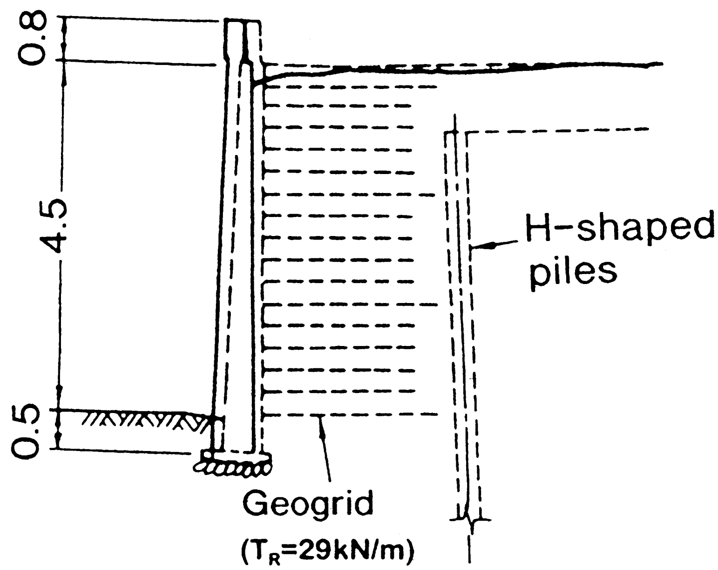

Figure A-5. Cross section of geogrid-reinforced wall that suffered some damage (Tatsuoka et al. 1996a)

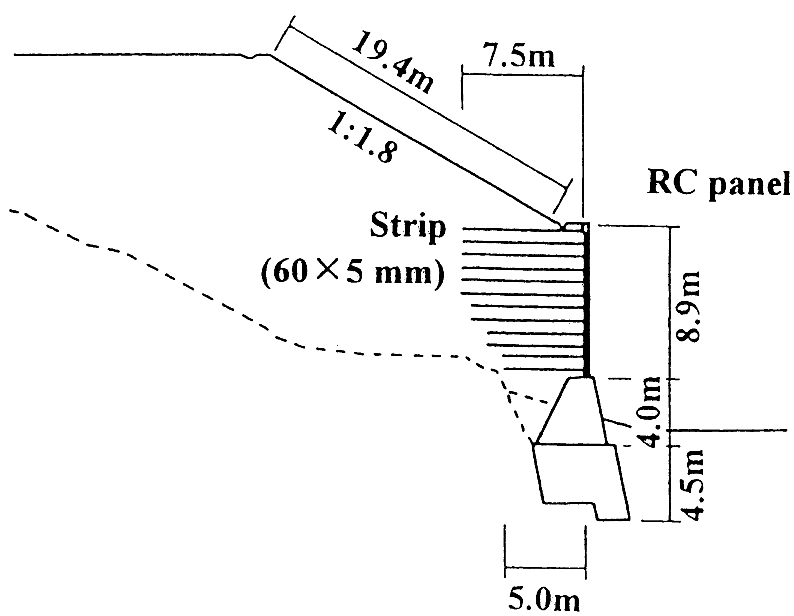

Figure A-6. Cross section of the Reinforced Earth wall that suffered some damage (Tatsuoka et al. 1996a)

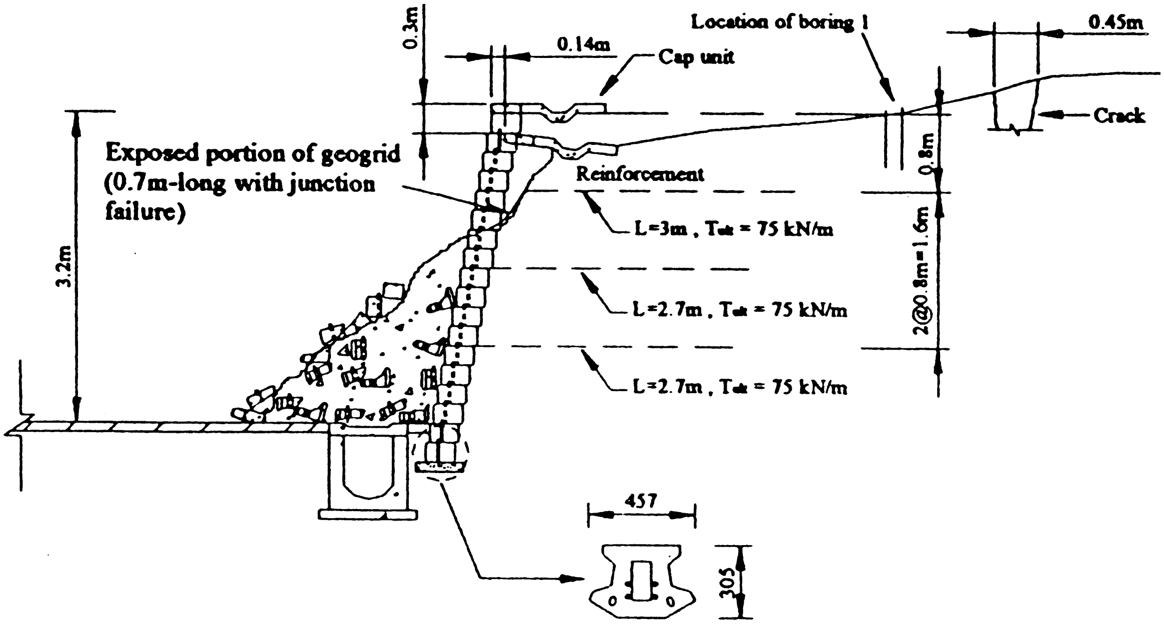

Figure A-7. Cross section of the collapsed geosynthetic reinforced modular block wall (Huang 2000)

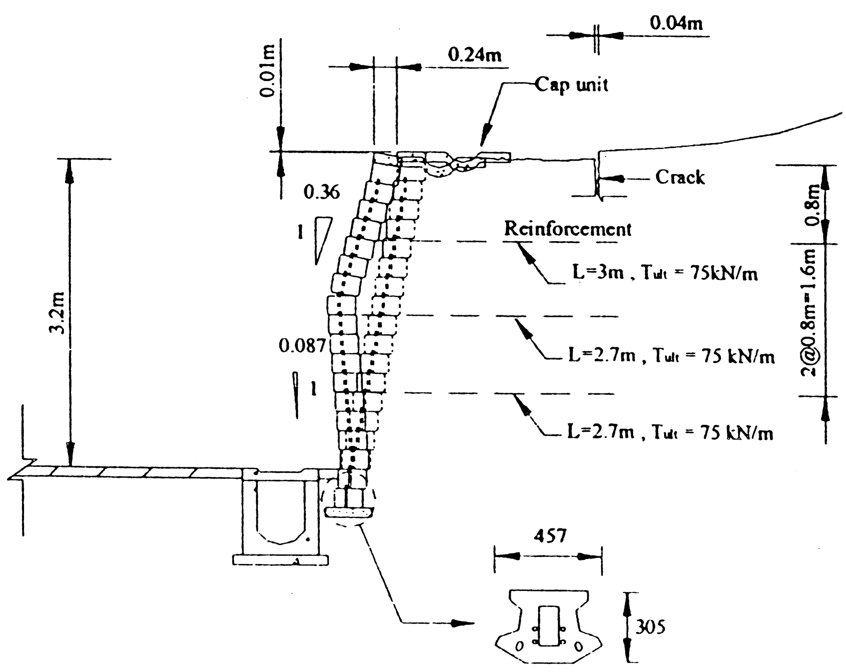

Figure A-8. Large deformations at the geosynthetic reinforced modular block wall (Huang 2000)

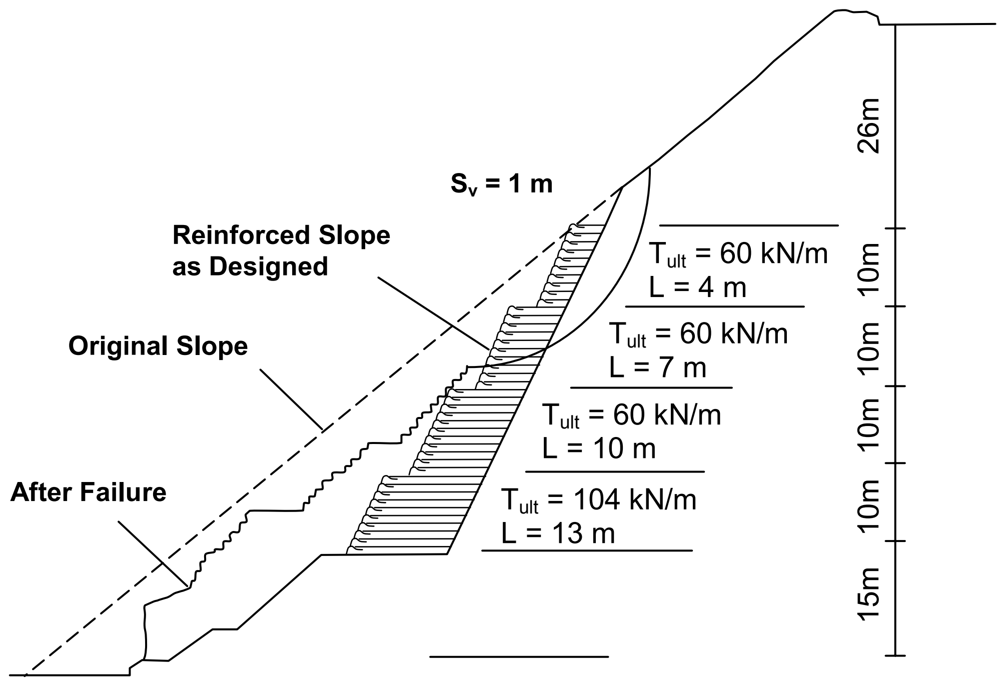

Figure A-9. Cross section of the failed reinforced slope (adapted from Huang 2000)