Geo-hazards and mitigation

New Horizons in Earth Reinforcement – Otani, Miyata & Mukunoki (eds)
© 2008 Taylor & Francis Group, London, ISBN 978-0-415-45775-0

Earthquake performance of reinforced earth embankment subjected to strong shaking and ground deformations

C.G. Olgun & J.R. Martin II
Virginia Tech, Blacksburg VA, USA

H.T. Durgunoğlu
Boğaziçi University, Istanbul, Turkey

T. Karadayılar
Zetaş Earth Technology Corporation, Istanbul, Turkey

ABSTRACT: The 1999 Kocaeli Earthquake (M7.4) in northwestern Turkey provided the opportunity to study the field performance of a double-walled reinforced-earth (RE) embankment designed according to procedures essentially same as current guidelines used in the US. The embankment was located just a few meters from the ruptured fault and subjected to strong ground shaking and significant permanent ground deformations. Unanticipated liquefaction-related settlements also occurred beneath the embankment. Ground shaking at the site was much higher than what was accounted for using the k_h design value of 0.1. Estimated peak ground acceleration was about 0.25 g, corresponding to an estimated equivalent k_h of about 0.3. Despite being subjected to ground motions that significantly exceeded the design levels, the RE system performed well, suffering only minor damages related mainly to the ground deformations.

1 INTRODUCTION

Recent decades have seen an increased usage of reinforced soil structures, and designs are becoming more aggressive with taller walls and a wider variety of reinforcing elements and facing materials. The August 17, 1999 Kocaeli Earthquake ($M_w = 7.4$) that struck northwestern Turkey provided an important opportunity to study the field performance of a number of mechanically-stabilized and soil-nailed walls located in the affected region. The earthquake setting is shown in Figure 1. Of particular significance was the performance of a Reinforced Earth® approach embankment at the Arifiye Bridge Overpass. The embankment was immediately adjacent to the ruptured fault and underlain by soft and liquefiable soils. Peak ground accelerations during the earthquake are estimated at about 0.25 g for this site (Olgun 2003). The wall was designed using specifications similar to FHWA, but was designed for shaking levels much lower than those estimated to have occurred. Following the earthquake, a detailed field reconnaissance was made that included measurements of wall displacements, ground settlements, and fault-related ground movements. Despite being subjected to ground shaking levels above the design values, the RE system performed well, suffering only minor damage. This paper provides a description of the RE embankment, the seismic design of the structure, observed performance, and the numerical studies performed to better understand the seismic behavior.

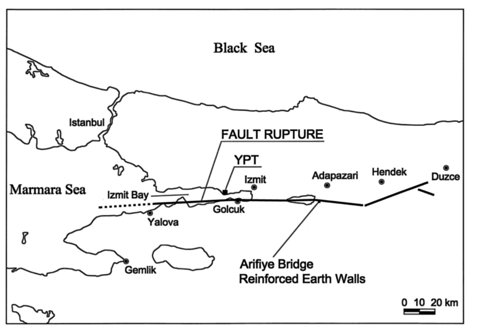

Figure 1. Setting of the August 17, 1999 Kocaeli Earthquake.

2 ARIFIYE BRIDGE OVERPASS AND RE EMBANKMENT

The Arifiye Bridge Overpass, which was constructed in 1988 and destroyed in the 1999 earthquake, consisted of four simply-supported spans resting on approach abutments and three mid-span pier supports. The two wing walls of the northern approach embankment were constructed using Reinforced Earth® (RE) technology, whereas the southern approach was a conventional earth embankment with sloping sides. The site is located along the Trans European Motorway adjacent to the ruptured fault as shown in Figure 2.

The northern approach ramp was 145 m long and 12.5 m wide with one traffic lane in each direction. The maximum height of the RE walls was 10 m. The bridge deck rested on a reinforced concrete abutment supported by a pile foundation. The RE walls that formed the embankment were of conventional design, consisting of cruciform and/or square, interlocking reinforced-concrete facing panels and ribbed, galvanized steel reinforcing strips. The facing panels were 150 cm × 150 cm in frontal area, and the reinforcing strips had a cross section of 40 mm × 5 mm. At the 10 meter high section, five strips per panel were used for the two lower panels, and four strips were used for the upper panels. The reinforcement length was 7 m along the section of the wall where the height ranged between 8–10 m. Reinforcement length was decreased progressively at shorter sections of the approach embankment. The reinforcing elements from each side of the wall overlapped 1.5 m at the center and were not connected.

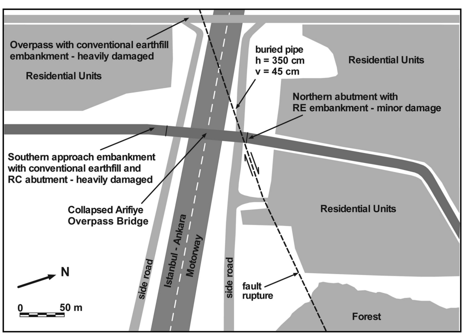

Figure 2. Plan view of Arifiye Overpass and the fault rupture.

The backfill soil was of high quality, consisting of sand and gravel that was compacted in lifts during wall construction.

As shown in Figure 3, a reinforced-concrete culvert passed beneath the wall. The culvert is located in a creek channel that runs beneath the site. Suspected liquefaction in the creek-bed soils beneath this culvert led to significant earthquake-induced settlements in this area of the wall. Also, slip joints were used along the height of the wall on both sides of the concrete culvert, as well as between the RE wall and the reinforced concrete bridge abutment. These special joints were used to mitigate anticipated static differential settlements, but apparently played an important role in limiting damage from earthquake-induced differential settlements.

The Arifiye Overpass site is situated within a deposit of Quaternary alluvial sediments consisting of alternating layers of medium clay/silt and loose sand with a shallow water table. The sounding from one of the Cone Penetration Tests near the culvert is shown in Figure 4. The upper 5 m of the profile consists of 2 m of silty/clayey sand fill underlain by a 3 m-thick medium clay layer with Q_c values of about 1.5 MPa. The clay is underlain by a 1 m-thick stratum of clean sand with an $I_c < 1.5$ and an estimated fines content <5%. Average value of normalized clean-sand-equivalent tip resistance ($q_{c1,CS}$) for the sand is 120. This layer is liquefiable under moderate levels of ground shaking. A mixed stratum of clay with lenses of interbedded sand is encountered below the sand between 5–8 m depth. Normalized clean-sand-equivalent tip resistance ($q_{c1,CS}$) for this stratum averages at 80. A medium-to-stiff clay stratum with an average Q_c of 2 MPa extends from a depth of 8 m down to 22 m where the CPT was terminated. The water table was encountered at a depth of about 2 m.

The sandy levels are potentially liquefiable under moderate shaking as evidenced by their low penetration resistance (Olgun 2003). Furthermore it was not possible to sample and test the clayey levels for a more detailed assessment of their cyclic vulnerability.

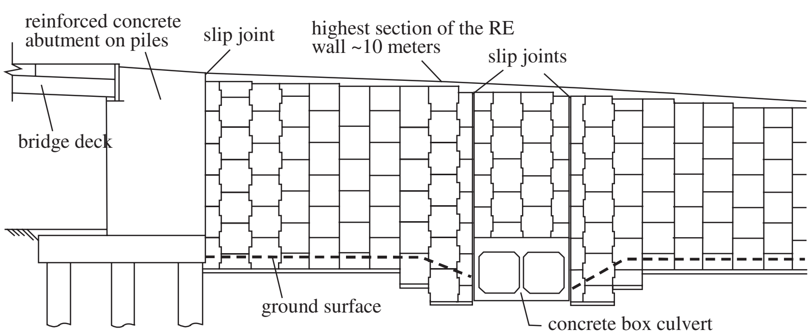

Figure 3. Side view of the Arifiye Overpass RE wall (northern approach embankment).

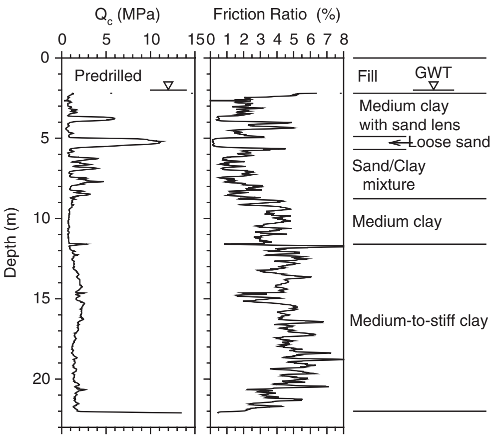

Figure 4. CPT sounding between the culvert and the reinforced concrete abutment.

However the observed ground failure pattern and the surface cracks near the base of the embankment strongly suggests such a potential vulnerability with these soils.

3 OBSERVED FIELD PERFORMANCE

Field reconnaissance at the Arifiye Overpass site was conducted following the August 1999 M7.4 earthquake (Mitchell et al. 2000). Peak ground acceleration at the site was in the range of 0.25 g, close to that recorded at other near-fault sites underlain by similar soil conditions such as the YPT station (Safak & Erdik 2000). In addition to significant ground shaking, ground displacements within a few meters of the RE walls were large, as the surficial fault rupture passed between the northern abutment and the center pier. The maximum horizontal and vertical ground displacements near the northern abutment were estimated at 3.5 m and 0.45 m, respectively, as inferred from the measured offset of a nearby ruptured pipe. Four spans of the bridge collapsed in a "saw-tooth" manner due to the resultant relative displacements between the piers and abutments along with beam-seat widths that were insufficient to accommodate the movements.

In addition to fault-related ground deformations, foundation settlements of up to 25 cm were observed. The resulting differential wall settlement caused the facing panels to become separated and misaligned, which allowed spillage of some backfill material. The maximum out-of-plane wall panel displacement was about 10 cm and occurred at 3 m or one-third of the wall height from the base. It appears that the heavy embankment and the relatively strong fill underneath the wall (2-m thick dry crust) punched through the soft foundation soil forming a localized "cone of depression" with the maximum settlement of 25 cm concentrated at the wall section overlying the culvert and creek bed. Numerous ground cracks running perpendicular to the wall were observed at ground surface near the embankment. The location and orientation of the cracks suggests they were associated with differential settlements along the base of the wall. In particular, the culvert appears to have settled during the earthquake, probably due to the presence of the soft and/or liquefiable creek bed sediments. As discussed later, it is likely that liquefaction occurred in the underlying 1-m sand layer located at a depth of 5 m. No sand boils were observed at the site or neighboring sites. However, the presence of a 2-meter thick dry crust would have likely prevented any surficial manifestation of liquefaction at depth. It is possible too that that the fined-grained silty and clay strata suffered strength loss and softening and allowed earthquake-induced undrained shear distortions and settlement under the weight of the structure. Significant earthquake-induced settlements of similar fine-grained soils under loaded areas were reported at other sites in the region during the earthquake (Sancio et al. 2002, Martin et al. 2004).

The slip joints used along each side of the concrete culvert and other sections of the wall apparently added to the flexibility and the structure's ability to tolerate differential settlements without being overstressed. The facing panels were intact and no signs of distress on the panels were noted. The wall was demolished several weeks following our site reconnaissance and no signs of tensile failures in connections or reinforcing strips were found.

Overall, the most notable observation was the relative lack of significant damage to the RE embankment despite being subjected to ground shaking levels higher than the design values and unanticipated large ground displacements. In stark contrast to this behavior, the conventional embankment at the southern approach suffered heavy damage and had been demolished when the reconnaissance team arrived. Also, a conventionally-constructed 10 m high approach embankment located about 250 m from the RE wall (see Fig. 2) suffered heavy damages during the earthquake, experiencing settlements of more than 1 m. The good performance of the RE embankment is thought to be particularly meaningful in demonstrating the seismic stability of conventionally-constructed walls of this type.

4 SIMPLIFIED SEISMIC ANALYSIS AND DYNAMIC DEFORMATION ANALYSIS

The Arifiye Overpass RE double-wall, which was constructed in 1988, was designed with a pseudo-static approach using a seismic coefficient $k_h = 0.1$ (Sankey & Segrestin 2001). To better interpret the performance of the RE walls and assess the adequacy of the seismic

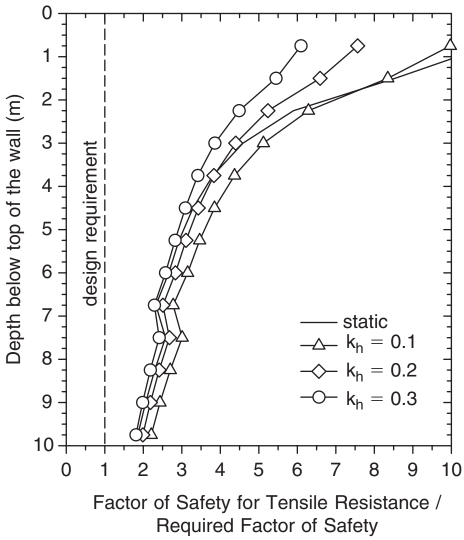

Figure 5. Ratio of the allowable tensile resistance (capacity) to that required for static and seismic design (demand).

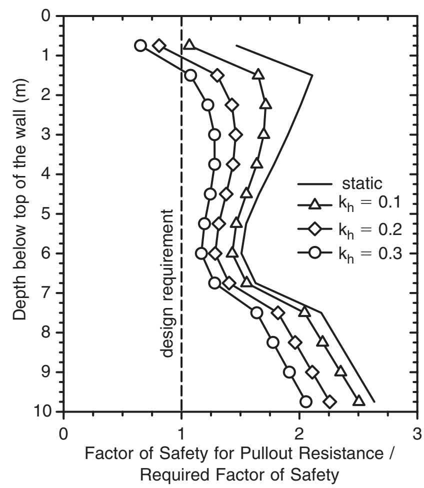

Figure 6. Ratio of the allowable pullout resistance (capacity) to that required for static and seismic design (demand).

design used, the authors performed a simplified seismic analysis as well as a numerical analysis to estimate the earthquake loading on the wall. As mentioned earlier, peak ground accelerations at the site were estimated at about 0.25 g during the M7.4 event.

For the simplified analysis, seismic loads on the RE walls, and the corresponding factors of safety for internal stability, were estimated using a pseudo-static approach where the earthquake-induced forces were represented via a horizontal seismic coefficient (k_h) as per the current FHWA design guidelines (Elias et al. 2001, AASHTO 2006). This approach provides a reasonable perspective for assessing the observed embankment performance, especially in that this approach is most often used in practice.

Simplified analyses were performed using three different seismic coefficients: $k_h = 0.1$, 0.2, and 0.3. The analyses were performed for $k_h = 0.1$ because this was the value used for design of the wall at the time. The value of 0.3 is the calculated "equivalent k_h" value that corresponds to the peak ground acceleration of 0.25g estimated to have occurred.

The calculated design forces for each case are compared with the allowable tensile resistance of the steel strips and the available pullout resistance at each level. Allowable tensile resistance is estimated as 55% of the tensile yield strength of steel ($\sigma_{\text{yield}} = 450$ MPa and $\sigma_{\text{allowable}} = 0.55 \cdot \sigma_{\text{yield}} = 247.5$ MPa). The design guidelines allow the use of 75% of the static factor of safety for seismic design (AASHTO 2006). In essence, this results in the allowable stress for seismic design as $0.73 \cdot \sigma_{\text{yield,steel}}$ (330 MPa). Allowable steel strength for static and seismic cases and the available reinforcement cross-section area per unit wall width were calculated at each elevation. Estimated design forces from the simplified approach and the allowable tensile resistance were calculated and comparison of these values are presented in Figure 5. It can be seen that the allowable capacity/demand ratio is well above 1.0, and is most critical at the lower reinforcement levels, ranging between 1.8–2.2 at the bottom reinforcement levels. The results suggest that the reinforcements should not have been stressed beyond their allowable strengths during shaking, but they were stressed much higher than the design values based on $k_h = 0.1$.

Design guidelines recommend the use of a minimum 1.5 for the factor of safety against pullout at each reinforcement level. Similar to the tensile capacity calculations, 75% of the static design requirement is allowed for the seismic design against pullout. The pullout resistance is calculated using the friction coefficient and the surface area of the reinforcement anchored to the passive zone defined by design guidelines. The friction coefficient used for seismic design is 80% of the value used for static design. The extra allowance by the reduced factor of safety is more-or-less offset by the use of a reduced frictional resistance for seismic design. Comparisons of the allowable pullout resistances required by design (pullout capacity/required factor of safety) to the estimated forces at each elevation are shown in Figure 6. Although not shown above, the actual pullout factor of safety ranges between 2.2 and 4.0 along the height of the wall for the static case where 1.5 is the required minimum for design. Therefore, the allowable-pullout-capacity to demand ratio varies between 1.5–2.7 for the static case as shown in the figure. The values of allowable

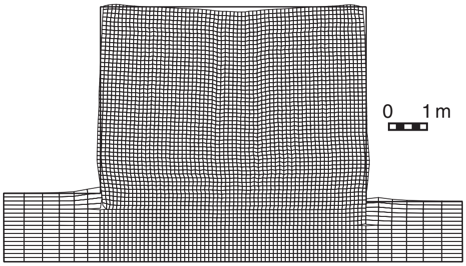

Figure 7. Deformed shape of the model wall (deformations at 1x, no exaggeration).

capacity-to-demand ratio drop slightly to 1.1–2.5 for a seismic coefficient of 0.1 which was used for design. Thus, seismic considerations probably did not govern the design of the wall and caused little or no increase to the reinforcement amount in comparison to the static case, as noted by Sankey and Segrestin (2001).

Values of allowable pullout capacity/demand ratio drop to a range of 0.8–2.3 for seismic coefficient of 0.2, and 0.7–2.1 for a seismic coefficient of 0.3. The values of allowable capacity to demand ratio for the reinforcements at the top 6 meters are only slightly above 1.0. The results suggests possible problems with pullout resistance at $k_h = 0.3$, the level suspected to have been induced during the earthquake.

In addition to the simplified seismic analysis, numerical analyses were also performed to better understand the seismic behavior of the structure (Olgun 2003). Using FLAC (Itasca 2000) with a two-dimensional analysis, we were successful in closely predicting the observed earthquake-induced deformation pattern and displacement magnitudes of the embankment. The deformed mesh shape from the seismic analyses is provided in Figure 7. For brevity purposes, the numerical analysis details are not provided, but the main findings are summarized:

1. A maximum earthquake-induced lateral wall displacement of 12–14 cm was predicted, compared to an observed value of 10-15 cm, and the predicted backfill settlement was 27 cm, consistent with the observed value of 25–30 cm.
2. The observed lateral wall bulging was due mainly to seismic shaking, whereas the backfill settlement and panel misalignment were due mainly to the differential ground distortions from liquefaction- and fault-related movements.
3. The earthquake-induced forces in the strips were on average 1.5–2 times larger than the design values. These higher than design values indicate that the reinforcements were most likely overstressed in the field during the earthquake.
4. Intolerable serviceability problems due to significant distortion of the structure, such as panel misalignment and significant backfill settlement, were likely to occur long before internal failure.
5. Consistent with the observations, the wall should have maintained its overall structural integrity despite the severe loading conditions it experienced. The numerical analysis added confidence to our interpretation of the field behavior and post-earthquake investigation of the structure.

5 SUMMARY AND CONCLUSIONS

The 1999 Kocaeli Earthquake (M7. 4) in northwestern Turkey provided the opportunity to study the field performance of a double-walled RE embankment designed according to procedures essentially same as those used for current FHWA guidelines and subjected to significant ground shaking and permanent ground deformations. The RE embankment formed the northern approach embankment for the Arifiye Bridge Overpass. The site was located immediately adjacent to the ruptured fault causing large horizontal and vertical ground movements within a few meters of the embankment. Unanticipated liquefaction-related settlements also occurred in the foundation, and the ground shaking levels that occurred were higher than those accounted for in design. The site was carefully documented and following the earthquake. Key observations and findings from the study are summarized as follows:

1. The structure was designed using guidelines similar to current FHWA Standards. Minimum FS values recommended for static and seismic conditions were maintained. Based on procedures in Turkey at the time, a seismic coefficient k_h of 0.1 was used for design of the RE walls and embankment. Liquefaction and/or fault-related movements in the foundation soils were not anticipated or designed for.
2. Ground shaking at the site was much higher than what was accounted for using the k_h design value of 0.1. Estimated peak ground acceleration was about 0.25 g, corresponding to an estimated "equivalent k_h" of about 0.3. Simplified and numerical analyses show reinforcement forces during shaking and pullout resistance greatly exceeded the design values.
3. In addition to strong ground shaking, the embankment was subjected to significant settlements and horizontal movements. Liquefaction and/or cyclic soil failure were responsible for differential foundation settlements, especially near the culvert that ran beneath the embankment- settlements of up to 25 cm occurred in this section. Also, the site was located immediately adjacent to the ruptured fault, as the fault rupture passed through the northern abutment and adjacent pier causing a lateral offset of more than 3.5 meters and a vertical offset of nearly 0.5 m.

4. Simplified and numerical analysis predicted that the steel reinforcements were probably not stressed beyond their yield strengths during shaking, but the induced forces in the strips were on average 1.5 times larger than the design values. Also, the pullout capacity may have been reached during the earthquake for the upper levels of the embankment.
5. Despite the high levels of shaking and unanticipated ground movements, the RE structure maintained overall integrity, exceeding the design provisions. (Although a peak acceleration of 0.25 g is estimated, it is also possible that the peak ground accelerations at the site were larger than 0.29 g which is the upper boundary of seismic loading where FHWA/AASHTO simplified design guidelines are applicable). The facing panels and reinforcements were undamaged, and if not for the ground movements, we suspect the RE embankment would probably not have suffered significant damage.
6. Numerical analyses predicted the observed lateral wall bulging was due mainly to seismic shaking, whereas the backfill settlement and panel misalignment were due mainly to the differential ground distortions from liquefaction- and fault-related movements. Also, intolerable serviceability problems due to significant distortion of the structure, such as panel misalignment and significant backfill settlement, were likely to occur long before internal failure. And, consistent with the observations, the embankment should have maintained its overall structural integrity despite the severe loading conditions experienced during the Kocaeli Earthquake.
7. Special slip joints were used to mitigate anticipated static differential settlements between different sections of the approach embankment (i.e., between the pile-supported abutment and RE embankment). The joints played important role in limiting damage associated with liquefaction-related foundation movements, allowing the wall to sustain significant differential foundation deformations without being overstressed.
8. In stark contrast to this behavior, two conventionally-constructed approach embankments located near the RE suffered heavy damages during the earthquake, experiencing settlements of more than 1 m. The good performance of the RE walls is thought to be particularly meaningful in demonstrating the seismic stability of conventionally-constructed walls of this type.
9. The study implies that well-designed conventional RE embankments constructed according to current design guidelines have an inherently high resistance to earthquake shaking and differential foundation movements. This performance is thought to be especially meaningful for illustrating the seismic resilience of these types of earth structures.

ACKNOWLEDGEMENTS

This research was performed under the auspices of the National Science Foundation awards CMS-0085281. and CMS-0201508. A major debt of gratitude is due to engineers and technicians at Zetas Earth Technology Corporation in Istanbul, Turkey. In particular, we are indebted to Dr. Canan Emrem who was instrumental in the data collection for this study and provided invaluable input. We also thank Mr. Murat Ozbatir, general manager of Reinforced Earth Corp.-Turkey (RECO) for kindly providing the design and construction details of the structures.

REFERENCES

AASHTO. 2006. LRFD Bridge Design Specifications. American Association of State Highway and Transportation Officials, Washington, D.C., 3rd Edition with 2005 and 2006 Interims.

Elias, V., Christopher, B.R. and Berg, R.R. 2001. Mechanically stabilized earth walls and reinforced soil slopes, design and construction guidelines. FHWA-NHI-00-043, Washington, D.C., 394 p.

Itasca Consulting Group. 2000. Fast Lagrangian Analysis of Continua, User Manual - Version 4.0. Itasca Consulting Group, Minneapolis, MN.

Martin, J.R., Olgun, C.G., Mitchell, J.K., Durgunoglu, H.T. 2004. High modulus columns for liquefaction mitigation. Journal of Geotechnical and Geoenvironmental Engineering, ASCE, vol. 130, no. 6, pp. 561–571.

Mitchell, J.K., Martin, J.R., Olgun, C.G., Emrem, C., Durgunoglu, H.T., Cetin, K.O. and Karadayilar, T. 2000. Chapter 9 - Performance of improved ground and earth structures. Earthquake Spectra, vol. 16, Supplement A to Volume 16, pp. 191–225.

Olgun, C.G. 2003. Performance of Improved Ground and Reinforced Soil Structures during Earthquakes - Case Studies and Numerical Analyses. PhD. Dissertation, Virginia Tech, Department of Civil and Environmental Engineering.

Sancio, R.B., Bray, J.D., Stewart, J.P., Youd, T.L., Durgunoglu, H.T., Onalp, A., Seed, R.B., Christensen, C., Baturay, M.B. and Karadayılar T. 2002. Correlation between ground failure and soil conditions in Adapazari, Turkey. Soil Dynamics and Earthquake Engineering, Volume 22, Issues 9–12, pp 1093–1102.

Safak, E. and Erdik, M. 2000. Chapter 5 – Recorded main shock and aftershock motions. Earthquake Spectra, vol. 16, Supplement A to Volume 16, pp. 97–112.

Sankey, J.E. and Segrestin, P. 2001. Evaluation of seismic performance in Mechanically Stabilized Earth structures. International Symposium on Earth Reinforcement Practice, IS Kyushu 2001, A.A. Balkema, K. Omine ed., November 14–16, 2001, Fukuoka, Kyushu, Japan, vol. 1, pp. 449–452.