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The performance of reinforced earth structures in the vicinity of Kobe during the Great Hanshin Earthquake

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ABSTRACT: 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.

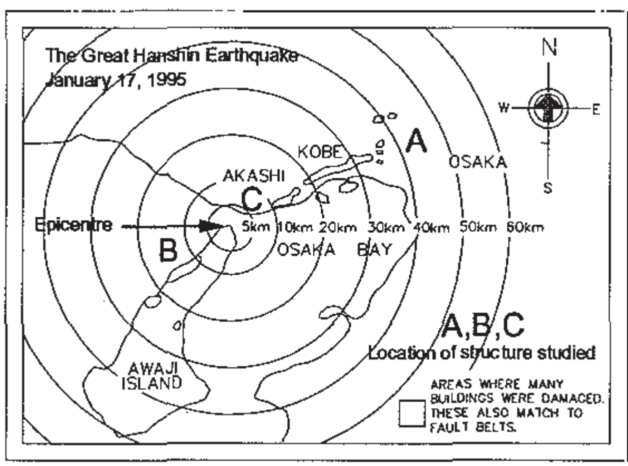

Figure 1. Site Map and Location of Structures

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.

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, 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 later 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 carried out by Uezawa at the Japanese National Railways (JNR) and Chida at the Japanese Institute of Research of Public Works, which has formed the basis for the development of seismic design methods adopted in Japan.

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.

Full-size Reinforced Earth structures have been tested by subjecting them to forced vibrations. One such test has been carried out in France at Triel in 1975 and another 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

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

3. PERFORMANCE OF REINFORCED EARTH STRUCTURES AT KOBE

3.1 *Inspections*

As of February 1995, a total of 812 Reinforced Earth structures had been constructed in the Kinki region (280 in Hyogo Prefecture, 140 in Osaka Prefecture, 90 in Nara Prefecture, 163 in Wakayama Prefecture and 139 in Mie Prefecture)

A total of 124 Reinforced Earth structures were inspected within a 40km 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 reported in detail 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 2). 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

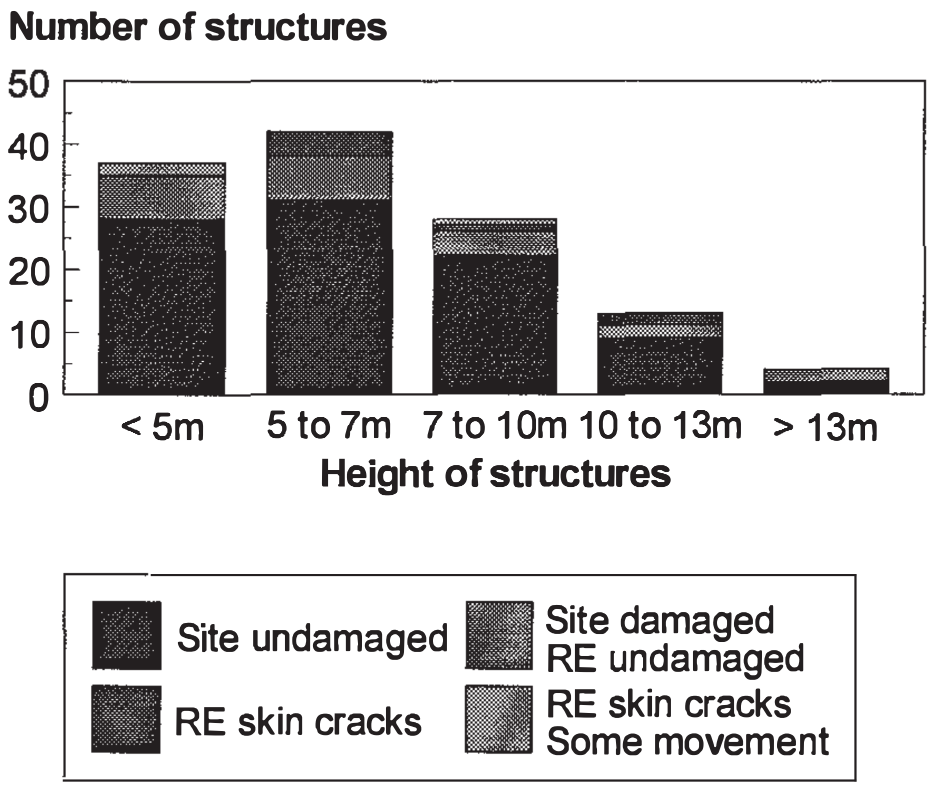

Figure 2. Summary of damage

- Damage to the parts of the structure connecting to other structures.
- Movement of the wall (displacement of facing panels, lateral movement of the wall, or 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

Of the 8% of the structures which exhibited some damage, it can be said that all of them are located in areas that suffered relatively heavy damage from the earthquake, judging by the condition of the houses and structures in their vicinity. In no case was the damage to the surveyed structures great enough to deprive them of their function and ground surface repairs being all that was required from a visual point of view.

The characteristics of the damage which was revealed on the 8% of structures was as follows:

- displacement of the concrete panels
- cracking in the concrete panels
- opening of the vertical joints
- damage at connections to other structures
- damage to foundation or capping structures
- subsidence and cracking of the ground surface

The relationship between the degree of damage and the wall height is shown in Figure 2.

3.3 *Study of Displaced Structures*

The three structures which have exhibited some movement have been studied in more detail.

The observed damage and deformation is described and a simple comparative psuedo-static analysis has been carried out using current seismic design methods developed for Reinforced Earth structures.

These 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.

3.3.1 Structure A

This structure forms an embankment for a housing lot platform in the City of Itami (Hirose, 1995). It is approximately 40km 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 3.

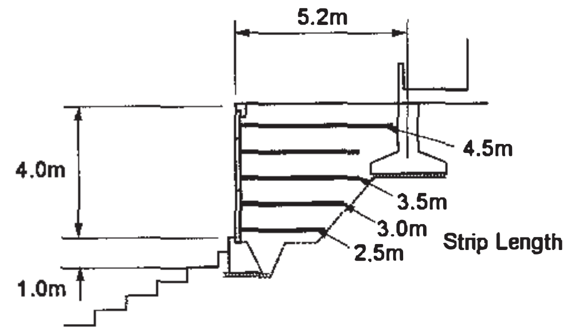

Figure 3 Structure A, Section

The wall face deformed up to 94nun in the middle, corresponding to only 2.5% of the wall height. This is less than the nonnal construction tolerance.

Cracking was observed in 19 panels, or 35% of the total number surveyed. The locations of the cracks were usually in the vicinity of the dowels or the corners where the panels were constrained during oscillations.

Up to 180mm opening of joints with adjacent structures was observed and cracking of the foundation was observed in several places. One joint in the capping concrete had opened by 37nun (at the place where the forward inclination of the wall was the greatest). Other joints with an opening of 10mm were also observed.

Subsidence of the upper ground surface ranged from 200mm to 1,000mm, presumeably due to a shakedown phenomenon from the earthquake.

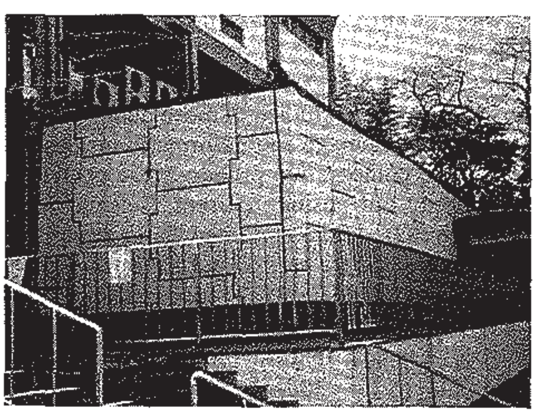

Figure 4. View of Structure A

Psuedo-static analysis indicated lower, but adequate, factors of safety for base sliding and reinforcement tension, despite a potential increase in reinforcement tension by a factor of 2.3 (top layer) reducing to 1.2 (base layer). Despite marginal friction capacity indicated in the top layer, there was no observed loss of pull-out resistance

3.3.2 Structure B

This structure forms an embankment for a roadway in Hokutancho, Awaji Island, approximately 13km from the epicentre (Kawasho, 1995). 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 as shown in Figure 5

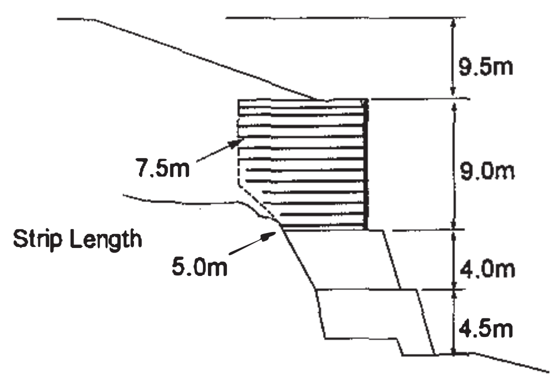

Figure 5 Structure B, Section

The defonnations of the wall face relative to the toe of the wall were measured before the earthquake and three weeks after (on 8th Feb 1995). The maximum forward movement at height 7.5m was measured at 109nun at the higher section (1), and 55nun 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 was, however, less than 1.5%.

Some cracking at the corners of concrete panels was considered to be the result of panels knocking against each other during the motion (particularly pitching)

Damage of the concrete infill between the wall face and the side gutter resulted from differential movement and some displacement was observed between the capping and facing panels.

An adjacent blockwall structure was observed to have displacements of 500mm, while another displayed large cracks.

Psuedo-static analysis indicated potential low base sliding factors of safety (reducing to 0.8), however, the minimum reinforcement tension factors appeared to reduce only from 2.0 to 1.7 despite a possible increase in reinforcement tension by a factor of 2.0 (top layer) to 1.2 (base layer).

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.

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.

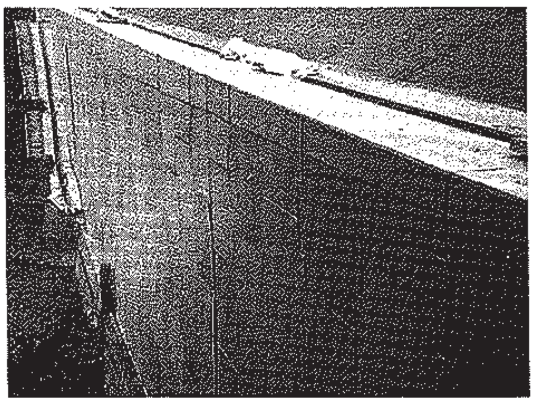

Figure 6. View of Structure B

3.3.3 Structure C

This structure forms an embankment for a parking site in Tarumi-ku, near Akashi City, approximately 6km from the epicentre (Kawasho, 1995). 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 7).

In this site there is an almost centrally located corner panel sandwiched between a front and a lateral face, forming an 'L' shape in plan. As a result of the earthquake, the entire lateral face has been displaced outward by up to 100mm. This appears to be main cause of the observed damage.

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 up to 59mm. The relative (tilting) movement was less than 2%

Cracking of the facing panels was observed in several places on the lateral face. The vertical joints between the panels have opened up by as much as 200mm, due to the relative displacement of the front and lateral walls. Large cracks measuring up to 150mm occurred in the top ground surface, due most likely to the forward displacement of the face. The cracks occured only within the range of the upper part where there was no strip reinforcement.

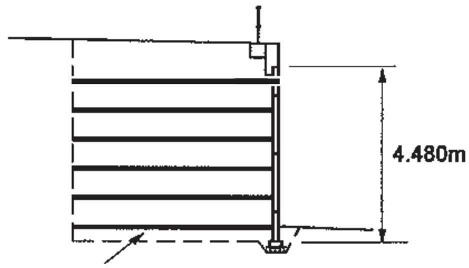

Figure 7. Structure C, Section

Damage to many houses in the hilly residential area of this site was observed - particularly to outer walls, roof tiles and retaining walls.

Psuedo-static analysis indicated similar factors of safety for base sliding and reinforcement tension factors as Structure A. Reinforcement tension is more uniformly increased by a factor of 1.7 (top layer) reducing to 1.4 (base layer) but the minimum factor of safety appeared to remain over 3. Reinforcement friction capacity appeared to be possibly exceeded in the top layer. (minimum factor of safety 0.8) .

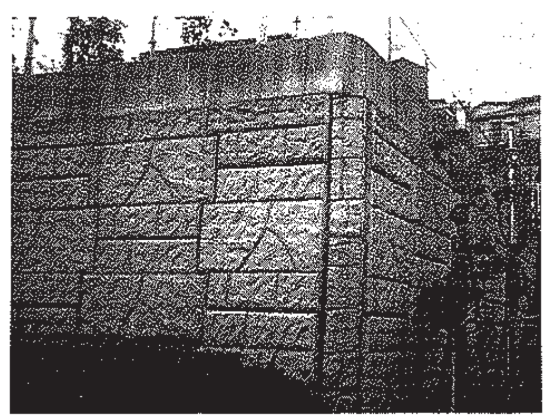

Figure 8. View of Structure C

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

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 damage which was observed on the facing panels was due to the oscillation of the panels during the movement and their restraint at connecting dowels and edges where they came in contact. The relatively minor damage of the panels under such strong movement, highlighted the importance of the flexibility of the whole system (earth mass, reinforcement and facing) to the minimisation of damage.

Slightly more damage occured where the facing came in contact with less flexible contiguous structures (capping, drainage gutters and adjacent walls)

The observed damage was relatively greater for small scale structures, which indicated no magnification of effect for larger structures.

The structures which exhibited some degree of movement (A, B and C) were checked using the typical psuedo-static design method and slip circle analyses, for the general seismic conditions expected in these locations. Neglecting external variables such as foundation conditions and embankment properties, it is seen that despite a possible increase in tensile loading of the reinforcement (and face pressure) between 1.5 and 2.5 that which is predicted from static dead loading conditions, for seismic accelerations up to 0.4g, the structures have remained stable and fully serviceable with no loss of long term capacity. Furthermore, 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 the usual psuedo-static design methods and global stability analyses appear to be conservative given the apparent conditions of these structures.

5. REFERENCES

Richardson, G.N. and Lee, K.L. 1975 Seismic design of Reinforced Earth walls, *Journal of the Geotechnical Engineering Division, ASCE, 101:GT2*, pp 167-188

Richardson, G.N. 1978, Earthquake resistant Reinforced Earth walls, *Proc. Symposium on Earth Reinforcement, Pittsburgh, ASCE*, pp 664-684

TAI 1989, Design for Reinforced Earth Structures subject to Seismic Forces, Documentation, Synthesis and Research results, *Research Report No. 48,* Terre Armee Internationale, Paris

Seed, H.B. and Mitchell, J.K. 1980, *Report for The Reinforced Earth Company*, USA

Kawasho Corporation. 1995, Report of Research on Terre Armee Structures Exposed to the Great Hanshin Earthquake, February.

Hirose and Company. 1995, Field Inspection Report on Reinforced Earth Structure (the Southern Hyogo Prefecture Earthquake), February.

Soga, K 1995, Kobe Earthquake, *ISSMFE News, International Society for Soil Mechanics and Foundation Engineering*, Vol.22, No.2, April.