Performance of a reinforced soil retaining wall during the Christchurch earthquakes

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ABSTRACT

This paper presents a case study of a reinforced soil retaining wall built in the centre of Christchurch in 1991. The wall has a maximum overall retained height of about 3.4m, consisting of a full height pre-cast concrete panel facing with HDPE punched and drawn geogrid attached by casting in during fabrication, and reinforcing a granular backfill material. The wall forms one bank of the River Avon over a 50m length, so that the foundation is situated below river level, and it supports a busy main road and a bus stop. The wall was designed originally by the tie-back wedge method. During the strong earthquakes which shook Christchurch severely during 2010 and 2011, together with many aftershocks, the wall was subjected to intense shaking. Many structures in the area were damaged by this seismic activity, but the reinforced soil retaining wall performed extremely well, with neither signs of distress to the wall itself nor to the supported road. This paper gives full details of the wall as designed, summarises the seismic shaking which took place and presents a back-analysis using the two-part wedge method of analysis, which is being used widely today to design such structures, for both static and seismic conditions. The analysis takes into account peak ground accelerations which are likely to have been experienced in this part of Christchurch during the earthquakes.

Keywords: retaining wall, MSE, reinforced soil, earthquake, seismic performance, back-analysis

1 INTRODUCTION

Reinforced soil structures subjected to strong shaking during recent earthquakes have generally performed very well. Tatsuoka et al (1996) describe a wide range of retaining wall types that were affected by the Hyogo-ken Nanbu (Kobe) earthquake (M = 7.2), which took place in Japan in 1995. They note that reinforced soil retaining walls constructed using polymer geosynthetics performed particularly well and such techniques were used later in the reconstruction which took place. Nishimura et al (1996) describe ten reinforced soil structures constructed using high density polyethylene (HDPE) geogrid reinforcement in the Kobe area, all of which performed very well during the same 1995 earthquake. Four years later in 1999, the Chi-Chi earthquake (M = 7.3) caused extensive damage in central Taiwan, and many reinforced soil structures were affected. Some failures took place, which have been extensively reported. Holtz et al (2001) describe the Chi-Nan University slope failure, and Huang & Tatsuoka (2000) report the failure of two small modular block walls. Many other papers have been devoted to these two failures, which may be attributed to inadequate design and detailing. However the majority of the reinforced soil structures in the epicentral region of the Chi-Chi earthquake performed well, and Dobie (2006) describes two large reinforced soil slopes which survived with negligible effect.

The city of Christchurch in the South Island of New Zealand was shaken severely by a number of earthquakes in late 2010 and 2011. The two main shocks took place on 3 September 2010 and 21 February 2011, with a further significant shock on 13 June 2011. The earthquakes had slightly lower magnitude compared to Kobe and Chi-Chi, but share similarities due to their shallow focal depth and proximity to built-up areas. 20 years prior to these earthquakes, Christchurch City Council had designed and built a reinforced soil structure near the city centre, on the north edge of Hagley Park. The structure is situated between the Avon River and Carlton Mill Road, and is referred to here as the Carlton Mill Road wall. This paper presents a case study of this wall, covering: design & construction, likely ground motions experienced during the recent earthquakes, and finally a back-analysis of the section based on design methods used today.

2 THE REINFORCED SOIL STRUCTURE AT CARLTON MILL ROAD, CHRISTCHURCH

2.1 Outline of the structure

The overall length of the reinforced soil structure at Carlton Mill Road is approximately 100m, with maximum retained height of about 3.4m. The structure has two distinct sections of roughly the same length: the west half consisting of a low height wrap-around steep slope and the east half consisting of a full height pre-cast concrete panel facing. The concrete panel facing forms the higher part of the structure and is founded in the edge of the Avon River. This part of the structure is described in further detail in the sections which follow, and is investigated by back-analysis.

2.2 Full height panel reinforced soil retaining wall

A typical cross-section of the full height panel reinforced soil retaining wall is shown in Figure 1. The pre-cast panels are 2.55m high and 5m long, resting on a pre-cast foundation beam. The geogrid reinforcement used has a width of 1m, so that each panel has four adjacent pieces of geogrid at each level. It is worth noting that this was probably the first use of this form of construction in New Zealand (ie. full height pre-cast panel with cast-in HDPE reinforcement). The upper T-shaped section was cast in-situ to form the pedestrian footpath, which partly overhangs the retaining wall. The upper layer of geogrid reinforcement was designed to take into account the moment generated by loads on the overhang. The design included a live load of 12 kPa on the road and footpath. When viewed today, the pre-cast foundation beam is visible in the edge of the river, so that the retained height effectively extends from the base of the foundation to the top of the fill.

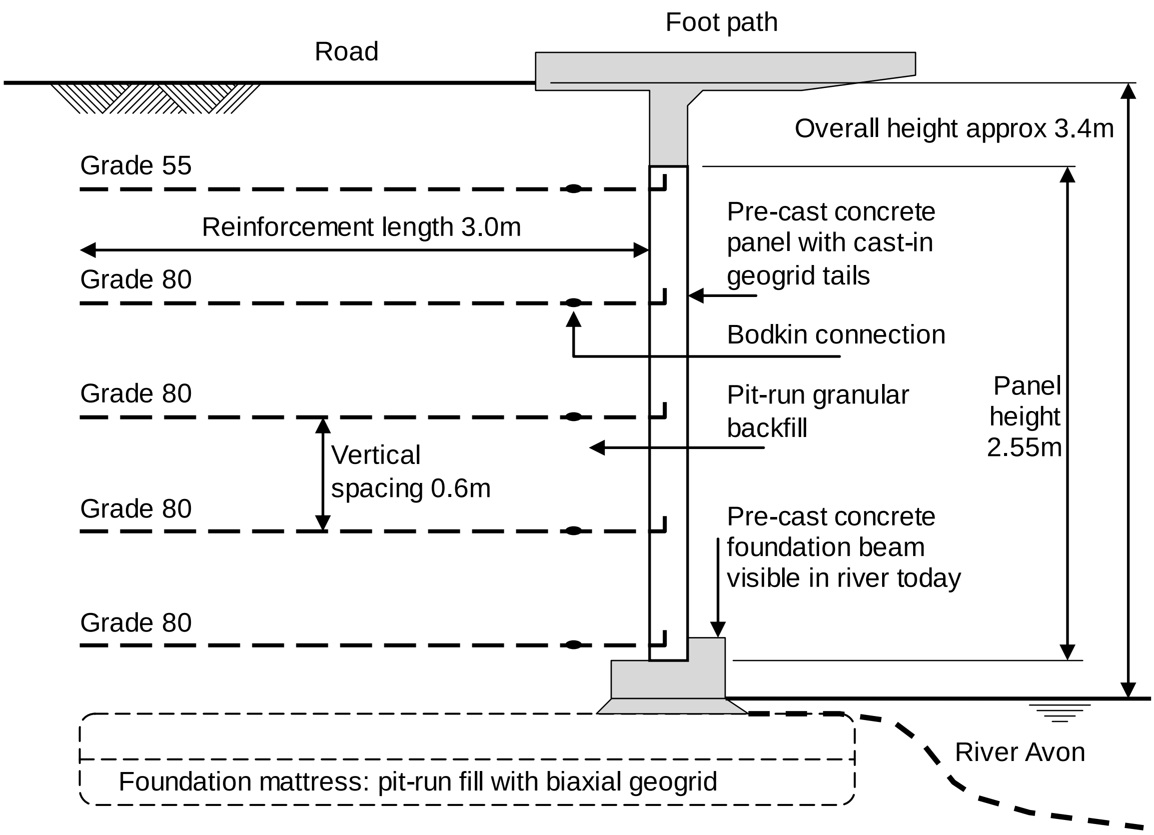

Figure 1. Section through the reinforced soil retaining wall at Carlton Mill Road

At the time of design and construction there were concerns about soft foundation soils, so that a foundation mattress was used to support the reinforced soil structure consisting of 0.5m of pit-run granular material with three layers of biaxial geogrid. The punched and drawn polypropylene biaxial geogrid effectively encapsulates the fill, thereby helping to minimise erosion due to flow in the River Avon.

2.3 Materials and design properties

The fill material is described as "pit-run", and based on photographs it has a high gravel content. The original design was carried out using methods based on ϕ'_{cv} and the value used was ϕ'_{cv} = 32° with bulk unit weight of 20 kN/m^3. Two grades of HDPE geogrid were used as shown on Figure 1, with design properties as given in Table 1. Because the geogrids are cast into the panel, there is a gap between adjacent layers, and the reinforcement coverage is 80%.

Table 1: Properties of the HDPE geogrid reinforcement

Grade	Tensile strength (kN/m)	120 year design strength (kN/m)	Installation damage factor for coarse fill
55	57.0	22.5	1.5
80	81.5	31.6	1.3

Source: BBA Certificate 99/R108 Tensar SR geogrids for reinforced soil retaining wall and bridge abutment systems

2.4 Design

The original design was carried out for the static case using the tie-back wedge method, based on a trapezoidal pressure distribution, using ϕ'_{cv} as mentioned above. This method is not used today, and would be considered conservative compared to current methods. The design indicated that there would be considerable spare capacity in the upper three layers of geogrid, with the lower two being closer to design capacity. The resulting design layout is as shown on Figure 1, and this is examined further in Section 4 of this paper.

3 THE CHRISTCHURCH EARTHQUAKES

3.1 The main shocks

The first of the three main earthquakes occurred in the early hours of 3 September 2010. Although this caused a lot of damage, the epicentre was relatively far from the city centre, so that damage there was not so severe, and it was approximately 36km from the Carlton Mill Road wall. The second earthquake was significantly smaller as regards magnitude and energy released. However the focus was much closer to the city centre and damage there was much more severe. The distance of the second earthquake from the wall was about 10km. Details of both main earthquakes are given in Table 2, together with a third shock which occurred on 13 June 2011.

Table 2: Main details of the Christchurch earthquakes

Date	Magnitude (M_w)	Epicentre	Focal depth (km)	Duration of shaking (sec)	Distance from wall (km)
3 September 2010 4:35:46am	7.0	172.120° E 43.530° S	5	20	36
21 February 2011 12:51:43pm	6.3	172.710° E 43.600° S	5	15	10
13 June 2011 02:20:50pm	6.0	172.740° E 43.580° S	9	15	11

Source: http://earthquake.usgs.gov/earthquakes/

3.2 Strong motion data

There is an extensive network of strong motion recording stations around New Zealand, especially in the more seismically active areas. Detailed data from these stations may be obtained from the Center for Engineering Strong Motion Data (CESMD, website: www.strongmotioncenter.org). For each recording site, the peak horizontal (two components) and vertical accelerations may be obtained. These are generally considered to be PGA (peak ground acceleration) although in some cases the instruments appear to be located significantly above ground level. In order to view this information in a concise fashion, these three values are plotted against epicentral distance on Figures 2 and 3. The three components are denoted by different symbols, with vertical shown as solid diamond shapes.

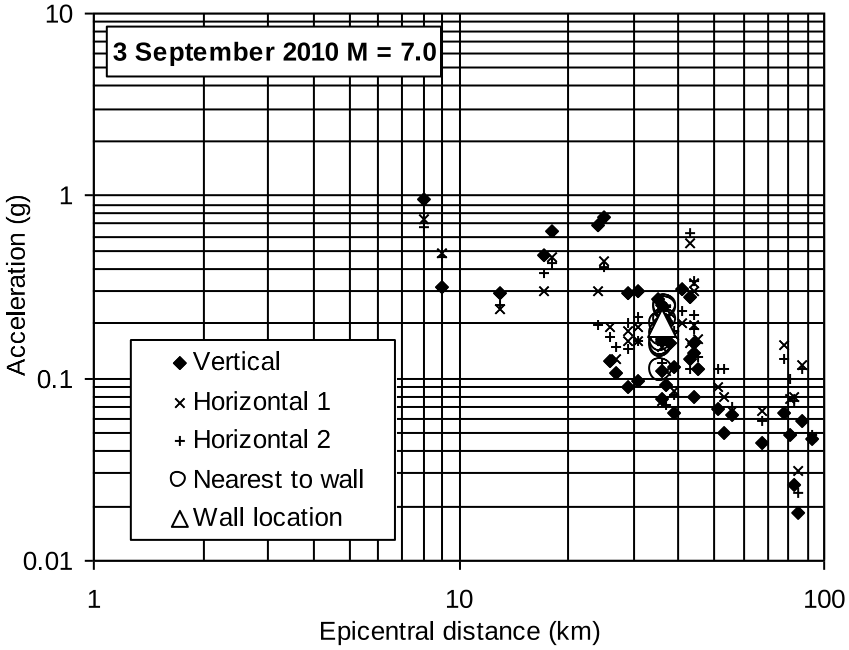

Figure 2. Attenuation of peak acceleration with distance for the 3 September 2010 earthquake

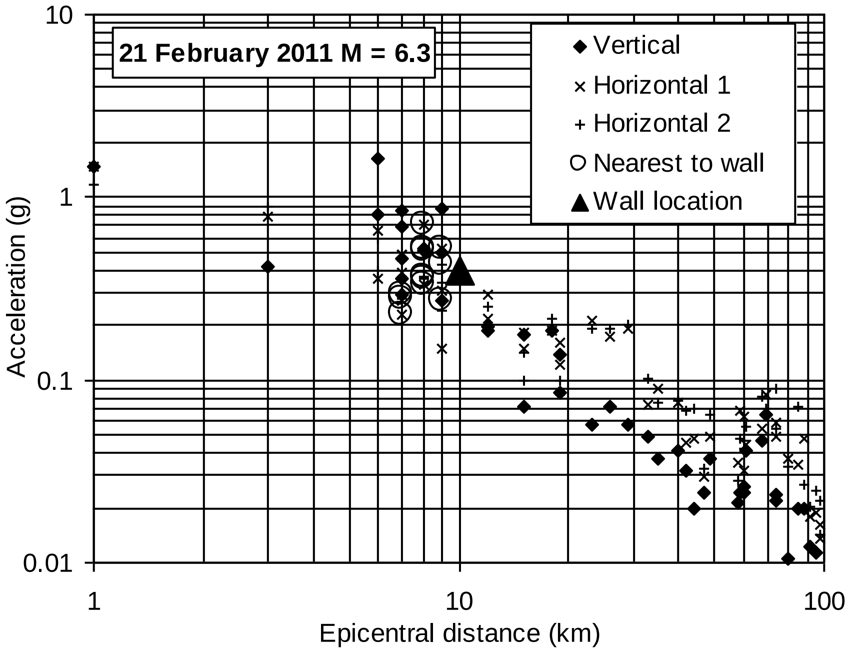

Figure 3. Attenuation of peak acceleration with distance for the 21 February 2011 earthquake

Some important observations may be made from these two plots. Firstly, close to the epicentre, vertical acceleration is generally higher than horizontal, but as epicentral distance increases this trend reverses. This is not surprising based on the nature of the fault movement and the proximity of the closest instruments to the source of shaking. The closest instrument during the 21 February 2011 earthquake is only 1km from the epicentre (ie. horizontally), but depth is 5km, so the massive vertical component is to be expected. A second important point is the nature of the attenuation. There is significant scatter, but at any particular epicentral distance, PGA for the 3 September event is significantly higher than for the second event, reflecting the higher magnitude of the first earthquake.

Figures 2 and 3 both indicate the location of the Carlton Mill Road wall, and the recording sites closest to the wall are indicated by an additional circular symbol. Based on this, the likely PGA at the wall for the 3 September 2010 earthquake is about 0.2g, and for the 21 February 2011 earthquake is about

0.4g. In addition the vertical and horizontal components are similar. The data from the 13 June 2011 event is similar to Figures 2 and 3, indicating PGA of around 0.2g at the wall.

There are two further issues of relevance to the design of reinforced soil structures. Firstly, do the horizontal and vertical peak values occur at the same moment in time? From examination of many of the strong motion records, generally there is between 1 and 5 seconds between the peaks, however in the case of the 21 February 2011 earthquake, the very strongest shaking at the HVSC station shows the peaks coinciding almost exactly, as do two more records (CMHS and CHHC) closer to the wall. Secondly, is the peak vertical acceleration upwards or downwards? In the case of the 21 February 2011 event, the majority of the records closest to the epicentre show it to be upwards, whereas for the 3 September 2010 event, it is mainly downwards, with the notable exception of the closest record (GDLC). These observations indicate that, with regards to design, it would be justified to consider the peak values as occurring simultaneously, and the vertical acceleration should be considered as both upward and downward.

3.3 Post-earthquake observations at Carlton Mill Road

Following the first two main shocks, but before the 13 June 2011 event, an inspection was made of the reinforced soil structures at Carlton Mill Road. This inspection indicated that both the panel wall and the wrap-around slope were in good condition and no damage or other issues were observed. There were some cracks in the road surface, which appeared to be associated with lateral spread of the river bank on the upstream (west) side adjacent to the wrap-around section. The cracks extended across the road mainly away from the reinforced zone. There was a patch repair of the road surface located at the end of the wrap-around zone, and there was also a small crack extending parallel to the face from this patch repair approximately 4m back from the face, which is slightly beyond the back of the reinforced soil zone. An apartment block a few 100m from the wall on the downstream (east) side was severely cracked and had been cordoned off.

One of the big issues has been lateral spread along the river banks (no passive resistance). At the location of the Carlton Mill Road wall and slope the competent blocks of reinforced soil have been sufficiently large to resist sliding and internal stability was sufficient to prevent deformation of the face, which is a highly satisfactory result when considering the damage on either side of the structure. The river bank opposite the wall had settled significantly and it had been necessary to build ramps along the footpath at the point where it runs up to a pedestrian bridge. Therefore a lot of movement had taken place all around the area, except in the location of the reinforced soil wall and slope.

4 BACK-ANALYSIS

Routine design of reinforced soil structures is generally carried out using limiting equilibrium techniques, based on either lumped safety factors or limit state analysis (using partial load and material factors). In the case of design for seismic loading, the routine approach is to treat the increased loads caused by the earthquake as additional components of static load, which is referred to as pseudo-static design. In some situations of high complexity or risk, dynamic analyses may be carried out using numerical methods, but this approach would be considered the exception rather than the rule. Pseudo-static methods are normally formulated to allow for some external deformation to take place during the earthquake, but to avoid collapse. The aim of this section is to investigate the question: would the Carlton Mill Road wall be expected to survive the recent Christchurch earthquakes based on routine design methods? The results summarised below are only concerned with internal stability, ie. adequacy of the reinforcement layout used to stabilise the structure.

Dobie (2012) outlines a two-part wedge method for the design of reinforced soil retaining walls, which has been formulated to follow the requirements of AS 4678-2002 (the Australian code for retaining wall design). The internal stability of the Carlton Mill Road structure has been examined based on the section shown on Figure 1, assuming 3.4m retained height. The reinforcement properties are as given in Table 1, assuming a Category C structure, and with additional partial uncertainly factors $\Phi_{up} = \Phi_{ud} = 0.95$ on the reinforcement strength. The actual frictional strength of the pit-run fill is not known, so that a range of values has been analysed ($\phi' = 34°$, 37° and 40°) and assumed to be "Controlled fill class 1" in terms of AS 4678-2002. Analyses have been carried out to find the critical PGA values for three cases: $K_h = K_v$, K_h fixed as 0.5g and K_v fixed as ±0.5g. The results are given in Table 3, where the K_h

and K_v values listed are the highest possible values (to 2 decimal places) before internal wedges become unstable. It should be noted that the critical wedges (ie. the first wedges to become unstable as K_h and K_v are increased) are low angle wedges starting from the lowest reinforcement layer at the facing and rising at 30° or less, with K_v upwards always being the critical case (-ve K_h indicates upwards acceleration).

Table 3: Summary of internal stability calculations in terms of critical acceleration

ϕ'	**ϕ' (des)**	**tanϕ' (des)**	**Condition**	**K_h (g)**	**K_v (g)**	**$K_h/(1 + K_v)$**
34	32.65	0.641	Max $K_h = K_v$ K_h = 0.5g, max K_v K_v = -0.5g, max K_h	0.3 0.5 0.22	-0.3 -0.01 -0.5	0.429 0.505 0.440
37	35.60	0.716	Max $K_h = K_v$ K_h = 0.5g, max K_v K_v = -0.5g, max K_h	0.36 0.5 0.28	-0.36 -0.15 -0.5	0.563 0.588 0.560
40	38.56	0.797	Max $K_h = K_v$ K_h = 0.5g, max K_v K_v = -0.5g, max K_h	0.41 0.5 0.34	-0.41 -0.27 -0.5	0.754 0.735 0.680

It can be assumed that the pit-run fill used for the Carlton Mill Road wall construction would have a ϕ' value near the top end of the range in Table 3. Based on this it can be seen that the critical K_h and K_v values are in the same range as those indicated on Figure 3 at the wall location. Table 3 includes the parameter $K_h/(1 + K_v)$. As this parameter approaches tanϕ' (design), then the Mononobe-Okabe formula used to calculate the seismic increment of earth pressure on the back of the reinforced soil structure gives very high K_{ae} (seismic active earth pressure coefficient) values and then becomes unstable if $K_h/(1 + K_v) > \tan\phi'$ (design). Examination of the results from this analysis indicates that the pseudo-static method is working well if $K_h/(1 + K_v) < 0.75 \times \tan\phi'$ (design) for this case.

5 CONCLUSION

The important practical conclusion from this case study is that the reinforced soil retaining wall built at Carlton Mill Road in 1991 performed extremely well during the recent earthquakes which have shaken Christchurch. Back-analysis indicates that the pseudo-static two-part wedge method of calculation used is capable of predicting this performance, even for the high PGA values experienced at the site. The applicability of using this design method may be judged by comparing $K_h/(1 + K_v)$ with tanϕ' (design). Examination of the strong motion data from the earthquake indicates that K_v should be considered simultaneously and with similar magnitude to K_h, and it should be checked as both an upward and a downward seismic force increment in pseudo-static analysis.

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