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Seismic Performance of Earth Retaining Structures under Case Histories Earthquakes

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ABSTRACT

The stability of infrastructures like earth-retaining structures "ERSs" under the shaking of earthquakes is an important matter of reconnaissance for geotechnical engineers. Lessons recorded during such reconnaissance are essential for enhancing the future design and analysis, which, in turn, improve stability and reduce the failure cases. In this paper, the stability and instability recorded in case histories of selected ERSs under the hit of past earthquakes were presented, and lessons from them were drawn so as to be considered in future design processes. The various factors affecting the performance of earthquake-prone retaining walls were reviewed and systematically examined according to the studied cases. According to the presented case histories, ERSs exhibited different levels of performance under earthquakes. They performed very well in some cases, while they exhibited lower performance in others. The ERSs that are well designed under static loads are found to perform very well under both static and dynamic loadings, even though they aren't designed for the shaking loads. Many factors impact the ERS's stability, like the constraint degree, flexibility, face inclination, wall geometry, loading condition, backfill basic-properties and compaction, seismic thrust, and the rigidity of foundation-soil. However, the main damages recorded are summarized herein to include, for reinforced-soil ERSs, (reinforcement strips pulling out and corrosion, failure of connection and geogrid internal slippage, facing-unit damage, etc.), and for concrete and gravity ERSs, (structural failure, lateral offset, sliding, outward movement, rotation or excessive displacement, etc.). Accordingly, new assessments may be introduced regarding the seismic performance based on failure (or damage) recorded in the reconnaissance during earthquakes.

Keywords: Case histories earthquakes; earth-retaining structures; seismic load; seismic stability; retaining walls

INTRODUCTION

Earthquakes are one source of dynamic loads imposed on geotechnical structures (as in earth-retaining structures, ESRs) and soil. The soils' strength and behavior vary widely when subjected to dynamic loads compared to those in static conditions. The nature of dynamic loads related to earthquakes is of a random type. They vary widely with time, taking an irregular fashion; therefore, they are considered random loads. Figure 1 illustrates the accelerogram of the Halabjah Earthquake, Iraq, on November 12, 2017 (East-West component) (Das & Ramana 2011; Aryadi et al. 2024; Li & Shen, 2024; Octavia et al. 2024; Al-Taie & Albusoda 2019). In earth-retaining structures, the random load from earthquakes caused many failures or damages due to the excessive dynamic "lateral earth pressure," LEP, on these structures. Many failure modes were recorded during earthquakes due to increasing the LEP, including sliding and tilting (overturning) (Prakash 1981; Das & Ramana 2011).

In general, the instability problems of ERSs during past earthquake cases are important reconnaissance issues for scholars. Investigating, recording remarks, and learning from these cases are key matters to improve the dynamic performance of ERSs and avoid geotechnical disasters under earthquake shocks. Selecting historical earthquake cases from this vast research, specifically those involving earth-retaining structures, was no easy task. Choosing these cases from diverse regions of the world presented another challenge. Connecting the numerous factors influencing the performance of these structures was the greatest challenge. In this paper, the instability problems resulting from the hit of random seismic loading, from past historic earthquakes, on selected ERSs were explored, presented, and critically discussed. It aims to provide an insight into the damage cases and different failure patterns of these structures when subjected to ground-shaking. The selected systems are reinforced soil walls (including different reinforcement materials like geogrid, steel strips, etc.), different types of cantilever and gravity structures, U-box walls, gabion, abutment walls, basement and semi-basement walls, block stone, quay walls, masonry ERSs, concrete block ERSs, and anchored and tied—back systems. The historic earthquakes presented in this paper are from Chile, Japan, Costa Rica, New Zealand, the USA, South Asia, Turkey, India, Greece, Haiti, and Nepal, as illustrated in Figure 2.

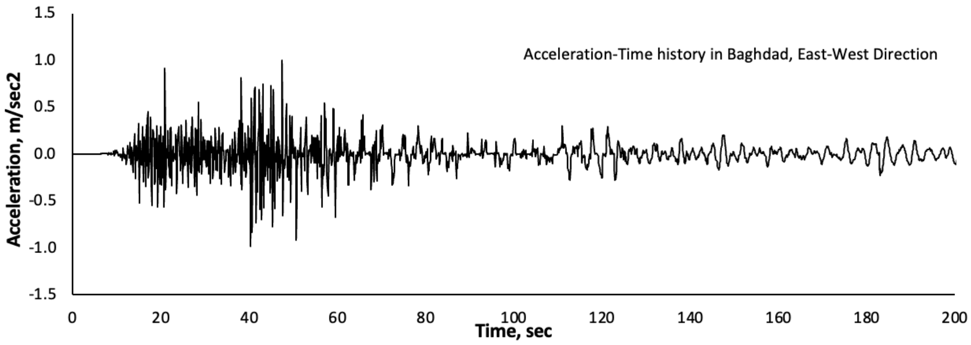

FIGURE 1. Accelerogram of Halabjah Earthquake, November 12, 2017 (E-W component) (Al-Taie & Albusoda, 2019)

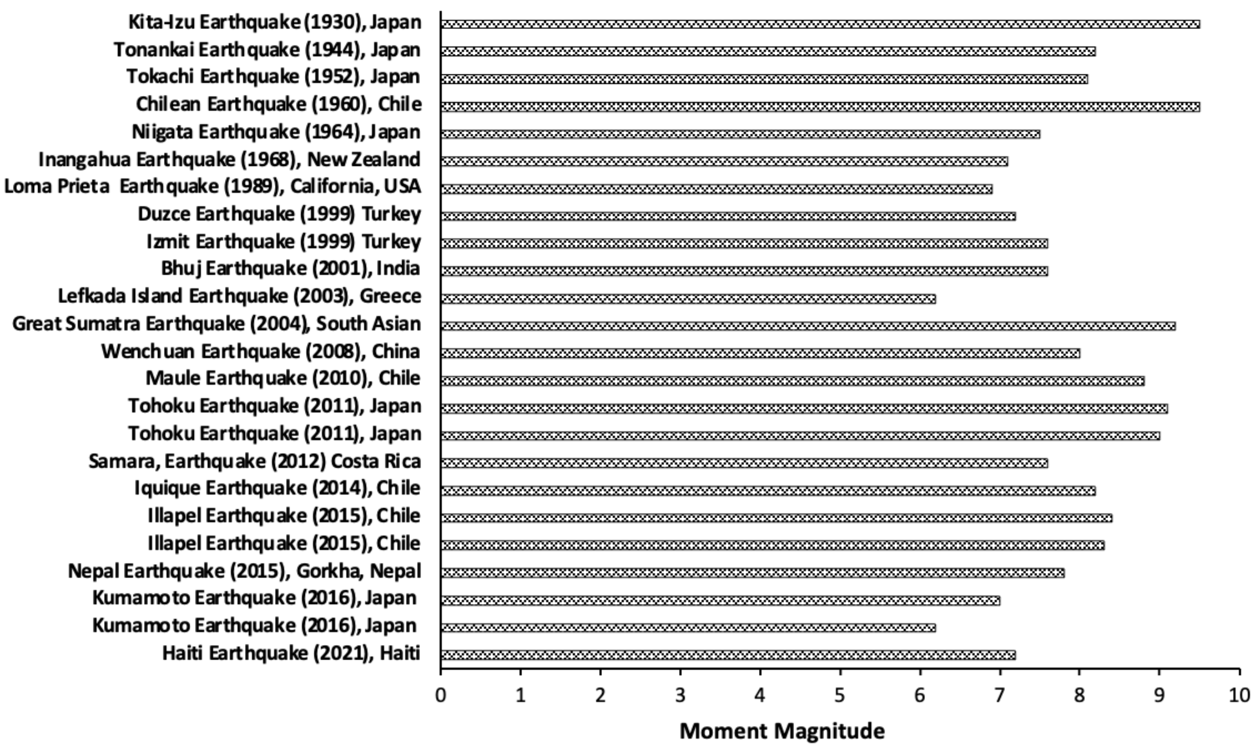

FIGURE 2. The historic earthquakes presented in current investigation

OBSERVATIONS OF ERSS' DAMAGE AND FAILURES FROM DIFFERENT EARTHQUAKES

DAMAGES DURING THE INANGAHUA EARTHQUAKE (1968), NEW ZEALAND

In May 1968, an earthquake with a magnitude of 7.1 (moment magnitude) struck at a depth of 12 km close to Inangahua town, Murchison, New Zealand; this was the Inangahua earthquake. This earthquake killed, injured, and evacuated several people; it also damaged many infrastructures (chimneys, roads, bridges, retaining structures, and railways) and caused a number of landslides (Dowrick & Sritharan 1993; McSaveney 2017).

Regarding ERSs, (Richards & Elms 1979) inspected a number of bridges to report the damage to their abutment walls during the 1968 Inangahua earthquake. They stated that about 40% of the inspected abutments suffered damage; this form of damage was also mentioned by (Ostadan, 2005). As geotechnical damages, (Lew et al. 2020) classified the settlement of backfill in bridges' abutments into three main categories (minor, moderate, and major). These authors observed the damage in bridges' abutments during a selected number of earthquakes in New Zealand, including the 1968 Inangahua earthquake. They concluded that, based on the type of abutment, there were no differences noted between the geotechnical damages. The main geotechnical damages observed by these authors included the settlement of approaches, lateral displacement of the abutments, and abutment tilting. They attributed the majority of the stated damages to the soil liquefaction experiment.

DAMAGES UNDER LOMA PRIETA EARTHQUAKE (1989), CALIFORNIA, USA

Considerable damage to the infrastructure, more than 60 people's deaths, and billions of dollars in economic losses resulted from the occurrence of a 6.9 magnitude earthquake in California, USA. This was the "Loma Prieta Earthquake," which hit Santa Cruz city in October 1989 (Mitchell et al. 1990, 1991; Clough et al. 1994).

In many locations, during this earthquake, the infrastructures were affected to different degrees of severity. The earthquake effects were recorded for electrical substations (damaged), buildings (collapsed or partially or significantly damaged), bridges and causeways (showed structural failure or destruction), roads (cracked), and other damage to the infrastructure. From a geotechnical aspect, the earthquake caused soil liquefaction, downslope movement, ground ruptures, sand volcanoes, landslides, and damage to some of the ERSs like waterfront RWs (Eguchi & Seligson 1994; Fagan 1999; FEMA 2003; Gathright 2004; Scawthorn 2005; USGS 2008; Choudhury & Rajesh 2020; Javadi et al. 2021).

The severity of damage during this earthquake varied based on the type of ERS and design considerations. For example, the basement walls, mechanically-stabilized ERSs, and reinforced-earth concrete ERSs with a height of 5m to 10m (designed with some seismic consideration and a horizontal PGA of 0.1g to 0.55g) performed well without recording any damage during the Loma Prieta Earthquake. In contrast, damage represented by lateral movement was reported for 4.4m height geogrid-reinforced ERSs under horizontal PGA of 0.1g to 0.4g of "Loma Prieta Earthquake"; the reported lateral movement was 88cm at the top of these systems (Benuska 1990; Whitman 1991; Nova-Roessig 1999; Lew, 2012; Hazirbaba et al. 2019; Khan et al. 2024, 2025).

DAMAGES DURING THE LEFKADA ISLAND EARTHQUAKE (2003), GREECE

On the 14th of August, 2003, the Lefkada Island earthquake (with a 6.2-moment magnitude, a=0.42g, and a depth of 12 km) struck off Lefkada Island, Ionian Sea, Greece, and caused damage to the infrastructures (like the road network, tourist facilities, marine and port facilities, and wastewater and water systems) (Papadopoulos et al. 2003; Papathanassiou et al. 2005; Benetatos et al 2005; Pitilakis & Roumelioti, 2013).

It should be mentioned that the Lefkada Island Earthquake of 2003 had a significant number of geotechnical impacts, something rather uncommon in past Greek earthquakes. Numerous geotechnical impacts and failure instances were recorded due to the Lefkada Island Earthquake (e.g., rock falls, lateral spreading, landslides, ground settlement, liquefaction, and quay wall damage), and this is uncommon in past earthquakes in Greece. Some of the failure instances were attributed to poor conditions of the soils (loose soil at shallow depths), inadequate compaction of the backfill materials behind the retaining facilities, and the immoderate seismic lateral pressures from the soil (Margaris et al. 2003; Gazetas, 2004; Gazetas et al. 2005; Karakostas et al. 2005; Pitilakis & Roumelioti, 2013). These authors summarized the quay wall damages under the Lefkada Island earthquake to include the rotation or excessive displacement of the ERSs, complete overturning of the walls, lateral spreading, and cracking and settlement of backfill soil. (Gazetas 2004; Karakostas et al. 2005; Pitilakis & Roumelioti 2013) gave typical photographs as examples to illustrate these damages, as shown in Figures 3 and 4.

FIGURE 3. Leaning of RWs during Lefkada Island Earthquake (from Margaris et al. 2003)

FIGURE 4. Displacement of quay RWs during Lefkada Island Earthquake (from Pitilakis & Roumelioti 2013)

Among the investigated cases, an interesting instance was noted for the case of (5m x 5m) concrete rigid gravity quay retaining walls in the Lefkada Marina. (Gazetas et al. 2005) examined the performance of these walls during the Lefkada Island Earthquake (2003). These walls were designed, following the Greek seismic-code (EAK, 2000), using a ground acceleration value of 0.36g as a basic value in the analysis and based on the "Newmark sliding block method." Although the investigated systems have experienced damage, they performed well compared to other retaining systems designed according to conventional practice. In reality, the recorded damages were only small-scale. On the other hand, it was found that the Newmark method fails to calculate the considerable permanent displacement of quay walls resulting from the Lefkada earthquake. Furthermore, under the vertical acceleration of the Lefkada earthquake (of very high frequency), in agreement with various studies (Seed & Whitman, 1970; PIANC, 2001; OCDI, 2002; Gazetas et al. 2004), it was noted that, unequivocally, the vertical acceleration does not affect the response of the quay ERSs. Finally, the conclusions of Gazetas et al. 2005 indicated that the displacement of quay walls under earthquake results from both the soil elastic deformation (under the base of the wall) and sliding of the wall's base. Different values for ground acceleration have been adopted in international standards to reduce the damage to structures subjected to earthquakes. To ensure minor damages, a high level of requirement may be adopted. The effective "peak acceleration value" used in Greece for design is 0.36g (Gazetas et al. 2005).

DAMAGES DURING TOHOKU EARTHQUAKE (2011), JAPAN

The Tohoku Earthquake (2011), with a moment magnitude of 9.0, also known as the "2011 off the Pacific Coast of Tohoku Earthquake" and the "2011 Great East Japan Earthquake," hit the Pacific coast of Japan on the 11th of March. Globally, this earthquake is the fourth-largest after 1900, while locally, it is the first-largest. This earthquake had two main shocks that continued for more than two minutes. With excessively long vibrations and tsunamis, the Tohoku Earthquake (2011) caused great collapse or damage to many facilities, including buildings, dams, railways, roads, ports, power plants, etc. (Kamiya 2011; Branigan 2011; Kazama & Noda 2012; USGS 2012; Kuwano et al. 2014).

The Tohoku Earthquake (2011) produced serious impacts on embankments, slopes, river dykes, seawalls, and other geotechnical facilities. Also, it caused different damages to several waterfront ERSs and some damage to reinforced soil ERSs (Kuwano et al. 2014; Krishna & Katsumi, 2020; Javadi et al. 2021). According to (Choudhury & Rajesh 2020), many waterfront ERSs were damaged during the main shocks and the aftershocks of the Tohoku Earthquake (2011). These authors emphasized the need for appropriate attention to the choice of different combinations of forces for the seismic design of these earth-retaining structures. The emphasized forces by these authors are inertia forces in the wall (due to earthquake), dynamic LEP, buoyancy pressure (at the base of the wall), dynamic water forces, hydrostatic pressure, and wave forces.

The performance of many reinforced soil ERSs under the shocks of the Tohoku Earthquake (2011) was reviewed by (Kuwano et al. 2014). They reported three types of reinforced soil ERSs in the Tohoku area; these are multianchor, steel strip, and geogrid RWs. The effect of the Tohoku Earthquake (2011) on the performance of these ERSs varied from "no damage" to "serious damage." Actually, more than 90 % of these ERSs performed well with no recorded damages, while serious damage was reported for less than 1.0% of the walls. It is worth mentioning that the impact of the Tohoku Earthquake (2011) on these retaining systems was much greater than the value adopted in their design. In other words, the real performance of the reinforced ERSs under seismic loading is notably greater than their design target.

A sample of geogrid RW damage was reported by (Kaneko & Kumagai, 2011). In this case, a 5 m high geogrid RW (with facing of wire mesh and primary and secondary geogrid layers of 1.2 m and 0.6m spacing, respectively) collapsed during earthquake impact with estimated ground acceleration of about 0.306g; the collapse of the wall was attributed to connection failure and internal slippage of the geogrid. Later investigation showed that very high groundwater with a lack of a suitable drainage system led to the walls' collapse. Another collapse case was recorded for a 10 m high steel strip RW, with steel strips spaced every 0.75 m, constructed on poor foundation soil. The investigation for the wall's collapse exposed that it occurred due to the sliding of poor foundation soil about 7 meters horizontally under the seismic motion effect. Furthermore, damage during the Tohoku Earthquake (2011) was noted at a few points in the facing units (concrete panels) of steel strip RWs with multi-anchors (Kuwano et al. 2014).

DAMAGES DURING IQUIQUE EARTHQUAKE (2014), CHILE

On the first of April, 2014, the Iquique earthquake (with an 8.2-moment magnitude and 25-kilometer epicenter depth) struck off Chile's coast. In this earthquake, many people died and were injured, and infrastructures were damaged to different degrees or collapsed (CSN 2014; Moreno & Paz Núñez 2014).

Among the affected infrastructures, failure cases during the Iquique earthquake in 2014 were reported for some types of ERSs, such as reinforced concrete cantilever type, unreinforced masonry wall, mechanically stabilized soil ERS, quay wall, and gabion wall. It is worth mentioning that these failures were not related to immoderate seismic thrust; they were due to either poor design and/or construction materials or soil failure issues. For example, the failure of the no-footing concrete cantilever ERS, the failure of no-reinforcement steel masonry walls, the failure of mechanically stabilized soil ERS of corroded reinforced strips, the failure of quay walls due to liquefaction of backfilling soil, and the minor displacement (with no failure) of the gabion wall. Even though these ERSs' failed in the Iquique Earthquake (2014), no damages were reported for basement-type ERSs (Rollins et al. 2014; Wagner & Sitar, 2016a; Khan et al. 2024).

Rotation of the concrete cantilever wall (without footing) about its base was observed under the ground motion in the Iquique earthquake, 2014, as shown in Figure 5. The rotation of this wall might be due to either the lateral earth pressure or the inertia of the wall. In contrast, there is no distress problem seen for an adjacent wall with the same height (Rollins et al. 2014; Wagner & Sitar 2016b). Overall, the performance of ERSs after the Iquique earthquake, 2014, was considered satisfactory (Candia et al. 2017).

FIGURE 5. Rotation of concrete cantilever RW during the Iquique Earthquake (2014), (from Wagner & Sitar, 2016a)

DAMAGES DURING NEPAL EARTHQUAKE (2015), GORKHA, NEPAL

In April 2015, one of the damaging earthquakes of 7.8 (moment magnitude) and of a hypocenter depth of about 8.2km hit Nepal. This earthquake, called the "Gorkha Earthquake" or "Nepal Earthquake," is considered the largest in Nepal's history. This earthquake injured and killed about thirty thousand people and destroyed thousands of buildings (Parajuli & Kiyono, 2015; Okamura et al. 2015).

Surveying was conducted by researchers for the Nepal earthquake damages in selected areas. The survey showed that different parts of the infrastructure (including roads, slopes, retaining walls (RWs), and residential buildings) were damaged. Regarding the ERSs, two types of RWs were observed with the damaged infrastructure; these are reinforced RW and gravity RW, as shown in Figure 6. According to Okamura et al. 2015, damages were observed in the joints between the RWs. The damages in the gravity RWs are represented by cracking in some parts of these ERSs. Overall, minimal damages and no failures were reported to ERSs during this earthquake (Hashash et al. 2015; Khan et al. 2024).

DAMAGES DURING KUMAMOTO EARTHQUAKE (2016), JAPAN

In April 2016, exactly on the 14th and 16th of April, sequences of earthquakes (6.2 and 7.0 moment magnitude) hit the region of Central Kyushu, Japan; this is the Kumamoto Earthquake (Mukunoki et al. 2016; Kiyota et al. 2017; Anderson et al. 2023). This earthquake injured and killed more than three thousand people (273 people died and 2809 were injuries), destroyed numerous buildings, led to severe damage to infrastructure, and generated 1.0 million to 1.3 million of disaster debris (Kayen et al. 2016; Kiyota et al. 2017; Achour & Miyajima, 2020).

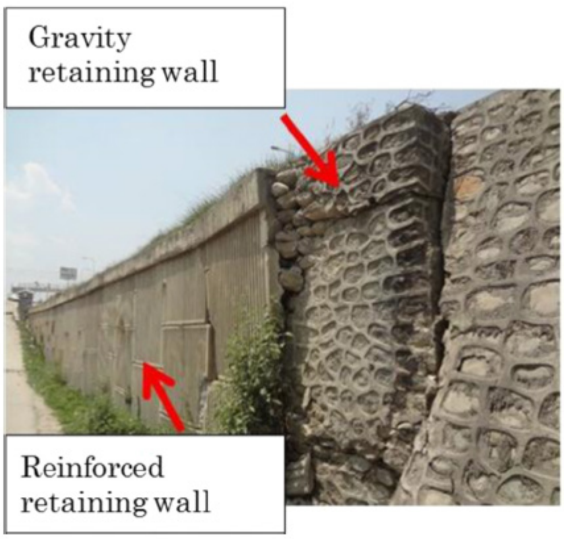

FIGURE 6. Damages of RWs during Nepal earthquake (from Okamura et al. 2015)

The severe damages caused by the Kumamoto Earthquake was attributed to the short period of intense shaking, where response spectra of this earthquake showed peak values below one second (Kiyota et al. 2017). Kayen et al. 2016 investigated different damages caused to several roads, hills, and RWs by the 2016 Kumamoto Earthquake. These authors reported damages to RWs of several types (including block stone RWs, unreinforced-concrete block RWs, and unreinforced-masonry RWs) and sizes (including RWs with different lengths and heights). The damages reported for unreinforced concrete block RWs are lateral offsets of these walls; see Figure 7.

FIGURE 7. Lateral offset for unreinforced-concrete block RWs during Kumamoto Earthquake (from Kayen et al. 2016)

The values of the recorded offset varied depending on the height of the RWs. A 20cm offset was recorded for taller RWs, while a higher value (40cm) was for the shorter. While the unreinforced masonry RWs showed diagonal cracks, these walls fractured as shown in Figure 8. Furthermore, the unreinforced stone masonry RWs were damaged.

FIGURE 8. Diagonal cracks for unreinforced masonry RWs during Kumamoto Earthquake (from Kayen et al. 2016)

Reconnaissance was conducted by researchers for the Kumamoto Earthquake geotechnical damages in the south of Kumamoto City, Mashiki Town, and the area of Aso

Caldera. The recorded and observed geotechnical damages in this surveying were soil liquefaction in different locations, significant deformation and settlement in some river levees and buildings, embankment damages, slope failures, and large-scale landslides (Mukunoki et al. 2016; Kiyota et al. 2017). Regarding the ERSs, three types of RWs were observed with the damaged infrastructure; these are cantilever RW, gravity RW, and reinforced soil walls, as shown in Figures 9 and 10. As can be noted, the cantilever and gravity walls were structurally damaged. In addition to the structural failure, the cantilever-type RW was excessively tilted. On the other hand, significant damage can be seen for reinforced soil walls, Figure 10. This damage was represented by the pulling out of the reinforcement strips from their position in the backfill soil side. The reason for this failure type was not identified, as stated by Kiyota et al. 2017.

FIGURE 9. Damages of RWs during Kumamoto Earthquake (from Kiyota et al. 2017)

FIGURE 10. Damages of reinforced soil walls during Kumamoto Earthquake (from Kiyota et al. 2017)

CRITICAL DISCUSSION ON THE PERFORMANCE OF ERSS DURING EARTHQUAKES

This paper reviewed the ERSs' performance during selected past earthquakes and addressed the damages and failures of various types of them in major world's seismic zones. The selected earthquakes include the (Kita-Izu Earthquake (1930), Japan), (Tonankai Earthquake (1944,) Japan), (Tokachi Earthquake (1952), Japan), (Chilean Earthquake (1960), Chile), (Niigata Earthquake (1964), Japan), (Inangahua Earthquake (1968), New Zealand), (Loma Prieta Earthquake (1989), California, USA), (Izmit Earthquake (1999), Turkey), (Duzce Earthquake (1999), Turkey), (Bhuj Earthquake (2001), India), (Great Sumatra Earthquake (2004), South Asian), (Lefkada Island Earthquake (2003), Greece), (Wenchuan Earthquake (2008), China), (Maule Earthquake (2010), Chile), (Tohoku Earthquake (2011), Japan), (Samara Earthquake (2012), Costa Rica), (Cephalonia Earthquake (2014), Greece), (Iquique Earthquake (2014), Chile), (Illapel Earthquake (2015), Chile), (Nepal Earthquake (2015), Gorkha, Nepal), (Kumamoto Earthquake (2016), Japan), and (Haiti Earthquake (2021), Haiti). These earthquakes struck with moment magnitude values ranging from 6.2g to 9.5g at epicenter depths varying from 8.2km to 45km as shown in Figures 2 and 11.

Depending on the magnitude, earthquakes can be classified into six categories. These categories are "minor,' "light," "moderate," "strong," "major," and "great" for magnitudes of (3 to 3.9), (4 to 4.9), (5 to 5.9), (6 to 6.9), (7 to 7.9), and (8 or larger), respectively. The historic earthquakes explored herein can be categorized into three groups in light of these categories. In the first group, 12% of the historic earthquakes are classed as "Strong," at which "damage may occur"; the second includes 38% of the earthquakes, which are of the "Major" class, where damage is expected, and the rest are classified as "Great," where significant damage is expected.

According to the current paper, and referring to the literature, varied geotechnical hazards happened after the hit of the earthquakes. The most notable hazards were soil liquefaction, foundation failures, ground movements, and landslides. Furthermore, many ERSs underwent different failure types and were damaged to varying degrees. In addition to the cases presented in the previous sections, further instability problems were explored to record the damage and failure (if any) of ERSs during the past earthquakes. Then a comprehensive summary of all cases was prepared and tabulated as illustrated in Table 1. According to the summary of Table 1, the seismic performance of different full-scale ERSs, under the historic earthquakes, was performed. It has appeared that some of the ERSs performed well, while the seismic performance of other types was low where they were damaged or failed.

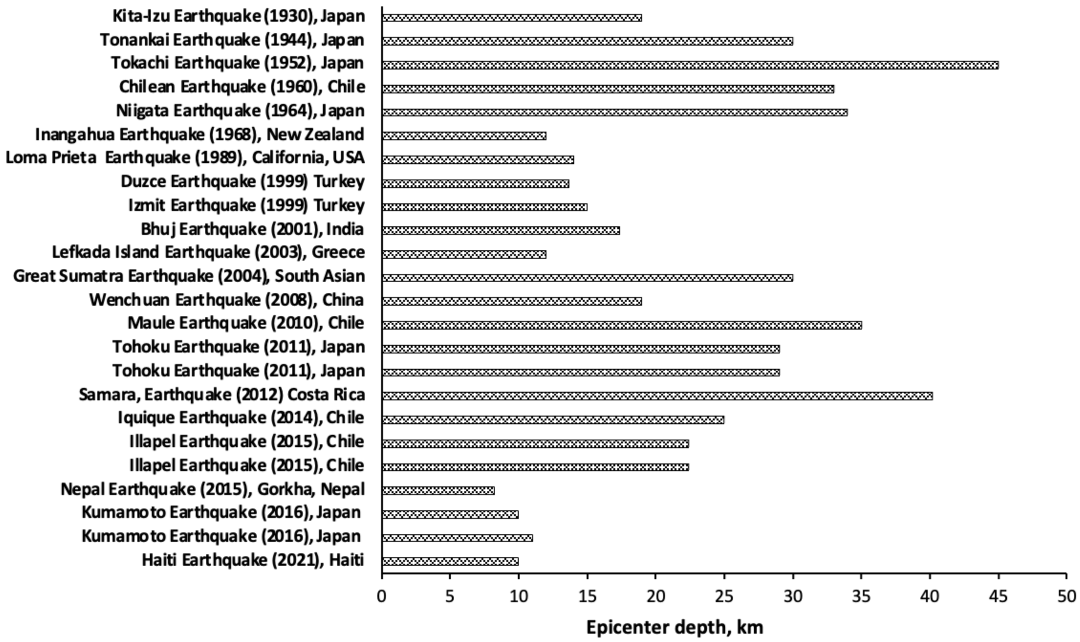

FIGURE 11. The epicenter depths for the Earthquakes considered in this paper

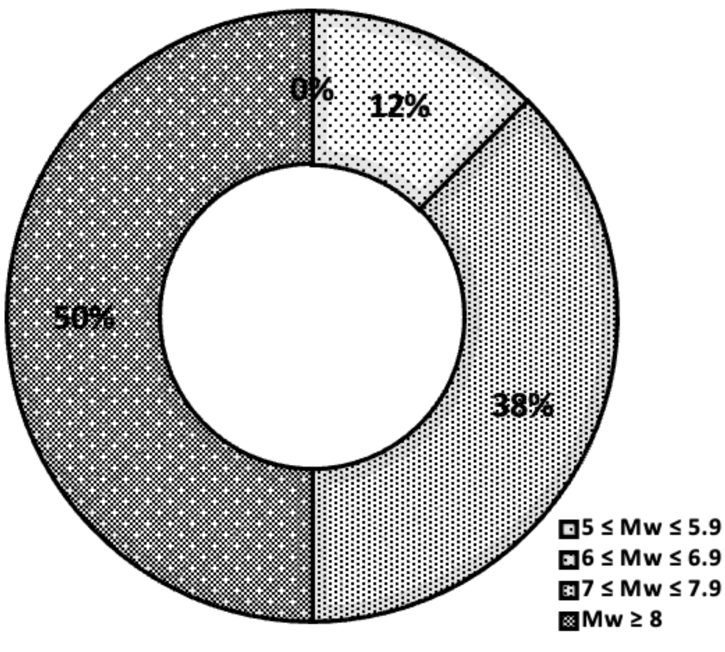

FIGURE 12. Earthquake magnitude classes

The recorded cases of serious or important failures or damages for some types of ERSs during "Strong Earthquakes" (like the Loma Prieta Earthquake (1989) in California (USA)), "Major Earthquakes" (like the Izmit Earthquake (1999) in Turkey and the Duzce Earthquake (1999) in Turkey), and "Great Earthquakes" (like the Wenchuan Earthquake (2008) in China and the Iquique Earthquake (2014) in Chile) are almost few or non-existent. These walls include reinforced-earth concrete RW, mechanically stabilized RWs, and concrete cantilever and basement walls. The good seismic performance of this group necessitates examining the retaining systems' features to benefit from them in future design. For instance, the basement RWs are U-shaped walls and considered non-yielding or stiff restraining walls and exhibit very limited deflection. Their behavior, as non-yielding walls, is greatly affected by the stiffness and rigidity of the foundation soil, especially under the impact of seismic load. It is worth mentioning that these structures have different behavior compared to the yielding type in terms of movement during lateral thrust and the distribution of this pressure, where the shape and value of the lateral pressure are affected by the aforementioned movement. The non-restraining walls can translate or rotate, while restraining ERSs may bend but do not translate. Preventing the movement requires taking special considerations during the design stage, such as adopting a safety factor of high value. (Fang & Ishibashi, 1986; Al-Taie, 2013; Yi, 2013; Wagner & Sitar, 2016b; Al-Taie & Ahmed, 2024c, 2025). This, however, reflects positively on the performance during seismic load. Scholars' observations (Seed & Whitman, 1970; Clough & Fragaszy 1977; Lew et al. 2010a; Mikola & Sitar 2013; Mikola et al. 2016) presented that non-yielding ERSs designed for a static loading (active LEP of the Rankine approach) perform in a good manner under random earthquake loads with ground acceleration of 0.4 g even if they are not designed for seismic pressures without any indication of distress.

On the other hand, the method supporting mechanism of lateral pressure has an important effect on the performance of ERSs. Accordingly, ERSs can be classified as internally stabilized and externally stabilized. The mechanically-stabilized ERSs depend mainly on reinforcing the soil itself; thus, they are internally stabilized. These retaining systems derive the lateral support and stability from soil and strip-reinforcement interaction. Keeping in mind that the reinforcing strips provide the retaining system with more flexibility, such flexibility is an essential parameter in controlling the behavior and performance of ERS. Under dynamic random loading, wall flexibility influences the amount of the moment on the retaining system and the value and shape of LEP. More flexible retaining systems are quite successful. Thus, the high flexibility makes ERSs perform better under the shaking of earthquakes. (Al-Taie 2011; Clayton et al. 2013; Wilson & Elgamal 2015; Das & Sivakugan 2019; Ma et al. 2020; Javadi et al. 2021; Salgado 2022; Yildiz & Kina 2023; Ahmed & Al-Taie 2024; Al-Taie & Ahmed 2025).

On the other hand, ERSs' failures and damage problems during past historic earthquakes can be noted in Table 1. Sundry past historic earthquakes (all investigated categories) made retaining systems fail and caused damage to a variety of degrees. Such instability cases impacted numerous ERSs, including bulkhead ERSs, concrete gravity RWs, waterfront anchored sheet piles, bridge abutments, sheet-pile bulkhead geogrid reinforced earth RWs, quay walls, unreinforced masonry RWs, gabion RWs, cantilever RWs, block stone RWs, unreinforced-concrete block RWs, and unreinforced-masonry RWs. These retaining systems exhibited variable seismic performance based on their type, mass, backfill and foundation conditions, and rigidity. The explored damages from these case histories are as follows:

1. Pulling out the reinforcement strips, or reinforced soil ERS, from their position in the backfill soil side.
2. The corrosion of reinforcement strips inside the backfill, for reinforced soil ERS
3. The failure of the connection and internal slippage of the geogrid, for reinforced soil ERS
4. The damage to facing units
5. The structural failure of the walls
6. The lateral offset of the ERS
7. The sliding of the ERS
8. The outward movement of the ERS
9. Rotation or excessive displacement of the ERS
10. Backfill settlement.
11. Liquefaction of backfilling soils
12. Cracking and settlement of backfill soil

According to the above-listed damages, the following factors can be concluded as the main reasons the ERSs suffer low performance under random loads of earthquakes:

1. observations in the detailing of the structural design process, deficiencies in construction (poor design and construction; non-engineered ERSs).
2. effects related to geotechnical issues, including failure of backfill and foundation soils due to liquefaction phenomena, poor condition of the foundation soils (weak soil at shallow depths), using low-quality geomaterials as a backfilling, inadequate compaction of the backfill materials behind the ERSs, the immoderate seismic LEP from the soil, very high groundwater with a lack

of a suitable drainage system, and the topographical conditions like the location of the retaining system concerning the slopes and mountains.

It was noted that the major instability mechanisms for retaining systems during the hit of strong random seismic loads are sliding, tilting (rocking), toppling (overturning), and collapse (fracture). The liquefaction of the soil beneath the retaining system footing and lateral movement led to some of these mechanisms. Furthermore, constructing a retaining system on shallow footing may cause excessive tilting and internal failure. Nevertheless, the internal failure of retaining systems can be minimized by utilizing different structural monotonic members, such as counterforts and relief shelves, or by adopting rigid footings to minimize the LEP. According to (Yi 2013), as mentioned by (Al-Taie & Ahmed 2024b), shallow footings are non-rigid bases; during seismic events, ERSs constructed on such footings may be subjected to 0.15 to 0.17 additional LEP compared to ERSs founded on rigid bases. (Yildiz & Kina 2023) stated that the movement of heavy RWs constructed on rock is relatively restricted. At the same time, the magnitude and shape of LEP behind retaining systems are greatly affected by random earthquake loads; they are unique and nonlinear (Annapareddy & Pain 2021). Accordingly, it is necessary to emphasize the importance of adopting relevant design criteria in the evaluation of the seismic performance of retaining systems. Criteria based on the permanent displacement of the retaining system are recommended in some literature (Takahashi et al. 1999).

According to this summary, it seems that the type and frequency of failures and damages are a critical concern in the design of ERSs under seismic loadings. In brief, it is clear that, in many cases, the damage has taken place due to the augmentation of lateral thrust and the generated movement of ERSs under the shaking of earthquakes. More importantly, considerable damage has occurred due to the failure of backfill and/or foundation soil, and, additionally, internal failure of non-engineered ERSs.

TABLE 1. ERSs damages due to earthquakes from different countries

Earthquake	Earthquake Details	Type of ERSs	Recorded Damages	References
Kita-Izu Earthquake, Japan, 1930	Northern Izu Peninsula, Japan., Ms=9.5, and depth of 19 km	Gravity RWs	Failure was reported for gravity-type RWs	(Amano et al. 1956; Seed & Whitman 1970)
Tonankai Earthquake, Japan, 1944	Empire of Japan, Tōkai region, Ms=8.1, Mw=8.2, and depth of 30 km	RWs	Sliding was reported for RWs	(Amano et al. 1956; Seed & Whitman 1970; NGDC/WDS 1972)
		Bulkhead RW with relieving platform	Outward movement reported for the wall	
		RWs with pile support	The ERS was moved outward	
Tokachi Earthquake, Japan, 1952	Tokachi District, Hokkaidō, Japan, .Mw=8.1, and depth of 45 km	Gravity RWs	Outward movement reported for the wall	(Amano et al. 1956; Seed & Whitman 1970; Utsu 2004)
Chilean Earthquake, Chile, 1960	The coast of southern Chile, M=9.5, and depth of 33 km	About 274 m of length reinforced-concrete gravity wall built on concrete caisson filled with soil	Over the total length, the wall overturned	(Seed & Whitman 1970)
		Waterfront anchored sheet pile RW	The wall was pushed about 91cm outward over a length of 381m.	
Niigata Earthquake, Japan, 1964	Niigata Prefecture, Japan, M=7.5 or 7.6, and depth of 34 km	Sheet-pile bulkhead	Extensive failures (outward movement) were reported due to liquefaction of backfilling soils.	(Hayashi et al. 1966; Seed & Whitman 1970)
Inangahua Earthquake, New Zealand, 1968	Inangahua town, Murchison, M=7.1, and depth of 12 km	Bridge abutments,	Geotechnical damage was recorded as backfill settlement in approaches.	(Ostadan, 2005; Lew et al. 2020)

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Loma Prieta Earthquake, California, USA, 1989	Santa Cruz, M =6.9, and a depth of 14 km	Reinforced-earth concrete RWs with a height of 5m to 10m	The ERS was performed well without recording any damages	(Benuska 1990; Whitman 1991; Nova-Roessig 1999; Lew 2012; Wagner & Sitar 2016a, 2016b; Choudhury & Rajesh 2020; Krishna & Katsumi 2020)
		A 4.4m high geogrid-reinforced earth RWs	A lateral movement of 88cm was recorded at the top of the wall	
		Mechanically-stabilized RWs	No indications of important residual movements were observed.	
		Waterfront RWs	Damage was reported for many of these structures	
		Basement RWs	Damages were not reported for basement-type RWs	
Izmit Earthquake, Turkey, 1999	The Kocaeli Province, Turkey, 17 August, 1999 M=7.6, and a depth of 15.0 km	Reinforced soil RWs	These walls exhibited superior performance during the Izmit Earthquake, 1999	(Pamuk et al. 2005)
Duzce Earthquake, Turkey, 1999	Duzce, Turkey, 12 November, 1999 M=7.2*, and a depth of 13.7 km	ERSs	Under the Duzce Earthquake, no substantial damages were experienced for ERSs	(Rathje et al. 2006; Gur et al. 2009)
		Semi-basement RWs	Due to the poor design, the RWs underwent damages under the Duzce Earthquake.	
Bhuj Earthquake, India, 2001	Kachchh District, Gujarat, India, M =7.6 and a depth of 17.4 km	Waterfront RWs	Damage was reported for many of these structures	(Gupta et al. 2001; Choudhury & Rajesh 2020; Krishna & Katsumi 2020)
Lefkada Island Earthquake, Greece , 2003	Lefkada Island, Ionian Sea, Greece, M =6.2, depth 12km	Small gravity-type concrete quay RWs	Rotation or excessive displacement of the RWs	(Gazetas 2004; Gazetas et al. 2005; Karakostas et al. 2005; Pitilakis & Roumelioti 2013)
		Quay wall	The summary of damages under earthquake includes the rotation or excessive displacement of the RWs, complete overturning of the RWs, lateral spreading, and cracking and settlement of backfill soil	
		(5m x 5m) rigid gravity (concrete) quay RWs	The recorded damages include rotation and lateral displacement of the quay RWs	
Great Sumatra Earthquake, South Asian, 2004	South Asian Sumatra, M=9.2 and depth 30 km	Waterfront RWs	Damage was reported for many of these structures	(Lay et al. 2005; Choudhury & Rajesh 2020; Krishna & Katsumi, 2020)
Wenchuan Earthquake, China, 2008	Province of Sichuan, M=8.0*, and a depth of 19 km	ERSs	No collapse or substantial damage was reported under the Wenchuan Earthquake	(Sitar et al. 2012; Mikola et al. 2016; Khan et al. 2023)

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Tōhoku Earthquake, Japan, 2011	Oshika Peninsula, the Tōhoku region, JapanM=9.0 to 9.1 and depth 29 km	Seawalls	Damage was reported for many of these structures	Kaneko & Kumagai 2011; Kuwano et al. 2014; Choudhury & Rajesh 2020; Krishna & Katsumi 2020)
		Reinforced soil RWs	Under the Tohoku Earthquake (2011), about 90% (or more) of the reinforced soil ERSs were undamaged, while a small number of these walls (less than 1%) were seriously damaged.	
		A 10 m height steel strip RW, with steel strips spaced every 0.75 m, constructed on poor foundation soil.	The collapse occurred due to the sliding of poor foundation soil about 7 meters horizontally under the seismic motion effect	
		A 5 m height geogrid RW (with facing of wire mesh and primary and secondary geogrid layers of 1.2 m and 0.6m spacing, respectively)	The system collapsed during earthquake impact due to connection failure and internal slippage of the geogrid. The very high groundwater with a lack of a suitable drainage system led to the walls' collapse	
		Steel strip RWs with multi-anchors and facing units (concrete panels)	Damage during the Tohoku Earthquake (2011) was noted at a few points in the facing units,.	
		Reinforced soil RWs	A number of reinforced soil RWs were barely damaged. The real performance of the reinforced ERSs under seismic loading is notably greater than their design target.	
Samara, Costa Rica, Earthquake, 2012	Costa Rica, M =7.6, depth of 40.2 km	ERSs	Failures were noted for poor quality ERSs with poor soil condition.	(Rollins et al. 2013)
Iquique Earthquake, Chile, 2014	Iquique, Chile's coast, Chile, on 1 April, 2014, M =8.2, and a depth of 25 km	Concrete cantilever RW	Rotation of concrete cantilever RW about its base was observed under the ground motion in the Iquique earthquake, 2014	(Wagner & Sitar 2016b)
		Unreinforced masonry RWs.	The walls failed due to poor design, where the wall was constructed with no-reinforcement steel	(Rollins et al. 2014; Wagner & Sitar 2016a)
		Mechanically stabilized soil RW.	The walls failed due to poor construction materials where its reinforcement strips were corroded inside the backfill	
		Quay walls.	The walls failed due to liquefaction of backfilling soil	
		Gabion RW	Minor displacement (with no failure) was reported for the gabion RWs	
		Basement RW	No damages were reported for basement-type ERSs	

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Illapel Earthquake, Chile , 2015	Coquimbo region, Chile, on 16 September, 2015, M=8.3-8.4, and a depth of 22.4 km	ERSs Waterfront gravity RWs	No damages were reported for ERSs backfilled with non-liquefiable soil. Failure was noted for gravity RWs and this was attributed to the susceptibility of soil under footing to liquefaction	(De Pascale et al. 2015)
Nepal Earthquake, Gorkha, Nepal, 2015	Gorkha, Nepal, M=7.8 and depth 8.2 km	Reinforced RW and gravity RW	The joints between the RWs were damaged. Cracks were observed in the gravity RW	(Okamura et al. 2015)
		Different ERSs	It was reported that no failures and minimal damage to ERSs during the 2015 Gorkha, Nepal earthquake.	(Hashash et al. 2015)
Kumamoto Earthquake, Japan, 2016	Kumamoto, Kyushu, Japan, Sequences of Earthquakes M=6.2 at a depth 11 km, M=7.0 at a depth 10 km	Gravity and cantilever-type RWs (with heights ranging from 2 m to 3 m), and reinforced soil walls.	The RWs were structurally damaged. In addition to the structural failure, the cantilever -type RW was excessively tilted. Significant damage was noted for reinforced soil walls. This damage was represented by the pulling out of the reinforcement strips from their position in the backfill soil side.	(Mukunoki et al. 2016; Kayen et al. 2016; Kiyota et al. 2017; Anderson et al. 2023)
		Block Stone RWs Unreinforced-Concrete Block RWs Unreinforced-Masonry RWs	The unreinforced stone masonry RWs were damaged. The damages reported for unreinforced concrete block RWs are lateral offset (20cm offset was recorded for taller RWs and 40cm for the shorter RWs. The unreinforced masonry RWs showed diagonal cracks; these walls were fractured	
Haiti Earthquake, Haiti, 2021	Haiti, M=7.2 at a depth 10 km.	ERSs	During the Haiti Earthquake, in 2021, failure was reported for ERSs due to the impact of filling rock.	(Dashti & Ganapati 2021)

CONCLUSION

The systematic review and analysis of the performance issues of ERSs are regarded as of great significance in geotechnical engineering. These issues are related to a large number of features concerning the retaining wall itself and the foundation and backfilling geomaterials. The kind and geometry of ERSs, characteristics of foundation and behavior of backfill geomaterials, drainage provisions, lateral thrust and its degree of linearity, etc. In reality, the evolution in the analysis of ERSs is critically dependent on a good understanding of the failure modes for such structures. Within the circumstances of the explored earthquakes, it is obvious that some ERSs exhibit good performance under earthquake loading (e.g., basement walls, RWs with tied-back systems, and RWs with reinforced soil), while others do not (like bulkhead and seawalls, concrete gravity and cantilever RWs, block stone and unreinforced-concrete and masonry blocks RW, segmental-type and geosynthetic reinforced-soil RW, geogrid reinforced and crib RWs, etc.).

It can be said that the first effective factor that ensures high seismic performance of walls is good design, even if only for static loads. It has been proven in the literature that satisfactorily designed retaining systems for static loadings exhibit good performance when earthquake shocks occur. Such performance is noted under earthquakes with acceleration values ranging from 0.2g to 0.4g, or more. This, however, is related to the type of retaining system and design method used. The flexible systems show higher performance as they have higher ductile capacity. Meanwhile, the methods that allow permanent displacement are preferable in the design.

There are probably several reasons that lead to the weak seismic performance of retaining walls during an earthquake. Such as failure to adhere to technical design and construction specifications, mass and rigidity of the retaining system, various geotechnical issues of backfill and foundation soils, or the immoderate seismic lateral thrust.

The recorded main seismic instabilities of the retaining systems are mainly represented by sliding, tilting (rocking), toppling (overturning), and collapse or fracture. The lateral spreading or the liquefaction of soil behind or beneath these systems and shallow foundations under their bases are among the reasons for such mechanisms. Anyhow, the internal failure of retaining systems can be minimized by utilizing different structural monotonic members, such as counterforts and relief shelves, or by adopting rigid footings to minimize the LEP, or by including suitable reinforcing materials (distributed with a relevant vertical distance) in the backfill soil, or by using geomaterials of an appropriate quality with good compaction.

In the context of what has been presented regarding the varying effects of destruction and damage caused by earthquakes on different types of ERSs, it is necessary to improve seismic design methods for these important structures. To ensure minor damage, a high level of requirement may be adopted. Among the proposed methods for improvement is introducing a reduction factor into the values of the "horizontal acceleration coefficient" calculated from "peak horizontal ground acceleration." The reduced values of this factor are to be adopted when conducting stability calculations (internal and external) for the RWs based on the method of Mononobe and Okabe.

DECLARATION OF COMPETING INTEREST

None.

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