PERFORMANCE OF REINFORCED EARTH® BRIDGE ABUTMENT WALLS IN THE 2010-2011 CANTERBURY EARTHQUAKES

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SUMMARY

Reinforced Earth bridge abutment walls were subjected to strong ground shaking in one or more of the earthquakes in the Canterbury earthquake sequence of September 2010 to December 2011. Although the walls at three sites were subjected to ground motions of intensity greater than the design level none of the walls were damaged by the earthquakes.

The paper describes the earthquake design procedure used for the Reinforced Earth abutment walls and back-analyses carried out after the earthquakes to investigate their performance. Calculations based on probable material strengths rather than the dependable design values, and assuming no strip corrosion, gave critical accelerations to initiate sliding movements of the walls that were about 20% greater than predictions based on the design parameters. No significant outward movements of the walls were observed following the earthquakes. This was consistent with the predicted critical acceleration levels for the walls in their condition at the time of the earthquakes.

INTRODUCTION

Reinforced Earth® abutment and approach walls associated with four bridge structures located near the city of Christchurch were subjected to strong shaking in one or more of the two main earthquake events and two large aftershocks in the Canterbury earthquake sequence of September 2010 to December 2011. One of the bridges, Blenheim Road Overpass, carries a main city arterial road across the South Island Main Trunk Railway and the other three, located at Barrington Street, Curletts Road Interchange and the Heathcote River, are located on the recently completed extension of the Christchurch Southern Motorway (CSM). At the time of the earthquakes the bridge abutment walls were the only Reinforced Earth walls in the vicinity of Christchurch.

The Heathcote River Bridge was not constructed at the time of the main earthquake events although the abutment walls were complete at the time of the large aftershock that occurred on 23 December 2011. The ground shaking at the bridge site in this aftershock was estimated to have a peak ground acceleration (PGA) of about 0.17g and this was significantly lower than estimated at the other wall sites during the main events. The Heathcote River walls were undamaged in the aftershock and because of the moderate level of shaking that they experienced and their similarity to the Barrington Street walls their performance is not considered in this paper.

Reinforced Earth walls act as gravity retaining structures with a coherent gravity block consisting of facing panels, steel strip reinforcing and associated granular fill within the reinforced block behind the facing. The Christchurch walls were constructed with precast concrete cruciform shaped facing panels, nominally 1.5 x 1.5 m in elevation and 140 mm thick. The panels were connected to ribbed galvanised steel strips designed for a 100 year life with sufficient allowance for a reduction in strength due to corrosion.

BRIDGE AND WALL DETAILS

Details of the bridges are summarised in Table 1 and the lengths and heights of the walls in Table 2. Heights in Table 2 are taken from the top of the facing foundation levelling pad to the road surface near the front of the reinforced block.

Table 1. Bridges with Reinforced Earth abutment and approach walls.

Bridge Name	Year of Construct-ion	Distance From City Centre km	Bridge Length m
Blenheim Road Overpass	2007	2.3	27 Single span
Barrington Street	2012	3	31 Single span
Curletts Road Interchange	2012	5	46 Two spans

Elevations of the walls are shown in Figures 1 to 3.

SITE GROUND CONDITIONS

Christchurch city is located on Holocene deposits consisting of river flood plain sediments and loess. The surface sediments are fluvial gravels sands and silts of maximum thickness 20 m that overlie 300 to 400 m thick inter-layered gravelly formations (Cubrinovski *et al* [1]).

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Table 2. Reinforced Earth wall dimensions.

Wall Name	Max. Height m	Min. Height m	Wall Length m	Wall Area m^2
Blenheim Road. Walls 1, 2 & 3 are at NW end of bridge.				
Wall 1: Approach	8.3	1.3	203	923
Wall 2: Abutment	8.3	7.9	20.5	140
Wall 3: Approach	7.9	1.1	179	757
Wall 4: Approach	7.1	2.6	149	673
Wall 5: Abutment	7.1	7.1	20.5	121
Wall 6: Approach	7.1	1.7	171	714
Barrington Street. Walls identical at either end of bridge.				
Wall 1: Wing wall	8.3	2.0	14.7	
Wall 2: Abutment	8.3	8.3	29.7	294 total
Wall 3: Wing wall	8.3	2.0	14.7	
Curletts Road. Single wall at west end of bridge.				
Wall 1: Abutment	8.3	8.3	61.6	134

Figure 1: Blenheim Road Bridge. Junction between Walls 1 and 2 (NE corner of bridge).

Figure 2: Barrington Street Overpass. SE corner.

Blenheim Road Bridge Site

Site foundation soils are river plain gravels, sands and silts, overlaying deep gravel deposits. Medium dense upper sand layers were considered to be susceptible to liquefaction when subjected to strong earthquake shaking.

Figure 3: Curletts Road Overpass. West abutment looking towards the south.

To mitigate against potential liquefaction damage the abutment sections of the walls over the 20 m width of the embankment and for a length of 9 m at either abutment were supported on 800 mm diameter stone columns at 1.8 m centres extending for a depth of between 4 m to 8 m below the wall foundations. Large differential settlements along the lengths of the approach walls were expected under gravity loads even after preloading and placing the stone columns.

Barrington Street and Curletts Road Interchange Overpasses

Both the Barrington Street and Curletts Road sites are underlain with alluvial soils, comprising silt, sand, gravel and organic silt layers. These soils were considered to be compressible with significant settlements expected to occur under the walls.

At Barrington Street and Curletts Road the reinforced blocks were founded on 600 mm diameter stone columns arranged in triangular grids with a drainage blanket over the top of the area covered by the stone columns. At Barrington Street the columns were at 1.75 m spacing and extended to a depth of 3.5 to 4.5 m below the wall panel foundation footing. At Curletts Road the columns were spaced at 1.5 m and extended to an approximate depth of 18 m. Further stone columns were located in the backfill area behind the block at a wider spacing of 1.75 m.

Liquefaction During the Christchurch Earthquake

There were no reports of significant liquefaction at any of the bridge sites in either the Darfield or Christchurch earthquakes. Following the Darfield earthquake a sand boil was observed near the low end of Wall 6 (south-east approach wall) at the Blenheim Road Bridge but there was no sign of significant settlement at this location. Silt was observed in storm water drains near the Barrington Street Bridge indicating some local liquefaction.

EARTHQUAKE EVENTS

The 4 September 2010, local magnitude (M_L) 7.1, Darfield Earthquake caused significant damage in Christchurch City. The epicentre was located 38 km to the west of the city and because of the attenuation of the seismic waves across the Canterbury alluvial plains the shaking intensity was less severe in the city than would have been the case had the epicentre been closer. Of the four bridges with Reinforced Earth abutment walls only the Blenheim Road Bridge, completed in 2007, was constructed at the time of this event. Construction had just commenced on the Barrington Street

Overpass, with the abutment wall levelling pad and the first row of panels in place on the east side of the bridge. There was minor settlement damage to the levelling pad. The PGA at the Blenheim Road Bridge site in the Darfield earthquake was estimated to be about 0.22g.

The Christchurch earthquake, a further shallow event of M_L 6.3, occurred on 22 February 2011. With an epicentre 8 km south-east of the Christchurch city centre it was centred much closer to the city than the Darfield earthquake and caused severe damage to many buildings in the central business area. Both the abutments at the Barrington Street and Curletts Road Interchange Overpasses were complete at the time of the Christchurch earthquake although the bridge superstructures were not in place. At the time of the earthquake the approaches to both bridges had been surcharged with fill 1.5 m higher than the final motorway surface level.

Following both earthquakes there were many significant aftershocks with the two largest on the fault that ruptured in the Christchurch earthquake reaching magnitudes of 6.4 and 6.0 (M_L) respectively. The first of these occurred on 13 June 2011 and the second on 23 December 2011. These two aftershocks caused strong shaking in the Christchurch city area and at the bridge sites on the CSM. At the time of the 13 June event the surcharge at Barrington Street Overpass had been removed and the abutment sill beams were largely completed. The surcharge at Curletts Road Overpass had also been removed.

At the time of the 23 December 2011 aftershock all three CSM bridges were essentially complete with the superstructures in place. At the Barrington Street Overpass work was still being completed on the west side wingwalls.

The location of the four bridges with Reinforced Earth abutment walls in relation to the earthquake epicentres, strong motion accelerographs (SMA's) and Christchurch City is shown in Figure 4.

All of the Reinforced Earth abutment walls were located within 10 km of the Christchurch earthquake epicentre and within 20 km of the east end of the Greendale Fault which ruptured in the Darfield earthquake. They were further from the epicentres of the two large Christchurch earthquake aftershocks than the epicentre of the main event.

Earthquake Locations

The epicentral location, date, time, magnitude, depth and distance from Christchurch ity Centre of the Darfield and Christchurch earthquakes and the two large aftershocks that followed the Christchurch earthquake are listed in Table 3.

Table 3. Details of earthquakes and large aftershocks.

Earthquake Event	Date	Local Mag. M_L	Depth km	Distance Epicentre from Chch. City km
Darfield	4-Sep-10	7.1	11.0	38
Christchurch	22-Feb-11	6.3	5.4	7
Christchurch Aftershock	13-Jun-11	6.4	6.9	10
Christchurch Aftershock	23-Dec-11	6.0	7.5	10

Strong Motion Accelerograph Array

The area surrounding Christchurch is well instrumented with SMA's and the records from these instruments enable good estimates to be made of the shaking intensity experienced at the bridge sites. Table 4 lists the SMA's located within 18 km of the Christchurch epicentre and the peak ground accelerations (PGA's) recorded in the two earthquakes and main aftershocks. Figure 4 shows the locations of the SMA's in relation to the bridges, towns and state highways.

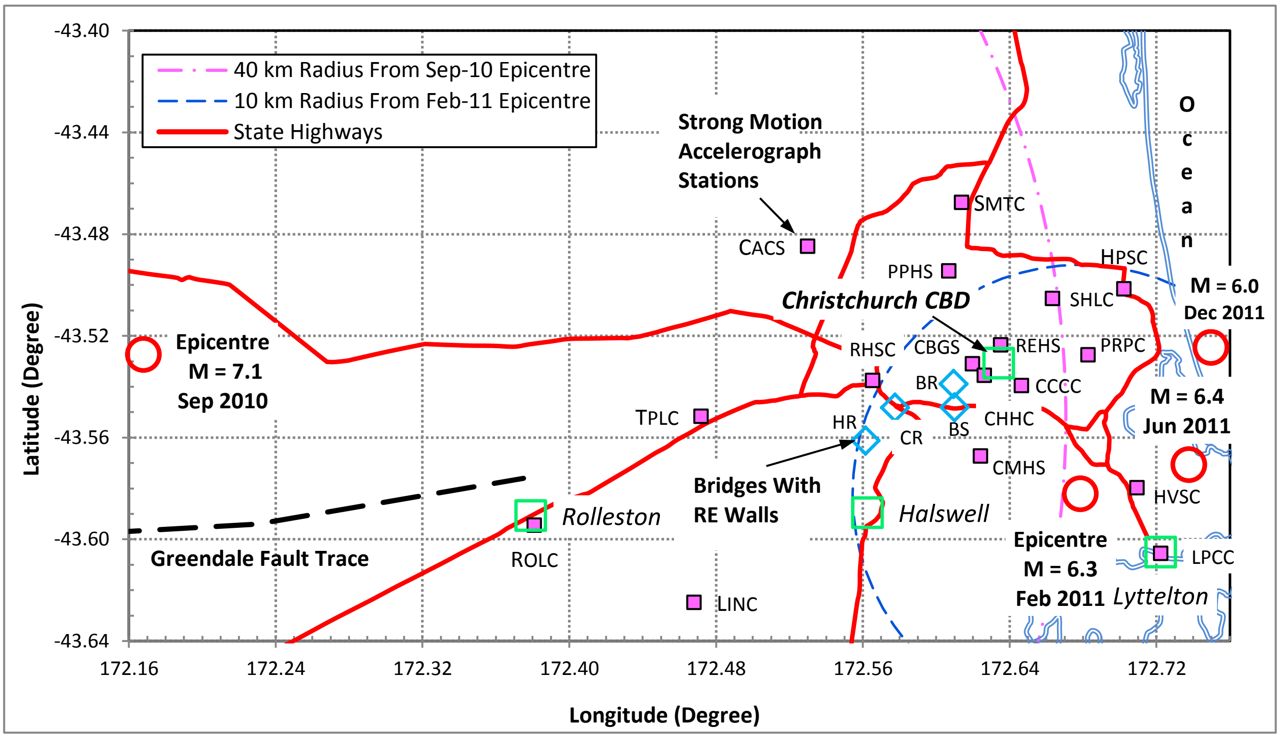

Figure 4: Location of bridges with Reinforced Earth abutments in relation to earthquake epicentres. See Table 5 for bridge name abbreviations.

Table 4. Strong motion recorders within 18 km of Christchurch earthquake epicentre.

SMA Code	Epicentral Distances, km		PGA's - Horizontal Components, g			
	Dar-field	Chch	Dar-field	Chch	Jun-11 A/S [1]	Dec-11 A/S
HVSC	43	1	0.62	1.46	0.98	0.61
LPCC	44	4	0.34	0.88	0.59	0.46
CMHS	36	6	0.25	0.40	0.19	0.19
CCCC	38	6	0.23	0.48	-	0.19
PRPC	41	6	0.22	0.67	0.46	-
CHHC	36	8	0.21	0.36	0.22	0.24
REHS	37	8	0.26	0.72	0.34	0.36
CBGS	36	9	0.18	0.53	0.17	0.23
SHLC	39	9	0.19	0.35	0.21	0.27
HPSC	43	9	0.17	0.25	0.43	0.27
NNBS	44	11	0.20	0.77	0.18	-
RHSC	31	12	0.23	0.29	0.19	0.15
PPHS	35	12	0.21	0.21	0.13	0.14
SMTC	36	14	0.17	0.18	0.09	0.16
CACS	29	18	0.19	0.22	0.15	0.10

Note 1. A/S = aftershock.

Out to a distance of 18 km from the Christchurch earthquake epicentre the largest PGA's were recorded in the Christchurch event except at the HPSC station where the PGA in the 13 June 2011 aftershock was greater than in the main Christchurch event. However, the HPSC station is 9 km to the north-east of the Blenheim Road Bridge, which is the closest bridge to the station, and there are 11 other stations closer to this bridge so the 13 June HPSC record does not provide a good indication of the PGA's for any of the bridge sites.

Shaking Intensity at Bridge Sites

From an examination of the PGA's in the four earthquake events and the relative location of the recorders and bridges it is clear that the Christchurch earthquake would have produced the largest PGA's at the Blenheim Road, Barrington Street and Curletts Road Bridge sites following completion or substantial completion of the abutment walls.

Distances of the bridges from the Christchurch earthquake epicentre, the names of the two nearest recorder stations to each bridge site and the mean of the PGA's recorded at the two nearest stations to each bridge in the Christchurch earthquake are given in Table 5.

Table 5. Distances of bridges from Christchurch earthquake epicentre and PGA's at nearest recorders.

Bridge Name and Abbrev-iation	Dist From ChCh Epicent km	Nearest SMA and Dist to Bridge km	2nd Nearest SMA and Dist to Bridge km	Mean PGA From Two Nearest SMA's g
Barrington Street (BS)	7	CHHC: 1.9	CBGS: 2.0	0.45
Blenheim Road (BR)	8	CBGS: 1.2	CHHC: 1.4	0.45
Curletts Road (CR)	9	RHSC: 1.5	CBGS: 3.9	0.41

The Blenheim Road, Barrington Street and Curletts Road Bridges were all within 4 km of the nearest two SMA stations to each of them. Because of the relatively small distances between these nearest SMA's and the bridges, a best estimate of the PGA at each site was taken as the mean of the PGA's from the two nearest SMA's to each bridge.

To verify the PGA best estimates they were plotted as shown in Figure 5 against the distance of the bridges from the Christchurch earthquake epicentre together with the PGA's recorded at each of the SMA's within 20 km of the epicentre.

Also shown in Figure 5 is the prediction of the median PGA and PGA plus or minus one standard deviation (SD) for Site Category D (NZS 1170.5 *Deep or Soft Soil* site category classification) given by the ground motion attenuation model of McVerry *et al* [2]. This model includes modifying parameters for fault mechanisms and for estimating either the geometric mean or the maximum of the two horizontal components. The regression curves in Figure 5 are for a reverse fault mechanism (identified as the Christchurch Fault mechanism type) and for the maximum of the two horizontal components. The two closest SMA's to each of the bridges are thought to be located on Site Category D sites although some of the PGA's plotted in Figure 5 were recorded on Site Category D-E or E (*Very Soft Soil*) sites.

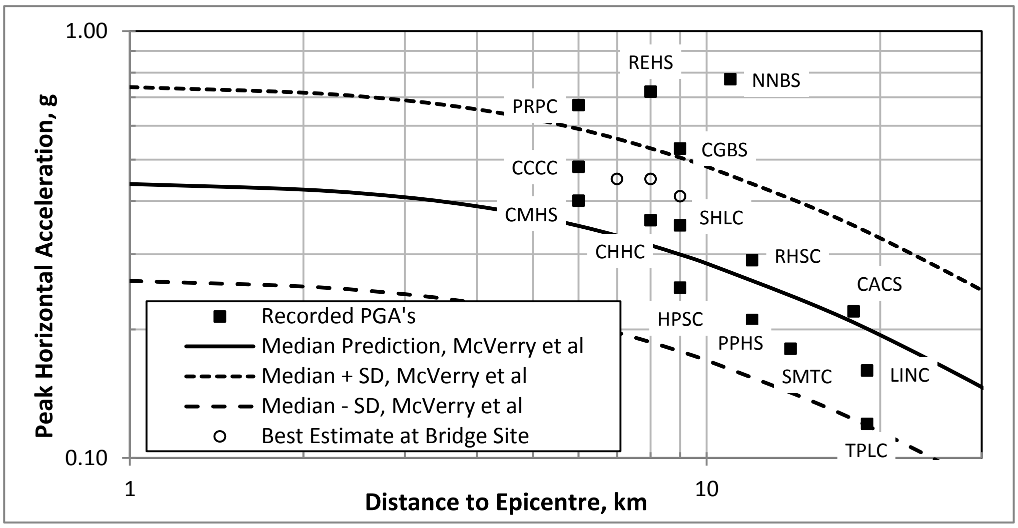

Figure 5: Recorded PGA's in Christchurch earthquake, best estimates for bridge sites and PGA predictions. (McVerry et al [2]).

From the geotechnical information provided by the bridge design consultants all three bridge sites were classified as Site Category D.

Figure 5 shows that the PGA's at distances less than 9 km from the Christchurch earthquake epicentre were higher than the McVerry *et al* median prediction with three of the eight recordings greater than the median plus one SD.

Comparison of the PGA values for the bridge sites calculated from the mean of the nearest two SMA's to each site (best estimate values) with the other plotted information in Figure 5 verifies that these are satisfactory predictions. However, as indicated by the scatter in the recorded points, and the ± 1.0 SD curves, it is not possible to make very precise predictions. Note that the logarithmic axes used in Figure 5 tend to disguise the magnitude of the scatter. For example, at a distance of 7 km from the epicentre (Barrington Street bridge site) the McVerry *et al* ± 1.0 SD curves give a PGA range from 0.2g to 0.55g.

At the bridge sites the duration of strong shaking in the Christchurch earthquake was less than in the Darfield earthquake; however examination of the stronger of the two horizontal time histories from the CBGS records, showed that there were ten peaks greater than 0.3g (total for both positive and negative peaks) compared to only one peak exceeding 0.15g in the Darfield earthquake.

Because of the close proximity of some of the SMA's (and bridges) to the Christchurch earthquake epicentre very strong vertical accelerations were recorded. Although not usually considered in Reinforced Earth wall design, strong vertical accelerations will increase the soil pressures on the wall panels and the amount of outward movement if failure planes develop in the backfill. Peak vertical ground accelerations recorded in the Christchurch earthquake by the SMA's closest to the bridges are compared in Table 6 with the peak horizontal ground accelerations.

Table 6. Vertical acceleration in Christchurch earthquake

SMA Recorder	Distance From ChCh Epicentre km	Peak Ground Acceleration, g		Ratio Vertical/ Horizontal
		Hor.	Vert.	
CMHS	6	0.40	0.80	2.0
CHHC	8	0.36	0.51	1.4
CBGS	9	0.53	0.27	0.5
RHSC	12	0.29	0.19	0.7

EARTHQUAKE PERFORMANCE OF WALLS

Wall Damage

Following the earthquake sequence of main events and aftershocks there were no reports of panel cracking or significant outward movement on any of the completed Reinforced Earth walls. Surface sliding on the backfill slopes that had been increased in height by the surcharge on the approach embankment occurred at Curletts Road but this did not damage the wall.

Settlement of Walls at Blenheim Road Bridge

The Darfield earthquake occurred a few days after the levels on the Blenheim Road walls had been surveyed. The walls were re-levelled nine days after the earthquake and the recorded levels showed settlements of up to 13 mm since the previous measurements. Settlements were probably higher than recorded as the levelling benchmark may also have settled.

Settlement of Walls at Barrington Street Bridge

Levels near the tops and bottoms of both main abutment walls were surveyed on 14 and 28 February 2011; that is eight days before and six days after the Christchurch earthquake. At the time of the earthquake the main walls were complete and the approaches to the bridge surcharged. Recorded settlements near the middle sections of the walls were about 40 mm and near the ends of the walls between 18 to 25 mm. The reliability of the level datum following the earthquake is uncertain and the settlements could have been greater than these values.

A total station instrument was used to survey reference points on the walls and this measurement method gave the horizontal movements of three points on the top and bottom of each of the main abutment walls. The mean movement outwards of the tops and bottoms of the walls was 10 mm and 22 mm respectively with similar movements at each abutment. The order of accuracy of these measurements is unknown but they are probably only accurate to ± 10 mm. Greater movement at the bottom than the top of the walls may indicate movements in the soil layers at some depth below the base of the walls. Silt was observed in nearby storm water drains so there had been some liquefaction which may have contributed to both the settlement and outward movement.

Settlement of Walls at Curletts Road Bridge

Levels near mid-height of the abutment wall and at three points along the wall were measured on 21 February and 2 March 2011; that is one day before and nine days after the Christchurch earthquake. At the time of the earthquake the wall was complete with surcharges on the approach embankments. Recorded settlements along the wall varied from 8 mm to 18 mm. As was the case at the Barrington Street Bridge site the reliability of the level datum following the earthquake is unknown.

The total station measurements indicated that the wall had moved outwards a distance varying from zero at the north end to 13 mm at the south end. These small movements are not much greater than the likely order of accuracy of the measurement method.

Settlement of Walls During Construction

Much greater settlements of the walls at the three bridge sites occurred during construction than during the earthquakes with the total settlements during the construction periods ranging from 200 mm to 290 mm.

At Blenheim Road there was a 30 mm differential settlement over a 5 m length where a concrete encased storm water pipe crossed under a section of one of the approach walls. At Barrington Street a differential settlement of 70 mm occurred over a 14 m length of the West Abutment wall and at Curletts Road a differential settlement of 90 mm occurred over a length of 11 m on the abutment wall. At Barrington Street the differential settlements were mainly a result of the difference in vertical stresses in the foundation soils across the width of the embankments.

The walls were undamaged by the settlements during construction.

EARTHQUAKE DESIGN ACCELERATIONS

Design Code Requirements

The design of the abutment walls at the three bridge sites was based on the New Zealand Transport Agency (NZTA) Bridge Manual (BM). At the time of the design of the walls the BM was published as the Second Edition [3] with a Provisional Amendment, December 2004. This amendment adopted the earthquake loading provisions of NZS 1170.5 [4] (the BM referred to a draft revision of this Standard but the draft was subsequently adopted as NZS 1170.5).

The BM defines the design acceleration for earth retaining walls using the following equation:

$$C_o = C_h(T{=}0)\, S_p\, Z\, R_u \qquad (1)$$

where: C_o = design acceleration coefficient,

$C_h(T{=}0)$ = spectral shape factor for T = 0 specified in NZS 1170.5

S_p = is the structural performance factor specified in the BM

Z = zone factor specified in NZS 1170.5

R_u = return period factor specified in BM for the bridge and abutment walls

The BM requires walls supporting bridge abutments to have no permanent outward displacement under the design level acceleration.

The parameter values used in Equation 1 for the wall designs are summarised in Table 7. A different S_p factor was used for the Blenheim Road Bridge than for the three CSM bridges. The value of S_p = 1.0 was specified in the Principal's Requirements for the CSM bridges but no reason was given for adopting this value which is higher than the minimum specified in the BM. At the time of the design of the bridges and the walls the zone factor for Christchurch was 0.22 but following the earthquakes this value was increased to 0.3.

Table 7. Earthquake design coefficients for walls.

	Blen-heim Road	CSM Bridges	
$C_h(T{=}0)$	1.12	1.12	Spectral Shape Factor. Soil Site Category D.
S_p	0.67	1.0	Principal's Requirements for CSM bridges.
Z	0.22	0.22	Specified in NZS 1170.5.
R_u	1.8	1.8	2500 year return period. BM for bridge design.
C_o	0.3	0.44	Calculated from above parameters.
$C_h(T{=}0)$ x $Z\, R_u$	0.44	0.44	PGA for 2500 year return period.

The Blenheim Road and Barrington Street Bridge superstructures are supported on abutment spread footings (sill beams) located close to the wall faces. At the abutments of the Curletts Road Bridge the superstructure is supported on a capping beam founded on 310 UB 137 piles. The bridge designer indicated that an allowable outward movement of 30 mm for the pile supported abutment would be acceptable. The design coefficient of 0.44, derived on the assumption of no outward movement, was reduced by a factor of 0.7 to give a design acceleration coefficient 0.31 consistent with permitted level of movement. The reduction factor was estimated from the sliding block displacement equation presented by Wood and Elms [5]. The walls at the abutments of the two bridges with spread footings were designed for the unreduced coefficients given in Table 7.

Comparison of Design and Recorded PGA's

The best estimate of the PGA at both the Blenheim Road and Barrington Street Bridge sites in the Christchurch earthquake was 0.45g which is marginally higher than the 2,500 year return period design value of 0.44g used for the design of the Barrington Street abutment walls.

Because of the adoption of an S_p factor of 0.67 the design acceleration of 0.3g for the Blenheim Road walls was significantly less than the design 2500 year return period PGA of 0.44g. Part of the S_p reduction is to account for the lack of coherence in the ground motions over the large area covered by the walls and backfill. If the PGA exceeds the critical acceleration required to initiate permanent outward movement by less than 20% the outward movements are of the order of a few millimetres and where this level of movement is acceptable it is reasonable to take this into account in selecting the S_p factor as well as a reduction due to lack of coherence in the accelerations.

The best estimate of the PGA at the Curletts Road site in the Christchurch earthquake was 0.41g which is about 30% higher than design level acceleration of 0.31g. The intensity of shaking estimated at the Curletts Road site corresponds to a return period level of about 2,000 years.

Although there was a close array of SMA's near the bridge sites the estimated PGA's contain significant uncertainty which needs to be considered when comparing observed with predicted performance.

DESIGN METHODS

Earthquake Design

The design method specified in the BM was used for all the Reinforced Earth abutment walls. This method is based on a limiting equilibrium (LE) analytical analysis developed by Bracegirdle [6] and verified by model shaking table tests carried out at the University of Canterbury (Fairless [7]). A bilinear failure surface is assumed to develop at the toe of the wall and to propagate up through the reinforced block (RB) and the retained soil behind the RB. An upper-bound failure criterion is applied to find the critical failure surface inclination angles and the acceleration at which sliding develops. The disturbing forces acting on the sliding block are the imposed forces from the bridge, RB soil weight, RB inertia force, and the Mononobe-Okabe (M-O) pressure on the back of the RB (Wood and Elms [5]). These are resisted by soil friction and cohesion (usually zero) on the failure plane and the tension forces in the reinforcing strips that cross the failure surface. Forces acting on the failure wedge are shown in Figure 6. The response acceleration acting on the sliding block is obtained by reducing the PGA by the S_p factor.

External stability analyses were also undertaken as part of the wall design using horizontal equilibrium equations for sliding on a horizontal plane through the base of the wall. The base vertical pressures were estimated using moment equilibrium equations. Gravity and earthquake forces acting on the RB for the external stability analysis are shown in Figure 7. For the bridge abutment walls the friction angle on the back of the block was taken as 10^o.

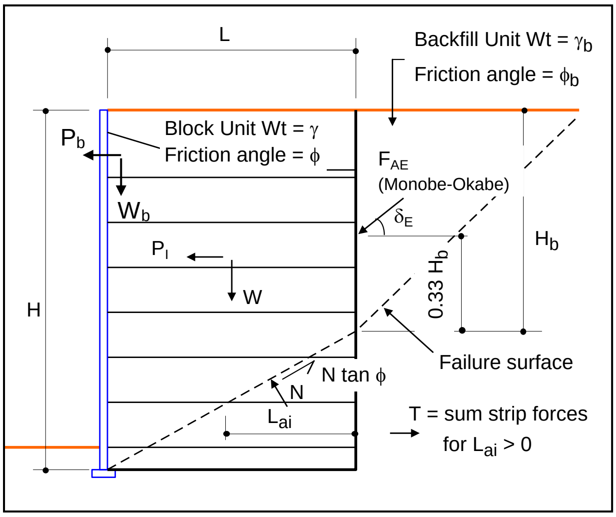

Figure 6: Reinforced Earth block LE analysis definition diagram.

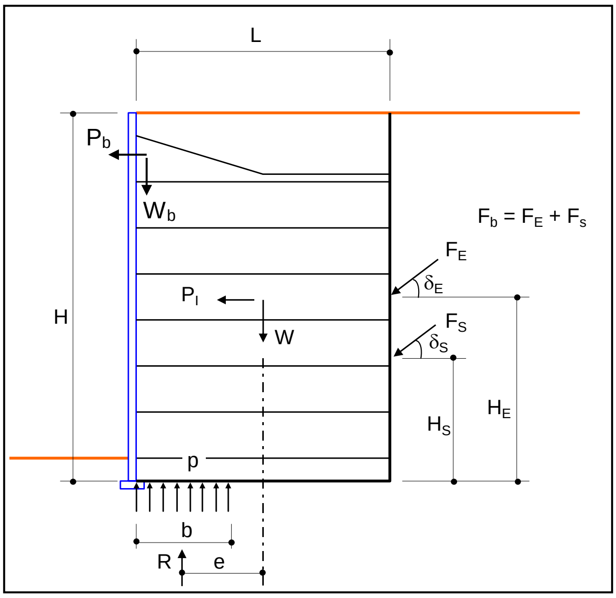

Figure 7: Reinforced Earth block external stability analysis.

The earthquake inertia forces from the bridge superstructure and abutment concrete seating beams were assumed to be distributed equally to the abutment walls at either end of the bridge, and were applied uniformly across the wall with the line of action at the same height as the centre of gravity of the combined superstructure and abutment beams. Usually the passive resistance of the soil behind the abutment structure can resist the total of the superstructure and abutment inertia loads but relatively large displacements are required to develop the full passive resistance and therefore providing resistance at both abutments is considered necessary to limit displacement damage to wall facings and abutments.

For the earthquake load case, unfactored gravity and earthquake loads were used and no live load was applied on the bridge and approach carriageways.

The LE and external stability calculations were performed using a special purpose spreadsheet.

The LE analyses were verified using the STARES software program (Balaam [8]). The analysis method used in STARES is based on the Bishop [9] simplified procedure for unreinforced slopes and is modified specifically for investigating the stability of Reinforced Earth structures. A circular rupture surface is assumed and the limiting equilibrium of the sliding mass considered taking into account the stabilising influence of the tensions developed in the reinforcing strips. The analysis is repeated for a large number of trial failure circles to estimate the minimum critical acceleration to initiate sliding of the mass. The method is similar in principle to the LE analysis method but the LE method assumes a bilinear failure surface rather than a circular surface.

Dead and Live Load Design

For the static load case of dead plus live load (G + Q) the design analysis was based on the method described in the Terre Armee Internationale (TAI) Design Guide [10].

In the TAI internal stability analysis procedure, the vertical pressures on the maximum tension line at each strip level using factored ultimate limit state (ULS) design loads from the soil mass and surface live loads, are calculated by moment equilibrium and assuming a Meyerhof vertical stress distribution. Vertical pressures from the factored bridge and abutment ULS loads are calculated using a simplified theory of elasticity solution. The combined vertical stresses on the maximum tension line are converted to horizontal pressures by pressure coefficients that vary from at rest at the surface to active at a depth of 6 m. The strip resisting length is taken as the length behind the maximum tension line.

In the TAI procedure three possible maximum tension lines are investigated; one behind the facing panels, one running to the rear of the abutment sill beam and one intermediate line that intersects the sill beam. For the present abutment walls the line running through the rear of the abutment sill beam was found to be most critical line. Factors of safety against pull-out and tensile failure of the strips were computed and compared with the factors of safety specified by TAI for ULS loads.

The strip density over most of the height of the abutment walls was more critical under earthquake loading than the G + Q load cases. On the approach and wing walls where there were no bridge inertia loads, the strip density near the top of the walls was more critical under the G + Q load cases.

Design Parameters

The soil and structural parameters adopted for the design analyses of all the walls are summarised in Table 8. All soil was assumed to have zero cohesion. Details of the strip lengths and densities used on the walls are shown in Figures 8 to 19.

At Blenheim Road the abutment sill beams were anchored with 8 m long strips at 300 mm centres providing an ultimate resistance of 85 kN/m. These strips were not included in the earthquake analysis of the wall. They were specified by the bridge designer and the decision to include them was made after the wall design was completed.

Loads From Bridges and Load Combinations

The unfactored loads imposed by the bridge superstructures on the walls are summarised in Table 9. These loads were supplied by the bridge designers and were related to the response accelerations predicted on the superstructure and abutment structures. The abutment wall length is the length considered to be effective in resisting the imposed loads and for all three bridges was taken as the length of the abutment sill beam.

The BM load cases and load factors used for the design of the walls are summarised in Table 10.

Table 8. Reinforced Earth wall design parameters.

Parameter	Value	Comment
Soil maximum unit weight in reinforced block and backfill	24 kN/m^3	
Soil minimum unit weight in reinforced block and backfill	21 kN/m^3	
Soil maximum unit weight in general backfill	24 kN/m^3	22 kN/m^3 at Blenheim Rd
Soil minimum unit weight in general backfill	21 kN/m^3	
Soil friction angle in reinforced block	36^o	
Soil friction angle in general backfill	36^o	
Soil friction angle on base of reinforced block	36^o	30^o at Blenheim Rd
Soil friction angle on back of reinforced block	10^o	External stability
Maximum vertical spacing of strips (nominal)	750 mm	
Strip width	45mm	
Strip thickness	5mm	
Strip corrosion allowance.	1.5mm	
Strip minimum UTS. (Used in G + Q load cases.)	520 MPa	
Strip minimum yield stress. (Used in EQ Load Case.)	350 MPa	
Strip friction coefficient at surface, f* max	1.5	
Strip friction coefficient at 6 m depth, f* min	0.73	
Strip yield Capacity Reduction Factor: EQ load	0.9	
Strip pull-out Capacity Reduction Factor: EQ load	0.8	
Strip factor of safety on tensile failure: G + Q load	1.65	
Strip factor of safety on pull-out: G + Q load	1.5	
Panel concrete 28 day compressive strength	40 MPa	
Panel structural thickness	140mm	

The dead load factor of either 1.0 or 1.35 was applied to both the unit weight of the reinforced soil block and the general backfill soil. The design was based on the combination of load factors that resulted in the lowest factors of safety for strip pull-out or tension failure.

Panel Strengths

All the concrete panels in the abutment walls, and the panels deeper than 4.5 m below the pavement surface in the approach and wing walls were reinforced. Sufficient vertical reinforcing is placed in the panels in the form of deformed 12 mm diameter stirrups to provide a flexural strength that exceeds the pressure load that could arise if the strips reach their ultimate tensile strengths. The panels are therefore unlikely to be damaged by the maximum pressures expected at the design level earthquake accelerations.

Table 9. Loads imposed on abutment walls (unfactored).

	Blenheim Road	Barrington Street	Curletts Road
Effective length of abutment	20 m	28 m	32 m (on skew)
Dead load superstructure	180 kN/m	268 kN/m	Abutment on piles
Dead load super + abutment	230 kN/m	348 kN/m	
EQ load super + abutment	54 kN/m	159 kN/m	
Live load from superstructure	110 kN/m	100 kN/m	
Live load breaking force	8.4 kN/m	6.1 kN/m	
Live load on RB & approach	20 kPa	12 kPa	12 kPa

Table 10. BM load cases and load factors.

BM Load Case	Load Factors EQ	G	Q	Break -ing	Earth Pre- ssure
Load Case 1A	0.0	1.0 or 1.35	2.25	0.0	1.82
Load Case 2C	0.0	1.0 or 1.35	1.67	1.35	1.35
Load Case 3A	1.0	1.0	0.0	0.0	1.35

Factors Affecting Performance

A number of conservative assumptions are made in the wall design which could result in a better than predicted earthquake performance. These include:

- Use of dependable strength parameters rather than probable strength parameters.
- Assuming that the accelerations in the backfill and reinforced block are in phase with the superstructure inertia forces.
- Assuming parallel approach walls act independently.

For design the soil friction angles for the backfill and reinforced block are not usually established by tests specific to the site and conservative values are adopted so that there is a low probability that the actual soil shear strength will be less than assumed. A strength reduction factor of 0.8 is also applied to the apparent soil/strip friction to cover uncertainty in this parameter. A characteristic yield strength is adopted for the strips so that there is only about a 5% probability of the actual yield strength being less than the design value. A strength reduction factor of 0.9 is also applied to the yield strength.

Depending on the type of bearings that a bridge superstructure is supported on at the piers and abutments, it may have a longitudinal period of vibration significantly greater than zero resulting in a response that does not correlate closely to the input ground motions. There may also be some lack of coherence between the ground acceleration in the reinforced block and in the backfill on large walls.

At the Blenheim Road Bridge the approach walls on the ramps were about 20.5 m apart. The predicted failure planes for the design configuration extend a distance of about 13 m from the back of the walls so there is overlapping of the failure planes which is likely to reduce the backfill pressure forces on the reinforced blocks. At Barrington Street Bridge the spacing between the wing walls is about 29 m so interaction between the failure planes of the wing walls on either side of the bridge approaches is unlikely.

The influence of the conservatism in the strength parameters on the critical accelerations for the walls was investigated by carrying out LE and STARES analyses using probable strength parameters instead of the dependable values used for the design. The factors used for these analyses are summarised in Table 11.

Table 11. Dependable and probable strength parameters.

Parameter	Dependable Design Value	Probable Value for Assessment
Reinforced block and backfill friction angles	36^o	37^o
f* reduction factor	0.8	1.2
Strip yield strength reduction factor	0.9	1.1

ANALYSIS RESULTS

Critical Accelerations

Results from the earthquake load analyses to determine the critical accelerations for the abutment and associated wall sections described above are presented in Table 12. These were calculated using the dependable design parameters for both the case of the design corrosion allowance and for no corrosion with the results for no corrosion representing the strip condition at the time of the earthquakes. Surcharges were present on the Barrington Street and Curletts Road Bridges at the time of the Christchurch earthquake and these were included in the analyses undertaken with no strip corrosion. A further analysis was carried out for the condition at the time of the earthquakes (no corrosion of the strips and surcharges where in place) using the probable strengths given in Table 11.

On the Curletts Road sections shallow surface failures on the slopes above the wall occurred at critical accelerations of 0.18 g. (Failures of this type were observed in the Christchurch earthquake). For the estimated PGA of 0.41g at the site in the Christchurch earthquake, Figure 20 indicates that these slides might move downslope a distance of up to 80 mm. This amount of movement of shallow slides would be unlikely to result in damage to the bridge abutments or the walls.

The results in Table 12 are for the case when the failure surface passes through the base of the wall. Provided the strip density does not reduce rapidly from the base to the top of the wall this is usually the case. However, for the Curletts Road sections with sloping backfills behind the walls the STARES analyses indicated that failure surfaces running through the panels above the base occurred at critical accelerations lower than for the case with the failure surfaces through the base. The analyses were based on the assumption of no shear strength resistance from the panels and in practice there will be significant resistance which will increase the critical accelerations.

Table 12. Critical acceleration analysis results.

Bridge Name	Section Name	Critivcal Acceleration, g Dependable With Corrosion	 Dependable No Corrosion	 Probable No Corrosion
Blenheim Road	A: Abut.	0.35	0.39	0.49
	B: Appr.	0.30	0.33	0.43
Barrington Street	A: Abut.	0.46	0.46	0.52
	B: Wing	0.44	0.44	0.49
Curletts Road	A: Wing	0.27	0.26	0.31
	B: Abut.	0.37	0.30	0.38
	C: Abut.	0.28	0.29	0.33

From the results obtained using the material probable strength values it is apparent that the critical accelerations for well constructed walls could be up to 25% higher than calculated using the material design strength parameters.

Failure Plane Locations

Locations of the failure planes from earthquake loading on the main sections of the walls are shown in Figures 8 and 9 for the Blenheim Road Bridge, Figures 10 to 13 for the Barrington Street Bridge, and Figures 14 to 19 for the Curletts Road Bridge.

For the Barrington Street and Curletts Road Bridges failure plane plots are shown for both the design configuration (including bridge loads and strip corrosion allowances) and for the configurations at the time of the Christchurch earthquake with preloading in place. At Barrington Street the toe of the preloading was retained by concrete blocks with a soil slope of about 2: 1 (horizontal: vertical) behind the blocks. As exact dimensions of this arrangement were not available the surcharge slope was modelled by a 1:1 slope which was considered sufficiently accurate for the present assessment. At Curletts Road the preloading above the wall was sloped at about 2: 1.

For the Blenheim Road Bridge failure planes are shown for only the design configuration as the bridge superstructure was in place at the time of both the Darfield and Christchurch earthquakes. Eliminating the corrosion allowance (configuration at the time of the earthquakes) reduced the failure plane inclination angle on Section A by about 3^o but made no change to the failure plane inclination on Section B.

Outward Displacement

At both the Blenheim Road and Curletts Road sites the estimated PGA's in the Christchurch earthquake were greater than the calculated critical accelerations for the walls based on the design strength assumptions and in their uncorroded strip and surcharged (Curletts Road) configurations at the time of the earthquake. If the PGA estimates and critical acceleration calculations are correct then outward movements of the walls would be expected.

If the accelerations acting on the walls and backfill are assumed to be fully coherent with no reduction in the S_p factor for this effect, the outward movement resulting from

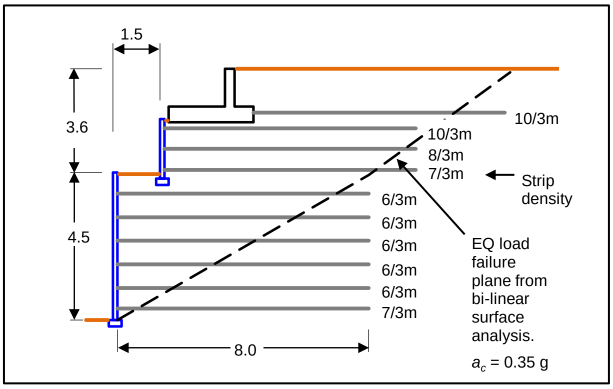

Figure 8: Blenheim Road bridge. Failure plane Section A. Design configuration.

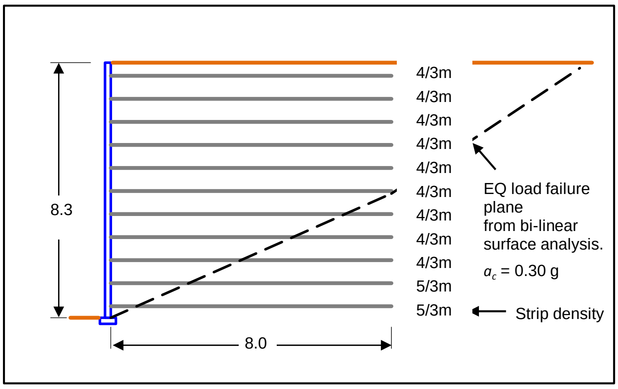

Figure 9: Blenheim Road bridge. Failure plane Section B. Design configuration.

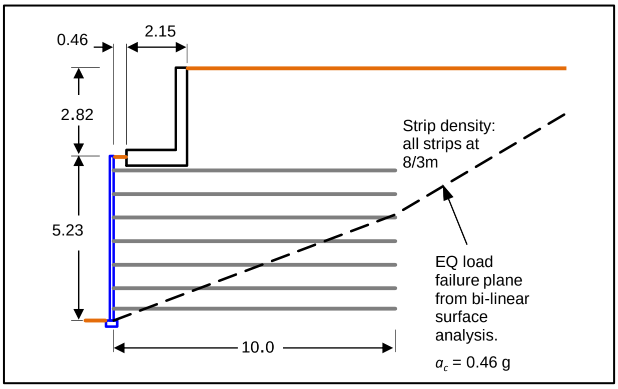

Figure 10: Barrington Street bridge. Failure plane Section A. Design configuration.

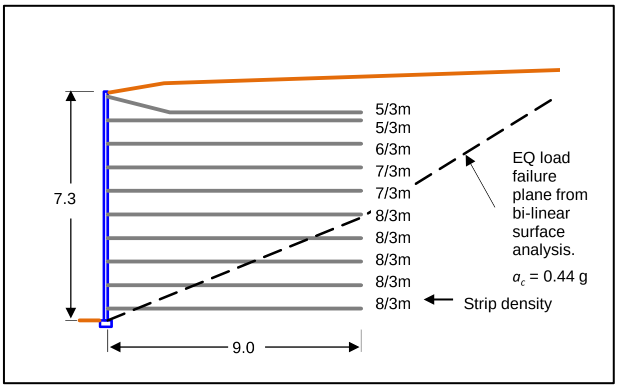

Figure 11: Barrington Street bridge. Failure plane Section B. Design configuration.

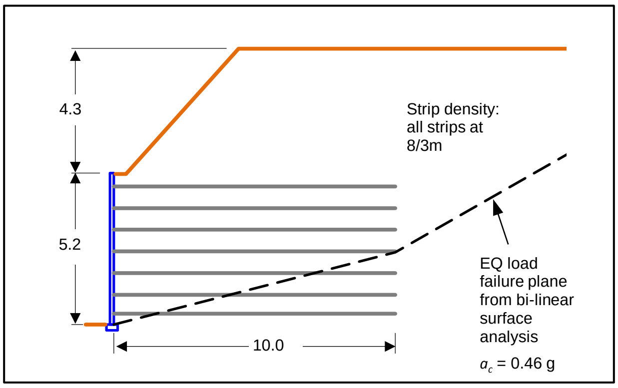

Figure 12: Barrington Street bridge. Failure plane Section A. Surcharge configuration.

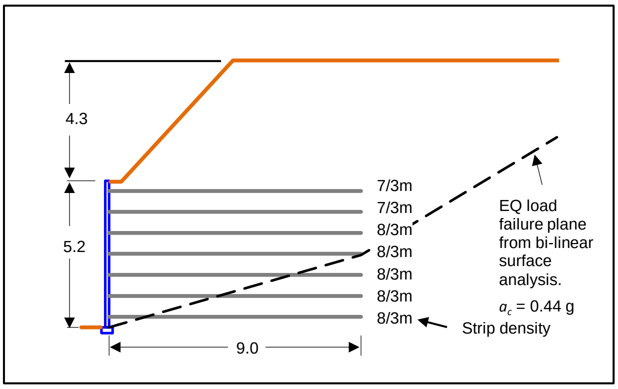

Figure 13: Barrington Street bridge. Failure plane Section B. Surcharge configuration.

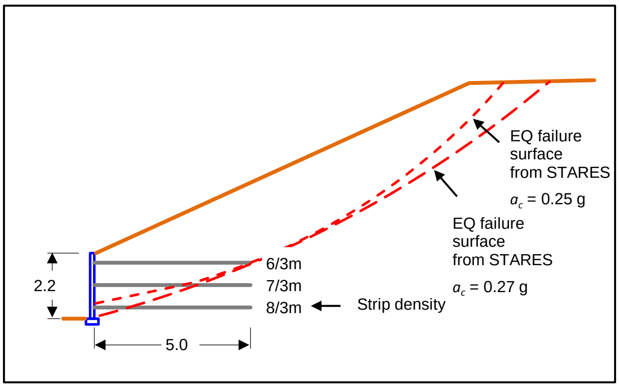

Figure 14: Curletts Road bridge. Failure planes Section A. Design configuration.

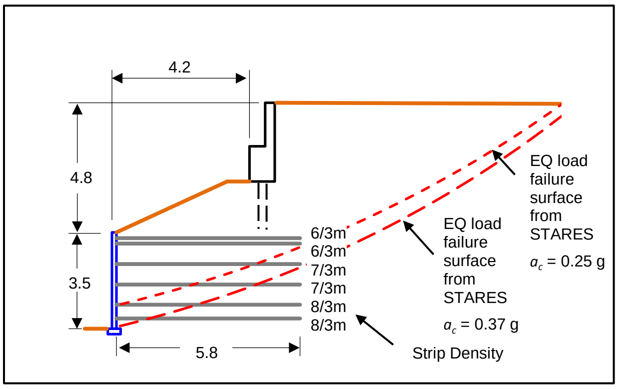

Figure 15: Curletts Road bridge. Failure planes Section B. Design configuration.

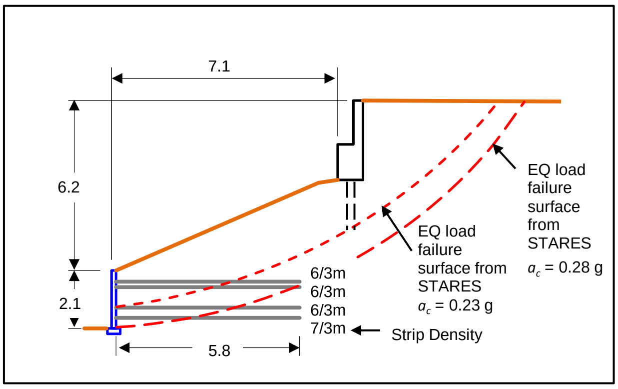

Figure 16: Curletts Road bridge. Failure planes Section C. Design configuration.

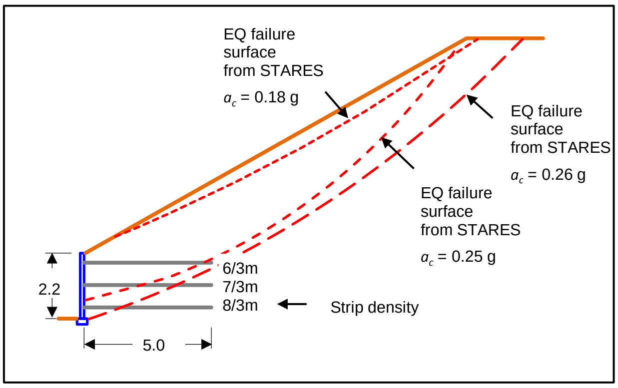

Figure 17: Curletts Road bridge. Failure planes Section A. Surcharge configuration.

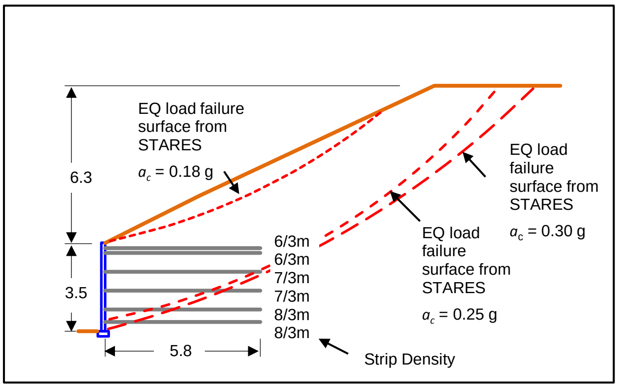

Figure 18: Curletts Road bridge. Failure planes Section B. Surcharge configuration.

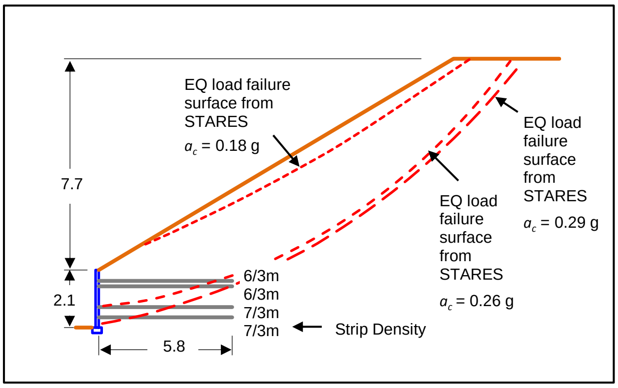

Figure 19: Curletts Road bridge. Failure plane Section C. Surcharge configuration.

the site PGA's in the earthquake exceeding the calculated critical accelerations can be estimated using the Newmark sliding block theory (Newmark [11]).

In the application of this theory the soil mass is assumed to be a rigid block that fails in a rigid-plastic manner when the ground acceleration exceeds the critical or yield acceleration of the slope. Once sliding commences, it is assumed that the rigid mass continues to slide under the actions of the inertia force from the ground acceleration pulse and a constant resisting force. When the acceleration pulse diminishes in magnitude or reverses in direction, the relative velocity between the sliding block and the supporting ground or base eventually reduces to zero and the movement relative to the base ceases. Successive displacements take place each time the ground acceleration exceeds the critical value.

Outward displacements can be estimated using the correlation equations of Jibson [12], Ambraseys and Srbulov [13], and Ambraseys and Menu [14]. Jibson used the Newmark sliding block theory for unsymmetrical sliding (one-way movement) to integrate 2,270 horizontal component strong-motion records from 30 earthquakes of magnitudes between 5.3 and 7.6, and performed regression analyses of the computed displacement data, with critical acceleration ratio and earthquake magnitude as variables, to obtain the following expression for the permanent displacement, *d*, expressed in centimetres:

$$\log(d) = -0.271 + \log\left[\left(1 - \frac{a_c}{a_{max}}\right)^{2.335}\left(\frac{a_c}{a_{max}}\right)^{-1.478}\right] + 0.424Mw \pm 0.454 \qquad (2)$$

where: a_c = critical acceleration to initiate sliding failure

a_{max} = peak ground acceleration (PGA) in the acceleration record

Mw = earthquake moment magnitude

The last term in the equation is the standard deviation of the model.

Ambraseys and Srbulov used records from 76 shallow earthquakes with magnitudes ranging between 5.0 to 7.7 and regression analysis to develop a similar expression to the Jibson equation. Their relationship included a source distance for the earthquake which was not used by Jibson. Ambraseys and Menu used 26 sets of two-component records from 11 earthquakes of magnitudes 6.9 ± 0.3 and performed a multiple-variable regression analysis on the computed data with a number of ground motion parameters as independent variables. Because of the small magnitude range of the source earthquakes, magnitude and duration were not found to be important parameters and their prediction equation was similar to Jibson's but without a magnitude term.

Upper-bound outward displacements computing using the three regression equations are shown in Figure 20 for a M_w 6.2 earthquake. (A moment magnitude of 6.2 was estimated for the Christchurch earthquake). The displacement for a 5% probability of exceedance is plotted against the a_c /a_{max} acceleration ratio.

Outward displacements for the walls at the Blenheim and Curletts Road Bridge sites were estimated using the Jibson and the Ambraseys and Srbulov correlation equations for 5% probability of exceedance. The displacement results are summarised in Table 13 for critical accelerations calculated using the design strength assumptions with uncorroded strips and surcharges in place.

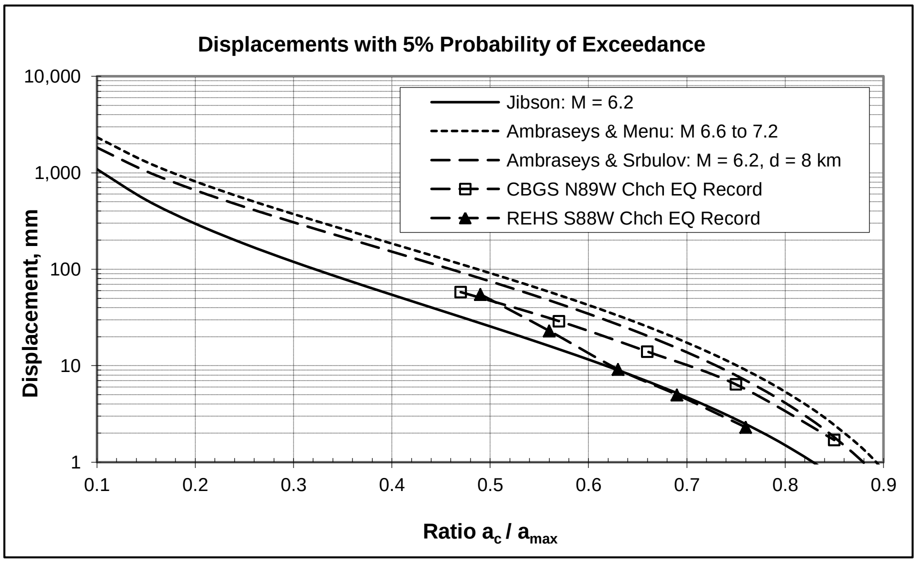

Figure 20: Outward displacements in Christchurch earthquake for 5% probability of exceedance. Lines labelled CBGS and REHS were computed from recorded accelerograms.

The estimated PGA in the Christchurch earthquake at the Barrington Street site was about the same level as the calculated critical accelerations for the walls so outward displacement was not expected.

Table 13. Outward displacements in Christchurch earthquake.

Bridge Name	Section Name	Crit. Accel g	Site PGA g	Outward Displacement 5% P of Exceedance mm	
				Jibson	Ambra-seys & Srbulov
Blenheim Road	A: Abut	0.39	0.45	0	1
	B: Appr	0.33	0.45	3	10
Curletts Road	A: Wing	0.26	0.41	9	26
	B: Abut	0.30	0.41	3	10
	C: Abut	0.29	0.41	4	12

The estimated outward displacements given in Table 13 for the Blenheim and Curletts Road Bridge walls in the Christchurch earthquake were based on the design material strength parameters are therefore likely to be overestimated. The critical accelerations calculated using probable strength values indicate that no outward displacement of the walls would occur at Blenheim Road and only a few millimetres would be likely for the Curletts Road abutment wall.

Numerical Computation of Displacements

To verify the outward displacement estimates from the correlation equations sliding block displacements were calculated using two acceleration time-histories recorded in the Christchurch earthquake. The analysis was carried out using a special purpose software program (DISPLMT, Houston *et al* [15] and time-histories aligned in approximate east-west directions recorded at the CBGS and REHS stations. The largest PGA's near the bridges were recorded at these two stations (see Figure 4) and the strongest components were in the east-west directions. The main abutment walls of the Barrington Street and Curletts Road Bridges are aligned approximately north-south so the east-west acceleration components are the most critical for loading on these walls. The main abutment walls and approach walls at Blenheim Road are aligned approximately southwest-northeast and northwest-southeast so the east-west acceleration components need to resolved and combined with the other horizontal component to estimate the accelerations normal to the walls. This degree of refinement was not warranted for the present assessment.

CBGS was one of the two SMA's closest to three of the bridges (see Table 4). Although REHS was not one of the two closest SMA's to any of the bridges it was relatively close being 2.7 km and 3.4 km from the Blenheim Road and Barrington Street Bridges respectively.

The acceleration input and displacement output time-histories are shown in Figures 21 and 22 for the CBGS and REHS records respectively. For the CBGS record critical accelerations of 0.30g, 0.35g and 0.40g were used. For the stronger REHS record these were increased to 0.35g, 0.40g and 0.45g. (Sliding block displacements depend on the sign direction of the acceleration input. The plotted results are for the sign direction that gave the greatest displacement.) In Table 14 the calculated sliding block displacements are summarised and compared with predictions made using the Jibson, and Ambraseys and Srbulov correlation equations for the same ratios of a_c/a_{max} as used in the numerical analyses.

Displacements from the numerical analyses using the recorded accelerograms are also compared with the correlation results in Figure 20. The numerical results lie between the Jibson and Ambraseys and Srbulov correlation estimates indicating that for the Christchurch earthquake the correlation results give satisfactory upper and lower estimates.

Table 14. Outward displacements in Christchurch earthquake from numerical analysis.

Time-History Record	PGA g	Crit Accel g	Disp from Num-erical Analysis mm	Disp from Correlation: 5% P of Exceedance, mm	
				Jibson	Ambra-seys & Srbulov
CBGS N89W	0.53	0.30	29	15	43
		0.35	14	7	20
		0.40	6.4	3	8
REHS S88E	0.72	0.35	55	28	81
		0.40	23	16	48
		0.45	9.2	9	27

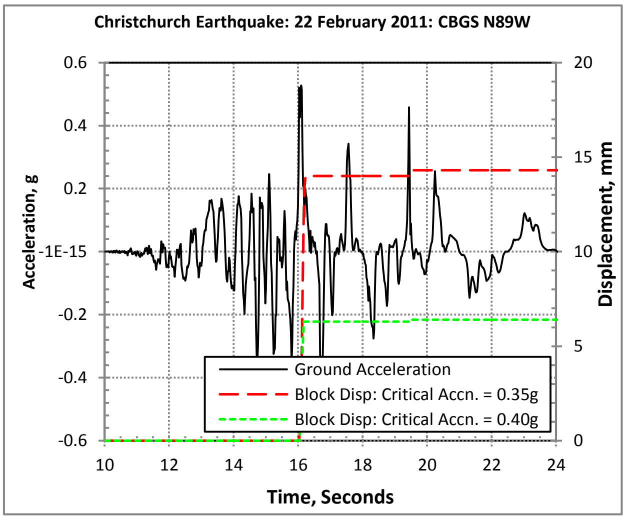

***Figure 21:** Sliding block displacements from CBGS N89W acceleration time-history.*

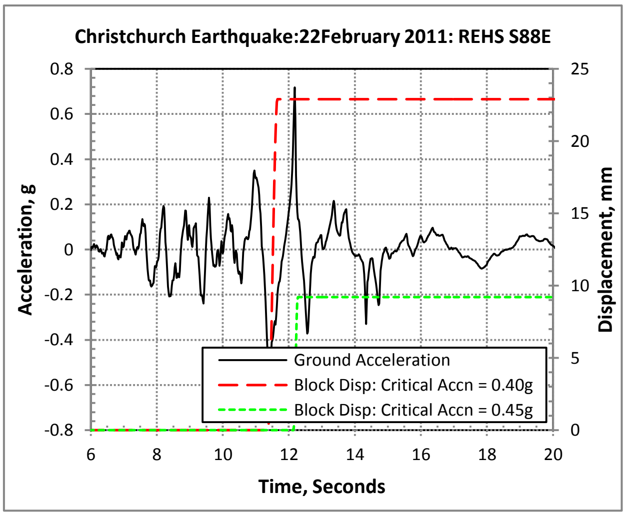

***Figure 22:** Sliding block displacements from REHS S88E acceleration time-history.*

The best estimate of outward displacement can be taken as the numerical results for the CBGS east-west accelerogram as the accelerations recorded at the REHS station were probably higher than at the bridge sites. The numerical results confirm that any outward movement of the walls at the Blenheim and Curletts Road Bridges would be less than 30 mm.

Base Sliding and Base Pressures

The factors of safety against sliding on a horizontal plane through the base of the reinforced block and the maximum base pressures obtained from the earthquake load external stability analyses are summarised in Table 15. The results are for the design configuration with the superstructures in place and without surcharges. The wall design level horizontal acceleration was used for the external stability analyses and a 10^{o} friction angle was assumed on the back face of the reinforced block.

At the design acceleration of 0.31g the 2:1 (horizontal: vertical, 26.6^{o}) slopes above the wall facing on the Curletts Road wall sections are unstable. Shallow surface failures occur on the slopes at an acceleration of about 0.18g and the Mononobe-Okabe pressures on the back of the reinforced blocks where the slope extends above them tend to infinity at about this acceleration level. It was therefore not possible to undertake a simple external stability analysis for Section A of the Curletts Road wall.

Table 15. Factors of safety against sliding and base pressures.

Bridge Name	Section Name	Design Level Acc g	Factor of Safety Against Sliding	Maximum Vertical Pressure at Base kPa
Blenheim Road	A: Abut	0.30	1.2	460
	B: App	0.30	1.1	370
Barrington Street	A: Abut	0.44	1.2	550
	B: Wing	0.44	1.1	430
Curletts Road	A: Wing	0.31	-	-
	B: Abut	0.31	1.3	250
	C: Abut	0.31	1.7	160

The analysis for Section B was based on assuming a horizontal surface behind the reinforced block. For Section C the block was assumed to extend back to the abutment piles with a horizontal surface behind the piles. Analyses for these sections are approximate but they will give results of the correct order for the maximum base pressures. Sliding will be resisted by the abutment piles but this has not been taken into account.

The factor of safety results in Table 15 indicate that sliding on the base of the walls was unlikely. The maximum vertical pressures under earthquake loading are moderately high but because of the transient nature of the loading are unlikely to cause significant deformations in the foundations strengthened with stone columns.

CONCLUSIONS

(a) The close array of SMA's in the Christchurch area enabled reasonably reliable estimates to be made of the ground shaking intensity at the three bridge sites where Reinforced Earth abutment and approach walls were constructed or under construction during the Canterbury earthquake sequence.

(b) At two of the bridge sites the estimated PGA's in the Christchurch earthquake exceeded the wall design level accelerations and also the critical accelerations calculated for the Reinforced Earth walls based on the material strength parameters used in the design, assuming no corrosion of the strips and making allowance for any bridge inertia load or surcharge present at the time of the earthquake.

(c) No significant outward movements or other damage to the walls occurred during the two main earthquakes and aftershock sequences. Critical accelerations calculated on the basis of probable material strengths, rather than the dependable strengths used in design, showed that it was only the Curletts Road abutment wall that had critical accelerations less than the PGA estimated for the site in the Christchurch earthquake. For this wall where the bridge abutment was supported on piles the design level acceleration was reduced to less than the design level PGA as outward movement of up to 30 mm was considered acceptable. Although the critical acceleration based on probable material strengths was about 30% less than estimated PGA at the Curletts Road Bridge site for one of the wall sections, predictions of the outward movements based on Newmark sliding block theory showed that they would be less than 15 mm.

(d) The LE method used for the earthquake design of Reinforced Earth walls in New Zealand is not widely used elsewhere. The Darfield and Christchurch earthquakes presented for the first time an opportunity to observe the performance in strong ground shaking of high abutment walls designed by this method.

(e) Prototype testing (Richardson [16]) and experience in earthquakes elsewhere (Wood and Asbey-Palmer [17]) has indicated that outward movements of up to 2% of the height of the wall are unlikely to result in damage to the panels or affect the post earthquake performance. Walls designed by the LE method for no, or small outward displacements, are therefore expected to perform satisfactorily in earthquake events with shaking intensities significantly greater than the design level.

(f) Reinforced Earth walls with precast concrete facing panels are particularly well suited to construction on weak soils. The Reinforced Earth abutment walls at the three Christchurch bridges constructed on soft soils were undamaged by total settlements of up to 290 mm and differential settlements of up to 90 mm over a 11 m length of wall (0.8%) that occurred during construction. Maximum settlements of the abutment walls in the Christchurch earthquake were of the order of 50 mm and these settlements were also accommodated without damage to the facing panels.

REFERENCES

1. Cubrinovski, M. and Green, R.A. (Editors) (2010), "Geotechnical Reconnaissance of the 2010 Darfield (Canterbury) Earthquake", *Bulletin New Zealand Society for Earthquake Engineering,* Vol. **43**, No. 4.

2. McVerry, G.H., Zhao, J.X., Abrahamson N.A. and Somerville P.G. (2006), "New Zealand Acceleration Response Spectrum Attenuation Relations for Crustal and Subduction Zone Earthquakes", *Bulletin New Zealand Society for Earthquake Engineering,* Vol. **39**, No. 1.

3. NZTA. (2003), "Bridge Manual", Second Edition, Including amendments to December 2004. New Zealand Transport Agency, Wellington.

4. NZS 1170.5: 2004. "Structural Design Actions. Part 5: Earthquake Actions – New Zealand", Standards, New Zealand.

5. Wood, J.H. and Elms, D.G. (1990), "Seismic Design of Bridge Abutments and Retaining Walls", *RRU Bulletin 84, Vol. 2.* Transit New Zealand.

6. Bracegirdle, A. (1980), "Seismic Stability of Reinforced Earth Retaining Walls", *Bulletin NZ National Society for Earthquake Engineering*, Vol. **13**, No. 4.

7. Fairless, G.J. (1989), "Seismic Performance of Reinforced Earth Walls", Report 89-8, Department of Civil Engineering, University of Canterbury, Christchurch.

8. Balaam, N.P. (2006), "Stability Analysis of Reinforced Soil", *User Manual for Program STARES for Windows.* Centre for Geotechnical Research, University of Sydney, NSW.

9. Bishop, A.W. (1995), "The Use of the Slip Circle in the Stability Analysis of Earth Slopes", *Geotechnique,* London, **5**, pp. 7-17.

10. TAI. (1990), "Design of Reinforced Earth Abutments", *Design Guide A-06/10.* Terre Armee Internationale.

11. Newmark, N M. (1965), "Effects of Earthquakes on Dams and Embankments", *Geotechnique*, London, **15**(2), pp. 139–160.

12. Jibson, R. W. (2007), "Regression Models for Estimating Coseismic Landslide Displacement. *Engineering Geology*", Vol. **91**, Issues 2-4, pp. 209-218.

13. Ambraseys, N. and Srbulov, M. (1995), "Earthquake Induced Displacements of Slopes", *Soil Dynamics and Earthquake Engineering,* Vol. **14**, pp. 59 - 71.

14. Ambraseys, N. N. and Menu, J. M. (1988), "Earthquake–induced Ground Displacements", *Journal of Earthquake Engineering and Structural Dynamics*, Vol. **16**, pp. 985 - 006.

15. Houston, S.L. Houston, W.N. and Padilla, J.M. (1987), "Microcomputer-Aided Evaluation of Earthquake-Induced Permanent Slope Displacements", *Microcomputers in Civil Engineering,* Vol. **2**, pp. 207-222.

16. Richardson, G.N. Ferger, D. Fong, A. and Lee, K.L. (1977), "Seismic Testing of Reinforced Earth Walls", *Proceedings American Society of Civil Engineers,* Vol. **103**, No GT1.

17. Wood, J.H. and Asbey-Palmer, D. (1999), "Performance of Reinforced Earth Retaining Walls in the 1995 Hyogoken-Nanbu (Kobe) Earthquake", *Proceedings New Zealand Society for Earthquake Engineering,* Annual Conference, Rotorua.