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The Performance of Reinforced Earth Structures in the vicinity of Kobe during the 1995 Earthquake

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Summary The Great Hanshin Earthquake caused widespread damage on 17th January, 1995, particularly in the Kobe area. The magnitude was measured at 7.2 on the Richter scale and the seismic intensity reached 7. Maximum horizontal ground accelerations greater than 0.8g and vertical accelerations greater than 0.4g were recorded. More than 120 Reinforced Earth structures were inspected, which were close to the main fault line. Of these, 70% were higher than 5 metres. The damage to the Reinforced Earth structures was minimal. Only 8% showed minor cracking of isolated concrete panels and 2% demonstrated some relatively minor movement of the structure. No structures failed. Inspections of the Reinforced Earth structures have confirmed their strength and flexibility. Pseudo-static analyses of selected structures has indicated that present design methods may be very conservative as the structures were likely to have withstood conditions greatly in excess of their design condition.

1. THE EARTHQUAKE

The Great Hanshin Earthquake which shook the Southern Hyogo Prefecture of Japan, occured at 5:46 am on 17th January, 1995. The earthquake was not in the plate, but in the earth crust above the plate. It came from the reactivation of an active fault (the scars of an old earthquake) rather than the release of strain built up between plates. For its type, it was one of the most severe in about 50 years. It measured 7.2 on the Richter scale, with its epicentre at the northern tip of Awaji Island and at a depth of about 14km.

Seismic intensity of 7 occured over areas of Hanshin and Awaji Island along the main fault lines where many buildings were damaged. Such intensity equates roughly to ground acceleration greater than 0.4g. According to the Kobe Meteorological Observatory, the horizontal acceleration exceeded 0.8g (north/south) and 0.6g (east/west) and the vertical acceleration was relatively high compared with the horizontal acceleration - over 0.3g.

The damage was most concentrated along the direction of the main fault lines radiating from the epicentre (Figure 1). Local conditions of the sites contributed to the ground intensity and many structures founded on alluvial soils were destroyed. Transportation infrastructure was severly damaged in some areas and many were out of operation for several months.

The elevated Hanshin Expressway, Kobe Line, toppled over 600m. Also the Chugoku Motorway and the Meishin Highway elevated bridges collapsed and over 100 piers fractured. Japanese Railways new Sankyo Trunk Line had over 700 bridge piers broken and bridge beams collapsed in 8 places, while over 170 bridge beams were seriously damaged on the new Tokaido Trunk Line.

The Port of Kobe was extensively damaged by soil liquefaction and lateral spreading, with all but 7 of its 186 berths unable to be used. About 900m of quay walls around the islands collapsed.

In the Nishinomoya area numerous landslides and rockfalls occured with one in Nikawa burying 34 people.

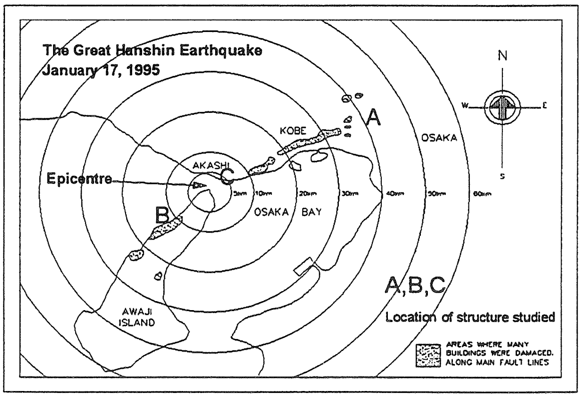


Figure 1. Site Map and Location of Structures

Significant ground movements were observed in Awaji Island - up to 1.7m of strike slip and 1.3m of vertical slip. The new Akashi Kaikyo suspension bridge (under construction) linking Awaji Island and Akashi spanned the epicentre and the overall length of the bridge increased by 1.2m, with one tower moving 1.3m from its original position, increasing the main span of 1,990m by 0.8m.

2. REINFORCED EARTH

2.1 Application

Since the Reinforced Earth technology was invented by Henri Vidal over 30 years ago, more than 20,000 projects have incorporated more than 10 million square metres of Reinforced Earth around the world. It was introduced to Japan over 20 years ago and since been used in more than 5,000 structures incorporating more than 2 million square metres of wall face through its licensees, Sumitomo Corporation (in conjunction with Hirose and Company) and Kawasho Corporation.

2.2 Seismic Behaviour

2.2.1 Studies

Experiments and studies related to the performance of Reinforced Earth structures subjected to seismic motion began before 1970. Researchers were particularly active in countries where earthquakes are a fact of life.

Richardson and Lee (1975) undertook the first major study into the seismic behaviour of Reinforced Earth which was refined by Richardson (1978) to take into account dynamic strain, damping and relative stiffness factors. Seed and Mitchell (1980) reviewed and evaluated the seismic design of Reinforced Earth for Terre Armee Internationale (TAI), from which were synthesised a number of simple rules for pseudo-static design, whose validity has been confirmed since by finite element studies carried out by TAI (1989).

In Japan, research has been reported by Uezawa at the Japanese National Railways (JNR) and Chida at the Japanese Institute of Research of Public Works

2.2.2 Performance

The excellent performance of Reinforced Earth in regions exposed to earthquakes has been confirmed by its experience in events such as Italy in 1976, Japan in 1983 and 1995, Belgium in 1983, Mexico in 1985, New Zealand in 1987 and California in 1989 and 1994.

Two full-size Reinforced Earth structures have been tested by subjecting them to forced vibrations - one in France at Triel in 1975 and the other in the United States at Millville in 1983. Generally, it has been demonstrated that there is no residual deformation for accelerations up to 0.3 - 0.4g. Furthermore, there is relatively little amplification of the average horizontal acceleration or modification of the active zone geometry, and the dynamic reinforcement tension is well predicted from the inertia force of the active zone.

More recently, full scale dynamic testing of Reinforced Earth structures by the Ministry of Construction in Japan undertaken on a large (12m x 12m) vibration table at the Science and Technology Agency at Tsukuba city, near Tokyo, on walls up to 6m high, have confirmed their performance under seismic accelerations up to 0.6g.

The key features which have been identified in the Reinforced Earth system which are important from a seismic performance perspective include

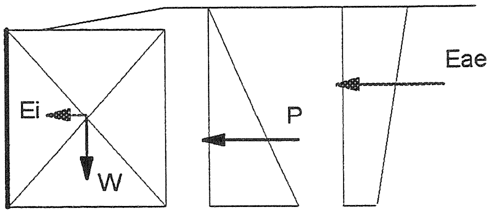
- the high tensile strength of the steel reinforcing strips
- the high shear strength of granular earth fill
- the flexibility of the system

Reinforced Earth structures are designed with the ability to deform and dissipate energy under extreme dynamic loads. In particular, the facing system is specifically designed to articulate as the earth mass deforms to allow the forces to be redistibuted within the system. This is a critical property which is often ignored at great risk by less experienced practitioners.

2.3 Seismic Design

The psuedo-static seismic design of a Reinforced Earth structure developed from the research of the TAI Group, involves the determination of both internal and external dynamic forces which are additional to the static earth pressure and applied loads. Test data by TAI has indicated a relationship between ground acceleration and peak acceleration in the structure which allows for the determination of horizontal and vertical dynamic loads.

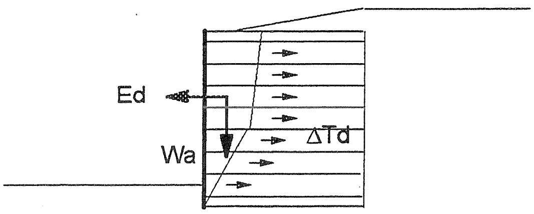
External stability is determined from the combined effect of the dynamic load of the structure mass and an additional applied earth pressure, typically calculated according to the Mononobe-Okabe method. This is described in Figure 2. The combined effect is factored to allow for the reduced likelihood of concurrent action.



Ei = W (a/g)
Eae from 'Mononobe Okabe' calculation

Figure 2. External stability

Internal stability is determined by calculating the horizontal dynamic force due to the weight of the active zone inside the structure, as shown in Figure 3. This load is distributed to the reinforcement in proportion to the resistance available at each level. This usually results in a larger dynamic load increment near the base of the structure where more reinforcement is in the resistant zone.



$Ed = Wa\,(a/g)$
ΔTd in proportion to strip resistance

Figure 3. Internal stability

3. PERFORMANCE OF REINFORCED EARTH STRUCTURES AT KOBE

3.1 Inspections

A total of 124 Reinforced Earth structures were inspected within a 50km radius of the epicentre. Seismic intensities varied from 4 to 7. Approximately 21 of these structures were in areas subject to seismic intensity 7, where the greatest damage was observed.

The results of the inspections have been detailed by Kawasho (1995) and Hirose (1995)

The structures ranged in height from 1.5m to 16.5m, with a majority (70%) greater than 5m high and 13% higher than 10m (Figure 4). The inspections of the Reinforced Earth structures focussed on the following main areas:

- Damage to the wall facing panels (cracking or breaking of the concrete skin), joints, coping and footings
- Damage to the parts of the structure connecting to other structures
- Movement of the wall (horizontal displacement of the face, settlement of the wall) or associated ground

The structures in this region were not designed for a ground acceleration greater than 0.15g.

3.2 Observations

The overall observations on the structures were as follows:

- 3 structures (2%) indicated some damage to the wall facing and movement of the wall and adjacent ground
- 7 structures (6%) indicated some damage to the wall facing and to the adjacent ground
- 22 structures (18%) indicated no damage to the wall despite some damage to the adjacent areas.
- 92 structures (74%) indicated no wall or adjacent area damage

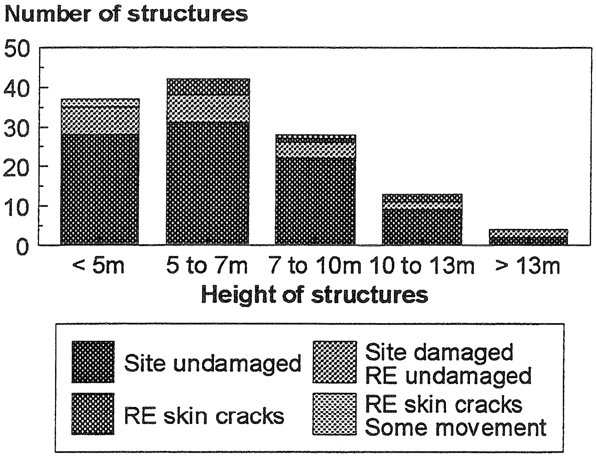


Figure 4. Summary of damage

3.3 Specific Structure Studies

For the three structures which have exhibited some movement, the observed damage and deformation is described and a simple analysis has been carried out using the seismic design methods developed for Reinforced Earth structures (TAI, 1989) to assess the degree of effect anticipated for the inferred site and loading conditions of each structure. In addition, a dynamic slip circle analysis has been undertaken using STARES, a modified Bishop method which takes into account the effect of reinforcement. STARES was jointly developed by Centre for Geotechnical Research at The University of Sydney and The Reinforced Earth Company

The analyses were carried out for both static and dynamic conditions (ground acceleration = 0.4g) assuming standard strength parameters for the Reinforced Earth structural fill (ϕ = 36deg) and retained fill (ϕ = 30deg) for the purposes of comparison. Live load was neglected.

Results of the analyses are summarised in Table 1.

3.3.1 Structure A

This structure was reported by Hirose (1995) and is in Atami City, approximately 41km away from the epicentre, but which is situated along the fault line in an area subject to seismic intensity 5 to 6 (approximately 0.08 to 0.40g acceleration). The height of the wall is only 3.75m with reinforcing strip lengths between 2.5 and 4.5m. A section of the structure is shown in Figure 5.

Some minor cracks and breakages were found in the concrete facing panels, together with some opening of the joints. Settlement was observed on the ground surface and in the foundation section of the adjacent gymnasium building.

The wall face deformed up to approximately 100mm, however the relative wall (tilting) movement was less than 3% of the wall height.

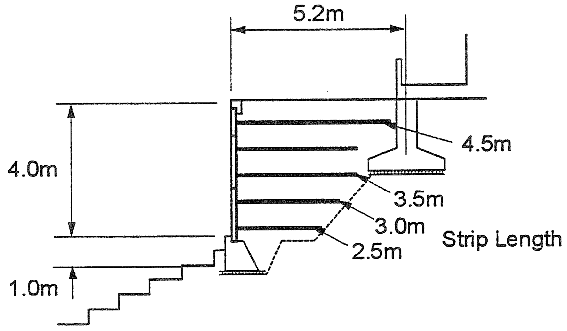


Figure 5. Structure A, Section

Under the dynamic loading conditions, the TAI design method indicated lower, but adequate, factors of safety for base sliding and reinforcement tension, despite an increase in reinforcement tension by a factor of 2.3 (top layer) reducing to 1.2 (base layer). Reinforcement friction capacity was predicted to be exceeded only in the top layer. (minimum factor of safety 0.7).

Analysis using STARES indicated a reduction of the factor of safety from 1.3 to 0.8 (see Figure 6)

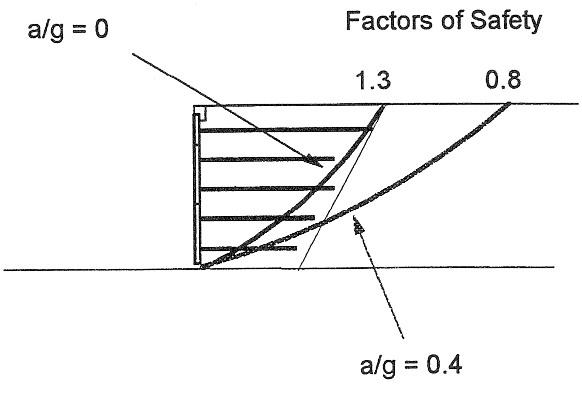


Figure 6. Structure A, Stability

Inspection of the structure has not, however, indicated any loss of frictional resistance in the upper layers, or basal movement.

3.3.2 Structure B

This structure, reported by Kawasho, is on Awaji Island, approximately 13km from the epicentre. It is in an area subject to significant damage and a seismic intensity of 7 (greater than 0.40g acceleration). The wall is a maximum height of 9m supporting a 1 in 1.8 slope approximately 10m high. Reinforcing strip lengths vary from 5 to 7.5m (Figure 7).

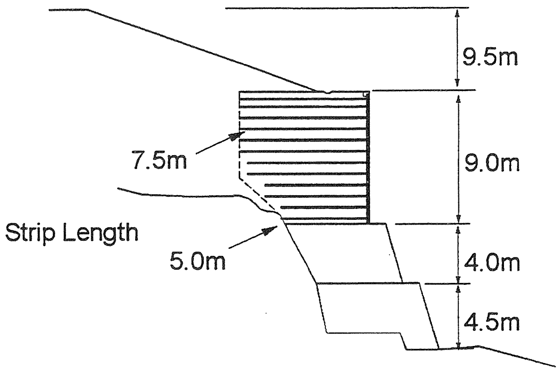


Figure 7. Structure B, Section

Some minor cracks were found in the concrete facing panels, but no cracking was observed in the coping on top of the facing. Water channels in front of the structure had collapsed and cracks were observed in the natural ground slope. Masonry blockwork structures adjacent to the wall also collapsed.

The deformations of the wall face relative to the toe of the wall were measured three weeks after the earthquake (on 8th Feb 1995). The maximum forward movement at height 7.5m was measured at 109mm at the higher section (1), and 55mm at the lower section (2). At a height above the base of 3.0m, the forward movement was generally less than 20mm. The relative (tilting) movement has been less than 1.5%.

Under the dynamic loading conditions, the TAI design method indicated low base sliding factors of safety (reducing from 1.5 to 0.8), however, the minimum reinforcement tension factors reduced only from 2.0 to 1.7 despite an increase in reinforcement tension by a factor of 2.0 (top layer) to 1.2 (base layer). Reinforcement friction capacity, was maintained with a minimum factor of safety of 1.4.

Analysis using STARES indicated a reduction of the factor of safety from 1.2 to 0.7 (see Figure 8).

This was a more critically loaded structure due to the sloping embankment which significantly increased the calculated dynamic load applied to the structure and effected the external stability calculation. This was more noticeable in the STARES analysis which considered the full dynamic mobilisation of the mass above the slip circle.

Internal stability was maintained due to the increased frictional resistance available from the longer reinforcement strips and the depth of overburden due to the slope.

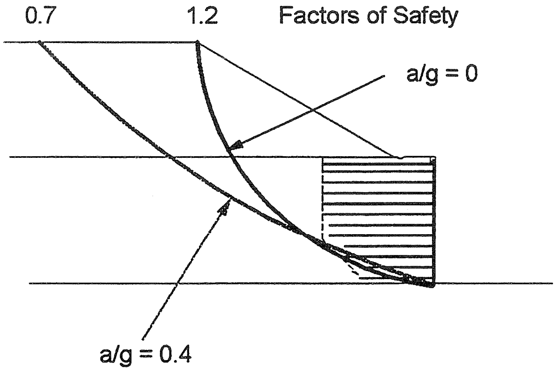


Figure 8. Structure B, Stability

Inspection of the structure has not indicated any base movement.

3.3.3 Structure C

This structure, reported by Kawasho (1995), is near Akashi City, approximately 6km from the epicentre. Seismic intensity in the area is estimated at 6 (approximately 0.25 to 0.40g acceleration). The wall is 4.5m high supporting a horizontal surface, with reinforcing strip length of 5m (Figure 9).

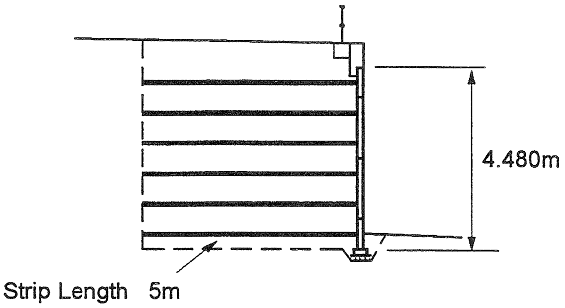


Figure 9. Structure C, Section

Some minor cracking and opening of panel vertical joints was observed in the wall facing. Cracks were observed on the fill surface, adjacent stone masonry was cracked and private house retaining walls collapsed.

The deformations of the wall face relative to the toe were measured at four locations along the wall, three weeks after the earthquake (on 7th Feb 1995). The forward movement at height 4.0m varied up to 74mm, while at height 1.5m, the movement varied from 13mm (forward) to 59mm (backward). The relative (tilting) movement was less than 2%.

Under the dynamic loading conditions, the TAI design method indicated similar factors of safety for base sliding and reinforcement tension factors as Structure A. Reinforcement tension more uniformly increased by a factor of 1.7 (top layer) reducing to 1.4 (base layer) but the minimum factor of safety was 3.2. Reinforcement friction capacity was however exceeded slightly in the top layer. (minimum factor of safety 0.8) .

Analysis using STARES indicated a reduction of the factor of safety from 1.7 to 0.9 (see Figure 10)

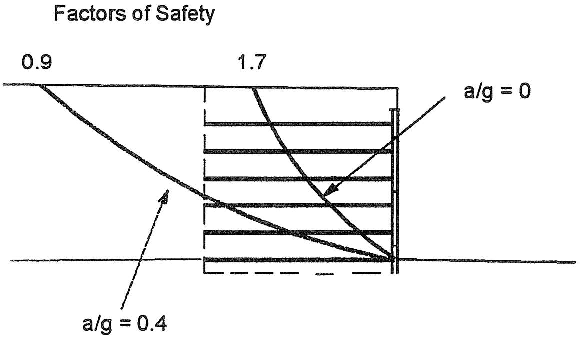


Figure 10. Structure C, Stability

The more uniform increase in tension in the reinforcement and the higher slip circle factors of safety than Structure A were due to the uniform section (reinforcement strip length).

Inspection of the structure has similarly not indicated any loss of frictional resistance in the upper layers, or global movement.

4. COMMENTS AND CONCLUSIONS

The minimal damage to the Reinforced Earth structures, confirmed the experience of the Reinforced Earth Group with the behaviour of such structures in seismically active regions.

The observed deformation of some walls is varied, but relatively minor (less than 3% of wall height). The variability of the movement is most probably due to the variability of the foundation and embankment conditions which have the most influence on structure deformation. The movement itself reflects the flexibility of the system and its ability to absorb energy and redistibute loads, rather than to resist and, therefore, attract loads. No evidence of tensile or frictional failure was observed in the Reinforced Earth structures, notwithstanding that they were not designed for the high accelerations experienced.

The structures which exhibited some degree of movement (A, B and C) have been checked using the TAI design method and a global slip circle analysis, for the general seismic conditions expected in these locations.

With the TAI design method, the response of the structures varies depending on the size and configuration of the structure and its elements. Neglecting external variables such as foundation conditions and embankment properties, it is seen that the increase in tensile loading of the reinforcement (and face pressure) could be between 1.5 and 2.5 that which is predicted from static dead loading conditions, for seismic accelerations up to 0.4g. Despite this, the structures have remained stable and fully serviceable with no loss of long term capacity.

Global stability analyses indicated significant reduction in factors of safety under the assumed conditions, however, this was not reflected in the observed performance.

The simple comparative analysis indicates that both the psuedo-static design method used by TAI and the more traditional global stability analyses appear to be conservative given the apparent conditions of these structures.

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Table 1. Summary of Factors of Safety assessed for Selected Structures

	Structure A	Structure B	Structure C
Base sliding			
- static	4.2	1.5	4.3
- dynamic (0.4g)	1.4	0.8	1.4
Reinforcement tension			
- static	5.1 to 23	2.0 to 10	4.4 to 10
- dynamic (0.4g)	4.2 to 9.9	1.7 to 5.0	3.2 to 6.1
Reinforcement friction			
- static	1.1 to 2.1	1.6 to 4.4	1.2 to 2.3
- dynamic (0.4g)	0.7 to 1.2	1.4 to 2.2	0.8 to 1.7
Slip circle (STARES)			
- static	1.3	1.2	1.7
- dynamic (0.4g)	0.8	0.7	0.9