5. PERFORMANCE OF IMPROVED GROUND

The performance of four improved ground sites is discussed in this chapter. Three of the sites were treated with stone columns (i.e., Ferrmead Apartment Building, AMI Stadium, and Motorway Bridge Approaches). The last site (i.e., St. Andrews School) was "treated" with screw piles.

Ferrymead Apartment Building

The Waterside apartment building (-43.5577°, 172.7066°), indicated by the dashed white line in Figure 5-1, is situated between the Avon-Heathcote Estuary to the north and Ferry Rd to the south, and immediately west of the Ferrymead Bridge. The structure consists of a 6-story precast concrete panel building with a single basement level carpark (Figure 5-2). The geotechnical engineer for the project indicated that the building is supported on shallow foundations overlying stone columns. Large volumes of ejecta were evident in the area, with sand boils still present in the estuary adjacent to the structure. Two lateral spread cracks were noted on the north side of the building between the water and the building. The crack closest to the water had a maximum width of 13 cm and the crack closest to the building had a maximum width of about 4 cm. The larger crack extended along the top of the embankment north of Tidal View Rd, and additional cracking was present between the road and building.

Figure 5-1. Aerial view of Ferrymead apartment building indicating surrounding ejected material (LINZ 2011). (-43.5577°, 172.7066°)

Conversations with the property owner suggested that the building had settled and had a slight tilt toward the water. Measurements by the GEER team indicated that the building has settled between 4 and 8 cm. On the north side of the building, separation walls on the ground surface showed differential movement (Figure 5-3). The separation walls sloped downward towards the building at an angle of about 0.4 to 0.5 degrees (Figure 5-3a). This caused the caulk in the expansion joint to be compressed at the top of the wall (Figure 5-3b). Three of the glass panels that were mounted on top of the separation walls were shattered (Figure 5-3a), likely due to the compression of the glass against the deck above; cracking of the wall connection beneath this was also evident. The separation walls on the south side of the building also sloped downward toward the building at an angle of about 0.8 degrees. Cracking along concrete walkways extending out from the structure also indicated differential settlement of the building relative to the ground north of the building (Figure 5-4).

Significant flooding occurred in the basement (Figure 5-4), with sand having flowed up through the drains in the basement slab. Upon the GEER team's arrival on 3 March 2011 a work crew was repairing a ruptured water line on the north side of the building, and a pump was being used to remove water from the basement that had been flooded. Workers were also removing sand from the basement. The water level had dropped to within a few inches at the time of the team's inspection, as indicated by high water marks on the basement wall.

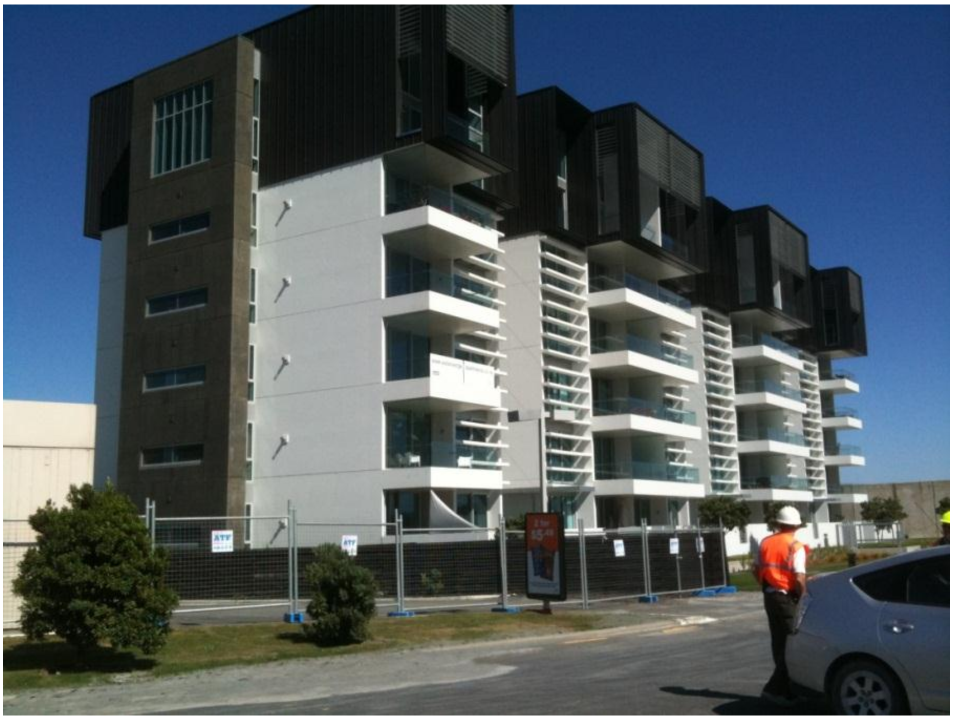


Figure 5-2. Photograph of apartments looking toward the south west. (-43.5577°, 172.7066°)

(a) (b)

Figure 5-3. Photograph of separation walls on the north side of the building. Note the (a) sloping of the separation walls and shattered glass panels, and (b) compression of caulk at top of expansion joint. (-43.5577°, 172.7066°)

Figure 5-4. Differential settlement along concrete walkway between patios and surrounding ground north of the building. (-43.5577°, 172.7066°)

Figure 5-5. Photograph of the basement parking garage. Note the high water mark on the walls. (-43.5577°, 172.7066°)

AMI Stadium

AMI Stadium (-43.5419°, 172.6541°), Figure 5-6, is located in an area that extensively liquefied during the 22 February earthquake. All four stands has shallow foundation systems, and suffered varying levels of damage. The Hadlee Stand, with no ground improvement measures, suffered severe structural damage and has been recommended for demolition. The Tui Stand, constructed on a fill platform to raise its level, and suffered less severe structural damage during the earthquake.

Both the Paul Kelly and Deans Stands were constructed on stone columns installed within their footprint. The Deans Stand has shallow foundations connected by grade beams built upon 8 m deep stone columns, while the Paul Kelly Stand has a slab foundation up to a meter thick founded on 9 m deep stone columns. Both stands suffered structural damage from differential settlements and the ground shaking, which was approximately 30% larger than design levels.

The Paul Kelly Stand, constructed in 2000, settled by up to 400 mm, with settlement variations of approximately 70 mm The thick slab beneath the structure prevented any ejected material from coming up within the structural footprint. The Deans Stand, built in 2008, developed similar overall settlements, but with much larger variations in settlement across the structure of up to 300 mm.

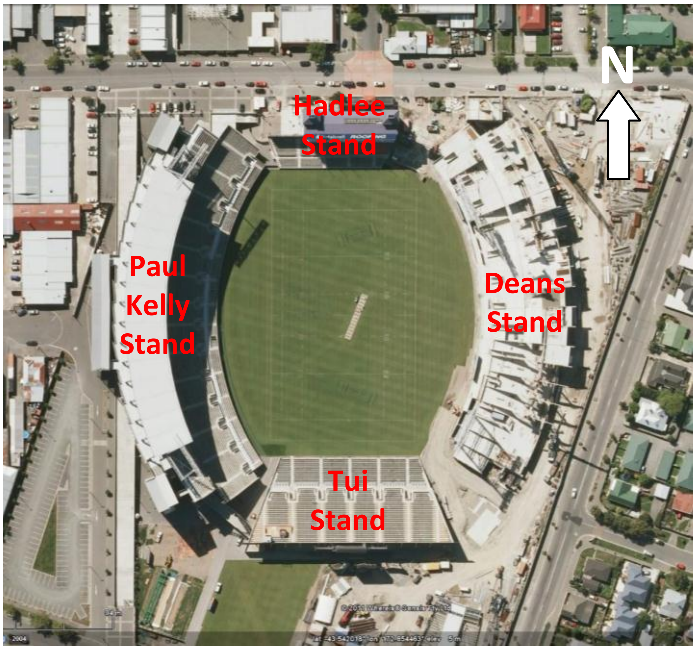


Figure 5-6. AMI Stadium. (-43.5419°, 172.6541°)

The Paul Kelly stand settled by up to 400 mm, with settlement variations of approximately 70 mm The thick slab beneath the structure prevented any ejected material from coming up within the structural footprint. The Deans stand developed similar overall settlements, but with much larger variations in settlement across the structure of up to 300 mm. Stones columns (600 mm in diameter) were installed beneath the Deans Stand to a depth of 8-10 m below grade. The stone columns were installed on approximately 2.5 m c-c spacing in an arc pattern that extending slightly outside of the footprint of the stand (see Figure 5-7). No liquefaction was evident in the northern part of the Deans Stand, however, there was a large area with surface evidence of liquefaction beneath the southern part of this stand.

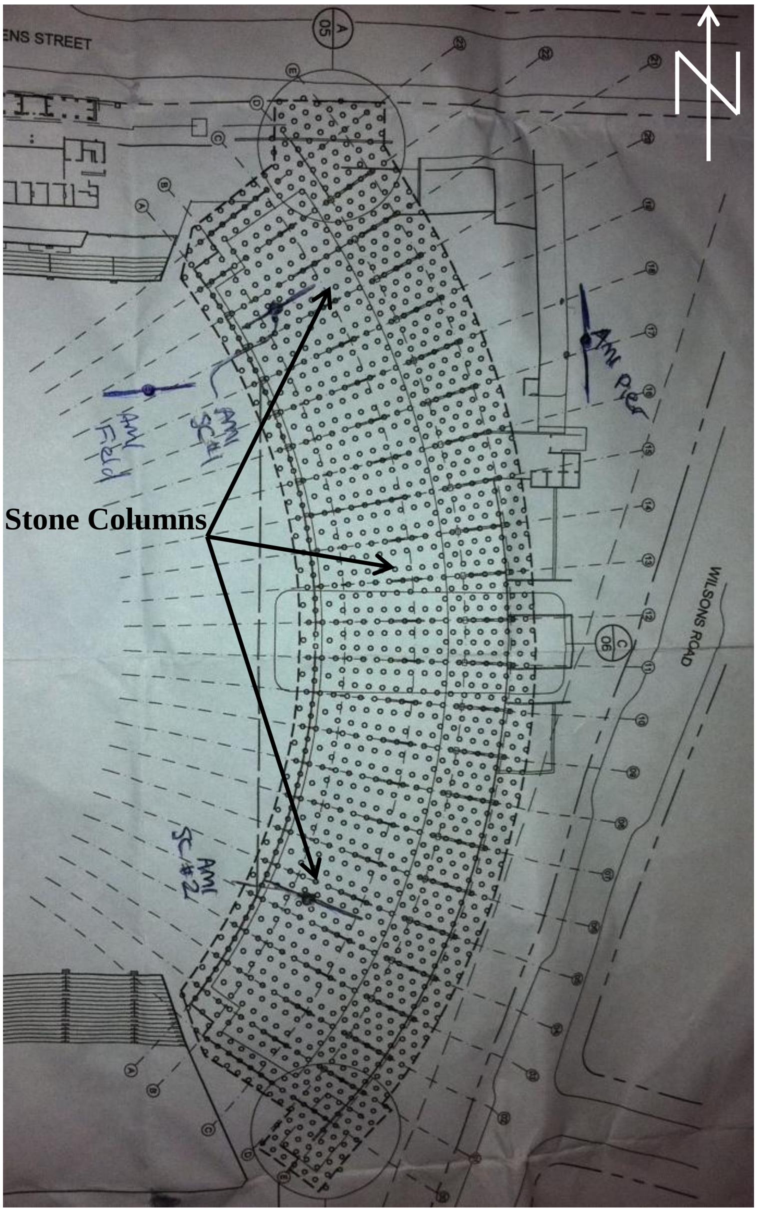


Figure 5-7. Stone column installation plan for the Deans Stand of AMI Stadium

Significant liquefaction occurred on the field. However, due to mesh below the turf liquefaction "blisters" formed, Figure 5-8, more so than sand boils. The blisters formed undulations up to 70 cm high across the field.

Figure 5-8. Liquefaction “blisters” and sand boils on the AMI Stadium field. (-43.541397°, 172. 654521°)

Shear wave Velocity Measurements

SASW tests were performed at four locations at AMI Stadium, one on the playing field (AMI Field), two under Deans Stand (AMI SC1 and AMI SC2), and one just outside of Deans Stand (AMI Columns). The locations were the SASW tests were performed are shown in Figure 5-9.

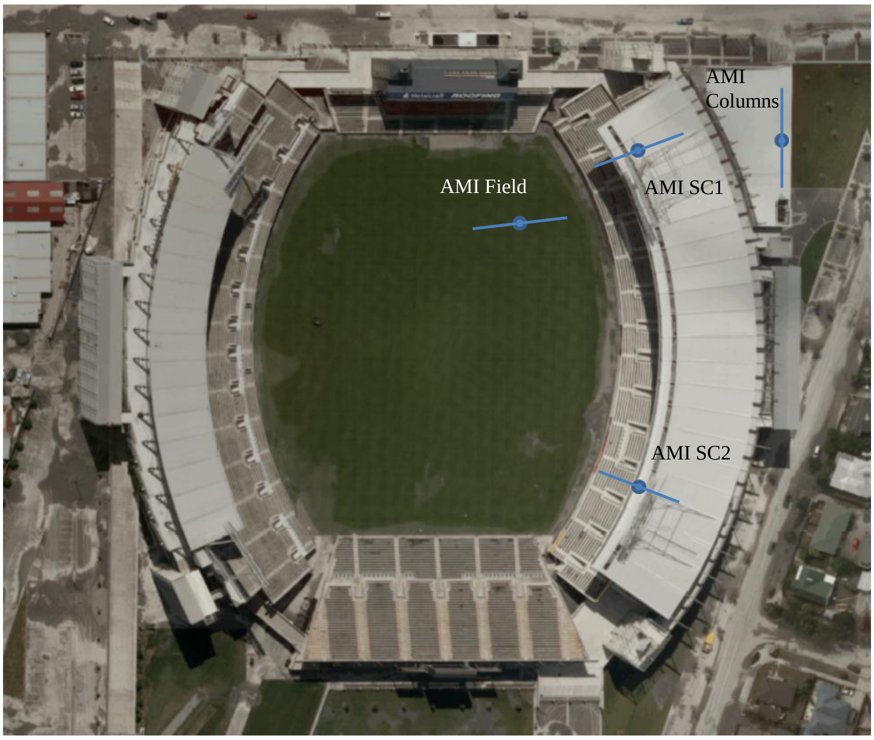


Figure 5-9. Aerial photo of AMI Stadium indicating the locations of four SASW arrays: AMI Field: -43.54161°, 172. 65452°; AMI Columns: -43.54128°, 172. 65562°; AMI SC1: -43.54138°, 172. 65501°; AMI SC2: -43.54236°, 172. 65514°.

The SASW array at the AMI field test site was located on the northeastern portion of the playing field in an area surrounded by sand blisters and boils Figure 5-10. The results are presented in Figure 5-11 and Table 5-1.

Figure 5-10. SASW array at AMI Field. (-43.54161°, 172. 65452°)

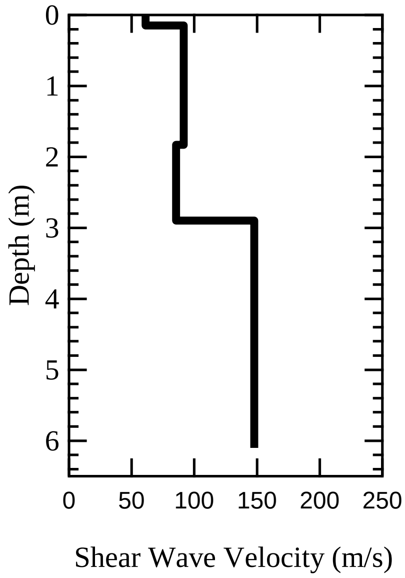


Figure 5-11. Shear wave velocity profile for AMI Field test site. (-43.54161°, 172. 65452°)

Table 5-1. Shear wave velocity profile for AMI Field test site. (-43.54161°, 172. 65452°)

Layer No.	**Thickness (m)**	**Depth to bottom (m)**	**P-wave velocity (m/s)**	**Shear wave velocity (m/s)**	**Poisson's Ratio**	**Unit Weight (kN/m^3)**
1	0.2	0.2	110	60	0.30	13
2	1.7	1.9	170	90	0.30	13
3	1.1	3.0	1520	90	0.50	13
4	3.2	6.2	1520	150	0.50	14

Note: maximum depth of the profile is approximately equal to maximum experimental wavelength/two.

AMI Columns test site is located just outside the northeastern stadium walls. The array was located within 1 meter, and parallel to, a row of drilled shaft-supported columns (Figure 5-12). The columns were placed on approximately 7.5 m c-c spacing. Ground deformations were present around the columns and sand boils were present in and around the array. The results are presented in Figure 5-13 and Table 5-2.

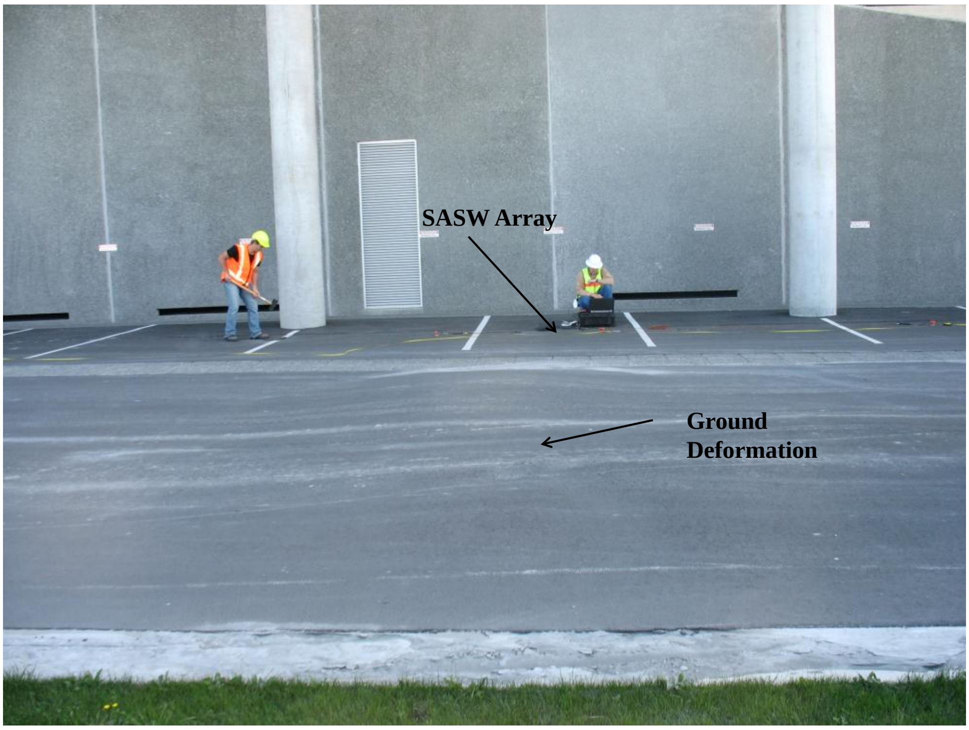


Figure 5-12. SASW array at AMI Columns test site. (-43.54128°, 172. 65562°)

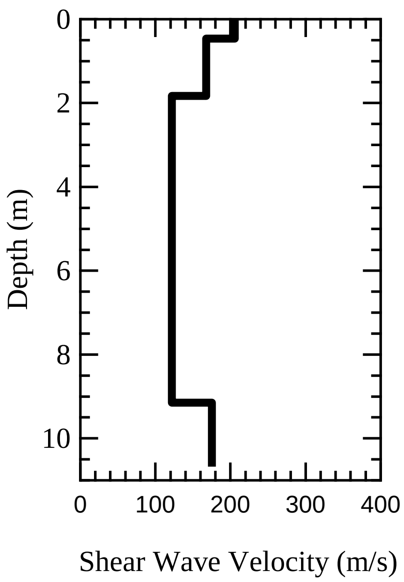


Figure 5-13. Shear wave velocity profile for AMI Columns test site. (-43.54128°, 172. 65562°)

Table 5-2. Shear wave velocity profile for AMI Columns test site. (-43.54128°, 172. 65562°)

Layer No.	Thickness (m)	Depth to bottom (m)	P-wave velocity (m/s)	Shear wave velocity (m/s)	Poisson's Ratio	Unit Weight (kN/m^3)
1	0.5	0.5	380	210	0.30	16
2	1.4	1.9	310	170	0.30	14
3	7.3	9.2	1520	120	0.50	14
4	1.5	10.7	1520	180	0.49	19

Note: maximum depth of the profile is approximately equal to maximum experimental wavelength/two.

AMI Stone Columns 1 (SC1) test site was located under the north end of the Deans Stand in a parking facility. A photo of the array setup is shown in Figure 5-14. In this area, there was about 5-8 cm of asphalt pavement and 10 cm of base material. There was no evidence liquefaction around the array or in the most northern part of Deans Stand. The results are presented in Figure 5-15 and Table 5-3.

Figure 5-14. SASW array at AMI SC1 test site. (-43.54138°, 172. 65501°)

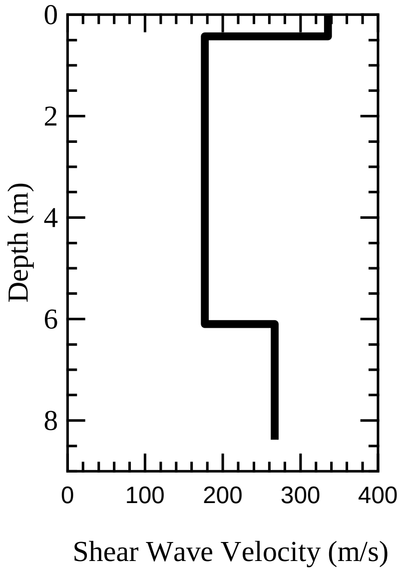


Figure 5-15. Shear wave velocity profile for AMI SC1 test site. (-43.54138°, 172. 65501°)

Table 5-3. Shear wave velocity profile for AMI SC1 test site. (-43.54138°, 172. 65501°)

Layer No.	**Thickness (m)**	**Depth to bottom (m)**	**P-wave velocity (m/s)**	**Shear wave velocity (m/s)**	**Poisson's Ratio**	**Unit Weight (kN/m^3)**
1	0.4	0.4	630	340	0.30	19
2	1.4	1.8	330	180	0.30	16
3	4.3	6.1	1520	180	0.49	16
4	2.3	8.4	1520	270	0.48	19

Note: maximum depth of the profile is approximately equal to maximum experimental wavelength/two.

AMI Stone Columns 2 (SC2) test site was located under the south end of Deans Stand. Again, this area was treated with stone columns, as described above. However, significant liquefaction occurred in the area of this test site, as shown in Figure 5-16. The results are presented in Figure 5-17 and Table 5-4.

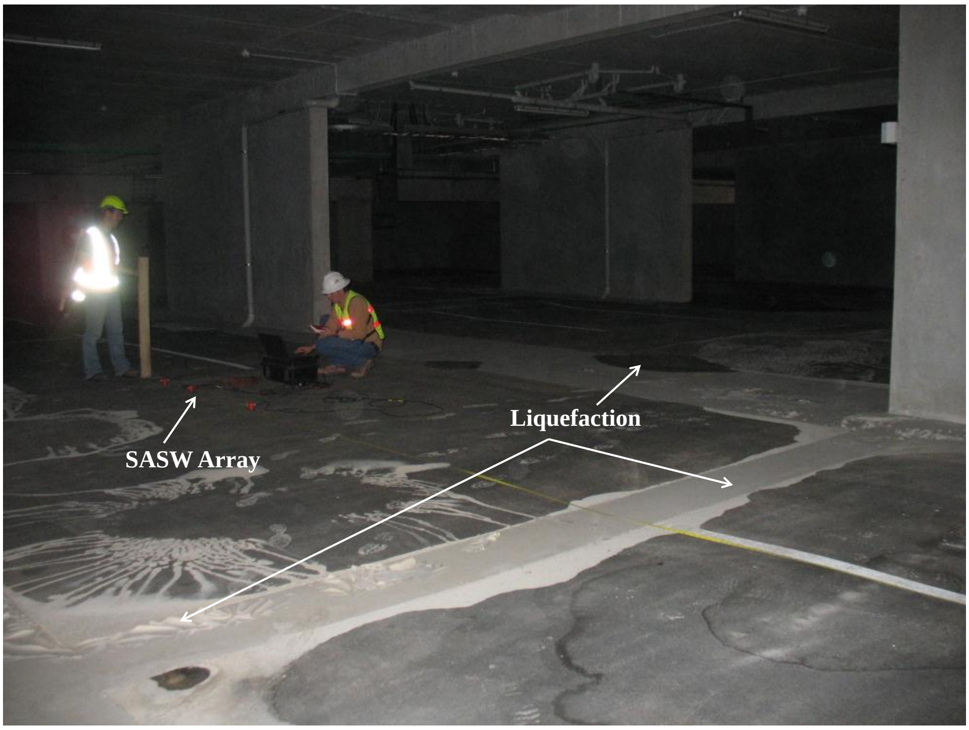


Figure 5-16. SASW array at AMI SC2 test site. (-43.54236°, 172. 65514°)

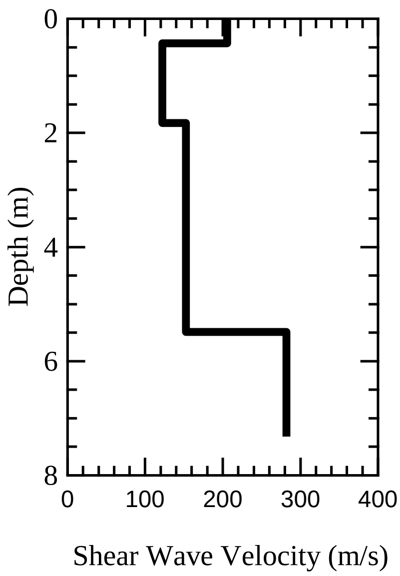


Figure 5-17. Shear wave velocity profile for AMI SC2 test site. (-43.54236°, 172. 65514°)

Table 5-4. Shear wave velocity profile for AMI SC2 test site. (-43.54236°, 172. 65514°)

Layer No.	Thickness (m)	Depth to bottom (m)	P-wave velocity (m/s)	Shear wave velocity (m/s)	Poisson's Ratio	Unit Weight (kN/m^3)
1	0.4	0.4	380	210	0.30	16
2	1.4	1.8	230	120	0.30	14
3	3.7	5.5	1520	150	0.49	14
4	1.8	7.3	1520	280	0.48	19

Note: maximum depth of the profile is approximately equal to maximum experimental wavelength/two.

Motorway Overpass Approaches

As part of the Christchurch Southern Motorway construction project, stone columns were installed beneath overpass bridge approaches at Barrington St (-43.5480°, 172.6101°) and Curletts Rd (-43.5478°, 172.5789°) interchanges. Stone column construction was still underway during the September 2010 Darfield earthquake, however no damage to these structures was reported. The stone columns had been completed prior to the Christchurch earthquake, with embankments built up and undergoing consolidation when the earthquake occurred. The greatest depth of stone columns at these sites is 18 m. The embankments at both sites were constructed using MSE abutment walls, consisting of concrete wall elements and embedded steel strips. No evidence of liquefaction was found at either site.

At Barrington St interchange, MSE wall construction had been completed and the embankments had been built up and were undergoing consolidation for approximately 6 months. Figure 5-18 shows the layout of the site; no damage to the MSE walls noted as a result of the earthquake. Curletts Rd interchange in Figure 5-19 not as far along, with the full height of the MSE walls yet to be reached on the western abutment (Figure 5-20), and construction had yet to commence on the walls of the eastern abutment. Preload had been placed and consolidation was underway on the western abutment, where up to 600 mm of settlement was expected at the western abutments at this site [NZTA 2011]. No damage was evident at this site and only minor surface slope movement occurred in the preload material.

Figure 5-18. Aerial view of Barrington St interchange MSE walls in later stages of construction. (-43.5480°, 172.6101°)

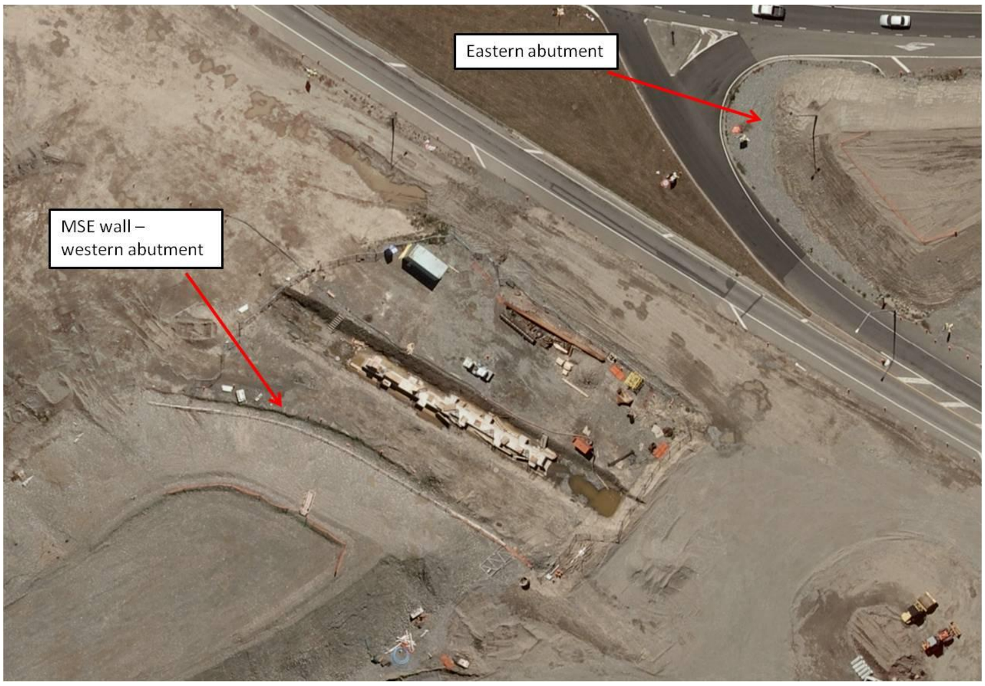


Figure 5-19. Aerial view of Curletts Rd interchange western abutment MSE wall in early stages of construction. (-43.5478°, 172.5789°)

Figure 5-20. Partial construction of the MSE walls for the Curletts Rd interchange western abutment. (-43.5478°, 172.5789°)

St Andrews School

The St Andrews School (-43.508903°, 172.612942°) is located in the suburb of Merivale and was affected by liquefaction during both the 4 September 2010 Darfield and the 22 February 2011 Christchurch earthquakes. Following the Darfield event there was some evidence of liquefaction, with the majority of the ejected material coming to the surface along a depression or old creek bed on the edge of the school grounds (white circled area in Figure 5-21). In this area a small, a two storey building was damaged and subsequently condemned as a result of lateral spreading and settlement. Aerial photographs taken shortly after the February event are shown in Figure 5-21. In this photo, a significant amount of ejecta can be observed, which is from the 22 February event.

During the GEER teams inspection, large amounts of ejecta was still on the school grounds and the surrounding areas, and lateral spreading cracks were evident throughout. Witnesses indicated that a section of the newly poured hockey field slabs (paved area within dashed black square) uplifted by approximately 100 mm due to the ejection of liquefied material, before dropping back down to the original level after the event.

Figure 5-21. Aerial photograph of St Andrews School following 22 February 2011 (LINZ 2011). (-43.508903°, 172.612942°)

Prior to the 22 February event, renovation and demolition was carried out on the Preparatory School buildings at St Andrews. A new, two-storey structure was constructed that is founded on 18 m deep, 350 kN screw piles. This building connected two existing structures that are founded on shallow raft foundations. The layout of these structures is shown in Figure 5-22, both before and after the February earthquake. In Figure 5-22a, the buildings outlined in yellow were demolished as part of the renovations prior to the February event. However, these building were present at the time of the September event, with no evidence of damage or movement observed as a result of the earthquake. The buildings outlined in red in Figure 5-22a are the two structures that are founded on shallow raft footings that remained after the renovations, as shown in the post-renovation photo in Figure 5-22b. A schematic of the post-renovation layout of the structure is shown in Figure 5-23. (Note that this figure also shows the locations and directions of photographs presented subsequently in this chapter.). The ground to the northeast edge of structure was relatively flat, and sloped away gradually to the northwest. The ground to the southwest was level, before dropping down 2 m at the retaining wall shown in the figure. To the south the ground sloped down approximately 2 m from the southeast to the southwest corner.

(a)

(b)

Figure 5-22. St Andrews Prep School buildings (a) prior to renovation with buildings that remain outlined in red and those demolished in yellow (Google 2010); (b) post-renovation aerial image showing the new building that is founded on screw piles. Also shown in this image is an extension to shallow foundation sections of structure outlined in dashed red (LINZ 2011). (-43.508903°, 172.612942°)

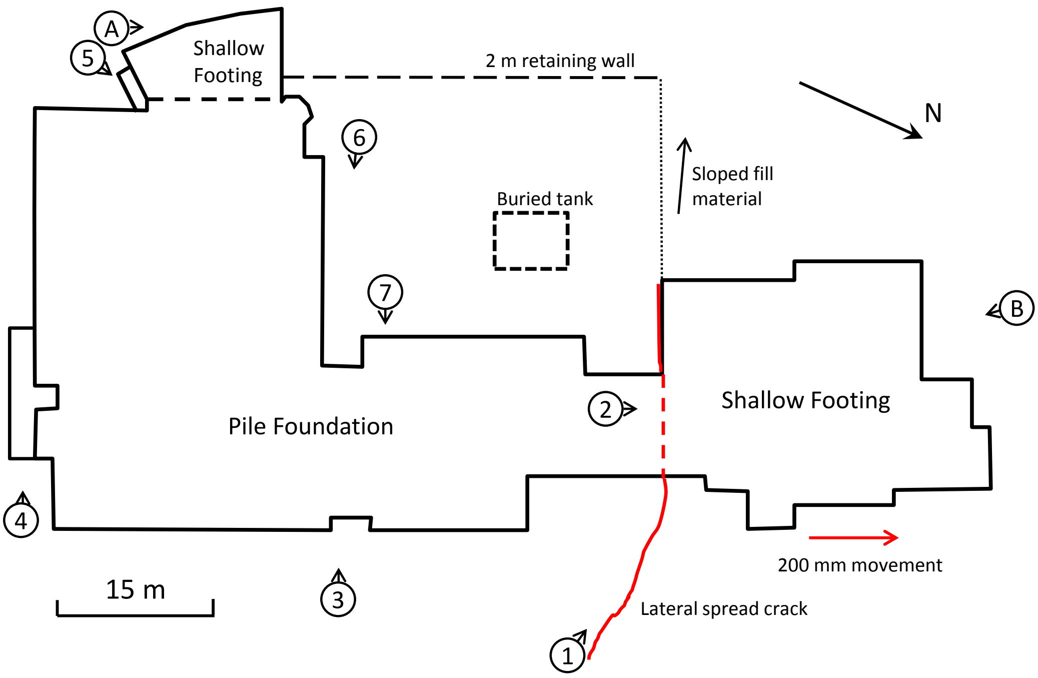


Figure 5-23. Layout of the Preparatory School indicating photo positions. (-43.508396°, 172.612766°)

Lateral spreading at the site was evident with movement to the northwest, as illustrated in Figure 5-23. A lateral spread crack passed underneath the building and resulted in the separation of an expansion joint between portions of the building founded on screw piles and raft foundation. The portion of the structure on the shallow foundation moved away from the pile founded portion by ~200 mm and settled by ~200 mm, while the piled portion of the structure remained in place. Internal and external photos of the gap formed by the spreading are shown in Figure 5-24. A ramp was constructed to account for the differential movement, as shown in Figure 5-25.

(a) (b)

Figure 5-24. (a) Lateral spread crack propagating through and opening seismic gap in structure (position 1); (b) Opening of seismic gap and settlement between structures on piles and shallow foundation (position 2). (Position locations shown in Figure 5-23.)

Figure 5-25. Ramp construction to mitigate differential movement between structures (position 2). (Position locations shown in Figure 5-23.)

A summary of the differential settlement between the structures and the surrounding ground is shown in Figure 5-26. These measurements show that shallow founded portions of the building settled almost evenly with the surrounding ground, while the pile supported portion did not move, rather the surrounding ground settled by up to 10” (25 cm). Some examples of these movements are shown in Figure 5-27. The settlement of 7.5” on the shallow founded section to the north seems to be due to movement of fill material away from the structure and down the slope, as indicated in Figure 5-23. Other than this position, Figure 5-28 shows that the level of the shallow foundation structure and the surrounds are almost even.

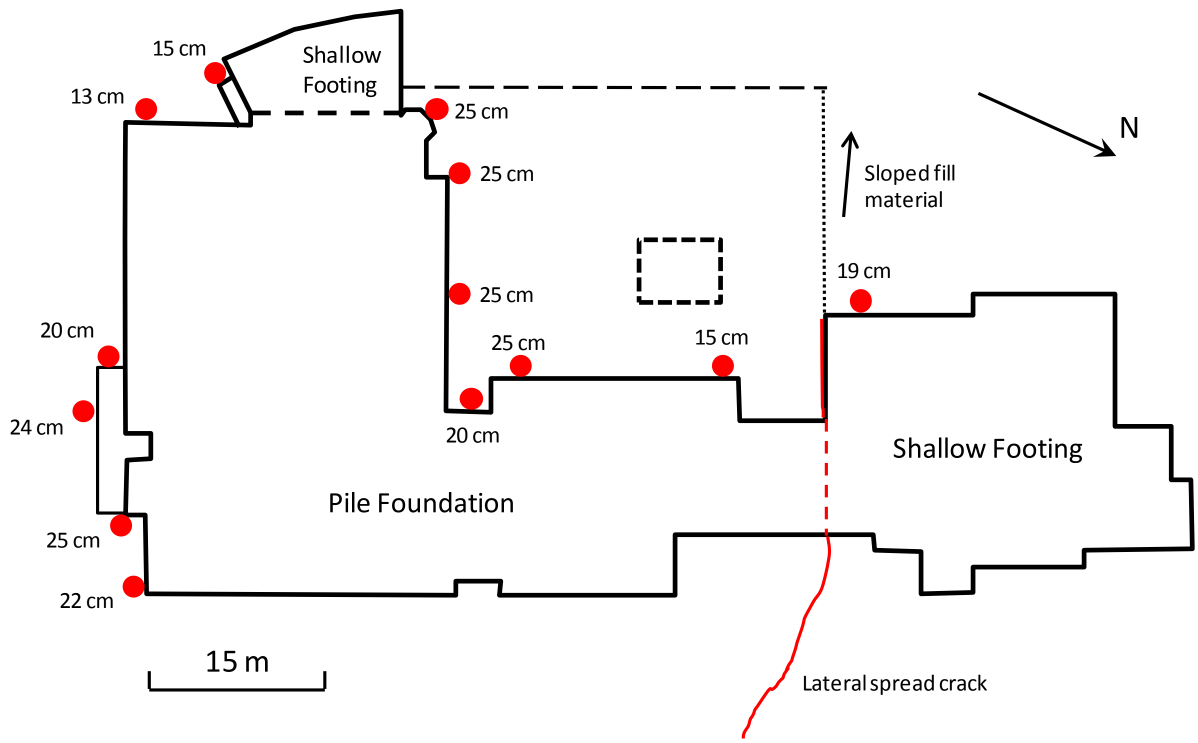
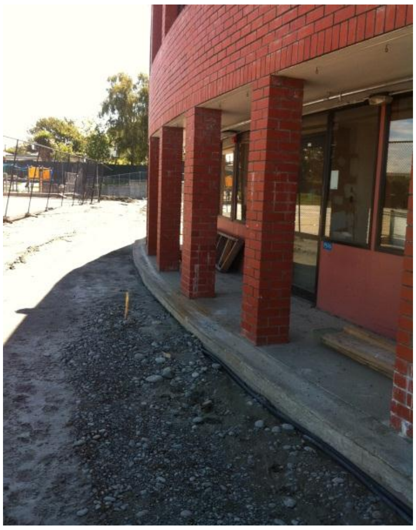


Figure 5-26. Measured ground settlement compared to structure. (-43.508396°, 172.612766°)

(a) (b)

(c) (d)

Figure 5-27. Settlement of ground around pile supported structure: (a) 20 cm, position 3; (b) 25 cm, position 4; (c) 25 cm, position 6; (d) 25 cm, position 7. (Position locations shown in Figure 5-23.)



(a) (b)

Figure 5-28. The difference between the level of the shallow founded portions of the building and the surrounding ground was minimal: (a) position A; (b) position B. (Position locations shown in Figure 5-23.)

Relative movement of the pile supported portion of the building and the surrounding ground resulted in damage to external stairs shown in Figure 5-29. The stairs were attached to the side wall of the original, shallow founded portion of the structure and lead to an entrance to the new, pile supported structure. When the area settled, the stairs were dragged down by the original structure as it settled, while the top of the stairs were restrained by the piled structure which remained in place. The result was a 7° slope of the stairs towards the original structure as shown in Figure 5-29a. This figure also shows damage to the anchorage reinforcement between the wall and the stairs, indicating both differential vertical and horizontal movement. The shallow founded building seems to have moved away from the stairs.

(a) (b)

Figure 5-29. Damage to stairs from differential movement of pile and shallow foundation supported structures at position 5. (Position locations shown in Figure 5-23.)

Apart from the gap in the construction joint that widened as a result of lateral spreading and stairs, none of the above ground portion of the structures suffered any distress. Settlements of the shallow founded sections were even, and the only required repair was backfilling of settled areas under and around the piled structure. A buried tank adjacent to the piled structure can be seen in Figure 5-30. As shown in this figure, the ground around the tank settled, exposing the tanks position at the surface.

Figure 5-30. Settlement of ground around the roped off buried tank structure. (-43.508389°, 172.612489°)

SASW and DCP tests were performed on a sports playing field at St Andrews School, as shown in Figure 5-31. As may be observed from this figure, the test site had sand blows throughout. A photograph of the SASW test array is shown in Figure 5-32, and the results from both the SASW and DCP tests are shown in Figure 5-33, with the SASW test data tabulated in Table 5-4. As shown in Figure 5-33, the ground water table is at a depth of approximately 1.3 m and the depth to the top of the liquefiable layer was around 2.84 m. (Note that a hole was hand augered to the top of the liquefiable layer and then the DCP test was started.).

Figure 5-31. Site photo of St. Andrews School (Post Christchurch EQ) showing the location of the SASW array. (-43.50953°, 172.61291°)

Figure 5-32. SASW array at St. Andrews School test site. (-43.50953°, 172.61291°)

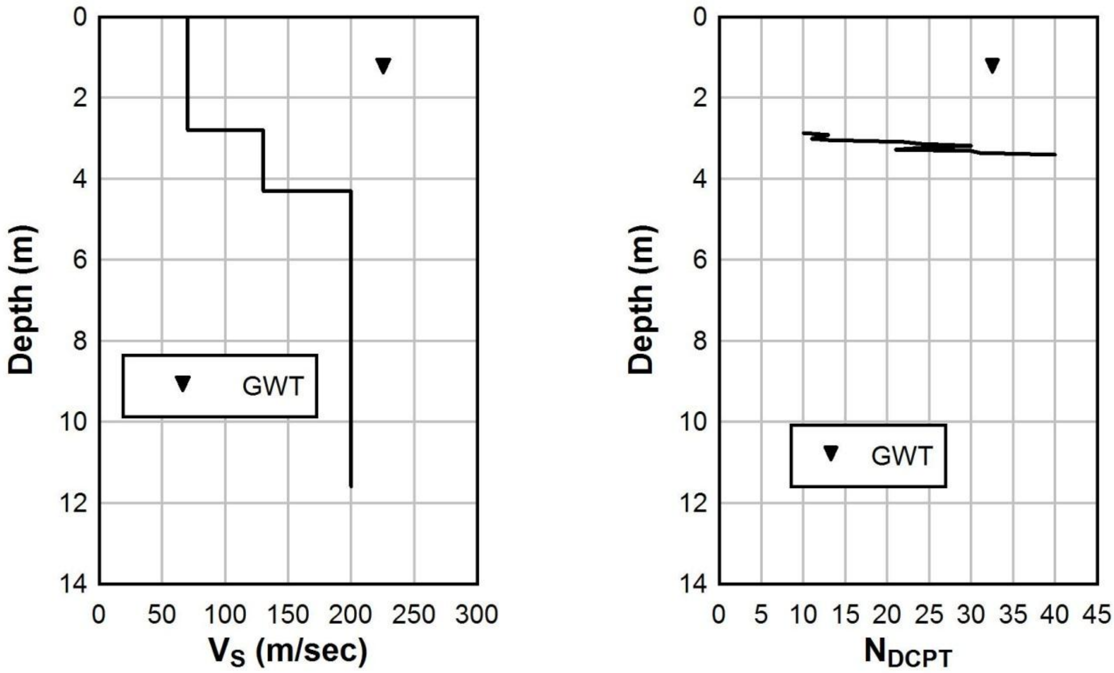


Figure 5-33. In-situ data from test site located on a sports field at St Andrews School: (a) SASW shear wave velocity profile; and (b) DCP blow count profile. (-43.50953°, 172.61291°)

Table 5-5. Shear wave velocity profile for St. Andrews School test site. (-43.50953°, 172.61291°)

Layer No.	**Thickness (m)**	**Depth to bottom (m)**	**P-wave velocity (m/s)**	**Shear wave velocity (m/s)**	**Poisson's Ratio**	**Unit Weight (kN/m^3)**
1	1.3	1.3	140	70	0.30	13
2	1.5	2.8	1520	70	0.50	13
3	1.5	4.3	1520	130	0.50	14
4	7.3	11.6	1520	200	0.49	16

Note: maximum depth of the profile is approximately equal to maximum experimental wavelength/two.

References

NZTA (2011) Christchurch Motorways, Project update, www.nzta.govt.nz, November 2011.