Earthquake Performance of Reinforced-Earth Embankment Subjected to Strong Shaking and Ground Deformations

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Abstract: Reinforced-soil structures, such as mechanically-stabilized embankments and soil-nailed walls, have proven to be feasible alternatives to conventional earth retaining structures. Although design methods for static conditions are well established, there is still a gap in our knowledge regarding seismic performance of such structures. 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 RE embankment formed the northern approach for the Arifiye Bridge Overpass and was investigated following the earthquake. 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 0.25g, corresponding to an estimated "equivalent k_h" of about 0.3. Despite being subjected to ground motions that exceeded the design levels, the RE system performed well, suffering only minor damages related mainly to the ground deformations. Numerical analyses were also performed to better understand the seismic behavior of the structure. 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. This study is particularly meaningful in terms of illustrating the high seismic resilience of well-built structures of this type.

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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. There is a critical need for well-documented earthquake field performance case histories. 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. MSE structures generally performed well, except for cases of poor foundation soils, global instability, and/or inadequate static design (*1*). Of particular significance was the performance of an MSE approach embankment at the Arifiye Bridge Overpass. The northern embankment was constructed using Reinforced Earth (RE) technology; the general site location is shown in Figure 1. 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 0.25g for this site (*2*). 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, along with large liquefaction- and fault-related ground displacements, the RE system performed well, suffering only minor damage. As a stark comparison, two conventional earth embankments located nearby suffered heavy damage. This paper provides a description of the RE embankment, the seismic design of the structure, observed performance, and the analytical and numerical studies performed to better understand the seismic behavior.

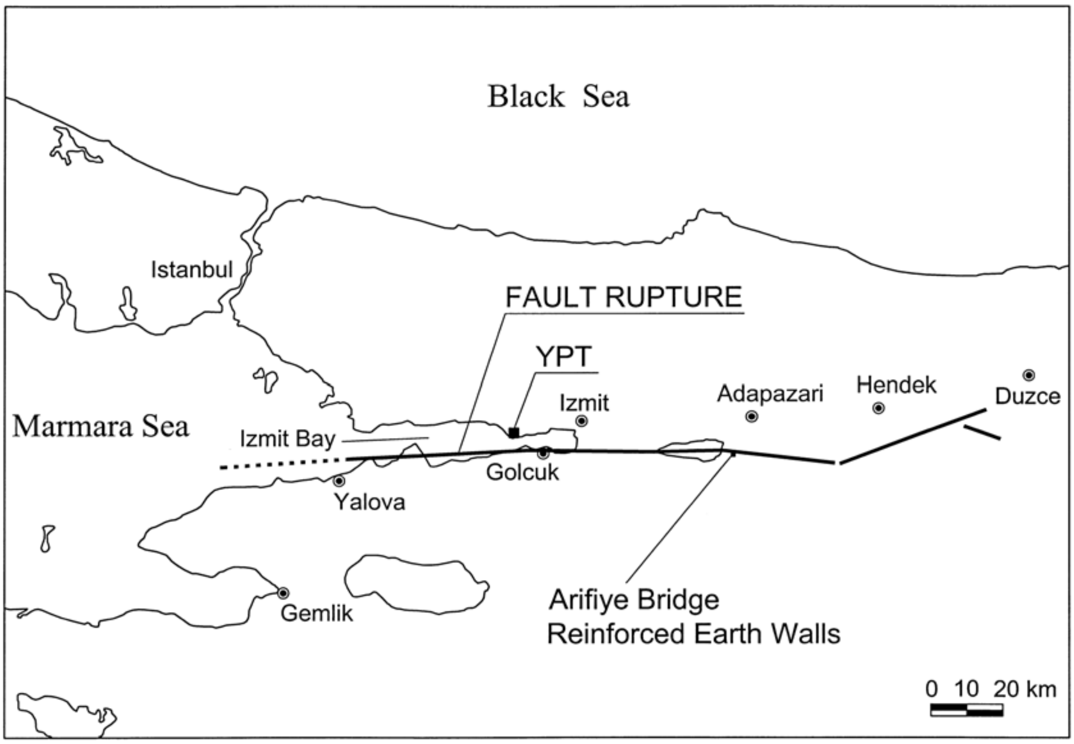

Figure 1 Setting of the August 17, 1999 Kocaeli Earthquake

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 the schematic 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 ramp was 10 m. The bridge deck rested on a reinforced concrete abutment supported by a pile foundation, as shown in Figure 3. 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. A cross-section of the maximum section of the RE embankment is shown in Figure 4. The facing panels were 150 cm x 150 cm in frontal area, and the reinforcing strips had a cross section of 40 mm x 5 mm. 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. The backfill soil was of high quality, consisting of sand and gravel that was compacted in lifts during wall construction.

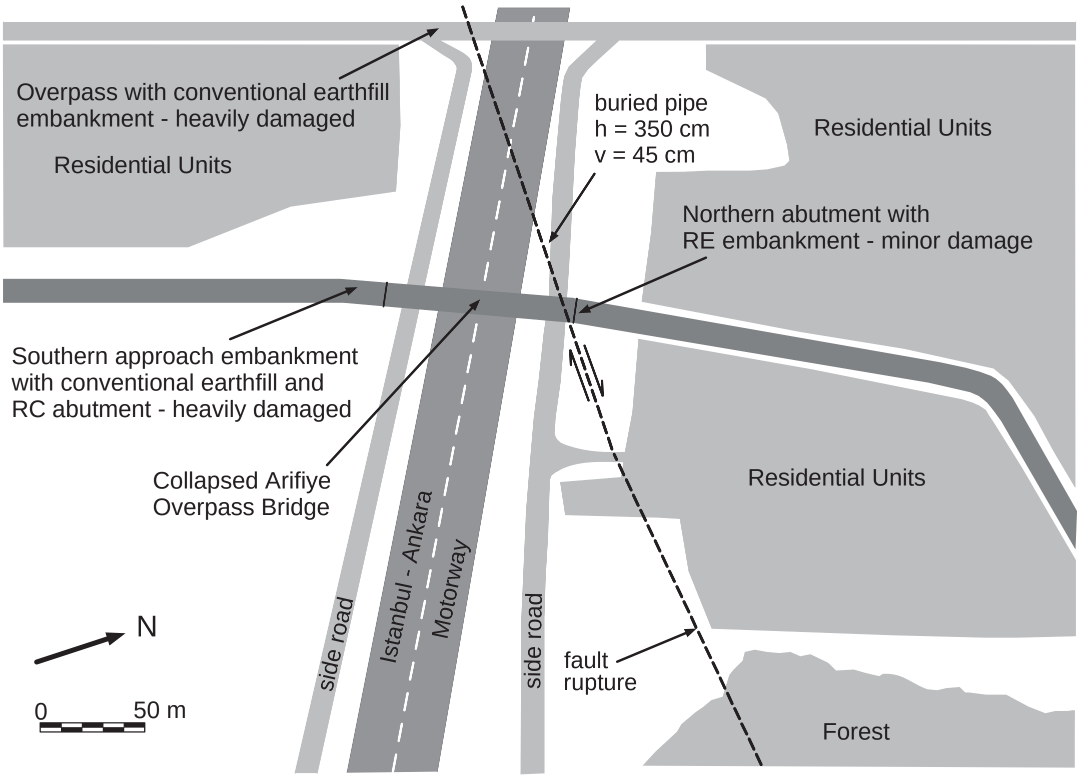

Figure 2 Plan view of Arifiye Overpass and fault rupture

As shown in Figure 3, a 4.8-m wide 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, note that “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 pile-founded 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.

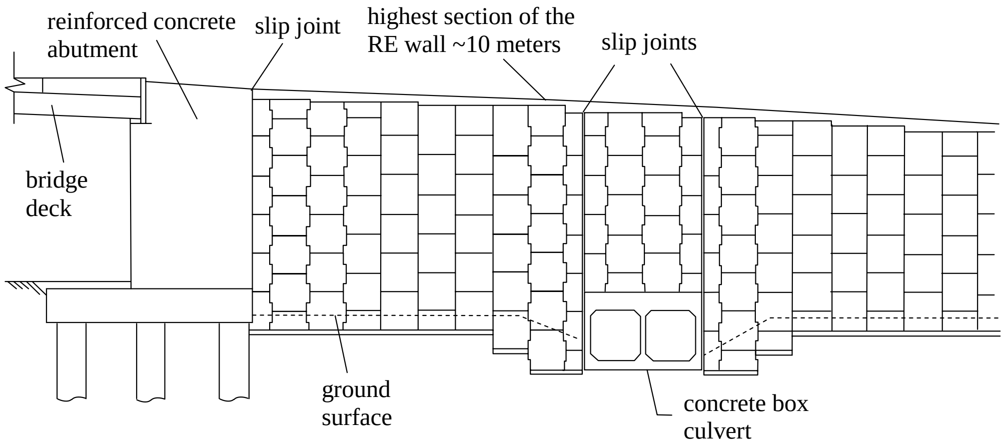

Figure 3 Front view of the Arifiye Overpass RE wall (northern approach embankment)

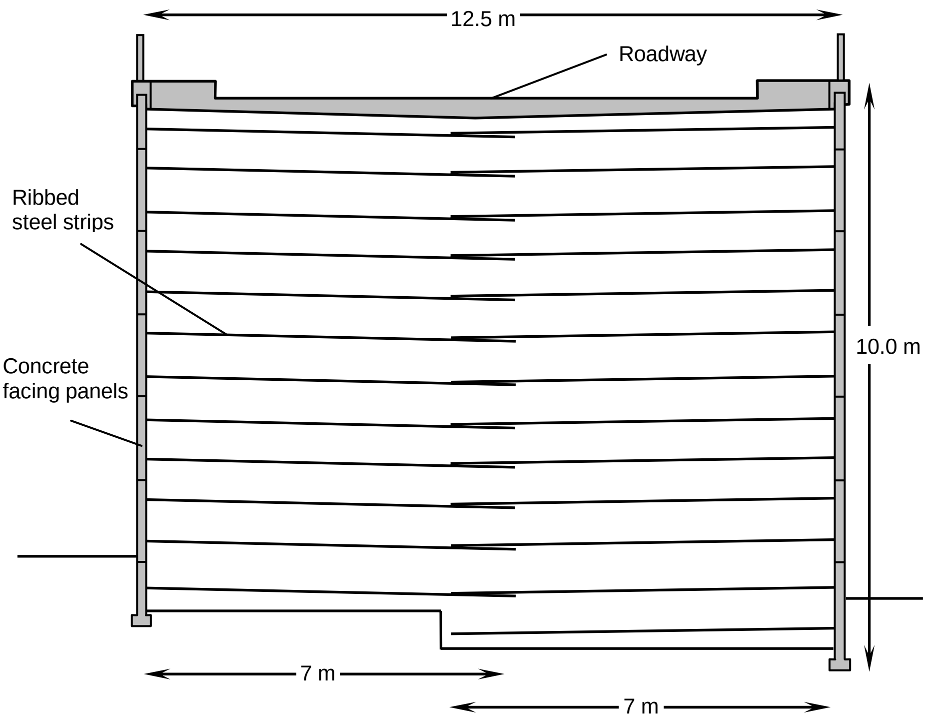

Figure 4 Cross-section view of the maximum section of the Arifiye RE embankment

SUBSOIL CONDITIONS

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. Following the earthquake, Virginia Tech personnel performed Cone Penetration Tests with shear wave velocity measurements (SCPT) at two locations along the northern approach embankment. CPT sounding and shear wave velocity measurements from one of the test locations are shown in Figure 5. The upper 5 m of the profile consists of 2 m of loose silty/clayey sand fill with average Q_c values of 2.5 MPa 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 values of normalized clean-sand-equivalent tip resistance ($q_{c1,CS}$) for the sand are 120. As shown later, 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 and extends from a depth of 5 m to 8 m. Normalized clean-sand-equivalent tip resistance ($q_{c1,CS}$) for this stratum is fairly uniform, averaging 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.

As shown in Figure 5, shear wave velocities average about 150 m/s throughout the 22-m deep profile, increasing from 70 m/s to 130 m/s within the first 10 m, and then quickly increasing to 150-200 m/s and remaining fairly uniform down to the explored depth of 22 m. The site classifies as NEHRP "F" due to the presence of liquefiable sediments; otherwise, based solely on the shear wave velocities, the site would classify as NEHRP "E" (*3*).

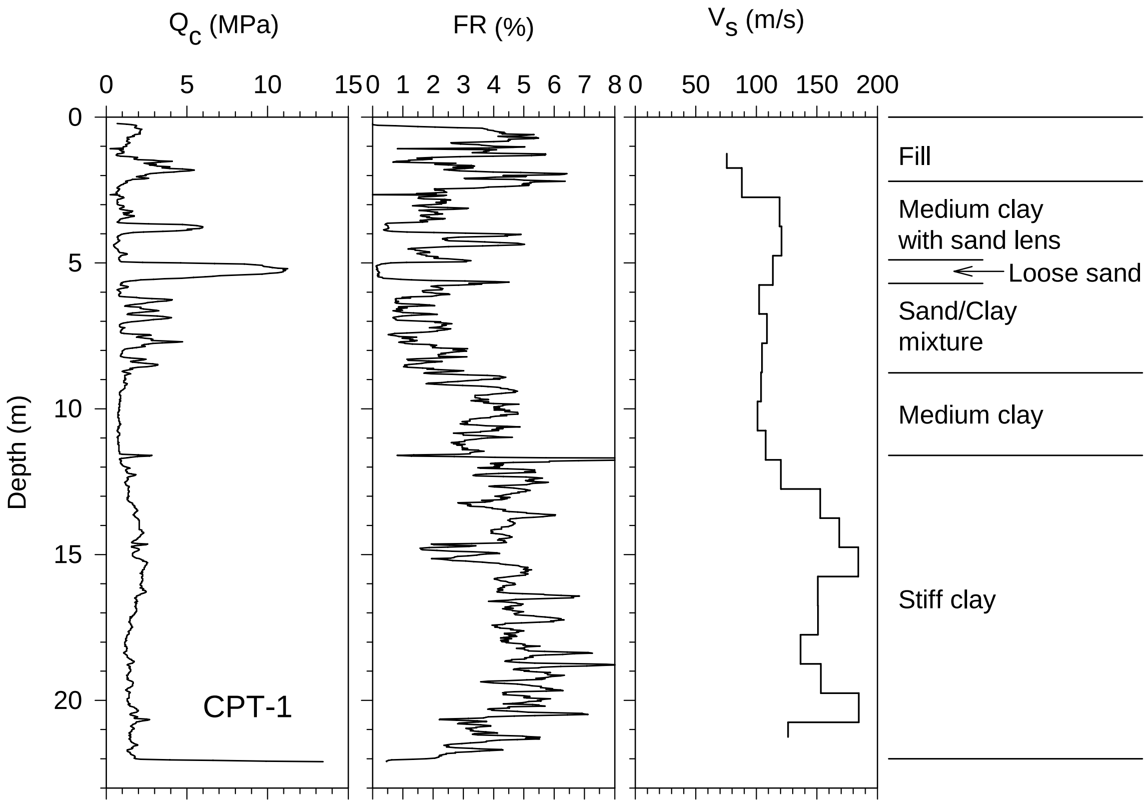

Figure 5 CPT sounding between the culvert and the reinforced concrete abutment (CPT-1)

RE EMBANKMENT SEISMIC DESIGN CONSIDERATIONS

At the time the RE walls were built at Arifiye (1988), the Turkish Earthquake Code required a pseudo-static horizontal seismic coefficient, k_h, of 0.1 be used for the seismic design of such structures (*4*). The design methodology of the code in Turkey is essentially the same as the current FHWA design procedures that utilize the pseudo-static limit equilibrium method for stability analysis of geosynthetic and metal-strip reinforced soil structures at sites where the anticipated ground accelerations are less than 0.29g (*5,6*). Accordingly, no rigorous seismic analysis was used for the wall design. The walls were designed in a simplified fashion using a pseudo-static seismic coefficient of 0.1 as required by the Turkish code, and the minimum factors of safety recommended by FHWA for both internal and external stability were used. Interestingly, as discussed later, this level of seismic coefficient resulted in little to no increase in required reinforcement compared to the static case. The actual estimated level of shaking that occurred at the wall corresponded to an “equivalent” k_h value much higher than 0.1, probably close to 0.3.

OBSERVED FIELD PERFORMANCE

Field reconnaissance at the Arifiye Overpass site was conducted following the August 1999 M7.4 earthquake (*1*). Peak ground acceleration at the site was in the range of 0.25g, close to that recorded at other near-fault sites underlain by similar soil conditions such as the YPT station. 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; see Figure 2. 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. The magnitude of horizontal movement that occurred directly beneath the RE embankment is unknown, but was probably significant.

In addition to fault-related ground deformations, foundation settlements of up to 25 cm were observed. The abutment and the damaged RE structure are shown in Figure 6, where ground cracks, settlements, and signs of distress on one side of the wall are visible. The resulting differential wall settlement caused the facing panels to become separated and misaligned, which allowed spillage of some backfill material, as shown in Figure 7. 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. A schematic illustrating the wall damage is shown in both side and plan view in Figure 8. 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. As illustrated in Figure 7, numerous ground cracks about 4 m long and running perpendicular to the wall were observed along 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 (*7,8*).

ground cracks

Figure 6 Northern approach embankment and damaged RE wall

Figure 7 Separation of the facing panels and spilled backfill material

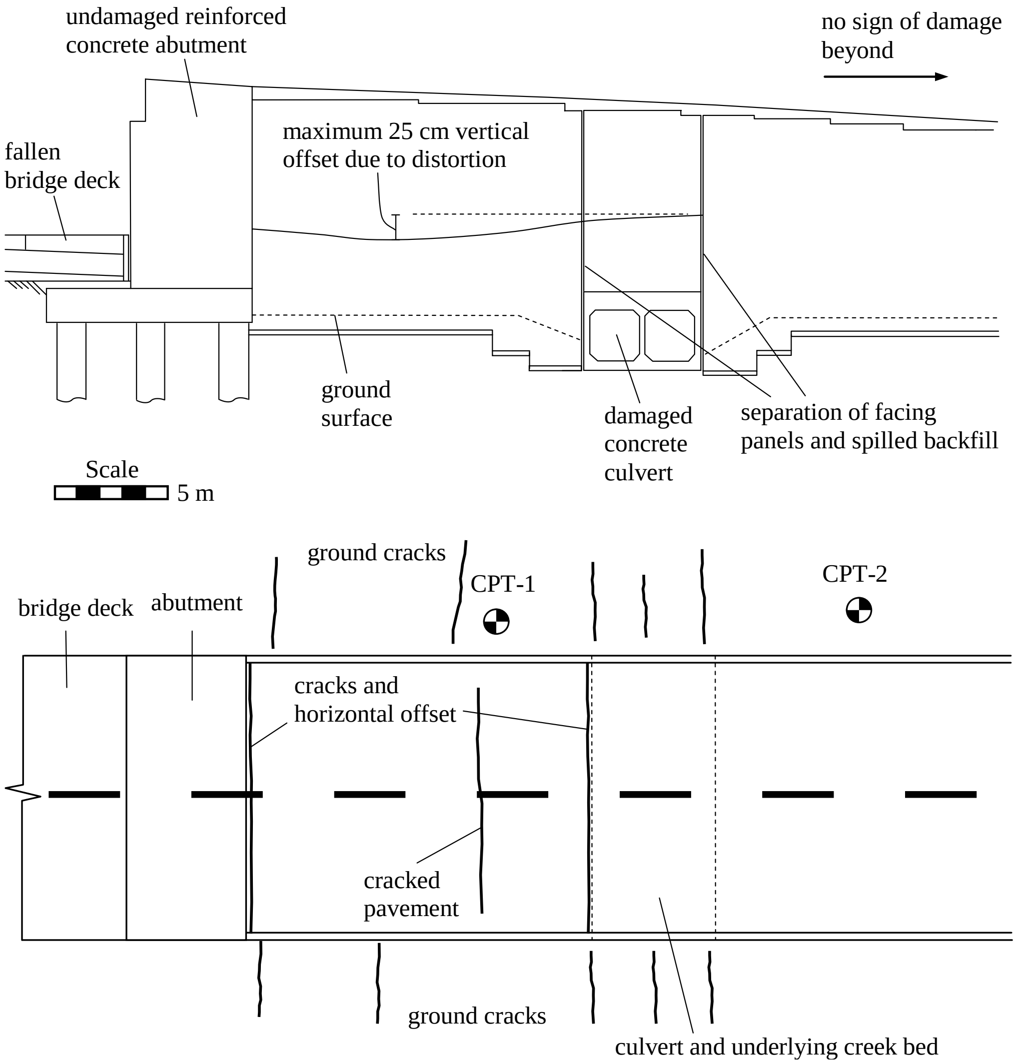

Figure 8 Damage pattern of the RE walls (front and plan views)

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 Figure 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.

SIMPLIFIED SEISMIC ANALYSIS

To better interpret the performance of the RE walls and assess the adequacy of the seismic design used, a simplified seismic analysis was performed to estimate the earthquake loading on the wall. As mentioned earlier, peak ground accelerations at the site were estimated at close to 0.25g during the M7.4 event. 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 (*5,6*).

A simplified analysis was used for this study because it is sufficient for providing perspective on the observed embankment performance, and because this approach is most often used in design. 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. The value of 0.3 is the calculated "equivalent k_h" value that corresponds to the estimated peak ground acceleration of 0.25g estimated to have actually occurred.

The use of a seismic coefficient of 0.1 results in an average reinforcement force increase of about 30% at the upper levels of the wall and 13% at the lower levels compared to the static case. A $k_h = 0.2$ corresponds to an average reinforcement force increase of nearly 50% near the top of the wall and about 25% at the bottom relative to the static case. A $k_h = 0.3$, results in an average reinforcement force increase of over 75% near the top of the wall and about 40% at the bottom compared to the static case. Using this approach, we estimate that the seismically-induced reinforcement forces were on average about 1.6 times higher than the static case and, most significantly, about 1.3 times higher than those assumed during the seismic design of the wall ($k_h = 0.1$).

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 ($\sigma_{yield} = 450$ MPa ≈ 65 ksi and $\sigma_{allowable} = 0.55\ \sigma_{yield} = 247.5$ MPa ≈ 36 ksi) and the available reinforcement cross-section area per unit wall width. Comparison of the available tensile resistances to the design forces and estimated actual forces are shown in Figure 9. It can be seen that the allowable capacity/demand ratio ranges from 1.9 - 11.0 along the height of the wall for the static case. These values fall to 1.7 – 7.5, 1.5 – 5.7, and 1.4 – 4.6 for conditions with seismic coefficients, k_h, of 0.1, 0.2, and 0.3, respectively. 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 $k_h = 0.1$.

The pullout resistance for seismic loading is calculated using the friction coefficient and the surface area of the reinforcement anchored to the passive zone defined by design guidelines. Design guidelines recommend the use of a minimum 1.5 for the factor of safety against pullout at each reinforcement level. Comparison of the pullout resistances to the design forces are shown in Figure 10. As seen, the static factor of safety ranges between 2.2 and 4.0 along the height of the wall for the static case. These values drop to 1.5 – 3.5 for a seismic coefficient of 0.1. The factors of safety drop to a range of 1.1 to 3.0 for a seismic coefficient of 0.2, and 0.9 to 2.9 for a seismic coefficient of 0.3. The results suggests possible stability problems with pullout resistance during the earthquake ($k_h = 0.3$) in the upper levels.

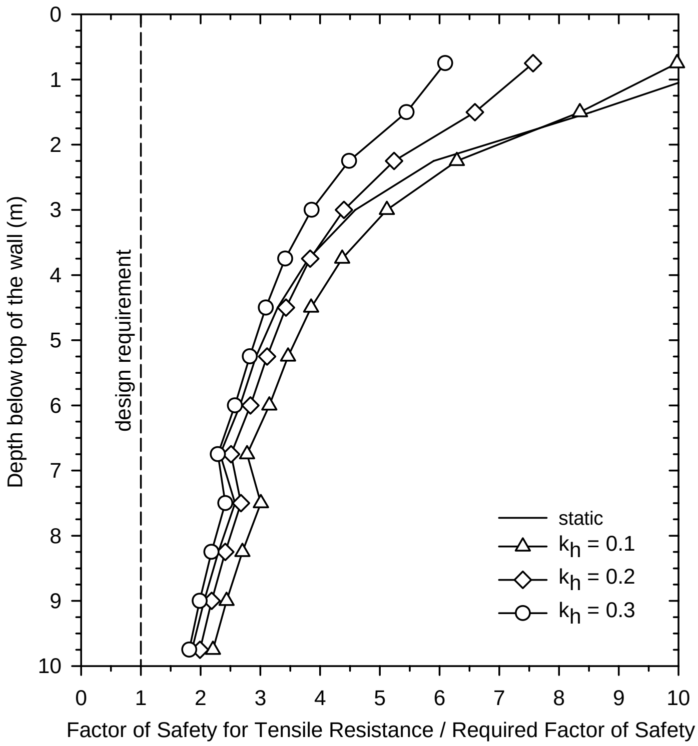

Figure 9 Tensile resistance - factor of safety for static and seismic design

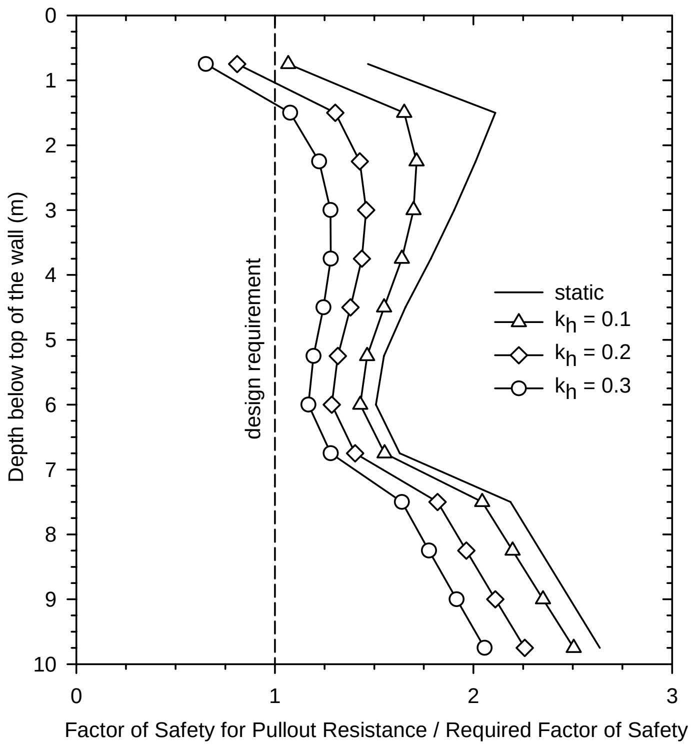

Figure 10 Pullout resistance - factor of safety for static and seismic design

LIQUEFACTION ANALYSIS

Because the wall suffered significant damages due to foundation settlement related to suspected liquefaction, a liquefaction analysis was conducted. For brevity purposes the full results are not presented here. This analysis found the soils had a moderate to high potential for liquefaction/cyclic failure under the loading of the Kocaeli Earthquake. The average FS against cyclic failure of soils in the upper 10 m was about 0.8, accounting for the presence of the overlying embankment.

DYNAMIC NUMERICAL MODELLING

In addition to the simplified seismic analysis, numerical analyses were also performed to better understand the seismic behavior of the structure (*2*). Using FLAC (*9*) 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 11. 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.) although the steel reinforcements were probably not stressed beyond their yield strengths, the earthquake-induced forces in the strips were on average 1.5 times larger than the design values; 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; and, 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.

SUMMARY AND CONCLUSIONS

The 1999 Kocaeli Earthquake (M7.4) in northwestern Turkey provided the opportunity to study the field performance of an 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 studied by Virginia Tech personnel 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.

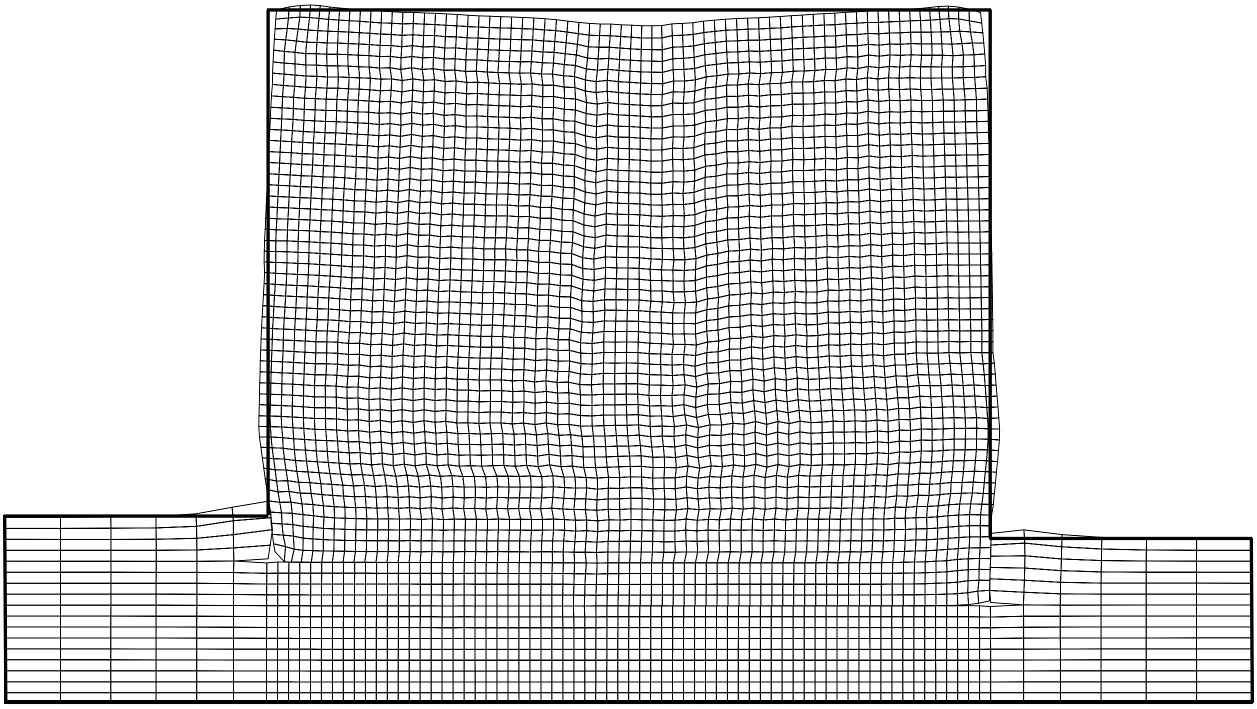

Figure 11 Deformed shape of the finite difference grid after shaking (deformations 1x - no exaggeration)

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 0.25g, 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. The magnitude of the fault-related movement that occurred directly beneath the RE embankment itself is unknown, but was probably significant.

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 PGA of 0.25g is estimated, it is also possible that the peak ground accelerations at the site were larger than 0.29g 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 FHWA 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 systems.

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