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Ahmet Pamuk
Rensselaer Polytechnic Institute, Troy, New York

Hoe I. Ling
Columbia University, New York, New York

Dov Leshchinsky
University of Delaware, Newark, Delaware

Erol Kalkan
University of California Davis, Davis, California

Korhan Adalier
Florida State University, Panama City, Florida

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Ahmet Pamuk
Rensselaer Polytechnic Institute
Troy, NY 12180
USA

Hoe I. Ling
Columbia University
New York, NY 10027
USA

Dov Leshchinsky
University of Delaware
Newark, DE 19716
USA

Erol Kalkan
Univ. of California Davis
Davis, CA 95616
USA

Korhan Adalier
Florida State University
Panama City, FL 32405
USA

ABSTRACT

A four-span bridge together with a 10 m-high and 100 m-long bridge approach fill was one of the highway facilities damaged due to surface faulting along the Trans-European Motorway during 1999 Kocaeli Earthquake (Mw 7.4). The fault rupture crossed beneath an overpass bridge within a few meters away from the bridge abutment while damaging the approach fill which was reinforced with a double-faced mechanically stabilized earth wall (MSEW) system. The faulting-induced excessive tectonic ground deformations including near-field seismic shakings were the main sources of damage in the walls. Such effects, along with the others, caused cracks and panel separations in wall faces as a result of a liquefaction-induced differential settlement in the cross section. The performance of the reinforced walls was satisfactory that there was no significant structural damage despite the total collapse of the bridge decks. The wall system provided a unique case history under extreme loading conditions, while proving that they are flexible and can withstand large ground deformations. This paper discusses how the walls performed based on post-earthquake reconnaissance studies. The faulting activity, geology of the site, strong ground motions and damage states in the reinforced wall are discussed in details.

INTRODUCTION

In 1999, Turkey was struck by a destructive earthquake that occurred on the western extension of 1500 km-long North Anatolian Fault (NAF) which resembles the San Andreas fault in California in many ways. The earthquake hit the most densely populated urban environments, namely Kocaeli and Sakarya provinces, situated on an alluvial fan at the western part of the NAF with magnitude (M_W) 7.4. This was one of the largest seismic events in the eastern Mediterranean basin in the last century causing substantial structural damage, casualties and economic loss. It also provided some of the most extensive strong ground motion data set ever recorded in Turkey within about 130 km of the surface fault rupture. Its impact on transportation infrastructures as well as on highly populated urban areas attracted the attention of many engineers and researchers worldwide. Initial reconnaissance efforts, including geotechnical observations in the earthquake-affected area were given by Ansal et al. (1999).

Figure 1a displays the intensity map of the northwestern Turkey combined with recorded peak horizontal ground accelerations (PHGA) in the disaster belt. The earthquake was felt well beyond the epicenter. Serious damage extended even to southwestern suburbs of metropolitan Istanbul, 120 km west of the epicenter. A track of the surface fault is also marked in this figure. Surface faulting started at the eastern end of the Marmara Sea, and then propagated eastward through the Adapazari region, while damaging the transportation infrastructure such as viaducts, bridges, bridge approaches and roadways. This was especially true along the Trans-European Motorway (TEM), a four-lane divided expressway between Istanbul and Ankara (Capital City), that had operated for about 10 years before the Kocaeli earthquake. The surface fault intersected the TEM at several locations westward from town of Arifiye, while being almost parallel to the highway system. Between the epicenter and Adapazari region, the slip displacement (i.e., lateral offset) averaged 2 to 3 m, with a maximum of 5.1 m (USGS, 2000) in the vicinity of Arifiye.

During the earthquake, the majority of the highway facilities performed well, except that the bridge overpass in Arifiye (Fig.2), located less than 50 km eastward of the epicenter, collapsed due to tectonic movement along the fault zone. The surface fault rupture passed beneath the northernmost span of the overpass while causing substantial surface deformations. The collapsed bridge blocked the TEM and caused serious delays in transporting immediate disaster-caused emergency needs in the epicentral area. Moreover, nearly a dozen people died when a passenger bus crashed into a collapsed deck while passing under the overpass.

Beyond the serious collapse of the bridge decks, the northern bridge approach fill (or ramp) that was reinforced with a pair of MSEW systems was also damaged mainly due to the excessive tectonic movement along the fault zone during the mainshock of

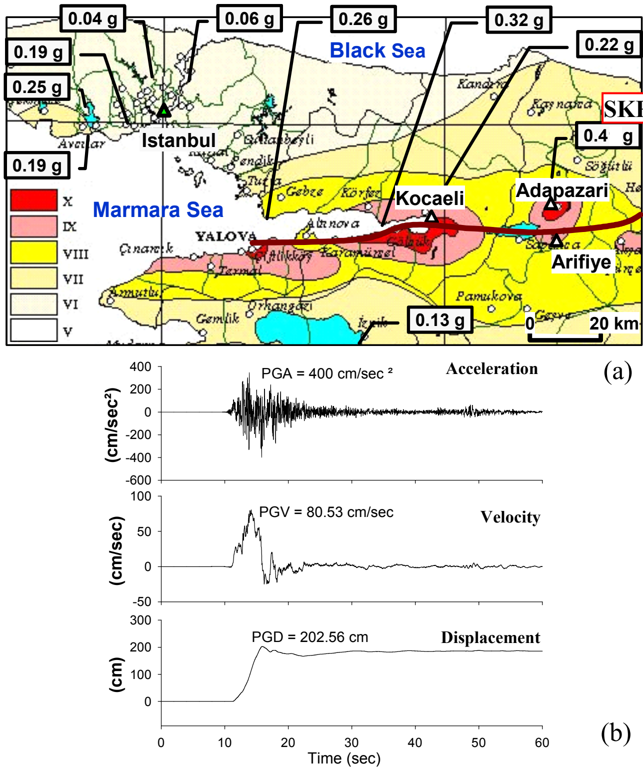


Fig 1. (a) Modified Mercalli intensities along the fault rupture and recorded PHGA's (ERD, 2000; USGS 2000; KOERI, 2002); (b) accelerograms recordings at Adapazari station (SKR) and calculated velocity and displacement time histories.

the earthquake. Major damage to the walls was not from seismic design, but a combination of adverse effects by the nearby fault movement and, possibly, bearing capacity problems associated with underlying foundation soil. Figure 2 depicts the overpass and the reinforced walls before and after the damage, respectively.

A detailed post-earthquake investigation revealed moderate-to-significant damage at about 80 locations along the TEM. Preliminary repair cost for all facilities along the motorway was estimated at about $40 million. Approximately 3.7 percent of this was for the demolition and reconstruction of the severely damaged Arifiye Overpass and its reinforced earth abutment system (GDH, 1999).

THE HISTORY OF DAMAGED MSEW AND COLLAPSED BRIDGE OVERPASS AT ARIFIYE

The four-span, two-lane, and 104 m-long bridge overpass was located in the town of Arifiye, less than 10 km south of the city of Adapazari (Fig. 1), and along a state highway (D-650) to cross the TEM as a skewed bridge as shown in Fig. 2. It was built in late 1980's in accordance with AASHTO Standards Specifications for Highway Bridges. After the construction of the overpass bridge, the 10 m-high bridge approach fill with a double faced MSEW system was installed because of space restrictions to accommodate minor roadways on both sides (Oztoprak, 1999). Both structures were in service until damaged by the earthquake.

A field test, conducted before construction of the overpass and walls, revealed very compressible sedimentary soil deposits at the bridge site with SPT-N blow counts averaging 1 to 4 for the first 15 m from the ground surface (Smith and Unal, 1992). This indicated that a deep foundation system was required to improve the bearing capacity as well as to reduce excessive settlements for the bridge abutment. Accordingly, the foundation of the northern reinforced concrete abutment was supported by a large pile-cap system (Fig. 2). The cap consisted of 16 concrete cast-in-place piles with a diameter of 1.2 m, extending to the depth of 48-50 m where a denser soil deposit lies below the weak soil deposits. The MSEW approach ramp for the overpass was then installed adjacent to the abutment.

The initial design of the approach was to construct a bridge system consisting of decks with supporting piers along its length in place of the MSEW approach fill. However, this design was

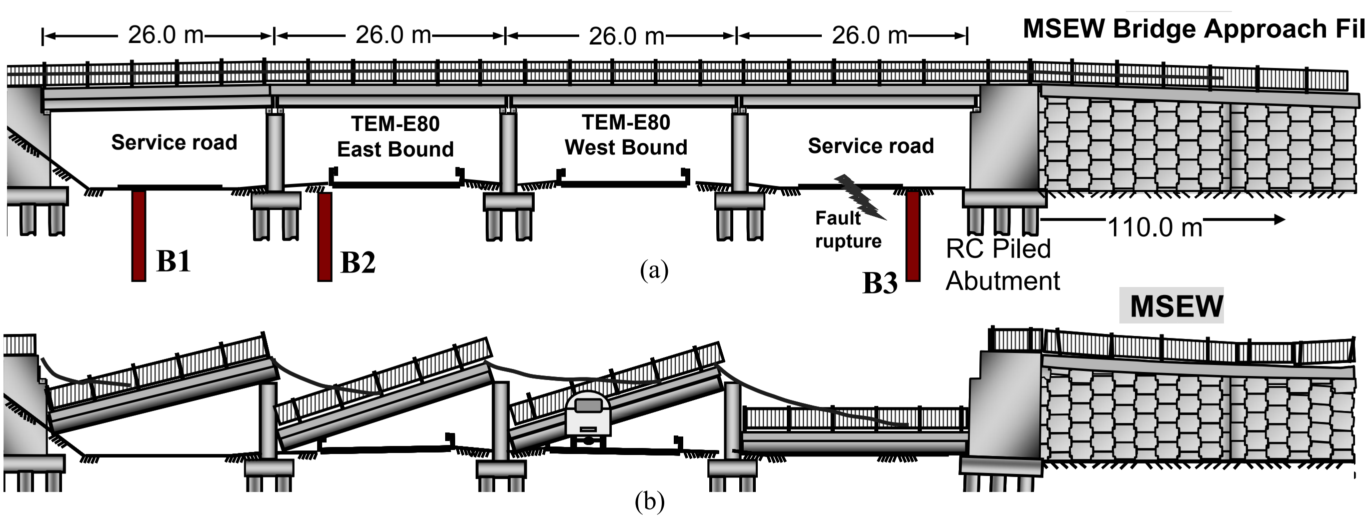


Fig. 2. The bridge overpass at Arifiye and mechanical stabilized bridge approach fill walls (a) before, and (b) after the earthquake.

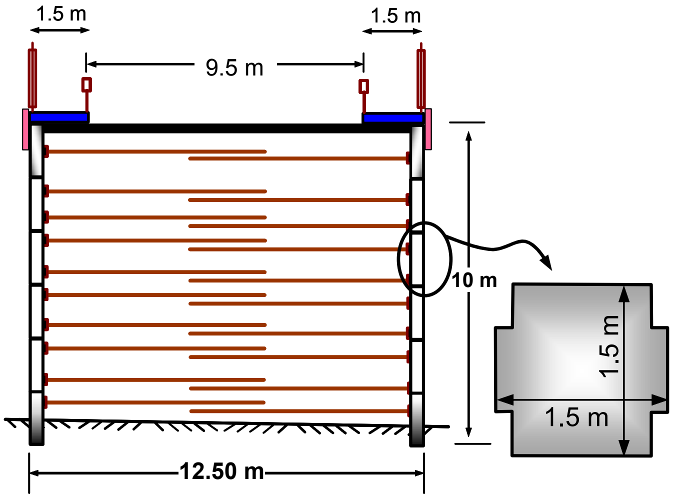


Fig 3. Schematics of double-faced gravity-type MSEW system with reinforced concrete facing panels in Arifiye.

found to be costly as it also required a pile-cap system for each pier. Thereafter, a gravity-type retaining wall system (i.e., Reinforced Earth®) was directly constructed on the ground. This reinforced wall system was selected not only as an economical construction method but also because it could be an effective method for withstanding large consolidation deformations from the compressible poor quality foundation soil (Smith and Unal, 1992). A 100 m-long MSEW system was built as a "double-faced" or "back-to-back" type wall, having parallel reinforced concrete facings with ripped metallic reinforcing inclusions to accommodate a two-way divided roadway as shown in Fig. 3.

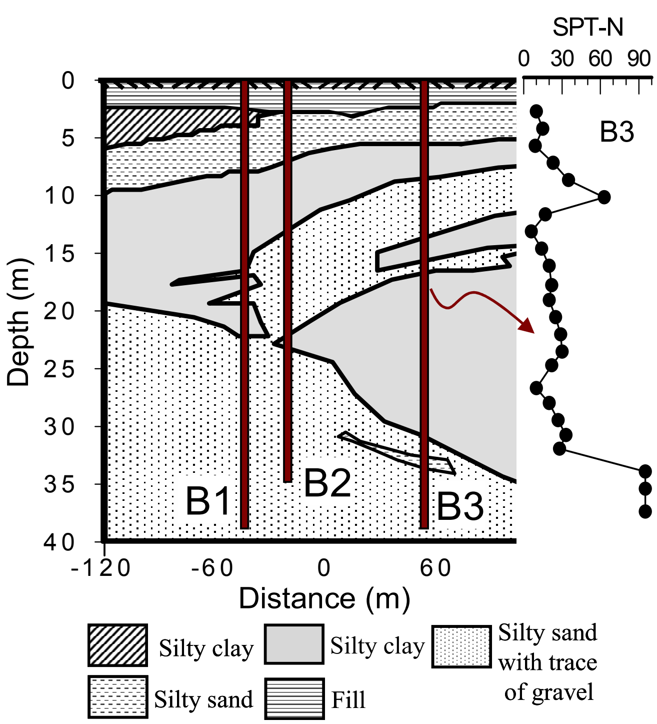


Fig 4. An approximate visualization of subsoil geology along the axis of Arifiye Overpass (modified based on ITU, 1999).

A reinforced concrete culvert was designed beneath the approach ramp possibly to facilitate storm or flood water discharge in approach ramp area (Fig. 5b). However, the culvert was not useful as it appeared inactive due to road fill at both sides of the approach ramp. Two slip joints (S1 and S2, Fig. 5) were also designed on each wall face on top of the rigid culvert to protect the walls damaged from differential settlement.

Despite the fact that the construction site for the reinforced wall system consisted of undesirable alluvial subsoil layers that were prone to significant seismic hazards, there was no special foundation preparation, such as removal or pre-consolidation of site soil against consolidation settlement, or any other ground remedial measures against earthquake-induced ground failures. The approach fill settled for about 40 cm during the construction period, followed by an additional settlement of 20 cm during post-construction monitoring period (Smith and Unal, 1992). These measured settlements were quite large compared to the design limitations, which constrain the individual differential settlements of a facing panel up to 1% of the sufficient joint width (<20 mm) along or perpendicular to the wall face (Elias and Christopher, 1997). Accordingly, the vertical settlements resulting from consolidation of the underlying soil was accommodated by the flexible joints of the facing panels without causing a major serviceability problem.

GEOLOGY AND SOIL CONDITIONS

The entire Adapazari region (Fig. 1a) is located in a large valley covered by alluvium deposits from a nearby lake and surrounding rivers. Soil deposition extends about 45 km long east to west, and 30 km long north to south with a varying thickness of more than 200 m-deep (SU, 2002). The geology of the bridge site in Arifiye is dominated by Pliocene to Pleistocene sedimentary rocks (i.e., dense soil deposits) which lie at least 50 m below the younger sedimentary deposits (ITU, 1999).

Standard penetration tests (SPT) were conducted by the Turkish General Directorate of Highways (GDH) to gain sufficient subsurface information between both ends of the bridge overpass soon after the earthquake. Three subsurface borings to a depth of 38 m were drilled. Their locations are shown as B1, B2 and B3 in Fig. 2a. A 2-D visualization for the local subsoil conditions along the axis of the bridge overpass is shown in Fig. 4. The ground water table was approximately 5 m below the ground surface.

B1 shows that very soft layers of soil deposits lie under the southern abutment and extend to a depth of 22 m where a dense (N_{30}=100) layer of sedimentary deposit (i.e., Pliocene base rock) of silty sand with some gravel was encountered. The loose layers became thicker to the depth of 34 m below the northern abutment as shown in B3, which was the nearest boring to the MSE walls. The soil profile at B3 consisted of a 2.5m-thick fill followed by varying thicknesses of silty sand and silty clay deposits. Loose silty sand and silty clay layers (with N_{30}< 20, Fig. 4) below the reinforced walls might have been prone to liquefaction or seismic-induced densification during the seismic event. These SPT measurements obtained after the earthquake might reflect denser states of the soil layers than those prior to the earthquake.

NEAR-FIELD EFFECTS AT BRIDGE SITE

The closest recording station to the Arifiye Bridge was Sakarya station (SKR), located between downtown Adapazari and Arifiye, for about 4 km northward from the bridge site. The largest peak horizontal ground acceleration of about 0.4g (EW direction), and peak vertical ground acceleration of 0.26g were recorded at this station during the main shock of the Kocaeli earthquake. Due to the malfunction of the transducer at Sakarya station, the NS component of motion could not be recorded. EW direction recorded acceleration and its computed velocity and displacement time-histories are presented in Fig. 1b. A clear evidence of impulsive motion (i.e., fling) can be observed from the velocity and displacement curves of this figure. It is also noteworthy that Sakarya (SKR) station was founded on a stiff soil site, whereas the bridge was located on soft soil. Thus, one may expect that the actual accelerations at this site would be even higher than what was measured at Sakarya station due to site amplification. But, intriguingly, no structural collapse or serious damage was observed on the neighboring residential units (at both sides of the surface fault) in the vicinity of the MSEW system in Arifiye. On the other hand, the structural damage gradually increased northward where it became most destructive in the center of Adapazari, located on a soft soil site.

Due to this paradigm and sparsely located strong motion transducers in the epicenter area, it is not possible to draw accurately the isoseismic map of peak ground acceleration at Arifiye. Thus, we refrain from estimating the probable peak ground acceleration at the site of MSEW based on the weak evidences. Rather than PGA at the site of interest, it is our contention that unseating of the bridge decks and their collapse as well as the damaged walls of the reinforced approach fill were the result of the static displacements due to the fault traversing the bridge and its associated strong near-field effects. Particularly, surface fault rupturing may cause an instantaneous energy demand and result in strong velocity and displacement pulses that force the structures (in the immediate vicinity of the rupture) to release such an energy with few cycles of plastic displacement excursions. The observed damage to the bridge, especially unseating of girders, conveys this conclusion, and emphasizes the detrimental consequences of near-source site effects typically observed in several places during the recent Turkish earthquakes. This issue is further discussed in details by Kalkan et al., (2004).

DAMAGE DETAILS IN MSEW BRIDGE RAMP

Field observations revealed that there were a number of factors that caused damage in the MSEW system in Arifiye. These are (i) large tectonic movements along the main fault line, (ii) presence of a drainage culvert, (iii) strong near-field shaking, and possibly, (iv) cyclic-induced soil densification and settlement.

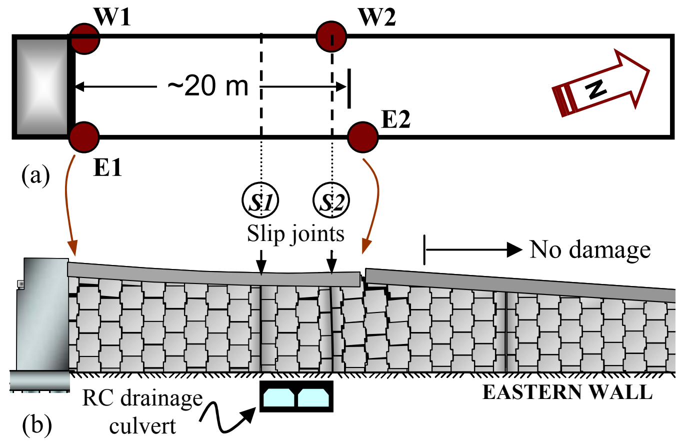
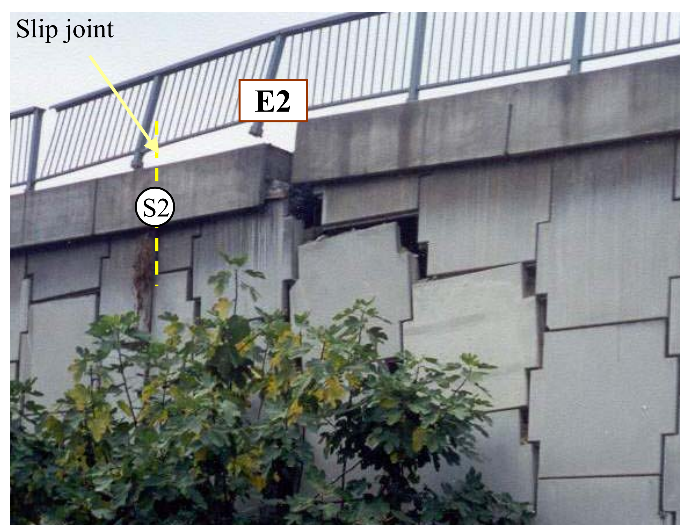


Fig 5. MSE bridge approach (a) plan view of approach fill with damage-concentration; (b) schematics of eastern wall after the earthquake.

(a)



(b)

Fig 6. Damage details for (a) E1 and (b) E2 on eastern MSEW face (photos after Ozbakir).

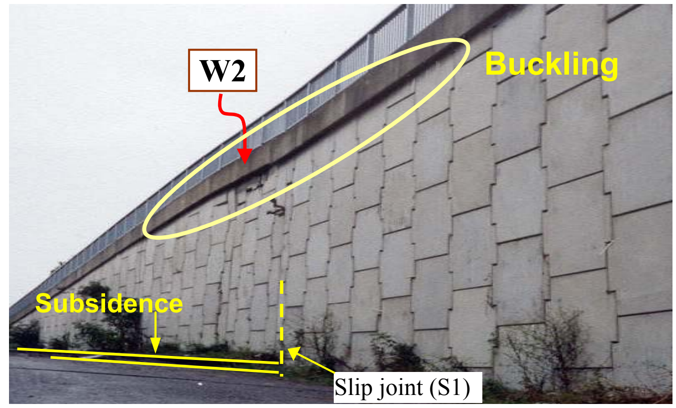
Only a limited section of the MSEW approach ramp was damaged to a great extent in between the bridge abutment and RC culvert. This section constituted approximately 20% of the entire length of the reinforced earth structure. The most damage-affected locations within this section along the eastern and western wall faces are highlighted in Fig. 5a as E1 and E2, and W1 and W2, respectively, whilst their detailed views are presented in Figs. 6 and 7.

Right-lateral strike-slip fault rupture along the main fault line passed under the northern span of the bridge (Fig. 2a) with large transverse and vertical displacements of approximately 3.5m and 0.5m (e.g., JSCE, 1999; ITU, 1999), respectively. Among these, the vertical ground deformation (or subsidence) appeared to be the main source of the damage state in the MSE walls of the approach ramp. The subsidence on the main fault rupture extended beneath the ramp, for about 20 m north from the bridge abutment where the RC culvert was located under the walls (Figs. 5 and 7). Cracks due to subsidence were clearly observed on asphalt-covered side roads, especially on the western side of the ramp (Fig. 7). It should also be pointed out that the final permanent ground deformation in this section may possibly include cyclic-induced settlement due to soil densification in addition to the subsidence from the fault rupture. However, the undamaged section of the wall did not exhibit any settlement due to earthquake shaking, indicating that the majority of the ground failure under the MSEW was from the nearby tectonic activity.

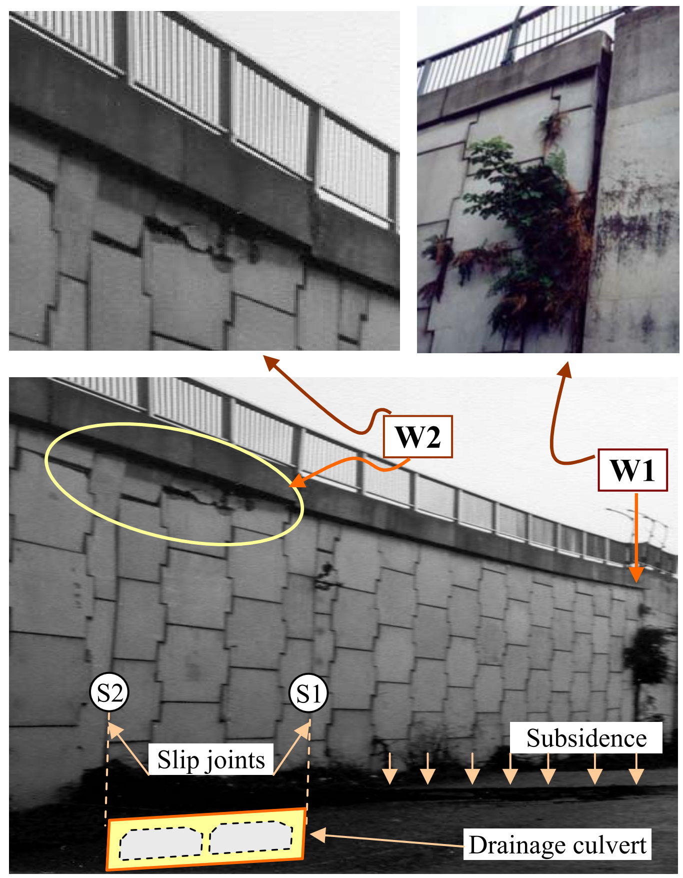
The greatest disturbance in the wall faces was concentrated at higher elevations above the culvert (at E2 and W2). Because the vertical displacement at E2 was larger than W2, the approach ramp tilted eastward in the cross section. That is, the ramp deformed in the horizontal direction as if an external force was applied perpendicularly to W2 and pushed the ramp eastward. This behavior (i.e., tilting) was most probably due to the presence of the rigid culvert which prevented interaction between the ramp and its foundation, therefore, the walls could not accommodate the underlying ground deformations that was induced by the fault rupture.

The tilting in the cross section resulted in different damage states above the culvert at E2 and W2 such that the western wall buckled in the vicinity of W2 (Fig. 7a), whereas the eastern wall face were stretched outward (Fig. 6). The buckled side increased compression on the facing panels at W2, whilst crashing and forcing the panels displaced (Fig. 7b). On the other hand, the largest damage in the reinforced walls was observed at E2 as shown Fig. 6b. At this location, the wall displaced both vertically and horizontally for about 25-30 cm. The displacements at this locality was so large that they exceeded the allowable design limitations for an independent panel movement. Thus, the panels could not accommodate the ground deformation, and finally, large panel separations and cracks (especially at lower elevations of the wall) occurred. However, the facing panel connections with the metallic reinforcements did not fail, and their flexible joints allowed large displacements and differential settlements.

At E1 (Fig. 6a) and W1 (Fig. 7b), the facing panels interacted with the pile supported bridge abutment. The damage states at



(a)



(b)

Fig 7. Western MSEW face: (a) buckled wall, looking from south to north; (b) damage details at W1 and W2, looking from north to south (photos after Ozbakir).

both locations were also different. At E1, the vertical ground deformation was so large that the flexible wall face was forced to be displaced both vertically and longitudinally. However, the movement in the longitudinal direction was greatly prevented by the rigid abutment. This caused large panel separations and cracks at the higher levels (Fig. 6a), but no damage observed at lower wall elevations.

At W2, the vertical ground deformation was not appreciably large compared to E1. On the other hand, a gap of about 10 cm occurred between the panels and the abutment as shown in Fig. 7b. This gap appeared to be resulted from the buckling in the same wall face in vicinity of W2 (Fig. 7a). That is, once the wall buckled at W2, the wall face was pulled longitudinally (northwardly) as a whole. This behavior along the western side of the approach ramp did not cause any damage in the facing panels between W1 and W2. This was an interesting observation indicating that the reinforced wall system was very flexible.

The near-field shaking effects at the site of interest was previously discussed in detail. The fact that there was no damage observed at most part of the walls along the length of the ramp (Fig. 5), it can be speculated that the shaking alone did not appreciably contribute a major source of the damage. However, the near-field effects might have increased the level of the observed structural displacement response due to its strong velocity and displacement pulses. The shaking may also contribute some additional settlement as previously discussed.

CONCLUSIONS

This paper summarizes the structural behavior and damage details of the double-faced MSEW system in Arifiye after the 1999 Kocaeli earthquake. The structure provided a unique full-scale field test of the reinforced soil structure under extreme loading conditions. That is, the wall system is the first one ever subjected to a significant near ground motions and deformations. The field observations indicated that the faulting-induced ground deformations remained as the main source of damage in the MSEW. Panel cracks and separations in wall faces were observed at certain location. The overall performance of the reinforced walls was satisfactory. That is, the internal stability (e.g., pullout, tensile and connection failure) and external stability (e.g., sliding, overturning, deep seated stability) of the wall system was satisfactory. The wall system proved that they are flexible and can withstand large deformations.

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