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Structural and geotechnical impacts of surface rupture on highway structures during recent earthquakes in Turkey

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Abstract

The most significant damage on highway bridges during the recent earthquakes in Turkey (Kocaeli and Duzce earthquakes) and Taiwan (Chi–Chi earthquake) was the result of fault ruptures traversing transportation infrastructure. This phenomenon and its consequences accentuate the need to examine surface rupture hazards and to identify those areas at risk. This understanding can help to develop remedial measures for both structural and geotechnical engineering. For that purpose, damage to highway bridges during the recent events was reviewed. The total collapse of the highway overpass in Arifiye, during the Kocaeli earthquake, was investigated. The major problems under consideration (in Arifiye) were: (i) dislodging of the bridge spans, and consequently, the total separation of the reinforced concrete girders from the piers; and (ii) the stability of a mechanically stabilized earth wall (MSEW) system under extreme loading conditions. The results of the structural and geotechnical investigations presented herein can be taken in consideration to improve transportation infrastructure against surface rupture hazards.

Keywords: Bridge; Kocaeli earthquake; Duzce earthquake; Seismic hazard; Retrofitting; Fault rupture; Arifiye Overpass; Mechanically stabilized earth wall

1. Introduction

Over the past 50 years, revolutionary developments have taken place in the design and construction of transportation facilities against seismic hazards. However, the earthquakes that occurred in 1999 in Turkey (M_w 7.4 Kocaeli and M_w 7.2 Duzce) and Taiwan (M_w 7.6 Chi–Chi) were opportunities which revealed the adverse consequences of near-fault site effects on transportation infrastructure, particularly surface fault rupture hazards. During the devastating Chi–Chi earthquake, a reverse-slip fault caused ground displacements of up to 3 and 9 m in the horizontal and vertical directions. Coupled strong shaking with surface faulting caused light-to-severe damage to more than 700 highway bridges. Almost a dozen of these were totally or partially collapsed due to direct or indirect interactions with excessive ground movement induced by the surface faulting [1]. The Bei-Feng Bridge (Fig. 1(a)) experienced a vertical ground movement of about 5–6 m which resulted in the collapse of its three spans. Similar damage was observed in a skewed bridge, the Shi-Wei Bridge (Fig. 1(b)), when the fault imposed large deformations on the southern abutment [2,4]. These are the typical examples of surface ruptures causing detrimental effects on highway facilities. During Turkey's earthquakes, several bridges were damaged significantly due to similar phenomena.

Recent earthquakes revealed that fault ruptures passing beneath or close to the foundation of transportation infrastructure were common source of damage even though most of the bridges and freeway viaducts were designed and constructed under modern seismic provisions. The veiling of this phenomenon in current design standards and practical engineering applications is the motivation for identifying the reasons causing damage both structurally and geotechnically. For that purpose, a totally collapsed highway overpass in Arifiye (in Turkey) was investigated in detail, while similar damage in other highway bridges affected by the strong shakings of the Kocaeli and Duzce earthquakes in Turkey was examined in general.

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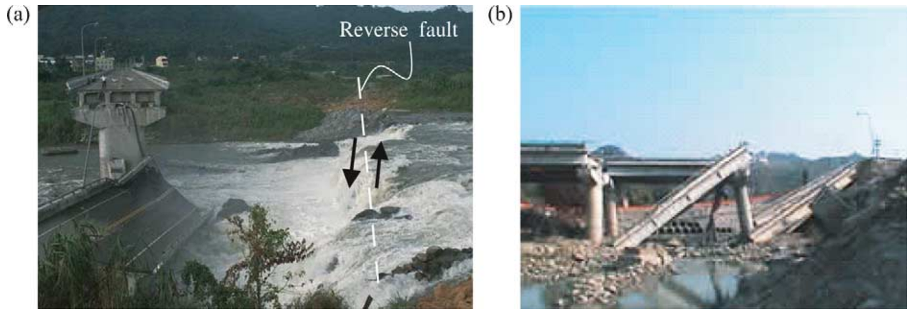

Fig. 1. Collapsed bridges in Taiwan due to fault rupture: (a) Bei-Fung Bridge [2]; (b) Shi-Wei Bridge [3].

The Arifiye Overpass was selected because strong near-fault effects were the main reasons of the damage. Two problems were of key importance, the first being the typical structural damage due to insufficiently detailed shear-keys, elastomeric bearings and column-to-cap connections. The second was the problem due to the geotechnical damage sustained by the Arifiye Bridge, particularly regarding the mechanically stabilized earth wall (MSEW) system. It is noteworthy to mention that such a MSEW system is the first ever subjected to a substantial near-fault ground motion and tectonic deformations. The Arifiye Overpass served not only as a typical case in terms of its structural damage pattern, but also as a unique case in terms of its geotechnical damage.

2. Bridge performance in recent earthquakes in Turkey

In 1999, two major earthquakes occurred in Turkey. These events were generated by the North Anatolian Fault (NAF), which is a characteristic shallow strike-slip fault. The first M_w 7.4 event (17 August 1999, Kocaeli earthquake) hit densely populated urban environments, namely Kocaeli and Sakarya provinces, situated on an alluvial fan at the western strand of the NAF. The second M_W 7.2 event (12 November 1999, Duzce earthquake) destroyed the city of Duzce which experienced strong shaking from the former event as well. Both earthquakes caused significant damage to the highway infrastructure, particularly in the north-western section of the Trans European Motorway (TEM). The TEM is a major east–west highway connecting Istanbul and Ankara, and has many bridges and long viaducts. The highway runs parallel to the fault rupture of the Kocaeli earthquake at a distance of less than 3 km. In some locations, the TEM was crossed by the surface rupture. The locations of the highway bridges along the TEM, and the recorded PGA at their nearest strong motion recording stations are shown in Fig. 2. Near-fault effects of Kocaeli and Duzce events, created large impulsive fault-normal ground shaking coupled with vertical accelerations and surface fault ruptures. These were the primary reasons why several spans had failed shear keys, were unseated from their bearings, or even collapsed. During the Kocaeli event, the TEM overpass at Arifiye collapsed as a result of surface rupture (explained in details subsequently). At the Mustafa Inan Viaduct (Fig. 3), a curved bridge located about 15 km southeast of the epicenter, girders were dislodged, seismic buffer stops located at the second-longest pier in the central span were damaged, and there was separation at the dilatations [6]. Failure of shear keys and damage of elastomeric bearings were observed in another viaduct (Sakarya Viaduct, Fig. 4) that was located less than 2 km north of the fault (Fig. 2(b)). This viaduct consisted of simply-supported prestressed concrete box girders seated on elastomeric bearings. Shear keys, which restrained transverse movements of the girders at the end of each box, and elastomeric bearing supports became dysfunctional [8].

The Duzce earthquake produced a powerful velocity fling, PGA's of up to 0.8 g and large ground movements.

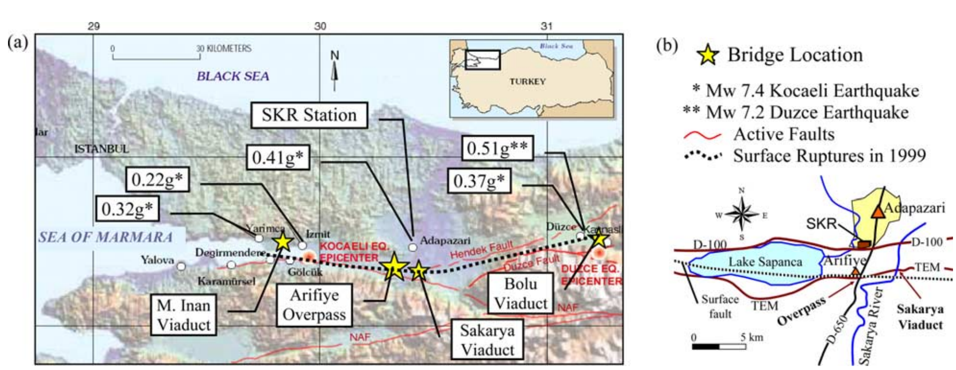

Fig. 2. (a) Locations of damaged highway bridges and recorded PGA at the nearest recording stations during the 1999 Kocaeli and Düzce earthquakes (map modified from [5]); (b) detailed map showing locations of Arifiye Overpass and Sakarya Viaduct.

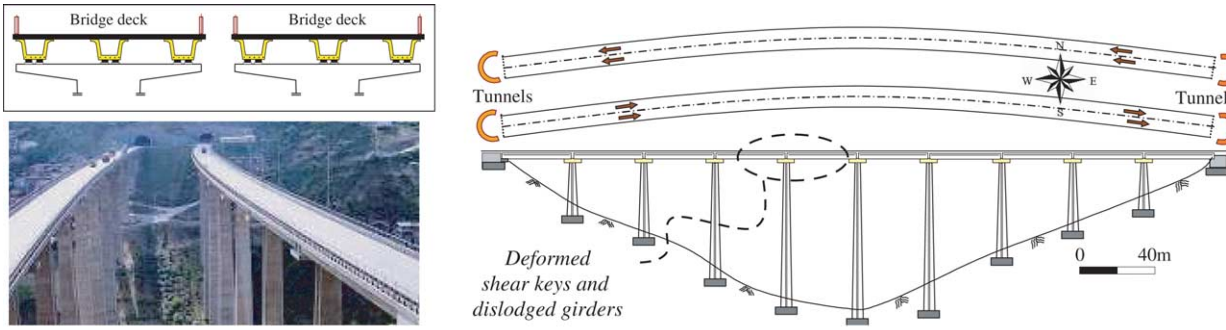

Fig. 3. Shear-keys failure at Mustafa Inan Viaduct.

Surface rupture crossed the bridge piers of the Bolu Viaduct, with a right lateral fault offset of more than 1.50 m (Fig. 5). This viaduct was composed of a pair of independent parallel bridge decks (each 40 m long and 17.5 m wide) supported by 60 piers, which makes it the longest (2.5 km in length) bridge on the TEM. It was still under construction during the Duzce earthquake. Although the bridge decks were equipped with seismic dampers, bridge girders could not accommodate the substantial displacements imposed by the main shock and the tectonic deformation. Surface faulting forced the piers to rotate as the rupture crossed the viaduct (Fig. 5(a)), resulting in damaged shear keys, bearing supports and dampers, and the dislodgement of girders (Fig. 5(b)).

3. Description of bridge overpass in Arifiye

The Arifiye Overpass, as shown in Fig. 6, was a four-span, 100 m-long, simply-supported prestressed concrete bridge located on the TEM near the city of Adapazari, Turkey (Fig. 2). The bridge was built during the late 1980s in accordance with AASHTO Standard Specifications for Highway Bridges [9]. Its two center and side spans overpassed the motorway and a local service road. Following the construction of the overpass bridge, a 10 m-high approach ramp with a double faced MSEW system was constructed adjacent to the northern bridge abutment (Fig. 6). The bridge had a skewed configuration (about 60°). Its girders were supported by three RC wall type piers and two end-abutments (Fig. 7). Its deck consisted of four simply-supported spans. Each span had five precast and prestressed concrete U-beams supported by five elastomeric bearings seated on pier-walls and seat-type abutments. The piers were 1.0 m thick and 13.65 m wide in the longitudinal and transverse directions of the bridge axis. The RC footings that supported the piers were 5.3 m wide and 14.4 m long. Each footing was supported by eight 1.0 m diameter cast-in-place RC piles extending 40–50 m below the ground surface. The northern abutment was supported by 16 cast-in-place RC piles ($D = 1.2$ m) extending to 48–50 m below the ground surface (Fig. 7(b)).

4. Near-fault ground motion at Arifiye bridge site and the resulting structural damage

The bridge overpass at Arifiye is located less than 50 km from the Kocaeli earthquake epicenter. The closest recording station to the bridge was the Sakarya station (SKR), located between downtown Adapazari and Arifiye, about 4 km north of the bridge site and 3 km from the nearest fault rupture (see Fig. 2(b)). During the strong shaking, this station recorded the largest peak horizontal ground acceleration of about 0.4 g (EW direction), and peak vertical ground acceleration of 0.26 g. The EW-direction acceleration and its computed velocity and displacement time-histories are shown in Fig. 8. This record exhibits typical near-fault characteristics with a displacement offset in the fault parallel direction (i.e. fling), and was characterized by strong velocity pulse of relatively long period [10]. Notably, the Sakarya (SKR) station was founded on a stiff soil site. Based on the site measurements, a shear-wave velocity of 400 m/s was reported for this station [11]. According to the SPT results, (explained in details later) the Arifiye Overpass was located on a soft soil site. Therefore, one may expect that the actual accelerations at this site would be even higher than what was measured at SKR due to site amplification effects. Nevertheless, no structural collapse or serious damage was observed in the neighboring residential units

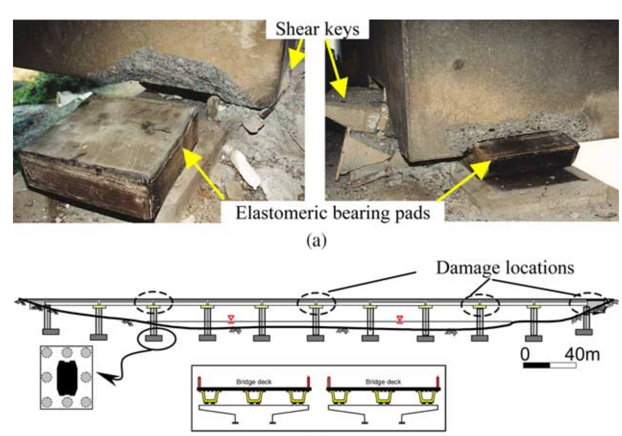

Fig. 4. Dislodgment of bearing systems at Sakarya Viaduct: (a) damaged bearing pads and shear keys [7]; (b) details of viaduct.

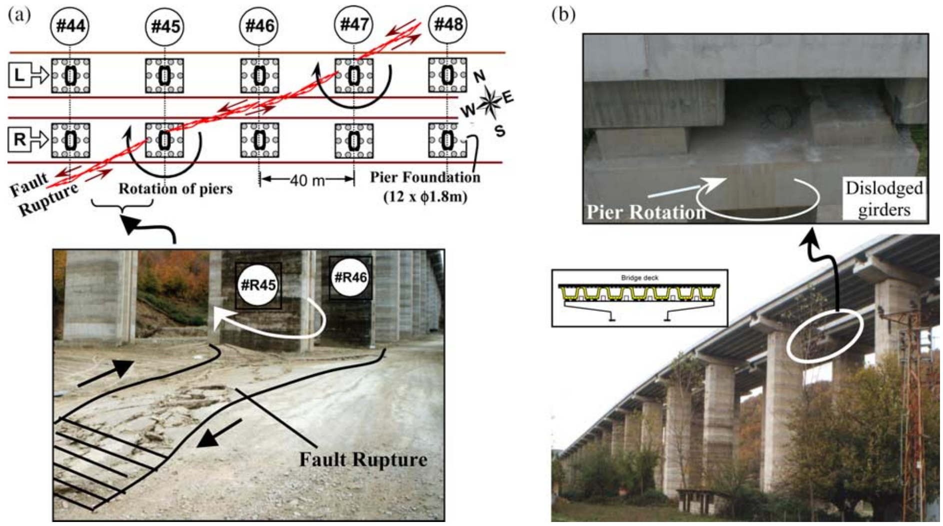

Fig. 5. Performance of Bolu Viaduct during Duzce earthquake: (a) piers traversed by surface fault rupture; (b) superstructural damage.

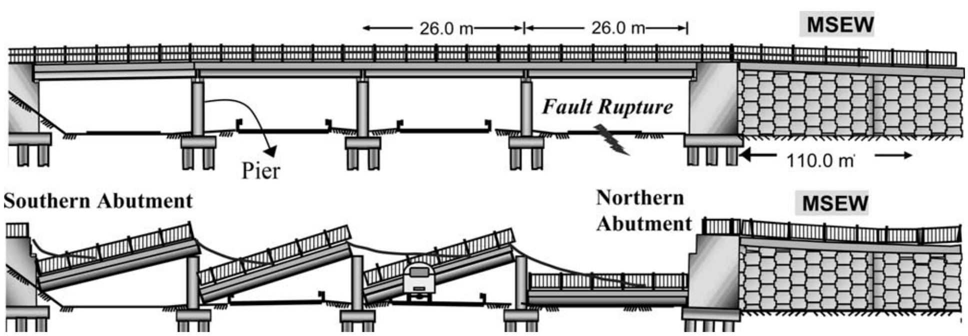

Fig. 6. Overpass bridge at Arifiye and mechanical stabilized approach fill walls before and after the Kocaeli earthquake.

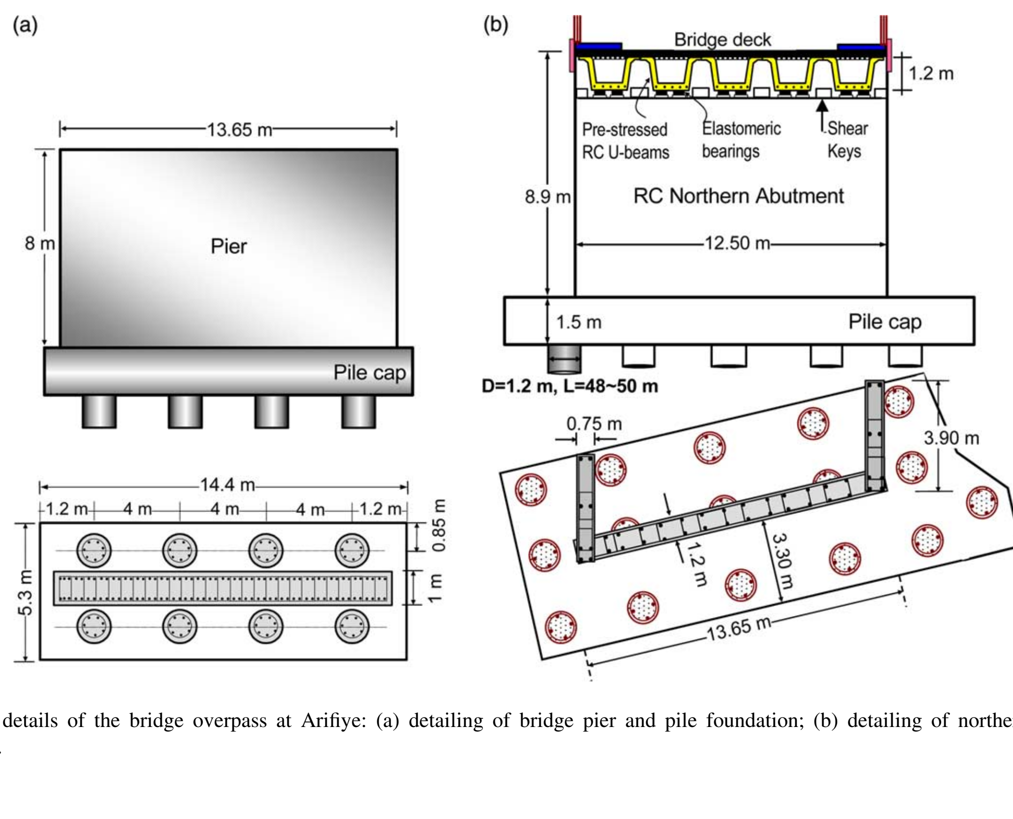

Fig. 7. Structural details of the bridge overpass at Arifiye: (a) detailing of bridge pier and pile foundation; (b) detailing of northern bridge abutment, deck and girders.

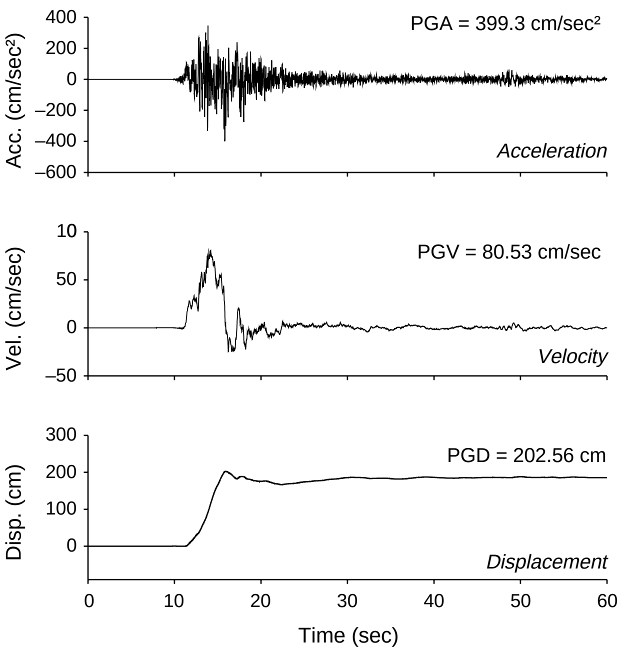

Fig. 8. Sakarya station (SKR) recording (EW direction) during the 1999 Kocaeli earthquake. (Note that the NS component of motion could not be recorded due to malfunction of the transducer).

(on both sides of the surface fault) in the vicinity of the Arifiye Overpass [12]. In fact, the structural damage gradually increased northward, where it became most destructive in the center of Adapazari (Fig. 2), 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 accurately draw the isoseismic map of peak ground acceleration at Arifiye.

Despite the lack of acceleration data at the bridge site, the permanent ground displacements due to surface fault rupture was clearly measured at Arifiye. Such deformations resulted in unseating of the bridge girders that was followed by a total collapse of the bridge spans as well as damage to the MSEW of the reinforced approach fill. The structural collapse of Arifiye Overpass revealed the following deficiencies. The large right-lateral fault traversing at Arifiye caused more than 1 m of longitudinal displacement under the northern span. Such permanent deformation was much greater than the existing seating length of the deck girders. Tilting of bridge piers located in the north direction and close to surface rupture (Fig. 7) exacerbated the dislodging of girders from their elastomeric bearing supports. Insufficient seating length of deck girders and elastomeric bearings, as well as dysfunctional shear keys (Fig. 9) triggered the catastrophic failure of the bridge span.

The observed structural damage of the Arifiye Overpass is typical such that other TEM bridges also suffered similar damage due to the shear key failures. These shear keys were not designed or constructed sufficiently against large transverse movement. In fact, had the shear keys been able to provide tolerable lateral restraining to those TEM bridges and overpasses, much of the associated damage could have been eliminated [14]. The collapse of Arifiye Overpass further indicates that bridges with skewed geometry are more vulnerable to support excessive movements, which may cause significant damage to their supports and bearing systems. For such structures, extended seating width provides a reasonable solution to avoid deck failures, particularly if the fault rupture has the potential to intersect with bridge structures.

Today many bridges along the TEM, especially those located in the vicinity of the Istanbul Metropolitan Area, are threatened by the North Anatolian Fault (NAF) and consequent surface ruptures. Strong earthquakes in this area may replicate the observed damage at Arifiye. For this specific reason, the seating length of the existing bridges has been widened by mounting additional L-shaped supports (Fig. 10(a)), and by redesigning elastomeric bearings and shear keys [9,14].

Another possible solution to prevent span failures is the installation of cable restrainers across deck joints. The restrainers utilized at expansion joints on the decks of the Bolu Viaduct prevented end girders from dislodging off of

Fig. 9. Arifiye Overpass traversed by surface fault rupture: (a) side view; (b) aerial view [13]; (c) damaged shear keys in the southern bridge abutment.

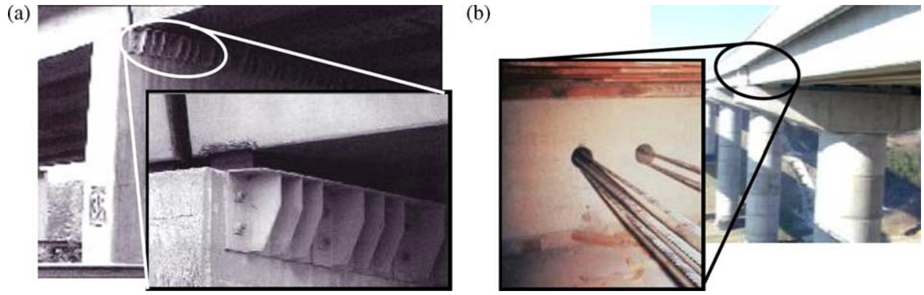

Fig. 10. (a) Retrofitting of bridge girders by increasing seating length of bridge girders with L-shaped steel profiles [14]; (b) hinge joint (cable) restrainers [15].

their supports during the Duzce earthquake [14]. Fig. 10(b) shows a typical hinge joint (cable) restrainer that has been installed to prevent excessive longitudinal joint separations in this viaduct. Such retrofitting measures were also shown to be effective during the 1989 Loma Prieta earthquake [16]. These are all applications commonly used to avoid catastrophic failures in highway bridges.

5. Geotechnical damage at Arifiye Overpass

The majority of damage at the Arifiye Overpass from a geotechnical standpoint concentrated on the 100 m-long MSEW system. This wall system was built as a 'double-faced' or 'back-to-back' type wall having parallel reinforced concrete facings with ribbed metallic reinforcing inclusions to accommodate a two-way divided roadway as shown in Fig. 11. A reinforced concrete culvert was designed beneath the approach ramp possibly to facilitate storm or flood water discharge (Fig. 11(c)). Two slip joints (S1 and S2, Fig. 11(c)) were also designed on each wall face on top of the rigid culvert to protect the walls from damage due to differential settlement. Even though the site below the MSEW system contained undesirable alluvial subsoil layers that were prone to significant seismic hazards, no subsoil remediation was ever done. Accordingly, the MSEW approach ramp experienced large settlements of about 60 cm during and after construction. However, this did not cause substantial damage to the walls [17].

The Adapazari region (Fig. 2) is located in a large valley covered by alluvium deposits formed by a nearby lake and surrounding rivers. Soil deposit extends about 45 km from east to west, and 30 km from north to south with a varying thickness greater than 200 m [18]. The geology of the bridge site in Arifiye is dominated by Pliocene to Pleistocene sedimentary deposits which lay at least 50 m below the younger deposits [19]. Standard penetration tests were conducted by the Turkish General Directorate of Highways to gain sufficient subsurface information between both ends of the bridge overpass soon after the earthquake. The locations of the subsurface borings are depicted in Fig. 12. This figure also indicates the 2D visualization of the local subsoil conditions along the axis of the bridge overpass. The ground water table was approximately 5 m below the ground surface. Boring No. 1 indicates that very soft layers of soil deposits were encountered beneath the southern abutment and extend to a depth of 22 m, below which there is a dense ($N_{30}=100$) layer of sedimentary deposit containing silty sand with gravels. The loose layers became thicker to the depth of 34 m below the northern abutment, where Boring No. 2 was made. Boring No. 2 was the nearest soil investigation to the MSEW and it showed a 2.5 m thick fill followed by varying thicknesses of silty sand and silty clay deposits. Loose silty sand and silty clay layers ($N_{30}<20$)

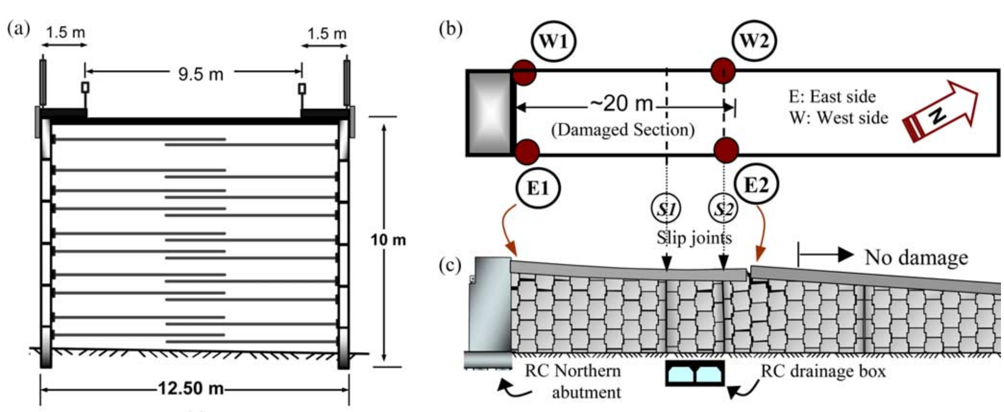

Fig. 11. Schematic of reinforced bridge approach fill of Arifiye Overpass: (a) cross section; (b) plan view of damage-concentrated locations; (c) damaged eastern wall face.

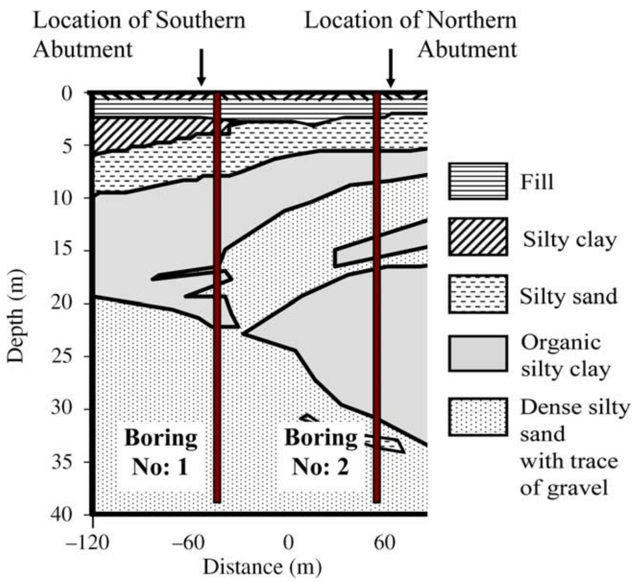

Fig. 12. Visualization of subsoil geology along the axis of Arifiye Overpass (modified based on [19]).

below the reinforced walls may have experienced liquefaction or seismic-induced densification during the seismic event.

These field measurements obtained after the earthquake may reflect denser states of the soil layers than those prior to the earthquake. Field observations revealed a number of factors that that damaged the MSEW system in Arifiye. These include (i) large tectonic movements along the main fault line, (ii) the presence of a drainage culvert, (iii) near-fault effects, and possibly, (iv) cyclic loading-induced soil densification and settlement. Only the sections of the MSEW approach ramp between the bridge abutment and RC culvert, about 20 m long, was significantly damaged (Fig. 11(c)). The locations most affected by the damage along the eastern and western wall faces are highlighted in Fig. 11(b) as E1 and E2 (E: Eastern side), and W1 and W2 (W: Western side), while their detailed views are presented in Figs. 13 and 14. The right-lateral strike-slip fault rupture along the main fault line passed under the northernmost span of the bridge (Figs. 4 and 7) with large transverse and vertical displacements of approximately 3.5 and 0.5 m, respectively [5]. The vertical ground deformation appeared to be the main source of damage to the reinforced walls of the approach ramp. The deformation on the main fault rupture extended through the RC culvert under the reinforced ramp (Fig. 11(c)). Cracks caused by vertical deformation were clearly observed on the asphalt-covered side roads, especially on the western side of the ramp (Fig. 14). It should also be pointed out that the final permanent ground deformation in this section possibly included the 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 to the wall faces was concentrated at higher elevations above the RC drainage box culvert. Because the vertical displacement in the eastern wall face was larger than the other side, the approach ramp tilted eastward in the cross section, especially at the location above the drainage box culvert. This tilting was probably due to the presence of the rigid culvert that prevented the ramp from moving with its foundation soil. The walls could not uniformly accommodate the underlying fault-induced ground deformations and cyclic-induced soil densification (i.e. ground settlement). 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, whereas the eastern wall face was stretched outward (Figs. 13(b) and 14(b)). The buckled side increased compression on the facing panels at W2 and forced the panels to displace at this location. The largest damage in the reinforced walls was observed on the eastern side at E1. At this location, the wall was displaced both vertically and horizontally for about 25–30 cm. The displacements here were so large that they exceeded allowable design limitations for independent panel movement. Thus, the panels could not accommodate the ground deformations leading to the large separation and cracking of the panels. However, the facing panel connections with the metallic reinforcements did not fail, and their flexible joints allowed large displacements and differential settlements.

At E1 and W1 (Figs. 13(a) and 14(a)), the facing panels interacted with the pile supported bridge abutment. The damage states at both locations were also different. At E1,

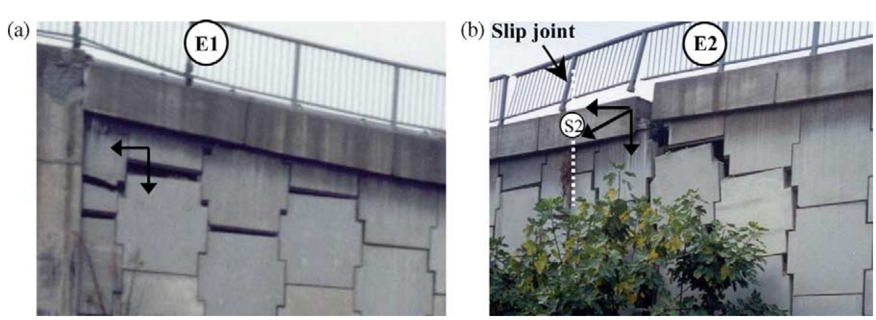

Fig. 13. Damage on the eastern MSEW face: (a) E1; (b) E2 (courtesy of M. Ozbakir).

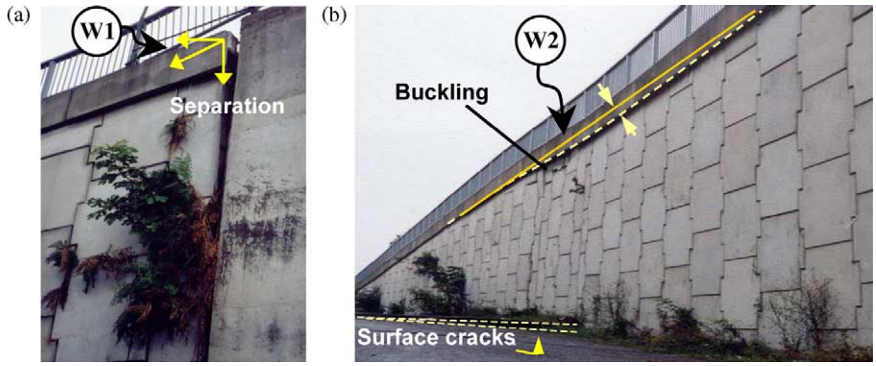

Fig. 14. Damage on the western MSEW face: (a) W1; (b) W2 (courtesy of M. Ozbakir).

the vertical ground deformation was so large that the flexible wall face was forced to displace vertically and longitudinally (i.e. along the wall). The movement in the longitudinal direction was prevented by the rigid abutment at the top of the wall that caused large panel separations and cracks, but no damage was observed at the bottom. At W1, the vertical ground deformation was not large compared to E1, however, a gap (i.e. panel separation) occurred between the panels and the abutment as shown in Fig. 14(a). This gap appeared to result from buckling in the wall as indicated in Fig. 14(b). That is, the buckling pulled the whole western wall face longitudinally at W2. This did not cause any damage in the facing panels between W1 and W2, indicating that the reinforced wall system was highly flexible, and the large ground deformations were accommodated by the flexible joints of the facing panels.

6. Discussion

Bridge failures that occurred during the recent earthquakes in Taiwan and Turkey showed similarities in terms of construction techniques and observed structural damages. These bridges were constructed as simply-supported reinforced concrete slab girders, and sustained damage in the form of dislodging of girders from their seating, shear key failures, and in some cases total collapse of bridge decks due to the surface fault ruptures. It was also observed that skewed and curved bridges were more vulnerable to near-fault effects.

The observed deficiencies in the structural systems of these bridges suggested that the wall type piers should have enough seating size with stabilized elastomeric bearings to accommodate for the larger possible movements. Partial continuous spans and/or hinge joint (cable) restrainer as well as shear keys and bearing elements that are consistent with design capacity of piers may help to prevent catastrophic failure of bridge spans as is being considered in recent designs in Turkey.

The MSEW system provided a unique case history under extreme loading conditions. Geotechnical observations showed that (i) the wall system had significant flexibility and was only lightly damaged and maintained its structural integrity, while withstanding large ground deformations (ii) differential settlement may adversely affect the wall performance, despite not being part of the design (such as in AASHTO Guidelines), (iii) the stiffness of the reinforced soil system in its longitudinal direction is also important although design is focused on the transverse direction only, and (iv) soft foundations should be improved using preventive measures (e.g. soil replacement, grouting and densification etc.) prior to wall construction in order to minimize settlement, especially in seismically active areas.

It should also be noted that in addition to the permanent ground deformations, surface fault rupture may also create an instantaneous energy release resulting in strong velocity pulses that force nearby structures to dissipate such an energy within only a few cycles of plastic displacement excursions. Although it was not quantified for Arifiye due to the sparcity of recording stations, these type of situations may be most detrimental to long period structures, such as bridges, where the resonance phenomenon may take place when exposed to coherent long period velocity pulses contained in near-fault ground motions (see Fig. 8). In some cases, vertical acceleration may contribute to increase levels of damage, especially at the near-fault. Although this situation was rarely quantified by recording stations during the Turkey earthquakes (e.g. Duzce station, DZC [20]), recent studies have shown significant vertical accelerations in the vicinity of the active faulting systems (e.g. [21,22]). The lack of both horizontal and vertical acceleration data in the vicinity of the bridge site does not allow us to establish a numerical projection that could convey these points. With that limitation, our investigation of Arifiye relies solely on site observations. We believe that this is an essential first step toward developing more profound design methodologies against surface rupture hazards.

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